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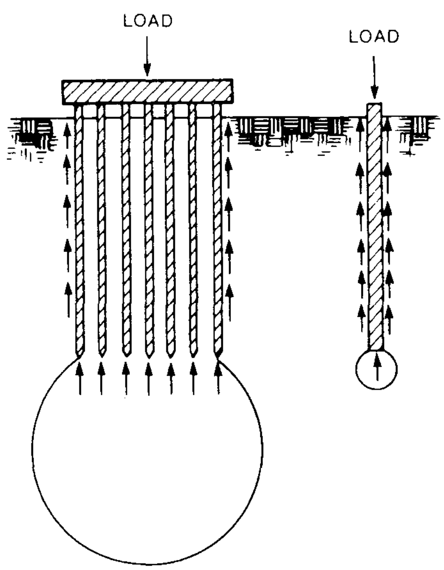
Pile Group Prediction Symposium: Summary

Volume I, Sandy Soil



U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike, McLean, VA 22101-2296

FOREWORD

This report presents the summary of several predictions of the behavior of a group of piles driven into sand layers existing on a site at the Hunter's Point Naval Base, San Francisco, California. The predictions were made by 11 foundation experts and were presented at a prediction symposium held on June 17 and 18, 1986 at the University of Maryland. The symposium was held to highlight the Federal Highway Administration's pile research and to discuss the various design methods for pile groups in sand.

Additional copies of the report can be obtained from the National Technical Information Service, Springfield, Virginia 22161.

R. J. Betsold
Director, Office of Implementation

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16. Abstract

This technical sharing report presents the summary of a pile group prediction symposium held on June 17 and 18, 1986 at the University of Maryland campus. Eleven foundation experts representing consulting firms, universities, and Federal and State transportation agencies presented their predictions on the results of single and pile group tests in sands, that were conducted on a site at the Hunter's Point Naval Base, San Francisco, California. The principal objective of the symposium was to evaluate current pile group foundations in sands. Complete reports on pile groups prediction symposium consists of the following:

Vol. No.	FHWA No.	Short Title
1	TS-87-221	Sandy Soil
2	TS-87-222	Clay Soil
3	TS-87-223	Appendix A (predictor's reports on sand)
4	TS-87-224	Appendix B (predictor's reports on clay)

* Geo-Resource Consultants, Inc., San Francisco, California

17. Key Words	18. Distribution Statement
Single and group piles, bearing capacities, end bearing, side friction, load transfer curves, load settlement characteristics	No restrictions. Copies of this report are available from National Information Service Springfield, Virginia 22161

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha
	MASS (weight)			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

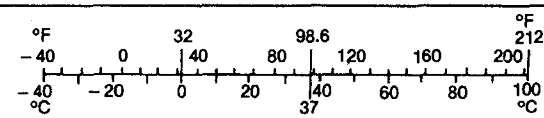
* SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac
	MASS (weight)			
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

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INTRODUCTION

This prediction symposium was established to examine various design methodologies for pile groups that are used by governmental agencies, consultants, and academic institutions in the United States. The predictions are being compared to the measured results of compression load tests on a full-scale pile group and an isolated control pile in cohesionless soils. The tests were conducted under a research contract between the Federal Highway Administration (FHWA) and Geo/ Resource Consultants, Inc., of San Francisco, California.

The design of pile foundations is primarily a problem of predicting the bearing capacity and settlement of a group of piles which typically involves the determination of bearing capacity and settlement of a single pile and extrapolating to get the group values. Simple and accurate guidelines are not available to perform a proper analysis; therefore, a conservative approach using a high factor of safety is typically used.

In the late 1970's, the Federal Highway Administration initiated a comprehensive research program for highway bridge foundations. One of the major emphasis areas involved the design of single piles and pile groups. This research was directed primarily toward evaluating available methods for predicting bearing capacity and settlement of pile foundations. A search was made for a comprehensive mathematical model of pile group behavior that could systematically convey engineering experiences from one site to another. After evaluating numerous existing methods, it was decided to modify one of the most promising models to develop a new method called the FHWA PILGP1. A series of laboratory and field load test programs was initiated to verify and refine the new method.

Due to budgetary constraints a large number of full-scale field tests was not considered practical. A series of three full-scale load tests to failure were designed to provide high-quality experimental data on a pile group in clay and two pile groups in sand. The first field test study was performed on a nine-pile group of steel pipe piles in clay at the University of Houston. The nine piles were driven in a 3 x 3 square array on a spacing of three-pile diameters. Two identical piles were driven apart from the group to serve as controls. Each of the piles and the surrounding soil was instrumented and monitored to provide the appropriate data for improving the PILGP1 method. A pile group prediction symposium was held in conjunction with this research effort to examine the various design methods for pile groups in clay. Details of the research test can be found in FHWA/RD-81/001, "Field Study of Pile Group Action," March 1981. Details of the predictions can be found in the proceedings of the symposium.

The second field test study was performed on an eight-pile group of timber piles in sand at the Lock and Dam No. 26 structure near Alton, Illinois, in cooperation with the Corps of Engineers. The timber piles were instrumented to measure load transfer and deformation up to and including the failure load. The acquired field data were used to refine the PILGP1 method. A prediction symposium was not held in conjunction with this field study.

The third field test study was performed on steel piles of various types in a sand deposit at the Hunter's Point Naval Shipyard Facilities in San Francisco, California, and is the subject of this prediction symposium. Complete details about the test site and the load test program will be given prior to the presentation of individual predictions.

In addition to the pile group load tests to failure, four full-scale field projects were initiated to observe pile group behavior under working loads. The short and long term behavior of the inservice piles has been compared with analytical predictions made by PILGP1.

One of the projects is located on the Natchez Trace Parkway in Mississippi, where a pile-supported bridge abutment is instrumented to obtain load transfer data on a group of six steel piles (12 HP 53) in soft clay and silt. Another project is located at Fort Belvoir, Virginia, where soil and pile conditions are similar to those at the Mississippi project. Instrument readings were taken weekly during the first year at each site, monthly during the second year, and will be taken quarterly for several years.

The third site, at the Mocks Bottom overcrossing in Portland, Oregon, near Swan Island, is underlain by a thick compressible clayey silt deposit. High downdrag loads were expected because of the approach embankment loads on the compressible soil. A bitumen coating was used to reduce downdrag loads by about 90 percent. Although the bitumen-coated piles cost about 15 percent more than the uncoated piles, fewer piles were required. Pile instrumentation included settlement and load transfer monitoring.

The fourth site is located at the West Seattle Freeway in Washington where a group of twelve 24 in. (0.6m) diameter concrete piles supports a pier in medium-dense sands. The bridge pier and pilecap were instrumented to measure the amount of load transferred to the pile cap, and each pile was instrumented to measure load transfer from the pile cap to the top of the piles. Three piles were instrumented for load transfer along the entire pile length of 100 ft. (30.5m).

The development and verification of PILGP1 is based primarily on the full-scale field tests of pile group behavior under both working loads and failure conditions. Because of the many variables involved, numerous full-scale field tests need to be conducted to provide a statistically meaningful data base. However, the high costs involved in full-scale field testing significantly restrict the number of tests that can be conducted. The alternatives to full-scale field testing are model field testing and laboratory model studies and centrifuge model testing.

FHWA initiated a comprehensive investigation of scale effects between model and full-scale pile groups at the Turner-Fairbank Highway Research center (TFHRC) in McLean, Virginia. The scaling factors identified in this study will be used to establish relationships between load deformation behavior of reduced-scale and full-scale piles and pile groups. These small-scale tests will provide data to validate PILGP1 at much less cost than full-scale field testing.

The first series of laboratory model tests are patterned after the timber pile field study at Alton, Ilinois and the Hunter's Point site. The sandy soils at these test sites were matched as closely as possible at TFHRC. Model load tests will be run on single piles and pile groups at 1/20, 1/15, 1/10, and 1/3 of full scale. A minimum of three load tests will be performed for each scale. Each pile is instrumented with strain gages to measure load transfer.

The second series of laboratory model tests is patterned after the full-scale load test on steel pipe piles in clay. Model tests will be run at 1/15, 1/10, 1/6, and 1/4 of full scale. The laboratory model tests on the 1/20, 1/15, and 1/10 scales are performed in a steel tank 5 ft (1.5m) in diameter and 5 ft (1.5m) deep. Because the 1/6, 1/4, and 1/3 scale models are too large to be tested in the laboratory test mold, outdoor test pits were constructed at the TFHRC site.

Small-scale models permit parametric studies at reasonable cost and allow soils and other conditions to be carefully controlled; however, it is difficult to achieve similitude between corresponding stresses and strains in the model and prototype. The response to load of a small pile and a large pile cannot be modeled by any simple, direct relationship derived by ordinary dimensional analysis. The question of scale effects must be resolved before any useful relationships for pile design can be developed.

Models of large heavy structures where gravity is a principal loading factor are not very effective indicators of prototype behavior because the state of stress in the model caused by self weight will be much lower than in the prototype. If the model can be placed in an artificially high gravitational field, the state of stress limitation can be counteracted almost entirely. A centrifuge apparatus provides the necessary accelerated gravity rate to load test the model under simulated gravitational forces. However, to accurately measure stresses and strains, the centrifuge must be able to accommodate a model that is large enough to handle the required instrumentation. The larger centrifuge capacity provides more accuracy in direct modeling of large prototype structures.

A pilot study validated the feasibility of using centrifuge techniques for corroborating the PILGP1 mathematical model. In a recently completed larger study the centrifuge was used to test models of the full-scale pile groups that were load tested to failure in the previously described field studies. The combination of centrifuge model testing and small-scale laboratory testing of conventional models at TFHRC will provide valuable physical data to establish relationships between pile groups of varying scales in the same environment.

Hopefully the preceding discussion of our research program will provide sufficient background information to allow you to place the role of this prediction symposium in the proper perspective. The main purpose for conducting the load tests at the Hunter's Point site was not to provide measured values to compare against various prediction calculations; but rather to provide high quality research data to validate the FHWA PILGP1 method. However, the field test results do present a good opportunity to compare various methods in common use today, and it is always beneficial to gather together a group of expert predictors and a knowledgeable audience to discuss and analyze pile design procedures. Without any further background discusssion, we will now set the stage for the ensuing prediction presentations by describing the test site and the data that was supplied to the predictors.

Each predictor was required to present an independent viewpoint and prediction of the behavior of the experimental pile group and the separate control pile. These predictions were required before the load tests were conducted and were based solely on the geotechnical and geometrical data. Each predictor was required to submit a detailed discussion of the methodology used to make the various predictions such as: failure mode, ultimate axial bearing capacity, corresponding settlement under the predicted ultimate load, and load distribution for the single pile and the five-pile group. Subsequently, load test results on the single pile were sent to each predictor to allow them to modify their predictions of the pile-group performance.

The test site has a dense, 5-foot (1.5 m) thick surface layer of clayey sand and gravel which overlies an approximate 35 feet (10.6 m) thick layer of uniform loose to medium dense, poorly graded sand that was placed in 1942 as a hydraulic fill over an approximately 5-foot (1.5) thick layer of silty clay. Serpentine bedrock was encountered at depths of 46 to 49 feet (10.6-14.6 m). The water table is approximately 8 feet (2.4 m) below the surface.

Each predictor was given a report of geotechnical and geometrical data which included details on instrumentation, reference and loading systems, reaction system, sequence of field observations, and loading procedures. The test site conditions included field investigation and laboratory testing results. The field data included boring logs, electric piezocone soundings, pressuremeter tests, dilatometer tests, and standard penetration tests. The laboratory data included grain size distribution , direct shear strength data, moisture contents and dry density values.

The test piles were hollow steel piles which were driven closed ended to a depth of 30 feet (9.1 m). The piles were closed at the tips by steel boot plates cut flush with the circumference of the pile. These piles had a wall thickness of 0.365 inches(9.27 mm) and a 10.75-inch (272 mm) outside diameter. The control pile was located aobut 13.5 feet (4.1 m) from the center of the group.

Detailed information on the preparation, load-test procedure, and the test results are presented by the research team in the papers that follow. A summary of the prediction results and panel discussion complete these symposium proceedings. Presentations of each prediction will be included in a separate volume of this symposium.

AXIAL LOAD TESTS ON A CONTROL SINGLE PILE AND FIVE-PILE GROUP IN COHESIONLESS SOILS.

BACKGROUND

As a continuation of pile research efforts by the Federal Highway Administration (FHWA), a research team led by Geo/Resource Consultants (GRC) has conducted several full-scale load tests on various types of single piles and a five-pile group in cohesionless soils. Since the focus of this symposium is on the prediction of the five-pile group behavior with respect to a single pile, only the load test on a single, control steel pipe pile and its corresponding five-pile group are presented in this article. The tests were performed under the direction of FHWA and represented Phases 2 and 3 of the current FHWA research on pile behavior in cohesionless soils. The Phase 1 study was completed earlier by Briaud, et al. (1983, 1984). At the time of this article, all field work for Phases 2 and 3 has been completed and the analysis effort has been partially completed. The results and findings of this study will be presented in a final report to be issued in December 1987.

The FHWA-sponsored symposium was held at the University of Maryland Campus on June 17 and 18, 1986. The purpose of the symposium was to examine the various design methods used by practicing engineers and academic institutions, and to provide a comparison between various pile design methodologies with measured results from the full-scale load tests in the field. The soil properties, pile characteristics, and test program details were given to all participating predictors. The results of load test data on the single pile were also given to the predictors to allow modification of the group pile predictions.

A summary review of these predictions is presented in a separate paper (O'Neill, 1986).

Apart from the participating predictors, the GRC research team also performed predictions based on various analytical methods (including FHWA PILGP2 Program) and provided analyses on the load test results. The details of these analyses are presented in a separate paper (Briaud and Tucker, 1986).

SITE SELECTION

The objective of the research was to evaluate the behavior of single and group piles driven into a cohesionless soil deposit. The selection criteria proposed for the test site consisted of about 50 feet of cohesionless soils with Standard Penetration Test data (SPT) of less than 30 blows per foot. The intent of selecting a test site with uniform soil characteristics was to provide a good baseline study for understanding pile behavior. On this basis, the site at the Hunter's Point, Naval Station in San Francisco, California was selected. The subsurface conditions at the test site consist of 40 to 46 feet of loose to medium dense sand and the soil deposits are relatively uniform. The soil characteristics of the test site are presented in the soil investigation discussion below.

The test site was located on an existing pier built in 1944-1945. It was constructed by driving cellular cofferdams along the perimeters, dredging the inside materials, and replacing the materials with hydraulically-placed sand fill. The thickness of this sand deposit is up to 120 feet towards the seaward end of the pier. The site is located towards the central portion of the pier where the depth of sand is about 40 to 46 feet. An aerial view of the site is presented in Figure 1.

SITE INVESTIGATION

The field investigation program at the test site consisted of standard penetration tests (SPT), cone penetrometer tests (CPT), pressuremeter tests (PMT), shear wave velocity measurement, and dilatometer tests (DT). These tests were performed from January to March, 1986, prior to any pile driving . The locations of the tests are presented on the Site Plan, Figure 2. The results of these tests are discussed below. SPT and CPT tests were repeated between July and September 1986, after the pile load tests were completed. The results will be incorporated in the final report.

The purpose of the test boring and sampling program was to: 1) explore subsurface conditions at the site; 2) obtain SPT data; 3) provide borehole location for performing cross-hole geophysical surveys; 4) obtain representative samples for laboratory tests. The test borings were performed under the direction of GRC engineers. Pitcher Drilling of Palo Alto, California provided a truck mounted Failing 1500 rotary wash drill rig. The borings were drilled through the overburden soils and cored into the underlying bedrock primarily with the use of REVERT drilling mud. The boring procedure consisted of augering through the top sandy gravel layer, which extends to approximately 4 to 5 feet, drilling and setting steel casing to a depth of about 9 feet, and using a rotary wash method to advance through the soil deposits. Where bedrock was encountered, coring was used to obtain core samples for bedrock.

Soil samples were recovered with either a 3-inch outside diameter Sprague and Henwood Sampler, driven by a 350-pound hammer falling 18 inches or, by performing a Standard Penetration Test (SPT). SPT tests were performed every 5 feet. A donut-type hammer was used to obtain SPT resistance in the pre-test investigation. SPT data and blow count data from Sprague Henwood Sampler are presented in Figure 3. During the post-test SPT test, both donut hammer and safety hammer were used for SPT data, providing an opportunity for comparison between the two hammers.

An electric piezocone mounted on a CME 750 drill rig was used to provide the cone soundings at the test site. The cone sounding was performed by InSituTech of Oakland, California. The cone was hydraulically pushed to the bedrock stratum. Typically, the cone encountered some high tip and friction resistance in advancing through the upper 5 feet of gravelly sandy fill. After penetrating through the fill, the cone was pushed steadily through the sandy soil and encountered high resistance at the bedrock. A portable data acquisition system in the field provided a profile of the cone tip and shaft resistance. The

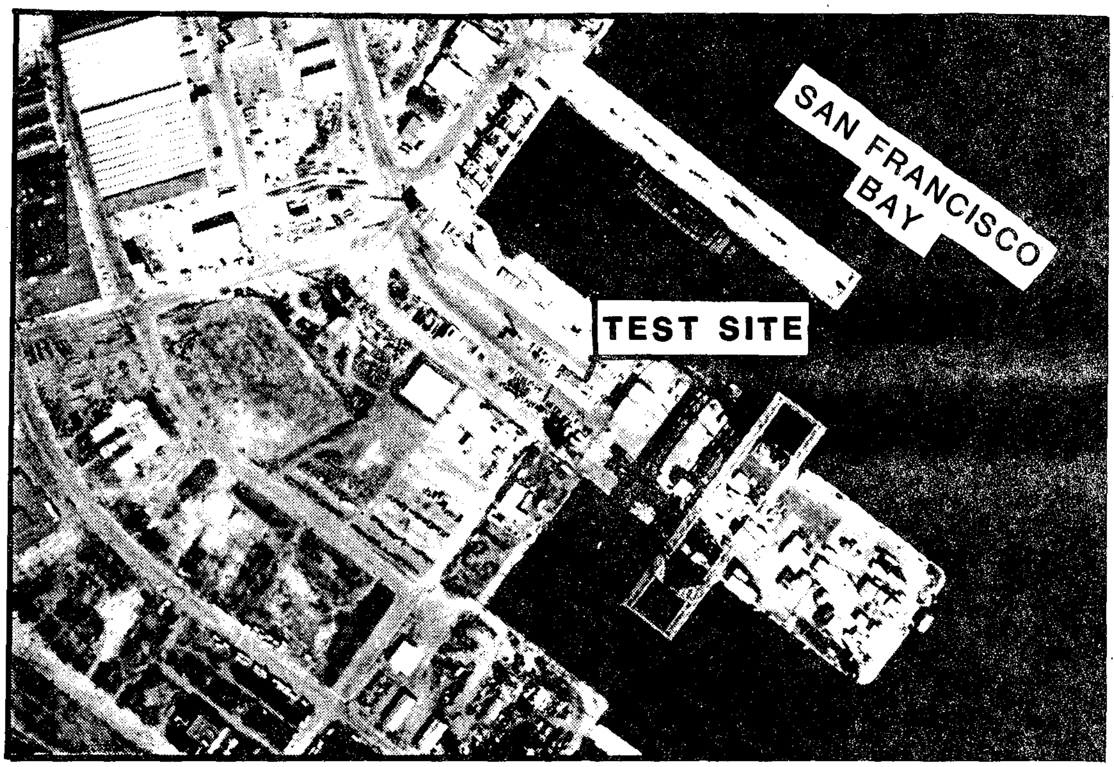

Fig. 1 Aerial view of Hunter's Point test site.

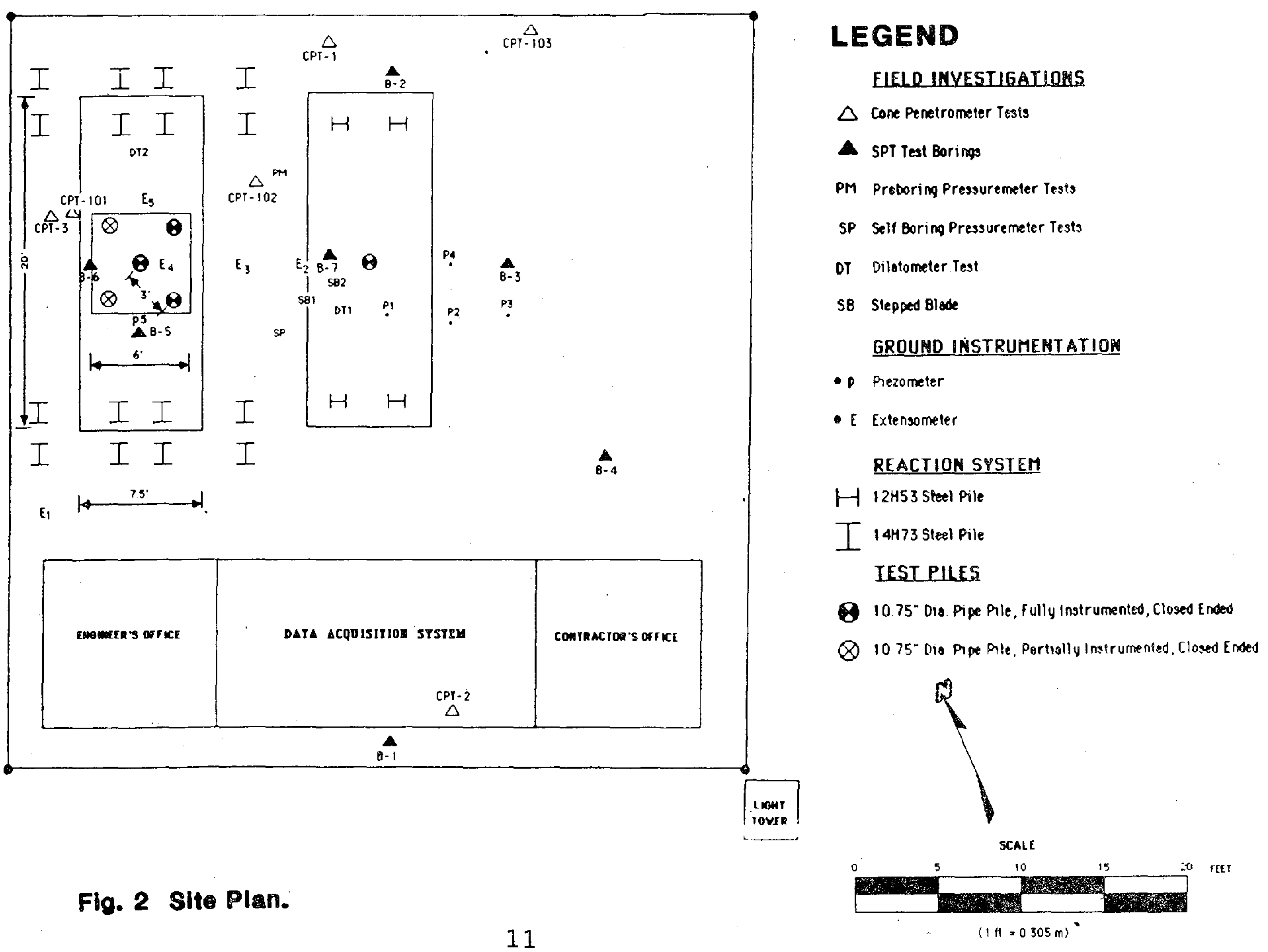

Fig. 2 Site Plan.

locations of the cone sounding are presented on Figure 2 and the results are presented on Figure 4.

A geophysical cross-hole survey was also performed using the array of test boreholes B-2, B-3 and B-4, with approximately 15 feet spacing between boreholes. The cross-hole measurement was performed by Mr. Bruce Auld and his staff in GRC. One hole in the array was cased with 4-inch I.D. PVC pipe, and the other holes with 3-inch PVC pipe. All holes were grouted into place with cement. A shear wave hammer was placed in one hole and vertical geophones were placed in the other boreholes. Cross-hole seismic wave travel times were determined at 5-foot intervals throughout the depth of the array. The results of both the compression and shear wave measurements are presented in Table 1.

Both preboring and selfboring pressuremeter tests were performed at the site by Dr. Jean-Louis Briaud and his colleague. The pressuremeters used at the site were the model TEXAM (Briaud, 1986). The preboring tests were conducted with a 2.91 inch (74 mm) diameter and 18.0 inch (457 mm) long probe. A total of 9 preboring pressuremeter tests were performed at PMT1 location to a depth of 51.5 ft. The probe was inflated to close to twice its initial volume in 10 minutes in equal volume increments. The borehole was prepared by setting casing to a depth of 6 feet and then by rotary drilling below the casing level with a 2-15/16 inch (75 mm) drilling bit. Axial injection of REVERT drilling mud was used with great success. The selfboring pressuremeter tests were conducted with a 2.76 inch (70 mm) diameter and 21.2 inch (538 mm) long probe at PMT2 location. The first two tests were used as pilot tests to determine the best selfboring technique for the soil conditions at the site. Casing was set at 2.5 feet above the testing depth. The selfboring probe was then lowered into the borehole and slowly advanced, through rotation of the cutting tool and injection of water to the testing depth. The SBPMT tests were carried out in 10 minutes by the volume increment method. The results of both tests, as reported by Briaud, are presented on Figures 5 through 7 (Briaud, et al., 1986).

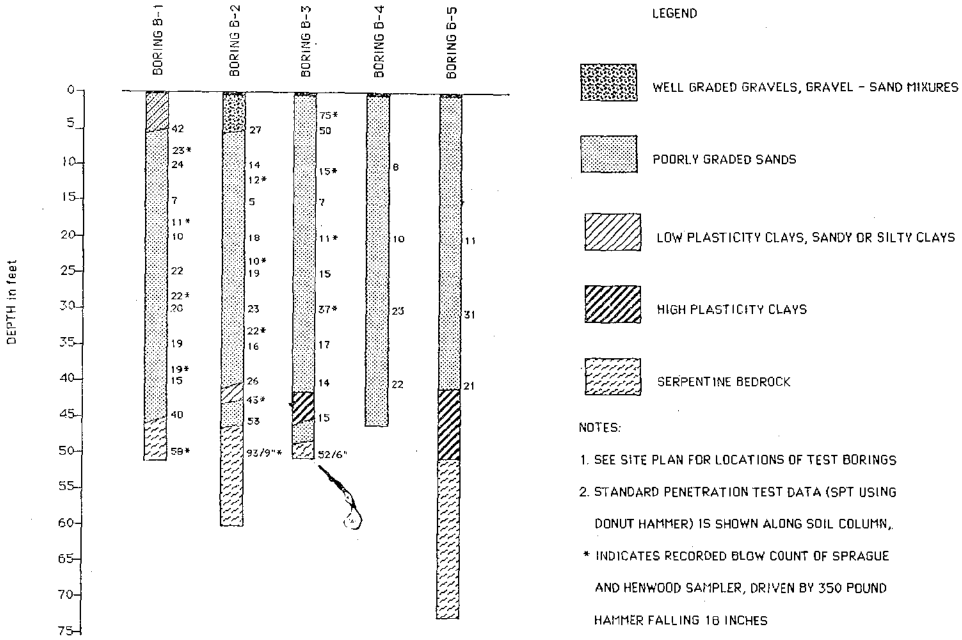

Fig. 3 SPT and borehole data.

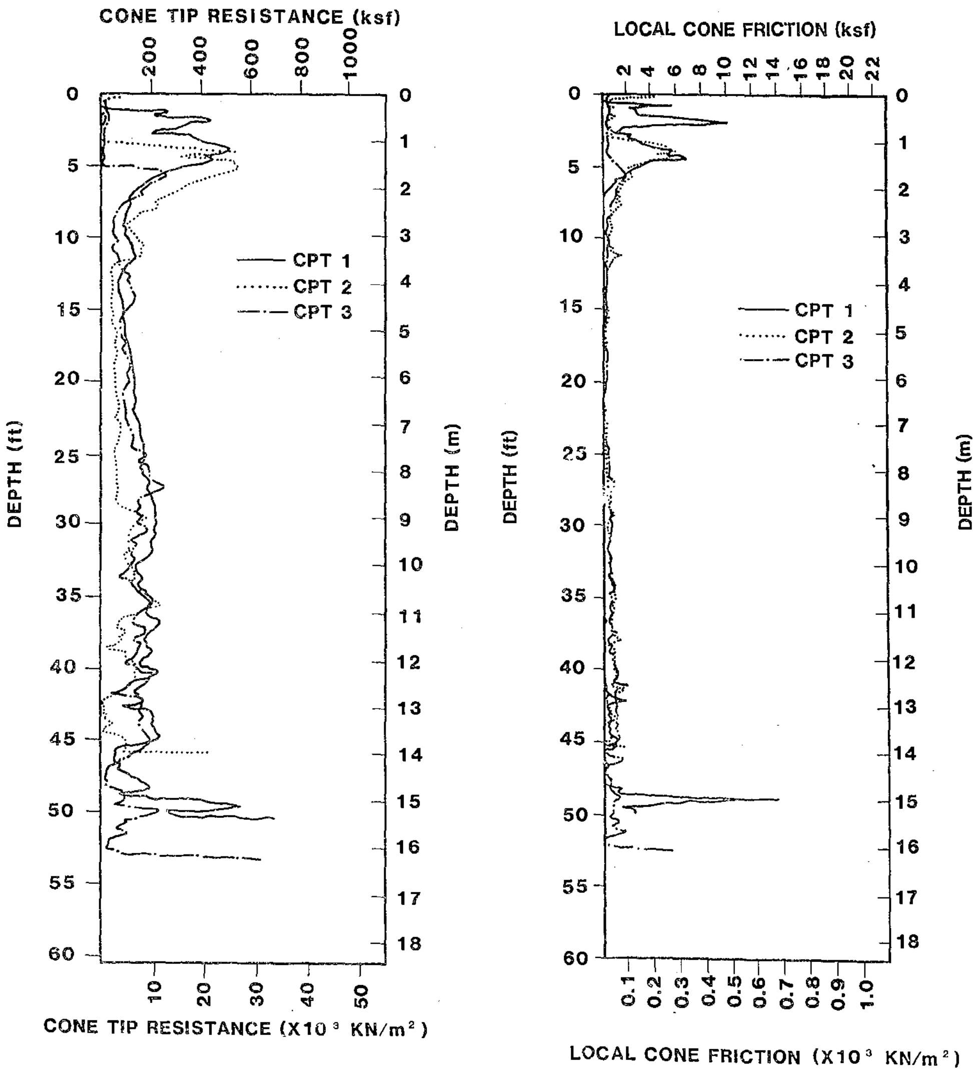

Fig. 4 CPT tip and friction resistance (after InsituTech).

CROSS HOLE VELOCITY MEASUREMENT

Depth (ft)	Vp (ft/sec) Compression Wave	Vs (ft/sec) Shear Wave
5	870 ± 90	650 ± 150
10	600 ± 150	430 ± 50
15	650 ± 150	400 ± 80
20	730 ± 250	450 ± 80
25	920 ± 400	470 ± 100
30	1080± 600	550 ± 150
35	1220± 600	620 ± 150
40	1060± 400	600 ± 120
43	950 ± 200	580 ± 100

Table 1. Cross hole survey data.

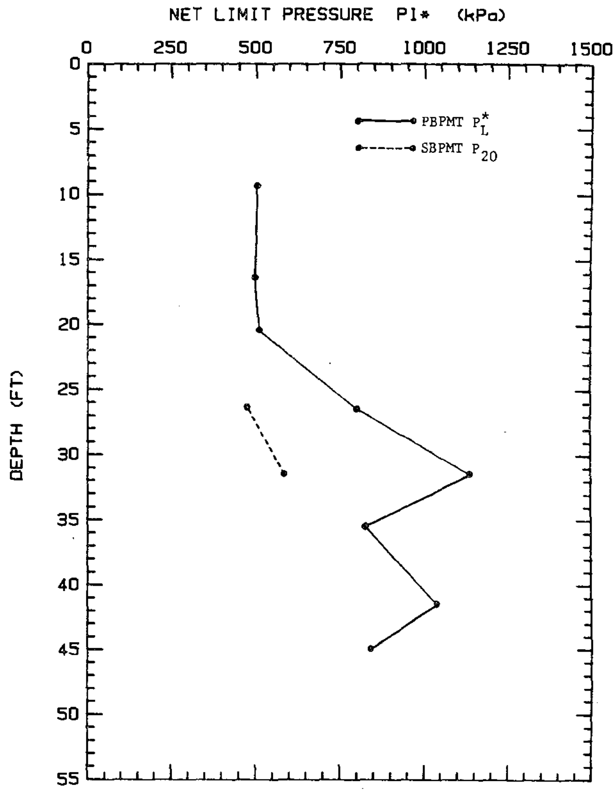

Fig. 5 Pressuremeter test – net limit pressure (after Briaud).

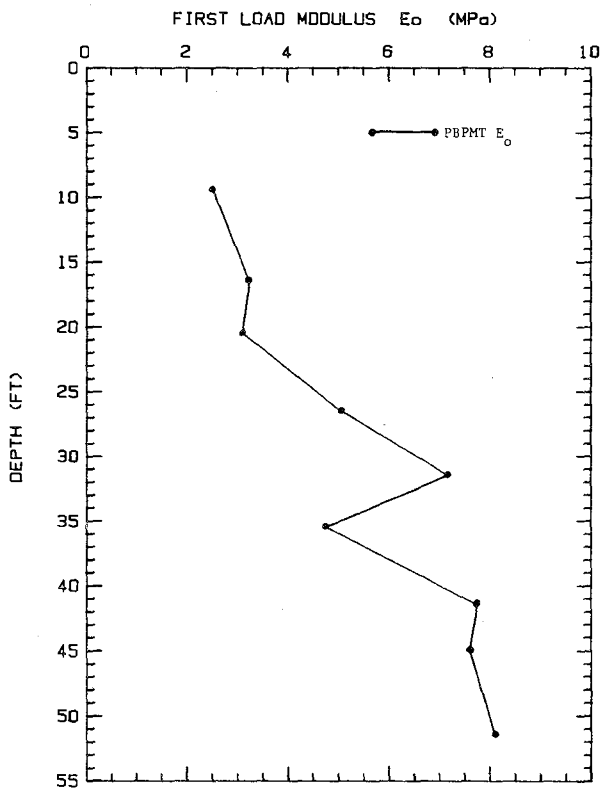

Fig. 6 Pressuremeter test, first load modulus (after Briaud).

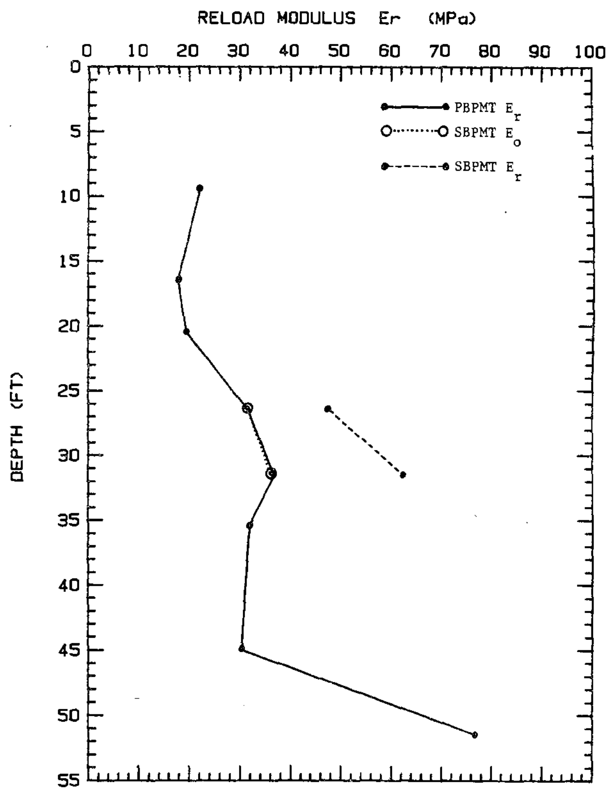

Fig. 7 Pressuremeter test, reload modulus (after Briaud).

Used dilatometer blade No. 1 ; gage range: 0-40 bars   Gage 0 = 0.03 (b)   diam. fr. red. = 7.15 cm

DM sounding location: Hunters Point Boring DHT 1 $Depth_{WT}$ = 3 m   eff. wt. rods = 7.74 kg/m

DM advanced by: push   1st depth = 2.44 m

(σ_{vt}) = 0.36 (b)

Membrane Calibration	BEFORE	AFTER	AVERAGE
(-)ΔA	0.23	0.19	0.21
ΔB	1.2	1.0	1.1

Depth		Advance dilat.		gage readings		Data reduction from HP-41C "D" program.									
ft	m	G, psi	F, kg	A(b)	B(b)	I_D	K_D	p_c(b)	K_o	c_u	ϕ°	M(b)	σ'_v(b)	OCR	Remarks
8	2.44	≈330	≈770	2.3	5.6	0.89	5.7	2.13	1.27	0.340	---	140	0.421	5.0	
9	2.74	≈330	≈770	0.9	5.1	3.24	2.0	0.68	0.54	---	30.6	115	0.491	1.4	Sand; 2 lengths
10	3.05	≈330	≈770	0.85	4.4	2.58	1.8	0.68	0.52	---	30.3	77	0.516	1.3	N rod
11	3.35	≈330	≈770	0.95	5.0	3.00	1.0	0.73	0.53	---	30.0	100	0.536	1.4	
12	3.66	≈330	≈770	0.9	4.9	3.21	1.6	0.68	0.51	---	30.0	88	0.555	1.2	
13	3.96	≈330	≈770	0.75	3.8	2.44	1.3	0.60	0.48	---	30.1	54	0.574	1.0	
14	4.27	320	724	1.1	4.45	2.03	1.8	0.84	0.55	---	29.4	66	0.594	1.4	
15	4.57	300	630	0.3	1.7	0.29	0.5	0.08	0.01	0.025	---	2.8	0.613	0.1	
16	4.88	300	630	1.1	4.4	2.10	1.6	0.89	0.58	---	27.5	62	0.627	1.4	3 lengths N rod
17	5.18	280	540	1.4	2.4	(-0.24)	2.1	0.72	0.58	0.155	---	(-10)	0.646	1.1	
18	5.49	340	817	1.6	4.95	1.49	2.2	1.18	0.62	---	28.9	76	0.660	1.8	
19	5.79	360	909	1.8	4.6	0.96	2.4	0.91	0.65	---	---	59	0.673	1.4	
20	6.10	370	956	1.55	5.8	2.41	1.8	0.98	0.54	---	30.3	104	0.694	1.4	
21	6.40	360	909	1.85	4.7	1.00	2.3	0.87	0.61	---	---	51	0.716	1.2	
22	6.70	330	770	1.9	4.7	0.95	2.2	0.87	0.60	---	---	54	0.737	1.2	
23	7.01	390	1050	2.15	6.3	1.66	2.4	1.42	0.62	---	30.0	116	0.758	1.9	
24	7.32	400	1000	2.4	8.1	2.38	2.5	1.60	0.65	---	29.3	198	0.779	2.1	
26	7.92	370	956	1.8	6.9	3.04	1.6	1.10	0.55	---	29.3	125	0.823	1.3	Lunch break,
27	8.23	390	1050	2.1	8.0	3.14	1.8	1.24	0.57	---	29.7	170	0.846	1.5	20 minutes.
28	8.53	460	1370	2.35	9.0	3.26	2.0	1.27	0.54	---	31.6	215	0.868	1.5	
29	8.84	490	1510	2.8	10.2	3.04	2.4	1.55	0.58	---	32.0	276	0.891	1.7	
30	9.14	530	1700	3.4	11.95	2.91	2.9	1.96	0.63	---	32.4	370	0.916	2.1	
31	9.45	610	2070	3.8	13.4	2.97	3.1	2.15	0.64	---	33.6	448	0.942	2.3	
32	9.75	640	2210	3.85	13.1	2.81	3.1	2.14	0.63	---	34.0	422	0.967	2.2	
33	10.11	690	2440	3.8	12.2	2.50	2.9	2.02	0.60	---	34.8	356	0.996	2.0	
34	10.4	610	2070	3.5	13.0	3.4	2.5	1.78	0.57	---	33.6	386	1.021	1.7	2 hour delay to
35	10.7	560	1840	3.5	9.6	1.87	2.6	1.99	0.60	---	32.4	213	1.046	1.9	repair drill rods.
36	11.0	480	1470	2.7	8.9	2.77	1.7	1.43	0.53	---	31.1	169	1.071	1.3	4 lengths N rod.
37	11.3	500	1560	2.9	9.0	2.48	1.9	1.54	0.54	---	31.3	171	1.093	1.4	
38	11.6	500	1560	3.1	9.7	2.56	1.9	1.67	0.56	---	31.2	199	1.115	1.5	
39	11.9	540	1740	3.2	9.5	2.32	2.0	1.69	0.55	---	31.8	186	1.140	1.5	
40	12.2	490	1510	2.8	9.5	3.13	1.6	1.45	0.52	---	30.8	173	1.165	1.2	
41	12.5	500	1560	2.6	8.2	2.76	1.4	1.31	0.49	---	31.2	133	1.187	1.1	
42	12.8	460	1370	3.1	10.0	2.88	1.7	1.72	0.57	---	29.7	192	1.209	1.4	
43	13.1	520	1650	3.9	9.9	1.72	2.3	2.28	0.62	---	30.7	189	1.234	1.8	
44	13.4	500	1560	4.3	9.6	1.285	2.6	2.74	0.67	---	30.0	171	1.257	2.2	
45	13.7	350	---	2.5	5.2	0.935	1.2	0.59	0.31	---	---	43	1.281	0.46	5 lengths N rod
46	14.0	400	1095	1.3	3.0	1.074	0.3	0.07	(-0.14)	---	---	12	1.298	0.05	
47	14.3	410	1140	2.6	6.2	1.544	1.2	1.54	0.55	---	28.4	71	1.316	1.2	
48	14.6	370	956	1.9	4.9	2.07	0.6	1.19	0.51	---	27.3	52	1.337	0.9	
48.5	14.8	≈370	≈956	2.0	4.6	1.41	0.7	1.27	0.52	---	27.1	40	1.350	0.9	
															Refusal; tip damaged.

Fig. 8 Dilatometer test, DT1 location (furnished by Handy).

Used dilatometer blade No. 15 mm; gage range: 0-40 bars | Gage 0 = 0.0 (b) | diam. fr. red. = 7.15 cm

DM sounding location: Hunters Point Boring DMT 2 | $Depth_{WT}$ = 3 m (est) | eff. wt. rods = 7.74 kg/m

DM advanced by: push | 1st depth = 2.44 m | (σ_{vt}) = 0.36 (b)

Membrane Calibration

	BEFORE	AFTER	AVERAGE
(-)ΔA	0.20	0.21	0.205
ΔB	1.2	1.05	1.025

Depth		Advance dilat.		gage readings		Data reduction from HP-41C "D" program.									
ft	m	G,psi	F,kg	A(b)	B(b)	I_0	K_D	p_c(b)	K_o	c_u	ϕ°	M(b)	σ'_v(b)	OCR	Remarks
8	2.44	400	1095	2.25	10.4	3.45	5.9	2.02	0.95	---	32.9	513	0.360	5.6	Sand; 2 lengths
9	2.74	570	1880	1.75	9.0	3.09	4.1	1.09	0.63	---	37.3	316	0.414	2.6	N rod
10	3.05	510	1605	2.2	9.25	2.90	4.6	1.62	0.75	---	35.2	384	0.463	3.5	
11	3.35	660	2300	2.35	10.25	3.20	4.5	1.47	0.68	---	37.7	438	0.488	3.0	
12	3.66	630	2160	2.6	10.4	2.86	4.7	1.7	0.73	---	36.8	440	0.513	3.4	
12'10"	3.91	*830	3099	4.0	12.3	1.97	7.0	3.48	0.98	---	37.9	560	0.534	6.5	Refusal; bored
14	---	---	---	---	---	---	---	---	---	---	---	---	---	---	to 14 ft
15	4.57	320	724	2.2	8.65	2.75	3.4	1.84	0.79	---	27.4	393	0.589	3.1	3 lengths N rod
16	4.88	460	1370	1.9	7.6	2.77	2.8	1.19	0.59	---	33.2	222	0.612	1.9	
17	5.18	530	1700	2.35	9.1	2.81	3.3	1.44	0.62	---	34.4	304	0.634	2.3	
18	5.49	590	1975	2.5	9.5	2.79	3.3	1.46	0.61	---	35.3	320	0.660	2.2	
19	5.79	580	1790	2.8	10.35	2.75	3.5	1.76	0.66	---	34.2	363	0.685	2.6	
20	6.10	580	1790	2.2	8.1	2.63	2.6	1.22	0.55	---	34.5	222	0.711	1.7	
21	6.40	520	1650	2.5	9.6	2.97	2.8	1.45	0.60	---	33.6	300	0.733	2.0	
22	6.71	610	2070	3.1	11.4	2.87	3.4	1.83	0.64	---	34.8	401	0.759	2.4	
23	7.01	820	3045	3.8	13.0	2.60	4.1	2.21	0.67	---	37.1	494	0.784	2.8	
23'6"	7.16	390	*2260	5.5	18.1	2.52	5.9	4.61	0.96	---	34.1	842	0.796	5.8	Refusal; bored
26	7.92	690	2440	5.1	20.3	3.56	4.7	3.49	0.81	---	34.5	942	0.870	4.0	to 26 ft
26'6"	8.08	*900	*3415	5.4	21.2	3.5	4.9	3.46	0.78	---	36.8	1002	0.886	3.9	4 N rods; Refusal; test stopped

Fig. 8a. Dilatometer tests, DT2 locations (furnished by Handy).

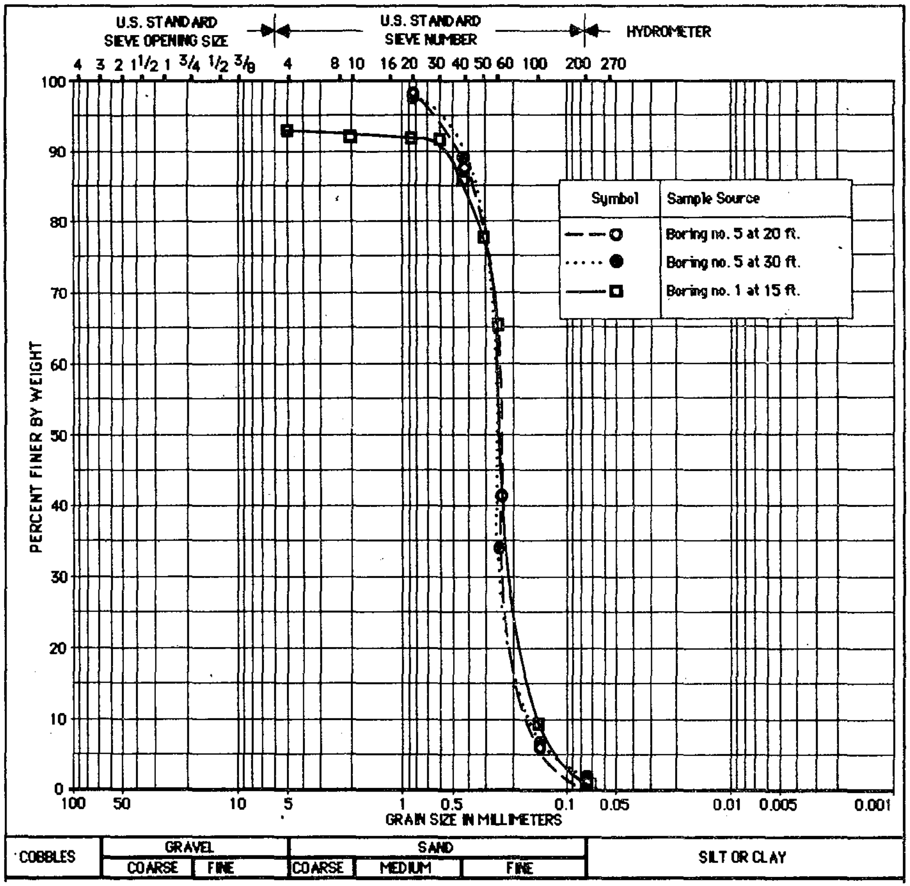

Fig. 9 Sieve analysis test data.

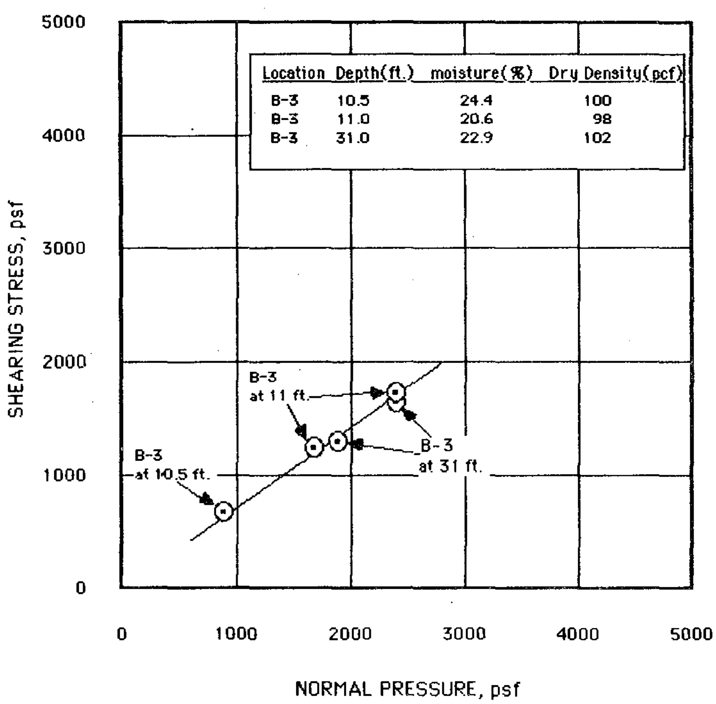

Fig. 10 Direct shear test data.

The dilatometer tests were performed at the test site by Dr. Richard L. Handy of Iowa State University. The dilatometer blade, which is flat and has a width of 3.7 inch (94 mm), was pushed by a rotary wash drill rig. The results of the dilatometer soundings, as reported by Dr. Handy, are presented on Figure 8 (Handy, 1986).

Samples obtained from the Sprague Henwood samplers were selected by GRC for laboratory tests. Direct shear strength tests, grain size analyses, and moisture/density determinations were performed. The results are presented in Figures 9 and 10.

PILE INSTRUMENTATION

The test pile selected for load tests consisted of a single control pipe pile and a group of five pipe piles, all closed end steel piles. The diameter of the pile was 10.75 inch (273 mm),and had a wall thickness of 0.365 inch (9.27 mm). The instrumentation of each pile was designed to obtain the load transfer behavior of these piles and to achieve residue stress measurements. The instrumentation was selected based upon their known durability during pile driving and their ability to provide steady results during load tests. A summary of the pile instrumentation program for this study is presented in Table 2. Dr. D. Michael Holloway provided consultation during design and supervised the installation.

Weldable resistance strain gages (Eaton 350 ohm half bridge) were selected for all pile strain measurements. The strain gages were installed in pairs at 7 levels along the length of the pile. The gages were welded on the pile surface, on diametrically opposite sides. The half bridge formation was selected to allow proper temperature compensation measurements. The bridge was completed at the digital Data Acquisition System (DAS). The locations of the 7 levels of strain gages along the pile for the single control pile and group piles are presented on Figure 11. For all five group piles, the top strain measurement level consisted of 4 strain gages spaced at 90 degree centers around the pile, above grade and below the pile cap to allow accurate measurement on axial loads delivered to each pile. To minimize damage from field conditions and pile driving, all strain gages were covered

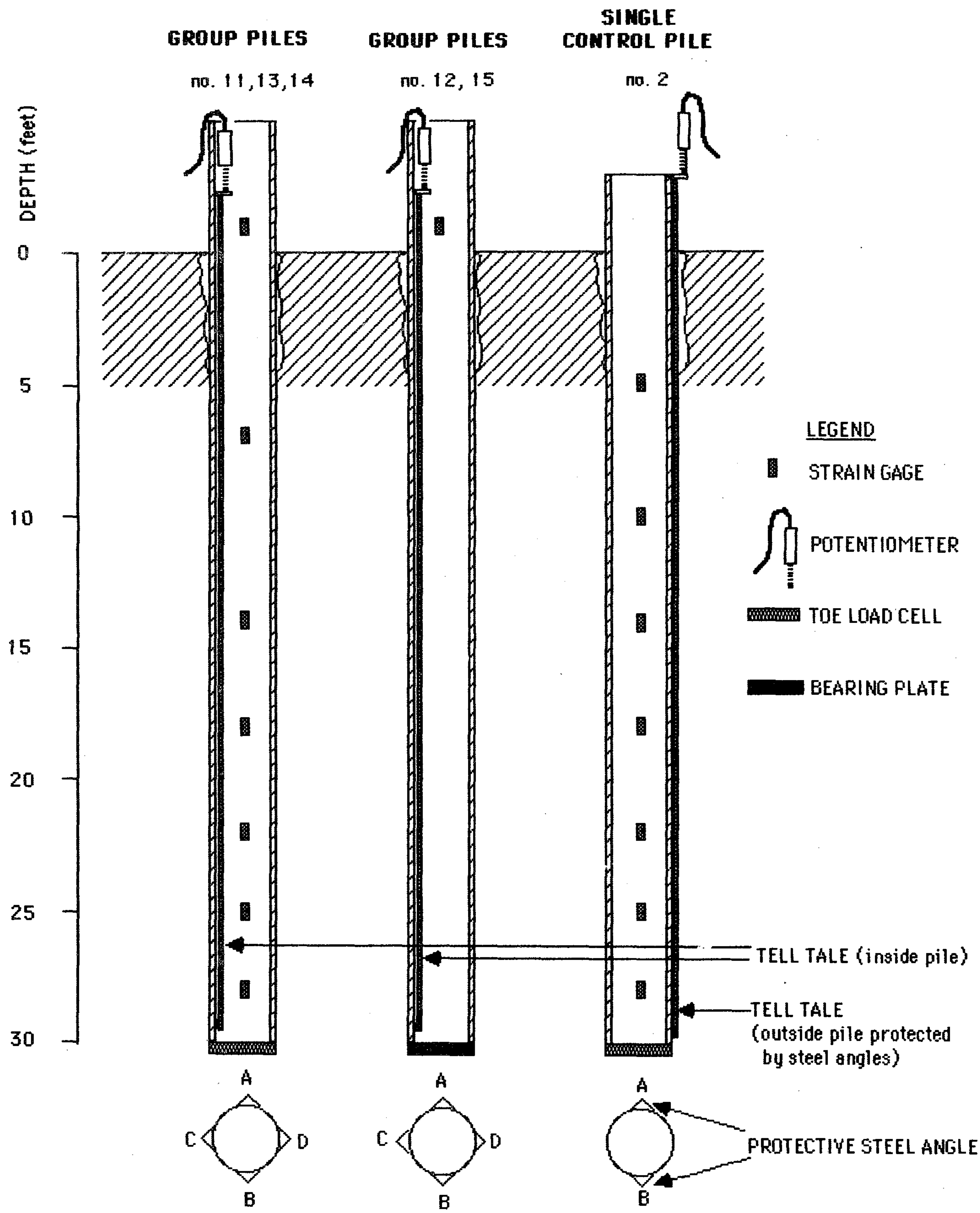

FIG. 11 - INSTRUMENT LOCATIONS ON PILES
(1 feet = 0.305 meter)

PILE TYPE		STRAIN GAGE no. of levels	TOE LOAD CELL in each pile	TELL TALE in each pile
SINGLE	Closed end 10.75" ∅ 0.365" steel pipe (Single control pile)	7	2	1
GROUP	Closed end 10.75" ∅ 0.365" steel pipe (Three piles)	7	2	1
	Closed end 10.75" ∅ 0.365" steel pipe (Two piles)	1	none	1

Strain gage - Eaton SG 359 weldable resistance gages, two at each level
Toe load cell - Hydraulic total stress cells with one pneumatic pore pressure sensor
Tell tale - 3/4" dia. steel rod anchored near the toe of pile

TABLE 2
SUMMARY OF PILE INSTRUMENTATION

EXTENSOMETER \ DEPTH (ft)	0	10	20	30	40
E1	4.1	4.3	2.6	0.6	0.6
E2	3.3	3.2	1.4	-0.3	-0.6
E3	3.3	3.0	1.5	0.5	-0.9
E4	5.5	5.3	3.6	1.5	0.7
E5	3.9	3.7	2.2	N.A.	-0.5

TABLE 3 - GROUND SETTLEMENT INDUCED BY REACTION PILE DRIVING
(INCHES)

EXTENSOMETER \ DEPTH (ft)	0	10	20	30	40
E1	0	0	0	0	0
E2	0.9	0.9	0.4	0.1	0
E3	0.9	0.8	0.5	-0.3	-0.1
E4	1.5	2.3	1.0	0.3	-0.1
E5	1.2	1.4	0.9	N.A.	-0.2

TABLE 4 - SETTLEMENT INDUCED BY TEST PILE DRIVING
(INCHES)

by 2-1/2 inch steel angles along the entire length of the pile. The space between the steel angle and the pile surface was filled with injected polyurethane foam.

The toe stresses at the tips of the piles were measured by hydraulic toe load cells furnished by Petur International of Seattle, Washington. The load cells were comprised of two half circle stress cells straddled over a central slot to allow placement of a pneumatic pore water pressure sensor. Stainless steel hydraulic tubings for the stress cells were led through the inside of the pile to the top of the pile. A silicone strain gage pressure transducer converted the pressure to analog voltage, which was read by the Data Acquisition System.

For the group piles, tell tales (rod extensometers) were fabricated from a 3/4-inch round steel rod. One end of the steel rod was welded to the inside wall of each steel pile, about 6-inches above the pile toe. The steel rods were lubricated with grease and sheathed in plastic tubing to facilitate free rod travel between the toe and the pile butt. The hollow pipe piles were then filled with insulating material in order to restrain lateral movement of the tell tale inside the pile. For the single pile, similar tell tale was welded to the outside of the pile surface protected by steel angles. The tell tale potentiometer was mounted on the butt of the pile such that the DAS readings recorded pile toe displacement relative to the pile butt. For the group piles, the potentiometers were mounted on the pile wall near the pile top. Under these conditions, the relative movement between the toe and top was recorded, such that toe displacement must be subtracted from pile top displacement in order to determine absolute pile toe displacements.

Both 2-inch and 4-inch-stroke linear potentiometers (models 518125 and 518126) were furnished by Slope Indicator Company of Seattle, Washington to provide displacement measurements utilizing the DAS. Two-inch potentiometers were used primarily to measure the tell tale and the pile cap displacements in the two horizontal orthogonal directions. Four-inch potentiometers were used for the vertical displacement. Companion dial gages were used to provide manual reading to verify electronic displacement readings. Except at the tell tale, all potentiometers and dial gages were securely mounted to the reference beam(s) by both magnetic and C-clamp stands in order to monitor absolute displacements at the prescribed measurement locations.

The applied load was measured by a 250-ton capacity, resistance strain gage butt load cell furnished by the FHWA. The FHWA load cell was used in the pile calibration efforts, and throughout load testings of all the piles. In the group pile load tests, three additional 250-ton load cells were rented from Dudgeon Co., for use in monitoring the axial loads applied to the pile group. In all cases, lubricated spherical bearing plates were employed in the loading scheme to minimize eccentricity effects. Each load cell was configured in full Wheatstone bridge circuits to minimize eccentric loading effects on the axial load readout. During load tests, the load cells were monitored by the DAS and cross-checked with hydraulic gage readings from the pump.

A multi-channel digital data acquisition system (DAS) for sampling, amplifying, storing, and presentating data from pile load tests was furnished by GRC. The software and signal processing hardware packages were developed by GRC. The DAS acquired data from up to 64 analog input signals, amplified the signals either directly or via a Wheatstone bridge, sampled each channel 100 times, averaged the 100 samples, and stored the results on a microcomputer using Unix operating system. Sampling intervals for the load test were controlled by the microcomputer. Continuous readout of the load cell(s) enabled close monitoring of the progress of the load test. The stored data were displayed on a graphics screen for real-time visualization of the pile load test. Hard copy plots of the results were displayed on a flat bed plotter as the test progressed or immediately after completion of the tests. Plots of load test data were immediately available upon completion of a load test. Some of these plots are presented later on in the discussion of load test results.

PILE CALIBRATION

An extensive pile calibration effort was performed at the Kiewit-Pacific Company Yard in Pleasanton, California, where all the test piles were prepared. The calibration effort was performed between March 14 to April 8, 1986. The objective of this calibration program was to ensure accurate measurement of applied load, repeatability of instrument readings and verification of DAS measurements with manual readout. A loading frame was designed and fabricated by Kiewit-Pacific Company for pile calibration under axial load. The FHWA load cell was re-calibrated in Pleasanton by Pacific Calibration Services (PCS) on March 13, 1986. The PCS calibrated load cell was used in series with the FHWA load cell in the pile calibration loading frame. The three load cells from Dudgeon Co. were also calibrated in a testing laboratory prior to their use.

A manual readout scheme (Vishay P350 Strain indicator and 10-Channel SB-2 Switch and Balance unit) was also available to monitor load cell and strain gage reading during calibration. The output from the pressure transducer for each toe pressure cell was obtained with a portable digital voltmeter, using the DAS external power supply for excitation.

Each pile was cycled in compression to its maximum anticipated load, and then unloaded. The process was continued until a reasonably repeatable load-strain cycle was observed. Thereafter, two to three "calibration cycles" were applied in stages to obtain sufficient data points to characterize the behavior at each gage level. The strain gages generally were configured as full bridges using the two half-bridge pairs at each level. On occasion when only one functioning gage was available, the half-bridge circuitry of the P350 unit was engaged. At least one load cycle was monitored with the automatic DAS for documentation purpose.

GROUND INSTRUMENTATION

Pneumatic transducers (SINCO MODEL 514178) were used to monitor the in-situ pore pressure data in the vicinity of the test piles. The 1-inch transducer was operated with a 2-tube configuration. The transducer was embedded in the borehole and was sealed off from other water-bearing areas with bentonite pellets. The fluid pressures sensed by the transducers were converted into pneumatic pressures and were relayed to a remote reading terminal by means of polyethylene tubing in a water-proof polyethylene jacket. The polyethylene tubings were protected from any possible damage by sand and wood covering during setup and by PVC conduit during the load testing program.

The settlement at the site during pile driving and load test were measured by a SINCO five-position, rod type borehole extensometer assembly. The extensometer assembly consists of five fixed anchors hydraulically expanded to lock the prongs firmly into the wall of the borehole at various depth points, approximately 10-, 20-, 30-, and 40-foot depths and firm bedrock. Each of the five fixed anchors was connected by a 1/4-inch diameter stainless steel rod within a PVC jacket to a reference head mounted on a 3 inch ID steel pipe embedded in the borehole at the ground surface. The fixed anchor locked into the lower bedrock served as a fixed reference. The relative movements were measured mechanically and periodically during different activities at the site. The location of piezometers and extensometers at the site are presented on Figure 2. The settlement data from the extensometers measured at various time in the field are presented on Tables 3 and 4.

TEST PILE INSTALLATION

The test piles were installed at the test site as shown on the Test Pile Configuration on Figure 12. All reaction piles required to support the load frame were installed first. For the single control pile, 4 steel piles 12H53 were used. For the group piles, 16 steel 14H73 piles were installed to support a 1,000-ton load frame. All pile locations including test piles, were predrilled to a depth of about 4.5 feet with a 14 inch diameter flight auger. The piles were driven by a Delmag D22 diesel hammer with a maximum rated energy of 40,000 foot/pound. The reaction piles were driven to about 55-foot depth, at least 5 feet into the serpentine bedrock. Each reaction pile (12HP53) was designed to have 100 kips pullout capacity. All group reaction piles were also driven to the same depth and were designed to carry 125 kip pullout capacity. Each type of reaction pile was load tested for pullout capacity to verify the design.

Prior to any test pile driving, it was ensured that all instruments had been properly calibrated. The test piles were placed horizontally on the ground at the test site and their zero reading taken. The single test pile was driven first and followed by group test piles. The order of pile driving and the blow counts recorded during driving are presented in Table 5. All test piles were driven 30 feet into the ground.

During test pile installation, the pile was monitored by a Pile Driving Analyzer (PDA), performed by Dr. D. Michael Holloway of InstuTech. The PDA monitoring device was Model GB PDA manufactured by Pile Dynamic Inc. The results of PDA monitoring on all test piles during the last 2 feet of pile driving are presented in Table 6.

At the end of pile driving, all ground and pile instrumentations were measured. Both ground and pile instrumentation were monitored periodically during load test set-up preparation.

LOAD TEST PROCEDURE

The load test was conducted first on the single control pile. A load frame was fabricated at the site by welding a steel cross beam onto four reaction H piles. A 300-ton hydraulic pump was used for the load test. The load was monitored throughout the test by using the 250-ton capacity FHWA load cell.

The displacements were measured by both potentiometers installed on the top of the pile for top movement, on the sides of the piles for horizontal movements and from the top of tell tale for toe movement. The potentiometers were connected to DAS. Manual dial gages were installed along side each potentiometer for backup measurements. The load transfer characteristics at the single pile was measured by reading the 7 levels of strain gages along the side of pile and by toe load cell at the tip of the pile together with the accompanying pore pressure data. Ground instrumentation data were read prior to the tests and periodically during the test.

Prior to applying the load, all gages were re-zeroed. The load was applied at an increment of 10% of estimated ultimate load.

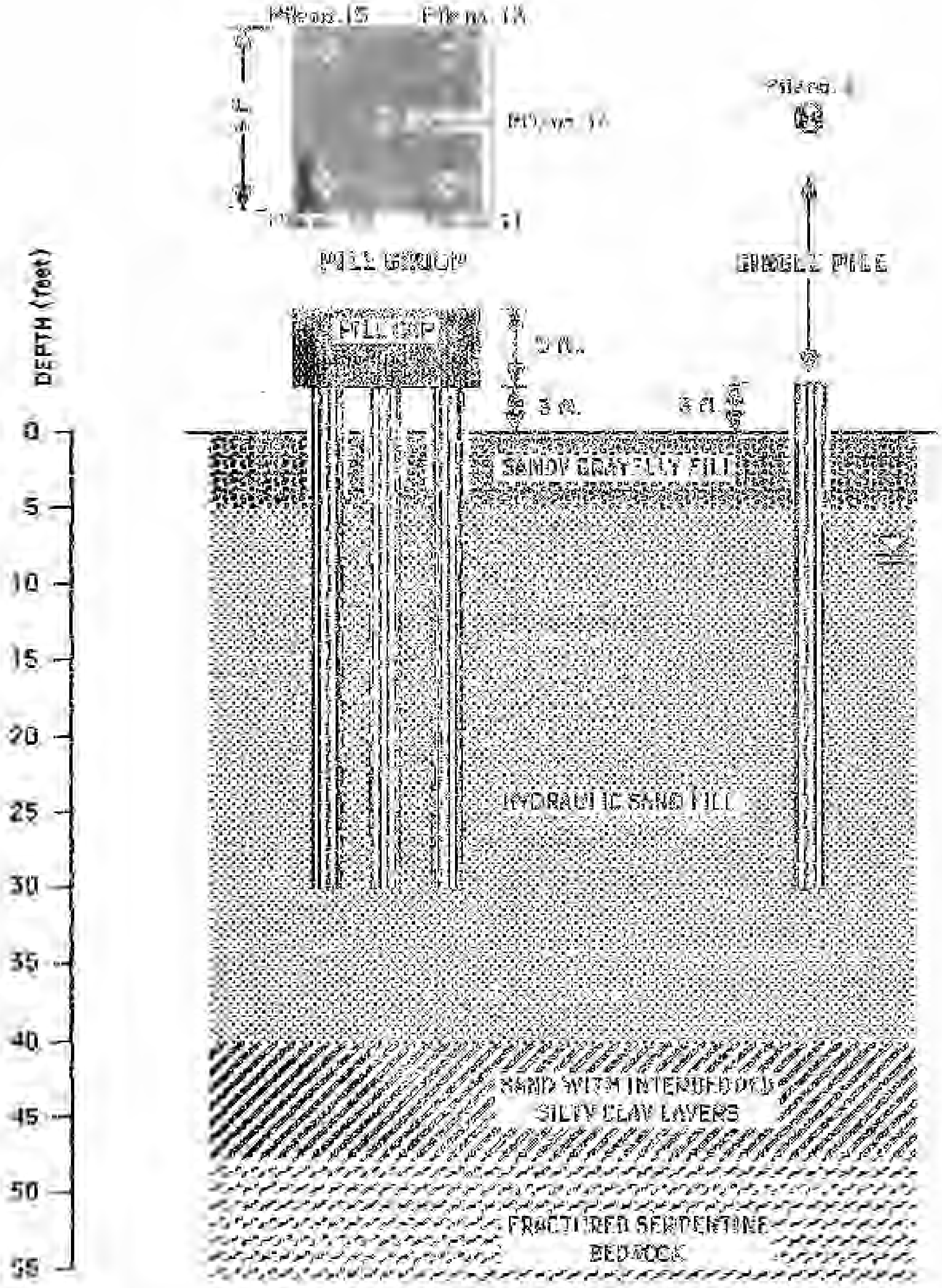

FIG. 12 – TEST PILE CONFIGURATIONS

(1 foot = 0.305 meter)

PENETRATION (FEET) \ PILE (DATE & TIME GIVEN)	SINGLE	GROUP				
	2	14	13	11	12	15
	4/30/86	4/30/86	4/30/86	4/30/86	4/30/86	4/30/86
	9:50	11:15	11:47	12:40	13:05	13:42
1	RUN ↓	↓	RUN ↓	RUN ↓	RUN ↓	RUN ↓
2	↓	↓	↓	↓	↓	↓
3	↓	4	↓	↓	1	↓
4	↓	↓	1	1	2	↓
5	↓	↓	2	2	2	↓
6	2	2	1	2	2	2
7	2	1	1	2	2	2
8	2	1	2	3	3	2
9	2	1	2	5	2	2
10	2	1	2	2	2	2
11	2	2	2	3	2	2
12	2	3	3	4	3	4
13	2	3	4	6	2	4
14	2	3	5	6	3	4
15	2	3	4	6	3	6
16	2	3	4	6	2	5
17	2	2	5	6	3	4
18	2	2	5	5	3	4
19	2	2	4	4	2	4
20	2	2	3	5	2	4
21	2	2	2	4	2	3
22	3	2	2	6	2	4
23	2	2	3	5	2	6
24	2	2	4	5	4	7
25	2	2	6	6	7	8
26	2	3	6	7	5	7
27	3	3	6	9	8	8
28	5	3	7	7	7	8
29	5	5	7	8	8	9
30	6	7	8	9	9	11

TABLE 5 - TEST PILES DRIVING RECORD (Blows/Foot)

SINGLE PILES

PILE NO.	BLOW COUNT, FINAL, ft.	EMAX,[1] kip-ft.	FMAX[2] kips	RMAD,EOI[3] kips
2	6	9-16	215-300	80-120

GROUP PILES

PILE NO.	BLOW COUNT, FINAL, ft.	EMAX,[1] kip-ft.	FMAX[2] kips	RMAD,EOI[3] kips
11	12	14-16	255-275	150-160
12	6	9-11	230-250	90-100
13	7	10-13	245-270	140-165
14	15	8-12	185-220	100-105
15	7	8-12	235-265	135-140

NOTE:
1. Maximum Energy
2. Maximum Force
3. Maximum Capacity At End of Driving

TABLE 6 - PILE DRIVING ANALYZER RESULTS
(AFTER HOLLOWAY 1987)

For the single pile, it was applied at an interval of about 10 kips. Each load increment was held for half an hour to allow the reading to stabilize. The pile instrumentation was read every 5 minutes or on demand. The load was continuously monitored by the DAS, allowing minor load adjustment to be made to maintain the load.

Manual reading of the dial gages was performed on the average of three times within the first half-hour during each load interval. The pile was pushed to ultimate failure as defined by tip movement of at least 3 inches. After achieving ultimate failure load, the pile was unloaded in 25% decrements of maximum load and each load interval was held for 5 to 10 minutes. The load test was completed when the load was totally released and its final readings recorded. During the test, all the reaction piles and the reference beam were surveyed periodically with a level to ensure that their position remained unchanged.

The general load test procedure was followed for the group pile test. A mobile load frame of 1,000-ton capacity was used. A reaction pile system consisting of 16 H piles was used to provide reactive support. The load was applied by four 250 ton jacks connected to a single pump. Each individual jack has the ability of providing fine adjustment to maintain equal and constant load. Three 300-ton and one 250-ton load cell were used to monitor the load. The load cells were symmetrically placed on the top of the 6 by 6 by 5 foot cast-in-place concrete pile cap. Both vertical and horizontal displacements were measured by potentiometers and dial gages at each of 4 corners of the pile cap. All pile instruments were electronically monitored. The ground instrumentation and level data were kept manually throughout the test.

The loading and unloading procedure generally followed the single pile procedure. The load increments were on the order of about 20 kips and the decrements were on the order of 25 percent of maximum load. The pile group was pushed to a maximum cap displacement of about 7 inches, in order to achieve ultimate failure.

During load tests, continuous monitoring of critical data channels, such as the load cell, was performed approximately once every second and displayed numerically on an auxiliary monitor screen. Additional numerical data from the last reading cycle were also displayed. Data sampling was controlled by the microcomputer. Typically, there was a 5-minute interval between each sampling cycle. At each sample cycle 100 samples of data were acquired from each channel. The 100 samples were averaged and the mean of the data from each channel was printed on a hard copy printer along with the number of samples that were within ± 2 digital units of the mean. These results were then stored on hard disk for later retrieval.

With a few exceptions, most of the pile and ground instruments performed satisfactorily during the test. The digital data acquisition system worked successfully on all pile load tests, providing accurate data and immediate results during load tests.

LOAD TEST RESULTS

Raw data from the load test for the single control pile are presented in Figures 13 through 16. In presenting these data, raw data from malfunctioned instrumentation was deleted from the presentation. Group pile test data are presented on Figures 17 through 26. These data consisted of the butt and toe loads versus time, load in pile as measured by strain gages, pile tip stresses as obtained from the toe load cells, top of pile displacement, and finally load settlement characteristics. All the data were measured without residue stress consideration since all the instruments were re-zeroed before the test. Analytical results including residue stresses consideration will be presented in a final report and are also summarized in the paper by Briaud and Tucker from these Proceedings.

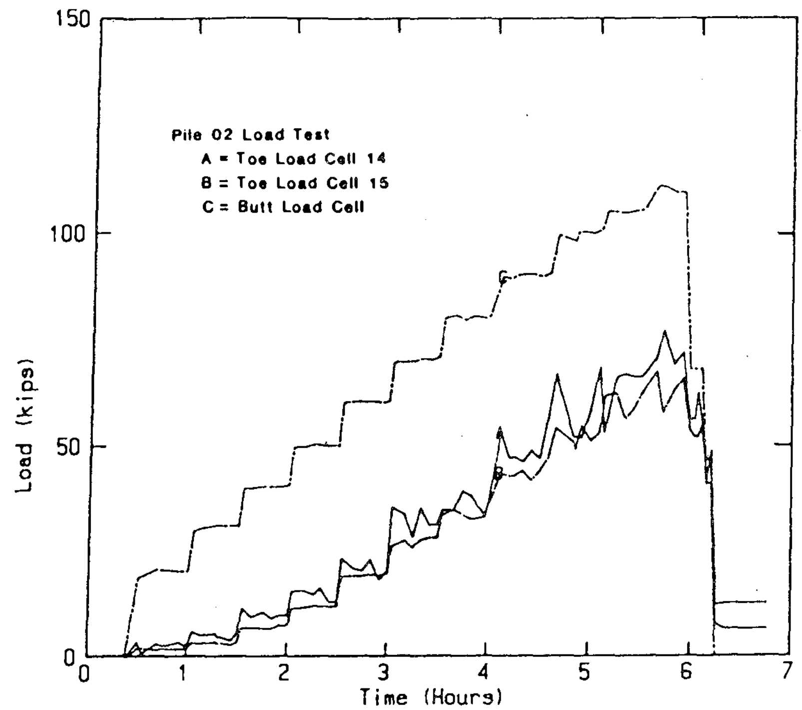

Fig. 13 Butt load and toe load cell results, single control pile.

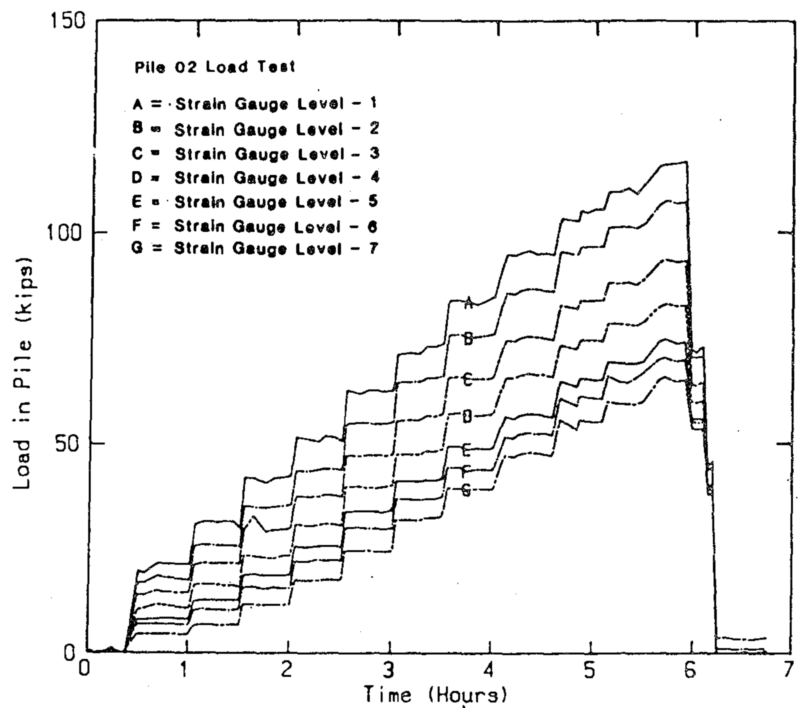

Fig. 14 Load in pile measured by strain gauges, single control pile.

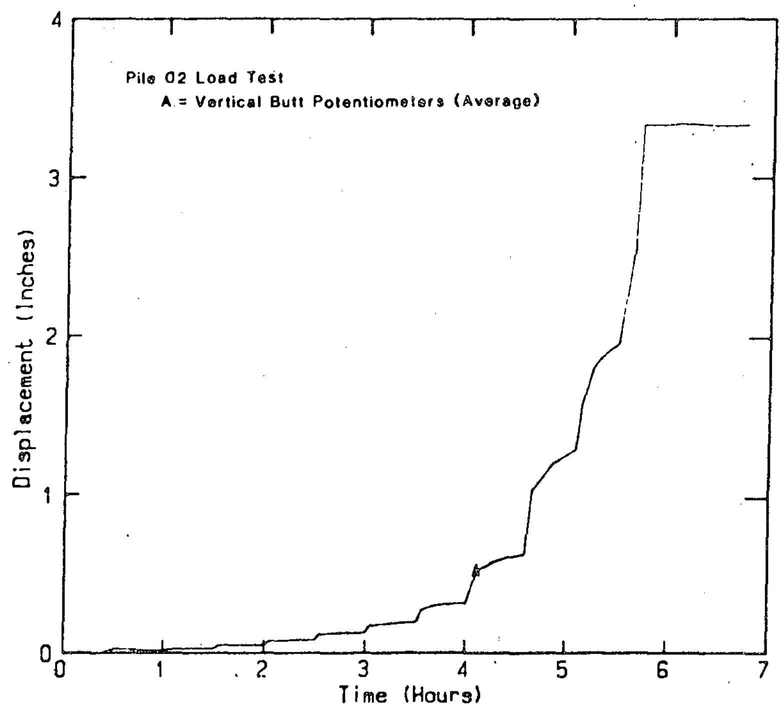

Fig. 15 Average pile top displacement, single control pile.

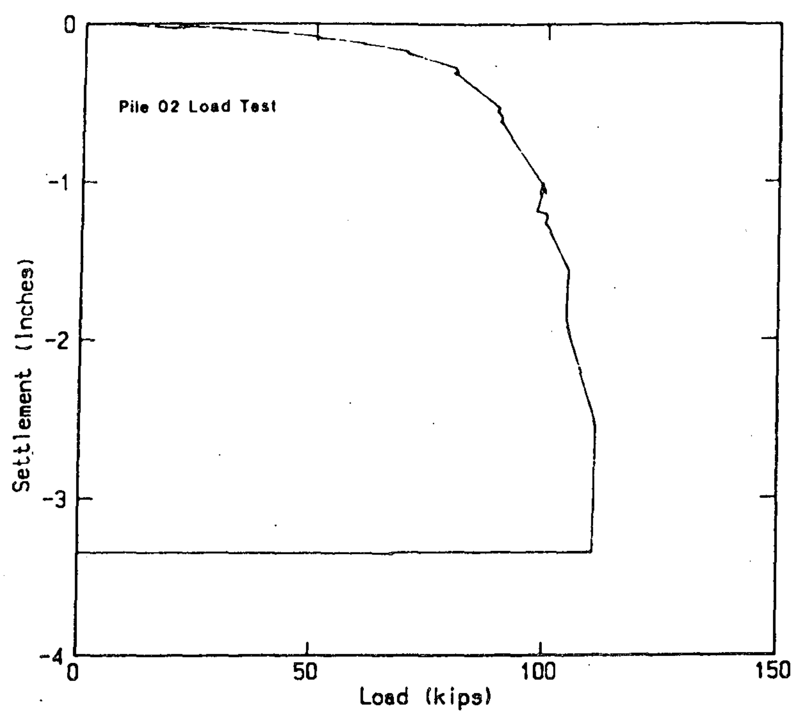

Fig. 16 Butt load–top settlement curve, single control

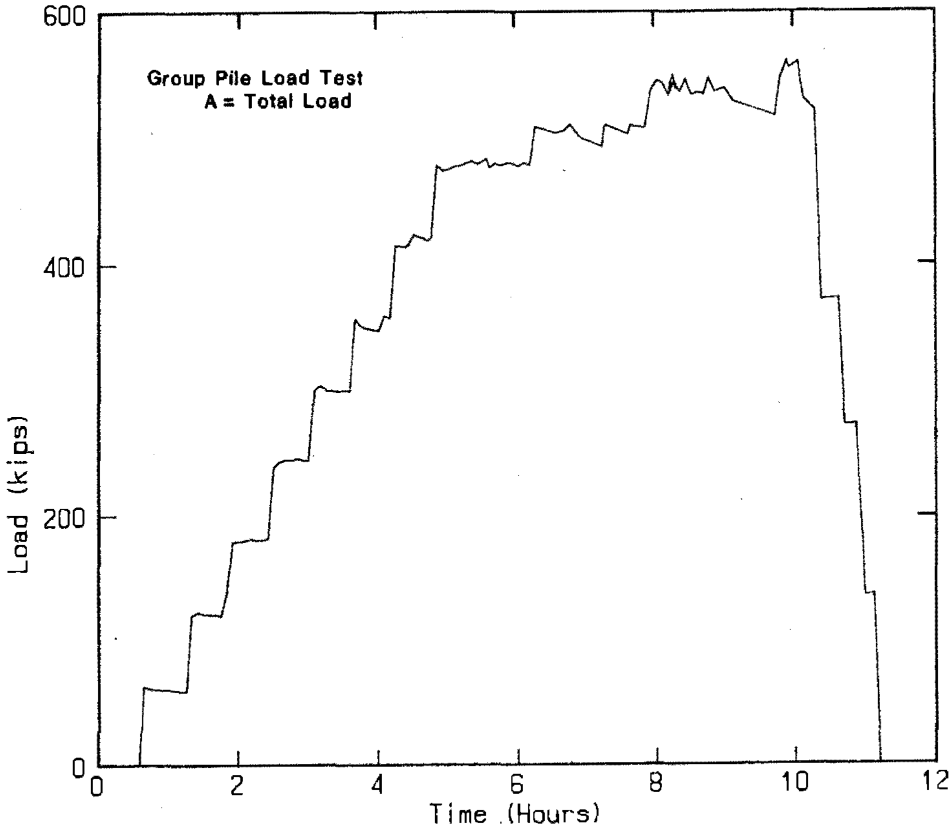

Fig. 17 Total butt load on pile cap (Group piles).

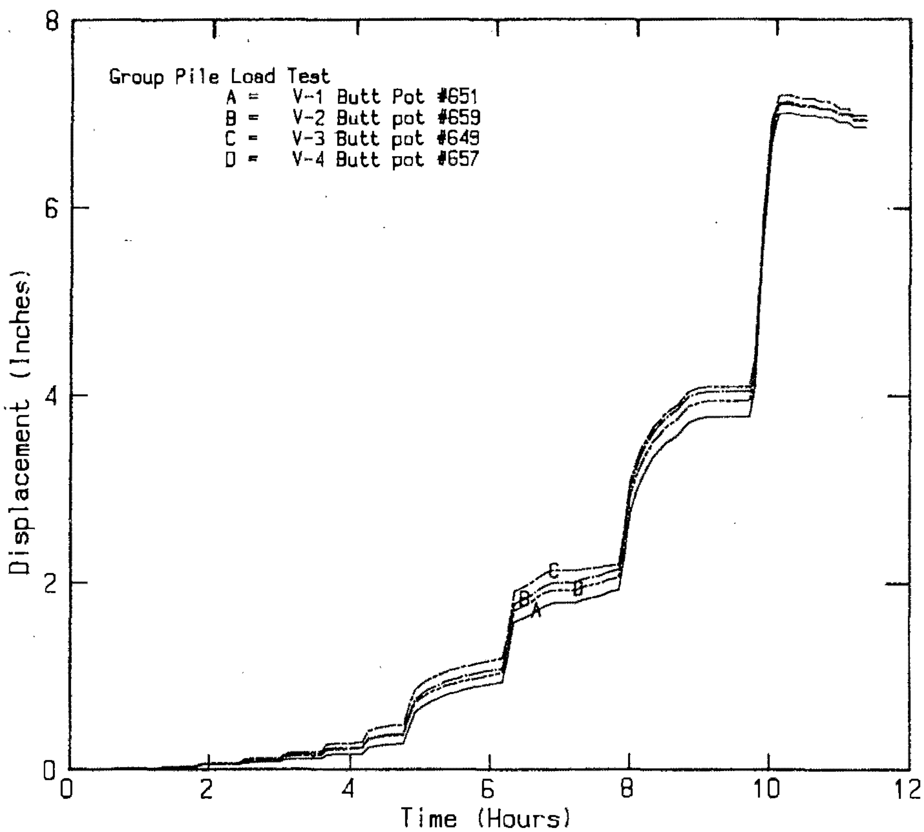

Fig. 18 Vertical displacement at four corners of pile cap, group piles.

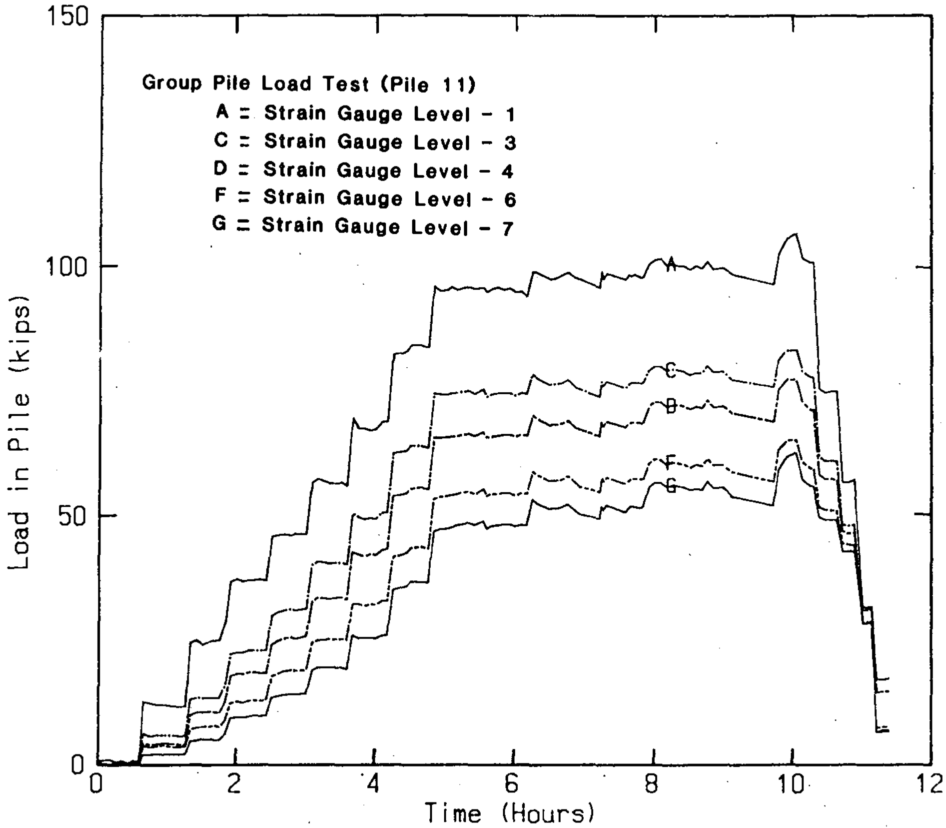

Fig. 19 Load in pile, Pile 11 of Group pile.

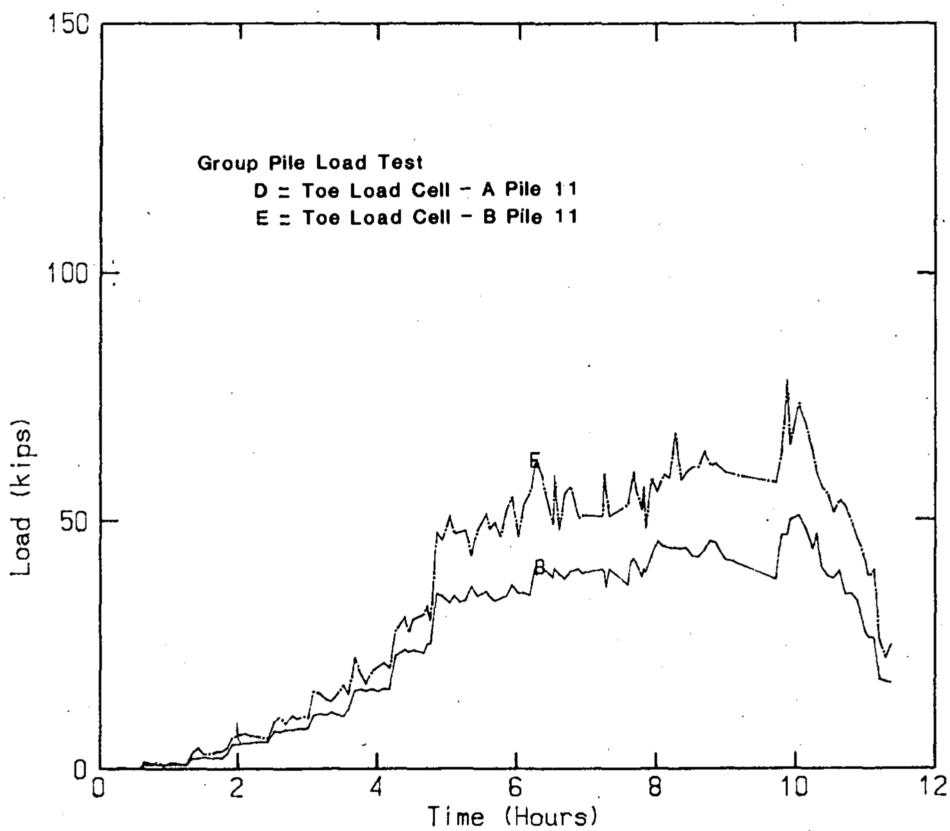

Fig. 20 Measured toe loads, Pile 11 of Group pile.

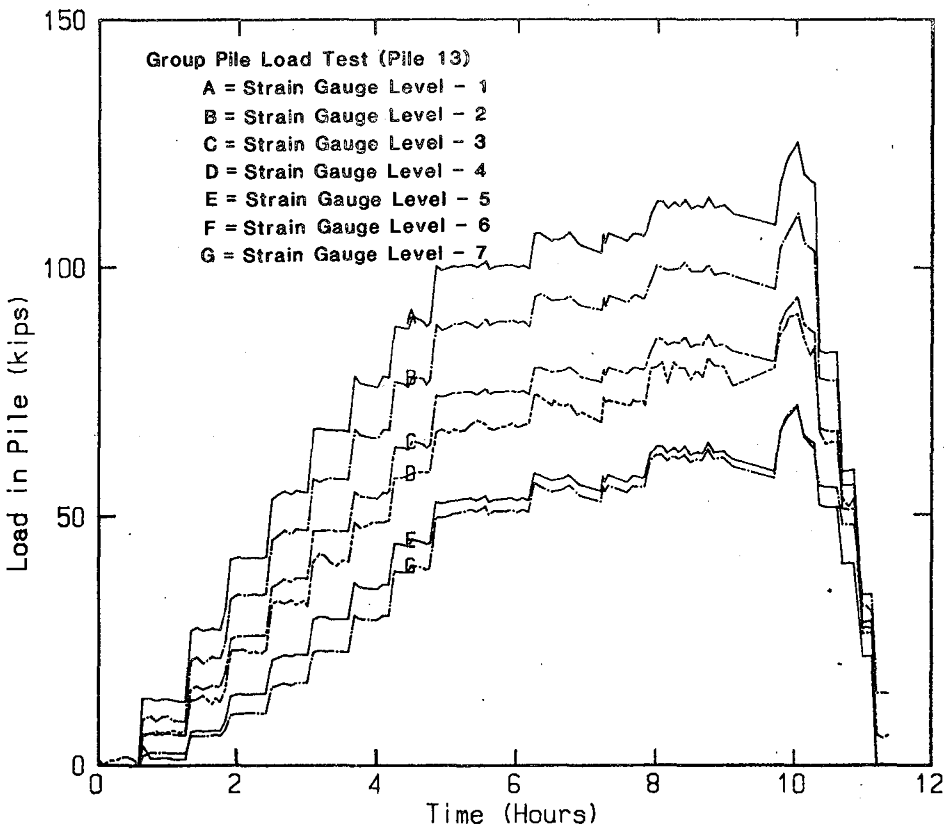

Fig. 21 Load in pile, Pile 13 of Group pile.

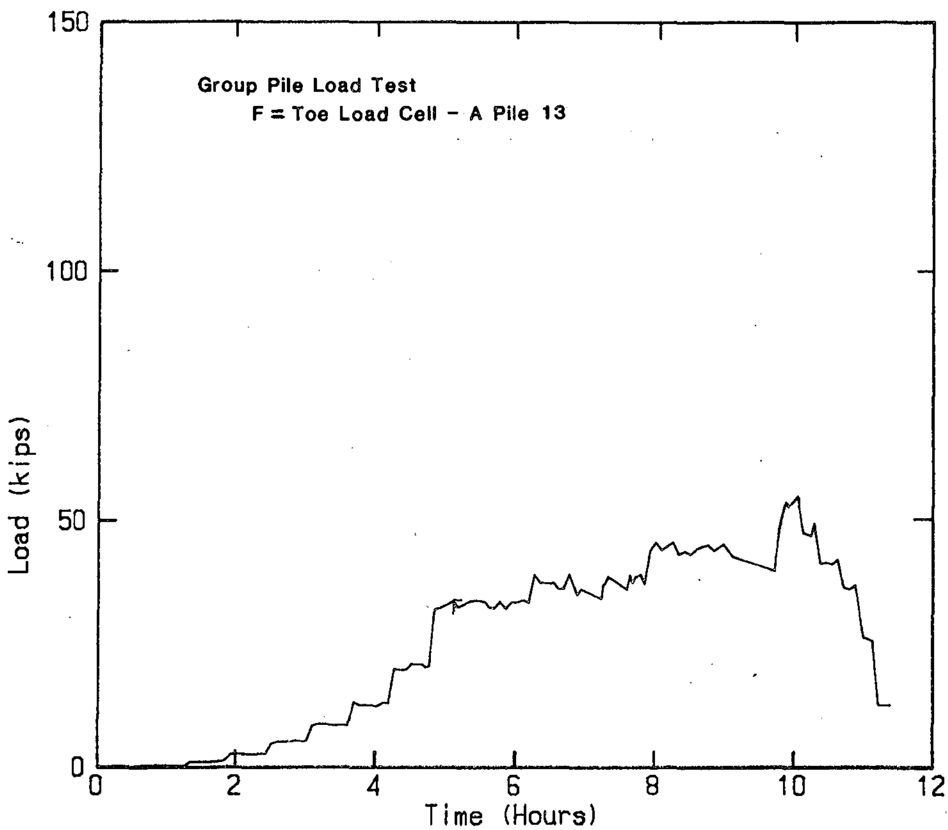

Fig. 22 Measured toe load, Pile 13 of Group pile.

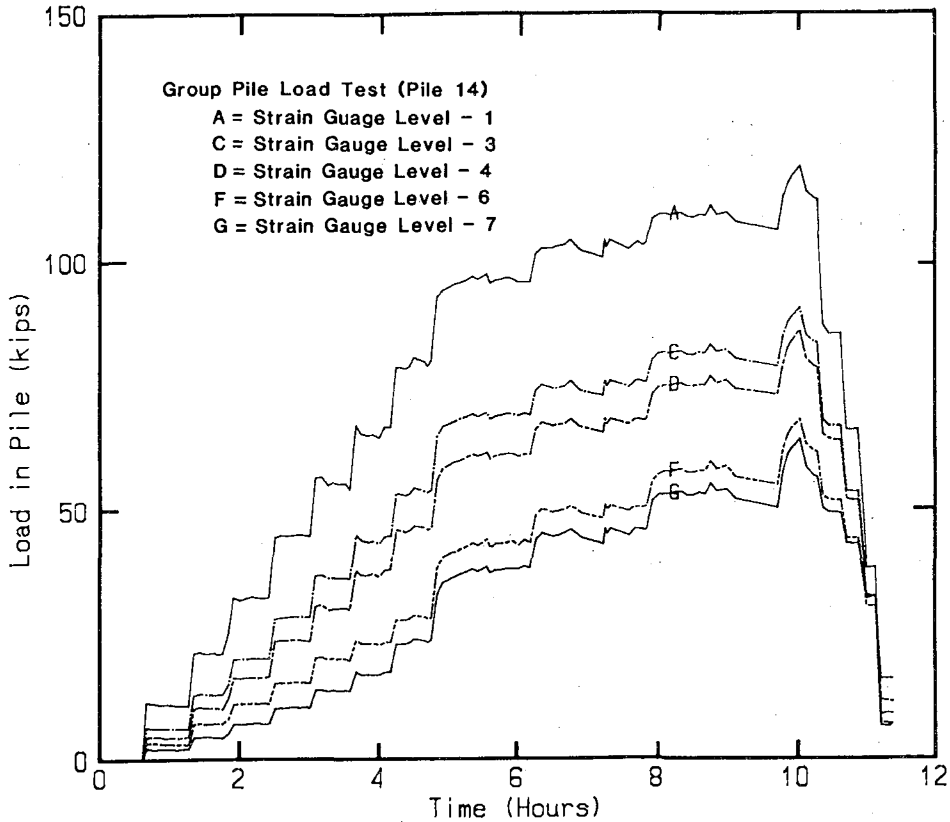

Fig. 23 Load in pile, Pile 14 of Group pile.

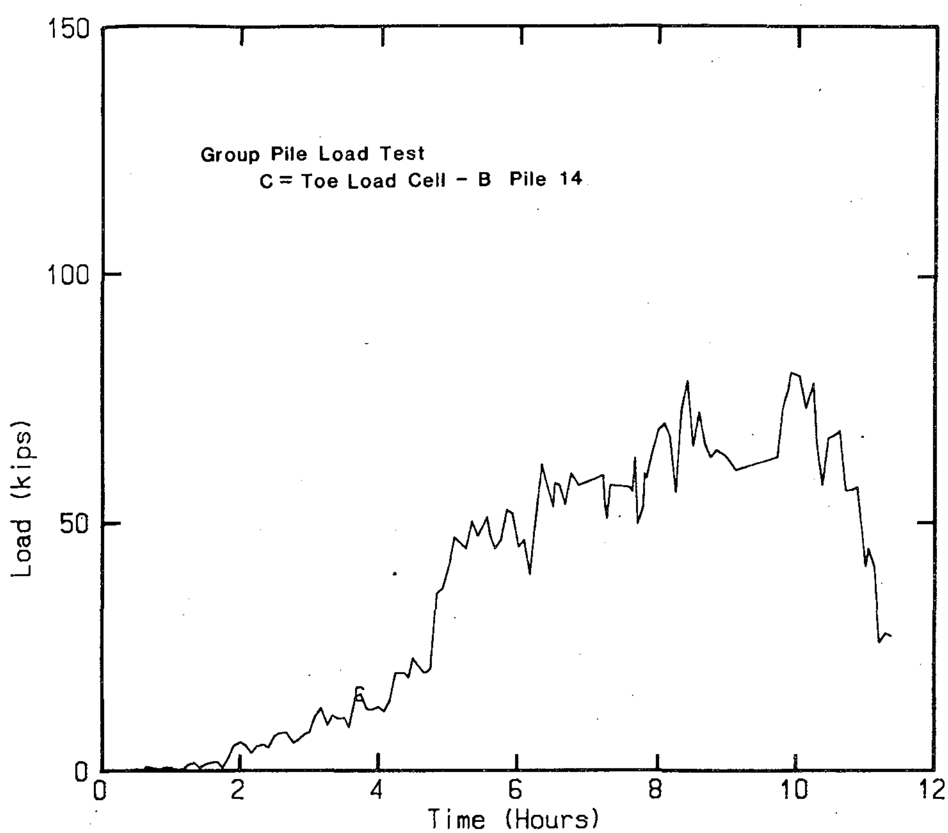

Fig. 24 Measured toe load, Pile 14 of Group pile.

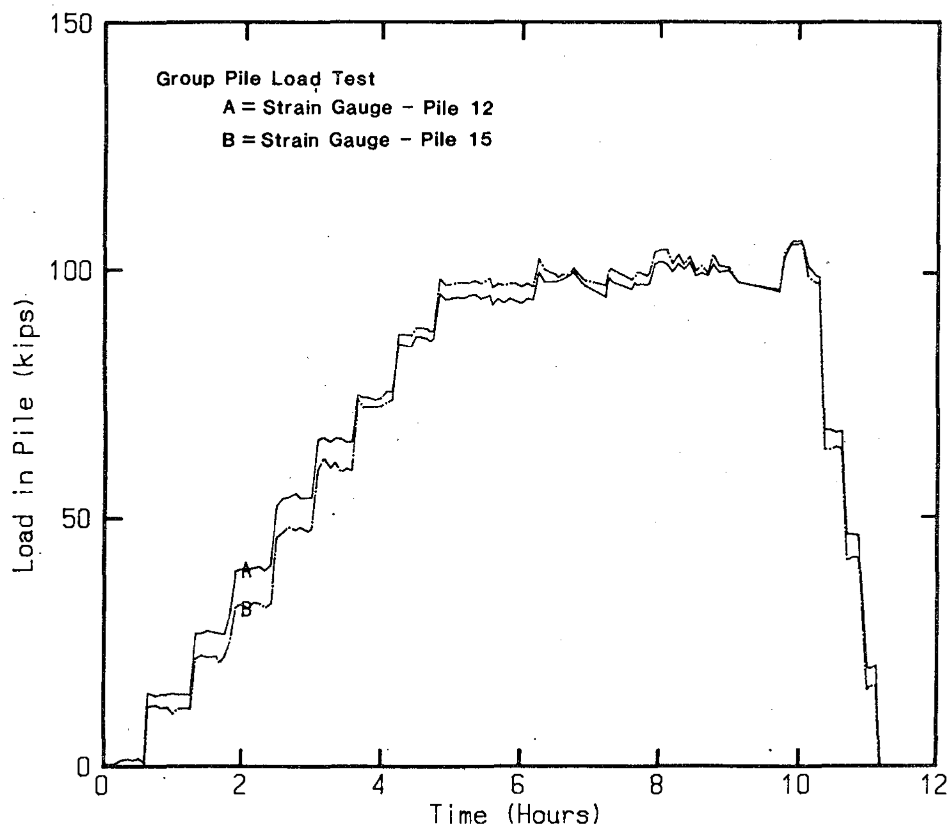

Fig. 25 Load in pile for Pile 12, 15, 1st level (Group Piles).

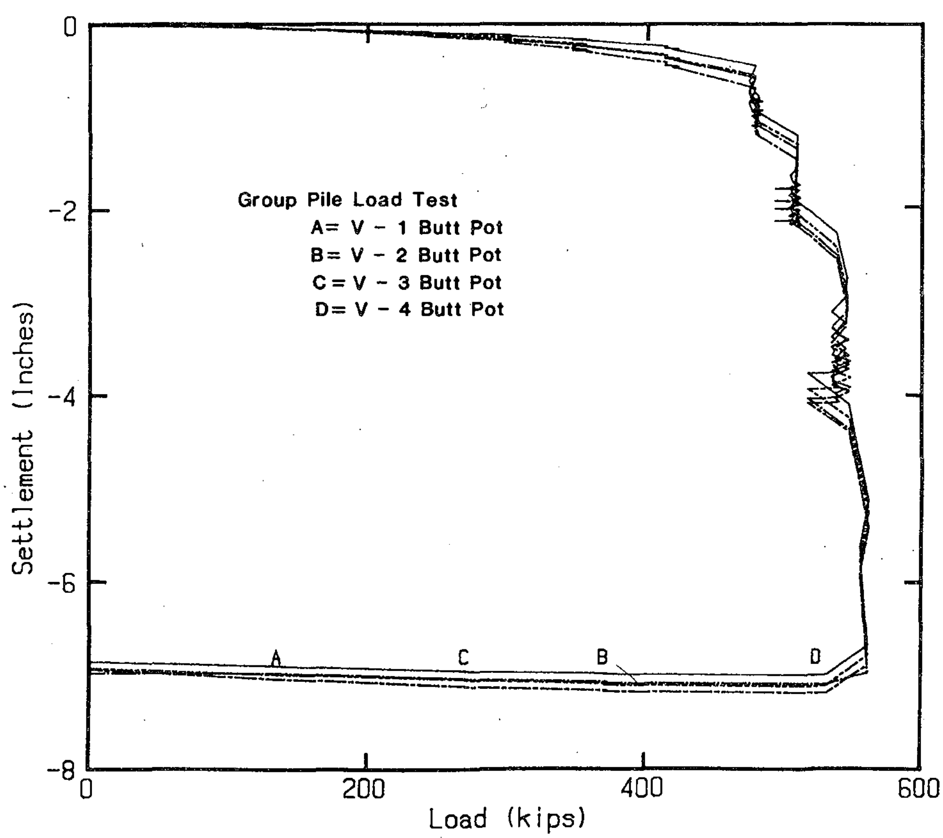

Fig. 26 Butt load - top settlement curves (Group Piles).

ACKNOWLEDGMENT

The authors would like to acknowledge the support and guidance of Mr. Carl Ealy of the Federal Highway Administration during all phases of this study. Special thanks should be given to Captain John Bauman and Mr. Donald C. Brown of Hunter's Point Naval Station for providing assistance and support throughout the use of the test site.

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ANALYSIS OF THE BEHAVIOR OF AN AXIALLY LOADED 5 PILE GROUP AND A CONTROL SINGLE PILE AT HUNTER'S POINT

Introduction

This article follows the article by Ng (1987) which described the instrumentation and the load testing of a five pile group and a control single pile at Hunter's Point in San Francisco (Figure 1). These load tests are the phases 2 and 3 of an FHWA sponsored project, in which Texas A&M University is responsible, among other things, for a set of predictions prior to load testing and the complete analysis of the load test data. The results of phase one have already been published (Briaud et al., 1983, Briaud, Tucker 1984a and b). At the time of this article (December 1986) the analysis of the load test results is partially complete and therefore the results presented here must be considered only as an update in a project which is progressing on schedule.

In a first part the soil properties, the pile characteristics and the test program are summarized. Then, the predictions for the behavior of the single pile and the pile group are presented. Finally, the results of the load tests are analyzed including residual stresses and load transfer curves.

The Soil

The soil has been described in detail by Ng (1987). Below a 4 in. thick asphalt concrete pavement is a 4.5 ft thick layer of sandy gravel with particles up to 4 in. in size. From 5 ft to 40 ft depth is a hydraulic fill made of clean sand (SP). Below 40 ft, layers of medium stiff to stiff silty clay (CH) are interbedded with the sand down to the bedrock. The fractured serpentine bedrock is found between 45 ft and 50 ft depth. The water table is 8 ft deep.

The hydraulic fill has the following average properties: 80% of the particles by weight smaller than 1 mm, 2% smaller than 0.075 mm, dry unit weight 100 pcf and water content 22.6% from Sprague-Henwood samples, friction angle from direct shear tests on Sprague-Henwood samples 35.4°, SPT blow count 15 bpf, CPT point resistance 65 tsf, PMT net limit pressure 7 tsf, shear wave velocity shear modulus 400 tsf. Many tests were performed at the sites including: Standard Penetration Tests with a donut hammer and a safety hammer, sampling with a Sprague-Henwood sampler, cone penetrometer tests with point, friction and pore pressure measurements, preboring and selfboring pressuremeter tests, shear wave velocity tests, dilatometer tests, step blade tests. CPT, SPT and step blade tests were repeated after driving and testing of the piles. All profiles will be reported in the final report. The location of selected soundings available prior to driving and load testing are shown on Figure 2. The corresponding profiles are shown on Figures 3 to 8.

The Piles and the Test Program

A large load testing program has taken place at Hunter's Point including 5 driven single piles, a group of 5 driven piles, and a series of vibrated single piles. Only the group of 5 driven piles and the corresponding control single pile are dealt with in this article (Figure 1). The piles are steel pipe piles with the following properties: 10.75 in. OD, 0.365 in. wall thickness, modulus E (assumed) 30 x 10^6 psi, cross section area A including instrumentation channels 15.41 in.^{2}. The AE value was measured to be 355614 kips during the calibration of the single pile. This AE value was used for all the piles.

The instrumentation on the piles consisted of strain gages, top and toe load cells and tell tales. The location of the instrumentation is shown in Figure 9. For the single pile the load was applied and measured 5 ft above the ground surface; the settlement was measured 3 ft above the ground surface. For the group the load was applied and measured above the cap and the settlement was measured at each corner of the cap. The instrumentation of the soil consisted of extensometers, and piezometers. A 12 in. diameter hole was predrilled for the first 4.5 ft, then the piles were driven close-ended to 30 ft below the ground surface with a Delmag D22 diesel hammer.

First, 28 reaction H piles were driven on April 15, 1986. This led to a settlement of the ground surface averaging 4 in. Two weeks later, on April 30, 1986, the single pile and the group of piles were driven and the ground surface settled an additional 1.1 in.; the order of driving is shown on Figure 1. The average blow count close to final penetration and some of the results of the pile driving analyzer are shown in Table 1. The piles were not redriven before testing.

The single pile was tested 24 days after driving on May 23, 1986.

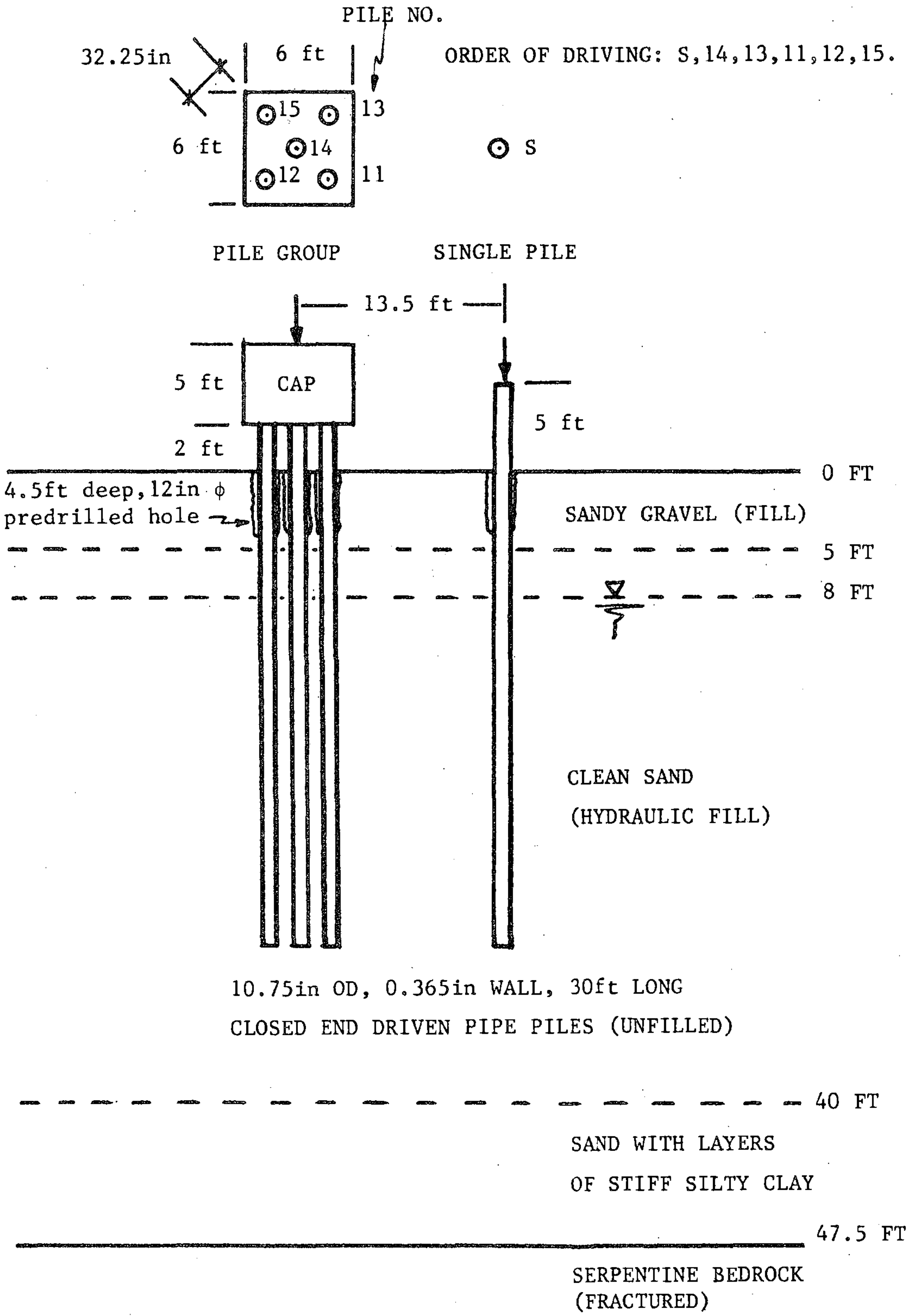
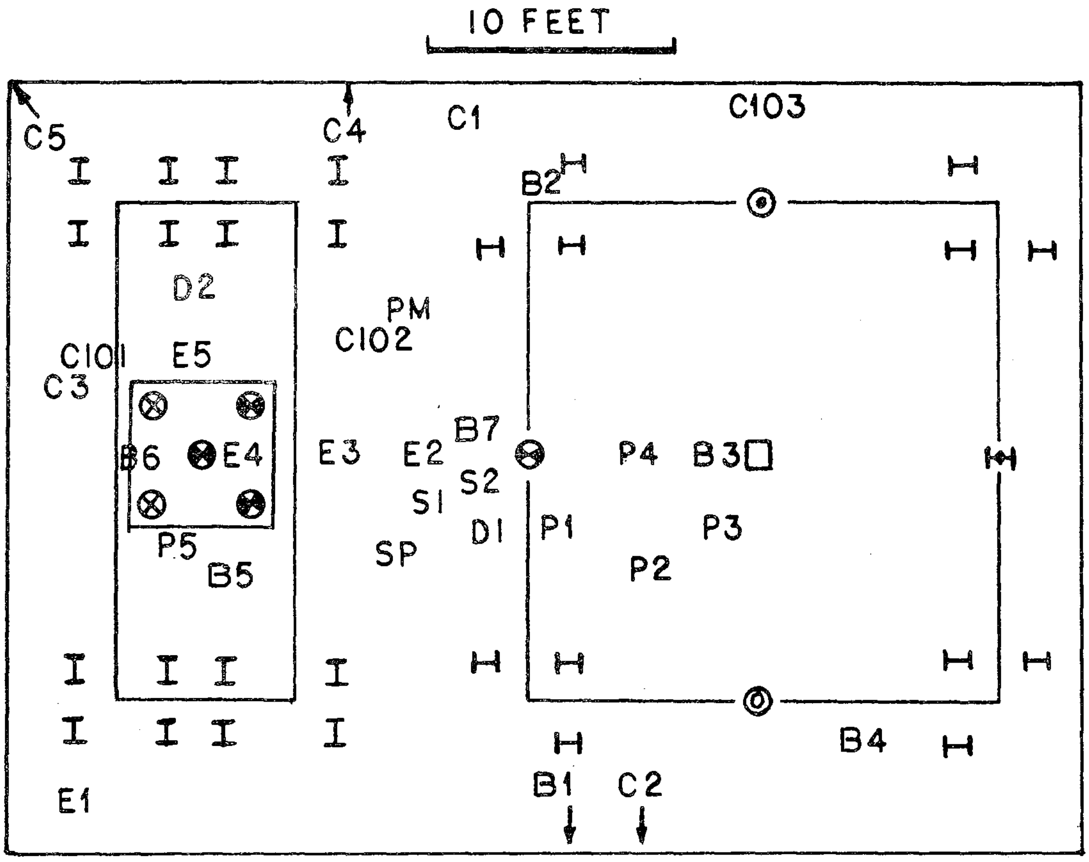


FIG.1- General Test Conditions.



S STEP BLADE TESTS
B SPT TEST BORINGS
C CONE PENETROMETER TESTS
D DILATOMETER TESTS
E EXTENSOMETER
P PIEZOMETER
PM PREBORING PRESSUREMETER TESTS
SP SELFBORING PRESSUREMETER TESTS
REACTION H PILES
TEST H PILE 14" 14H73 STEEL
10.75" DIAM PIPE PILE, FULLY INSTRUMENTED, CLOSED END.
10.75" DIAM PIPE PILE, PARTIALLY INSTRUMENTED, CLOSED END.
10.75" DIAM PIPE PILE, OPEN END.
12" SQUARE PRESTRESSED, PRECAST CONCRETE PILE.
12" TOP, 8" TIP, 0.14"/FT TAPER MONOTUBE STEEL PILE.

FIG.2- Location of Borings, Instrumentation, and Piles.

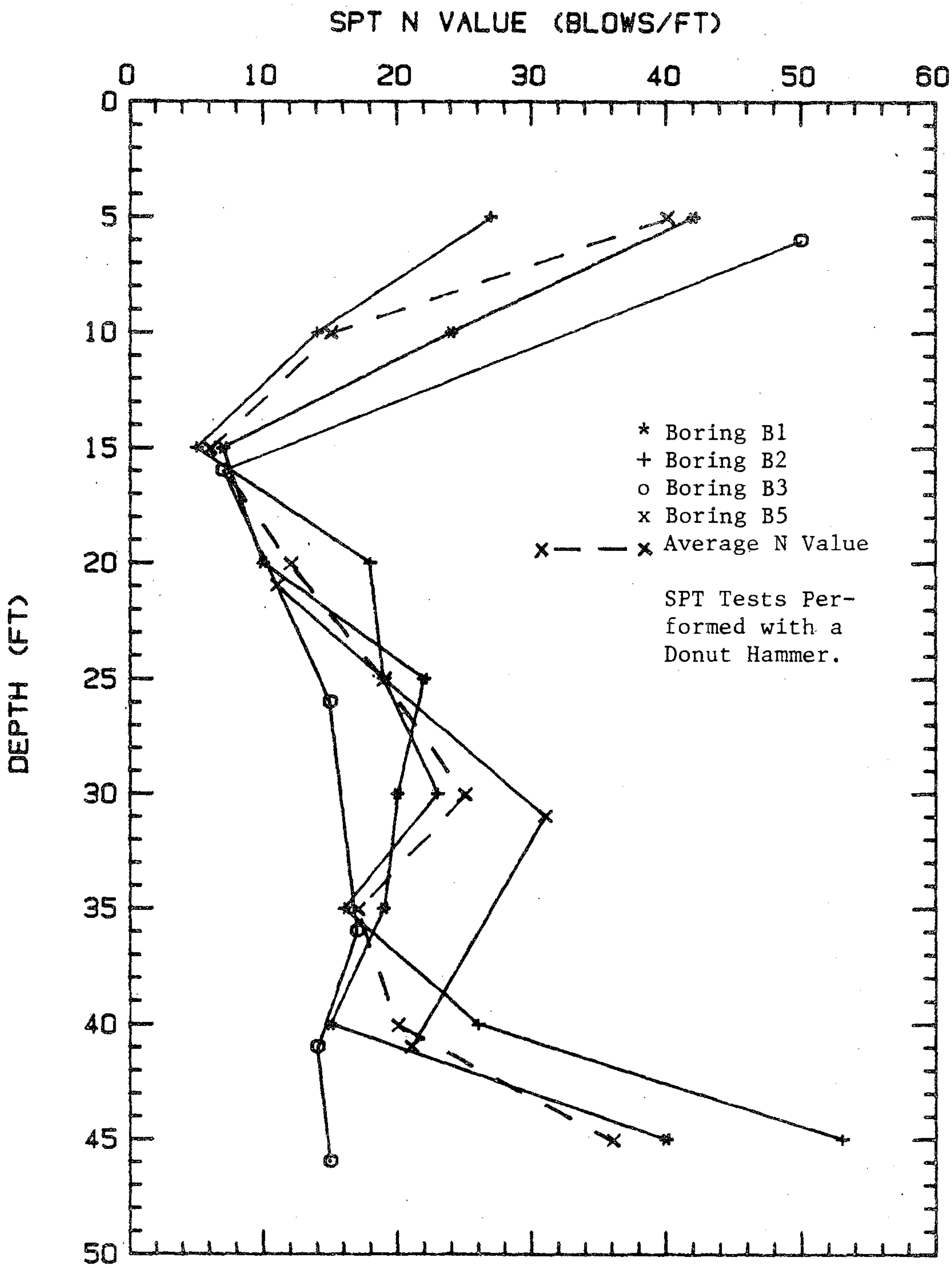


FIG. 3 - Profiles of Standard Penetration Test Blowcounts

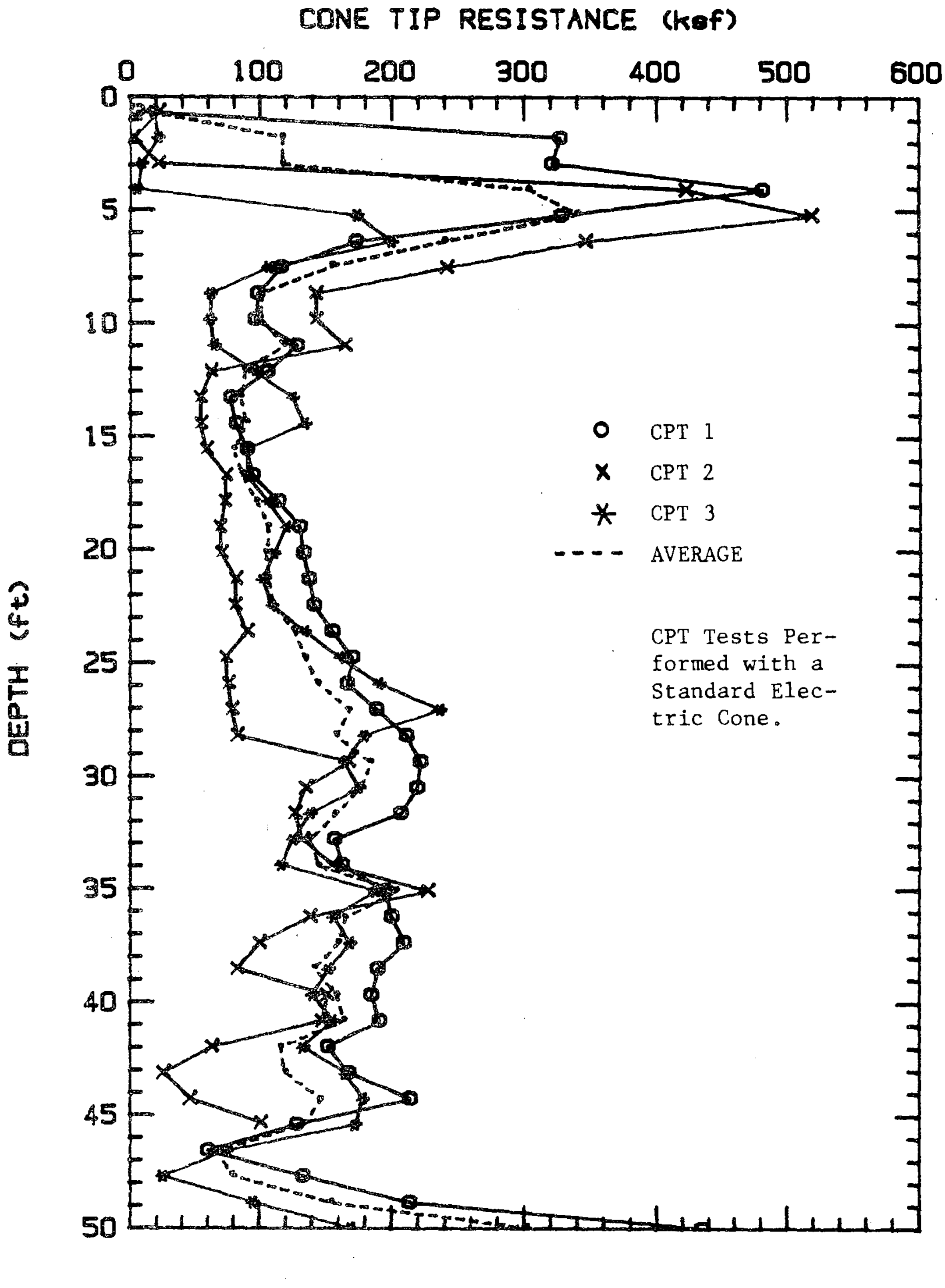


FIG.4- CPT Profiles For Point Resistance.

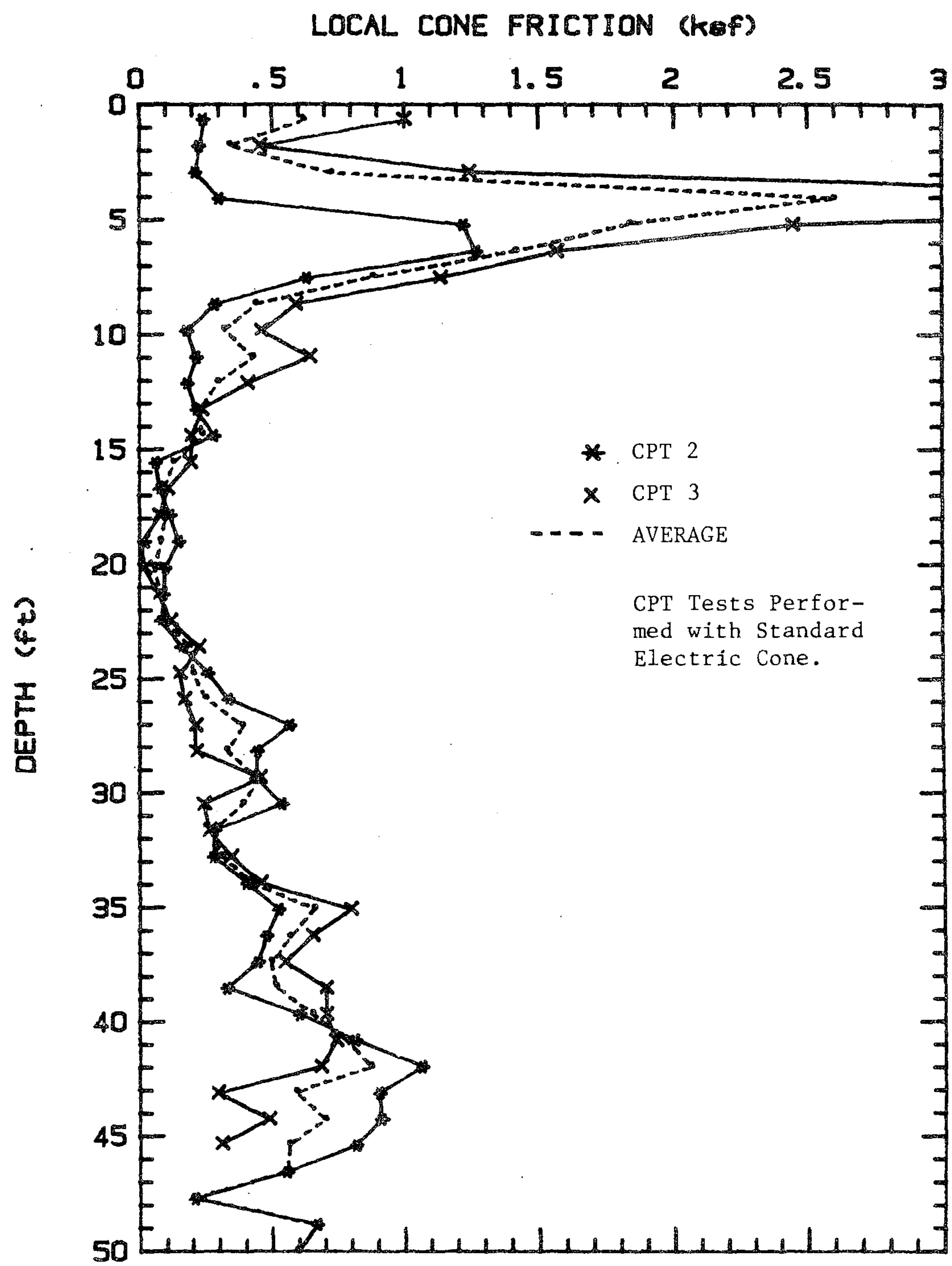


FIG.5- CPT Profiles for Friction Resistance.

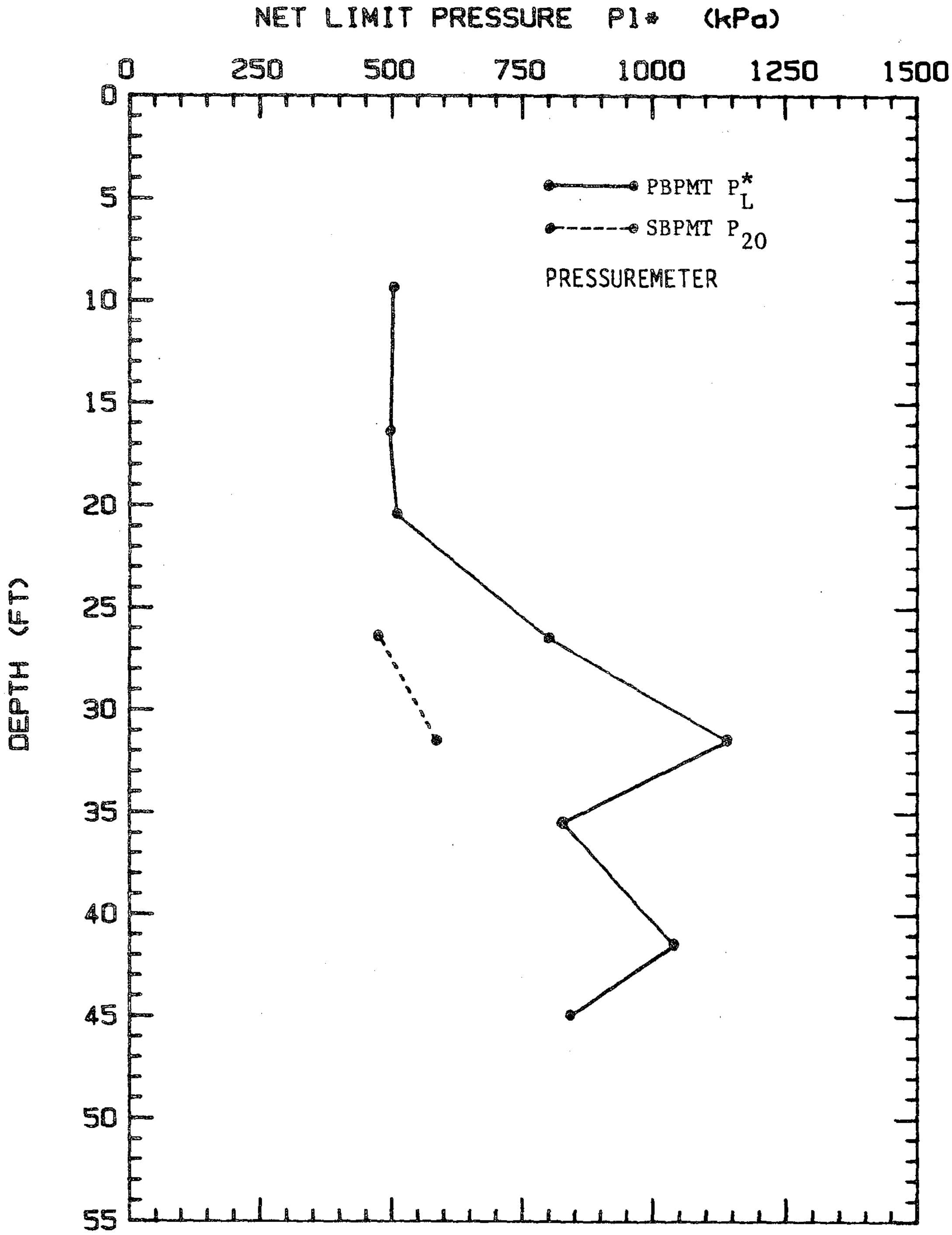


FIG.6 - Net Limit Pressure Profile

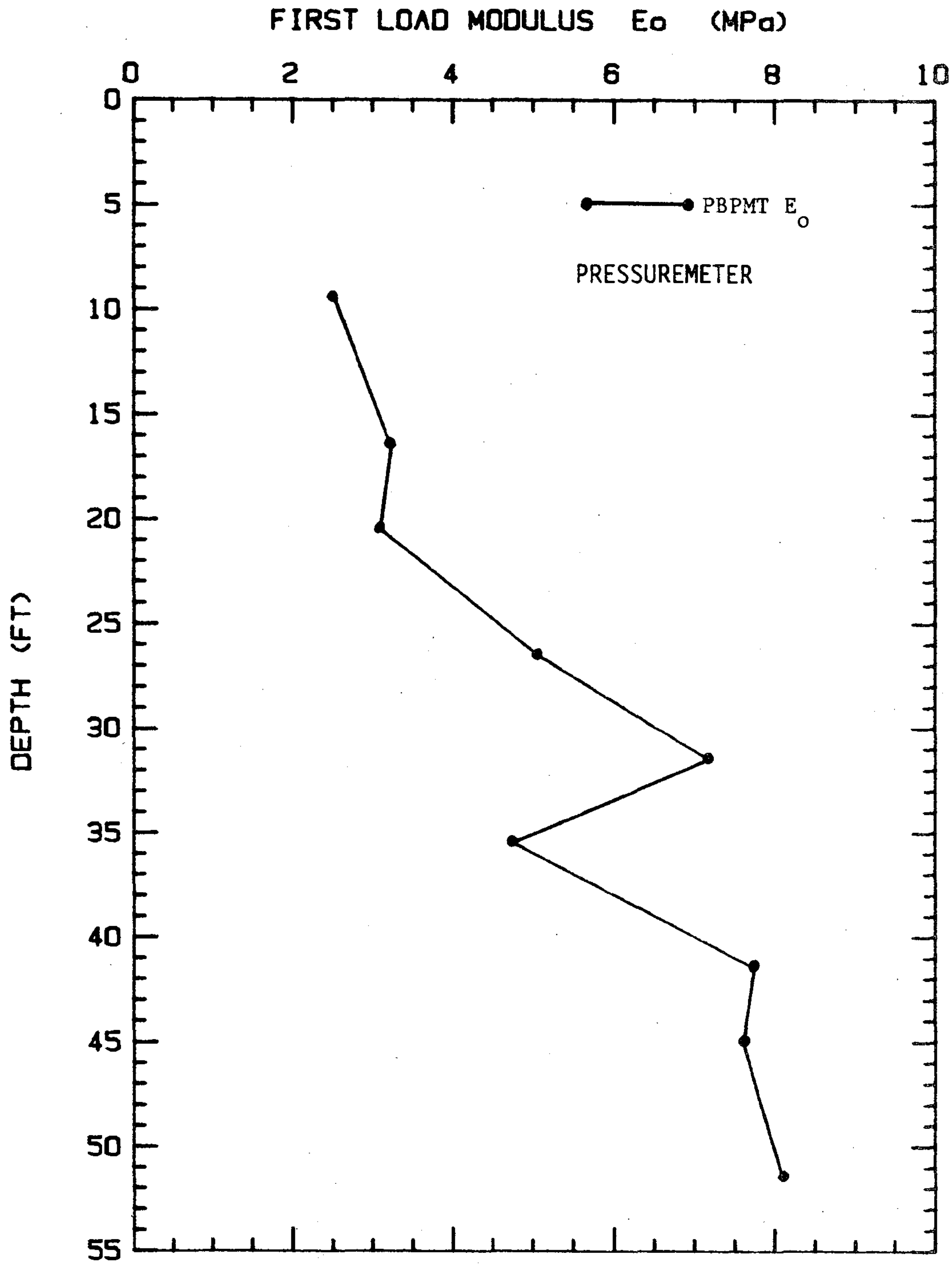


FIG.7 - First Load Modulus Profile

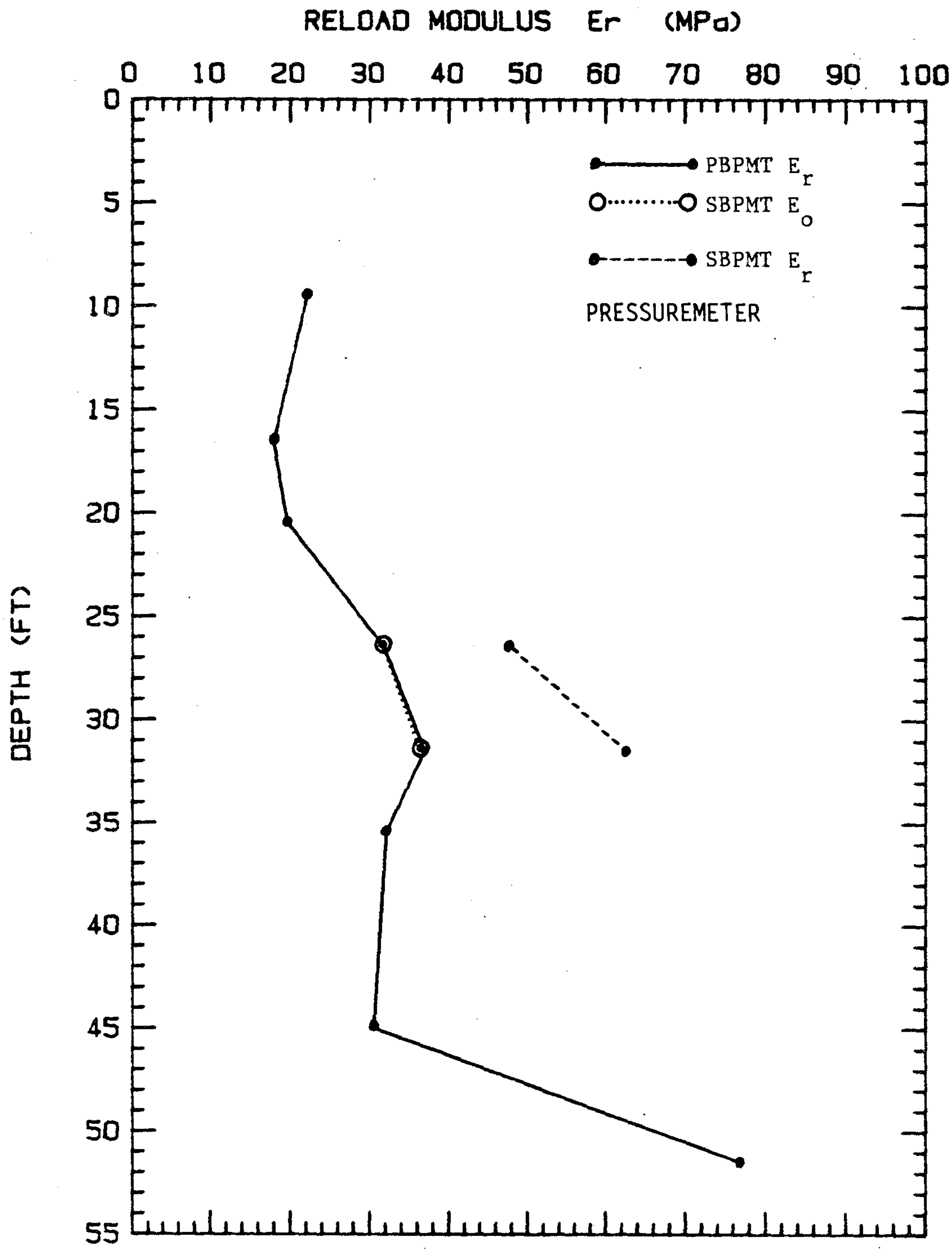


FIG.8 - Reload Modulus Profile

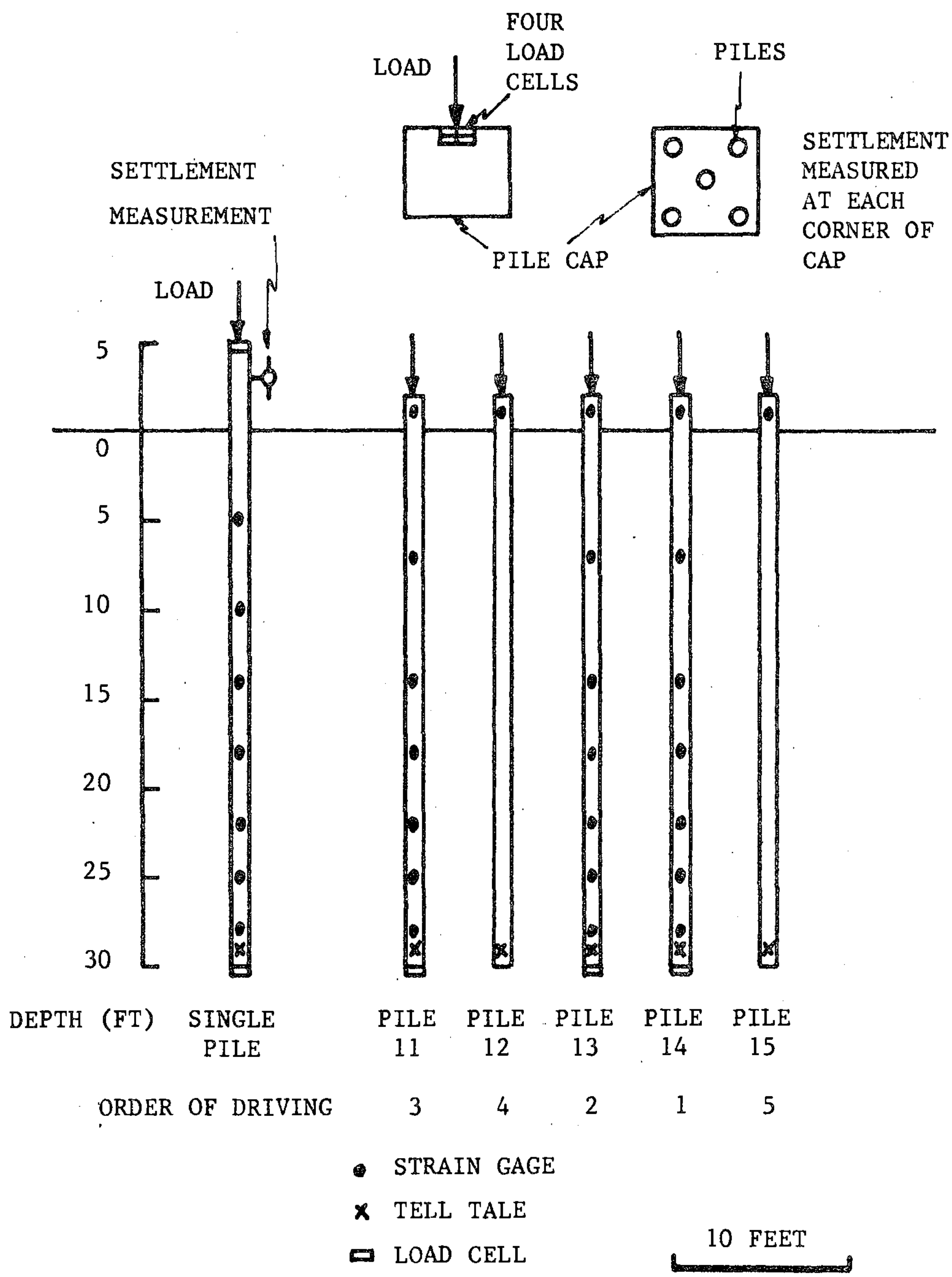


FIG.9- Details of the Instrumentation.

The loading sequence consisted of increasing the load in 10 kip increments, holding each load for 30 minutes and recording the displacement as well as all the instrumentation every 5 minutes. The pile group was tested on June 6, 1986, 14 days after testing the single pile. The loading sequence consisted of increasing the load in 60 kip increments holding each load for 30 minutes and recording the displacement as well as all the instrumentation every 5 minutes.

Prediction of Single Pile Behavior

The predictions consisted of three parts: prediction of residual loads, prediction of ultimate loads, prediction of load settlement curves. The residual loads which exist in the pile after driving and before load testing can be estimated by using the wave equation analysis (Holloway et al., 1975) or by using a semi-theoretical approach (Briaud, Tucker, 1984b). At this time in the project the wave equation analysis for residual loads has not yet been performed. The semi-theoretical approach gave a residual point load of 8.1 kips plus the weight of the pile of 1.6 kips. As will be seen later, the measured residual point load after driving was 13.8 kips including the pile weight.

A total of 10 different methods were used to predict the ultimate load of the single pile. They are listed in Table 2 together with the type of data necessary to use them. The Pile Driving Analyzer (PDA) data are in Table 1. The SPT profiles shown on Figure 3 were simply averaged in order to obtain the design profile for the methods requiring SPT data. For the CPT data, soundings 4 and 5 were considered to be too far from the piles to be applicable. Therefore, the tip resistance used for the predictions was an average of soundings 1, 2 and 3. The friction readings for sounding 1 were considered to be unreliable since the cone penetrometer had suffered damages while penetrating the top 4 ft. The sleeve friction used for predictions was an average of sounding 2 and 3. For the pressuremeter data no test was performed in the fill material near the surface; a limit pressure, p_L, and a reload modulus, E_r, were inferred from the cone point resistance, q_c, at a depth of 6 ft using the correlations between p_L and q_c, and E_R and q_c obtained from the other values in the borings. The total unit weight of the soil was assumed to be 120 pcf. The authors did not have the measured averaged total unit weight of 122.6 pcf at the time of the predictions. Using these design profiles the 10 methods of Table 2 were used and led to the results presented in Table 2. The actual ultimate load will be discussed later.

Four of the above 10 methods also included recommendations for load transfer curves. These curves allow to predict the entire load settlement curve for the pile. These predictions are presented on Figure 10. Note that for the load settlement calculations and for the ultimate load calculations it was assumed that no friction existed over the first 4.5 ft below the ground surface since a 12 in. diameter hole was drilled to that depth prior to driving. Also the load and settlement of the pile were calculated at the points where they were measured, that is to say, at 5 ft and 3 ft above the ground surface respectively. The actual load settlement curve is shown on Figure 10.

Prediction of Pile Group Behavior

The predictions included predictions of residual stresses, ultimate loads and load settlement curves. The residual point load for each pile in the group was predicted to be 8.1 kips plus the pile weight of 1.6 kips, the same as for the single pile since at this time the method does not distinguish between a single pile and a pile in a group. The average residual point load for all the piles in the group was 2.6 kips.

The ultimate load of a pile group in sand is calculated as:

$$Q_g = n\, Q_s\, e \qquad (1)$$

where Q_g is the ultimate load of the group, n the number of piles in the group, Q_s is the ultimate load of the single pile and e is the efficiency factor. O'Neill (1983) shows data from 10 full scale group tests in sand where e varied from 1 to 2 with a trend towards the higher e values for loose sands. Considering that the average SPT blow count was about 15 bpf, an efficiency value of 1.5 was selected. The predicted ultimate load for the group is shown on Table 3 for the various methods. The settlement ratio of a group of piles is defined as:

TABLE 1. Pile Driving Analyzer Results*

PILE NO.	ORDER OF DRIVING	TOTAL NUMBER OF BLOWS	BLOW COUNT, FINAL FT.	E_{MAX} KIPS	F_{MAX} KIPS	R_{MAX}, EOI** 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 InSituTech

TABLE 2. Methods Used for Ultimate Load Predictions and Results for Single Closed End Pipe Pile

	METHOD	TYPE OF DATA REQUIRED	POINT (kips)	SIDE (kips)	TOTAL (kips)
P	Nordlund (1963)	SPT	70.6	22.1	92.7
R	Coyle/Castello (1981)	SPT	70.5	48.4	118.9
E	Briaud/Tucker (1984b)	SPT	76.8	73.5	150.3
D	Meyerhof (1976)	SPT	128.7	49.8	178.5
I	API (1984)	SPT	10.7	20.2	30.9
C	Bustamante/Gianeselli (1983)	CPT	43.9	27.7	71.6
T	DeRuiter/Beringen (1979)	CPT	89.3	22.2	111.4
E	Schmertmann (1978)	CPT	89.3	20.3	109.6
D	Bustamante/Gianeselli (1982)	PMT	27.2	41.1	68.3
	Pile Driving Analyzer	PDA			80-120
M					
E	Measured with Strain Gages		75	42	117
A	Measured with Load Cells		86	24	110
S	Measured Average		80.5	33	113.5
U	D/10 + PL/AE (load cells)		71	27	98
R	Davisson Criterion (1972)(load cells)		50	30	80
E	Creep Load		56	32	88
D					

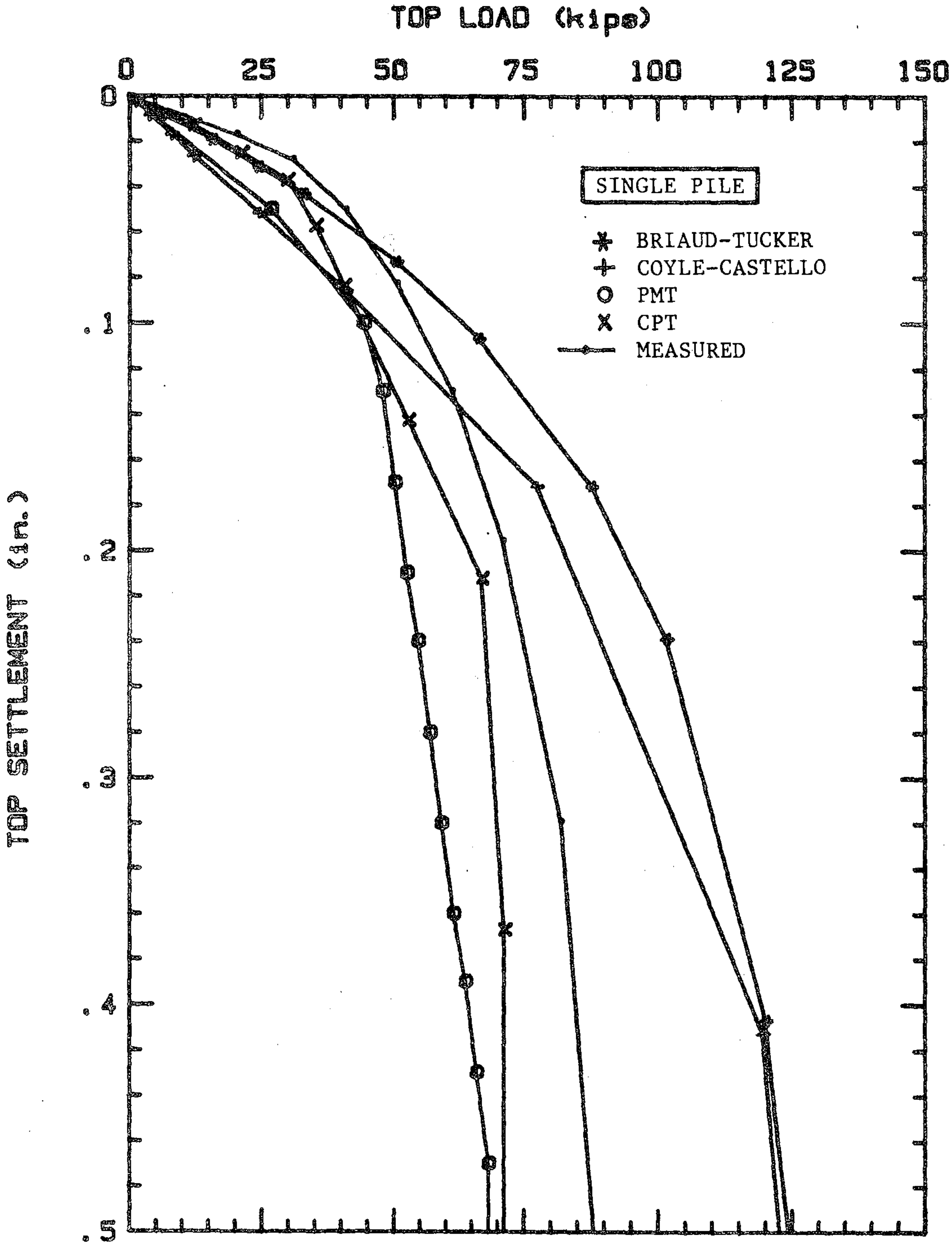


FIG.10- Predicted and Measured Load-Settlement
Curve for Single Pile.

TABLE 3. Methods Used for Ultimate Load Predictions and Results for the Group of 5 Closed End Pipe Piles

	METHOD	TYPE OF DATA REQUIRED	POINT (kips)	SIDE (kips)	TOTAL (kips)
PREDICTED	Nordlund (1963)*	SPT	529	166	695
	Coyle/Castello (1981)*	SPT	529	363	892
	Briaud/Tucker (1984)*	SPT	576	551	1127
	Meyerhof (1976)*	SPT	965	373	1338
	API (1984)*	SPT	80	152	232
	Bustamante/Gianeselli (1983)*	CPT	329	208	537
	DeRuiter/Beringen (1979)*	CPT	670	166	836
	Schmertmann (1978)*	CPT	670	152	822
	Bustamante/Gianeselli (1982)*	PMT	204	308	512
	Pile Driving Analyzer**	PDA			
	Pile No. 14	-			100-105
	Pile No. 13	-			140-165
	Pile No. 11	-			150-160
	Pile No. 12	-			90-100
	Pile No. 15	-			135-140
	Total PDA (Group)	-			615-670
MEASURED	Measured Plunging Load on Pile 14		57.5	59.5	117
	Measured Plunging Load on Pile 11		56.5	50.5	107
	Measured Plunging Load on Pile 13		49	71	120
	Measured Plunging Load on Pile 15				109
	Measured Plunging Load on Pile 12				107
	5 x Average of Piles 14, 11 and 13 (instrumented)		272	301	573
	Measured Plunging Load of Group (Strain Gages)		267	294	561
	Measured Plunging Load of Group (Load Cells on Cap)				561
	Creep Load for Group		132	300	432

* Assuming an efficiency of 1.5

** Furnished by Dr. D.M. Holloway of InSituTech

$$r_s = \frac{s_g}{s_1} \quad \text{. . . . .} \quad (2)$$

where s_1 is the settlement of the single pile for a load Q_1 and s_g is the settlement of the pile group for a load $Q_G = n\ Q_1$ where n is the number of piles in the group. Vesic (1977) proposed a simple formula to calculate the settlement ratio of a square group:

$$r_s = \left(\frac{B}{d}\right)^{0.5} \quad \text{. . . . .} \quad (3)$$

where B is the width of the group and d the pile diameter. Using this formula leads to a settlement ratio of 2.06 for this group.

The load-settlement curve for the pile group can be predicted by using the program PILGP2, a modified version of a program developed by O'Neill (1981) for the Federal Highway Administration. This program is based on a load transfer curve approach to calculate the settlement of a single pile coupled with an elastic interaction procedure to take into account the added settlement of that single pile due to the load on the other piles. Therefore PILGP2 leads to a ratio of the group settlement over the single pile settlement (settlement ratio) larger than 1 but to an efficiency of 1. Two major inputs are required for PILGP2: the load transfer curve for the single pile analysis, and the soil modulus for the elastic interaction. For the predictions, the load transfer curves from 3 single pile prediction methods were used: SPT (Briaud, Tucker, 1984b), CPT (Schmertmann, 1978; Verbrugge, 1981), PMT (Bustamante and Gianeselli, 1982, Briaud, 1986). The soil modulus for the elastic interaction was chosen to be:

$$E = 15\ N\ (\text{tsf}) \quad \text{for the SPT method . . . .} \quad (4)$$

$$E = 5\ q_c \quad \text{for the CPT method . . . .} \quad (5)$$

$$E = E_R \quad \text{for the PMT method . . . .} \quad (6)$$

where N is the number of blows per foot in the SPT test, q_c the CPT point resistance and E_R the PMT reload modulus. The results of the predictions are shown in Figure 11.

Analysis of the Single Pile Results

The analysis includes residual stresses, load settlement curve, load versus depth curves, load transfer curves, and a comparison with the predictions. The strain gages along the pile were zeroed while the pile was laying on the ground on April 29, 1986. The pile was driven on April 30, 1986 and the strain gages were read one day later on May 1, 1986, 7 days later on May 7, 1986 and 23 days later on May 23, 1986. The strain gage readings ε were transformed into loads Q by using:

$$Q = k\varepsilon \qquad (7)$$

where k was obtained from the calibrations. The profiles obtained at those 3 dates were somewhat different. Possible reasons for the evolution of the residual load profiles versus time include strain gage instability and release of residual stresses. The average of the three profiles was chosen as the residual load profile to be used for further analysis of the load test (Figure 12). All further analysis will take residual stresses into proper account unless otherwise specified. The profile of residual load satisfies approximately the boundary conditions since the load is near zero at the top and matches closely the toe load cell reading at the toe. Indeed, a load cell was placed at the toe of the pile and was zeroed before driving like the strain gages; after driving, this toe load cell was read and fluctuated a lot less than the strain gages. The toe load cell indicated a residual point load of 13.8 kips which includes the weight of the pile (1.6 kip). The average residual load profile shows that the friction acts downward down to the neutral point below which the friction acts upward. This conforms to the expected shape since the downward friction due to the pile trying to move up with respect to the soil corresponds to the downward force which keeps the point prestressed against the soil. Close to the point, the net pile soil movement even after rebound is downward since the point load is positive; therefore the friction close to the point is acting upward. This phenomenon is almost identical to the phenomenon of negative skin friction. At the end of the load test, the load was released and additional residual stresses were locked in. The average residual load profile after the load test is shown on Figure 12. Note that the residual point load has increased significantly.

The load settlement curve is shown in Figure 13. The data points correspond to the average readings at the end of each 30 minute load step of the top load cell and the top strain gage. The time to reach the end of the final load step was 6 hours. The failure load can be defined in many ways from the load test curve (Fellenius, 1975). The average load during the last load step was 110 kips as given by the top load cell and 117 kips as given by the top strain gage for an average of 113.5 kips; at the end of that load step the settlement was 3.33 in. As can be seen from the curve, little additional load could have been carried by the pile had the penetration continued passed 3.33 in. In that sense 113.5 kips can be considered as the true ultimate load for that pile. A common definition of the failure load is the load which corresponds to a top movement equal to 10% of the pile diameter plus the compression of the pile under that load as if it was a free column: this load is 98 kips. Another commonly used failure load (Davisson, 1972) is the load reached at a top movement of 0.15 in. plus 1/120th of the pile diameter plus the compression of the pile under that load as if it was a free column: this load is 80 kips. In addition to these failure loads, the creep load can be defined (as is routinely done in France, Bustamante, Gianeselli, 1981) as the load beyond which the settlement during a load step starts to increase more rapidly (Figure 14a and 14b). Alternatively it is proposed to define the creep load Q_c (Figure 14c and 14d) as the load for which the value of the creep exponent n becomes larger than 0.1, (Briaud, Garland, 1985):

$$\frac{s_t}{s_{t_o}} = \left(\frac{t}{t_o}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

where s_{t_o} is the first settlement reading taken during a load step, t_o is the time elapsed between the application of that load and the reading of s_{t_o}, s_t is the last settlement reading taken

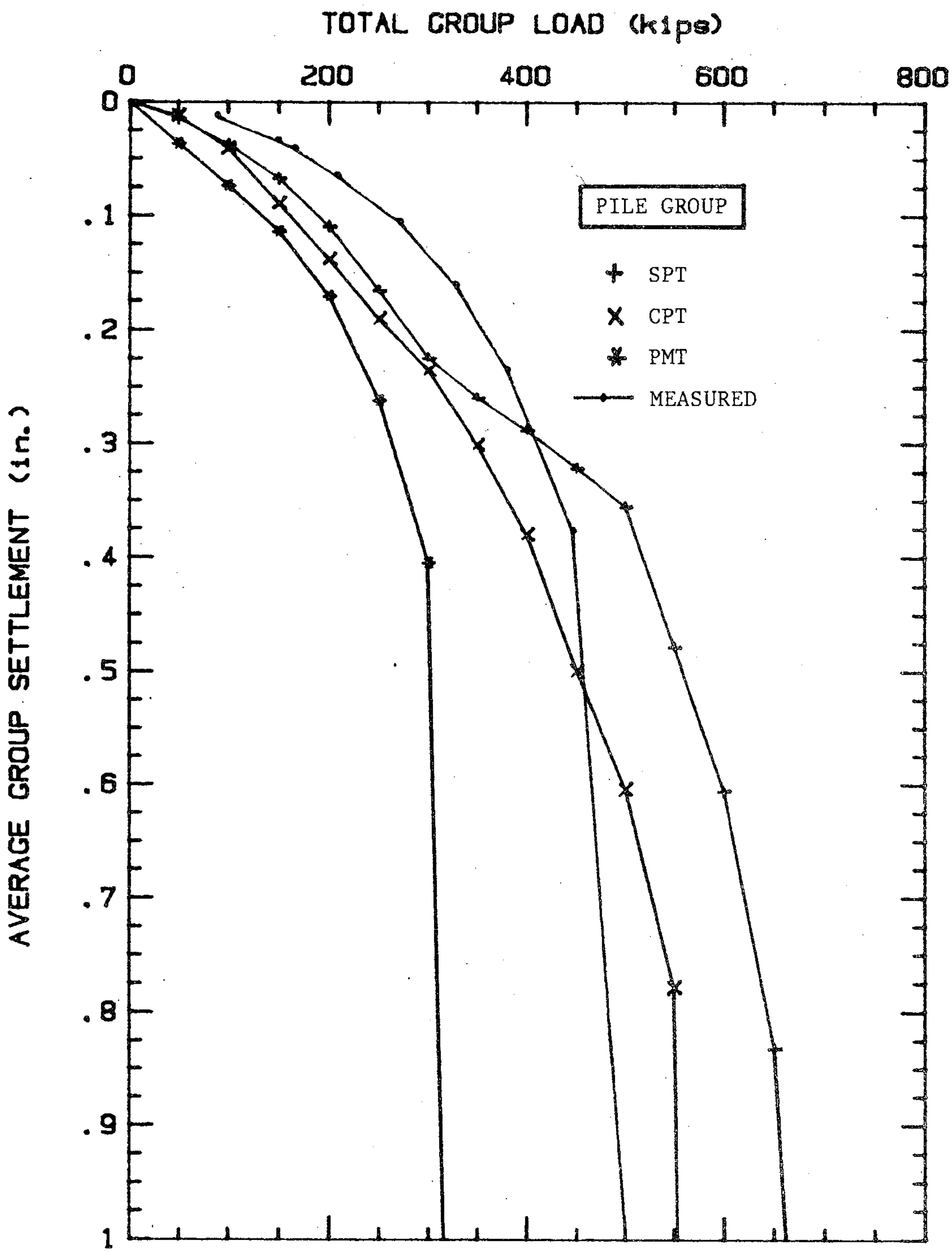


FIG.11- Predicted and Measured Load-Settlement Curve for Pile Group.

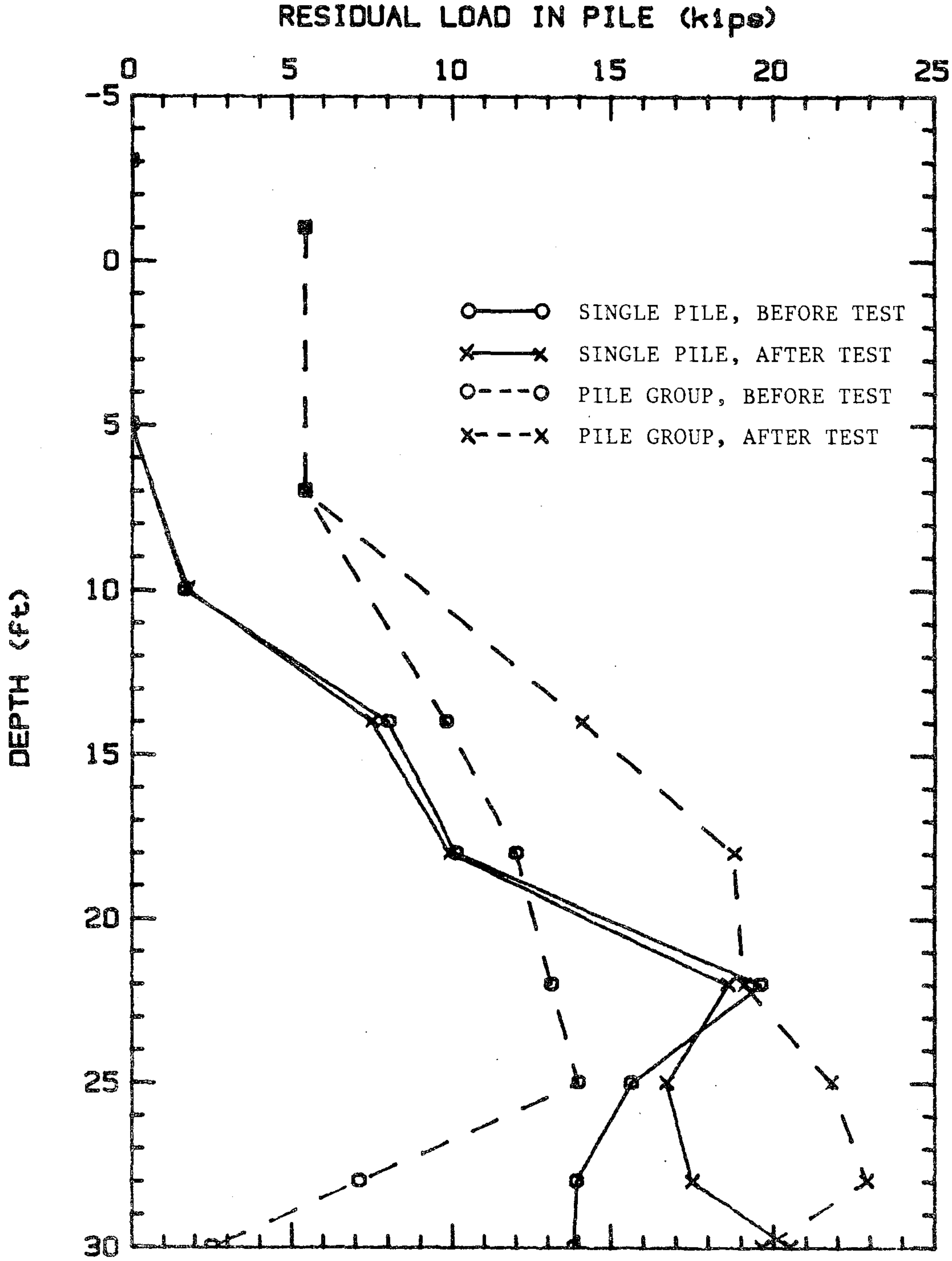


FIG.12- Residual Load Profiles

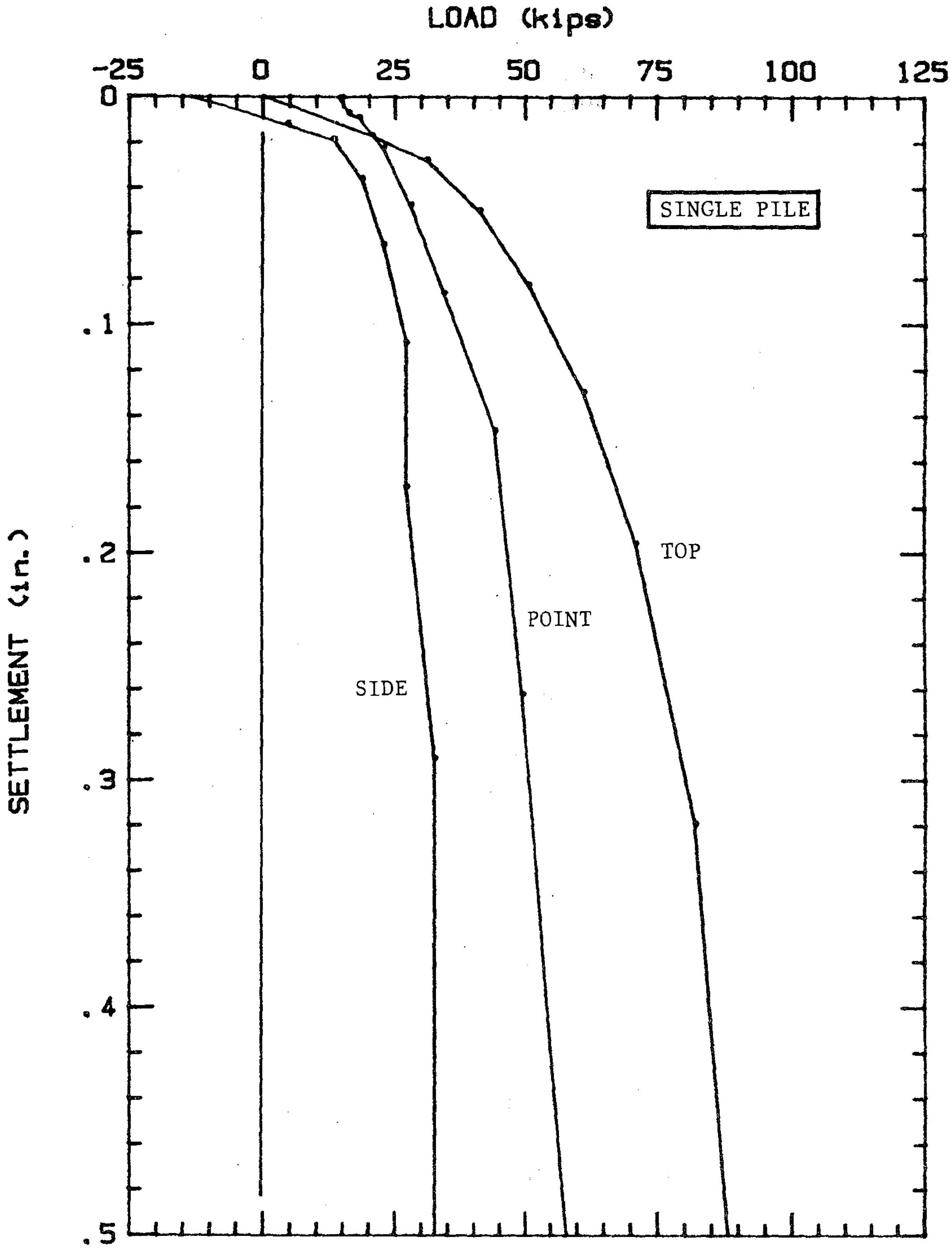


FIG.13a- Measured Load-Settlement Curve
for the Single Pile.

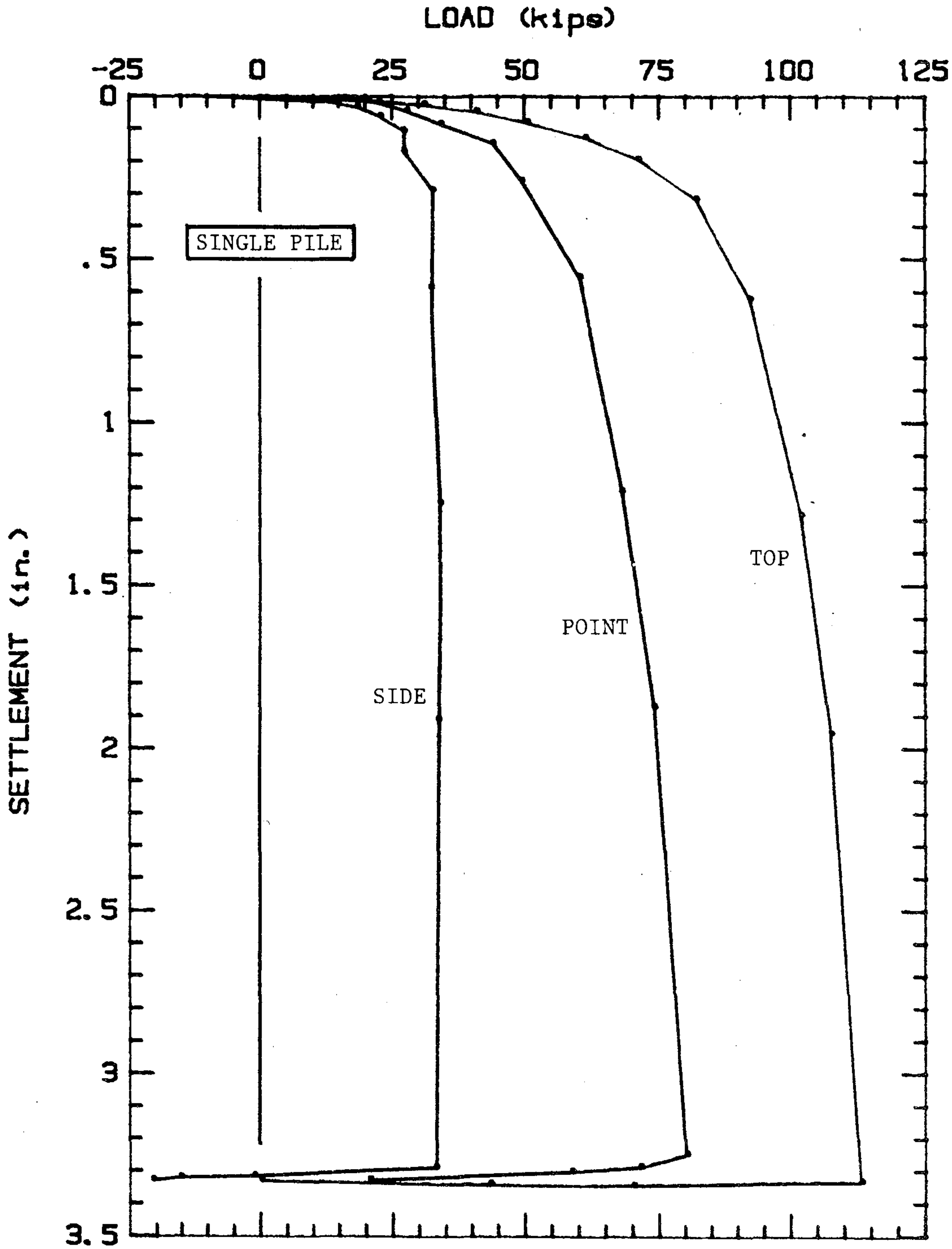


FIG.13b- Measured Load-Settlement Curve
for the Single Pile.

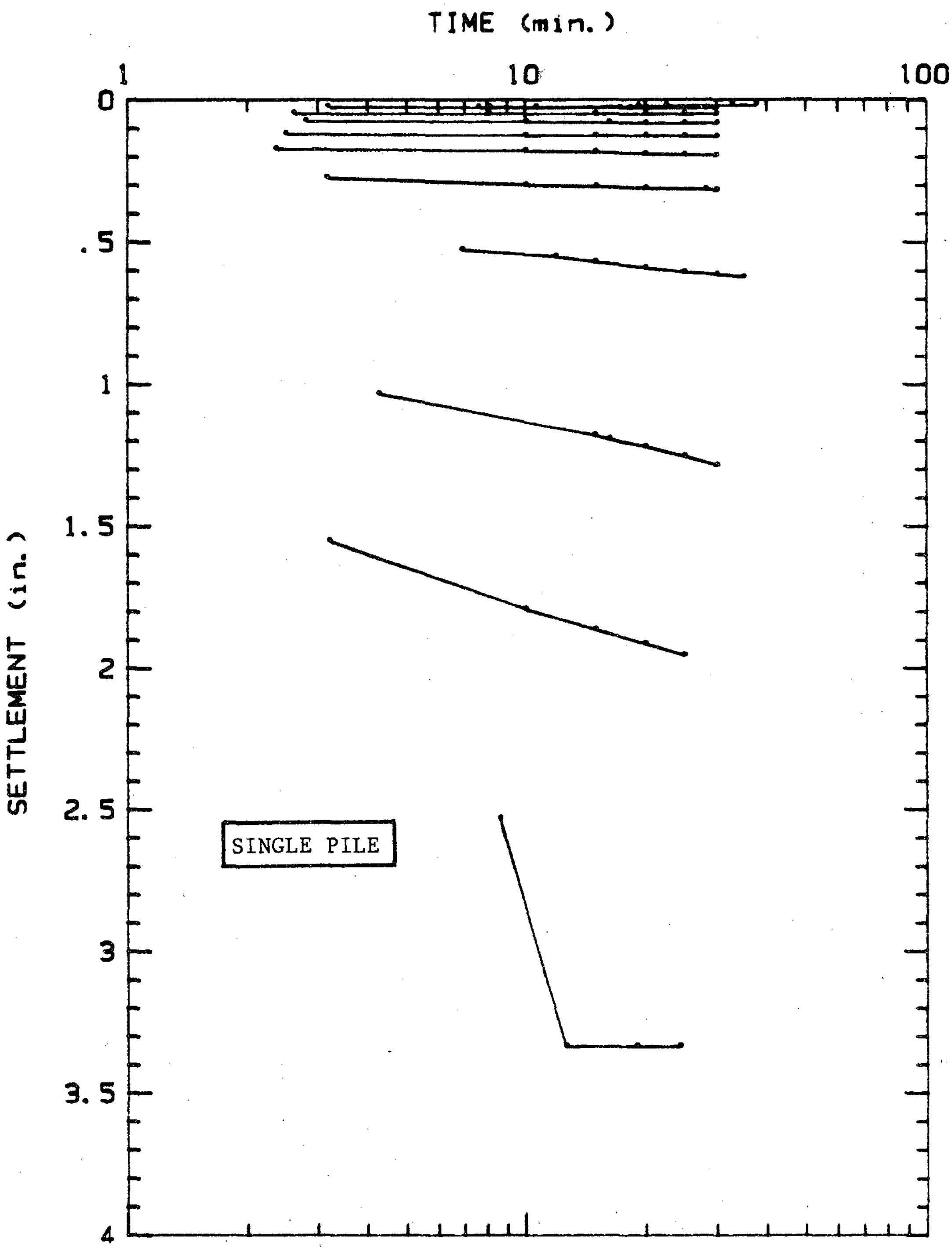


FIG.14a- LogTime vs. Settlement for Load
Steps of Single Pile Load Test.

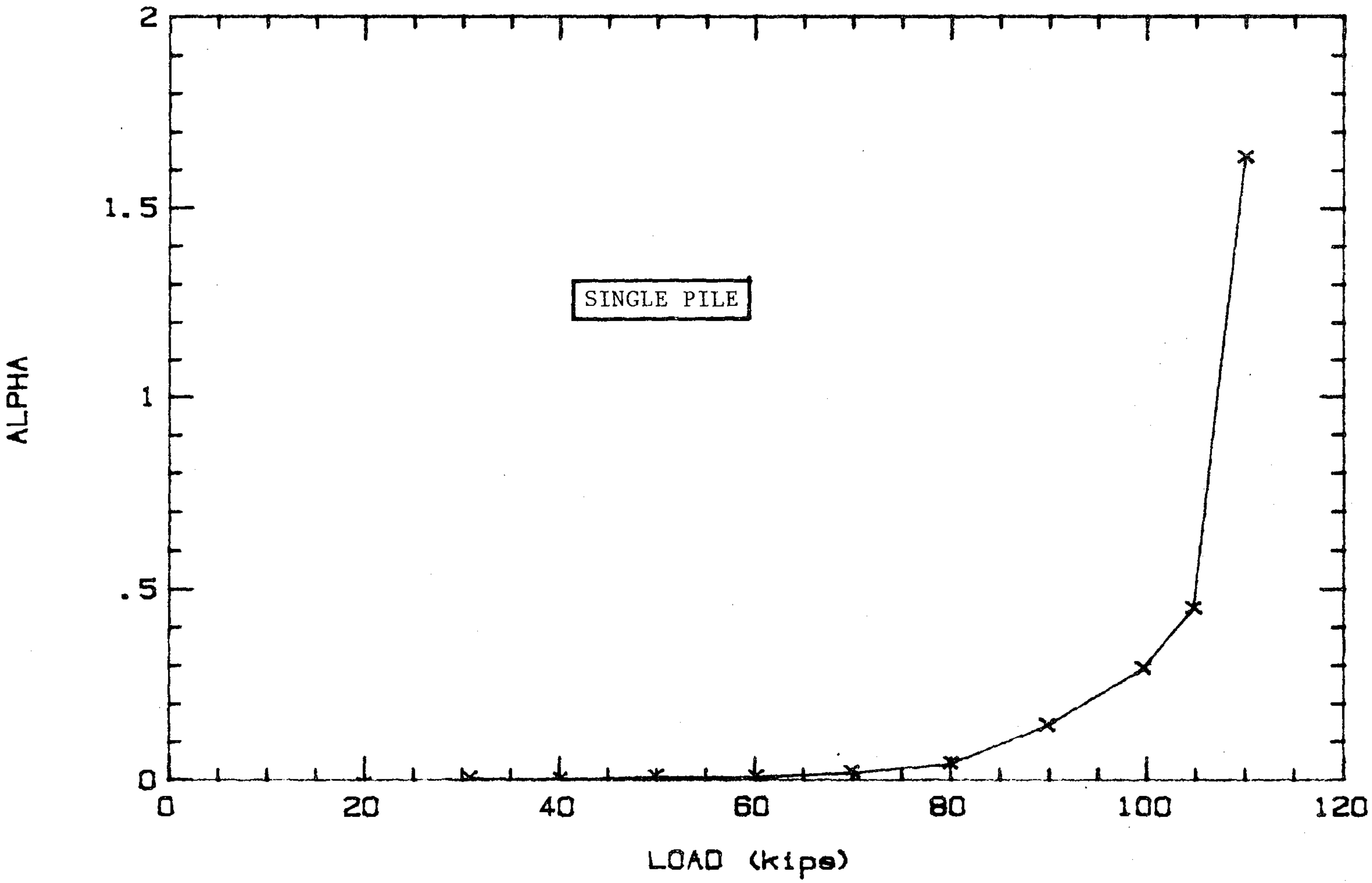


FIG.14b- Slope of LogTime vs. Settlement of Load Steps for Single Pile Load Test.

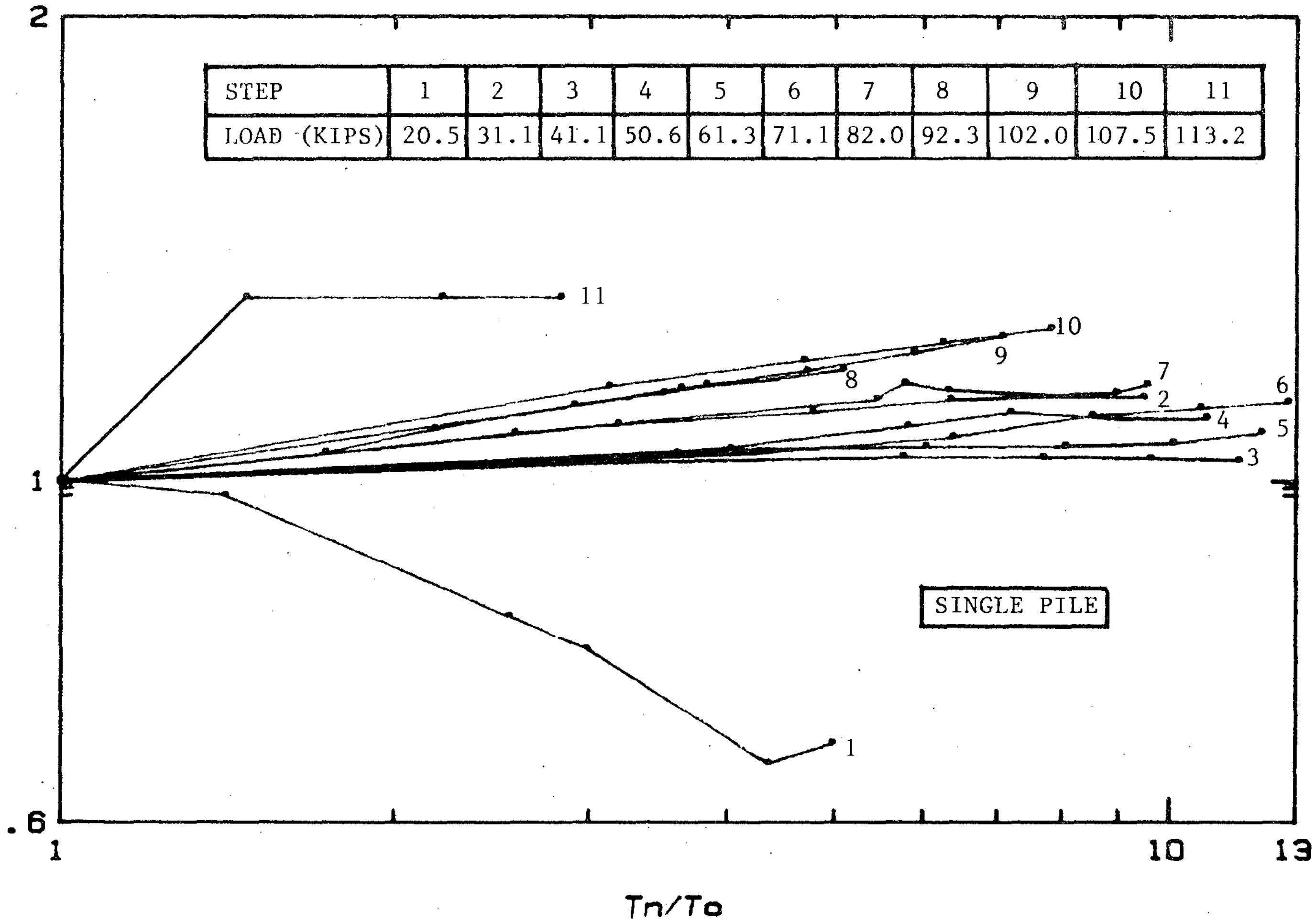


FIG.14c- LogTime vs. LogSettlement for Load Steps of Single Pile Load Test.

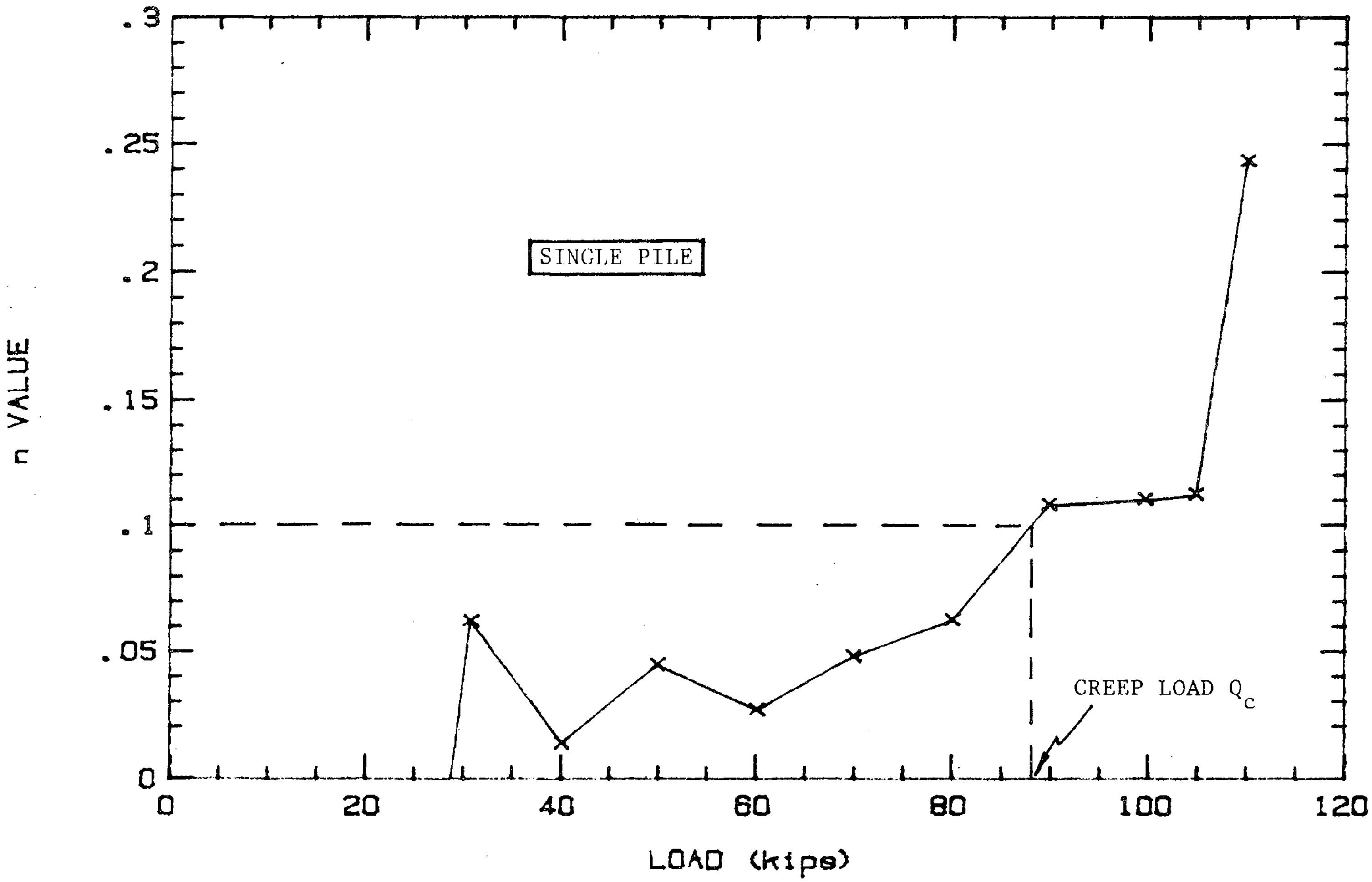


FIG.14d- Creep Exponent n for Load Steps of Single Pile Load Test.

during that load step and t is the time elapsed between the application of that load and the reading of s_t. The plot of n versus load shows the creep load Q_c (Figure 14d) to be 88 kips. Table 2 summarizes the ultimate, failure and creep loads.

The load versus depth profiles are shown on Figure 15. The load at a depth = -3 ft on the figure was obtained from the load cell. The next 7 load levels were obtained from the strain gage readings by using equation 7. The top load in the load settlement curves (Figure 13) was obtained by averaging the load cell and the first strain gage values. The load at depth = 30 ft was obtained from one of the two toe load cells (the other toe load cell was judged unreliable by Mr. Ng of the instrumentation team). The toe load in the point load settlement curves (Figure 13) was obtained by averaging the load given by the toe load cell and the load obtained by extension of the last 2 strain gages. The first load profile (top load = zero) is the residual load profile of Figure 12. The slope of these load profiles is the friction load per unit length of pile. At the maximum load the friction is negative (load increasing with depth) at three locations; this is due to measurement inaccuracies and not to real behavior. Overall, however, the shape of the profiles conforms to the expected shapes. For the maximum load and according to the top and bottom load cells, the top load is 110 kips, the point load is 86 kips and the friction load is 24 kips. For the same maximum load but according to the top strain gage and to an extrapolation of the bottom strain gage reading, the top load is 117 kips, the point load is 75 kips and the friction load is 42 kips. Mr. Ng of the instrumentation team stated that he had more confidence in the load cell than in the strain gages. After considering all the above comments, the interpreted load vs. depth profiles of Figure 16 were prepared and were used for the rest of the analysis. Note that if the residual loads are ignored, the load versus depth profiles are in error by a substantial amount.

The friction load transfer curves were obtained by using the two equations:

$$f_i = \frac{Q_i - Q_{i+1}}{P\,\Delta z_i} \quad \ldots\ldots\ldots\ldots \quad (9)$$

and $$w_i = w_T - \Sigma \frac{Q_i z_i}{AE} \quad \text{. . . . .} \quad (10)$$

where f_i is the friction at mid height of the i^{th} element of pile, Q_i is the load at the i^{th} strain gage level (as given by Figure 16), P is the pile perimeter including the instrumentation channels, z_i is the length of the i^{th} element, w_T is the movement at the top of the pile and $\Sigma\ Q_i\ z_i/AE$ is the compression of the pile between the top of the pile and the middle of the i^{th} element. The point load of the point load transfer curve was obtained by using the average of the load given by the toe load cell and the load obtained by extension of the last 2 strain gages. The movement for the point load transfer curve was obtained by using the bottom tell tale reading. The friction load transfer curves are shown on Figure 17 and are based solely on the strain gage data. The point load transfer curves are shown on Figure 18. In addition Figure 19 shows the average friction load transfer curve obtained by using the difference between the top load (average of load cell and first strain gage) and the bottom load (average of load cell and extension of the last 2 strain gages). The movement is the movement at the middle of the pile (average of movement at ground surface and bottom movement using tell tales). Figures 18 and 19 show that if the residual loads are ignored (apparent curves), the ultimate point load is too low by 18% and the ultimate friction load too high by 40%. Similar differences were documented in the first phase of this project (Briaud, et al., 1983). The true ultimate point load and friction load at a settlement of 3.33 in. are 86 kips and 24 kips respectively according to the load cells, 75 kips and 42 kips respectively according to the strain gages and 80.5 kips and 33 kips respectively on the average. Figures 17 through 19 also show that the actual load transfer curves do not start from the origin but from the residual stress values. The displacement necessary to mobilize the maximum friction resistance is very small ($\sim$ 0.3 in.) while the displacement to mobilize the point resistance is much larger. As a result at a working load of 50 kips (factor of safety ~ 2) the friction load is 22 kips or 88% of the maximum friction load, the point load is 28 kips or 32% of the maximum point load, and the settlement is 0.083 in. Note that at 50 kips the creep exponent n is 0.045 (Figure 14); equation 8 then allows to calculate the settlement at 50 years as being .083 x (50 yrs/5 hrs)$^{.045}$ = .154 in.

The profiles of maximum friction versus depth are shown on Figure 20. The actual friction profile (including residual stress) can be compared with the apparent friction profile (without residual stress). The shape of the apparent profile tends to support the idea of a critical depth (Meyerhof, 1976) below which the friction remains approximately constant (depth = 12 ft or 13.4 pile diameter). The shape of the actual profile does not show the same trend and tends to support the idea that the friction increases with depth (Coyle, Castello, 1981). Using the actual profile of f_{max} values (Figure 20) and the equation below, it is possible to back calculate the coefficient of horizontal pressure k at the soil pile interface:

$$f_{max} = K\, p'_{ov} \tan \phi \quad . . . . \quad (11)$$

where p'_{ov} is the vertical effective stress at rest at the depth of f_{max}, and ϕ is the soil-pile friction angle taken here as 2/3 ϕ. The friction angle ϕ was measured in direct shear tests to be 35.4°. The resulting profile is shown on Figure 21. The value of k averaged 0.82. For comparison purposes, the coefficient of earth pressure at rest k_o obtained from pressuremeter tests is also shown on Figure 21. The value of k_o averaged 0.85.

Comparison can be made between the predictions (Table 2 and Figure 10) and the measured results (Table 2 and Figure 13). This comparison consists of evaluating which method predicted best the ultimate total load, the ultimate friction load, and the ultimate point load. The methods which predicted these loads the closest were 2 methods based on the cone penetrometer data (DeRuiter-Beringen, 1978 and Schmertmann, 1978). Among the SPT methods Coyle-Castello (1981) and Nordlund (1963) performed best. Also the PDA predictions were good.

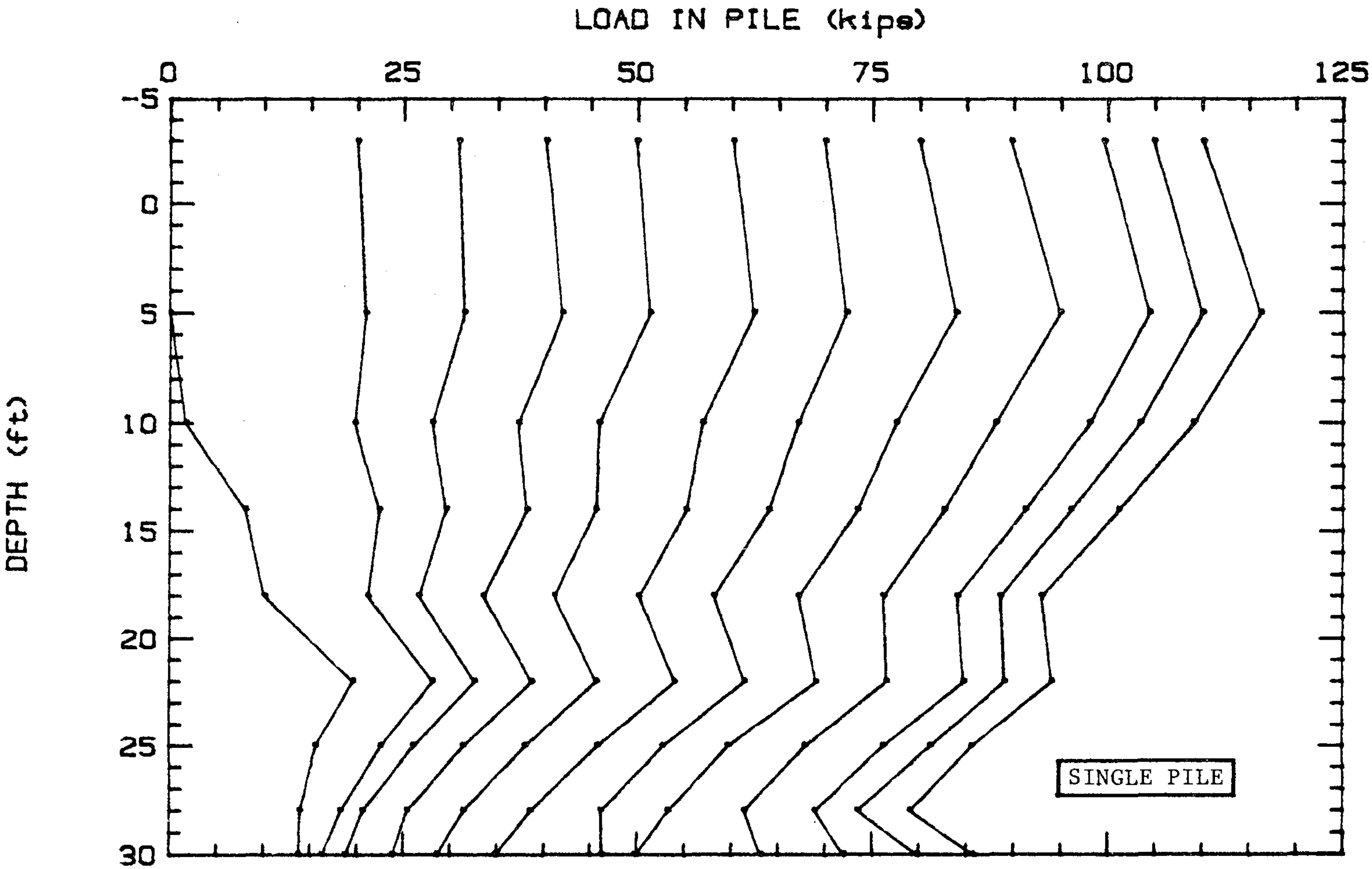


FIG.15- Load vs. Depth Profiles From Actual Measurements (Single Pile).

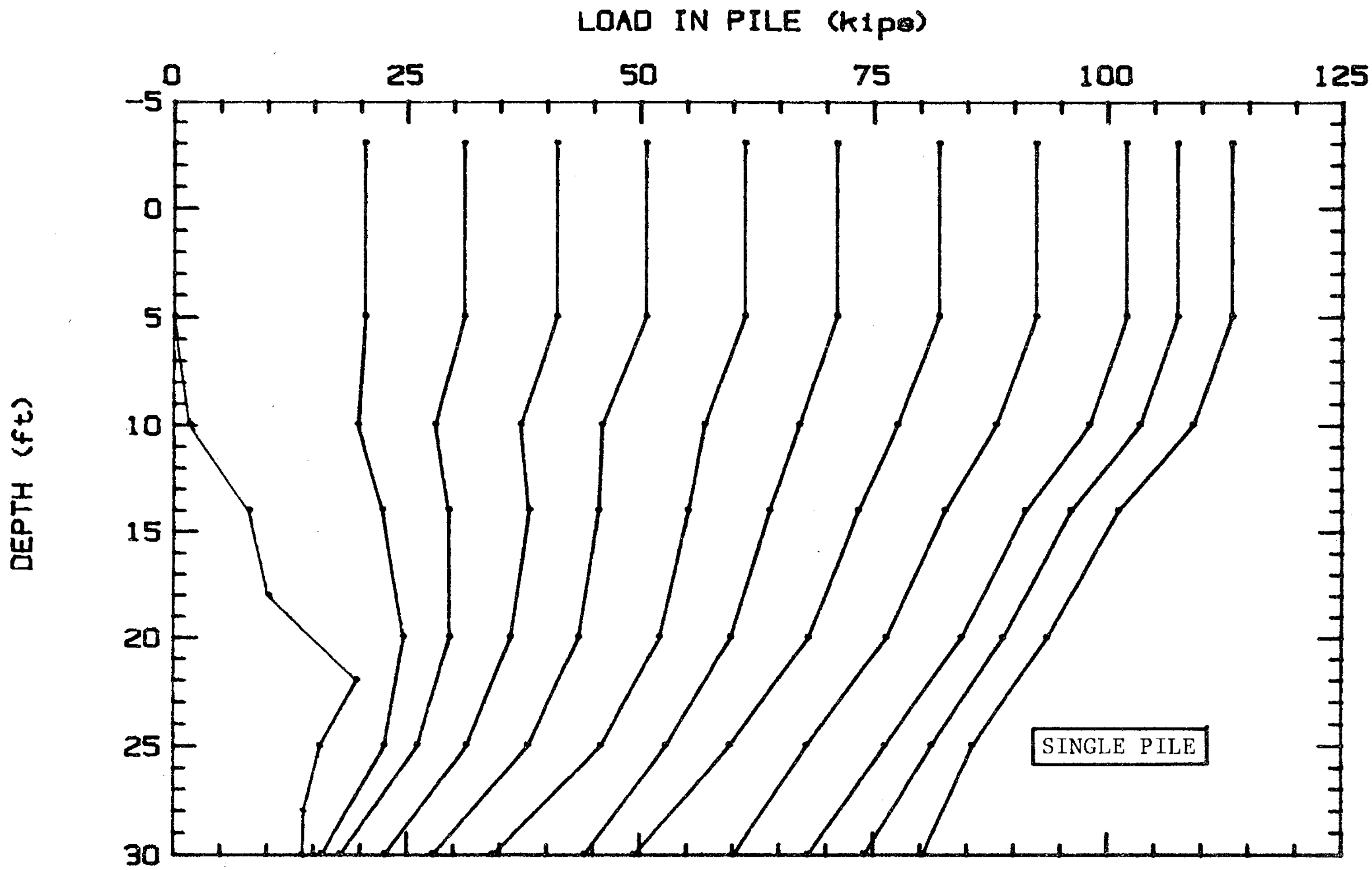


FIG.16- Load vs. Depth Profiles After Interpretation (Single Pile).

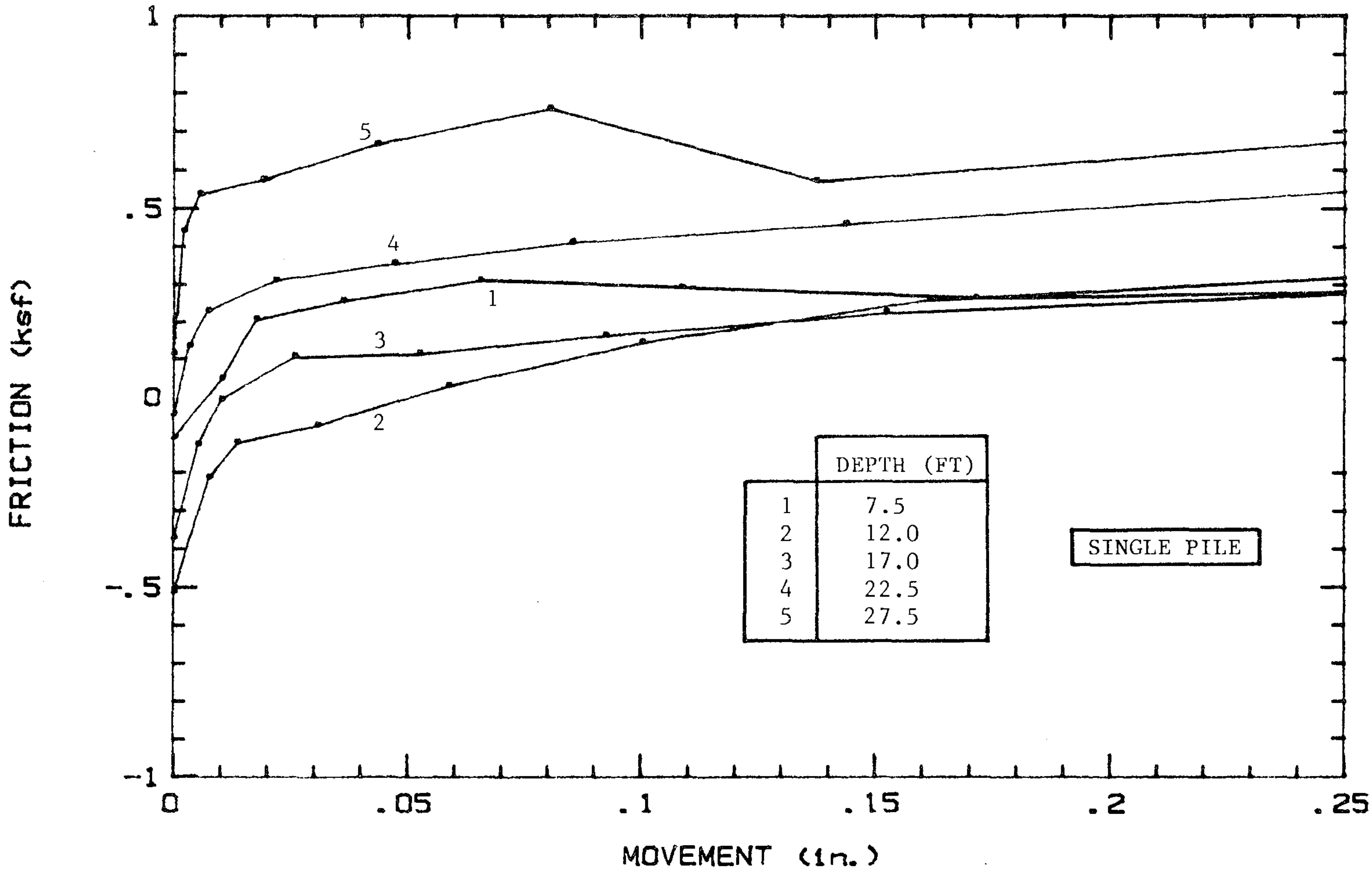


FIG.17a-Friction vs. Movement Curves for Single Pile.

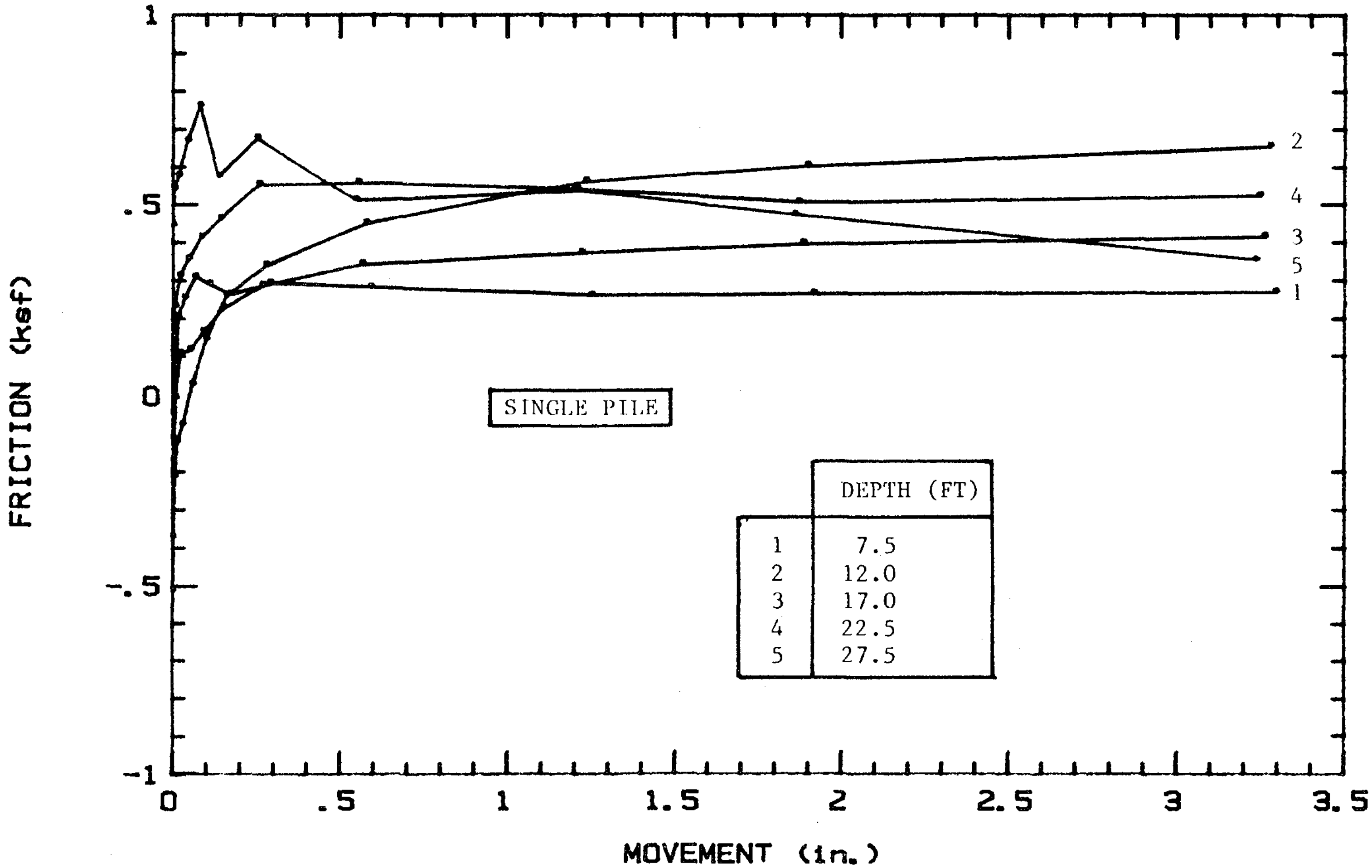


FIG.17b- Friction vs. Movement Curves for Single Pile.

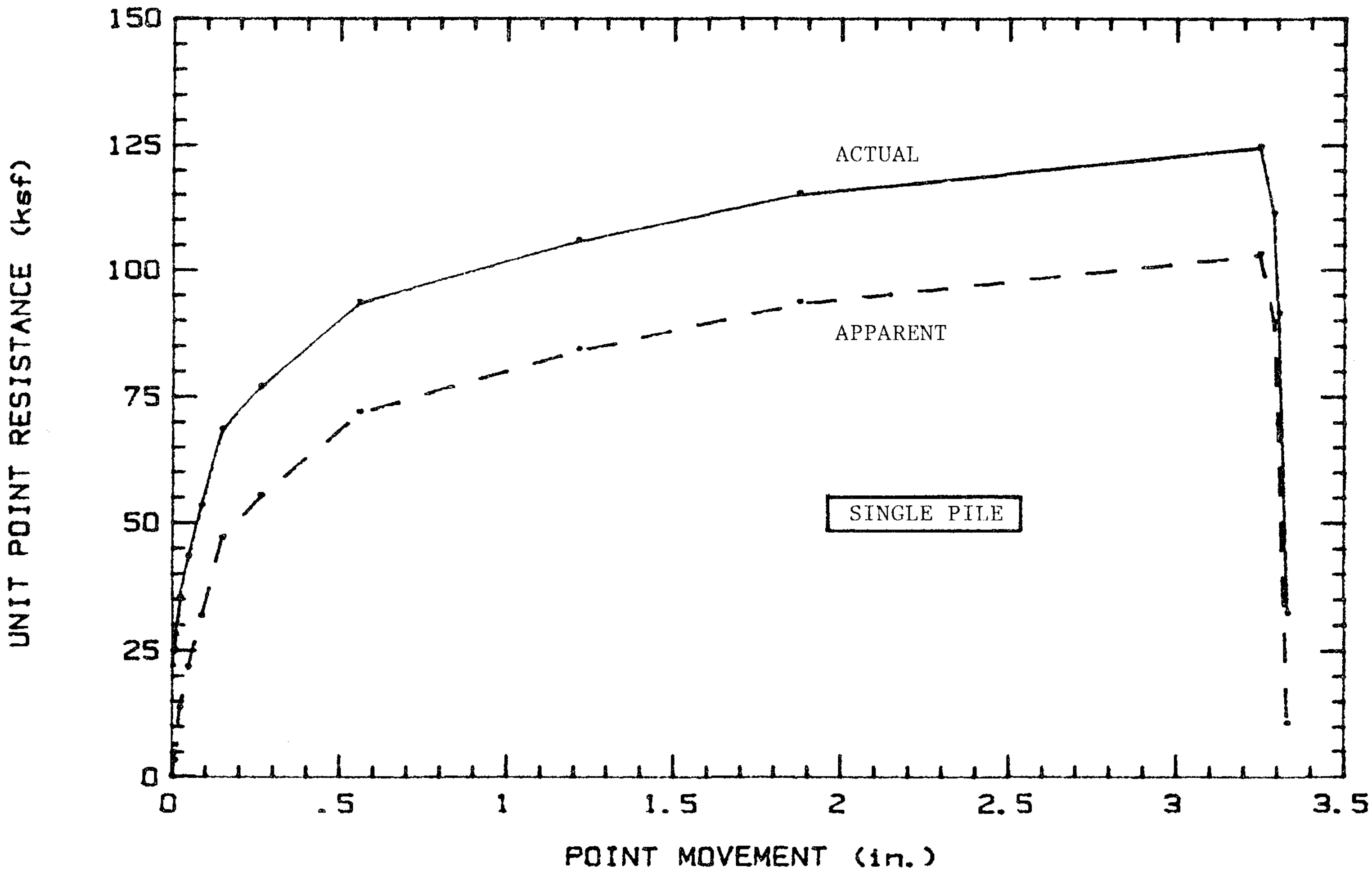


FIG.18- Point Pressure vs. Movement for Single Pile.

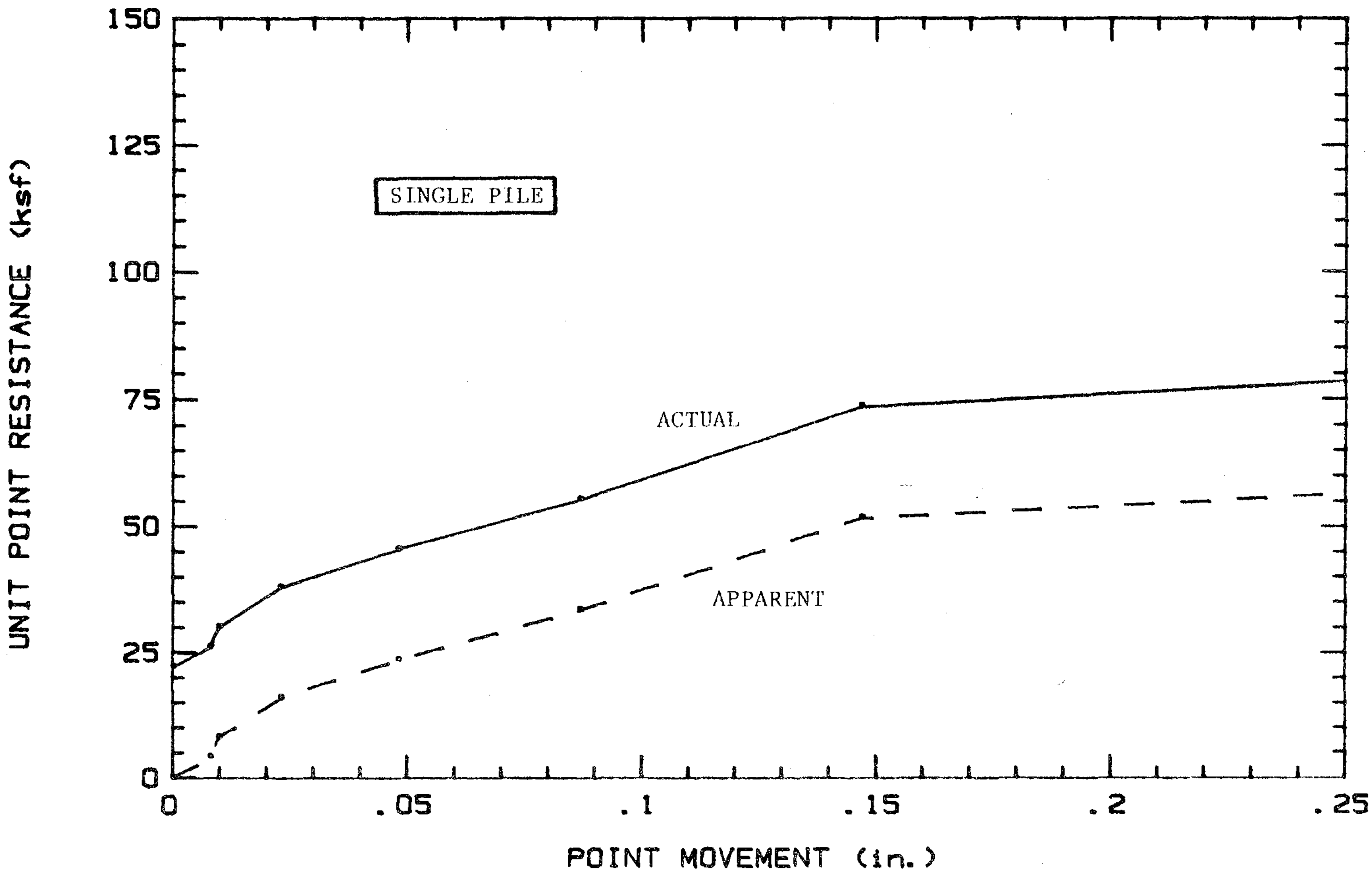


FIG. 18a- Point Pressure vs. Movement for Single Pile.

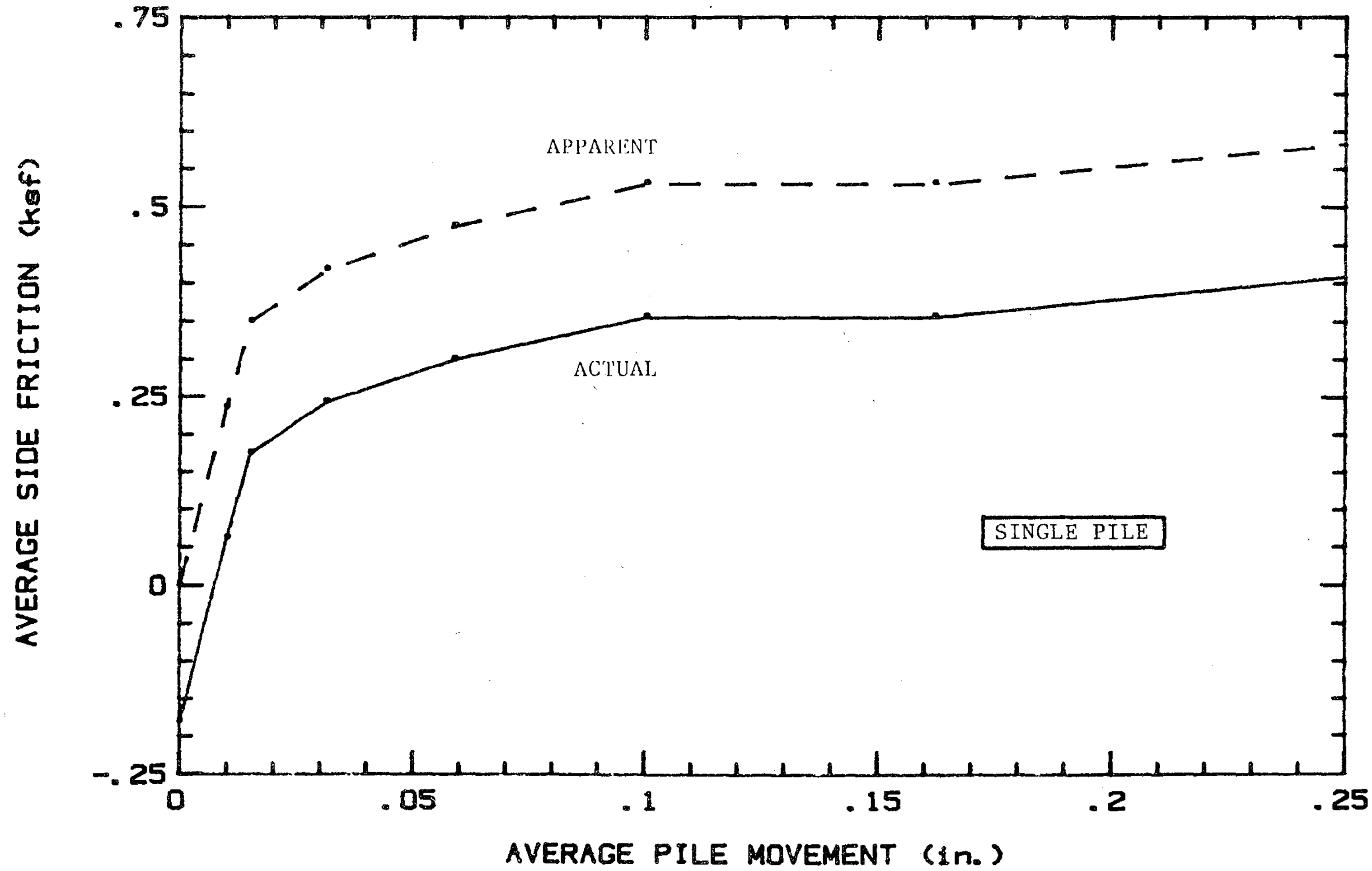


FIG. 19a-Average Friction vs. Movement Curve for Single Pile.

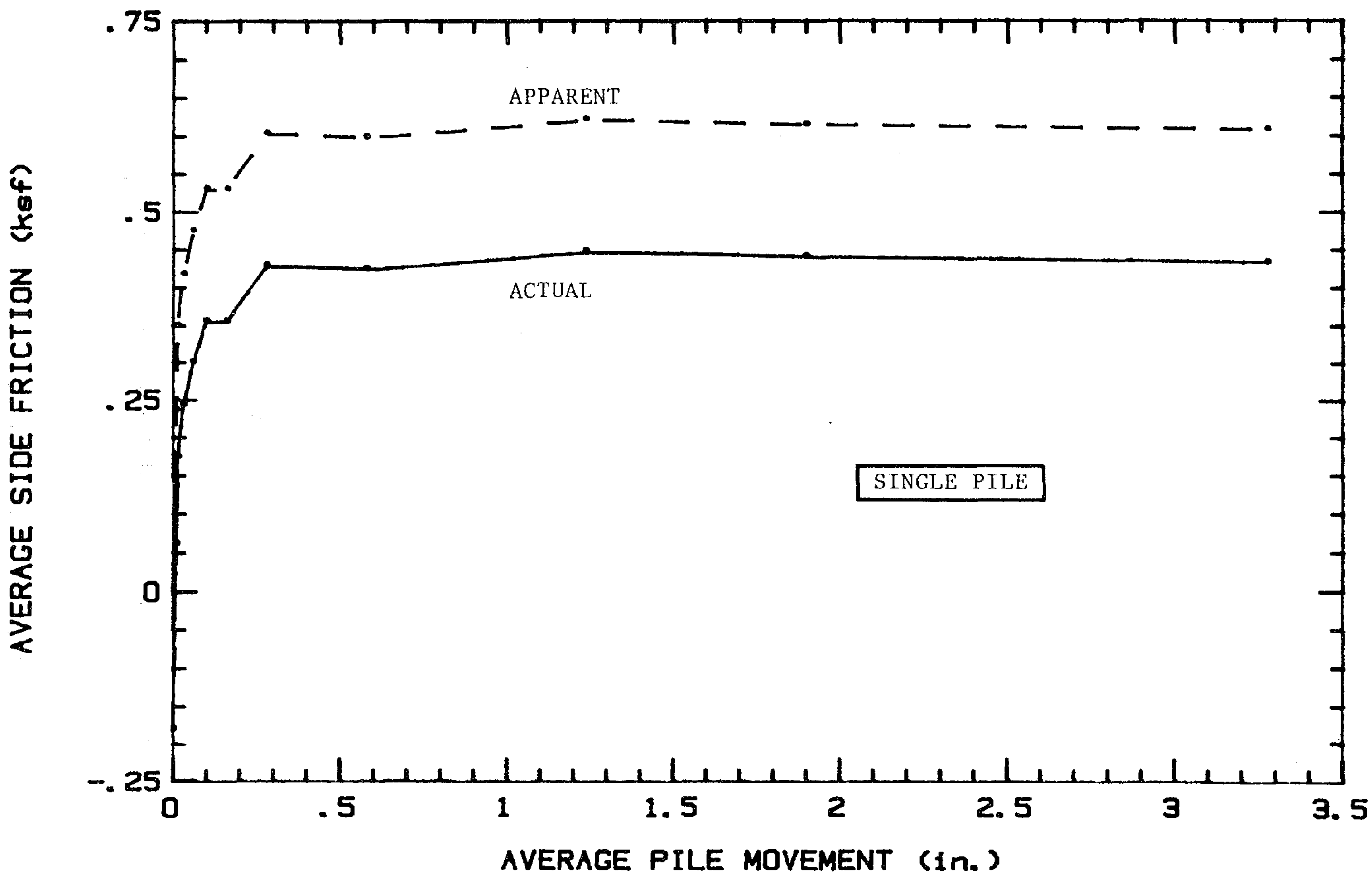


FIG.19b- Average Friction vs. Movement for Single Pile.

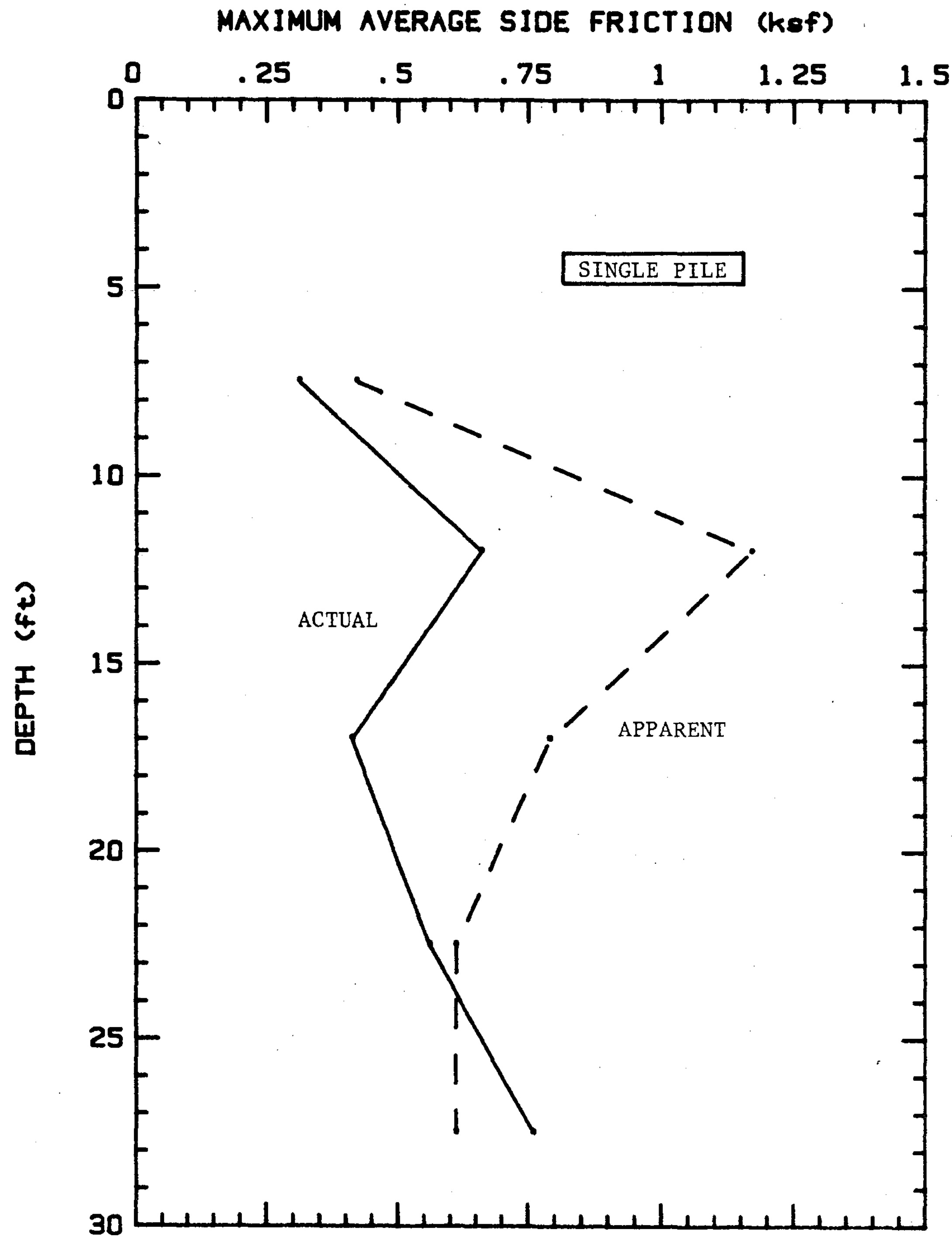


FIG.20- Maximum Friction vs. Depth Profile.

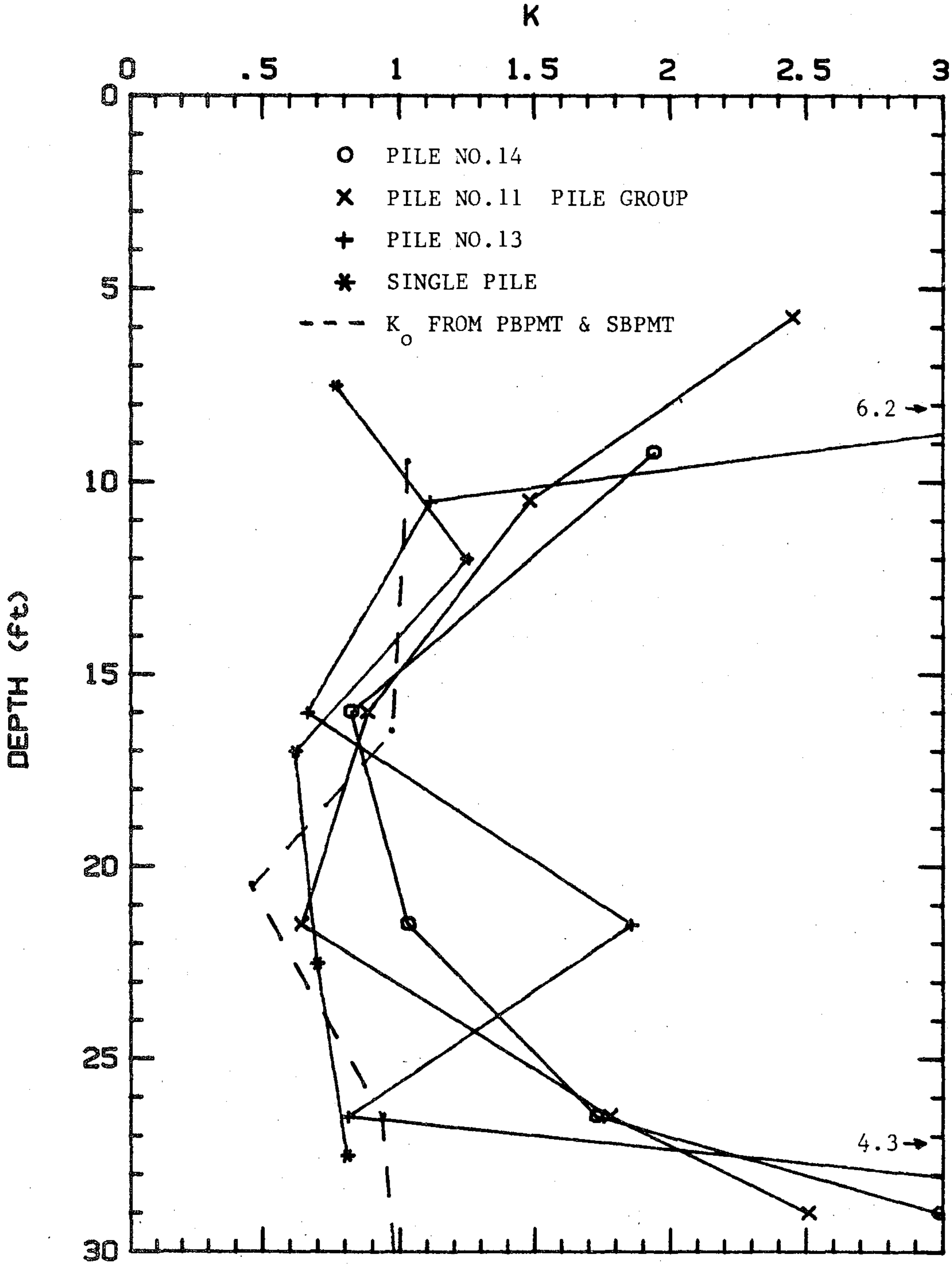


FIG.21- Horizontal Earth Pressure Coefficient.

Analysis of the Group Test Results

The analysis includes residual stresses in each pile, load settlement curve of the group and of each pile, load versus depth profiles for each pile, load transfer curves for each pile, and comparison with the predictions. Three of the five piles in the group had strain gages. These strain gages were zeroed while the piles were laying on the ground. The piles were driven on April 30, 1986 and the strain gages were read on May 1, June 2, June 3, June 4, June 6 in order to obtain the residual loads. The 6x6x5 ft concrete cap which weighed 27 kips was poured in early May and was not in contact with the soil. The load test was carried out on June 6, 1986. The analysis led to 5 profiles of residual loads versus depth for each of the three instrumented piles, one profile for each date above. Due to the scatter in the data and to the loss of some gages, it was decided after various trials to use the average of all the profiles of readings taken after the pouring of the cap. This average profile was modified at the top of the piles to match the weight of the pile cap (5.4 kips per pile). The final profile is shown on Figure 12. Note that the profile is quite similar to the single pile profile except for the residual point load which is very small compared to the single pile (2.5 kips versus 13.8 kips). Therefore in this case the driving of additional piles released the residual point load under the inplace piles but did not release the friction residual stresses; under the point of the inplace piles it was the soil which went down with respect to the piles, thereby decompressing the lower part of the piles, and not the pile which went up with respect to the soil. At the end of the load test, the load was released and additional residual stresses were locked in. The average residual load profile after the load test is shown on Figure 12. Note that this profile is very close to the single pile profile after loading.

The load settlement curve is shown on Figure 22. The data points correspond to the readings at the end of each 30 minute load step. The settlement is an average of the four settlement readings taken at the corners of the pile cap and the load is the sum of the 4 load cells. In the case of the group, the load cells and strain gage loads matched very well. The time to reach the end of the final load step was 9.7 hours.

The highest load during the last load step was 561 kips; at the end of that load step the settlement was 7.1 in. As can be seen from the curve, little additional load could have been carried by the group had penetration continued past 7.1 in. In that sense 561 kips can be considered as the true ultimate load for the group. Since the corresponding ultimate load for the single pile was 113.5 kips, the efficiency e is 0.99. This does not conform with the expected value since in this medium dense sand, one may have expected densification to lead to a higher efficiency. Careful inspection of the SPT data and CPT data close to the group (B5, B6, CPT3, CPT101) and close to the single pile (B7, CPT102) does not show any significant difference in soil strength (Figure 23). Boring B6 shows blow count lower than the others 5 ft below the base of the group, yet B5 shows a blow count higher than the others at the base of the group. CPT101 gives an average point resistance q_c slightly higher than CPT102 from 30 to 35 ft depth.

The settlement ratio r_s is defined here as the ratio of the settlement of the 5 pile group s_G at 5 times the safe load for the single pile (275 kips) over the settlement of the single pile s_1 at the safe load for the single pile taken as one-half the ultimate load (1/2 x 110 = 55 kips). The value of s_1 is 0.107 in. and the value of s_G is 0.138 in.; the settlement ratio is 1.29. The settlement of the group is only slightly larger than the settlement of the single pile; in fact the load settlement curve obtained by multiplying by 5 the load of the load settlement curve for the single pile is, in first approximation, close to the load settlement curve for the group (Figure 24).

The creep load for the group can be determined in the manner used for the single pile. Figure 25 shows the determination of the creep load which is 432 kips. Since the creep load for the single pile was 88 kips, the efficiency for the creep load is 0.98. The percent load carried by each pile during loading is presented on Figure 26. This figure shows that at working loads the corner piles carried on the average 12% more load than the center pile; however at the ultimate load the trend was reversed and the center pile carried 5% more load than the average of the corner piles.

The load versus depth profiles for pile 14 is shown on Figure 27. This profile is obtained directly from the strain gage readings. As can be seen it appears that two levels of strain gages did not work properly; it was decided to produce an interpreted profile (Figure 28). Note that since a 12 in. diameter hole was drilled down to 4.5 ft prior to driving, the load in the pile down to 4.5 ft should be close to constant. The interpreted profiles for piles 11 and 13 are shown on Figures 29 and 30. Note that, unlike in the case of the single pile, the load cells at the top and bottom of the piles agree quite well with the loads measured by the strain gage. As in the case of the single pile the first load versus depth profile is the residual load profile of Figure 12. At the ultimate load, the top loads, the point loads and the friction loads carried by the five piles are presented in Table 3. The tell tale readings on the 2 unstrain-gaged piles in the group (Pile 12 and 15) did not lead to any reasonable values of friction and point loads. No special trend is obvious when comparing the friction and point load carried by the various piles in the group. When comparing these loads with the loads in the single pile however it is clear that the piles in the group carried more load in friction and correspondingly less load in point resistance. This indicates that driving piles in groups tends to increase horizontal stresses on the pile shafts and to decrease the resistance of the soil below the pile points.

The friction load transfer curves and the point load transfer curves were obtained by using the same procedure as the one used for the single pile. The friction curves are shown on Figures 31 to 33 and the average friction curve for each pile is shown on Figure 34. Figure 35 shows the point curves. On Figures 34 and 35 the results of the single pile are shown for comparison purposes. As in the case of the single pile the displacement necessary to mobilize friction is very small (0.35 in.) while the displacement to mobilize the point resistance is much larger. As a result, at a working load for the group of 244 kips (Factor of Safety ~ 2), the friction load is 214 kips, the point load is 30 kips and the settlement is 0.1 in.

The profiles of maximum friction versus depth profiles are shown on Figure 36. These profiles are the actual profiles including proper

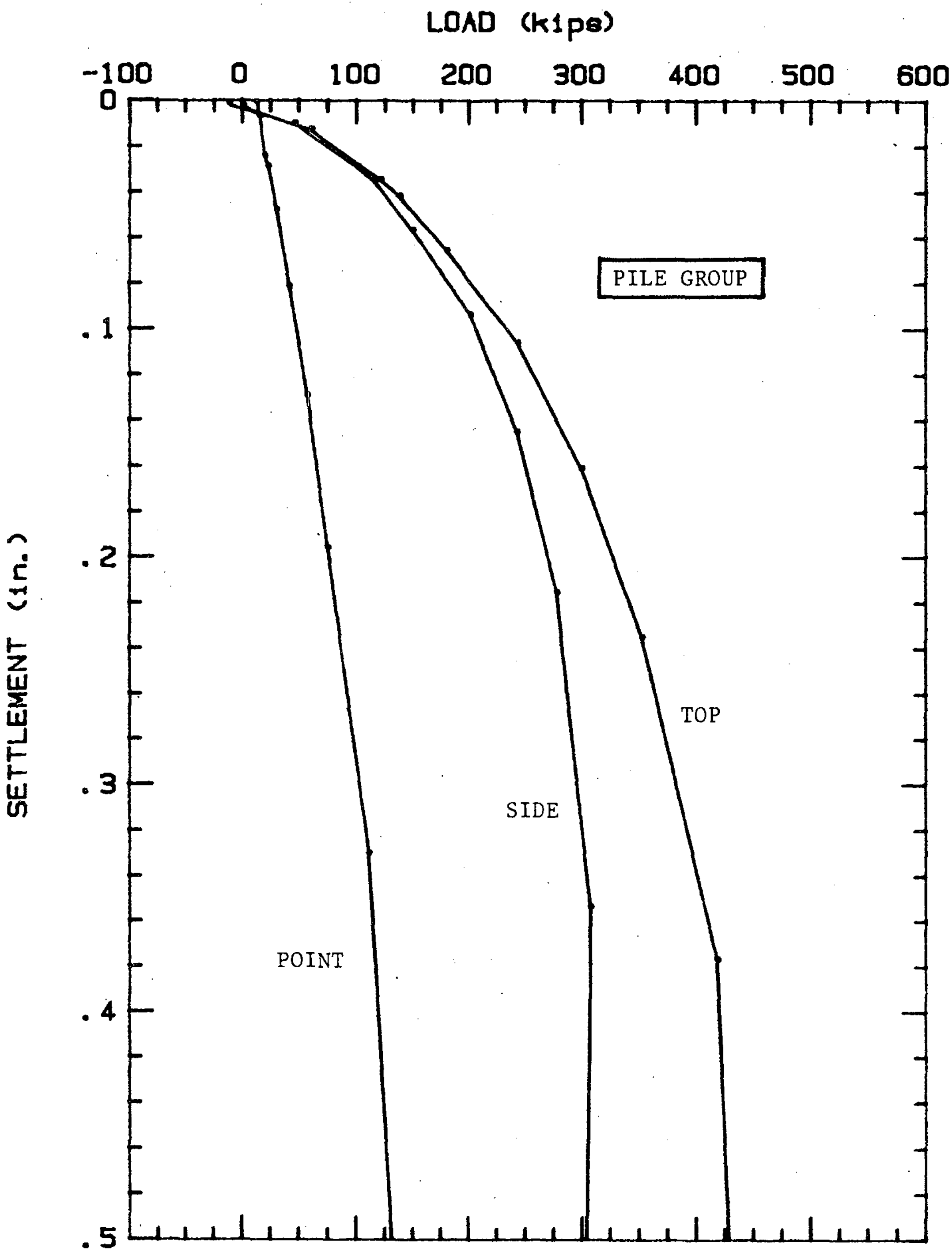


FIG.22a- Measured Load-Settlement Curve
for Pile Group.

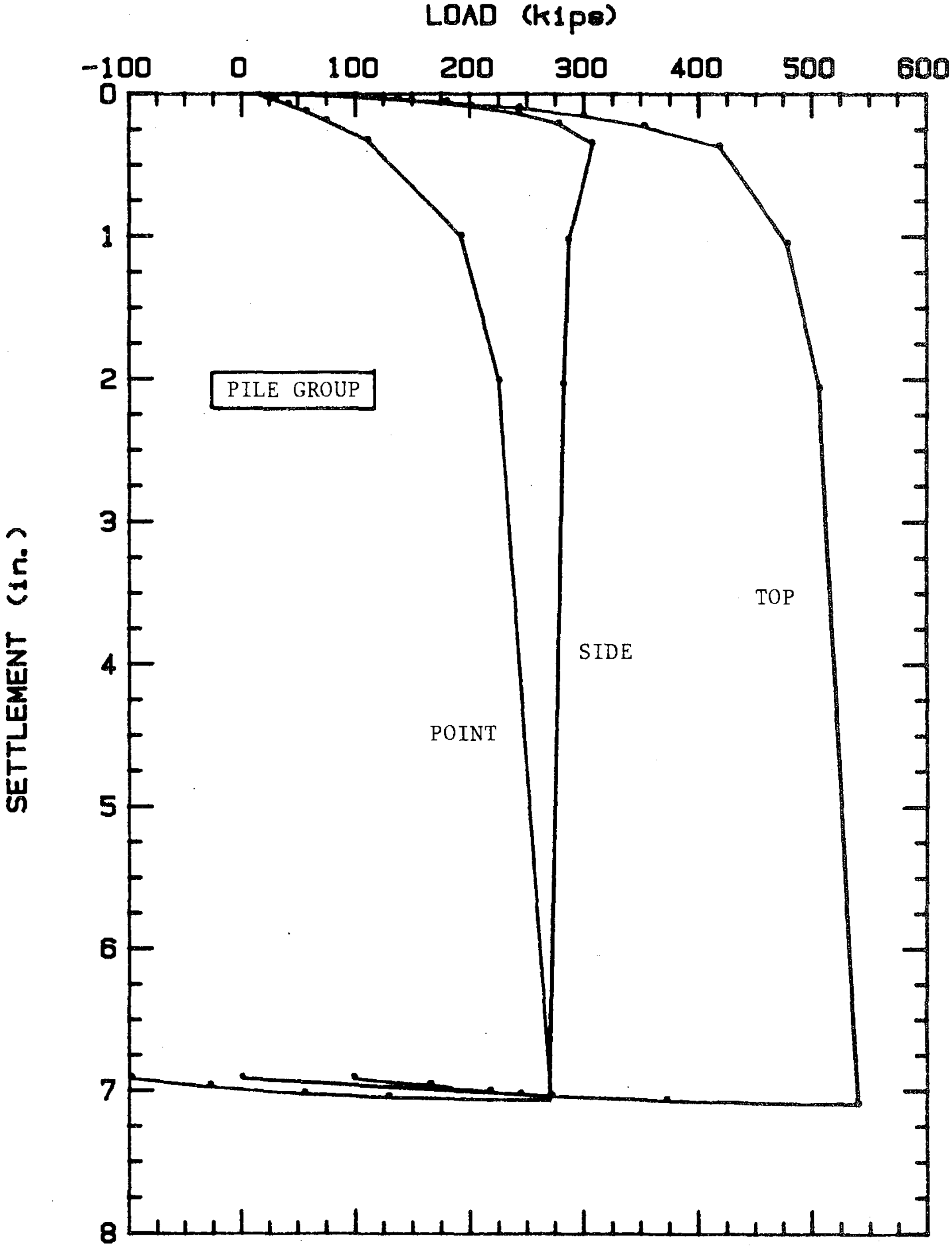


FIG.22b- Measured Load-Settlement Curve
for Pile Group.

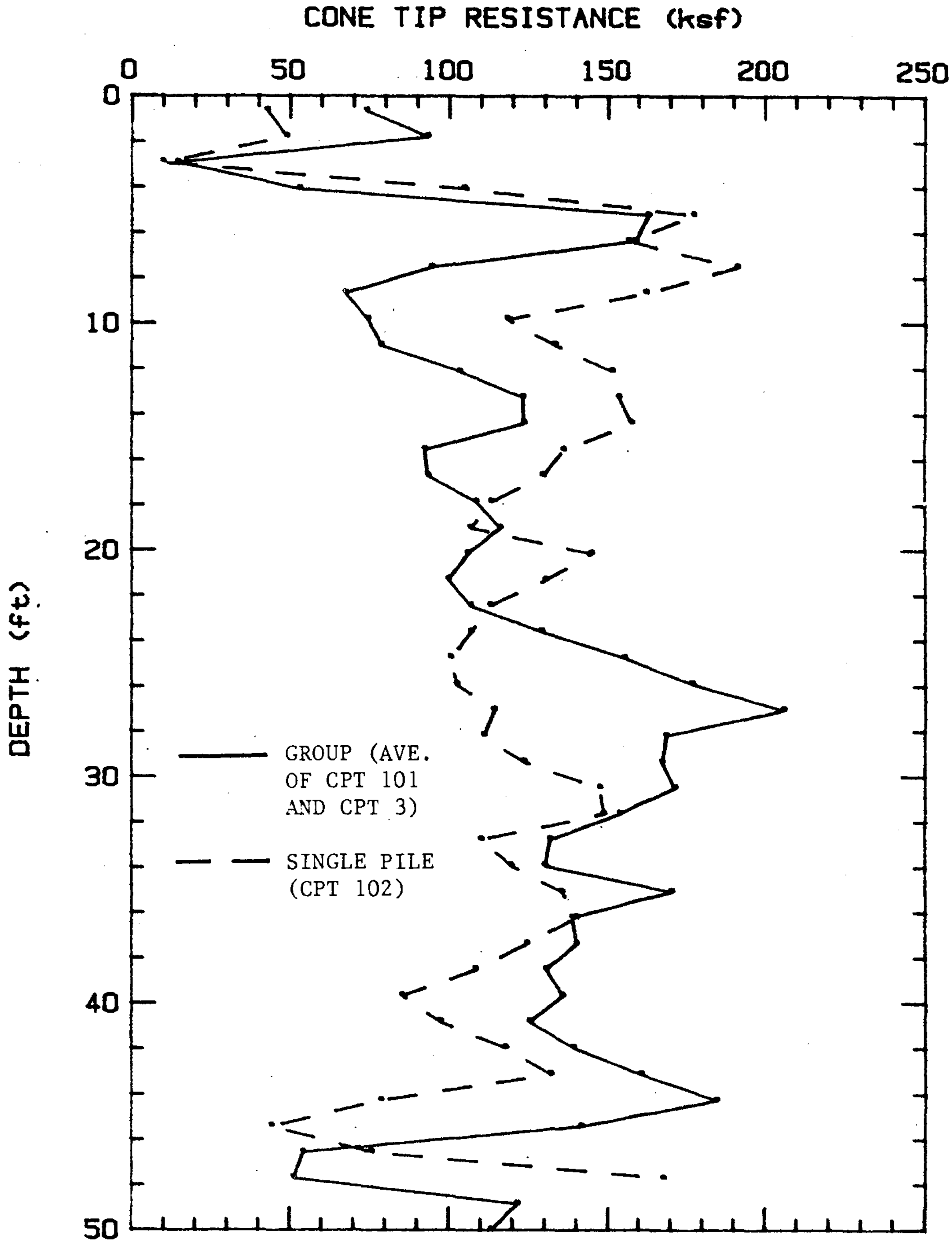


FIG.23a- Comparison of Soil Profiles at Single Pile and Pile Group Location.

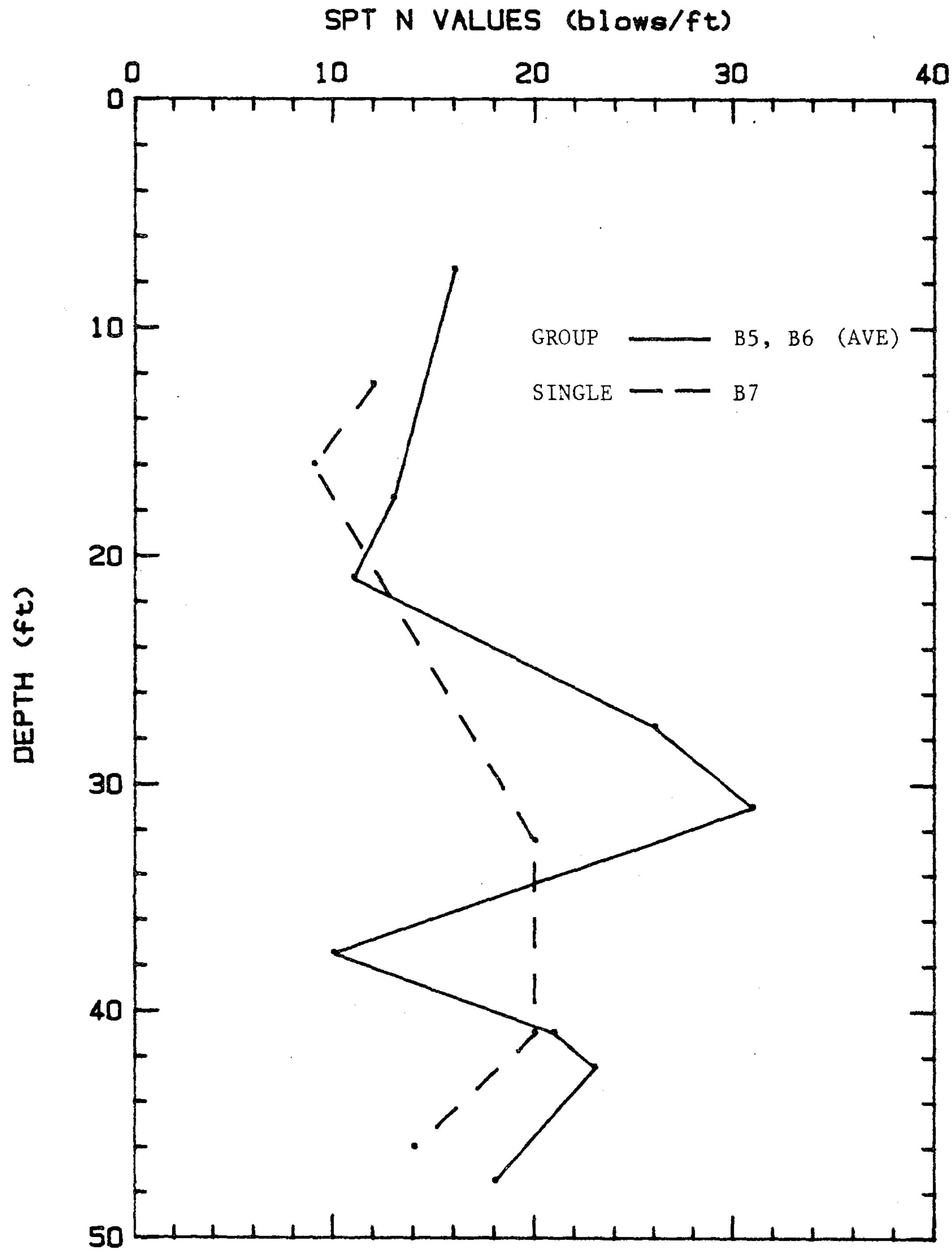


FIG.23b- Comparison of Soil Profiles at Single
Pile and Pile Group Location.

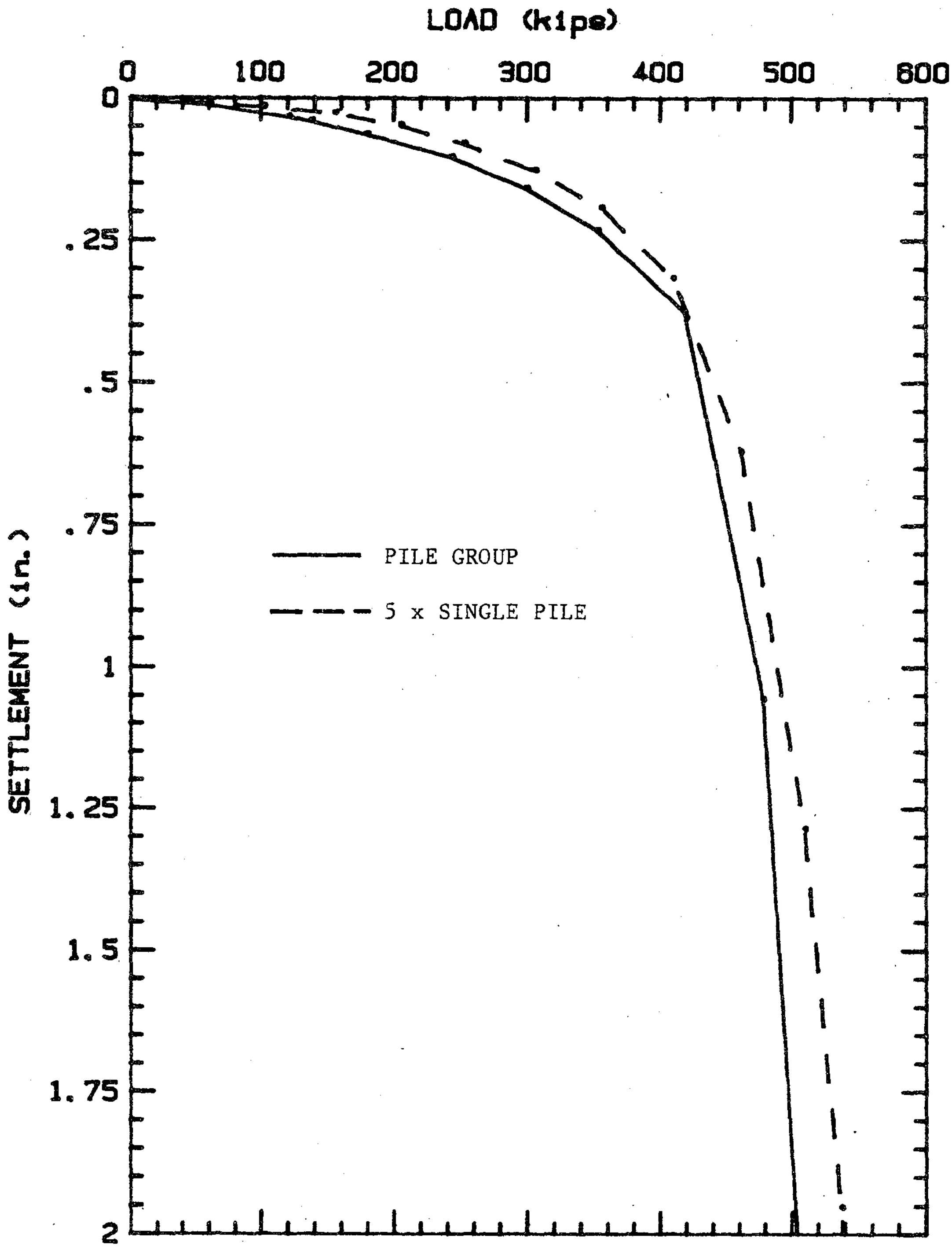


FIG.24- Comparison of Pile Group and Single Pile Load-Settlement Curves.

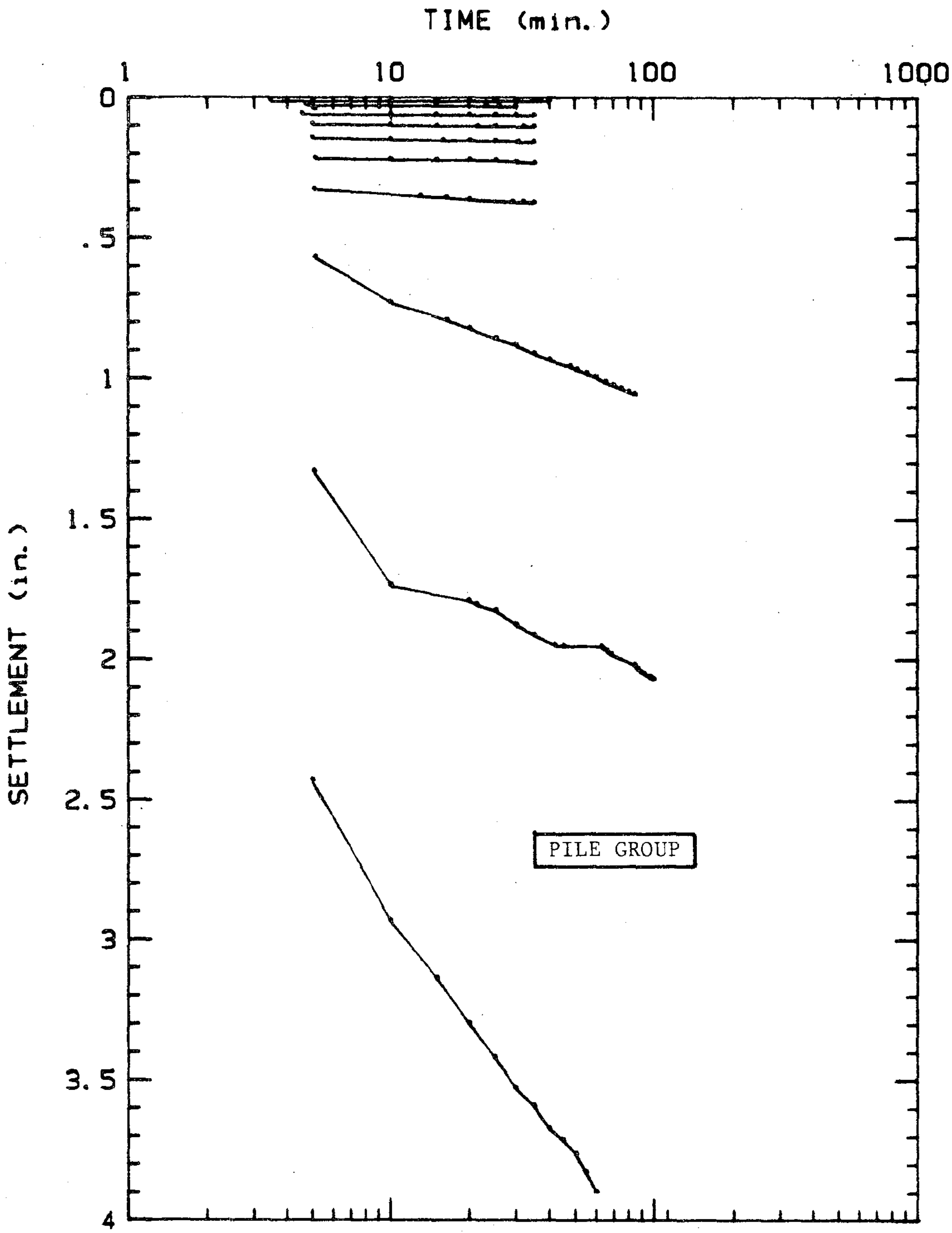


FIG.25a- LogTime vs. Settlement for Load
Steps of Pile Group Load Test.

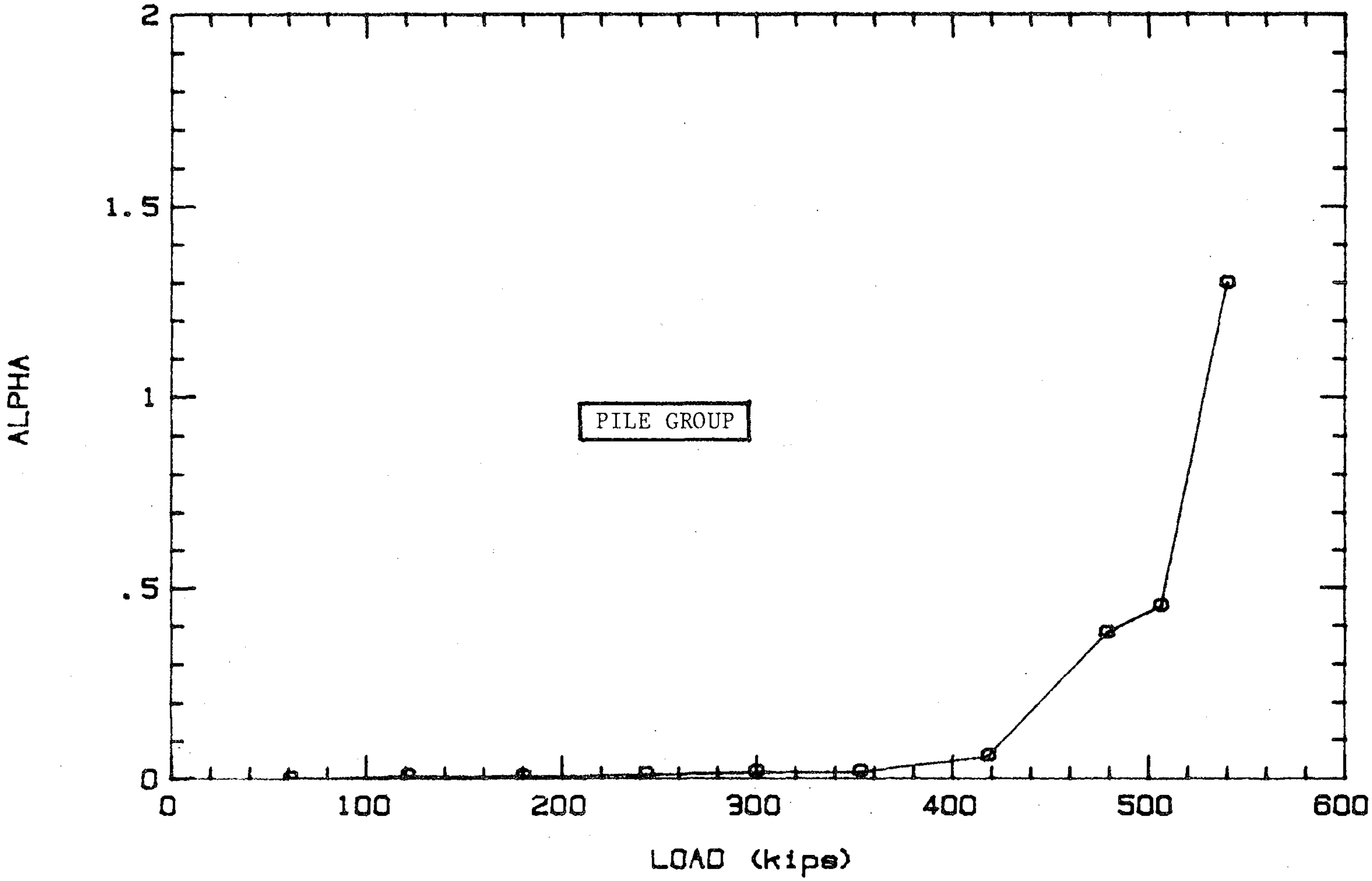


FIG.25b- Slope of LogTime vs. Settlement for Load Steps
of Pile Group Load Test.

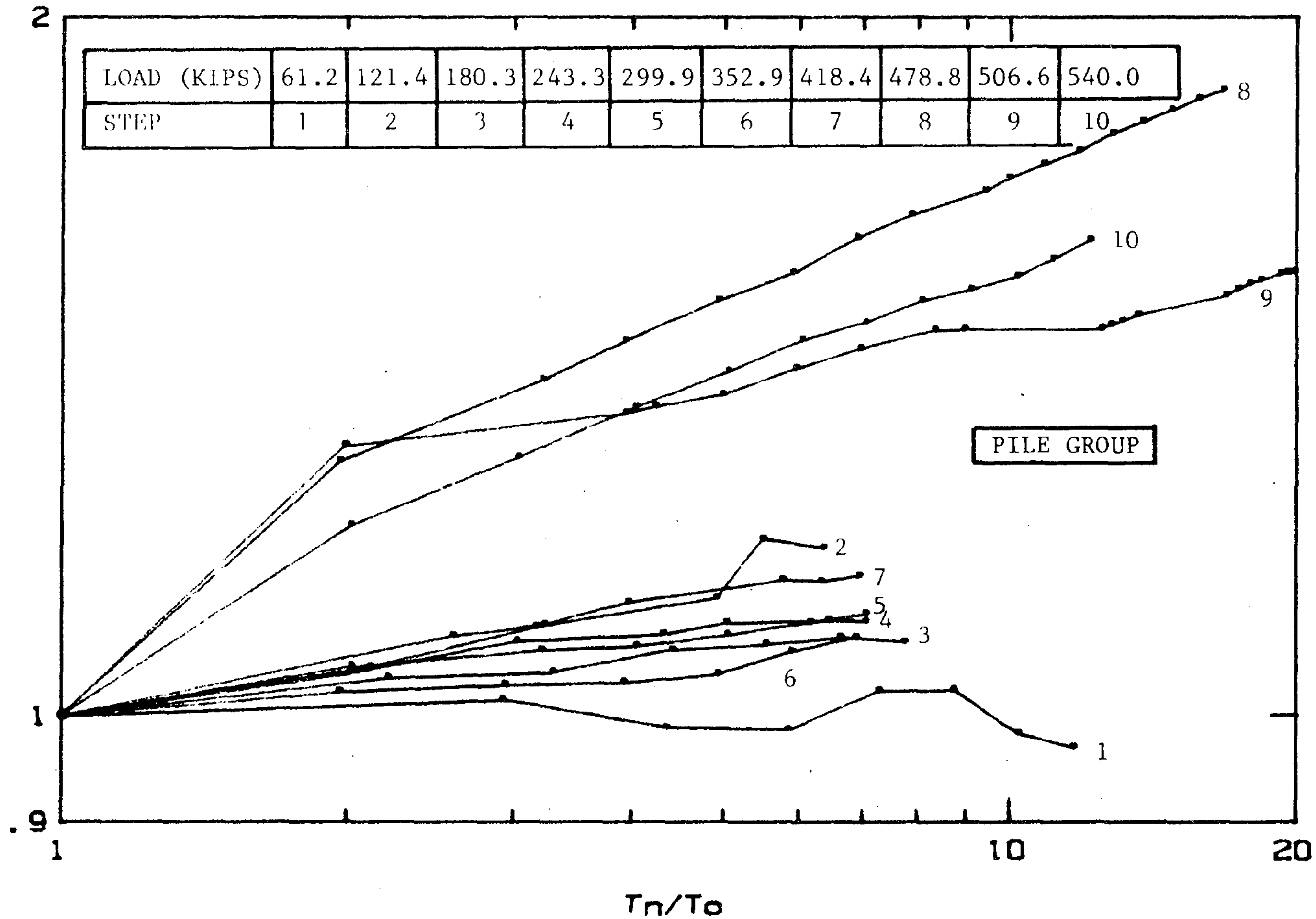


FIG.25c- LogTime vs. LogSettlement for Load Steps of Pile
Group Load Test.

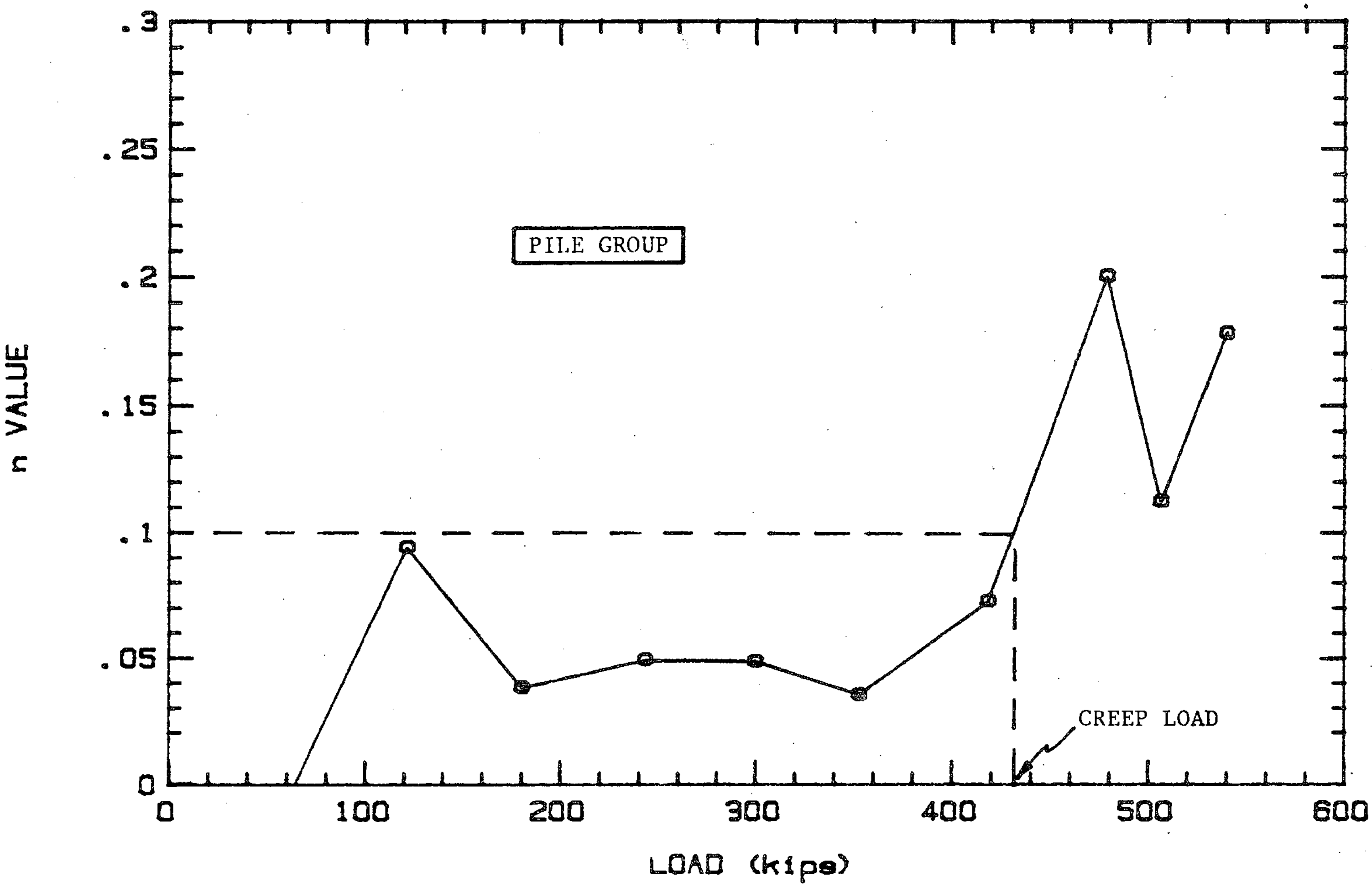


FIG.25d- Creep Exponent n for Load Steps of Pile Group Load Test.

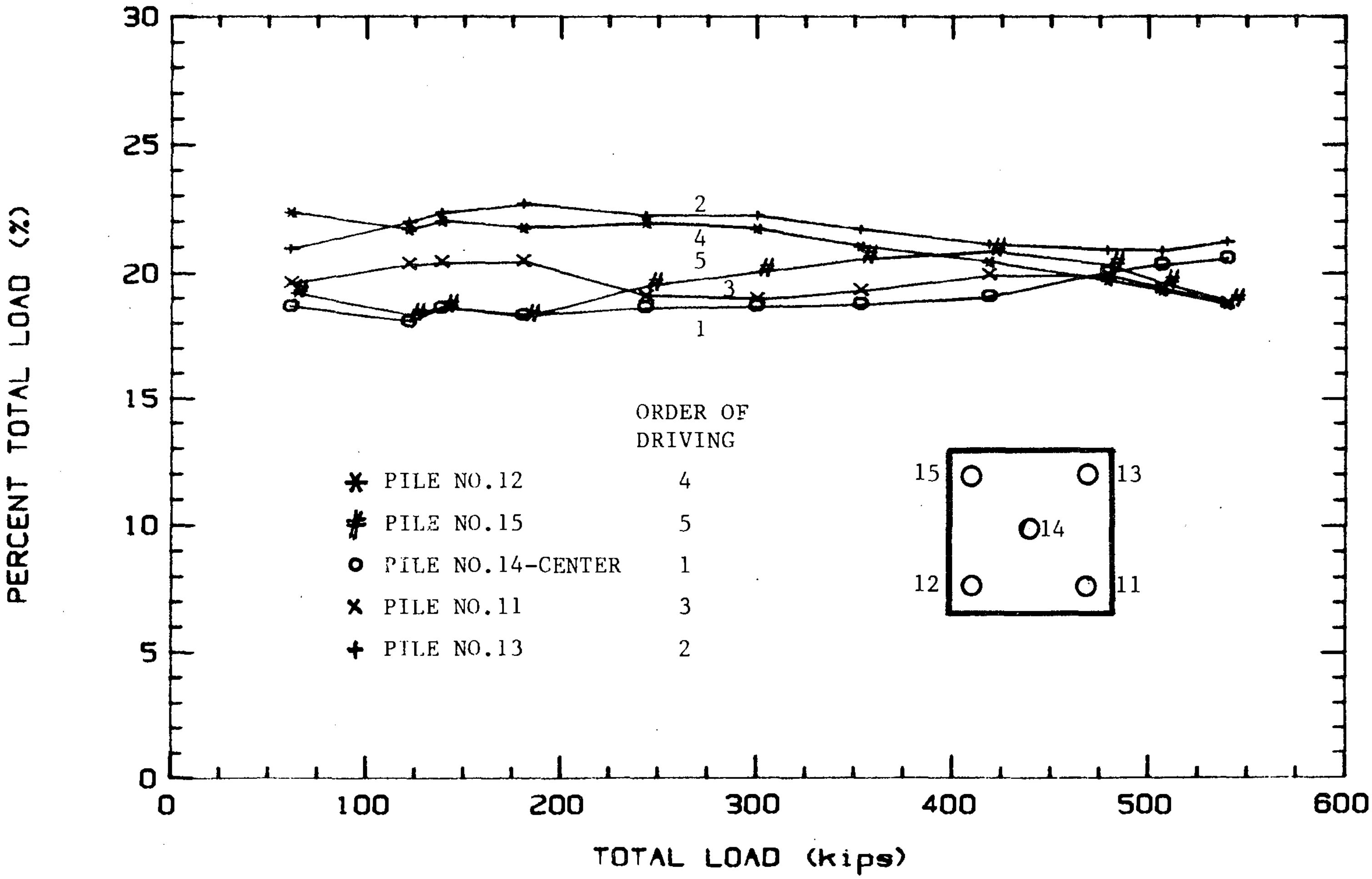


FIG.26- Percent of Total Load Carried by Each Pile in the Group.

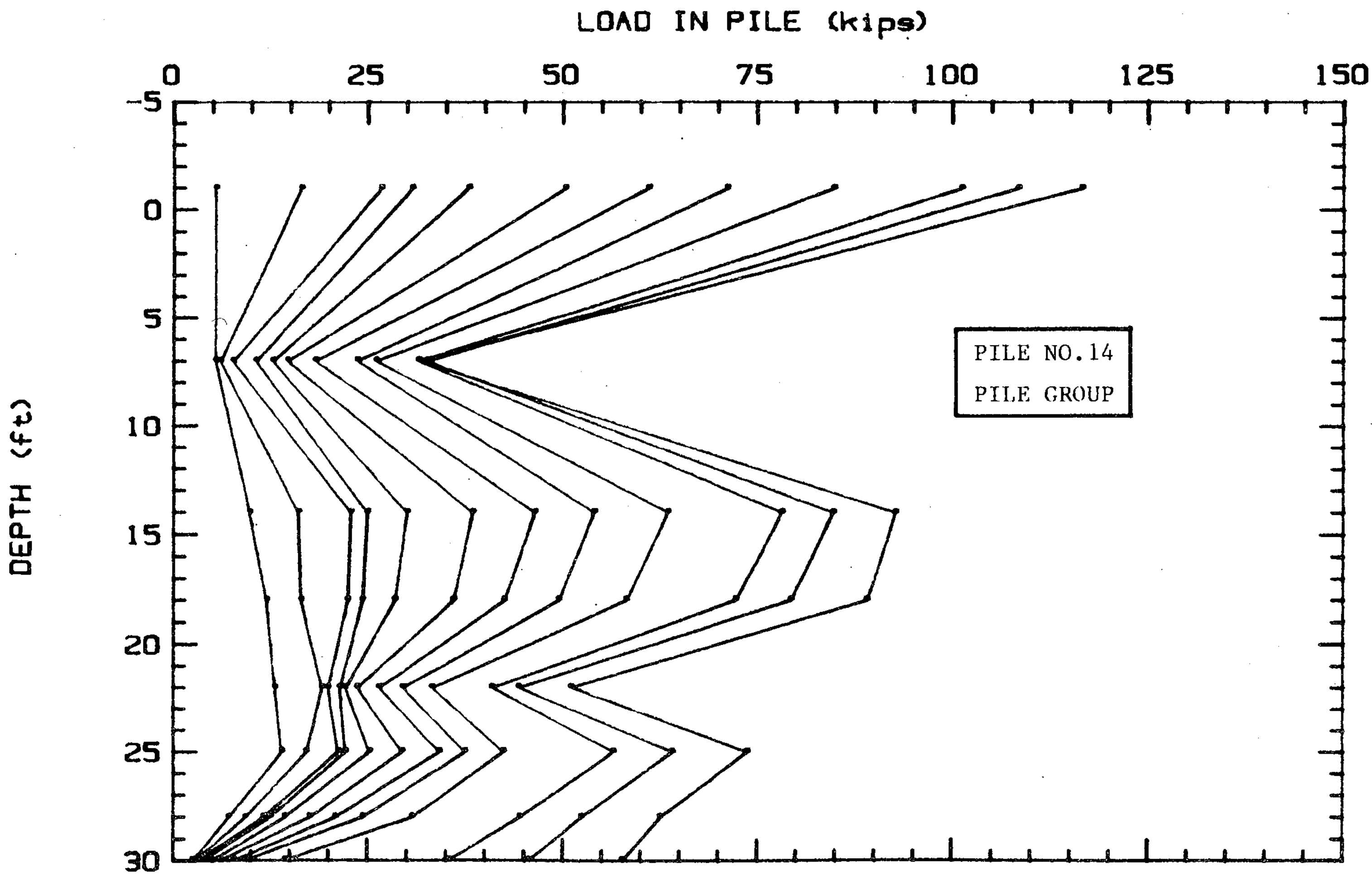


FIG.27- Load vs. Depth Profiles From Actual Measurement(Pile Group).

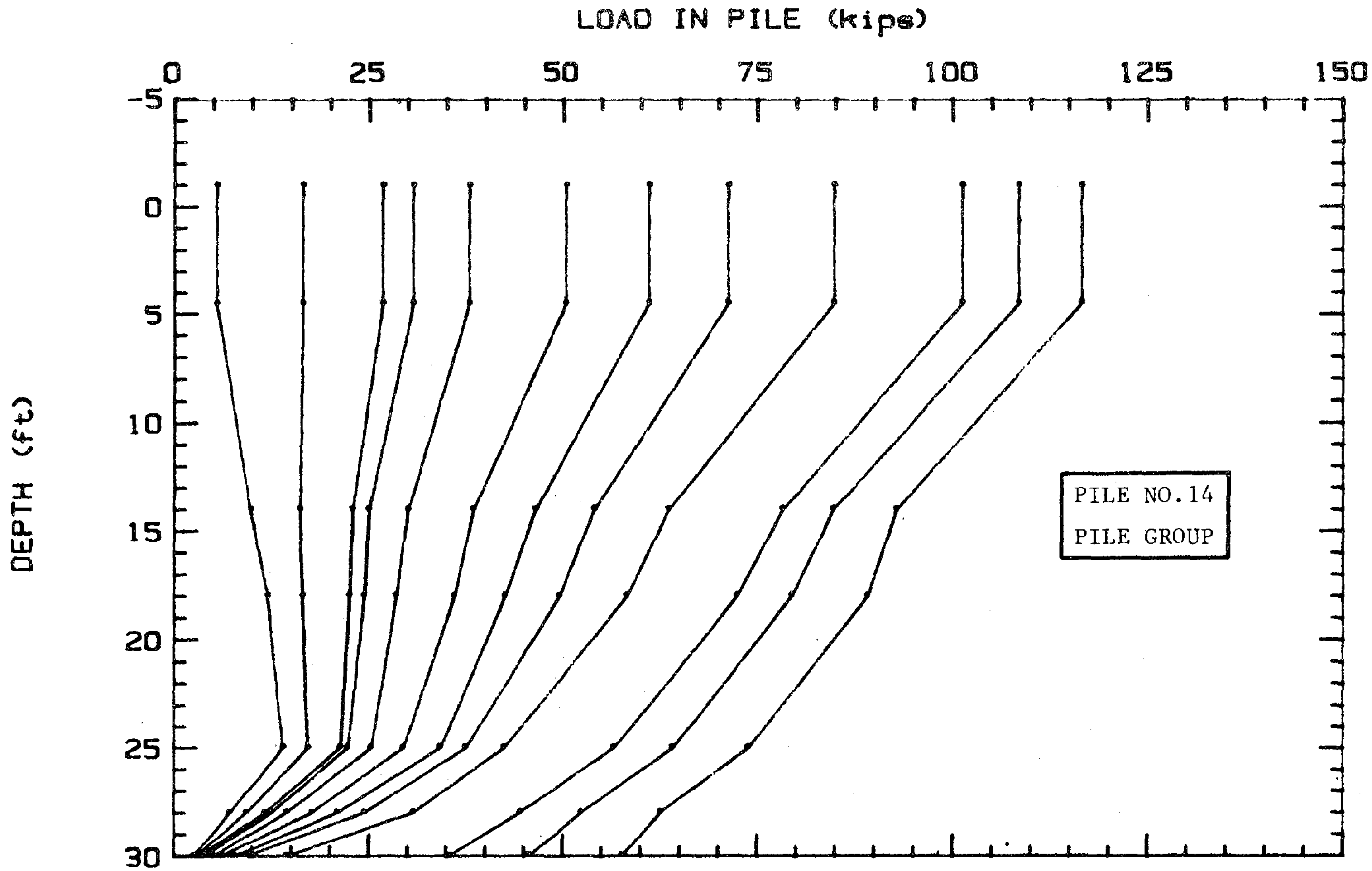


FIG.28- Load vs. Depth Profiles After Interpretation (Pile no.14 in Group).

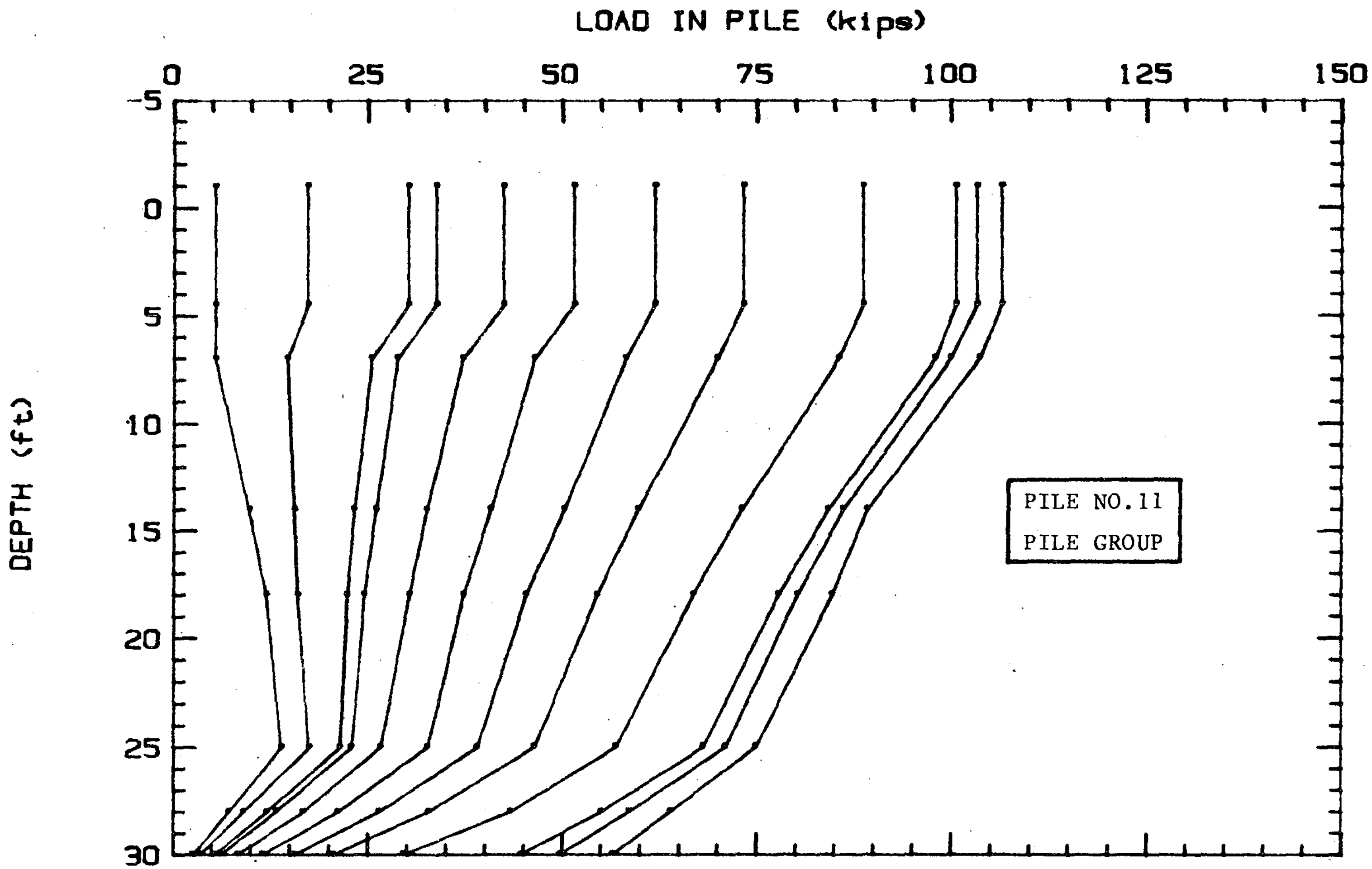


FIG.29- Load vs. Depth Profiles After Interpretation (Pile no.11 in Group).

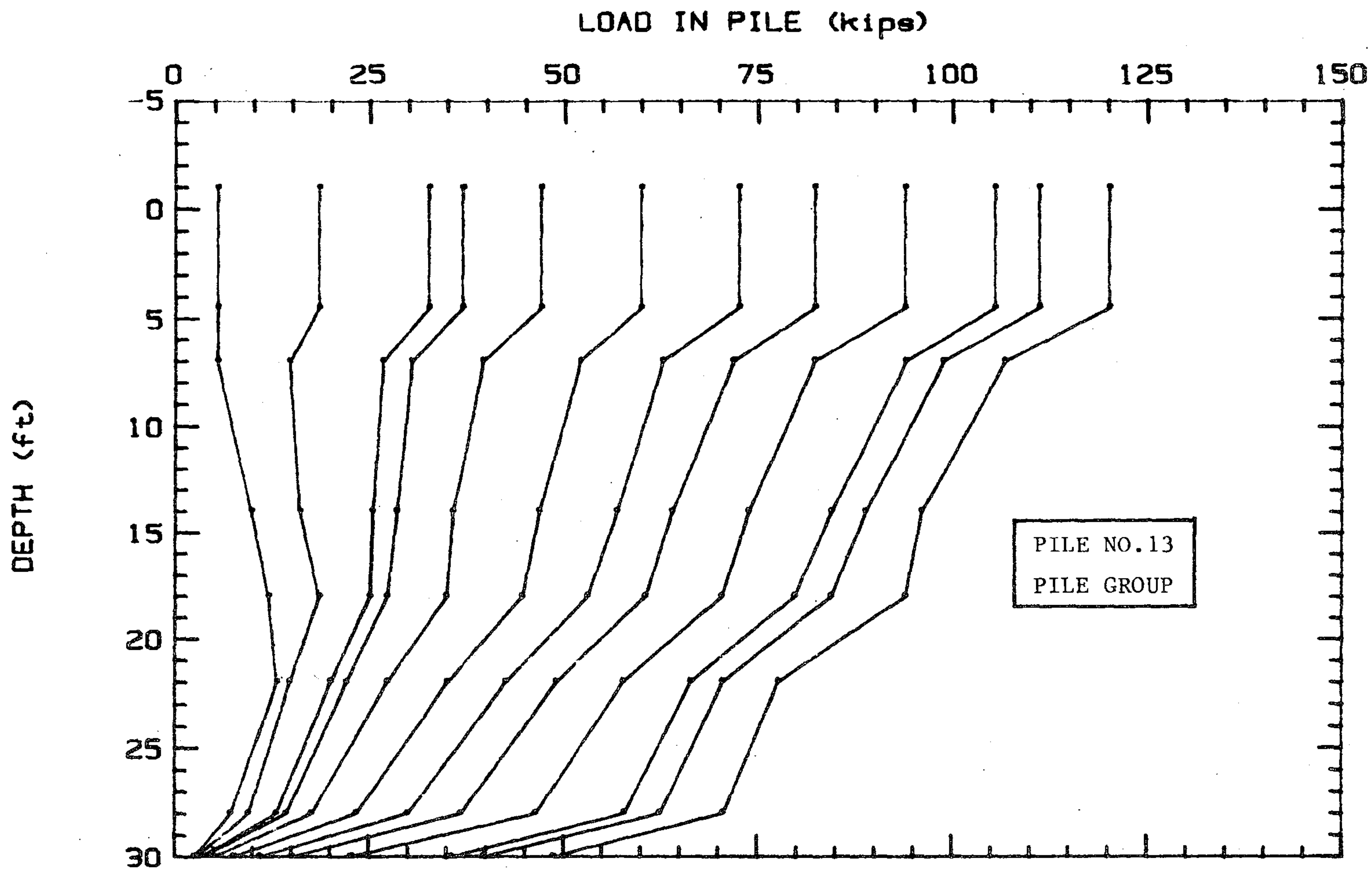


FIG.30- Load vs. Depth Profiles After Interpretation (Pile no.13 in Group)

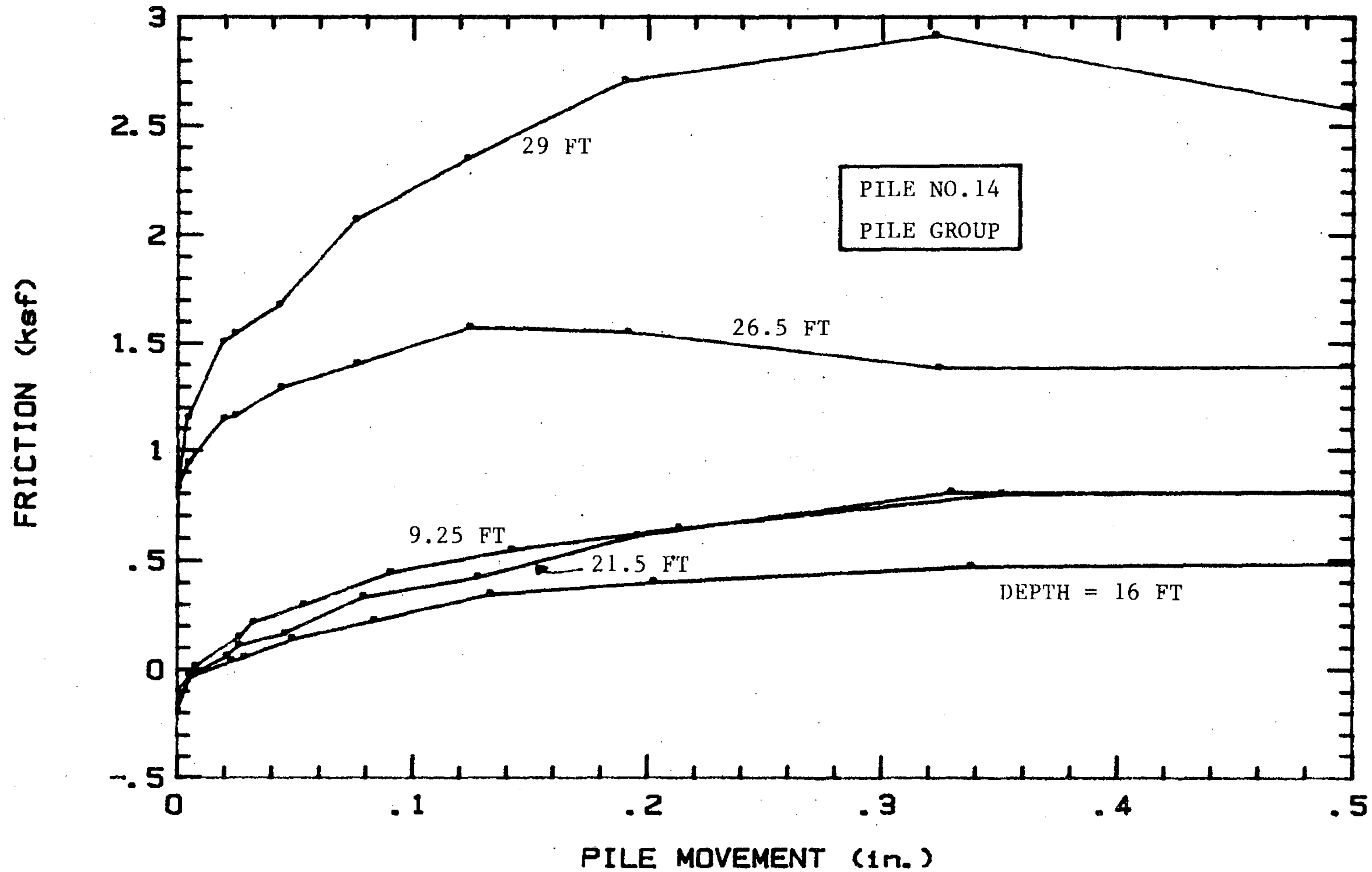


FIG.31a- Friction vs. Movement Curves for Pile No.14 in Group.

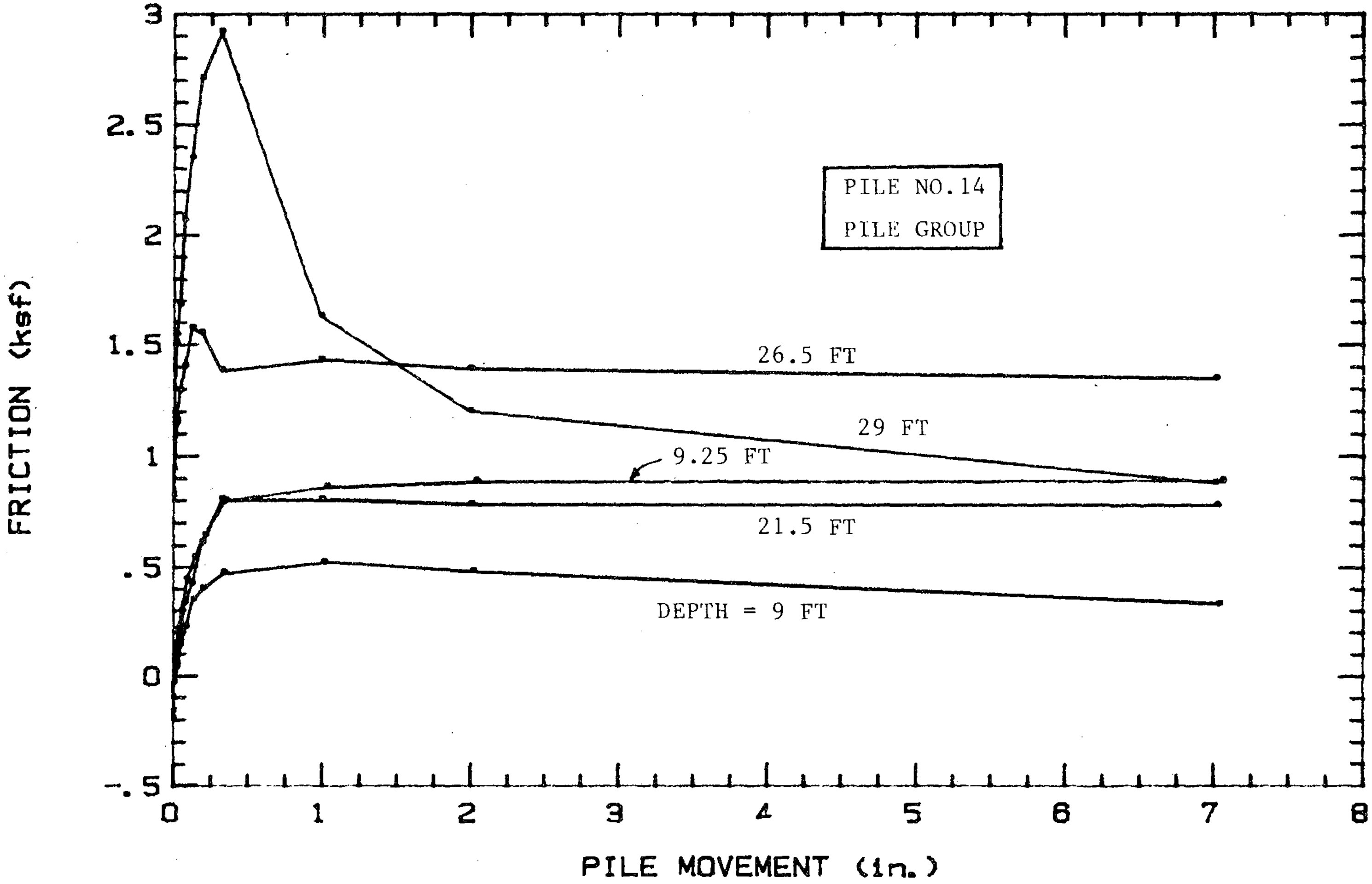


FIG.31b- Friction vs. Movement Curves for Pile No.14 in Group.

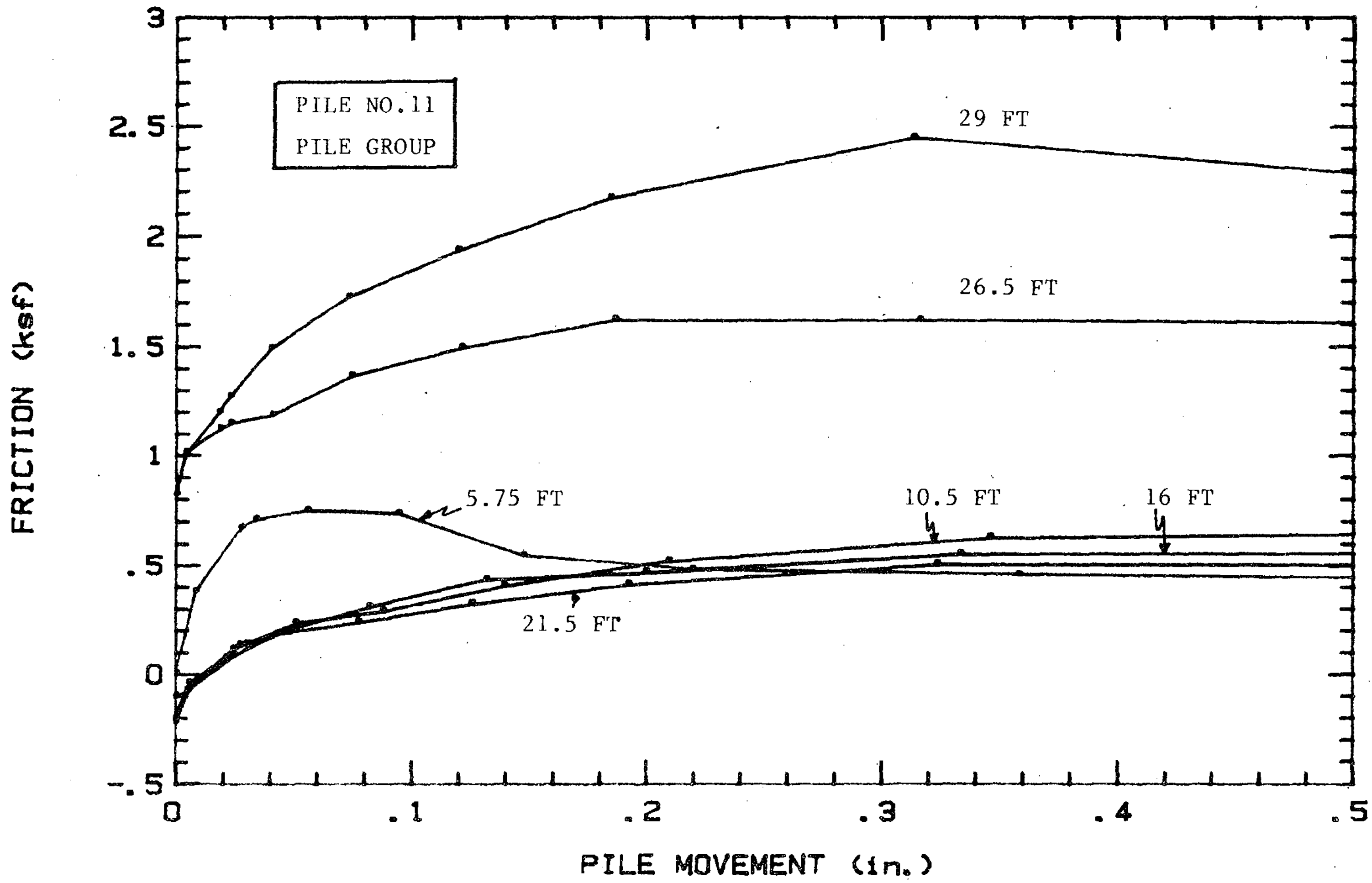


FIG.32a- Friction vs. Movement Curves for Pile No.11 in Group.

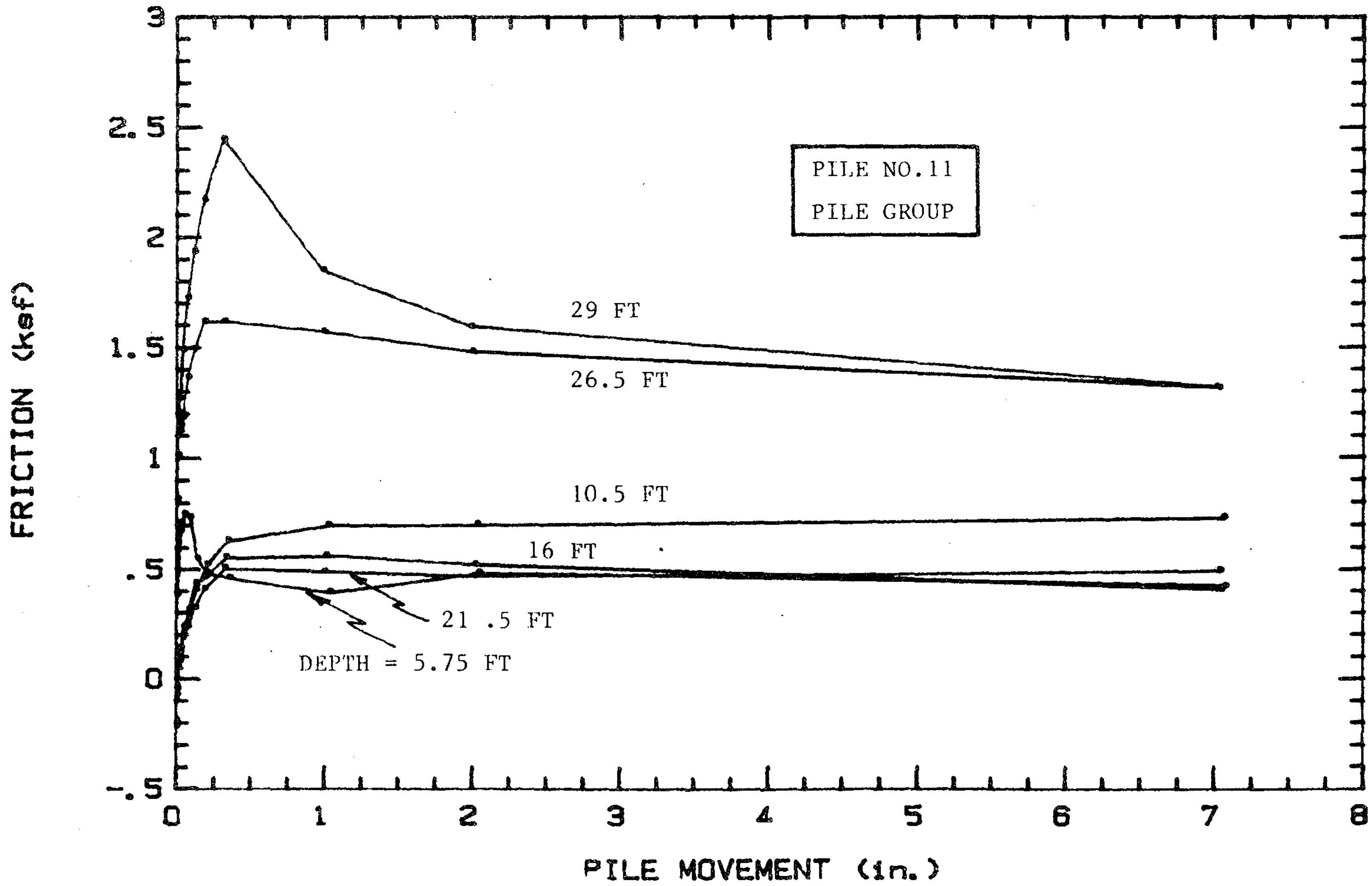


FIG.32b- Friction vs. Movement Curves for Pile No.11 in Group.

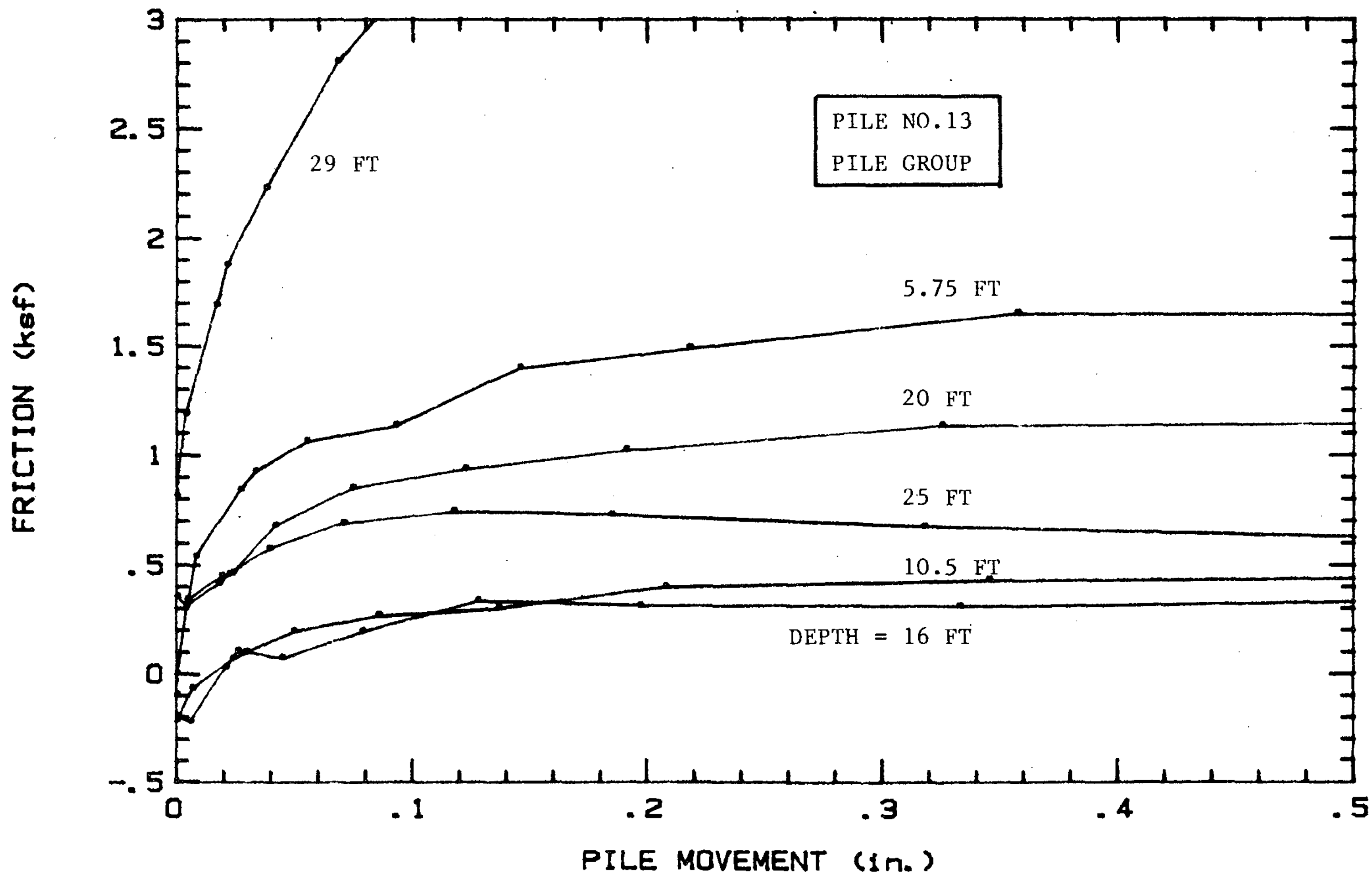


FIG.33a- Friction vs. Movement Curves for Pile No.13 in Group.

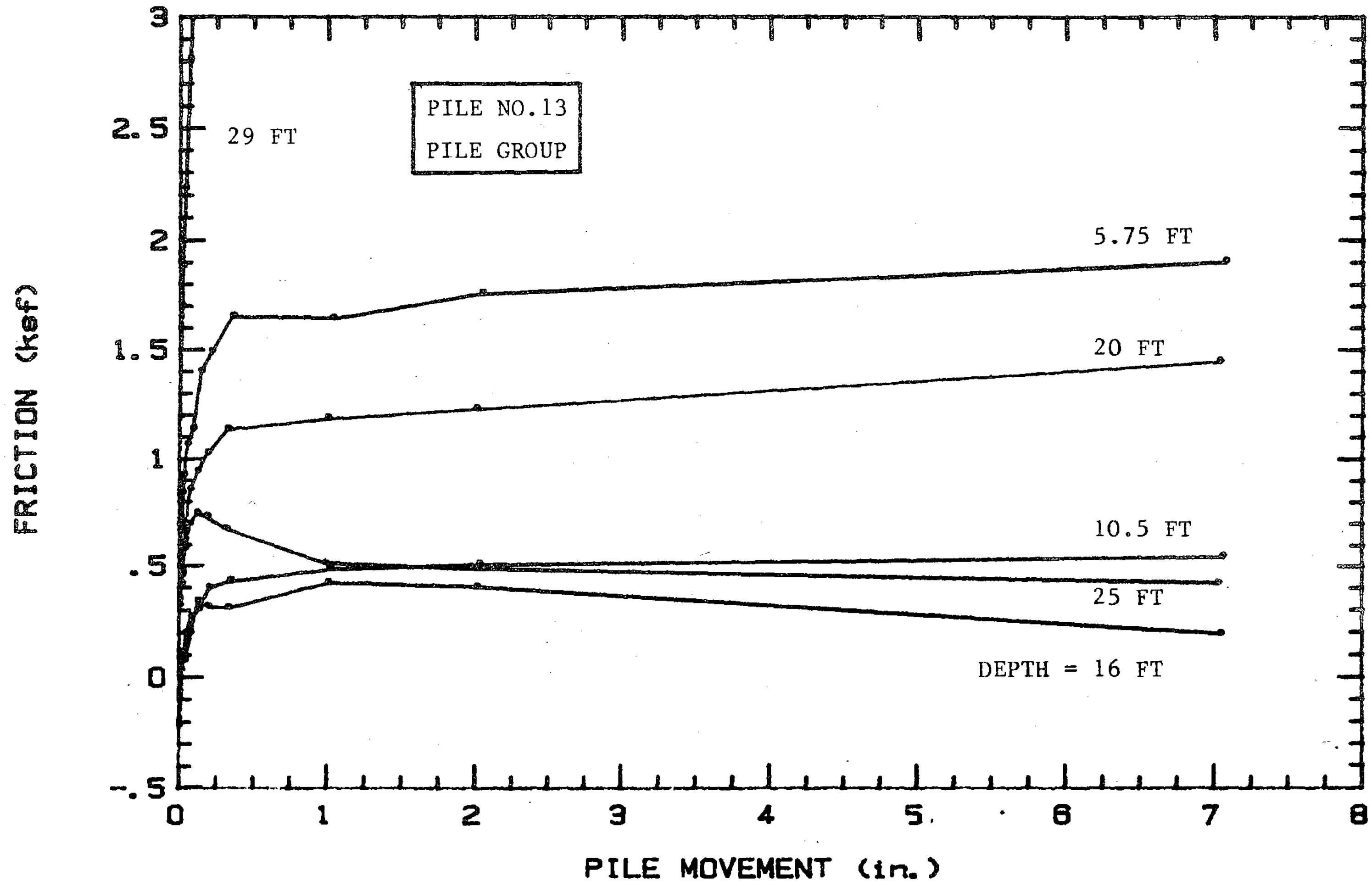


FIG.33b- Friction vs. Movement Curves for Pile No.13 in Group.

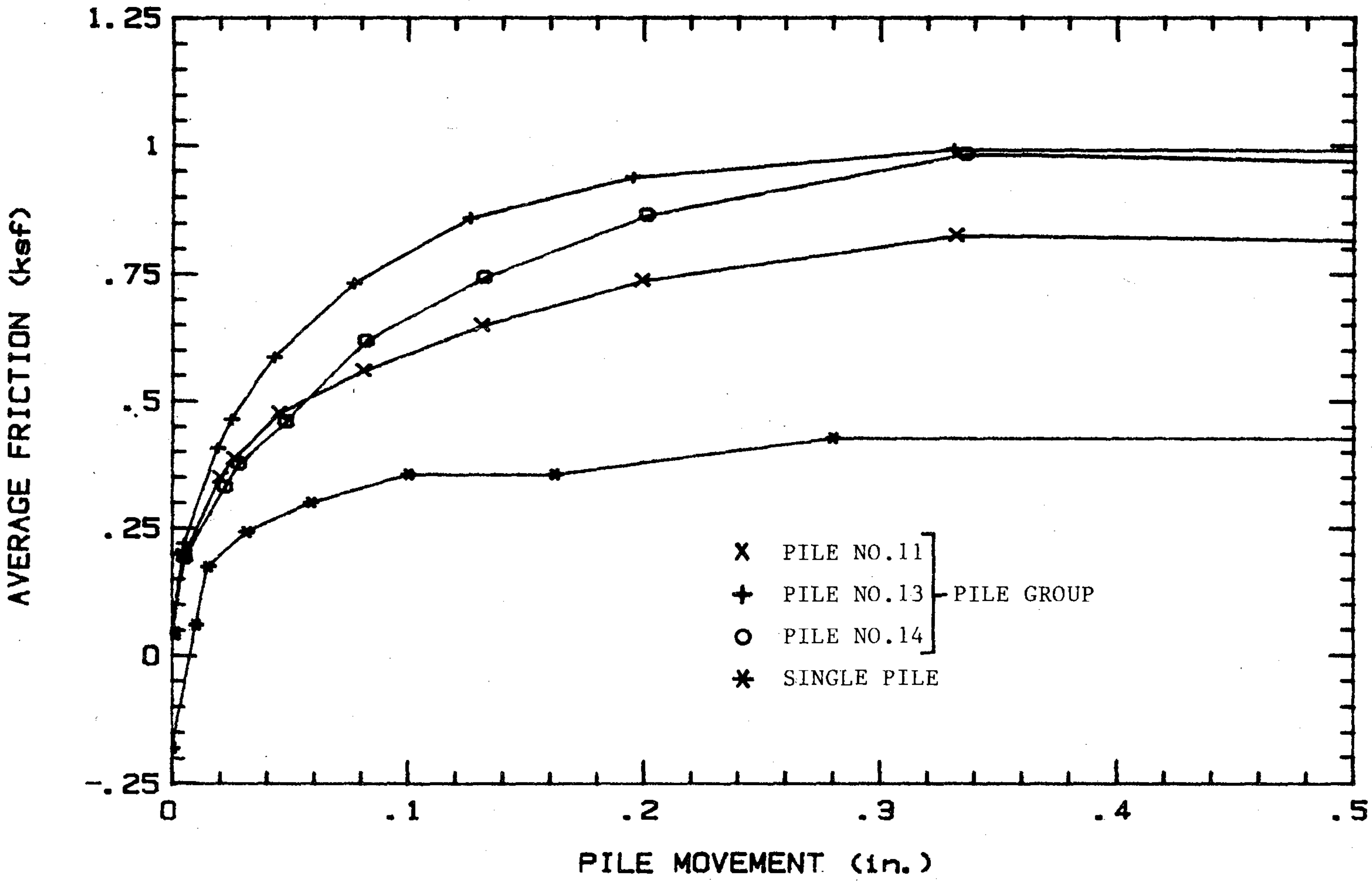


FIG.34a- Comparison of Friction-Movement Curves for the Single Pile and for the Pile Group.

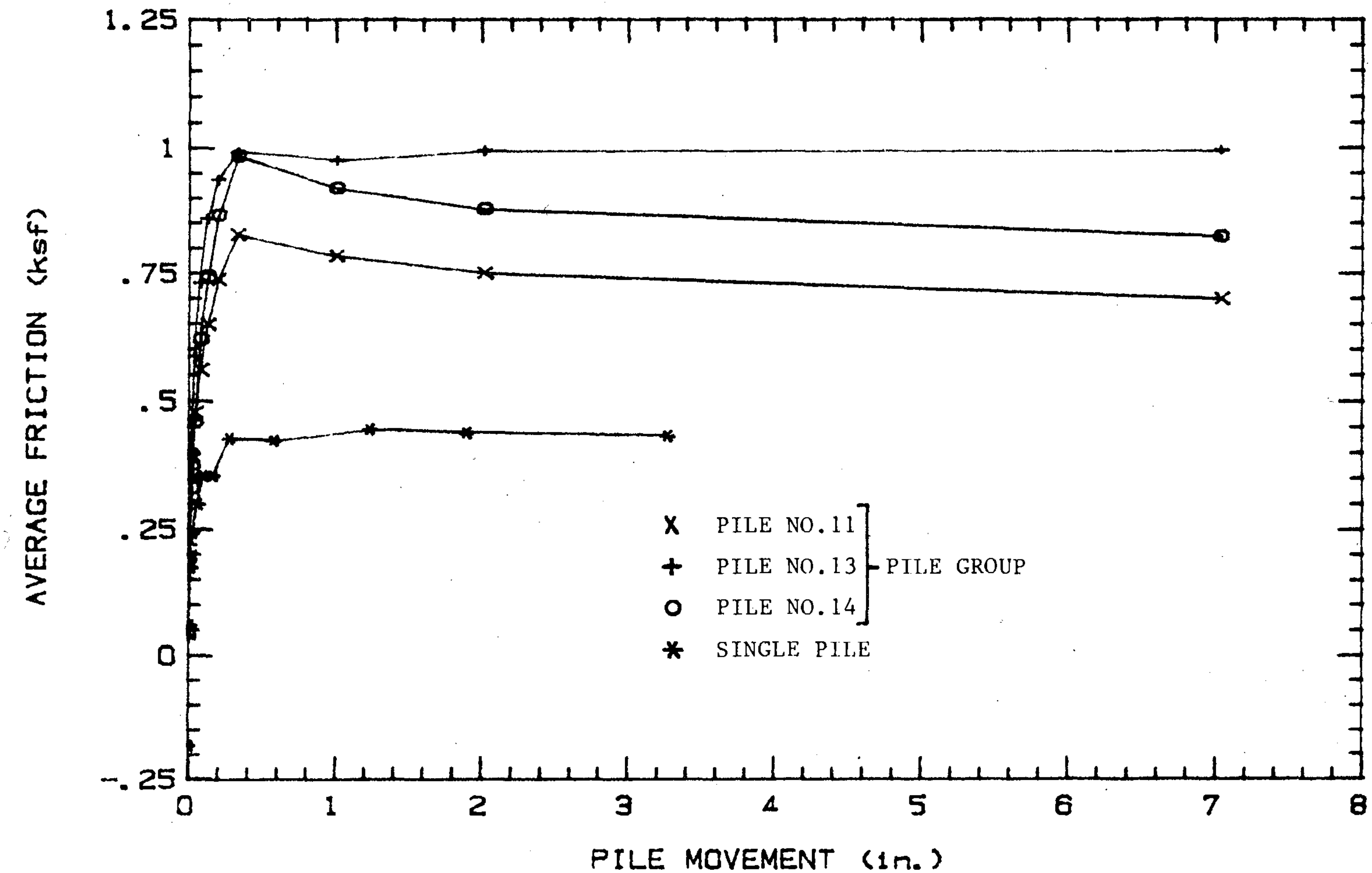


FIG.34b- Comparison of Friction-Movement Curves for the Single Pile and for the Pile Group.

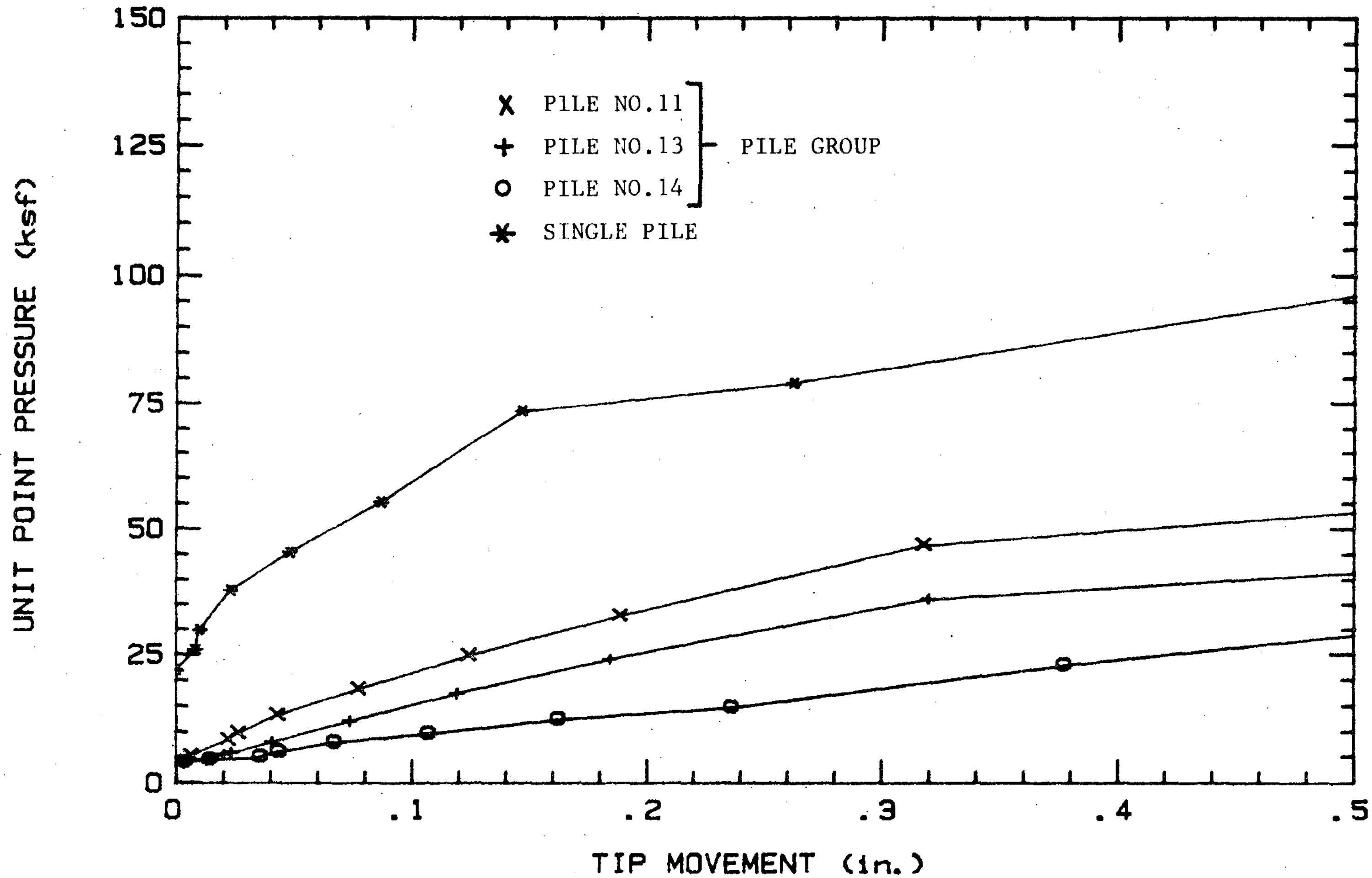


FIG.35a- Point Pressure vs. Movement: Comparison Between the Single Pile and the Pile Group.

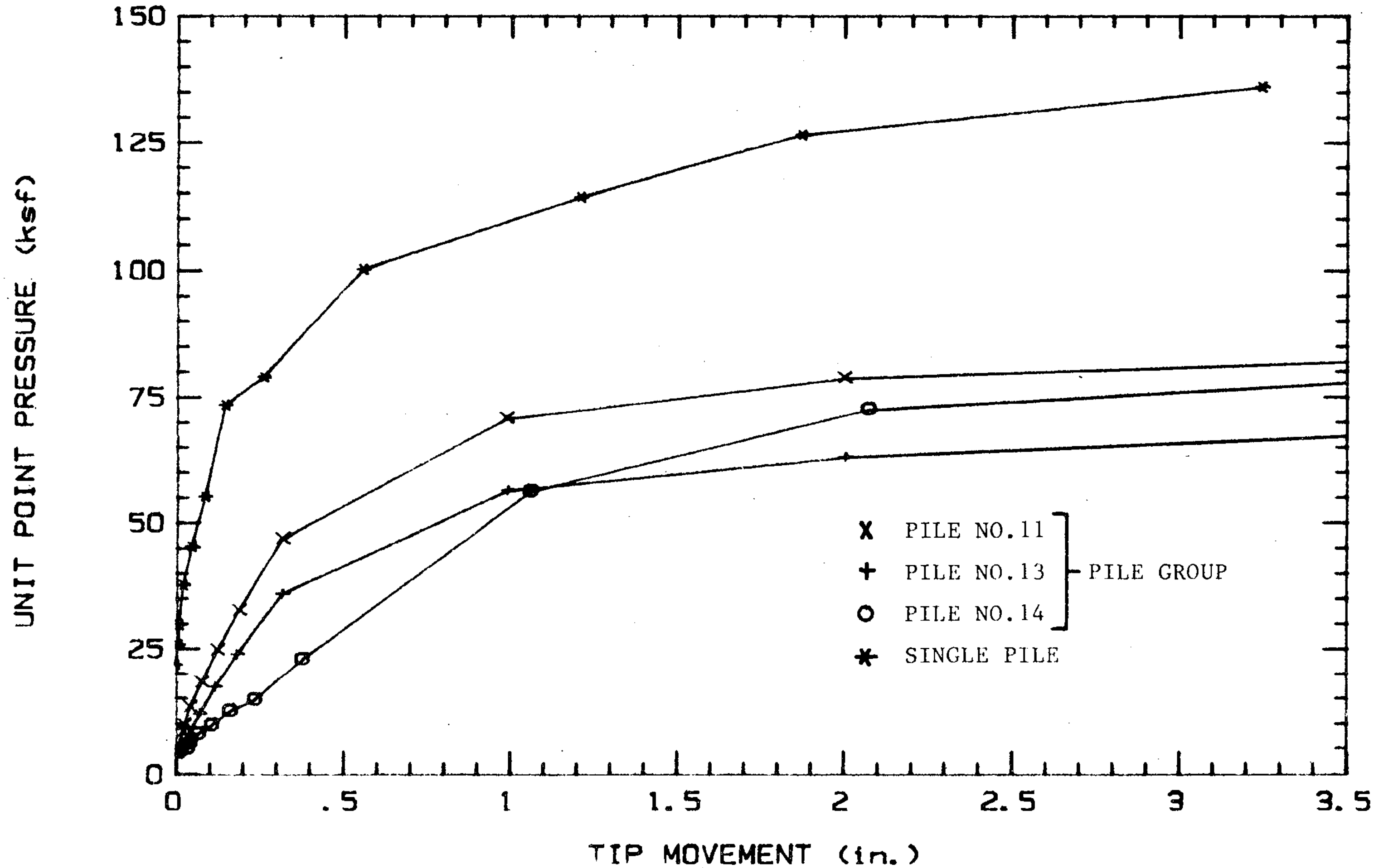


FIG.35b- Point Pressure vs. Movement: Comparison Between the Single Pile and the Pile Group.

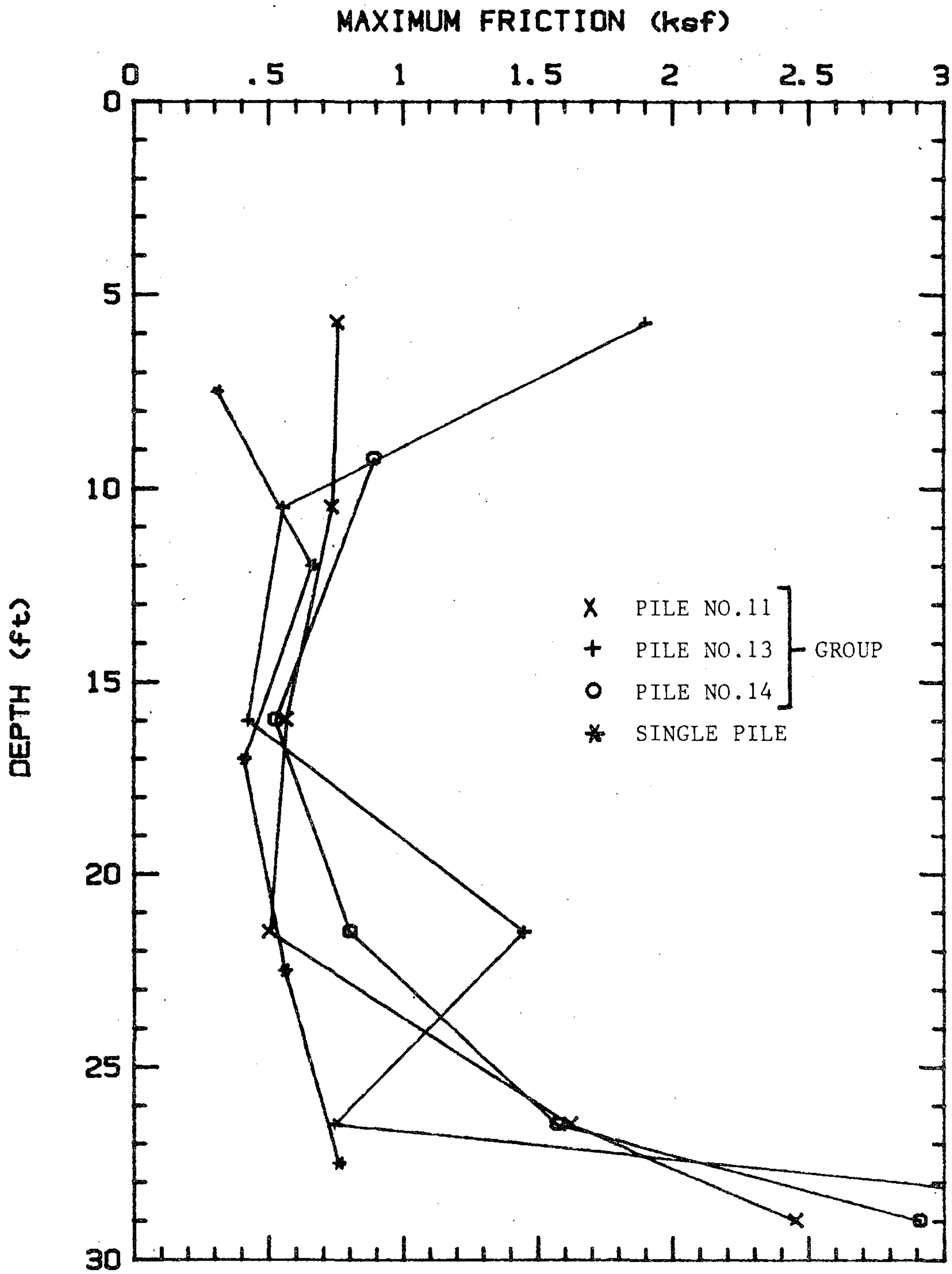


FIG.36- Comparison of Maximum Friction on Single Pile and on Pile Group.

account of residual stresses. The horizontal earth pressure coefficients k obtained from equation 11 are also shown on Figure 21. The average k value was 1.71 and is to be compared with the average value of k for the single pile which was 0.82 and the average k_o value which was 0.85.

Comparisons can be made between the predictions (Table 3 and Figure 11) and the measured results (Table 3 and Figure 22). The efficiency of the group was overpredicted since instead of 1.5 the efficiency was measured to be 0.99. As a result, the ultimate load for the group was best predicted by methods which underpredicted the ultimate load of the single pile (Table 2 and 3); these are the LPC cone penetrometer method and the LPC pressuremeter methods (Bustamante, Gianeselli, 1983 and 1982). The settlement ratio by the Vesic's formula (Eq. 3) was larger than the measured settlement ratio (2.06 versus 1.29). The use of the PILPG2 program (O'Neill et al, 1981) allowed to develop the complete load settlement curve for the group by 1 SPT method, 1 CPT method and 1 PMT method. Note that in PILGP2 the efficiency e is always 1.0. At working loads (250 kips on Figure 11), all three methods underpredicted the stiffness of the group (load/settlement) by a factor varying from 1.49 to 2.3. Therefore the soil moduli chosen for the elastic interaction between the piles and for the initial slopes of the load transfer curves were significantly too low.

Conclusions

As pointed out in the introduction, the analysis of the data is partially complete and much more can be done in order to take full advantage of the data. Nevertheless this article allows to reach some conclusions:

1. Residual stresses must be accounted for when analyzing instrumented load tests on single piles or pile groups. The single pile had a residual point load of 13.8 kips or 17% of the ultimate point resistance. The piles in the group had much smaller residual point loads (average 2.3 kips) but had significant residual friction stresses.

2. The plunging load for the single pile was 113.5 kips while the plunging load for the 5 pile group was 561 kips. The efficiency was

0.99.

3. At the plunging load the best estimate of friction load carried by the single pile was 33 kips while the average friction load as measured on 3 piles in the group was 60.3 kips. The efficiency on the friction load was 1.83.

4. At the plunging load the best estimate of point load for the single pile was 80.5 kips while the average point load as measured on 3 piles in the group was 54.3 kips. The efficiency on the point load was 0.67.

5. The concept of critical depth on ultimate friction values does not appear to exist if the residual friction stresses are taken into account. If the residual friction stresses are ignored a break appears in the ultimate friction profile consistent with the concept of critical depth.

6. The coefficient of horizontal pressure, k, was calculated from the ultimate friction values 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, 1.71 for the piles in the group. The average k_o value measured by PBPMT and SBPMT was 0.85.

7. The settlement of the single pile at half the plunging load (55 kips) was 0.107 in. while the settlement of the group at 5 times the load on the single pile (275 kips) was 0.138 in. The settlements at working loads are very small and the settlement ratio is 1.29.

8. At half the plunging load the single pile carried 41% of the load in friction and 59% in point resistance. Comparatively, at half the plunging load the pile group carried 88% of the load in friction and only 12% in point resistance.

9. The movement necessary to mobilize the maximum friction resistance was approximately 0.3 in. while an average of 1.7 in. was necessary to mobilize 90% of the point load. The shape of the friction curves is well approximated by an elastic-plastic model while a hyperbolic model fits better the point resistance curve.

10. The plunging load of the single pile and the corresponding friction load and point load were best predicted by 2 cone penetrometer methods (DeRuiter/Beringen, 1979, Schmertmann, 1978). The two SPT

methods which performed best were Coyle-Castello (1981) and Nordlund (1963). The PDA method also worked well.

11. The efficiency of the group was overpredicted by current methods. It appears that the common practice of using an efficiency of 1 is warranted.

12. The settlement ratio was overpredicted. However, both the predicted and measured settlement at working load for the single pile and for the group were very small.

13. The PILGP2 program was convenient in allowing to develop the complete load settlement curve for the group.

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PREDICTIONS OF THE BEHAVIOR OF AXIALLY LOADED CONTROL PILE AND FIVE-PILE GROUP AT HUNTER'S POINT TEST SITE

Introduction

The performance of high-quality, full-scale tests on components of engineered structures can afford the engineering profession the honest opportunity to examine the validity of design methods when "Class A" (before-the-fact) predictions can be made of the behavior of the components. The Federal Highway Administration has provided a vehicle for such an exercise by inviting ten engineers to participate in the prediction of the load capacity, load distribution and load-settlement behavior of a single, axially-loaded control pile in a deposit of sand and an axially loaded group of five, stiffly capped piles of the same design as the control pile. The predictors were free to analyze the control pile and pile group using any method or methods of their choosing. The detailed predictions are contained in Vol. 2 of the 1986 Pile Group Prediction Symposium.

General information provided the predictors, including the conditions of the test, is summarized on Fig. 1. The test site was at Hunter's Point in San Francisco, California. The geotechnical and geometric data listed on that figure were given to the predictors prior to their making their predictions. It is significant that the soil at the test site was a uniformly graded (medium-fine) sand placed as a hydraulic fill and that the near-surface soil was quite dense. A detailed description and commentary on the geotechnical test data is provided elsewhere in the Proceedings. Once the pile driving records and control pile load test data were received, they were furnished to the predictors, and the predictors were permitted to change their group predictions. Only two predictors did so. The actual load tests

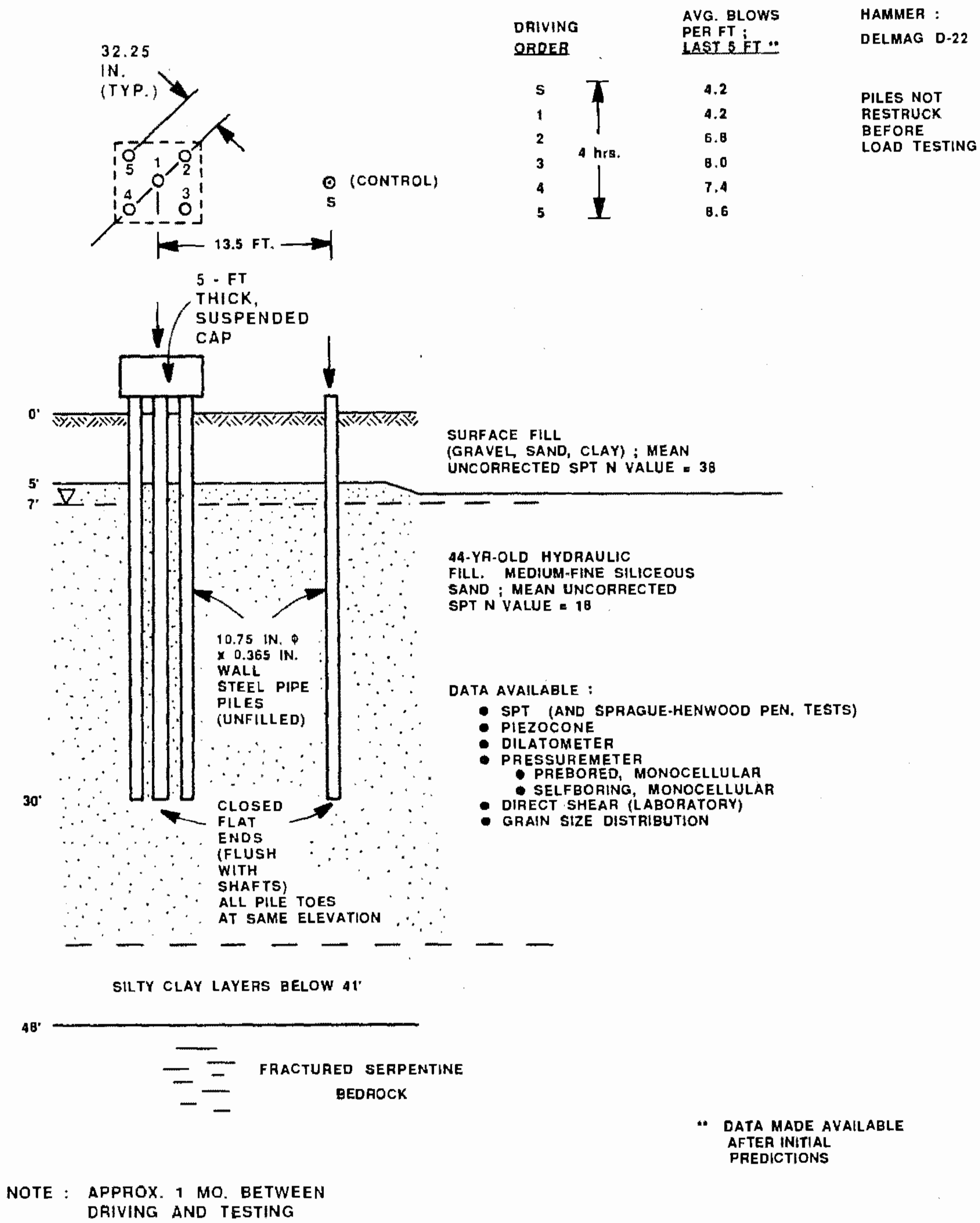

FIG. 1. GENERAL TEST CONDITIONS

were conducted in late May - early June, 1986, after the initial predictions had been completed.

Two pieces of apparently ambiguous data were given to the predictors: (a) The group spacing (Fig. 1) was shown as three pile diameters (32.25 in.) on one set of drawings and 3 ft (36 in.) on another set. The former value is correct, although some predictors assumed the latter. (b) Negative unit sleeve resistance (f_s) values were reported in one cone penetrometer sounding. It was later reported that these values occurred due to damage to the friction sleeve. None of the predictors ultimately employed these negative f_s values.

This paper attempts to summarize the methods used by the predictors, both to assess soil properties at the test site and to assess the behavior of the control pile and pile group. Predicted performances (i.e., "solutions") are also summarized. The author incorporates the latter summaries with considerable hesitation, because the close correlation of a predicted value and a measured value may be totally fortuitious and may therefore serve to propagate inappropriate design methods. The reader should therefore realize that there were really no "right answers" in this exercise and view it critically as a commentary on approaches to the design of driven piles in sand. Above all, it was not a contest to select the most accurate predictor.

Those invited by the Federal Highway Administration to make predictions were:

Umakant Dash, Pennsylvania DOT
Richard Engel, Ohio DOT
Bengt Fellenius, University of Ottawa
Robert Hood, New Mexico State Highway Department
Ashton Lawler, Virginia Department of Highways
Robert Lukas, STS Consultants, Ltd.
Vito Nacci, University of Rhode Island (Ret.)
R.L. Nordlund, Spencer, White and Prentis, Inc.
John Schmertmann, Schmertmann and Crapps, Inc. (with Umakant Dash)
Ray Schnore, New York State DOT
Suneel Vanikar, Federal Highway Administration

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General Commentary

The design methods described by the predictors can all be described as "rational," "intuitive" or "rational-intuitive," and, above all, practical. No strictly "theoretical" solutions were followed (e.g., finite element, boundary integral), possibly because the predictors all concluded that no totally appropriate and comprehensive theoretical method exists to solve the problem and possibly because the composition of the group of predictors (5 state DOT engineers, 1 FHWA engineer, 3 consultants, and 2 consultant/professors) preempted the use of research-level approaches. Thus, as with any problem in applied soil mechanics, a large measure of engineering judgment was applied by all of the predictors. All of the predictors gave the problem considerable thought and developed non-superficial solutions.

The problem of predicting the behavior of a pile group in sand is complicated by the fact that prediction of the behavior of a single pile in sand is far from a "solved problem." For example, Dennis and Olson (1983), using a large data base of pile load tests, demonstrated that the application of one popular method of static pile capacity prediction (American Petroleum Institute, 1984) resulted in a probable error in predicted capacity greater than one-half of the mean computed pile capacity. In clay soils, on the other hand, they found that errors in static capacity calculations were considerably smaller than those in sand. Several factors contribute to this effect:

(1) Prevailing practice in the USA (e.g., API) uses correlative methods for computing both shaft and toe capacities based on soil unit weight and angle of internal friction, usually taking the form of an equation similar to Eq. 1 (for prismatic piles):

$$
\begin{aligned}
Q_U &= Q_S + Q_T \\
&= \Sigma\,[(K\sigma'_v \tan\delta)\ C\ \Delta z] + \sigma'_{vT}\ A_T\ N_q \qquad \ldots \quad (1)
\end{aligned}
$$

where

Q_U = pile capacity (FS = 1)

Q_S = shaft capacity

Q_T = toe capacity,

K = earth pressure coefficient,

δ = angle of pile-soil friction,

σ'_v = vertical effective stress (T denoting toe level),

N_q = bearing capacity factor,

C = pile circumference,

Δz = incremental depth, and

A_T = area of pile toe.

K, δ and N_q, which are often expressed as functions of ϕ, are usually derived or modified through correlations with actual load test results. This leads to inaccuracies in the parameters (Coyle and Castello, 1981). For example, measured shaft and toe capacities may not be defined by consistent procedures among the tests used in developing parameter evaluations, due to the nonstandardization of the definition of failure and its effect on the derived capacity of piles that do not "plunge" (i.e., most piles in sand) (Fellenius, 1975). Furthermore, careful account has traditionally not been taken of the residual loads that exist in piles after driving, which tends to indicate shaft capacities that are too high and toe capacities that are too low relative to their true (absolute) values (Holloway et al, 1978, Briaud et al, 1985). Finally the values of ϕ at the test site are ascertained (usually) by inexact methods (e.g., the standard penetration test). These effects combine to make the design parameters themselves relatively inaccurate. The methods furthermore tend to be somewhat ambiguous in their application concerning whether the computed unit load transfer values are meant to be true values or are apparent values that exist relative to an in-place zero condition.

(2) Prevailing practice in the USA also therefore tends to use inputs for the design methods that involve correlating the results of some geotechnical field test at the site of the construction (e.g., SPT), to unit weight and angle of internal friction, which compounds the inaccuracy of the capacity prediction, since errors, sometimes significant ones, arise in these tests.

(3) Volume-change characteristics of the soil, essential in describing effective stress states in the soil after pile driving and pile-load deformation behavior, are routinely neglected, both in the data-gathering phase and the analysis phase of the project.

These difficulties are further compounded when predictions are extended to pile groups, where knowledge is almost exclusively empirical and, to an extent, conflicting. For example, Kishida (1967) suggested that driving piles in a group tended to densify relatively loose sand below the pile toes and thus increase capacities of piles in a group principally by increasing toe capacity. This effect may be more significant for the latter piles that are driven (unless the earlier piles driven are restruck). If the piles are driven in order from the inside of the group to the outside and all piles are driven to the same toe elevation, the perimeter piles may therefore carry greater loads than the interior piles and than an isolated pile. The driving resistances on Fig. 1 suggest this sort of behavior for the Hunter's Point group. On the other hand, Vesic (1969) has shown that in large models (in which all piles were inserted simultnaeously) the shaft capacities increase relative to those of single piles due to increased lateral stresses in the soil, whereas toe capacities do not. This phenomenon may suggest that greater loads would be carried by interior piles, or that loads would at least be more evenly distributed among the piles than predicted by models that consider continuum effects (Poulos and Davis, 1980).

Residual stresses in pile groups have been found to be lower than those in single piles (e.g., O'Neill et al, 1982), which leads to larger initial settlements in the group than would otherwise occur and to higher apparent toe capacities and lower apparent shaft capacities than exist in an isolated single pile. Only one predictor (Nordlund) explicitly considered the control pile and group piles to be residually stressed before loading in order to develop predicted load-settlement curves, although he judged that the residual stresses would not be significant in the short and relatively stiff piles at the test site. Another predictor (Fellenius) explicitly stated that he predicted true toe and shaft resistances (in the control pile) because true toe and shaft loads would be measured. Other predictors were not specific regarding whether they were predicting true or apparent shaft and toe loads.

At the Hunter's Point site the difficulty exists that the soil is uniform hydraulic fill and, as pointed out by Fellenius, that local zones of collapsing sand may therefore exist. If such zones were present below the pile toes, volume reductions produced beneath in-place piles by driving adjacent piles could cause loosening of soil beneath the toes of piles already driven and so produce reductions in toe stiffness if the piles remained "hanging" in the denser near-surface soil. On the other hand, compaction of soil around in-place piles might drag such piles down and possibly increase toe capacity. Since no explicit volume change characteristics of the soil were provided to the predictors, it was necessary to make some tacit assumption (consciously or subconsciously) concerning the likely volume change characteristics of the soil during pile insertion.

The effect of suspending the pile cap above the soil (Fig. 1) may also make empirical rules concerning group efficiency problematical for these tests, since most such rules have been developed from model tests in which the loading cap has contacted the soil.

In summary, a number of significant questions arise in predicting the control pile and group behavior, first, concerning the application of general single-pile design models and, second, concerning the physical effects that must be considered in extending such models to group behavior. It is hoped that careful analysis of the various predictions vis-a-vis the measured behavior will clarify such questions for situations similar to the specific case considered here.

Loading Procedures

The load test on the single, control pile was very capably carried out by Eric Ng and his colleagues at Geo/Resource Consultants, Inc., on May 23, 1986. The load test on the group was conducted by the same research team on June 6, 1986. The research team and the FHWA are to be commended for planning and executing this study, which when completed will undoubtedly provide a benchmark addition to our body of knowledge concerning pile behavior.

The control pile and the group were both loaded using the maintained load method, in which equal increments of load of about 10 per cent of the anticipated capacity were applied monotonically and maintained for 30 min. Settlement readings were taken continually during the hold period. For purposes of this paper, end-of-increment settlement values have been plotted against applied load, which makes the load-settlement curves somewhat more flexible than they would be if other settlement values had been plotted. The interpretation of failure load for the control pile was made by the author in four ways: (1) method recommended by Davisson (1972); (2) load at which the rate of settlement first reached 0.05 in./ton (which is the failure load criterion recommended by Nordlund, whose method is recommended by the FHWA (Cheney and Chassie, 1982)), (3) load corresponding to a head settlement of 10 per cent of the pile diameter and (4) maximum load applied in the test. These methods are admittedly arbitrary, and other established methods could have been applied (Fellenius, 1975). However, they represent a reasonable spectrum of values and coincide with failure definitions used explicitly or implicitly by most predictors. Hence, considering the interpretation of the load-settlement curve and of the "failure point" on that curve, a family of "measured values" corresponding to failure load and settlement at failure will be tabulated, as opposed to single, unique values.

No well-established methods are available, however, for establishing the failure load for a group of piles. Since failure of a pile group involves compression of a significant mass of soil or rock below the pile toes and may involve progressive failure of the piles within the group, the author has chosen to interpret group failure load as the load at which the creep settlement (settlement in the last one-half time increment (15 min)) of each loading period increases suddenly.

Methods Used in Analysis of Control Pile

The methods used to analyze control pile and group behavior will be summarized prior to describing the predictors' methods of interpreting geotechnical data.

Several predictors computed capacities, load-settlement and load distribution behavior for the control pile by several methods and then selected the solution that appeared to be the most realistic one as their primary solution. Another predictor made multiple analyses and recorded his solution as a numerical average of all methods. While the author advocates making multiple analyses for design purposes, describing all of the secondary methods of analysis considered by the predictors in this summary would prove unduly cumbersome. Therefore, only the primary methods selected by the predictors will be described. The reader is referred to the papers of the individual predictors for descriptions of secondary methods.

The prediction methods used for assessment of capacity and load-settlement behavior, as well as each predictor's definition of failure, are tabulated in Table 1. The majority of the predictors (e.g., all but one of the state DOT engineers) used some form of the capacity prediction method that has been suggested by the FHWA (Cheney and Chassie, 1982). The predictions are listed in an order that is dependent upon how far the capacity prediction method is perceived to deviate from the recommended FHWA procedure. Note that the final prediction (Dash/Schmertmann) has two distinctly different procedures.

The FHWA capacity prediction method utilizes Nordlund's (1979) static procedure for predicting shaft capacity and Thurman's (1964) method for computing toe capacity. The two resulting resistance values are directly added. In Nordlund's procedure (if the pile is prismatic), the unit load transfer f is given by

$$f = K_\delta \sigma'_v \sin\delta , \qquad \ldots (2)$$

where K_δ is dependent on an interpreted value of ϕ, and the pile volume, and δ (angle of wall friction) is dependent on ϕ, the pile volume and pile type. Unit toe capacity q is obtained from

$$q = \alpha N_q \sigma'_{vT} , \qquad \ldots (3)$$

where N_q is a bearing capacity factor dependent on ϕ (only) and α is a reduction factor that depends on ϕ and relative toe depth. Thurman's N_q factors are identical to the factors proposed by Berezantzev et al (1961), and his α factors are identical to Berezantzev's for shallow penetrations. σ'_{vT} is the value of vertical effective stress at the level of the toe. The key soil parameters that must be evaluated are therefore ϕ, γ (soil unit weight) and u (pore water pressure).

Nordlund's method is similar to the FHWA method, except that the Berezantzev et al (1961) bearing capacity equation common to both methods contains relative depth correction factors which have slightly less dependency on relative toe depth than do the Thurman factors. Nordlund's method, as understood by the author, predicts the load on the pile at the point that the rate of settlement first becomes 0.05

Table 1. Analysis Methods for Control Pile

Predictor	Capacity Prediction Method	Predictor's Definition of Failure	Settlement-at-Failure Prediction Method	Load-Settlement Analysis Method
Hood	FHWA	0.05 in./ton	Davisson*	Estimated from failure settlement
Lawler	FHWA	Plunging*	Vesic w/ C-factors	Avg. of linear load-settlement and superposition of toe and shaft load-settlement from Vesic with C-factors with max. shaft load at w_{ps}
Vanikar	FHWA	Plunging*	Vesic w/ C-factors	Vesic with C-factors assuming 50% of shaft capa- and 50% of toe capacity mobilized at one-half ulti-mate load. Settle-ment at one-third ultimate estimated
Engel	FHWA but with avg. of Thurman and min.-path CPT for toe capacity	Deformation Hardening	Davisson*	Superposition of toe and shaft load-settlement from Vesic with elastic moduli with max. shaft load at 0.05 in. deflection
Nordlund	Nordlund (0 shaft resist. in upper fill)	0.05 in./ton	Double tangent	Hand-drawn curve between the two tangents
Schnore	Nordlund	0.05 in./ton	Davisson*	Vesic with C-factors, assum-ing 60% of shaft capacity mobilized at one-third ulti-mate load and full shaft capacity mobilized at one-half ultimate load

Table 1. Analysis Methods for Control Pile (Continued)

Predictor	Capacity Prediction Method	Predictor's Definition of Failure	Settlement-at-Failure Prediction Method	Load-Settlement Analysis Method
Nacci	Static method w/K = 1.25, $\delta = 0.8\ \phi$, Thurman's toe capacity procedure, crit. depth = 12.5 diameters	Deformation Hardening	Davisson	Elasto-plastic t-z model (checked at failure w/continuum solution)
Fellenius	β for shaft capacity; toe stress at failure approx. equal to cone q_c at toe elevation. Checked (fortuitously) by direct correlation w/SPT	Plunging	Estimated based on toe failure deflection	-
Lukas	Avg. of 11 methods (4 CPT, 2 PMT, 1 N, 4 Static)	Deformation Hardening	Avg. of Vesic w/ C-factors, Davisson, Canadian Foundation Manual (1985)	Can. Fdn. Manual with variable elastic shortening tempered w/Vesic (1969) and Meyerhof (1976) at ultimate and one-third ultimate
Dash/ Schmertmann	Dash - CPT Method simplified from Schmertmann (1978) Schmertmann- Static method using K and ϕ from DMT and min.-path toe capacity using $N_{\gamma q}$ factors (Program "PCAP")	Deformation Hardening	Dash - 0.1 x pile diameter Schmertmann- elastic-plastic shaft load-sett. superimposed on elastic tip load-sett. carried to computed capacity	- Not documented

*Not consistent with capacity method in a rigorous sense, since Nordlund's method (used by FHWA) defines failure at settlement rate of 0.05 in./ton.

NOTE: FHWA method uses Nordlund (1979) for shaft capacity and Thurman (1964) for toe capacity.

in./ton. Technically, therefore, the definition of failure consistent with the Nordlund or FHWA method would be that point on the load-settlement curve and not the point defined by the Davisson capacity criterion, which is generally conservative.

One predictor (Nacci) used another static shaft capacity prediction procedure similar to Nordlund's along with Thurman's toe capacity method but explicitly limited σ_v' in all calculations to the value present at a depth of 12.5 pile diameters in order to account for a "critical depth" phemonenon. For this prediction it should be more appropriate to compare the shaft and toe capacities to those based on post-driving zeroes.

Two predictors (Dash and Fellenius) utilized the data from the electronic cone penetrometers to make their primary pile capacity computations. Dash used a simplified version of the formal procedure described by Schmertmann (1978), while Fellenius used the rate of increase in f_s with depth to deduce a value for β (= f/σ_v'), allowing for a twofold increase in β measured by cone tests because of soil compaction due to pile driving. Engle used the minimum-path toe capacity (Schmertmann, 1978) obtained from the cone q_c records averaged with the capacity computed from Thurman's theory.

Schmertmann used a static prediction method similar to Eq. 1 for shaft resistance in which K and δ (= ϕ) were evaluated directly from dilatometer tests. K was taken as a weighted average of K_o and K_D. Toe capacity was predicted using the $N_{\gamma q}$ factor, rather than N_q, in which

$$q = B\gamma N_{\gamma q} \qquad \ldots (4)$$

Where B = pile toe diameter. Below some critical depth (less than the pile penetration in the Hunter's Point tests), this factor, and hence q, becomes dependent only on ϕ. This approach, while ultimately not producing a value of toe capacity extremely close to that which was measured, has special merit among the static methods, because even though ϕ is used in the calculations, it is obtained from the site-specific DMT test through back-calculation using the same equation for $N_{\gamma q}$ that was used to compute the pile toe capacity.

defined.) This statement is not offered as a criticism but merely as a comment on how knowledgeable practitioners approach the design of piles.

Methods Used in Analysis of Group

Table 2 summarizes the procedures used to analyze the behavior of the five-pile group. Predictors were given the opportunity to modify their predictions once they had reviewed the results of the tests on the control pile. Two predictors (Engel and Fellenius) made changes. In those cases the original predictions are given here, and brief comments are made on the changes in the section "Addenda to Predictions." The methods of analysis and solutions contained herein, therefore, can be considered truly "Class A."

Methods used to predict group capacity and load-settlement behavior are difficult to classify; therefore, Table 2 is extensively footnoted. Since the effect of pile installation is so critical to the behavior of the group, each predictor had to employ considerable judgment in extrapolating control pile behavior to group pile behavior. The judgment varied from assuming that all piles would behave in a manner completely identical to the control pile (Vanikar), due to the wide spacing of the group piles, to increasing ϕ and/or K in a reasonably rational manner for the soil adjacent to the shafts of the group piles (Lawler, Nordlund, Schnore, and Nacci). Only one predictor deduced that the toe capacities would increase significnatly in the group (Schnore). That was accomplished by computing additional vertical effective stresses at the level of the pile toes due to shaft loads transferred to the soil above the toes. All other predictors appeared to follow Vesic's (1969) suggestion that "toe efficiency" in pile groups in sand is essentially 1.0.

Other predictors empirically increased shaft efficiencies by referring to charts relating measured efficiency in other group tests in sand to spacing or group size (e.g., O'Neill, 1983) and applying most or all of the capacity increase to shaft load. Efficiency formulae, relating group efficiency to pile group geometry, were used in part by two predictors, although the applicability of such formulae to the condition of this particular test is questionable in the author's opinion.

Table 2. Analysis Methods for Five-Pile Group

Predictor	Capacity Prediction Method	Predictor's Definition of Failure	Settlement-at-Failure Prediction Method	Load-Settlement Analysis Method	Load Distribution/ Load Transfer Analysis Method
Hood	Geometric efficiency formula	"Progressive" (block vs. individual pile not stated)	Assumed same settlement as control pile at equal loads per pile	Assumed same settlement as control pile at equal loads	Obtained directly from capacity prediction method
Lawler	Increased ϕ due to driving adjacent piles	Plunging of individual piles	Vesic method	Vesic method	Obtained directly from capacity prediction method
Vanikar	Deduced efficiency = 1.0, since S/B $\geq$ 3	Plunging of individual piles	Meyerhof method	Estimated settlements for loads less than ultimate	All piles have equal loads; same load transfer as in control pile
Engel	Deduced efficiency = 1.3 from published data. Efficiency factor applied to control pile plunging load, not failure load	Plunging of individual piles	0.05 x diameter of equivalent cylindrical pier	Equivalent pier unit load transfer curve method	Scott method for load distribution. Assumed further that the 30% capacity increase was distributed: 25% to the shafts and 5% to the toes

Table 2. Analysis Methods for Five-Pile Group (Continued)

Predictor	Capacity Prediction Method	Predictor's Definition of Failure	Settlement-at-Failure Prediction Method	Load-Settlement Analysis Method	Load Distribution/ Load Transfer Analysis Method
Nordlund	Increased lateral effective stress due to driving adjacent piles	Deformation hardening of individual piles with tilting	Double tangent method (based on avg. load per pile)	Hand-drawn curve between the two tangents	Obtained directly from capacity prediction method, adding 2K to center pile and 1K to 2nd and 3rd piles driven (corner piles) as shaft resistance in upper fill
Schnore	Increased ϕ (shaft) and increased vertical effective stress	Failure of individual piles	Elastic influence factors for group as a large loaded area	Elastic influence factors for group as a large loaded area	Obtained directly from capacity prediction method
Nacci	Increased ϕ due to driving adjacent piles and also increased K due to increase in ϕ	Ductile failure of individual piles	Two-pile-interaction factor method	Not documented	Obtained directly from capacity prediction method
Fellenius		No quantitative predictions for five-pile group			

Table 2. Analysis Methods for Five-Pile Group (Continued)

Predictor	Capacity Prediction Method	Predictor's Definition of Failure	Settlement-at-Failure Prediction Method	Load-Settlement Analysis Method	Load Distribution/ Load Transfer Analysis Method
Lukas	Deduced efficiency = 1.25 for S/B = 3 from published data	Analogous to failure of "deep footing"	Avg. of Vesic and Meyerhof methods	Estimated	Estimated center pile to carry 120% of avg. load and corner piles to carry 95% of avg. load. All additional load assigned to shaft resistance.
Dash	Deduced efficiency = 1.0 (offsetting effects of geometric efficiency loss and added densification)	-	-	-	Estimated that all piles carry same load at failure
Schmertmann	Deduced efficiency = 1.4	Failure of individual piles	Deduced ultimate settlement ratio = 3.4	Estimated by using settlement ratio in elastic range = 2.2	Estimated

<u>Notes for Table 2</u>

1. <u>Geometric efficiency formula</u>: Efficiency formula proposed by Bowles (1982) for reducing the capacity of piles due to block failure. Efficiency factors applied only to corner piles and only to shaft resistance reduction. Shaft resistance assumed in center pile was equal to one-half of that in the control pile. Toe resistances all equal to those for control pile.

2. <u>Increased ϕ due to driving adjacent piles</u>: ϕ (and δ) for <u>shaft</u> resistance (only) is increased by about $\{0.5(\phi+\phi_{max}) - \phi\}(1 - S/7r_p) \geq 0$ for each adjacent pile driven where S = center-to-center spacing and r_p = pile radius. ϕ_{max} typically = 40° or ϕ corresponding to D_r = 100%. Otherwise, capacity of each group

pile computed as per single pile method. Toe resistances are all equal to those for control pile.

3. <u>Increased lateral effective stress due to driving adjacent piles</u>: Lateral effective stresses (and therefore shaft capacity) = $\{1 + \Sigma(r_p/S)\}\sigma'_h$ (control pile), where summation is over the four adjacent piles. Otherwise, capacity of each group pile computed as per single pile method. ϕ reduced for toe capacity calculations to value lower than for control pile because N (30' depth) is lower at group test location.

4. <u>Increased ϕ (shaft) and increased vertical effective stress</u>: Same as 2 for shaft capacity. Vertical effective stresses (but not ϕ) increased for calculating toe capacity of each pile by 1/8 of shaft load on each adjacent pile divided by contributing area $\doteq 0.5\ S^2$.

5. <u>Vesic method</u>: w (group) = $w_o\ (B_{group}/B_{control\ pile})^{0.5}$, where w_o (pile head settlement of control pile) is obtained for the ratio of control pile load to control pile capacity = corresponding ratio for the average group pile. This is applied (nonrigorously) for failure as well as subfailure loads. B_{group} = width of group; $B_{control\ pile}$ = diameter of control pile.

6. <u>Meyerhof method</u>: w (group) = $(2\ Q_u/A_{group})(B_{group})^{0.5}(1 - D'/8B)$ + elastic compression of piles. A_{group} = cross-sectional area of pile group; D' = depth of mean load transfer ($\doteq$ 0.75 pile length); B = diameter of control pile.

7. <u>Double tangent method</u>: Settlement corresponding to load from a two-branched load-settlement curve, the first branch of which has a slope of sett./load = L/(AE) pile, and the second of which has a slope of 0.05 in./ton, beginning from a point on first line corresponding to a load equal to the true shaft capacity (considering residual stresses) plus one-half of the toe capacity.

8. <u>Equivalent pier unit load transfer curve method</u>: Used an equivalent pier with B = B_{group} and used Reese and Wright (1977) Q_t - w curve for large-diameter drilled piers superimposed on elasto-plastic Q_s - w curve, where $(Q_s)_{max}$ occurs at 0.005 x B_{group}.

9. <u>Two-pile-interaction-factor method</u>: Used Poulos and Davis (1980) interaction factors corrected for finite layer thickness and pile flexibility in a flexibility matrix equation in which single pile flexibility = $[w_o\ (failure)/Q_u]$ (control pile) and in which loads on piles are known from capacity analysis (requires compatible interaction factors).

10. <u>Scott method</u>: Method for computing distribution of loads to piles assuming that piles can be treated as disks in an elastic medium; described by Scott (1981).

Table 3. Methods of Interpreting Some Soil Properties

Predictor	Method of Correlation of N to ϕ	Values of ϕ (degrees) Shaft Avg.	Toe	Assessed Value of γ (pcf)	Assessment of E	Comments
Hood	Bowles (1968)	34	35.5	118 above w.t. 125 below w.t.	Not used	Water table at 9 ft depth
Lawler	Cheney and Chassie (1982)	33.5	34	120	Not used	Water table at 8-ft depth
Vanikar	Bowles (1968), verified by direct shear test results	36	35	126 (computed from given water contents and assumed specific gravity)	Not used	Water table at at 8-ft depth
Engel	Cheney and Chassie (1982) modified by direct shear test results	32.5	35	120	E judged to be 9000 psi; (used ν = 0.3)	Water table at 8-ft depth
Nordlund	Nordlund (1979)	31.6	34.4	112	Not used	Water table at 8-ft depth. Discarded unrealistically high or low 6-in. N's

Table 3. Methods of Interpreting Some Soil Properties (Continued)

Predictor	Method of Correlation of N to ϕ	Values of ϕ (degrees) Shaft Avg.	Values of ϕ (degrees) Toe	Assessed Value of γ (pcf)	Assessment of E	Comments
Schnore	Nordlund (1979), with ϕ increased by 0.33 ($40^\circ - \phi$) (after driving)	31	33	125	Linear correlation w/N (avg) and c (assuming lower clay norm. consol.) Sand - 4000 psi; clay - 1000 psi (used ν = 0.5)	Water table at 8-ft depth
Nacci	[Converted cone tip to Rel. Dens., Rel. Dens. to ϕ by Schmertmann (1975)] Verified generally by SPT blow counts. Increased ϕ by 0.5 ($40^\circ - \phi$)	38	38.5	112	E = 5000 psi based on avg. of PM reload modulus and CPT, DMT and SPT correlations	Water table at 7.2-ft depth

Table 3. Methods of Interpreting Some Soil Properties (Continued)

Predictor	Method of Correlation of N to ϕ	Values of ϕ (degrees) Shaft Avg.	Toe	Assessed Value of γ (pcf)	Assessment of E	Comments
Fellenius	[Used β for shaft resistance based on rate of increase of cone f_s with depth times 2 to account for compaction due to driving. Also used q_c directly for evaluation of toe capacity.]	-	-	100 (120 in surface fill)	Not used	Water table at 7.2-ft depth
Lukas	Peck, Hanson and Thornburn (1974) with no overburden correction	35	36	120	Not used	Water table at 8-ft depth
Dash/ Schmertmann	Dash - Used CPT f_s and q_c directly. Schmertmann - Used ϕ computed from dilatometer using Durgunoglu and Mitchell's (1975) $\xi_{\gamma q}$ and $N_{\gamma q}$ factors	30-45	33	80-125	E_D from DMT test (1500 - 3000 psi)	Water table at 8.2-ft depth

Only two predictors remarked that the boring data (e.g., SPT) were somewhat different at the general locations of the control pile and the group. In one case (Nordlund) group pile capacities were reduced because of this fact. Otherwise, predictors used average soil properties for the site for both group and control pile calculations.

Most predictors viewed the failure mode to be one of plunging of individual piles, as opposed to "block" failure. The settlement and load distribution/load transfer calculations were made using a wide variety of procedures. Table 2 will suffice in describing them. The author notes that no predictors used the FHWA pile group program PILGP1 (O'Neill et al., 1981), although one predictor (Engel) reported that he attempted to use that program but could not get it to run on his computer.

Interpretation of Soil Data

Most predictors chose to make static calculations of shaft and toe resistance using values of angle of internal friction (ϕ) based on correlations with SPT resistance values. Perhaps this route was chosen because it represents the practice with which most of the predictors were familiar. Unfortunately, the transformation of SPT N values into ϕ is highly problematical. Very little consistency existed among the predictors in this critical determination, which emphasizes the well-known lack of agreement in the profession on how to handle the problem. Interpretations are summarized in Table 3.

One predictor made automated static capacity calculations based on ϕ values obtained from Durgunoglu and Mitchell (1975) theory, which uses a single bearing capacity factor $N^*_{\gamma q}$ = base failure load/γ'B. $N^*_{\gamma q} = N_{\gamma q}$ x a shape factor. K values were taken as a weighted average of DMT K_o and K_D values, and ϕ was reduced to δ by a pile surface roughness factor. The $N_{\gamma q}$ factor was used with DMT ϕ values to compute pile toe capacity.

A method adapted by Bowles (1968) was chosen by two predictors, one of whom has automated it. In that method, which is ambiguous and therefore subject to individual interpretation, N values are corrected for overburden pressure and may be corrected for fines (not necessary for Hunter's Point site soils, since the fines content was less than 5 per cent). Two predictors used Nordlund's (1979) method, which was adapted from Peck, Hanson, and Thornburn (1974). That method also considers a correction for overburden pressure that reduces the raw N value more severely than the method adapted by Bowles. One of the predictors who used this method made the judgment that ϕ would increase beyond the value predicted by the correlation due to vibrations and compaction due to pile driving, using two-thirds of the increase in ϕ suggested by Kishida (1967) (= $40^\circ - \phi$ from SPT, as being applicable beneath the toes of driven piles) throughout the entire length of the pile, including the soil below the toe. Another predictor used the Peck et al. method but without correction for overburden pressure. This resulted in somewhat lower ϕ values near the surface but had relatively little influence on computed shaft resistance.

A third general procedure used by two predictors was that recommended by Cheney and Chassie (1982), using still a different rule for correcting for overburden pressure but using the same relationship between corrected N and ϕ described by Bowles (1968). Overburden correction factors recommended by Cheney and Chassie result in slightly lower corrected N values for σ_v' exceeding 1.5 ksf and slightly higher values for σ_v' smaller than 1.5 ksf in comparison with the method proposed by Peck et al. (1974). Two predictors used the results of the direct shear tests to confirm the general correctness of the correlated ϕ values.

Cone tip resistance (q_c) values, rather than N values, were used to obtain ϕ by one predictor, first through correlations of q_c with relative density and then with correlations of relative density with ϕ presented by Schmertmann (1975). This procedure resulted in the generally highest values of ϕ among the predictors.

One predictor had little confidence in ϕ values obtained from the preceding methods and determined a β factor from the f_s data from the cone soundings and an ultimate toe resistance from the q_c values at a depth of 30 ft, thereby bypassing the need to make any conversion from N to ϕ. Another predictor used both the f_s and q_c values directly in a formal, but simple, procedure patterned after Schmertmann (1978). (This general approach was also taken by other persons making secondary predictions or predictions using an average of several methods.)

The average interpreted ϕ values along the pile shaft among the predictors varied from about 31° to about 38°. The effect of this variation in interpreted ϕ can perhaps best be put into perspective by relating its effect to computed unit shaft resistance. Using the method for calculating shaft resistance most commonly applied by the predictors (Nordlund, 1979), this variation translates into unit shaft resistance being 70 per cent larger for the largest value of ϕ relative to that for the smallest. A similar but more pronounced effect exists regarding the variation of N_q due to the variation of ϕ at the pile toe.

Another source of inconsistency among those predictors making static capacity calculations (i.e., using some form of Eqs. 1 or 4) was the effect of the choice of soil unit weight, in which interpretations varied from about 100 pcf to 126 pcf, since its value is used to compute effective vertical stress and therefore unit shaft and toe resistances. Most of these interpretations were based on correlations with N values, grain size distributions supplied to the predictors and experience with sands at similar sites. One predictor stated that he used data supplied on one boring log which contained values of dry unit weight and water content obtained from drive samples taken with a Sprague and Henwood sampler.

Several predictors also used "elastic" soil parameters in some phase of their prediction (e.g., settlement of the pile group). Methods used and values obtained are also summarized in Table 3. Values of Young's modulus, E, for the soil obtained from various methods, ranged from 1000 psi to 9000 psi. These variations are reflected strongly, although not on a one-to-one basis, in the settlement computations.

Results

The results of the predictions are summarized in Tables 4-7 and in Figs. 2 and 3. "Probable error" in Tables 4 and 6 represent 50% confidence limits (plus or minus) for the set of predictions as a whole, assuming Gaussian distribution.

Table 4. Predictions of Control-Pile Capacity and Pile-Head Settlement at Failure

Predictor	Capacity (K) Total	Toe	Shaft	Settlement at Failure (In.)
Hood	140	94	46	0.5
Lawler	88	52	36	0.37
Vanikar	102	62	40	0.5
Engel	102	70	32	0.35
Nordlund	85	57	28	0.77
Schnore	81	52	29	0.32
Nacci	124	74	50	0.48
Fellenius	132	92	40	0.7
Lucas	128	76	52	0.37
Dash/	132	90	42	1.1
Schmertmann	177	104	73	0.8
Mean	117	75	42	0.57
Probable Error	19.5	12.2	8.6	0.16
Measured				
Davisson	79	40	39	0.35
0.05 in./ton	85	46	39	0.41
0.1 pile diam.	97	66	31	1.08
Sett.=3.33 in.	109	76	33	3.33

Note: True, not apparent, toe and shaft loads are presented here.

Table 5. N_q Values Used in Capacity Computations for Control Pile

Predictor	Method Used	Value
Hood	Thurman (1964)	68 (no explicit limit on σ_v')
Lawler	Thurman (1964)	34 (no explicit limit on σ_v')
Vanikar	Thurman (1964)	39 (no explicit limit on σ_v')
Engel	Thurman (1964)	42 (no explicit limit on σ_v')
Nordlund	Nordlund (1979)	45 (σ_v' limited to 3000 psf)*
Schnore	Nordlund (1979)	58 (σ_v' limited to 3000 psf)*
Nacci	Berezantzev et al. (1961) - Essentially identical to Nordlund (1979)	118 (σ_v' limited to 1000 psf) (Critical depth = 12.5 diams.)
Fellenius	[Toe bearing pressure directly from cone q_c = 146 ksf]	80 (Toe bearing value is a true value since residual loads will be measured)
Lukas	Various methods, e.g., Vesic (1965) Poulos & Davis (1980) API (1985)	 90 (σ_v' limited to 756 psf) 50 (σ_v' not limited) 40 (bearing pressure limited to 200 ksf)
Dash	[Used q_c (min. path) directly to obtain toe capacity = 143 ksf]	- (no explicit limit on q_c)
Schmertmann	Durgunoglu-Mitchell (1975) (w/min. path) $N^*_{\gamma q} \doteq 3300$	- (no explicit limit on $N^*_{\gamma q}$)

*σ_v' (at toe) < 3000 psf at Hunter's Point site.

Table 6. Predictions of Group Capacity and Pile-Head Settlement at Failure

Predictor	Capacity (K) Total	Corner Toe	Corner Shaft	Center Toe	Center Shaft	Settlement at Failure (In.)
Hood	677	94	46	94	23	1.0
Lawler	482	52	43	52	50	0.76
Vanikar	510	62	40	62	40	1.7
Engel	793	95	76	95	16	3.0
Nordlund	388	40	36-37*	40	42	0.56
Schnore	583	72	31	130	38	1.62
Nacci	744	74	80	36	90	1.11
Fellenius	Qualitative Predictions, but No Quantitative Predictions, Given					
Lukas	800	76	76	76	116	0.9
Dash	660	90	42	90	42	2.8
Schmertmann	1239	164	89	148	80	2.8
Mean	688	82	56	82	54	1.63
Probable Error	160	23	15	25	21	0.62
Measured (0.05 in./ton/pile)	421	23**	63	21**	58	0.37
Measured (point of increased creep)	479	41**	55	41**	55	1.05
Measured (maximum load applied)	561	59**	51	67**	53	6.83

*Corner piles will not carry equal loads, producing rotation.

**Includes 2 K residual toe loads in all piles. Toe load measurements were made only in Piles 1-3.

Note: Corner piles carried 50.7 K/pile (on the average) and center pile carried 45.1 K at load equal to one-half creep failure load.

Table 7. Predicted and Measured Efficiencies and Settlement Ratios[1]

Predictor	Efficiency	Settlement Ratio
Hood	0.94	1.2
Lawler	1.10	1.8
Vanikar	1.00	4.0
Engel	1.3	1.3
Nordlund	0.91[2]	0.9
Schnore	1.43	2.6
Nacci	1.20	1.4
Fellenius	-	-
Lukas	1.25	2.1
Dash	1.0	-
Schmertmann	1.4	2.2
Measured (0.05 in./ton/pile)	0.99 (Shaft - 1.60 Toe - 0.50)	1.0
Measured (creep load)	1.09 (Shaft - 1.45 Toe - 0.90)	1.3
Measured (maximum load)	1.03 (Shaft - 1.57 Toe - 0.87)	1.1

[1]Settlement Ratio at Load Per Pile = 0.5x(0.2 x Total Group Capacity), i.e., at 50% of Predicted Group Failure Load.

[2]Actually predicted shaft efficiencies of 1.44 to 1.60 and toe efficiency of 1.0 for condition in which control and group piles are in identical soils.

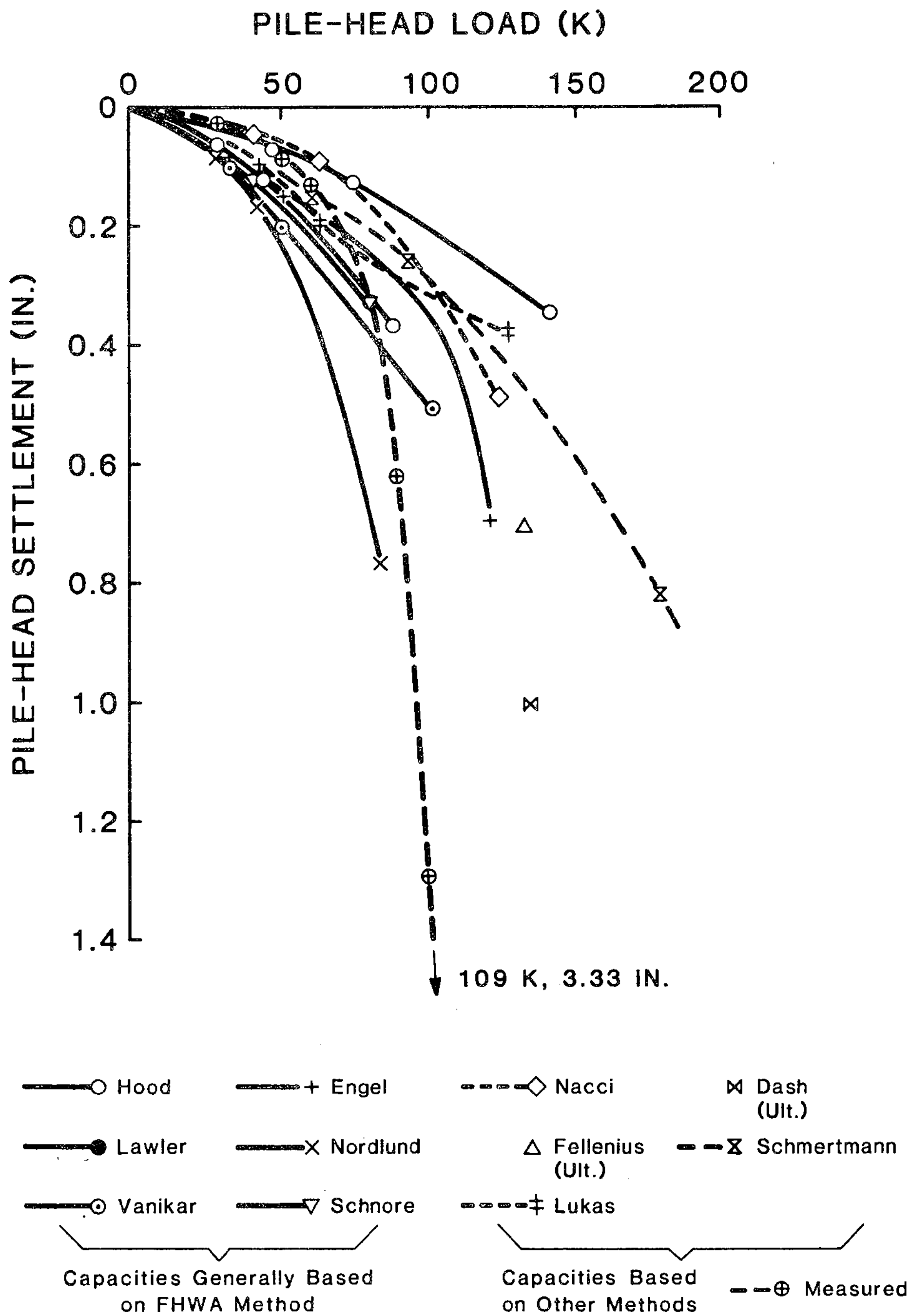

FIG. 2. PREDICTED AND MEASURED PILE-HEAD, LOAD-SETTLEMENT RELATIONSHIPS FOR CONTROL PILE

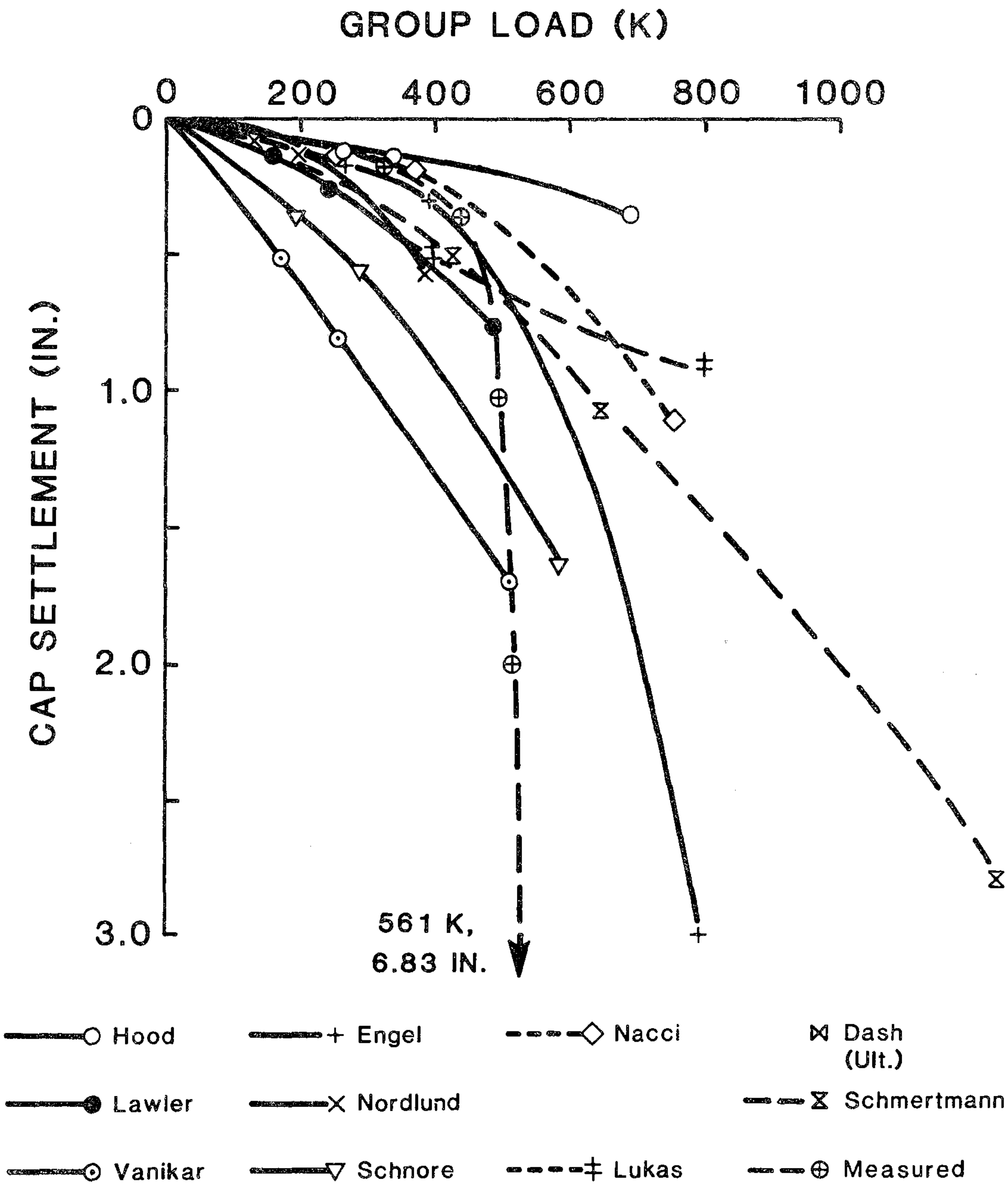

FIG. 3. PREDICTED AND MEASURED PILE GROUP LOAD-SETTLEMENT RELATIONSHIPS

A brief summary of the procedures for ascertaining N_q, referenced in Table 5, is in order. The method now recommended by FHWA, Thurman's method, employed by many of the predictors, uses Berezantzev et al's (1961) factors, which are functions of ϕ and pile shape, for the range of ϕ of interest here and reduces those factors by a multiplier (α), which depends on the value of ϕ and the value of relative embedment. Thus, those phenomena that tend to limit toe bearing stresses (residual loads, grain crushing, "arching") are apparently accounted for. An alternate interpretation of the use of Berezantzev's factors without α was provided by Nacci in which the vertical effective stress at the level of the toe was limited to the value at a depth of 12.5 diameters. Nordlund's method also uses the Berezantzev factors but with smaller reductions in N_q based on relative embedment. However, Nordlund recommends the use of a limiting value of σ'_v = 3000 psf to account for the tendency for unit toe resistances to be limited at large depths. Vesic's (1965) method and the API method also have been used with either real or implicit limits. The use of the Durunoglu-Mitchell factor also inplicitly requires a limiting toe capacity since the depth factor in the equation to evaluate $N_{\gamma q}$ is limited by the vertical distance from the toe to the point of vertical tangency of the failure surface.

If the limiting toe resistance is only apparent (a result of residual compressive streses that exist after driving), the method used to compute shaft resistance should explicitly or implicitly consider shaft resistance also to be apparent. Most procedures used by the predictors to compute shaft resistance in fact have been correlated to load test results in which only apparent shaft loads have been known and are therefore consistent, at least in principle, with the use of the methods that limit unit toe capacity.

The variations in N_q values among predictors using the same method for toe capacity calculation are attributable mainly to the manner in which each predictor assessed ϕ. The magnitudes of these variations are witness to the fact that methods based upon assigning a toe bearing capacity factor based on a single interpreted soil property parameter (ϕ) remains a problem. It is somewhat surprising to the author that no attempts were made to assign toe bearing capacity based on theories that consider soil compressibility or rigidity index (e.g., Vesic, 1977).

A gross summary of the group predictions is provided in Table 7, in which predicted load efficiencies and settlement ratios are tabulated. Most predictors determined that the group piles would carry more load than the control pile (except for Hood, who used a geometric efficiency formula, and Nordlund, who concluded that the soil was less dense at the group test site). Most predictors also deduced that the settlement ratio would be greater than unity.

Commentary on Test Results

Control Pile. The "measured" data reported in Table 4 for the control pile include an estimate of residual toe load of 4 K, which was included based on discussions with Eric Ng, who indicated that this was the approximate toe load after driving the control pile. (Actual data supplied the author were based on pre-test zeroes.)

The pile developed its maximum shaft resistance at a load approximately corresponding to the Davisson failure load, after which shedding of load from the shaft to the toe ensued. This effect apparently produced the rapid curvature in the load-settlement curve, shown on Fig. 2, in the load range of 80-90 K.

As expected, none of the predictions of control pile behavior were exact. However, the predictions that yielded values of failure load and distribution of failure load between shaft and toe closest to those measured were those that followed the FHWA procedure. One predictor using that method, however (Hood), selected ϕ values that gave excessive capacities (particularly at the toe) than the other predictors. Those using in-situ data (e.g., CPT, DMT) tended to predict somewhat excessive shaft capacities and toe capacities that generally approached the measured toe loads only at very large values of toe settlement (approximately 30 per cent of the toe diameter), although Lukas' computations using cone data from CPT-2 (greatest distance from test piles) yielded values consistent with measured values at much lower settlements. The true toe resistance of 121 ksf (76 K/0.63 ft^2) at large displacement (0.3 x toe diameter) compared favorably with the failure values of 140-150 ksf computed from weighted average q_c values near the toe depth by Fellenius and Dash, although those predictors deduced that less toe settlement would be required to develop such resistance. This fact suggests to the author that the soil under the pile toe was behaving as a very compressible granular medium and that "local failure" reduction factors may need to be applied to the q_c values in soils such as that at Hunter's Point. As it happens, Thurman's α factors seemed to perform this function reasonably adequately for this test, although it is not at all clear that these factors do so in a rigorous manner. The assumption of compaction beneath the pile toe by Nacci resulted in an N_q value that predicted excessive capacity. Unfortunately, the DMT test analysis using $N_{\gamma q}$ factors significantly overestimated toe capacity. This may be due to a systematic problem or may have been simply the result of the fact that the DMT probe hole used (DMT-1) was more than 10 ft from the nearest SPT or CPT boring and reflected actual differences in soil properties at that probe location. It is noted, however, that DMT-1 was only about 3 ft from the test pile.

The use of values of ϕ obtained from SPT N values and Nordlund's computational method appeared to produce reasonably accurate values of shaft capacity.

It should be mentioned that it is possible that the apparent error of any prediction was not totally the result of systematic attributes of the method used but instead may have been caused by anomalous soil behavior at the exact location of the control pile. Perhaps future CPT probing at the test site could be employed to produce a better statistical picture of areal and depthwise variabilty of soil properties than is presently available.

Regarding load-settlement behavior, most predictors underestimated the initial stiffness of the pile and also underestimated the curvature in the load-settlement curve at the point at which maximum shaft resistance occurred. The former effect, if important for a particular design, could possibly be forecast by explicit consideration of the release of residual stresses during loading through an appropriate computer simulation (e.g., Holloway et al., 1978). The unexpected load shedding and highly flexible behavior of the soil at the toe of the control pile also emphasizes the practical importance of obtaining pile driving and static load test data prior to embarking on a pile foundation design in unfamiliar soil. None of the predictions anticipated these effects.

Pile Group. The measured load-settlement curve is shown in Fig. 3. The near-plunging nature of the curve is indicative of individual pile failure, but its severity is unusual for piles in sand.

Several interpretations of group failure load are tabulated in Table 6. First, the definition of failure at a load corresponding to a rate of settlement of 0.05 in./ton/pile (an isolated-pile definition) gave the highest shaft resistance but also gave a very low average toe capacity. While this definition in probably adequate to define shaft failure, the actual failure load for the group can probably be taken to be a larger value.

The definition of failure preferred by the author, the creep method, is illustrated in Fig. 4. The "creep point" for the control pile was 87 K, which closely corresponds to the load at which rate of settlement equals 0.05 in./ton (85 K). The creep point for the group was at 95 K per pile, or 475 K for the group. This is nearly identical to a load to which the group was subjected (479 K). It is therefore sufficiently accurate to define 479 K as the creep failure load. At this load, shaft resistances had decreased from the values measured at the load corresponding to a settlement rate of 0.05 in./ton/pile (421 K), most notably in the corner piles, while toe resistances had nearly doubled. Further loading to a maximum load of 561 K produced further reductions in shaft resistances and increases in toe resistances.

By comparing the measured results in Tables 4 and 6, it can be seen that there was a clear increase in shaft capacity in the group piles. This phenomenon, which was recognized by most predictors, probably increased driving resistance in the latter piles driven in the group. It can also be observed in Table 6 that the loads were very evenly distributed among the group piles. This observation is consistent with an earlier study on a larger group of piles driven in clay (O'Neill et al. 1981) and casts serious doubts on mathematical models that are predicated on elastic behavior and that predict much larger loads in the exterior piles in an axially loaded group than in

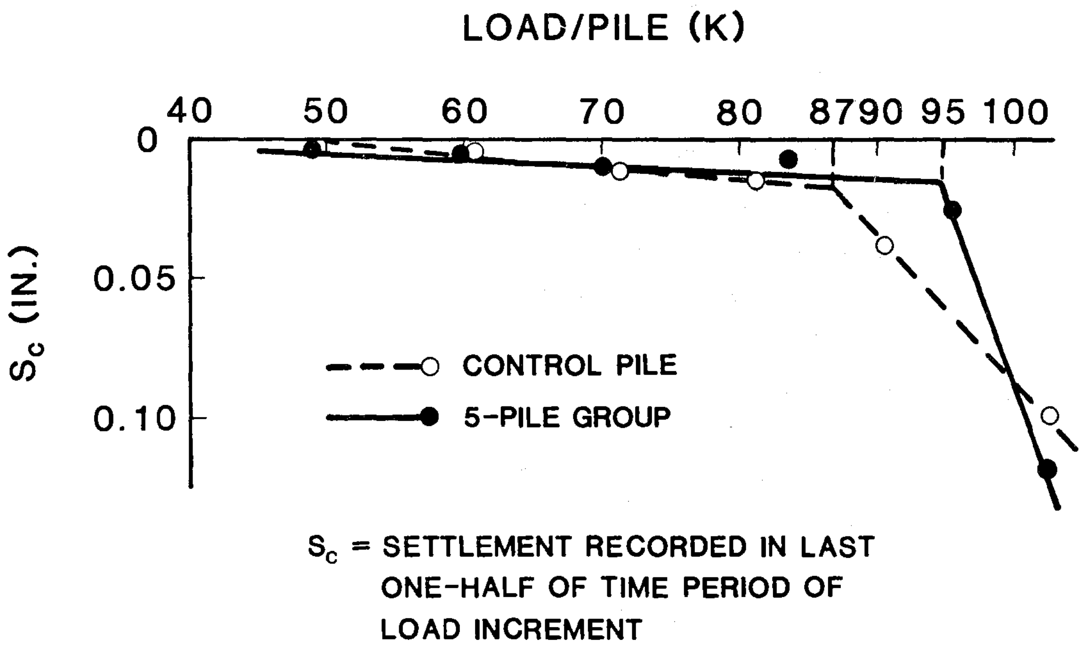

FIG. 4. INTERPRETATION OF FAILURE LOAD BY CREEP METHOD

the interior piles. Most predictors fortunately did not fall into the trap of using elasticity models without physical intuition.

Both the control pile and the group piles exhibited load shedding past their peak shaft resistances. This phenomenon, which was not forecast by any of the predictors, probably accounts partially for the high rate of settlement experienced by the group beyond a load of 400 K.

Measured shaft and toe resistance efficiencies are tabulated in Table 7, along with the predicted and measured total efficiencies. While the average shaft efficiency in the group was apparently of the order of 1.45-1.60, depending on the definition of failure, the toe efficiency was apparently less than unity, even when the toes were pushed in excess of 0.5 ft. (The word "apparently" is used because toe loads were measured on only three group piles: the first three that were driven. The toe load on the last pile driven should have been higher than that on the first piles driven, resulting in actual average shaft efficiencies that are slightly lower than those tabulated and toe efficiencies slightly higher than those tabulated, but still below 1.0). The low toe efficiency combined with the high shaft efficiency caused the total efficiency of the group to be about 1.0. This behavior may have been due to loosening during driving of the rather uniformly graded soil beneath the toes of piles that were driven before the last pile or may have simply been due to the fact that the soil at the toe elevation was looser initially at the group test location than at the control pile test location. The research team is encouraged to resolve this question.

Settlements were only slightly greater on a load-per-pile basis in the group than in the control pile. Measured settlement ratios tended to be much lower than those predicted. See Table 7.

In the author's opinion the predictions for the single pile taken as a whole were certainly satisfactory. However, unlike a similar set of predictions made for a group of pipe piles in clay (O'Neill et al., 1981) the group predictions, taken as a whole, indicated an overly optimistic assessment of group performance. For example, the average predicted capacity of 688 K is 43 per cent greater than the measured creep capacity of 479 K and 23 per cent greater than the maximum load applied (561 K, which produced settlements larger than most structures could tolerate). Whether this problem is a result of inadequate understanding of the mechanics of group action or the result of conditions peculiar to the case being considered here should be the focus of further study.

Addenda to Predictions

Engel revised his predictions for the pile group capacity downward to 715 K based on the performance of the control pile in the static load test. He also increased the shaft capacities by about 8 per cent and reduced the toe capacities of the group piles by about 20 per cent relative to the solutions described in this summary paper.

Fellenius analyzed the control pile using CUWEAP, a wave equation program. By assigning generally common pile and soil parameters and a 50 per cent toe resistance, he found generally good agreement between the predicted resistance of the control pile (both from the wave equation and his static procedures) and its measured capacity at the blow count that was observed. This suggests that there was nothing unusual in the performance of the isolated pile during driving and that the static predictions that he made were appropriate. However, the wave equation analysis revealed that the Delmag D-22 hammer had excessive energy for the conditions of the test.

Discussion

During the FHWA Pile Group Prediction Symposium held at the University of Maryland on June 17 and 18, 1986, each of the predictions was afforded the opportunity to discuss his predictions and the general conditions of the pile tests.

One major question that arose during the discussions was whether the SPT tests, upon which many predictors relied to estimate ϕ and γ, were conducted with a safety hammer or a doughnut hammer. The suggestion was made by Lukas and Schmertmann that the doughnut hammer delivers consistently less energy, and consequently produces higher blow counts (N values), than the safety hammer. Eric Ng confirmed that the doughnut hammer had been used, as is common practice in the San Francisco Bay area. The implication is that had a safety hammer been used and lower N values obtained, many predictors would have predicted lower pile capacities, which would have been closer to the measured capacities.

An equally significant question centered around the mineralogical composition of the sand. Since the sand at the toe levels appeared to be very compressible in the pile tests and since the cone friction ratio was lower than one normally expects in loose to medium dense siliceous sand, the hydraulic fill could possibly possess weak comentation (perhaps calcareous material).

Nordlund raised the issue that soil unit weights are important in arriving at pile capacities and that no clear definitions of unit weights had been supplied the predictors. The validity of unit weights obtained from drive samplers such as the Sprague and Henwood samplers was doubted by most predictors.

The areal consistency of the soils at the test site was also questioned. This is an important issue because no solid conclusions can be drawn regarding group action relative to single pile action until it is confidently confirmed that consistent conditions exist at the location of the group and the location of the control pile.

Eric Ng indicated that these issues will be addressed by the research team during the concluding period of the study.

Comments were also made by Hugh Lacy of Mueser Rutledge Consulting Engineers, Inc., concerning the appropriateness of the particular pile design (closed-end pipe piles penetrating 30 ft) selected for the study. The point was made by Mr. Lacy and others that had the project been a real engineering job, the piles of choice would probably have been either short tapered piles or piles driven to high end bearing capacity in the serpentine bedrock. In response, it was pointed out that this was a "research" project aimed at observing basic phenomena, as distinct from an "engineering" project.

Details of the pile instrumentation system were also discussed. The principal concern was the fact that the strain gages were mounted on the exteriors of the piles and were protected by protruding steel angle sections, 2 in. by 2 in. in size (to nearly the full pile length). How these angles may have influenced load transfer is at present unknown. However, since the angles were welded to all of the pipe piles tested, not just those with complete instrumentation, relative effects between control and group piles were thought to be minimal.

The possibility that the "soft" behavior of the pile toes in the group may have been caused in part by in-place piles being lifted off their toe bearing surfaces by the driving of adjacent piles was also raised. D.M. Holloway, of InSituTech, Inc., a consultant on the project, indicated that the pile driving contractor had monitored pile heave during driving of adjacent piles and had observed none.

Many of the predictors indicated that they tried to follow their normal practice for computing control pile and group capacities. A few, however, indicated that they made special efforts to predict group behavior that would not be made on an ordinary engineering project. The special solutions were intuitive and rational but did not use technology that is not generally available to bridge foundation engineers.

Jean-Louis Briaud of Texas A and M University, who was not an official predictor but who presented the test results at the symposium on behalf of the research team, described analyses made with Program PILGP2, which is a version of PILGP1, the FHWA pile group program. Several different sets of inputs were used. The best agreement between computations and measurements were for unit load transfer curves developed from CPT records and elastic moduli that were of the order of twice the values obtained from the pressuremeter reload moduli.

Given the nature of the problem, Bengt Fellenius offered the judgment that the predictions had been a "bull's eye," at least as they related to the control pile. Ashton Lawlor, a practicing state engineer whose numerical predictions were quite close to the measurements for both the control pile and the group, concluded that the prediction exercise had strengthened his confidence in the FHWA pile design method.

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