CENTRIFUGAL TESTING OF MODEL PILES AND PILE GROUPS

VOL. I EXECUTIVE SUMMARY

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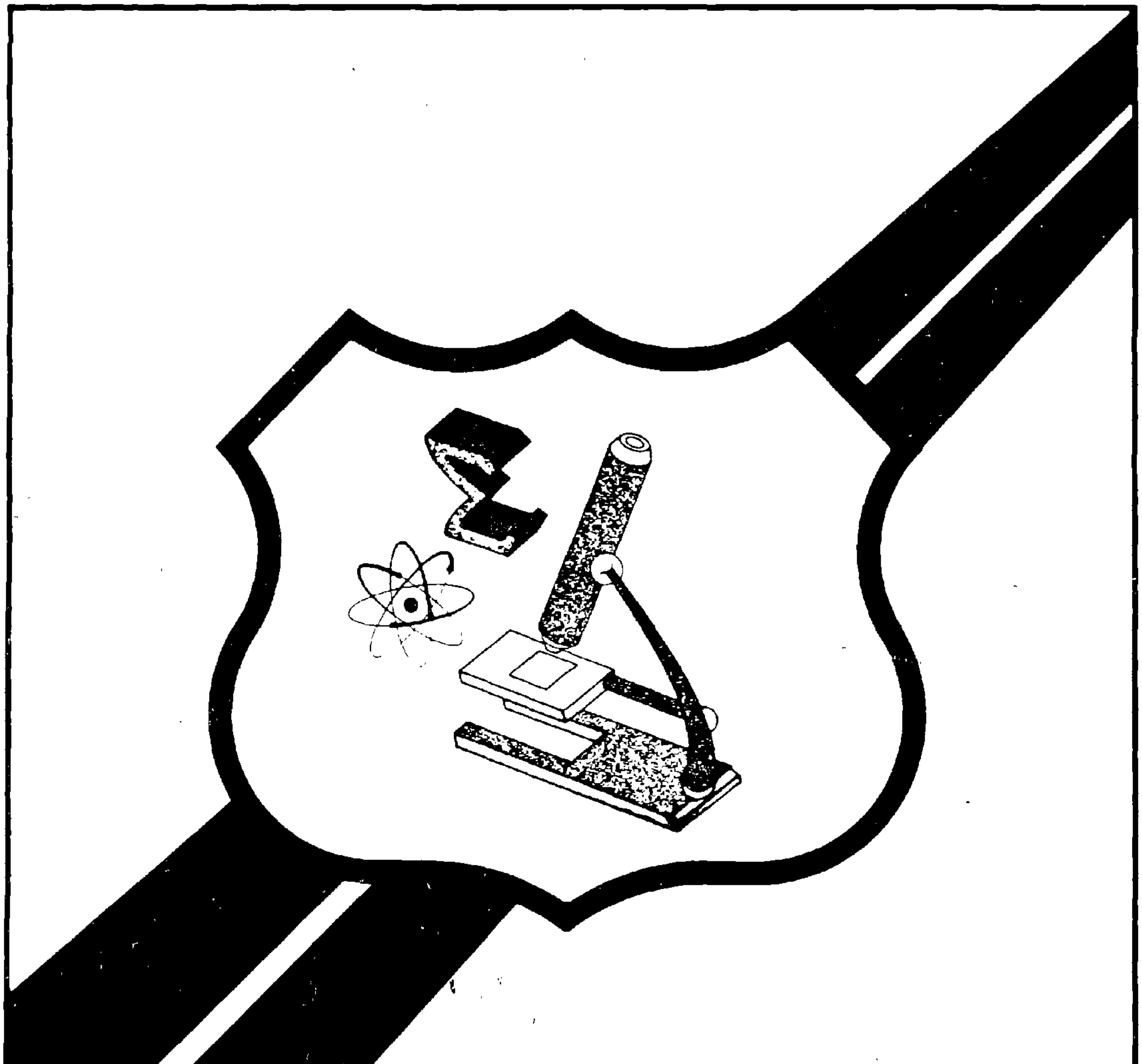

FOREWORD

This report presents a summary of results from a project to study the use of the geotechnical centrifuge to model piles and pile groups in both sand and clay projects. These results will be of interest to engineers designing pile supported foundations. This project has shown that centrifugal testing is a useful supplement and perhaps can serve as an alternative to expensive field strength tests.

This report summarizes the results of a University of Colorado research project, "Centrifuge Testing of Model Piles and Pile Groups." The project was conducted for the Federal Highway Administration, Office of Engineering and Highway Operations Research and Development, Washington, D.C., under contract DTFH61-81-R-00034.

Two full scale prototype pile groups were modeled, one in clay and the other in sand. Axial single pile and group model load tests to failure were conducted at scale factors of 50, 70, and 100. The load settlement and load transfer relationships obtained were compared with corresponding prototype load test results, with each other, and with analytical predictions.

Richard E. Hay, Director
Office of Engineering
and Highway Operations
Research and Development

NOTICE

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16. Abstract

This volume describes in summary form the research program conducted to evaluate the feasibility of conducting tests of model piles and pile groups using the geotechnical centrifuge. The goals of the program are described, the testing programs in both sand and clay are briefly discussed, important results are presented and conclusions and recommendations for future research are given.

Other reports developed in this study are FHWA/RD-84/003, Vol. II, Centrifugal Tests in Sand, and FHWA/RD-84/004, Vol. III, Centrifugal Tests in Clay.

Dr. R. H. Atkinson, Atkinson-Noland & Associates served as Project Director. Prof. H-Y. Ko and G. G. Goble served as Principal Investigators. Mr. F. Harrison conducted the test program in clay while Mr. M. Manzoori conducted the test program in sand. All experimental work was conducted using the geotechnical centrifuge of the Department of Civil, Environmental, and Architectural Engineering, University of Colorado, Boulder, Colorado.

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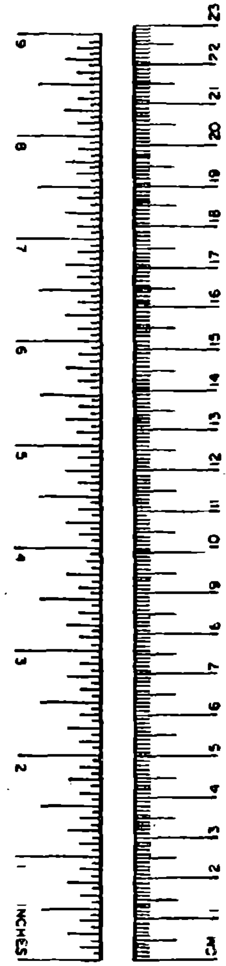
METRIC CONVERSION FACTORS

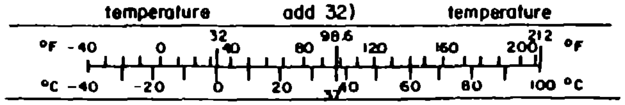
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
	LENGTH			
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares ($10,000 m^2$)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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CHAPTER 1. INTRODUCTION

1.1 Model Testing of Geotechnical Structures

Full scale testing of geotechnical structures, such as retaining walls, dams, foundations, etc., is rarely performed due to the great expense and difficulty of loading these structures to meaningful levels. When full scale field tests are conducted, they are usually of one design at a specific site and thus do not yield information on the influence of variables such as structure design or soil properties.

Model tests on scaled structures in the laboratory can overcome many of these problems. For structures located within or loaded by a soil mass, proper simulation of field (prototype) conditions requires that the proper magnitude and distribution of soil interaction forces be modeled. The stress-strain-deformation properties of soil are directly dependent on the magnitude and nature of the applied stress state which is, for most structures, usually generated by self-weight or body forces of the soil under normal gravity conditions. When testing small scale models in laboratory situations, however, proper modeling of the soil stress field and the resulting soil properties is usually not possible under 1 g conditions.

A method to achieve the required soil-stress field is to conduct a model test in an increased gravity field created by using a geotechnical centrifuge. The acceleration generated by the centrifuge increases the gravity-induced body forces on the soil particles and hence the apparent specific weight of the soil. Thus, the in situ stress field in a soil deposit can be correctly modeled.

The theory and application of centrifugal testing of soil-structure models which has been discussed in detail by Schofield (1980), is briefly reviewed. If λ denotes the scaling factor between model and prototype,

$$L_p = L_m \lambda \ , \tag{1}$$

then scaling relationships for quantities of importance in geotechnical engineering can be derived as listed in Table 1.

Table 1. Scaling factors for various quantities

Length	λ
Stress	1
Time	λ^2
Strain	1
Force	λ^2
Area	λ^2
Volume	λ^3
Specific Weight	$1/\lambda$
Gravitational Acceleration	$1/\lambda$
Mass Density	1
Mass	λ^3

These relationships are based on the assumption that the same soil material is used for both the model and prototype. Since, as discussed above, body forces cannot be neglected, the required scaling relation between model and prototype-specific weights can be achieved by centrifugal acceleration. As seen in Table 1, this modeling approach yields scaling factors of unity for both stress and strain and permits a direct comparison between prototype and model behavior at equivalent locations.

1.2 Model Testing of Piles and Pile Groups

The difficulties of full scale field testing discussed above apply to single piles and especially to pile groups where the magnitude of loads required to produce group failure become extremely difficult to generate. The deformation and failure of pile foundations is primarily influenced by the properties of the surrounding soil and its in situ stress state. For these reasons, results from the few model pile group studies at 1 g conditions (Whitaker, 1957; Saffery and Tate, 1961; and Sowers, 1961) have been largely qualitative due to the lack of adequate scaling relationships between prototype and model (Rocha, 1957).

After a limited application in the 1930's, centrifugal modeling of geotechnical structures was abandoned in the West until the late 1960's. The technique was first used at Cambridge University with other geotechnical centrifuge facilities following in Japan and the United States. An excellent discussion of centrifuge testing is provided by Schofield's 20th Rankine Lecture, 1980.

Research using the centrifuge specifically involving behavior of pile foundations has begun fairly recently. Scott's (1979) research on pile groups in silt subjected to cyclic lateral loads produced results which were internally consistent and demonstrated the feasibility of conducting pile load tests in a centrifuge. The lack of a prototype with which to compare results and thus verify the similitude relationships was a shortcoming of the study.

Axially loaded piles in sand were investigated by Hougnon (1980) at the University of Colorado. Although this was largely a feasibility study, some useful data regarding the effect of taper and soil density on bearing capacity was obtained. Problems encountered with uniform soil preparation and the loading apparatus limited the effectiveness of Hougnon's work.

In the last several years, many studies have been conducted on modeling of piles in the centrifuge. Some of these are in the proprietary domain, and many are still only available in the form of student theses. Clegg (1981) modeled axial loading of piles in stiff clay. Sabagh (1984) conducted cyclic axial load tests of piles in sand. Barton (1982) and Oldham (1984) carried out lateral loading of model piles in sand. Craig (1984) described the techniques used at the University of Manchester for pile installation for centrifuge tests. However, apparently no work has been done with respect to modeling of pile groups in the centrifuge.

In a recent meeting on the applications of centrifuge modeling to geotechnical design, Nunez and Randolph (1984) pointed out that each model pile testing project in the centrifuge requires development of novel experimental techniques with respect to (1) sample preparation to match prototype conditions; (2) scaling pile geometry and properties; (3) axial and lateral loading mechanisms to operate in a high gravity field; (4) in-flight pile installation techniques; (5) in-flight measurement of soil properties for the pile test; and (6) instrumentation to monitor pile performance including both internal stresses and external load-deformation. Of course, interpretation of the centrifuge test data, comparison with prototype performance and with analytical results are usually included in these investigations.

It is clear that the FHWA-sponsored work at the University of Colorado, described in this report, was started in 1981 at a time when several other studies were commenced in other centrifuge facilities. Many of the problems we faced were also encountered by other investigators and different solutions were used to solve these problems. It is the general conclusion of this group of investigators that centrifuge modeling is a viable method for studying pile foundation because it enables the effects of self-weight induced stresses to be properly modeled. Because it is inexpensive in comparison to prototype testing, it is possible to conduct parametric studies varying both soil and pile conditions to derive a clear understanding of the behavior of pile foundations.

The results of the present study have contributed to this growing reservoir of data from centrifuge modeling of pile foundations and the findings are consistent with those obtained elsewhere. It is the belief of the investigators that the centrifuge modeling technique has become well-established as an invaluable tool for the geotechnical engineer to use in modeling geotechnical structures.

CHAPTER 2. RESEARCH PROGRAM

2.1 Purpose

The research program reported herein on the centrifugal testing of model piles and pile groups was funded by the Federal Highway Administration. The program had three main objectives:

(1) To develop experimental techniques for testing model piles and pile groups in a geotechnical centrifuge. Among the experimental techniques to be developed were: 1) soil preparation and placement, 2) fabrication and instrumentation of model piles and pile groups, 3) pile installation equipment and techniques, and 4) pile load testing equipment and techniques.

(2) To perform a series of model pile tests in both sand and clay to verify both qualitatively and quantitatively the method of centrifugal testing. The method of modeling of models as described below was the approach used to provide quantitative verification of centrifugal model testing of piles.

(3) To perform single pile and pile group tests that model field tests on full scale piles. One of the field test programs was conducted on single piles and a 9-pile group at the University of Houston campus (O'Neill et al., 1981). This test was conducted in an overconsolidated clay deposit. The other field test to be modeled was conducted at Lock and Dam No. 26 on the Mississippi River near Alton, Illinois by Woodward-Clyde Consultants (1979). Single pile tests and tests on an 8-pile group in a cohesionless alluvial sand deposit were to be modeled.

CHAPTER 3. EXPERIMENTAL PROGRAM

3.1 Introduction

The research program consisted of two separate parts, tests of model piles in clay and tests of model piles in sand. These two studies have in common the use of the same centrifuge, the same pile loading mechanisms, and the same data acquisition system which are discussed below. The soil preparation, model pile design, instrumentation, test procedure and results for the sand and clay studies are discussed separately below.

3.2 Centrifuge and Pile Loading Mechanism

Tests were conducted using the 10 g-ton geotechnical centrifuge located at the University of Colorado, Figure 1. This machine can generate an acceleration of 100 g on a 200 lb (90.8 kg) package. This centrifuge is equipped with swinging "baskets" mounted at the ends of a symmetrical 42-inch (1.1 m) radius arm. The hinged baskets permit the resultant of gravity and centrifugal forces to act at all times perpendicular to the base of the model foundation. The centrifuge is equipped with 56 electrical slip rings for power and electrical signal transmission and two hydraulic slip rings for control of the loading mechanisms.

The soil container for both the sand and clay tests was a 15-inch (38.1 cm) diameter aluminum cylinder 12 inches (30.5 cm) deep. The driving and loading of the piles were performed in-flight using a Bellofram cylinder. A LVDT with a 5-inch (12.7 cm) range was used to monitor pile installation while a more sensitive unit with a 0.50 inch (1.27 cm) range was used to monitor pile motion during load testing. Applied loads were measured using special strain-gauge type load cells. Lateral loads were generated by a Bellofram cylinder applying tension to a thin wire cable which by means of a pulley arrangement was connected to the pile top. Deformation was measured using an LVDT and lateral force by means of a tension load cell connected to the cable. Figure 2 shows both the axial and lateral test configurations. Data was recorded using either analog plotters or a Hewlett-Packard 9825B data logging system which has a high scan rate.

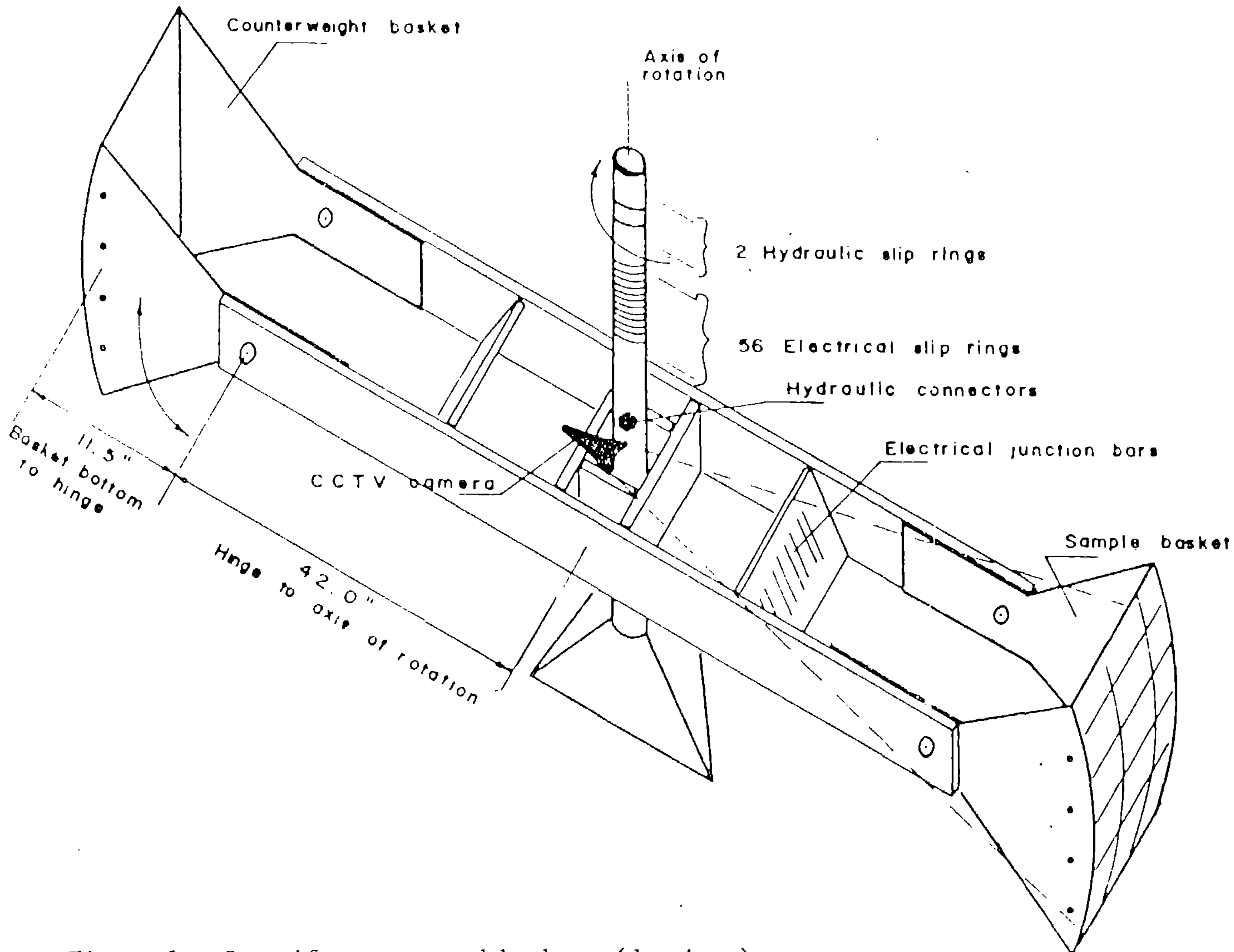


Figure 1. Centrifuge arms and baskets (drawings)

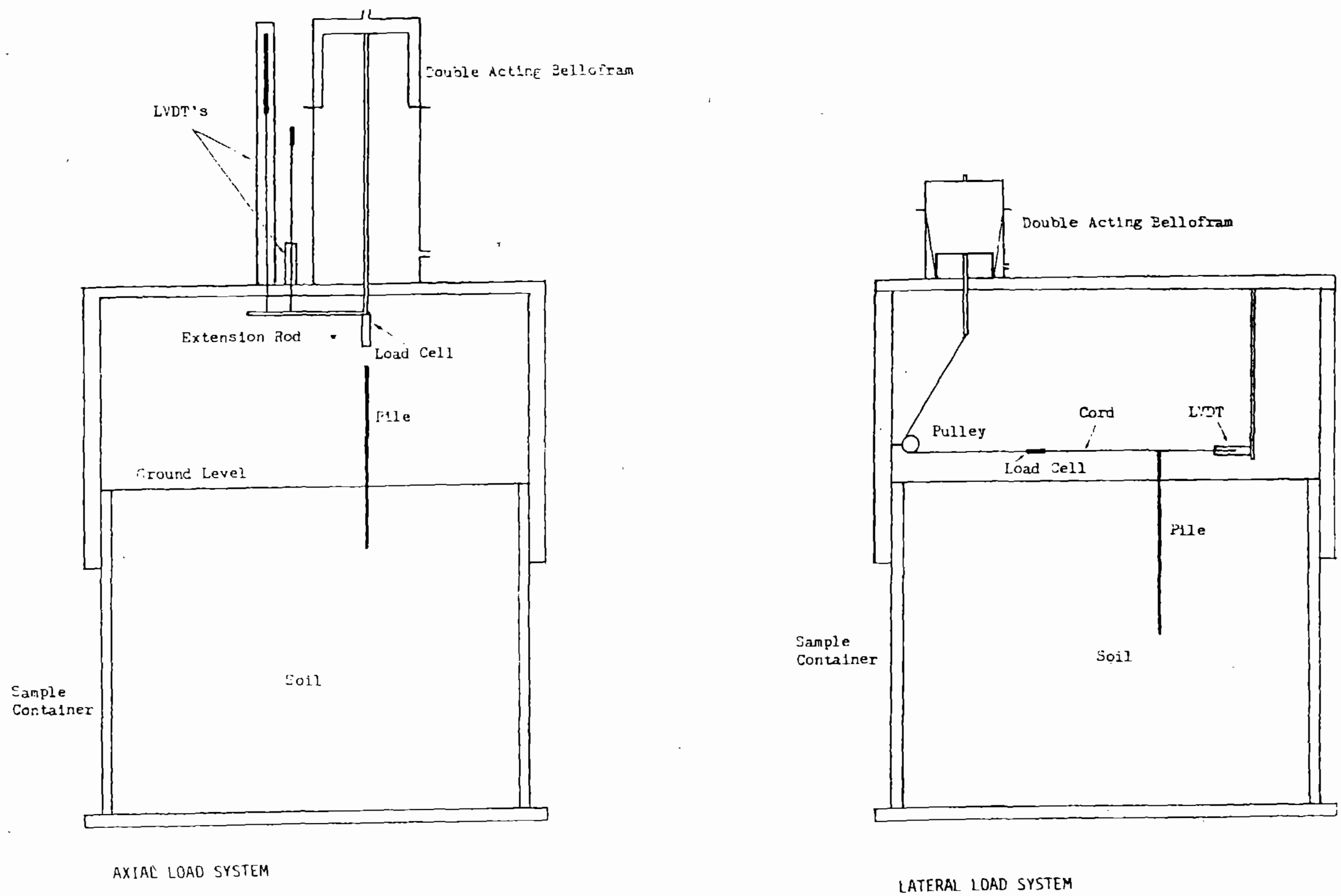


Figure 2. Mechanisms for loading model piles in the centrifuge.

3.3 Pile Tests in Clay

3.3.1 Soil Preparation and Pile Design

A Georgia kaolin was used to simulate the Beaumont clay from the University of Houston site, as the prototype soil was unavailable in block samples. The laboratory soil was prepared at an initial water content of 60 percent (L_w = 45%), placed in the soil container equipped with an extension tube and consolidated under a series of increasing static loads. The preconsolidation stress was determined using procedures developed by Atkinson and Bransby (1978). After the static consolidation pressure was relieved, the soil container was installed in the centrifuge and brought up to the test g level to produce the desired vertical distribution of undrained shear strength, Figure 3. Shear strength, S_u, of the clay soil was measured in-flight using a specially developed vane shear device.

Model piles were fabricated to 1/50, 1/70, and 1/100 scale factors. The 1/50 and 1/70 scale piles were fabricated from split aluminum tubing which permitted installation of internal strain gauges before the two halves were epoxied together. Strain gauges were installed at diametrically opposite positions at 5 levels to permit determination of both axial load and bending moment.

3.3.2 Test Program and Results

A summary of the experimental programs in clay is given in Table 2.

Considerable effort was expended in developing soil preparation and consolidation procedures to obtain the desired magnitude and profile of S_u versus depth. A total of 15 tests conducted in this effort yielded shear strengths at pile mid-depth and ultimate capacities for the single pile models. The desired S_u was 2.4 ksf (115 kPa) at pile mid-depth but the actual values ranged from 1.6 ksf (91 kPa) to 2.9 ksf (139 kPa). Figure 4 presents ultimate pile capacity of the model piles (at prototype scale) versus S_u. Good consistency is seen between pile capacity and S_u from the model tests as well as with the prototype result.

The correctness of the similitude relations assumed can be verified internally by comparing single pile results from the 50 g, 70 g, and 100 g

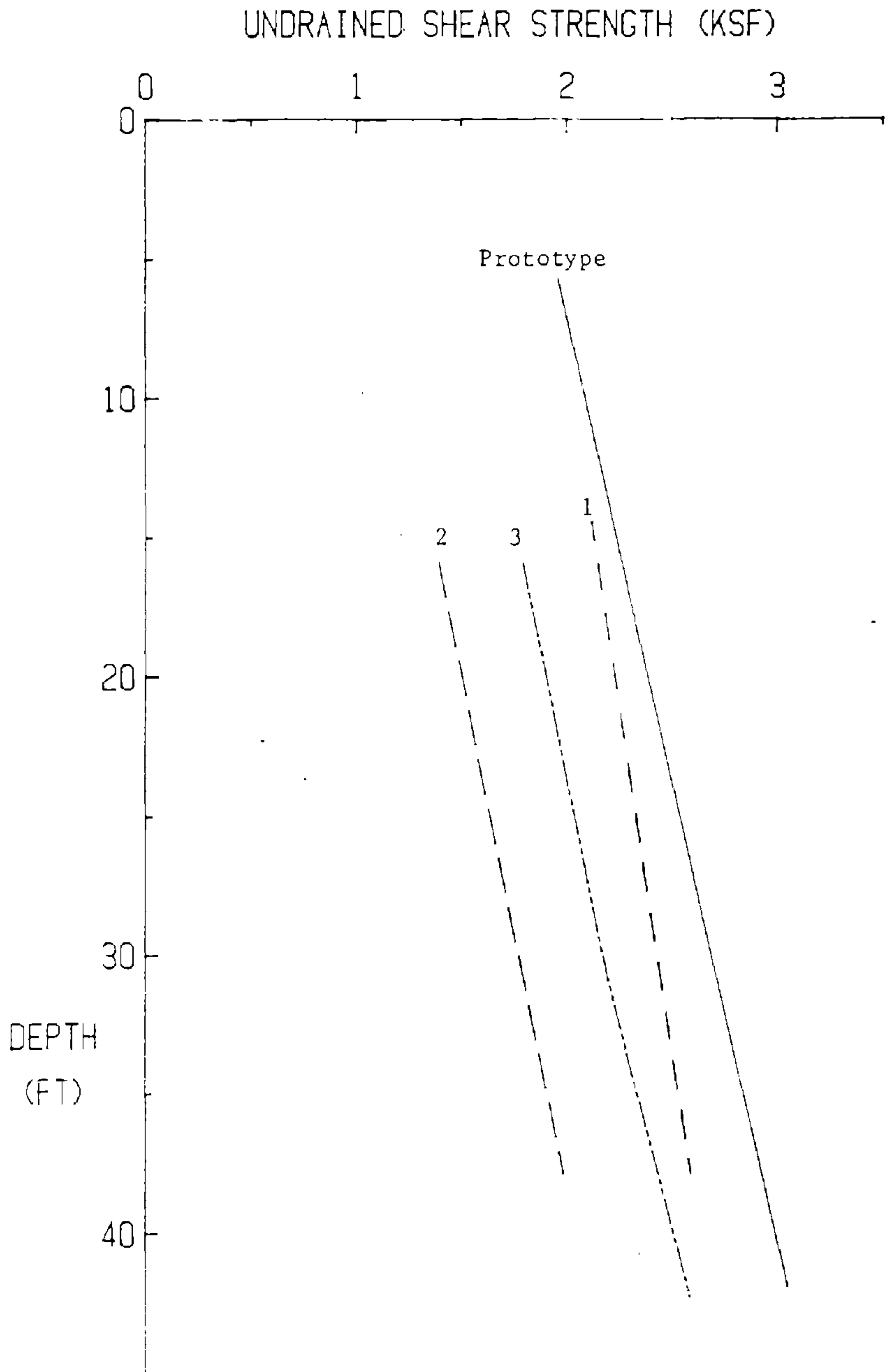


Figure 3. S_u profiles for initial tests (1 ksf = 47.9 kPa; 1 ft = 0.305 m).

Table 2. Summary of testing program in clay

Test No.	No. of vane tests	S_u at pile middepth	g level	Axial load test (single)	Axial load test (group)	Lateral load test	Load transfer (single)	Load transfer (group)	Moment vs. depth (lateral)
1	4	2.3	70	No displacement					
2	3	1.6	70	No displacement			X		
3	4	2.0	70	Good			3/5		
4	1	2.0	70	Good			4/5		
5	1	2.35	70	Good			3/5		
6	1	1.9	70	Good			4/5		
7	2	2.0	70	X					
8[1]	1	2.4	70	Good			3/5		
9[2]	2	2.1	70						
10[3]			70	X		Good			
11[4]	X		70						
12	2	3.0	70	No displacement		Good			
13	1	2.8	70	No displacement			4/5		
14[5]	2	very high	70						
15	1	2.35	70	Good		Good	5/5		4/5
16	1	2.75	50	Good		Good	3/5		3/5
17	1	2.45	70	No displacemt.	No displacemt		3/5	14/20	
18	1	1.7	100	Good	Good				
19[5]	1	very high	70						
20	1	2.35	50	Good		Good	5/7		5/7
21	1	3.2	100	Good	Good				
22	1	1.8	70	Good	Good		4/5	14/24	

Legend:

A blank space indicates that the particular measurement in question was not attempted.

"No displacement" indicates that an ultimate load was measured.

"Good" indicates that the measurement was successfully made

"X" indicates that the measurement failed due to an equipment malfunction.

"3/5" indicates that 3 of 5 strain gauge signals were successfully recorded.

Notes:

1. Specimen No. 8 dried out to such a degree that its results are deemed useless.
2. Specimen No. 9 was devoted to vane testing only
3. The vane apparatus was not available for test No. 10 as it was being serviced.
4. An accident during vane testing prevented further testing on No. 11.
5. Specimen Nos. 14 and 19 were so stiff that no further testing was performed on them.

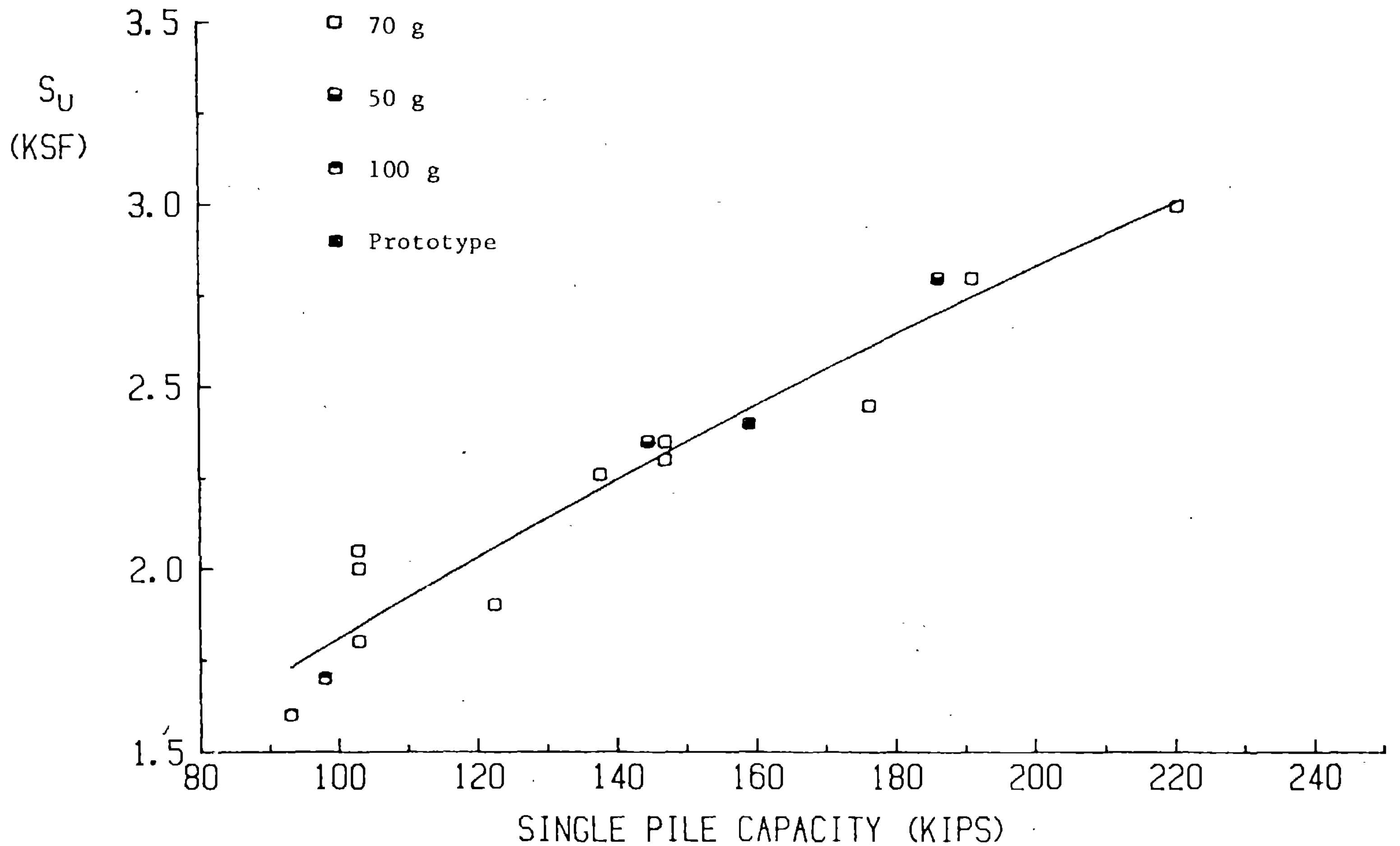


Figure 4. S_u vs. single pile capacity (1 **ksf** = 47.9 KPa; 1 **kip** = 4.45 KN)

tests as shown on Figure 4. These results, when plotted to prototype scale, yield consistent results. The prototype results also fall along the same curve thus verifying the similitude relations.

Pile group tests were conducted at 70 g and 100 g scales (Table 3). The measured group factor from the four groups (two at 70 g and two at 100 g) range from 0.96 to 1.04. The prototype had a measured group factor of 0.98. The higher factor measured in Test No. 21 may have resulted from underground pile deviations from the driven pile grouping. Upon excavation of this model pile group, several piles were observed to have deviated laterally several pile diameters at the pile tip depth.

Axial load transfer data obtained from the strain gauge output was consistent from all tests, both on single piles and pile groups, with the tip taking from 5 to 15 percent of the applied load with the remainder of the load transferred fairly uniformly along the pile length. Figure 5 shows a typical model load transfer curve and the load transfer curve from the prototype single pile test. The difference in the two curves at the pile top may reflect the fact that the prototype piles were driven in holes which had been preaugered for a depth of 10 feet (3 m). Attempts to auger holes at the model scale were unsuccessful.

Load-displacement curves are presented in Figure 6 for single piles and in Figure 7 for group tests. The single pile model results show slightly stiffer results which may reflect the effect of not driving in preaugered holes, as well as a slightly greater axial pile stiffness due to sizing the aluminum model piles to model the bending stiffness of the steel prototype pile. The influence of S_u on capacity is evident in the group load-displacement curves.

Lateral load tests were performed on 5 single pile models. Figure 8 presents measured lateral load deformation curves while Figure 9 presents the pile moment versus depth from one of these tests. The scatter observed in Figure 7 reflects both the varied soil strength and the fact that the lateral test piles did not have a consistent height above the surface.

Table 3. Group test data, clay

Test No. g Level	17 70	18 100	21 100	22 70	Prototype
S_u at pile middepth, ksf	2.45	1.7	3.2	1.8	2.4
Single pile capacity, kips	176	98	330	104	160
Pile group capacity, kips	1520	850	3100	940	1140
Group factor	0.96	0.96	1.04	1.00	0.98

Conversion factors: 1 ksf = 47.9 kPa, 1 kip = 4.45 KN

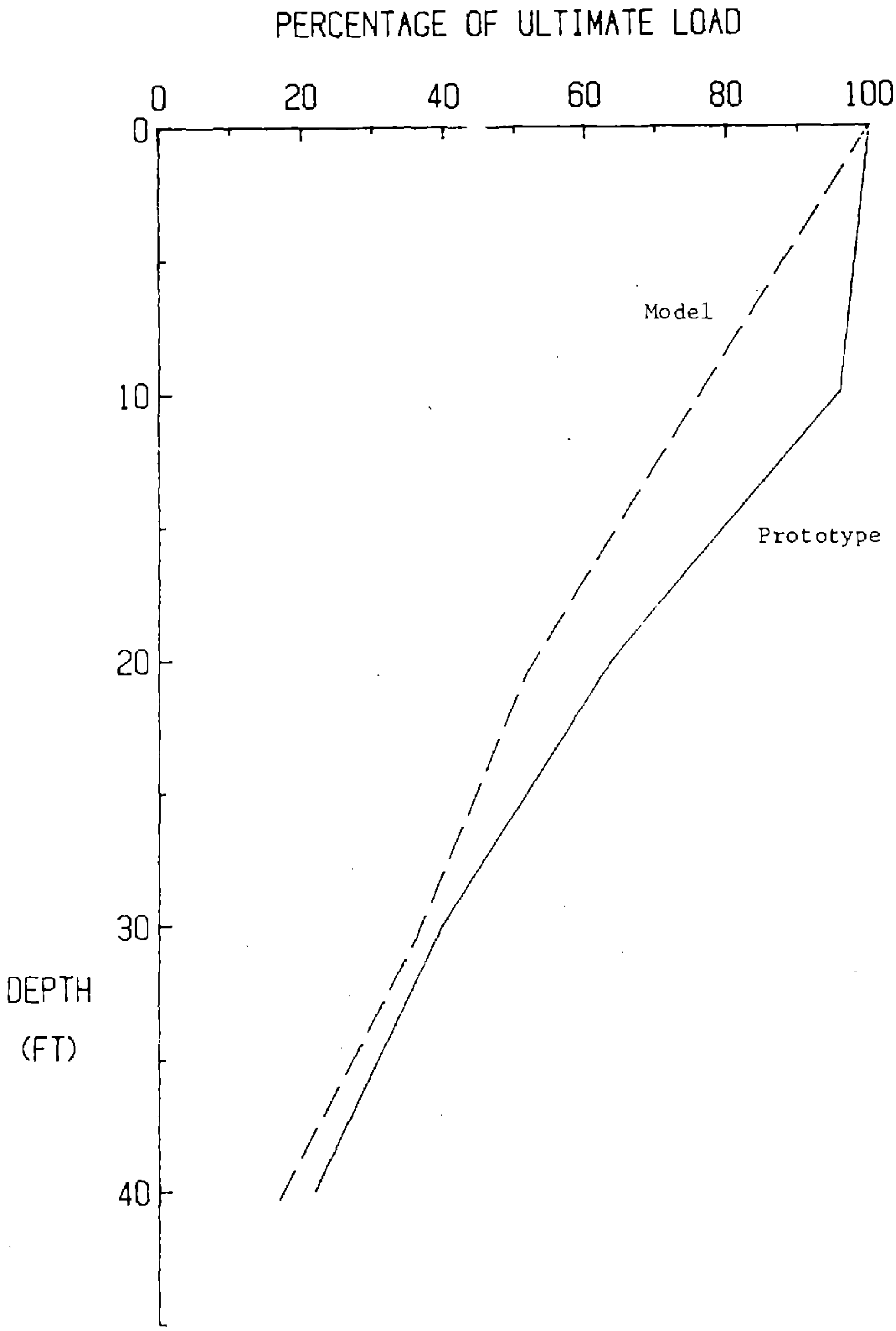


Figure 5. Load transfer curves from model and prototype tests in clay (1 kip = 4.45 KN; 1 ft = 0.305 m).

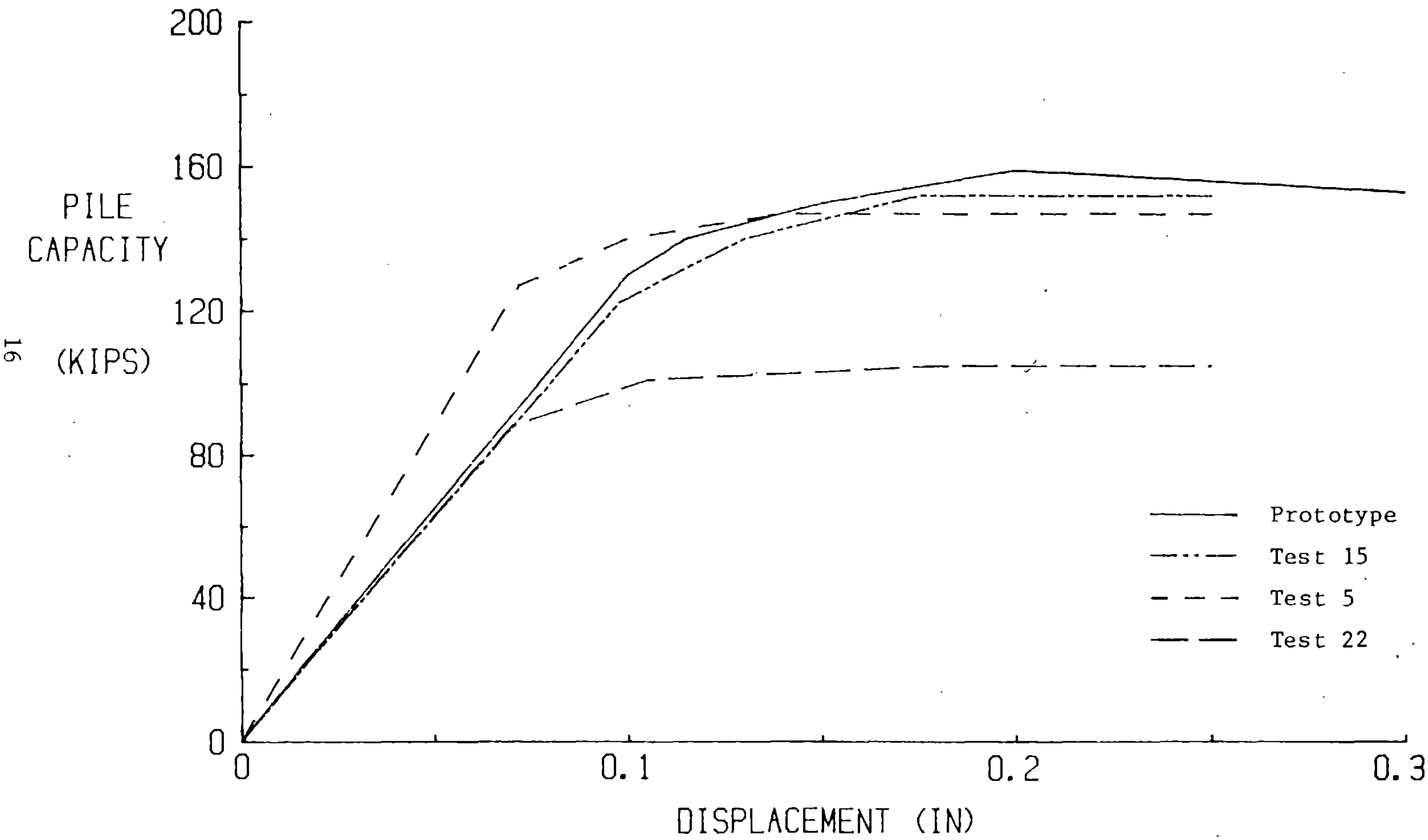


Figure 6. Load-deflection curves, single pile tests in clay (1 kip = 4.45 KN; 1 in = 2.54 cm)

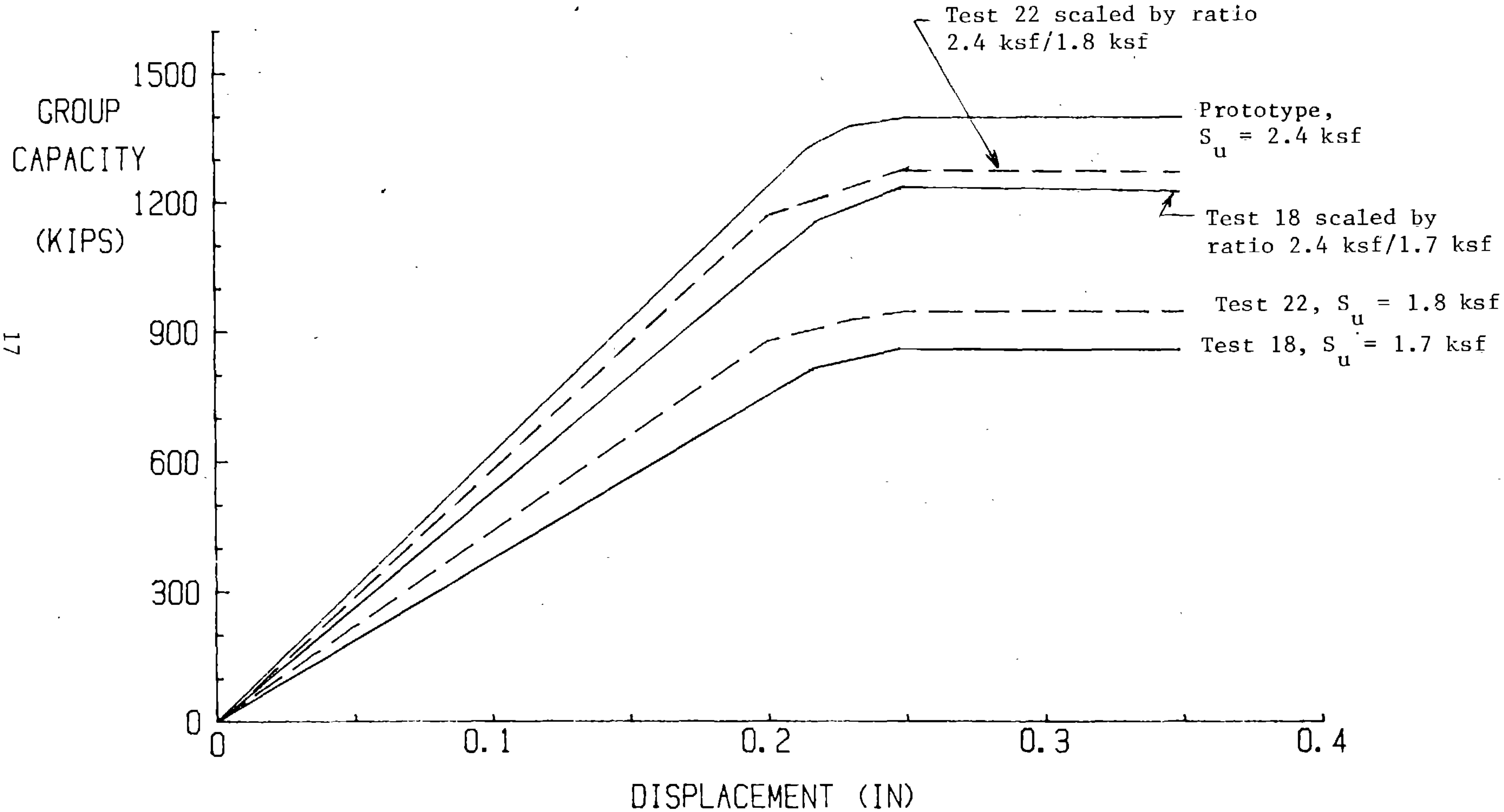


Figure 7. Load-deflection curves, group tests in clay (1 kip = 4.45 KN; 1 in = 2.54 cm)

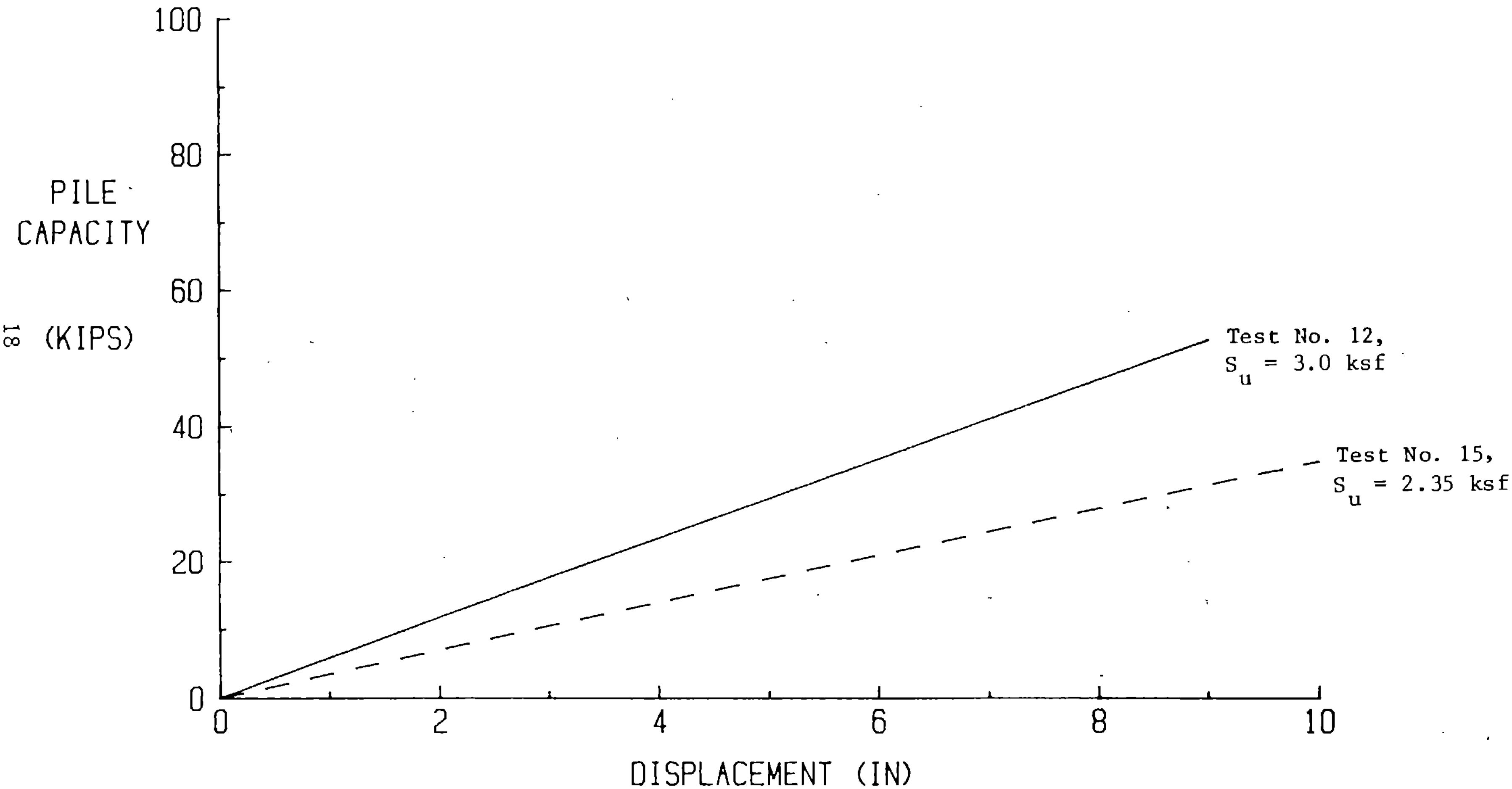


Figure 8. Lateral load-displacement curves, single pile tests in clay (1 kip = 4.45 KN; 1 in = 2.54 cm)

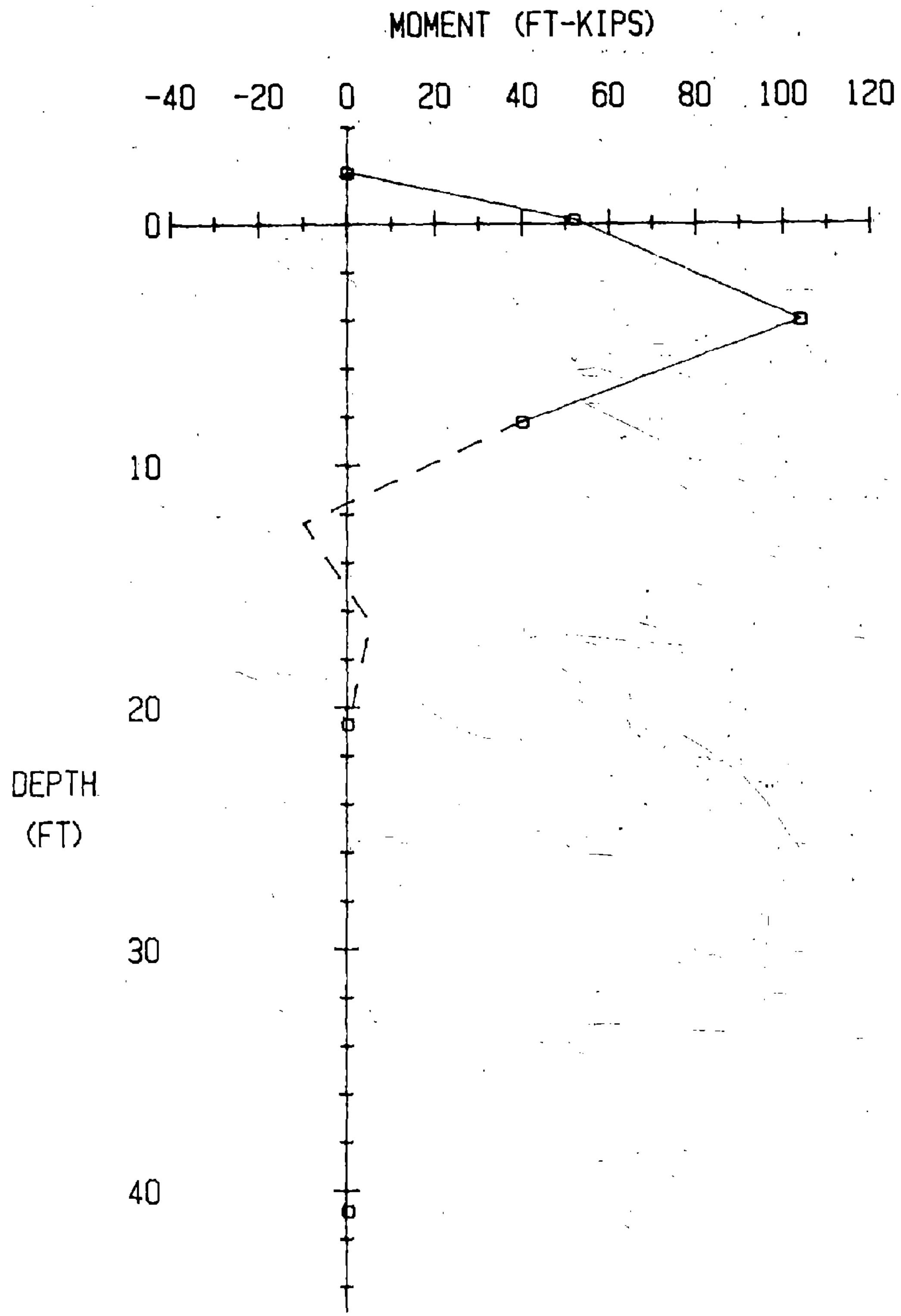


Figure 9. Moment vs. depth curve, model test in clay (1 ft-kip = 1.36 m-KN; 1 ft = 0.305 m)

3.3.3 Comparison to Predicted Results

A pile prediction program called PILGPI developed by Ha and O'Neill (1981) was used to predict model test results based on pile geometry and properties, soil properties and loading conditions. PILGPI uses a finite difference approach to model single pile behavior and the elastic equations of Mindlin to compute group behavior.

Soil behavior for single piles is defined by the f-z curve which relates unit shear stress versus shear deformation along the pile sides and the Q-z curve which relates tip load to tip deformation. The load-transfer data obtained at the 5-strain gauge locations and the load-deformation data at the pile top were used to determine the f-z and Q-z curves for the model pile tests, Figures 10 and 11.

Predicted results using model pile data for input to PILGPI are presented in Figure 12 for single pile test and in Figure 13 for the pile group test.

3.4 Pile Tests in Sand

3.4.1 Soil Preparation and Pile Design

Previous experience with centrifugal testing of model piles in sand at the University of Colorado, Hougnon (1980), indicated the sensitivity of pile performance to soil preparation methods and soil uniformity. Since the prototype soil profile contained 5 different layers, Figure 14, obtaining a representative sample was not possible. Another difficulty with using the prototype soil in the model tests would be the need to remove coarser fractions prior to use in centrifuge model testing.

After reviewing of the prototype soil profile data, it was decided to fabricate a uniform cohesionless model soil with a dry-unit weight of 100 pcf (15.89 KN/m^3) and an angle of internal friction, ϕ, equal to 40° from a commercially bagged, poorly graded, medium sand obtained from the FHWA Soil Laboratory, McLean, VA. The "raining" method was used to place the soil in the test container. The required density was obtained by properly controlling the rate of sand placement and the height of drop.

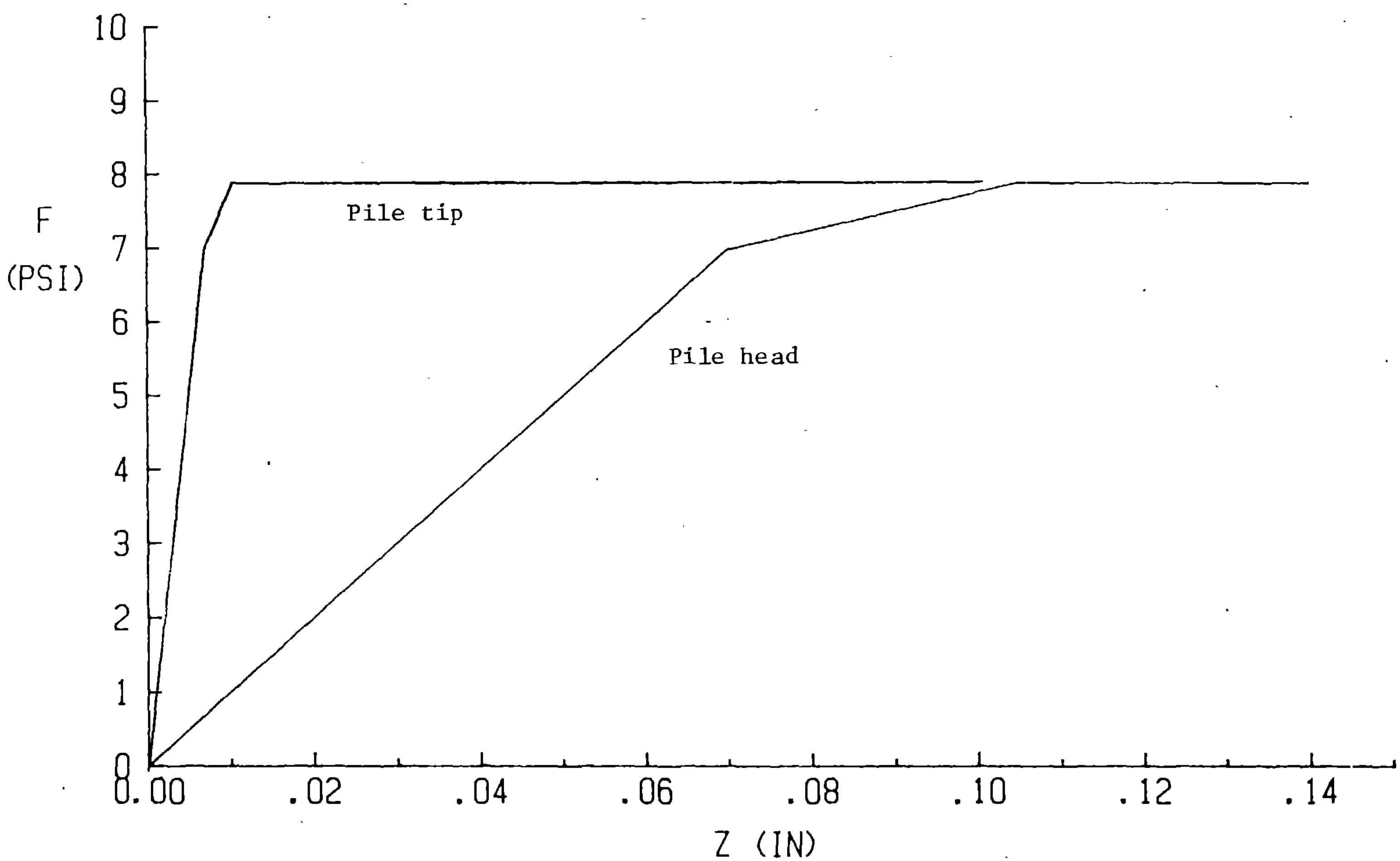


Figure 10. Unit skin friction vs. deflection, pile test in clay (1 psi = 6.9 KPa; 1 in = 2.54 cm)

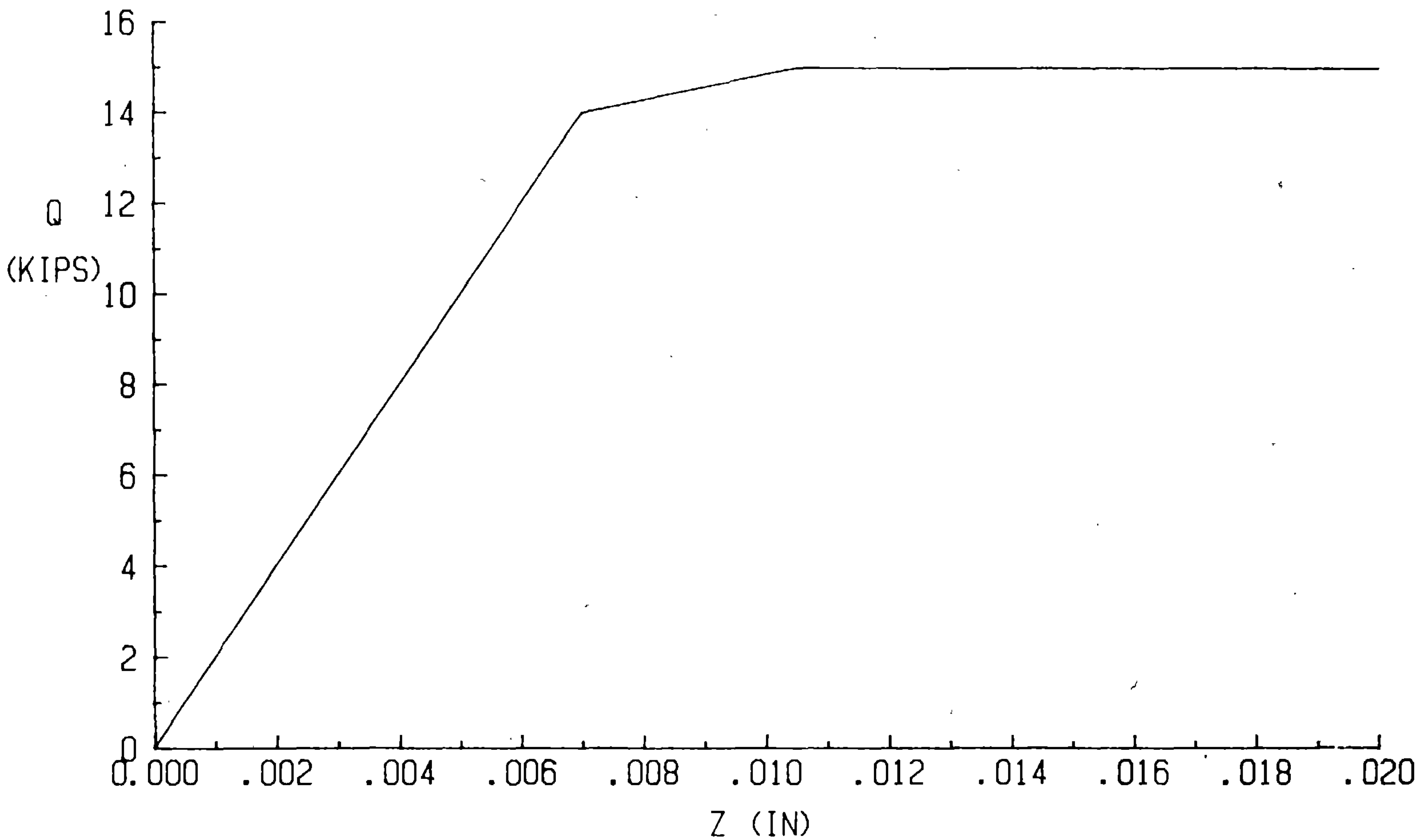


Figure 11. Tip load vs. deflection, test in clay (1 kip = 4.45 KN; 1 in = 2.54 cm)

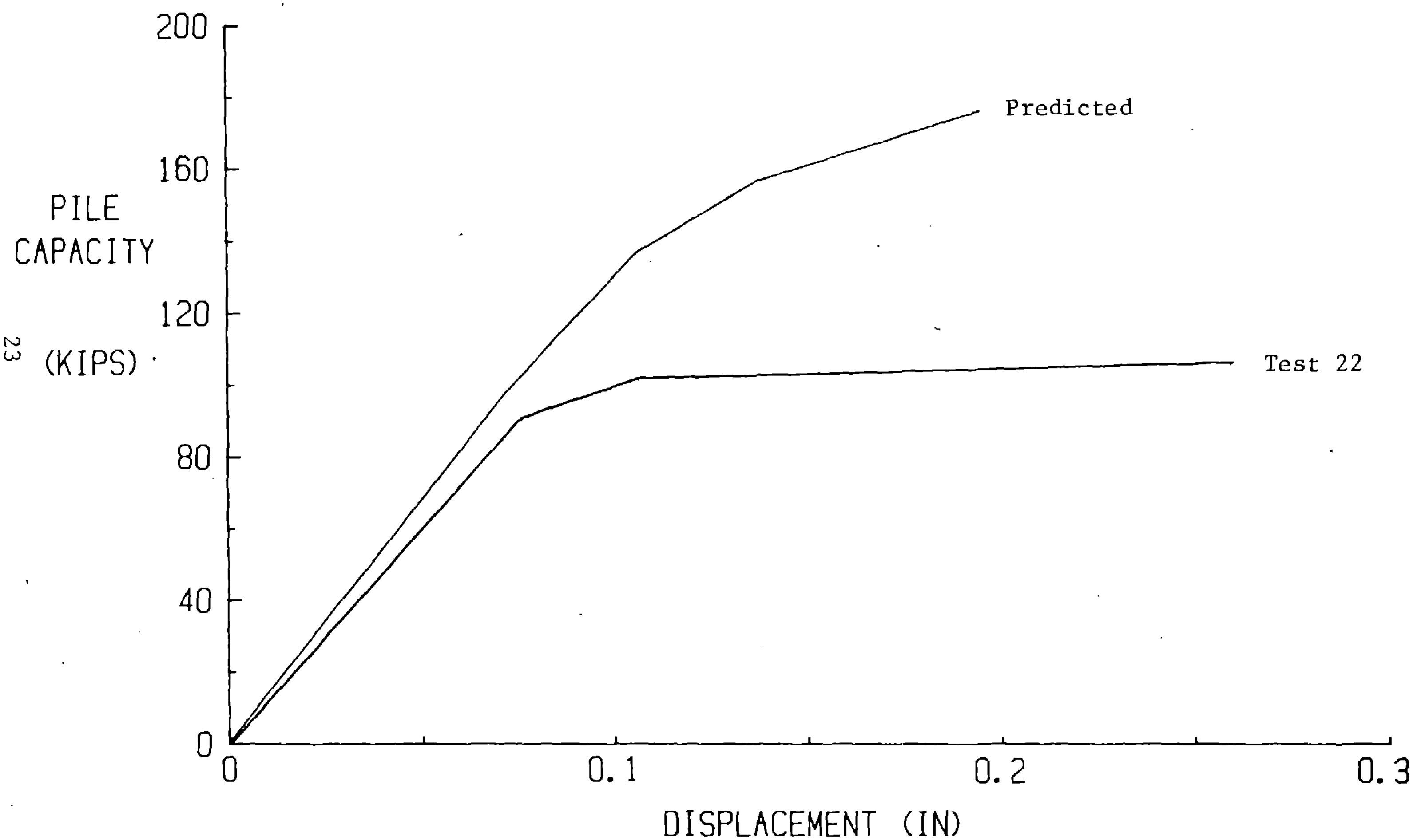


Figure 12. Single pile load-displacement as predicted by PILGP1 (1 kip = 4.45 KN; 1 in = 2.54 cm.).

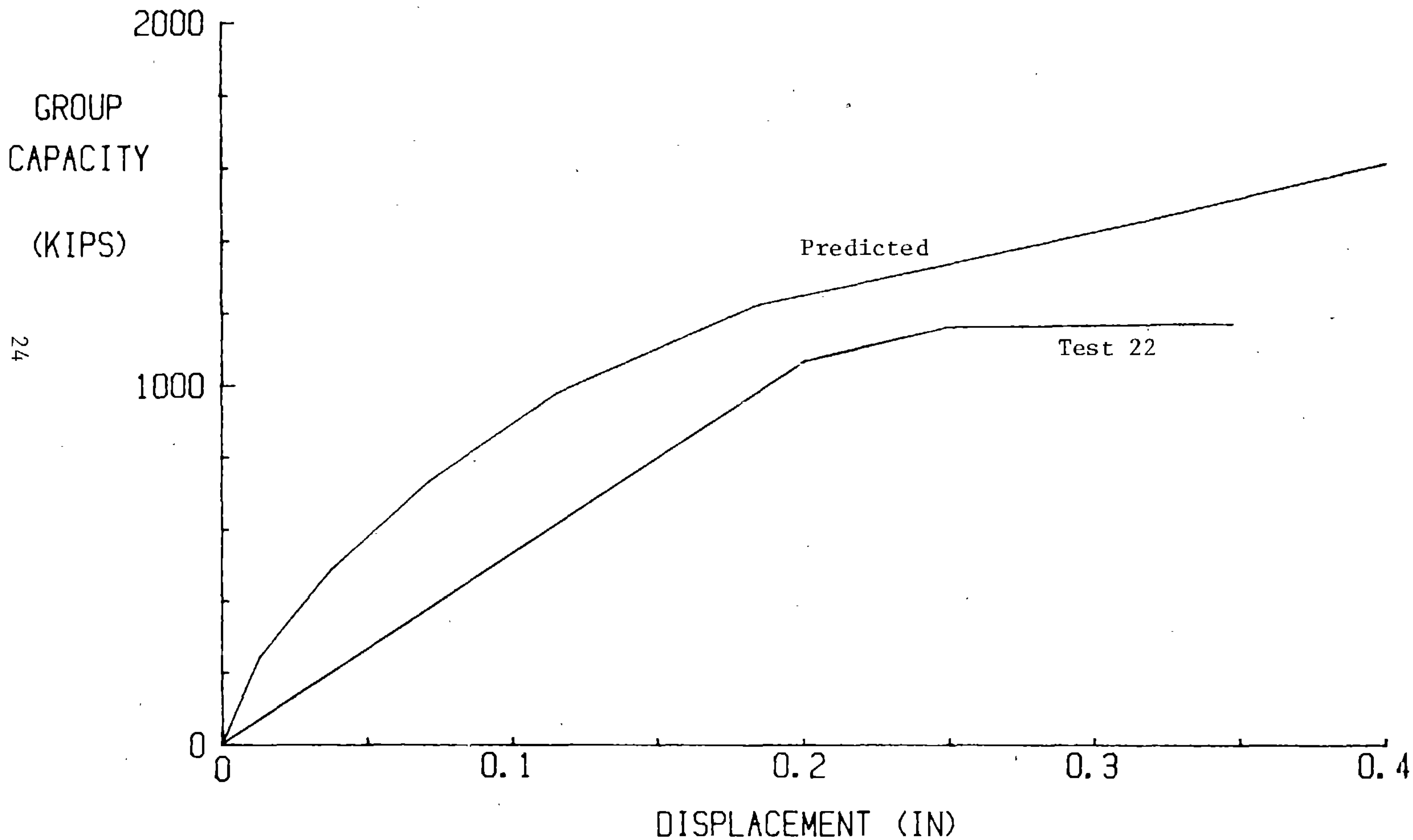
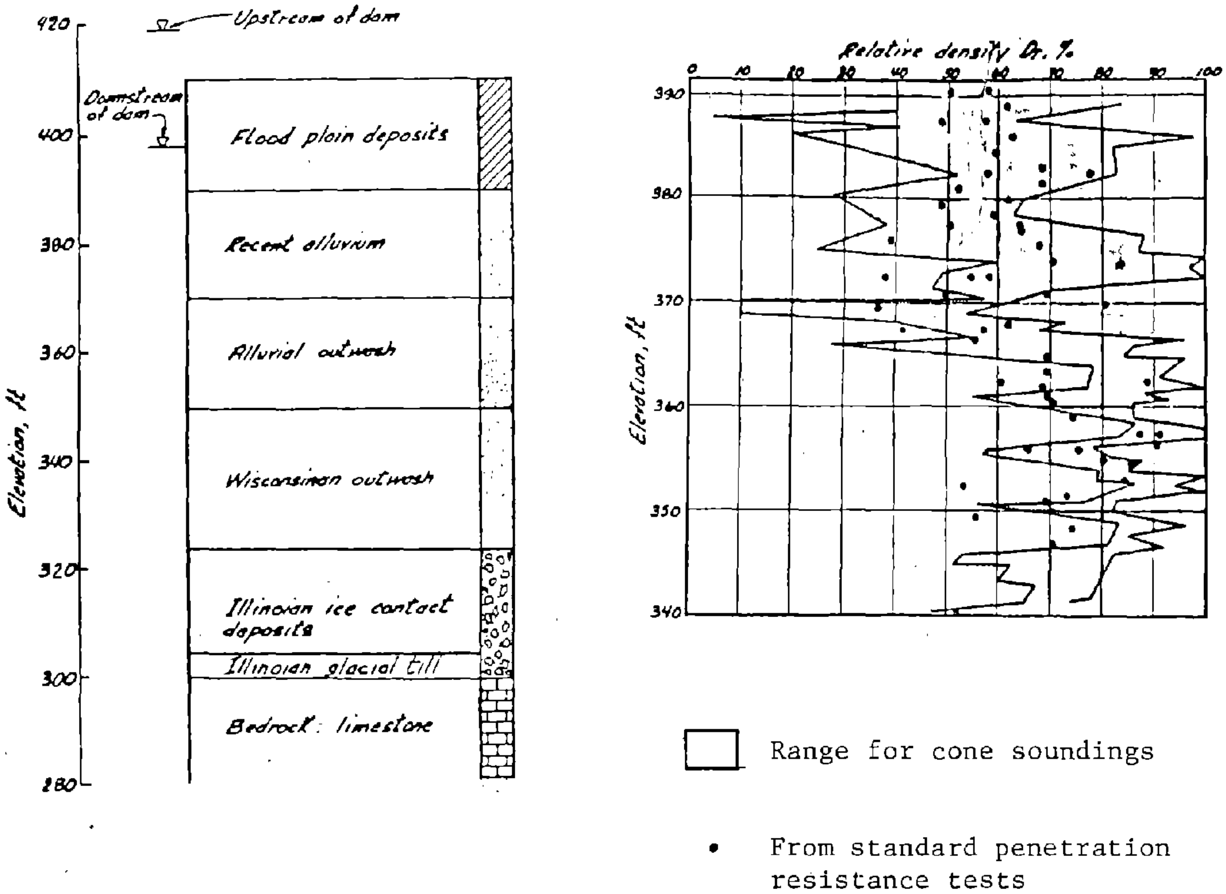


Figure 13. Pile group load-displacement as predicted by PILGP1 (1 kip = 4.45 KN; 1 in = 2.54 cm).



a. Generalized subsurface profile in vicinity of test area.

b. Relative density profile before timber pile installation.

Figure 14. Site conditions, Lock & Dam No. 26 (Ref. Woodward-Clyde Report, 1979).

The prototype piles were tapered wood piles, 40 ft (12.2 m) in length with a top diameter of 14 inches (35.6 cm) and a tip diameter of 10 in (25.4 cm). Model piles were fabricated to a correct dimensional scale for 50 g, 70 g and 100 g scales using wood having the same modulus as the Douglas fir prototypes.

The top portion of the tapered wood model piles were strain-gauged to measure the applied load. Each pile was calibrated individually. An aluminum pile at a 70 g scale was fabricated with five levels of strain gauges mounted internally. This pile, which had a uniform section with depth, was used to determine the load-transfer and tip loading of a pile in the test sand.

Single piles and the piles in the group tests were installed in-flight under full scale gravity conditions. A template on the soil surface was used to guide the piles and maintain proper spacing for the pile groups. After all piles in the 2 x 4 pile group had been separately driven, a rigid aluminum pile cap was installed to tie them together. This cap was made in three parts and was clamped to the model piles. After installation of the cap, the model piles were driven (0.2 in) as a group to minimize and disturbance produced by the pile cap installation.

3.4.2 Test Program and Results

The test program in sand is summarized in Table 4. Individual aspects will be discussed below.

The effect of in-flight installation vs. 1 g installation on single model piles was investigated at both 50 g and 70 g scale levels. Figure 15 shows the results of the 70 g test. The 50 g test showed similar behavior. These results show that it is necessary to have the proper simulation of the soil stress field both during the driving and testing of model piles. As it was necessary to stop the centrifuge to reposition the driving mechanism for each pile in the pile group, Test Series 2 was conducted to investigate the effect of such stoppage on subsequent pile performance. The results showed no significant difference in pile capacity due to centrifuge stoppage.

Table 4. Summary of testing program in sand

Test Series	Test Group No.	Purpose	Gravity Level g s	Test Condition Unit Weight pcf	ϕ deg.	Ultimate Load Kips	Special Point of Interest	Comments
1	1	Effect of in flight	70	98.23	42.5	520		
	2	installation vs.	50	103.61	46	1060		
		1g installation						
2	1	Effect of interuption between installation and load test	70	98.23	42.5	520		
3	1	Modeling	50	98.23	42.5	470–560		
	2	of	70	98.23	42.5	520		
	3	Models	100	98.23	42.5	540		
4	1		70	92.78	40.45	190		In 4.8 test group, at 13 inch left for installation to be completed, two of the piles broke at approximately 1.2 inch below the soil level.
	2		70	93.54	40.7	250		
	3		70	93.99	40.8	260		
	4	Parametric	70	95.70	41.35	340		
	5	study	70	96.96	41.85	470		
	6		70	98.23	42.5	520		
	7		70	103.16	45.7	890		
	8		50	103.61	46.0	1060		
							Efficiency	
5	1	Group	70	93.99	40.8	2800	1.32	The weight of the cap (103.8 Kip) is not included in the load-settlement curves but it is included in the calculation of efficiency factors.
	2	Tests	70	95.70	41.35	3100	1.15	
	3		70	96.96	41.85	4360	1.16	
	4		70	98.29	42.5	4790	1.15	
						Tap. Str.	Ratio: Tap/St.	
6	1	Tapered	70	95.32	41.25	— 270	-	The straight pile had the same diameter as the mid-height diameter of the tapered pile.
	2	vs.	70	95.70	41.35	340 290	1.15	
	3	Straight piles	70	98.23	42.5	520 415	1.24	
						Dry Sat.	Ratio: Dry/Sat.	
7	1	Saturated	70	93.99	40.8	2800 2350	1.19	Test 7.1 is a group test, and at the end of the test water level was 5.2 ft below soil level. Test 7.2 was a single pile test, and at the end of the test water level was 7.0 inch above soil level (prototype).
	2	Tests	70	96.96	41.85	470 290	1.61	
8	1		70	92.78	40.45	16.8		The lateral deflections were measured at 5 ft above the soil level. The ultimate loads at 5.5 inch of deflection (prototype).
	2	Lateral Load	70	93.54	40.7	19.5		
	3	Tests	70	98.23	42.5	24.0		
	4		70	103.61	45.7	27.0		
9	1	Instrumented Aluminum pile Test	70	95.32	41.25	170		

1 Kip = 4.45 KN, 1 pcf = .158 KN/m^3

All quantities expressed in prototype scale

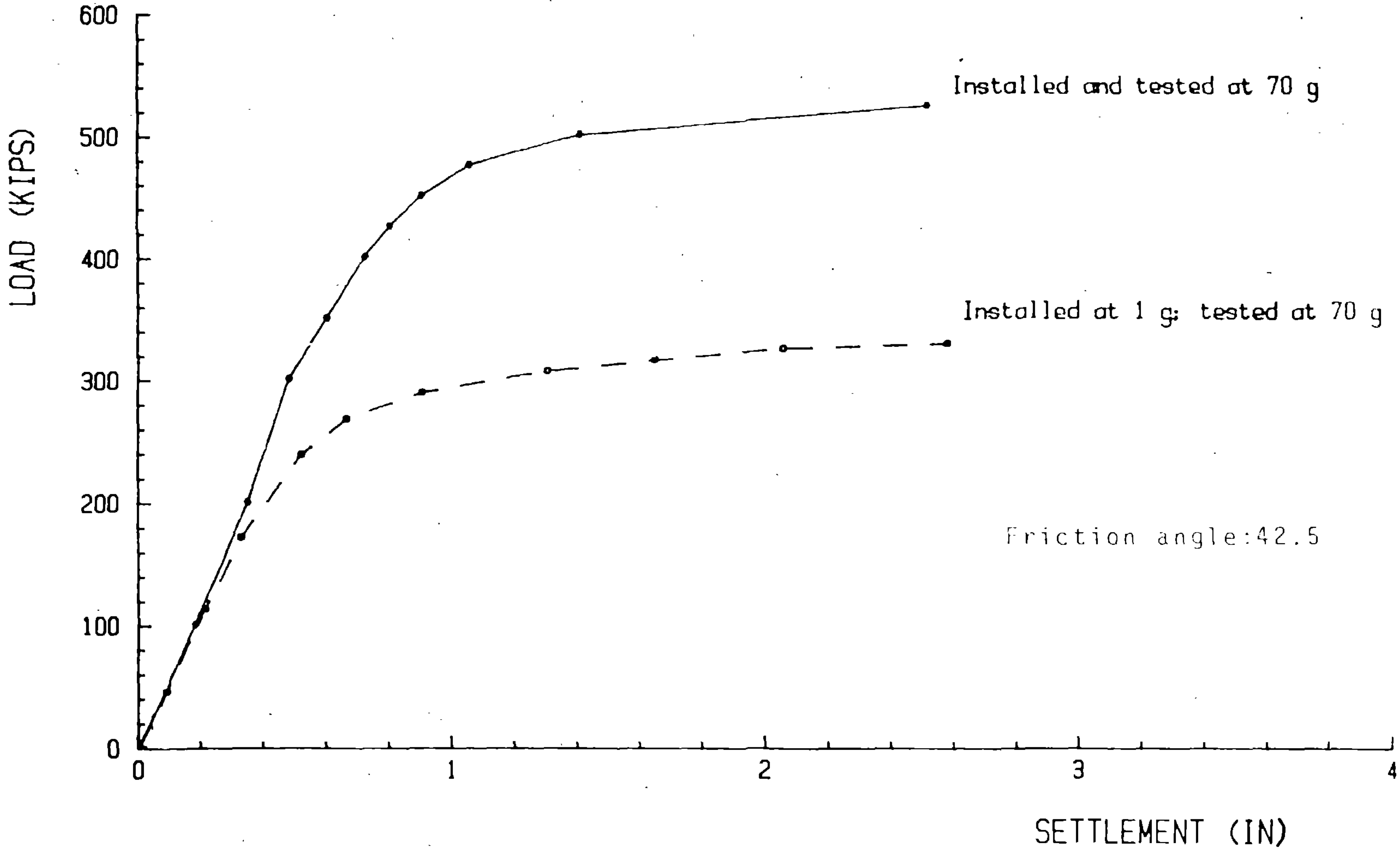


Figure 15. In-flight vs. 1 g installation of 1/70 scale pile in sand.

To verify the internal consistency of the similitude relations over a range of model scales, pile tests were conducted at 50 g, 70 g, and 100 g model scales. The load-settlement curves from these tests, converted to prototype scale, are shown in Figure 16. Five of the six curves are in good agreement, thus verifying the similitude assumptions.

To investigate the effect of angle of internal friction on pile performance, seven 70 g tests and one 50 g test were conducted on model piles in soil with density ranging from 92.2 pcf (14.66 KN/m^3) to 106.3 (16.37 KN/m^3). The ϕ angle for these soils ranged from 40.45° to 46.0°. The test results, expressed in prototype scale, are plotted vs. ϕ angle in Figure 17. The sensitivity of pile capacity to ϕ angle is evident.

To investigate the effect of straight vs. tapered piles, straight wooden piles having the same diameter as the mid-depth of the tapered piles were fabricated and tested. The tapered pile had 24 percent greater ultimate capacity than the straight pile for a soil density of 97.7 pcf (15.53 KN/m^3) and a 17 percent increase for a density of 95.2 pcf (15.12 KN/m^3). This suggests that the effectiveness of pile taper may be density dependent.

Four group tests were successfully conducted. Driving load records from the strain gauges at the top of the pile were obtained from two tests. Figure 18 shows one of these records where the influence of the driving order is clearly seen.

After installation of all piles and the pile cap, the individual pile loads were monitored as the pile group was loaded. Table 5 presents the individual capacities as well as the total group load. Although in all tests Pile No. 1 took less load than other piles, the influence of driving order is not pronounced. The group efficiencies computed for the four groups are presented in Table 5. Except for the test in the lowest density soil which had an efficiency of 1.32, the other tests all had an efficiency of 1.15.

One single pile and one group test were conducted to investigate the effect of soil saturation on pile performance. Because of test problems, the measured load from the group tests were not reduced by the ratio of $\gamma_{buoyant}/\gamma_{dry}$. These problems, which included swelling effects on the wooden

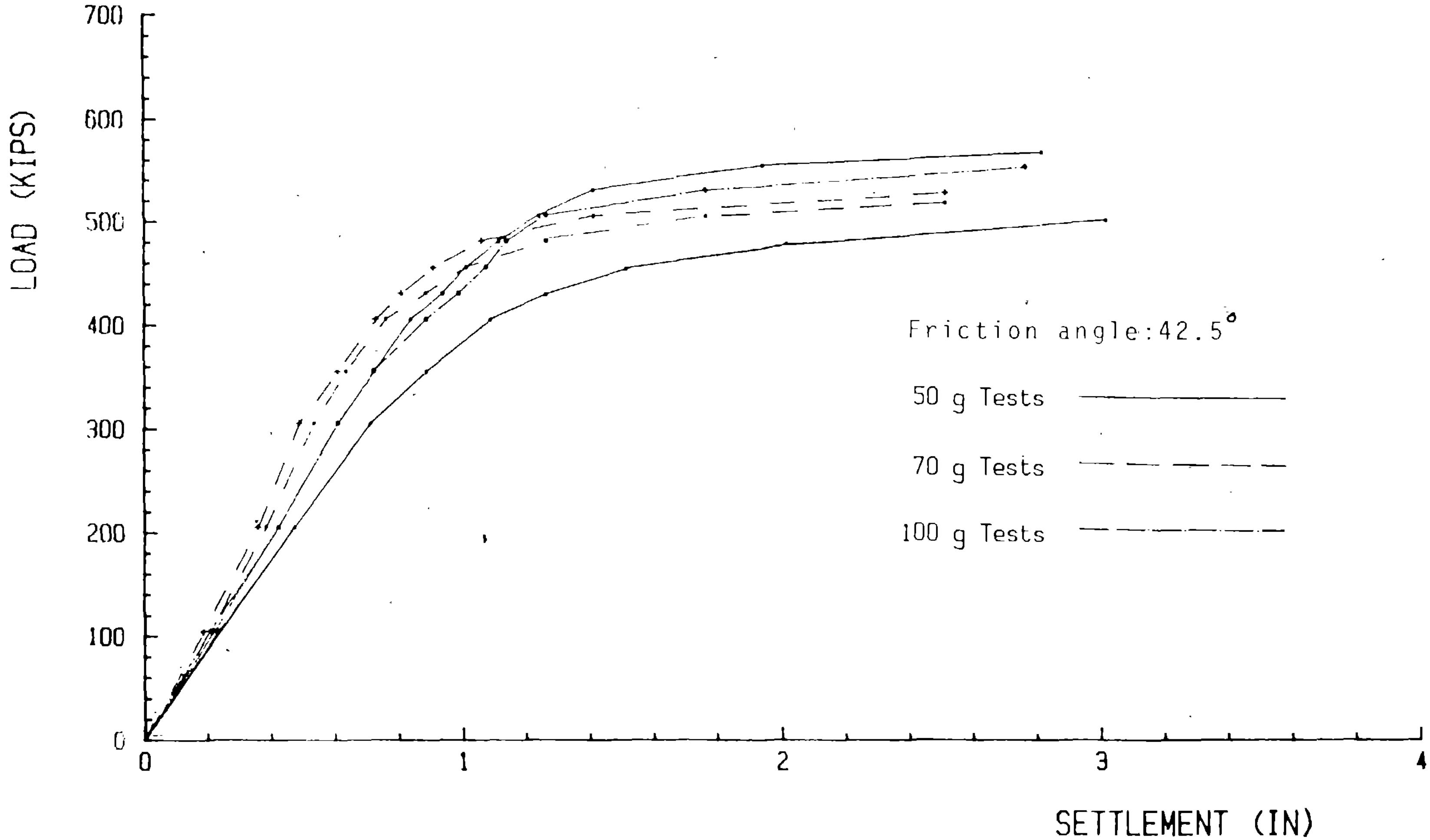


Figure 16. Load-settlement curves in sand.

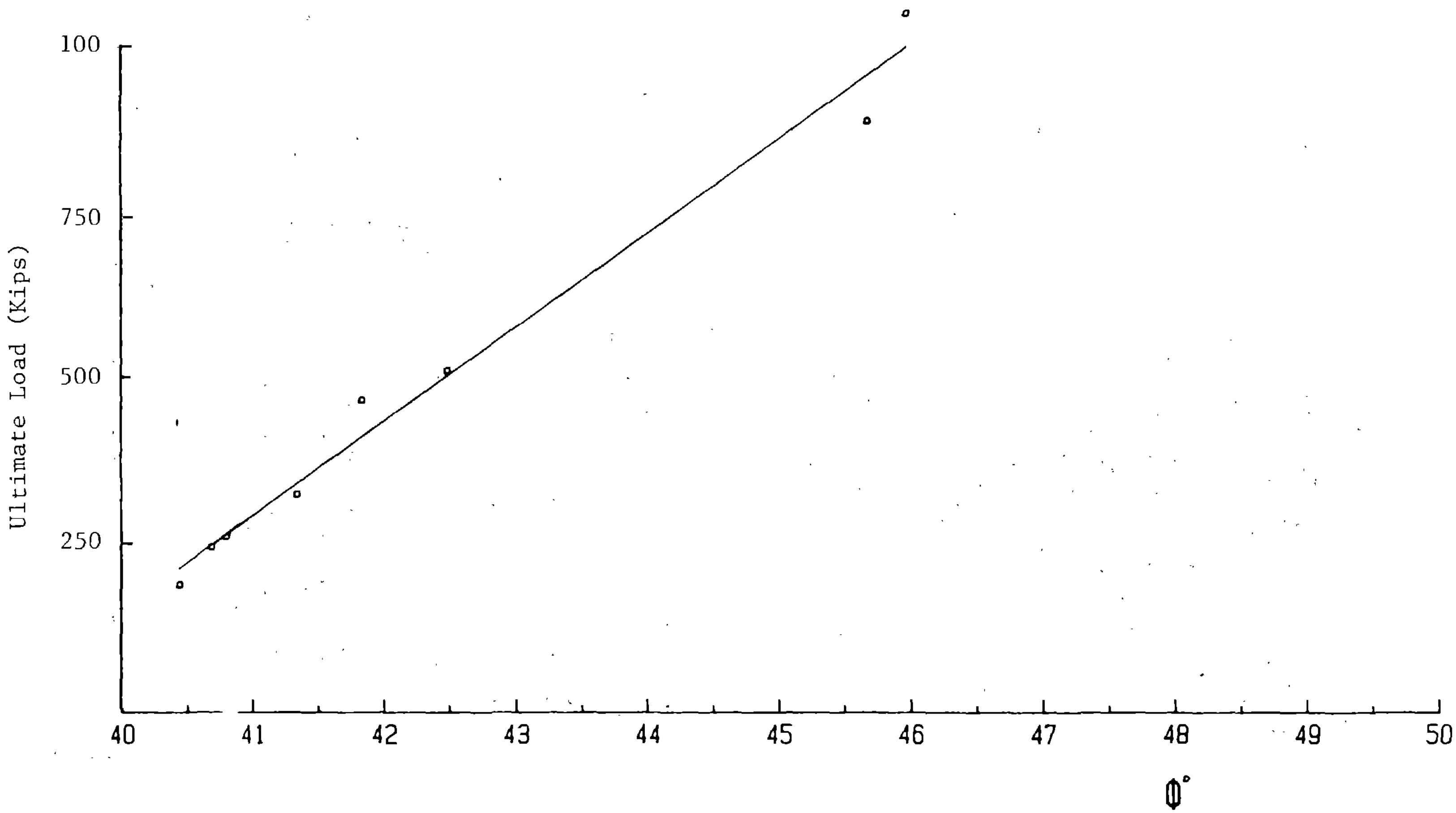


Figure 17. Pile capacity vs. soil friction angle.

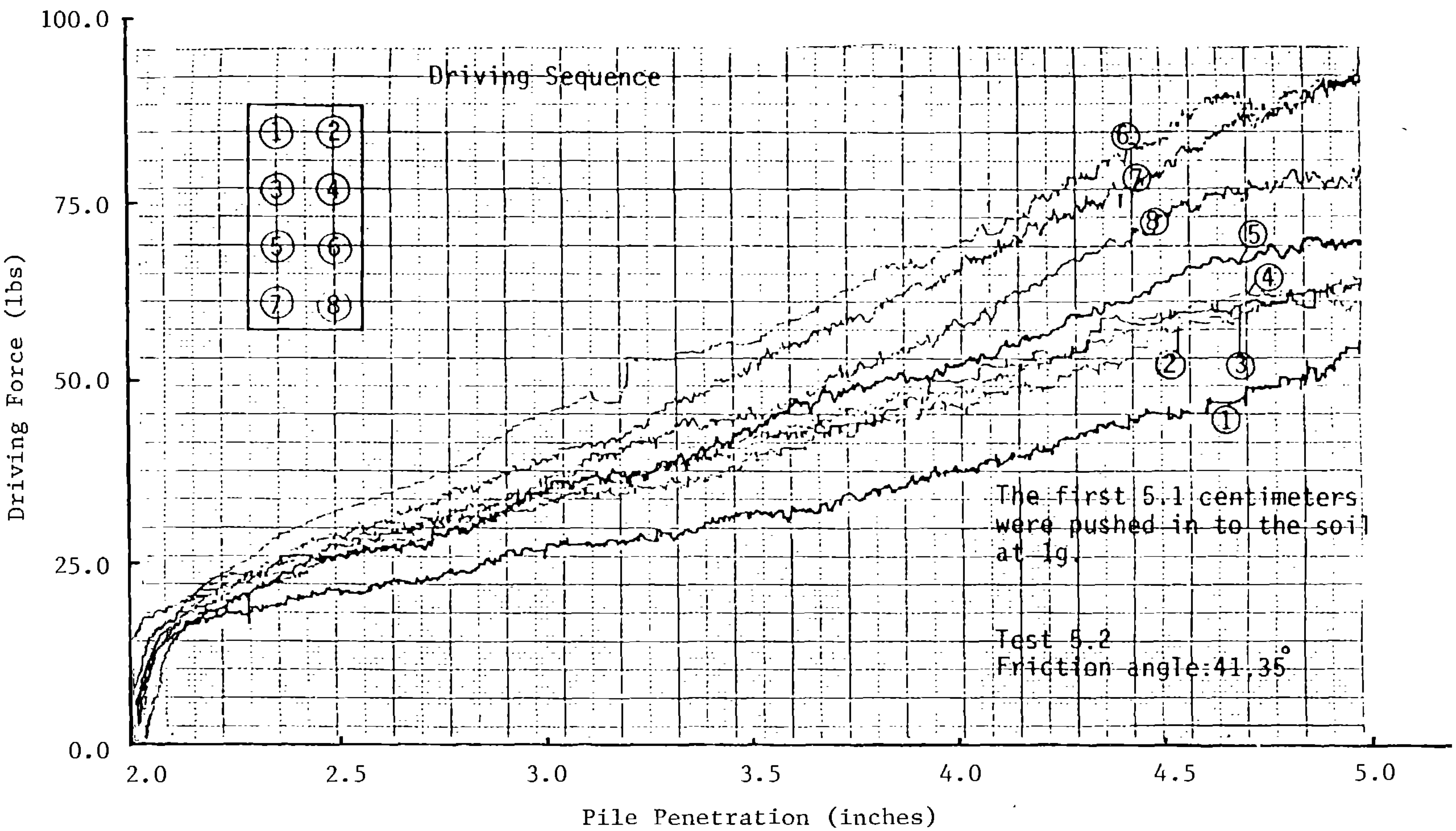


Figure 18. Influence of driving order, pile group in sand.

Table 5. Group test data, sand

	Individual Pile Capacities (lbs)			
Pile No.	Test 5.1	Test 7.1	Test 5.2	Test 5.4
1	54.7	57.6	57.1	102.4
2	67.1	59.1	72.8	103.0
3	69.2	60.2	74.4	114.9
4	70.8	*	73.6	114.9
5	*	58.4	75.6	144.4
6	76.6	63.5	82.4	114.2
7	70.8	58.6	72.8	142.6
8	77.0	64.0	80.8	113.5
Total load from individual piles	-	-	589	952
Total load from load cell	550	482	600	1016

* Gages did not work

Note: Weight of the cap not included

Scale: 70

pile, were corrected and the test rerun on a single pile, Figure 19. The reduction of both capacity and stiffness is clearly seen.

Four lateral load tests were conducted on 70 g scale piles. Figure 20 shows two load cycles from Test No. 8.1 with the second load-deflection curve stiffer than the first. Since the model piles were pushed into the soil a distance of 2.0 inches at 1 g before being driven to depth at test gravity, it is possible that disturbance in this upper region, which provided most of the resistance to lateral deformation, might have affected results.

The aluminum model pile was used to obtain the load transfer curve shown in Figure 21. Approximately 73 percent of the total resistance was provided by tip resistance. The load-transfer along the pile length is seen to be uniform.

3.4.3 Comparison to Predicted Results

The finite difference program PILGP1 of Ha and O'Neill which uses Mindlin's equations to compute group interaction was used to predict model pile performance. Because the available data from the aluminum pile test provided only ultimate values for side shear (f) and tip loads (Q), it was necessary to make a number of computer runs in which both the shape of f-z and Q-z curves and the magnitude of displacement at ultimate value, z_c, were varied. The best fit of predicted load and deformation to measured was obtained when z_c for the side shear stress (f) was 0.35 inch (prototype scale) while the value of the tip load (Q) was 0.65 inch (prototype scale) or 6 percent of the pile tip diameter. The shape of the Q-z curve was modified from that recommended by Ha and O'Neill to obtain a closer fit. The same f-z curve was used for the full length of the pile except for the top 5.8 feet (prototype scale) for which zero shear strength was assigned. The measured model vs. predicted pile response is shown in Figure 22 where the Q-z and total shear force versus z curves are also shown. Good agreement with the initial pile response is achieved, however, the curves begin to diverge after the side shear has reached ultimate value.

The field test was conducted in saturated conditions in a soil mass with variable properties as shown in Figure 14. The measured ratio of 1.61

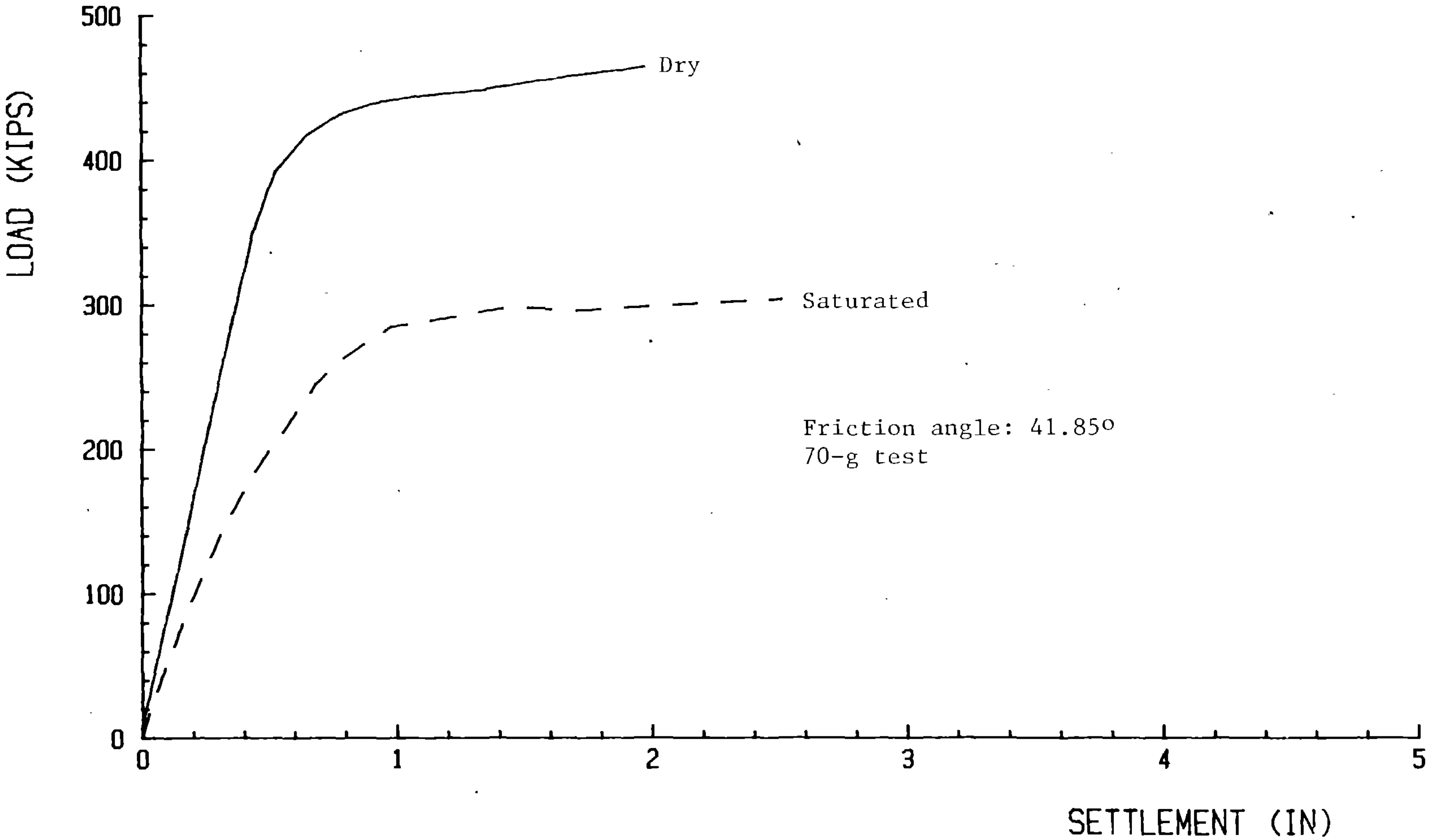


Figure 19. Effect of soil saturation on single pile load-settlement curve.

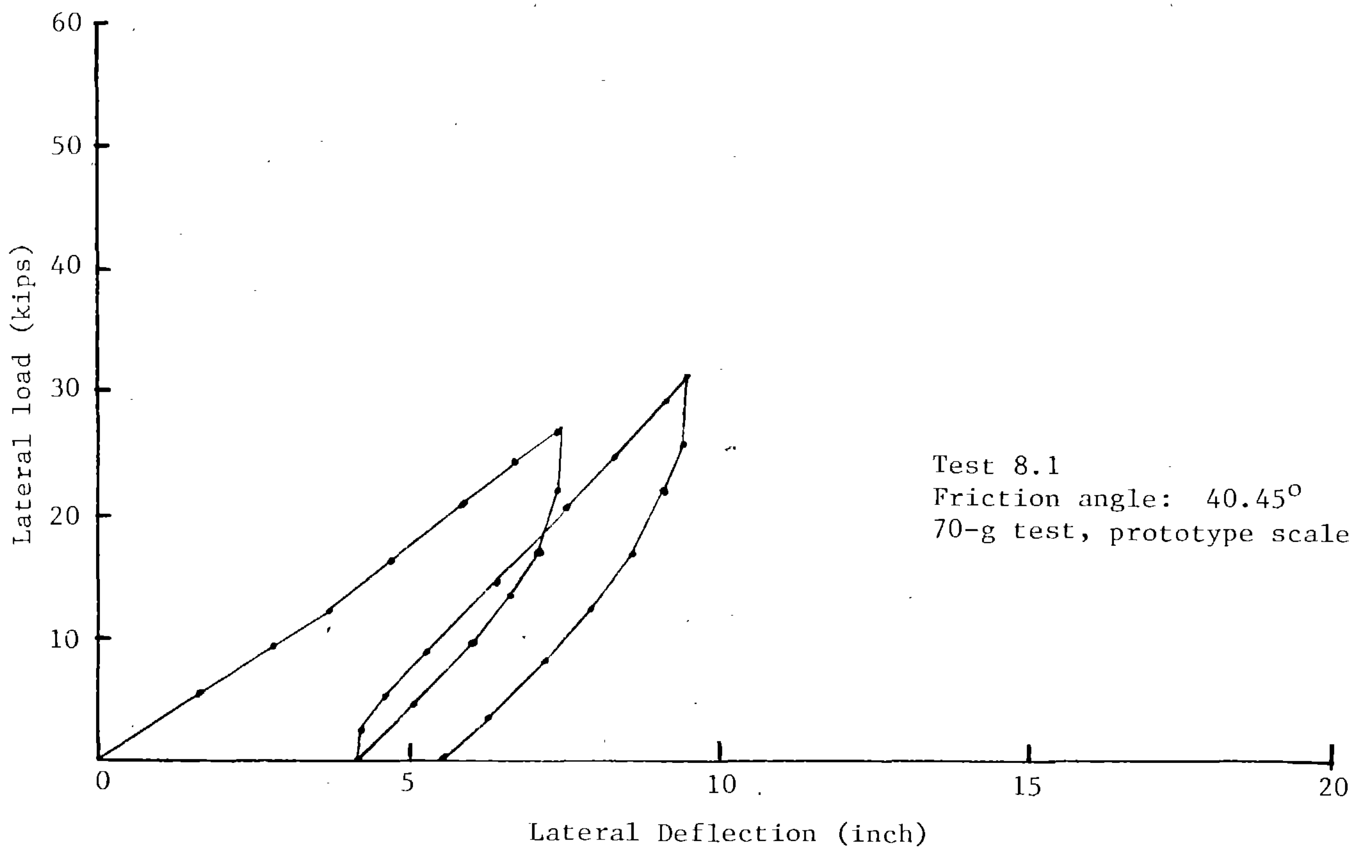


Figure 20. Lateral load-deflection curve of single pile in sand.

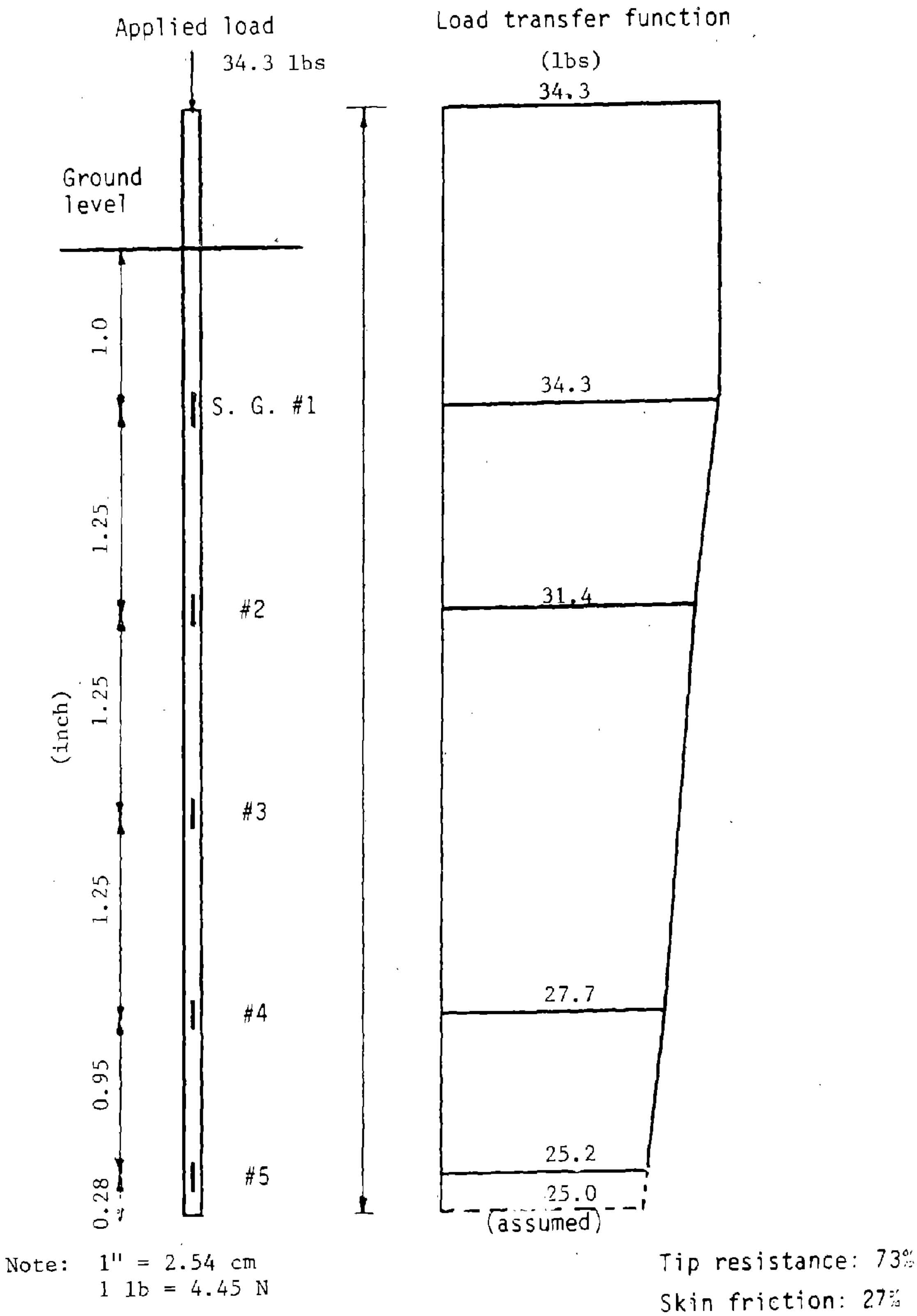


Figure 21. Load-transfer curve, aluminum pile in sand.

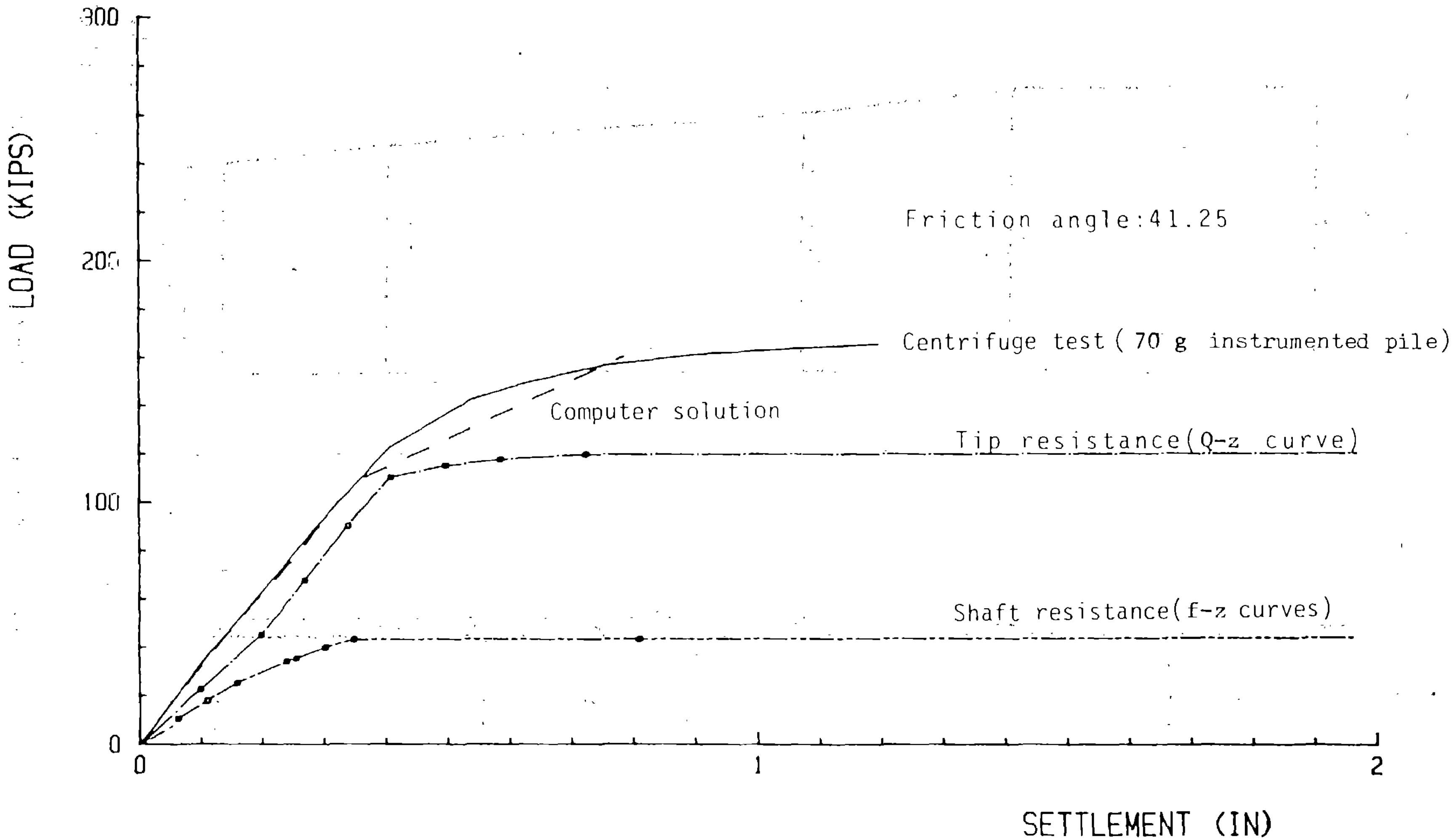


Figure 22. Measured model vs. predicted single pile response in sand (Prototype Scale).

for dry to saturated load from Test No. 7.2 was applied to the results of tests 4.1, 4.2, and 4.4 from the parametric series to obtain an estimate of saturated performance. These results are shown plotted in Figure 23 where the best fit is obtained for $\phi = 41.35^{\circ}$. The extreme sensitivity of model pile capacity to the ϕ value is evident. The field pile was driven by a dynamic hammer while the model pile was installed by a steady force or jacking procedure. It is speculated that this difference in pile installation may produce different types and magnitudes of soil disturbance in the cohesionless soil that may significantly affect pile behavior.

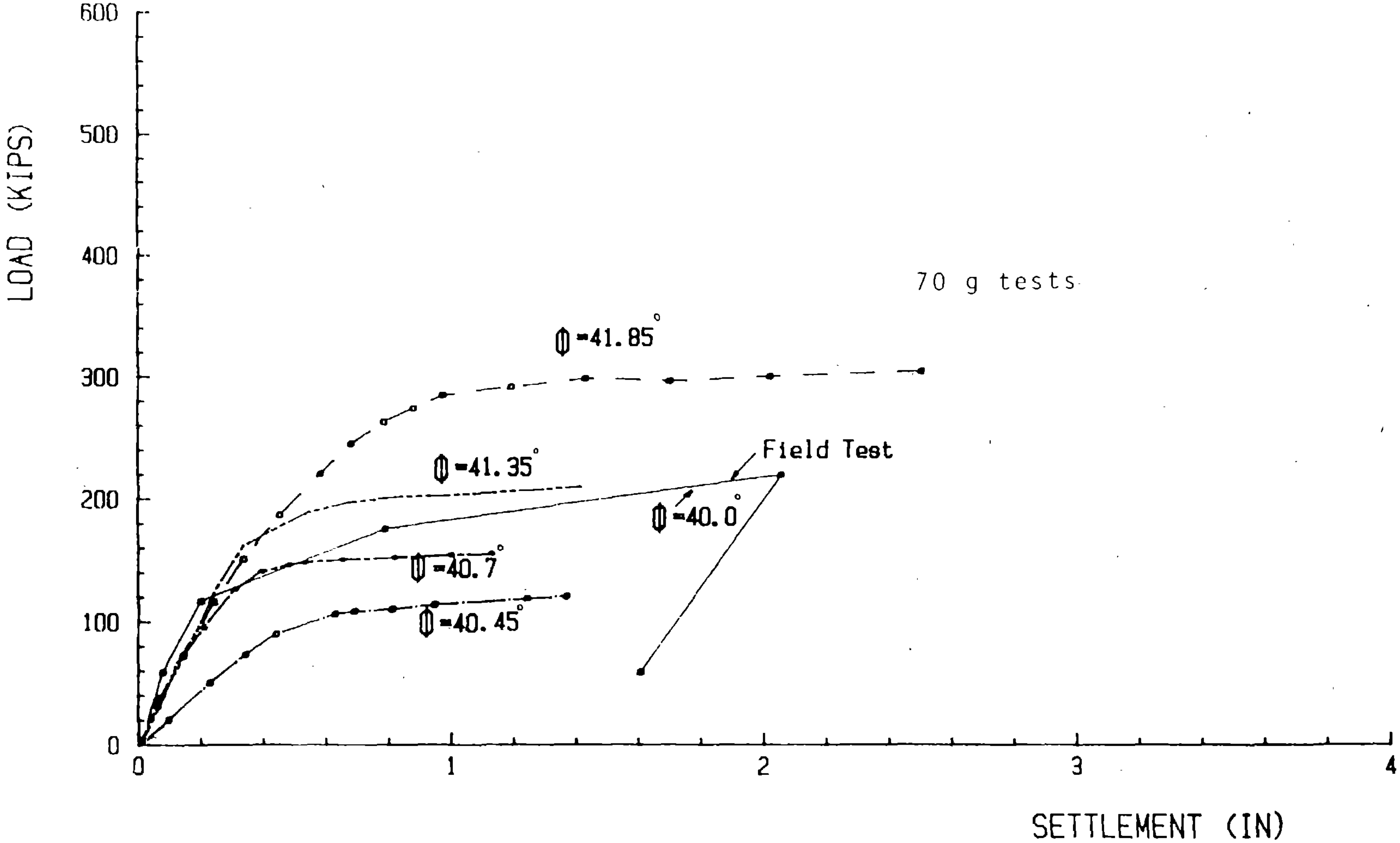


Figure 23. Comparison of extrapolated model response vs. prototype pile response in sand.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

A comprehensive research program has been performed on the use of the geotechnical centrifuge to model single piles and pile groups. Axial and lateral tests have been conducted on single piles and axial tests on pile groups both in sand and clay soils. Load-deflection and load-transfer data have been obtained. Details of the experimental programs are given in Volume II for the tests in sand and in Volume III for tests in clay. Important conclusions and recommendations generated by this research effort are presented below.

4.1 Conclusions

The results from both the sand and clay tests verify the assumed similitude relations as evidenced by the modeling of model results. This important conclusion gives the required credibility to the results obtained on pile behavior of the influence of various soil and pile factors. The modeling of models results when scaled to prototype scale gave a reasonable prediction of field test results considering the variable nature of the actual soil being modeled.

A pile behavior computer program was used to predict test pile behavior using load transfer data generated from the instrumented test piles. Only fair prediction of the full measured load-displacement curve was achieved even after several computer runs in which input variables were varied to improve the prediction. It is the opinion of the investigators that given soil property data and soil profile but without field test results, predictions from centrifuge model tests would give results as accurate as, and very likely much more accurate than, those generated from computer predictions.

The experimental procedures developed and the verification established in this program open the door to investigating in controlled laboratory setting the many factors that influence pile performance. The centrifuge technique should also be useful and very cost-effective in establishing the predicted behavior and the sensitivity to design changes of pile foundation for large projects.

4.2 Recommendations for Future Research

Among the many possibilities for future research on piles using the geotechnical centrifuge are:

The effect on pile behavior from different methods of installation should be investigated in detail. This will involve developing a pile driver for use in the centrifuge and subsequent load testing.

The high cost effectiveness of model testing in the centrifuge indicates that parametric studies should be conducted to examine the various parameters that influence pile behavior. Among the more important variables are the soil type, pile geometry, placement method, interface conditions, and time effects. When these results become available, the validity of existing pile capacity formulas and methods of predicting pile settlement can be critically examined. Based on these results, improvement on the predictive methods can be made.

A comprehensive program of testing model single piles and pile groups in lateral loading should be undertaken. The need to install a large number of strain gauges in the instrumented piles would be best served by using an approximately 1/25 scale pile. This in turn requires a soil container of a sufficient size such that the boundary effects are minimized. The University of Colorado has started the planning and design of a 400 g-ton centrifuge which has the capacity to carry a 1.5 m x 1.5 m x 1 m payload. When the construction is completed toward the end of 1984, this centrifuge would provide an excellent facility for the pursuit of additional modeling research on pile foundations. The smaller, 10 g-ton centrifuge which has been used in the program described in this report could be used in the meantime to develop the necessary techniques and procedures to be transferred to the larger facility.

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