IN-SITU MEASUREMENTS OF FRICTION AND BEARING CORRELATED WITH INSTRUMENTED PILE TESTS

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ABSTRACT

This study involved a series of field tests conducted with a recently developed in-situ testing device. The in-situ testing device was used to measure values of skin friction and point bearing taken during soil sampling operations. The test procedure was adapted for use with a standard core drilling rig. All load measurements were made at the ground surface.

The in-situ measured values of skin friction and point bearing taken during sampling operations were compared with measured values of skin friction and point bearing taken from two full-scale instrumented test piles. All tests were conducted at one test site in a clay soil. It was necessary to develop adjustment factors to correct for the size and shape effects. The adjustment factors were correlated with the plasticity index of the soil.

SUMMARY

This test program was conducted during the third year of a five-year study on "Bearing Capacity for Axially Loaded Piles." A test procedure was developed so that in-situ measurements of friction and bearing could be made during sampling operations in medium to firm clays. The procedure involved the use of standard drilling rig equipment with a load-measuring device at the ground surface.

The in-situ measurements of friction and bearing were compared with corresponding values from several instrumented pile tests. Adjustment factors were developed to correct for size and shape effects between the load-measuring device and the full-scale piles. It was possible to correlate the adjustment factors with the plasticity index of the soil.

IMPLEMENTATION STATEMENT

This is a technical progress report which presents the results of a test program conducted using a newly developed device for making in-situ measurements. These measurements of friction and bearing were made in medium to firm clays during sampling operations. The test procedure is acceptable for use with standard drilling rig equipment.

This study is considered preliminary in nature because measurements were made at one test site in one soil. Additional field tests in conjunction with fully instrumented test piles would be required for further verification of the results obtained in this study. Additional tests, particularly in very soft and very stiff soils, would be necessary in order to make the procedure applicable in all soils. A separate research study involving in-situ determination of soil strength in all soils is recommended.

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INTRODUCTION

Nature of the Problem

Ever since piles were first used to transfer structural loads to deeper soil strata, the problem of determining the bearing capacity of these piles has existed. A considerable amount of research has been done over the years in order to find an accurate and economical answer to this problem.

There are at present two static methods of predicting the bearing capacity of a pile (4)*

(1) a full-scale static load test performed at the actual site,

(2) a static formula in which the load carried by a pile in skin friction and point bearing are calculated and combined to give a static bearing capacity.

The first method is often very expensive and time consuming. The second method of predicting the ultimate capacity, Qu, of a pile relies on empirical data derived from model studies and full-scale load tests. The static formula is given as follows:

$$Qu = fAs + qAp \tag{1}$$

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* Numbers in parentheses refer to references at the end of this report. (The citations on the following pages follow the style of the Journal of Soil Mechanics and Foundation Division, ASCE.)

where As and Ap represent, respectively, the embedded surface area and the pile end area, and f and q represent, respectively, the unit skin friction and unit point bearing. Successful application of the static formula method depends upon selection of appropriate values of f and q that take into account the combined effects of soil conditions, pile type and dimensions, pile installation procedures, and the manner of loading (8).

The unit skin friction, f, at any point along a pile in clay, can be related to the shear strength of the clay. In order to obtain the shear strength of any soil, laboratory tests must be run on soil samples taken from the field. Experience has shown that in many cases it is impractical to impossible to obtain adequate undisturbed soil samples. Therefore, an in-situ measuring device which could measure unit skin friction, f, and unit point bearing, q, would be very desirable. The in-situ device should have the following characteristics:

(1) measurements can be made during sampling operations,

(2) measurements can be made in-situ,

(3) test procedure should be simple, reliable, inexpensive to perform, fast and trouble-free, and

(4) measurements can be made in soft material to hard rocks.

Present Status of the Question

Schmertmann (10) has recently done research at the University of Florida with an in-situ device known as the Dutch friction cone.

It is a device which makes separate measurements of skin friction and point bearing by static loading procedures. Schmertmann's procedure gives direct measurements of friction and bearing for use with the static formula method of determining bearing capacity of a pile.

At present, the Texas Highway Department (5) uses either laboratory shearing strength or the cone penetrometer test correlation to determine the soil properties needed for use with the static formula. The bearing capacity obtained in this manner is generally a conservative approximation (5). In many cases the Texas Highway Department performs full-scale static load tests on piles at an actual construction site.

A study involving the use of a small instrumented pile to measure dynamic and static skin friction and point loads in a variety of field soils has been presented by Korb (6). Korb's pile was 2.5 ins. in diameter and 3 ft. in length and was instrumented in such a way that separate measurements of skin friction and point bearing were made simultaneously. Korb recommended a test program with a longer pile which would make it possible to test at greater depths, and to examine the effects of over burden pressure.

Another study by Rehmet (9) was made using several in-situ measuring devices. Field tests in clay were made by Rehmet using these in-situ testing devices and limiting values of skin friction and point bearing were obtained. As a result of this study, Rehmet made recommendations concerning a practical test procedure which could be used to measure skin friction and point bearing during sampling operations. These recommendations involve a static test in which a device is pushed slowly into the soil at specified depth in a boring hole and load readings are taken at the ground surface to determine the skin friction and point bearing values.

Objectives

The objectives of this investigaiton are:

(1) To conduct a field test program at a test site which involves measurement or in-situ skin friction and point bearing during sampling operations following the recommended test procedures made by Rehmet (9).

(2) To correlate measured values of skin friction and point bearing taken during sampling operations with data obtained from the full-scale load tests on the instrumented pipe piles.

IN-SITU TESTING EQUIPMENT

General

In order to conduct the in-situ tests outlined in the first objective, the following equipment was needed:

1. A load measuring device,
2. Loading equipment,
3. And load recording equipment.

Since in-situ measurements were to be made during sampling operations, a standard drilling rig was used as the means of applying the load to the load measuring device. The load measuring device was connected to the top of the drill pipe in order to simplify measurement procedures. It was necessary that the load measuring device be simple, practical, and adaptable to the standard drilling rig. Also, it was necessary that the recording equipment be easy to operate and adaptable to field use.

Load Measuring Device

The in-situ load measuring device was made so that it was small, lightweight and attachable to standard drilling pipe as shown in Fig. 1. The load measuring device was fabricated from 2.375-in. O. D. steel tubing with two standard drill rod couplings welded to the tube with the male connection on one end and the female connection on the other. The load measuring device was 11.75 ins. in length with a wall thickness of 0.375 in. Four SR-4 Budd gage Rosettes were placed 5.875 in. from the bottom of the load measuring device. In order to increase the sensitivity of the gages, the walls of the drill pipe were machined to a thickness of 0.10 in. at the bridge location.

Loading Equipment

A standard Texas Highway Department drilling rig was used to apply loads to the drill pipe when running in-situ tests and when recovering soil samples after each in-situ test. The hydraulic system on the drilling rig was used to push the drill pipe into the

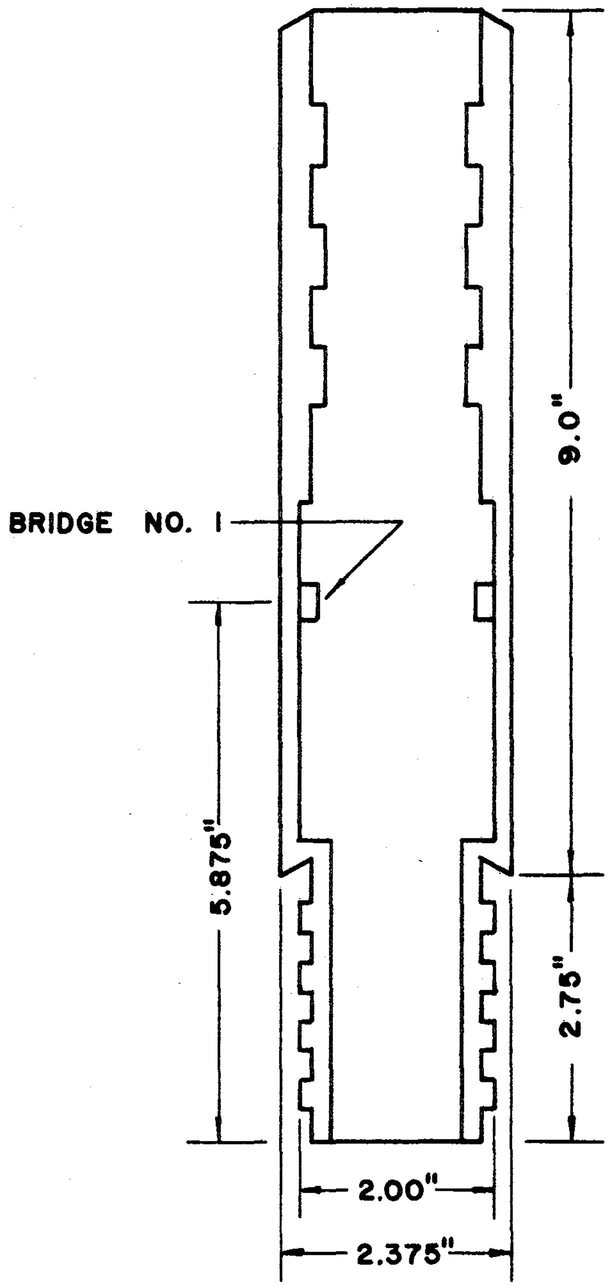

FIGURE 1 LOAD MEASURING DEVICE

boring hole the required penetration for each in-situ test at the required depth. Fig. 2 shows the hydraulic system being used to push the load measuring device and the drill pipe into the boring hole. Since a standard drilling rig is used primarily for soil exploration, it provided the means of obtaining soil samples at any depth desired. The operation of obtaining soil samples with Shelby tubes is shown in Fig. 3. The wench on the drilling rig, as shown in Fig. 4, provided a means of raising and lowering the drill pipe and load measuring device in and out of the boring hole.

Load Recording Equipment

The battery-operated Budd P-350 strain indicator which was used to record load data is shown in Fig. 5. Before each in-situ test, the Budd-P-350 strain indicator was balanced at full sensitivity with the in-situ load measuring device. During testing, the difference between the balanced load reading and the reading under a given load constituted the strain in the load measuring device for that load. Thestrains found for each load could then be converted to pounds by using predetermined load-strain (calibration) curves for the load measuring device. After each in-situ test the load measuring device was rebalanced in order to make sure that the balanced load reading did not change.

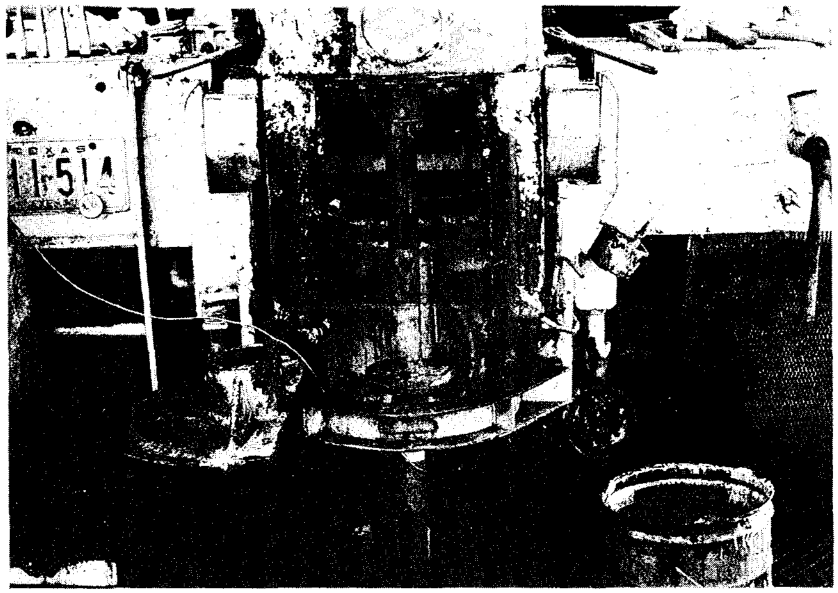

FIGURE 2 HYDRAULIC EQUIPMENT USED TO PUSH DRILL PIPE INTO THE BORE HOLE

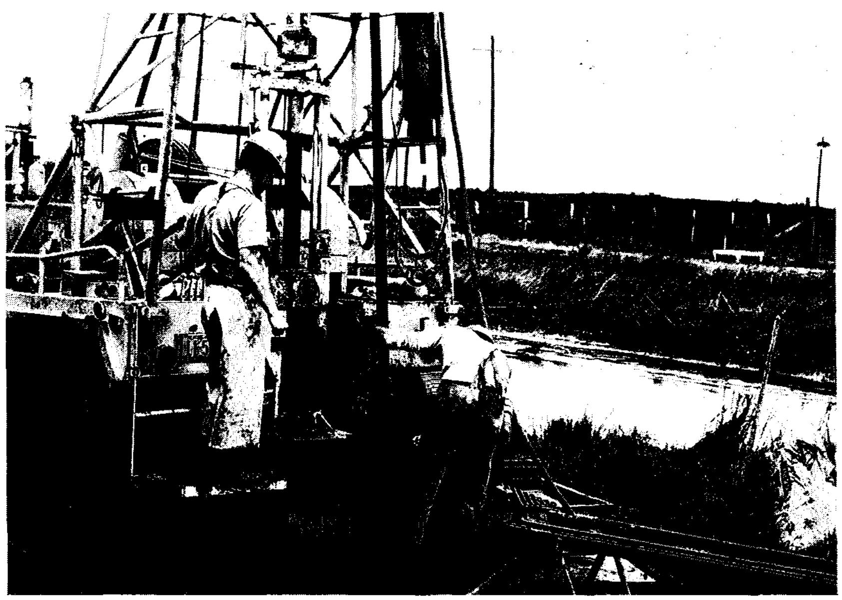

FIGURE 3 SHELBY TUBE SAMPLING

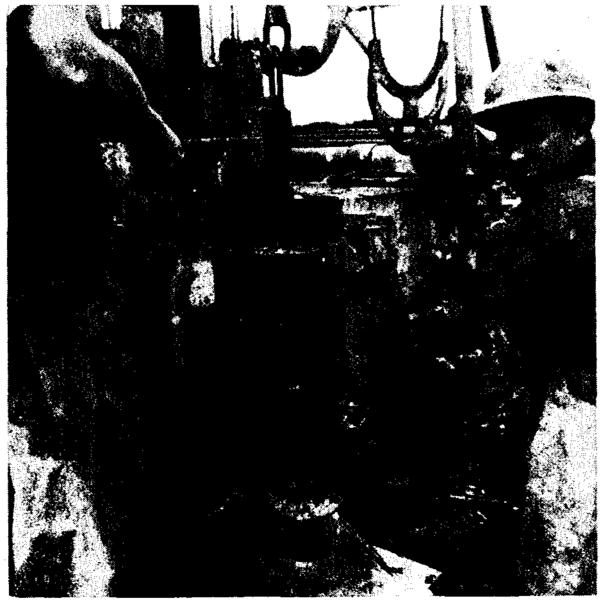

FIGURE 4 REMOVING IN-SITU LOAD MEASURING DEVICE AND DRILLING PIPE FROM BORE HOLE

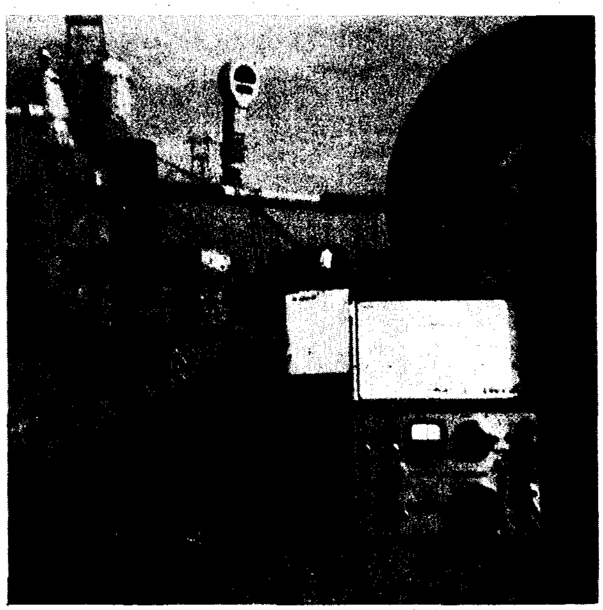

FIGURE 5 BATTERY OPERATED BUDD-350 STRAIN INDICATOR USED TO BALANCE AND RECORD STRAINS IN LOAD MEASURING DEVICE

Calibration

In order to calibrate the in-situ load measuring device, an accurate loading machine was needed. An Instron Loading Machine was available for this purpose. The load measuring device was loaded and unloaded several times in the Instron Loading Machine in order to eliminate permanent set that might occur. Once it was established that no more permanent set would occur under a maximum assumed loading capacity of 10,000 pounds, increments of 1,000 pounds were applied to the load measuring device and the corresponding strains for each increment were recorded on a Budd P-350 strain indicator. Loading and unloading from 0 to 10,000 pounds in increments of 1,000 pounds were accomplished several times in order to assure that corresponding strains for each load increment were in agreement. The resulting load-strain data was plotted and was found to be linear. This process of calibration as shown in Fig. 6 for the in-situ load measuring device was repeated after each series of in-situ tests.

FIELD TEST PILE INSTRUMENTATION

General

In order to satisfy the second objective of correlating measured values of skin friction and point bearing taken during sampling operations with data obtained from full-scale load tests on instrumented pipe piles, it was necessary to test instrumented piles. Two instrumented piles were driven and tested at a site in Port Arthur, Texas,

FIGURE 6 CALIBRATING IN-SITU LOAD MEASURING DEVICE ON INSTRON TESTING MACHINE

on State Highway 87 near the Intracoastal Canal Bridge. Pile No. 1 was a steel pipe 67 ft. long, 16 ins. in diameter, and had a wall thickness of 3/8 in. Pile No. 2 was a steel pipe 78 ft. long, 16 ins. in diameter, and had a wall thickness of 3/8 in. Pile No. 1 was driven and tested on the north side of the Intracoastal Canal Bridge while pile No. 2 was driven and tested on the south side of the bridge.

Instrumentation

Preliminary soil borings obtained from the Texas Highway Department, District 20 provided a means of determining the number of different soil stratum through which the instrumented piles were to be driven. These boring data are shown in Fig. 7 for site 1 and Fig. 8 for site 2. Strain gage bridges were attached to the piles at each elevation where a different soil layer came into contact with the pile when embedded. The location of these strain gage bridges is also shown in Figs. 7 and 8. At each of these locations four conventional 90^{o} strain gage rosettes were attached 90^{o} apart on the internal wall of the pile. These gages were wired into two separate wheatstone bridges with 4 active gages in each bridge in order to cancel bending in the pile and achieve temperature compensation and maximum sensitivity. The installation of these gages was made in accordance with the procedures used by Airhart (1).

Since non-uniform load distributions about the circumference of the pile were expected at the top and bottom of the pile, it was

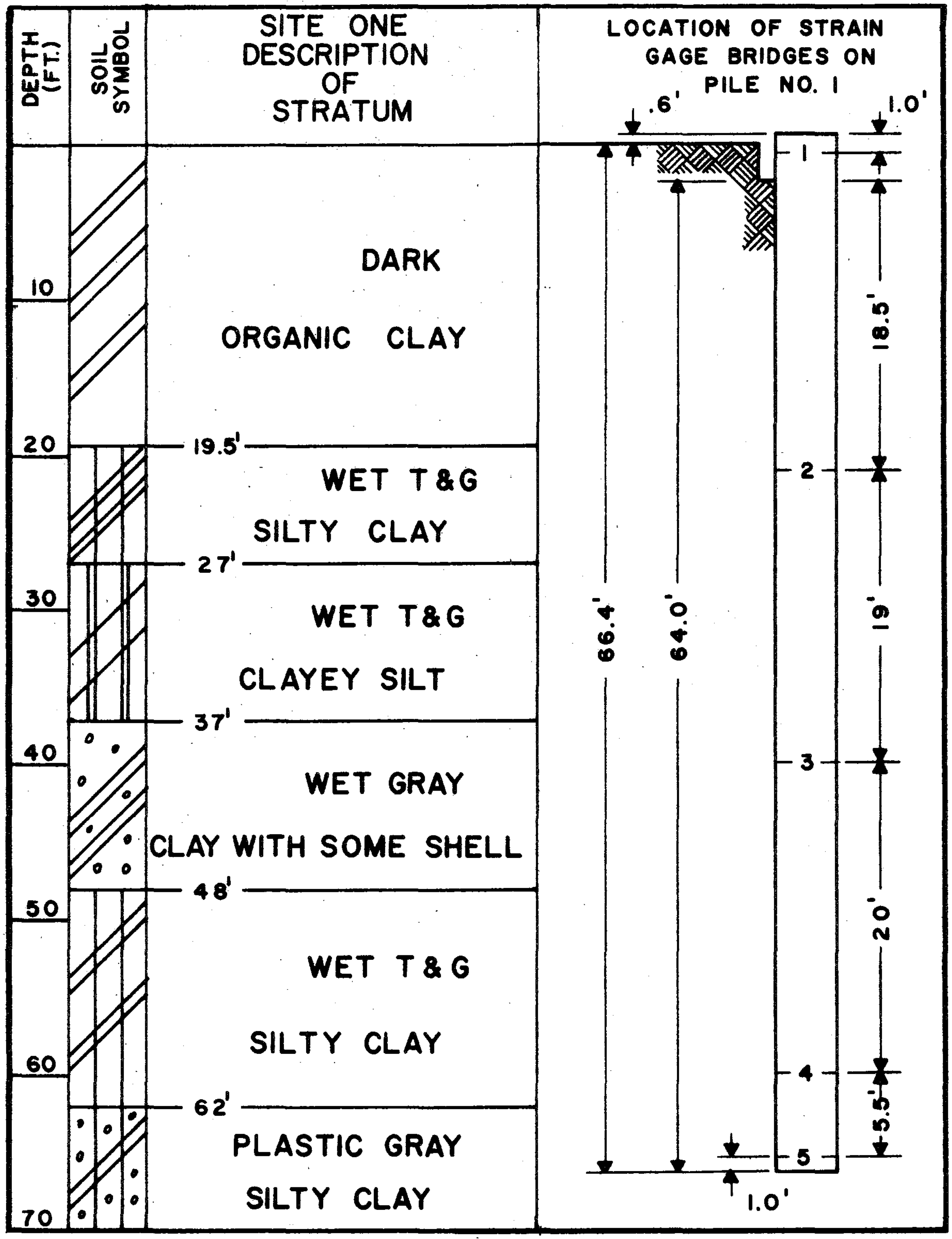

FIGURE 7 LOCATION OF THE 5 STRAIN GAGE BRIDGES (PILE NO. 1)

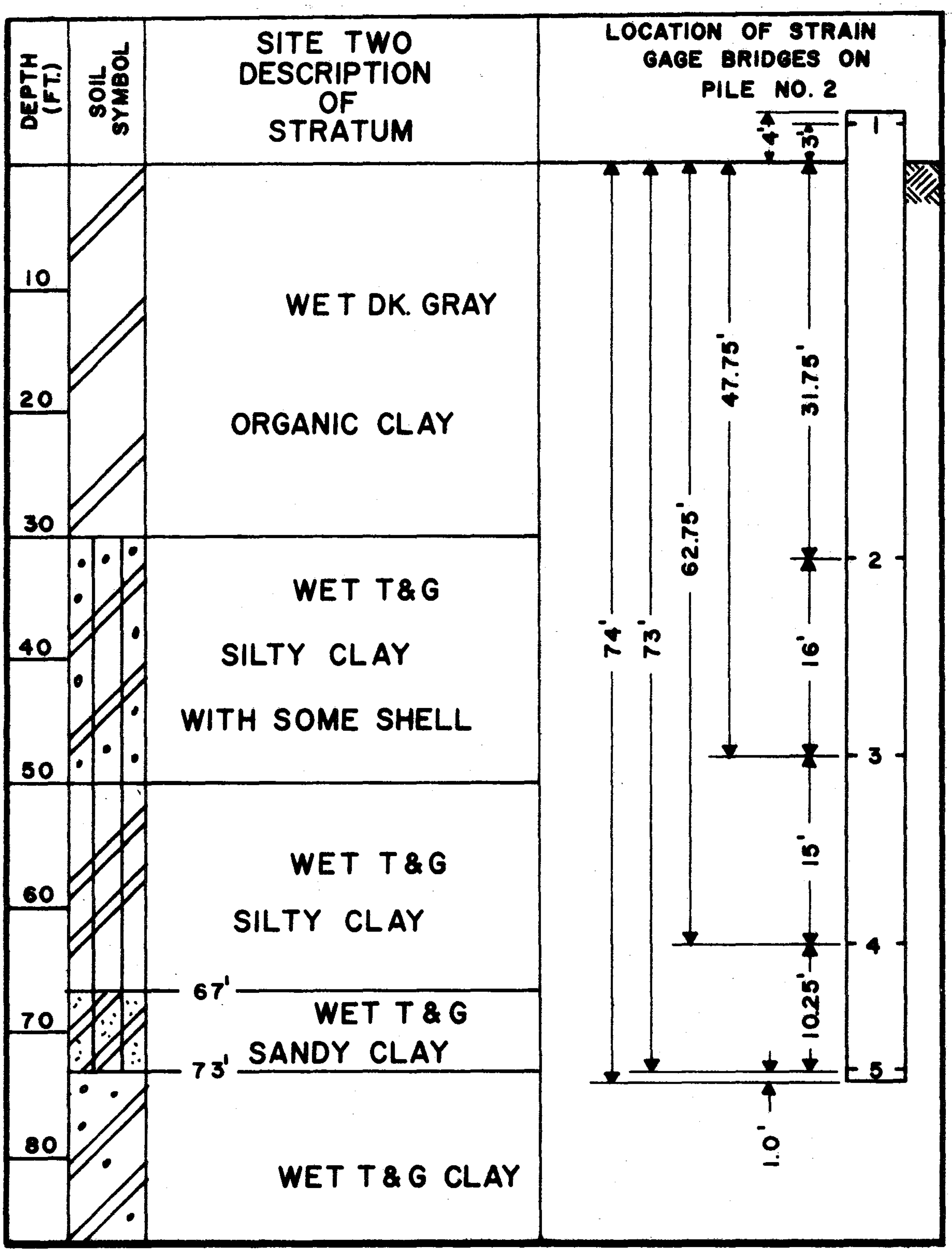

FIGURE 8 LOCATION OF THE 5 STRAIN GAGE BRIDGES (PILE NO. 2)

felt that averaging of these two bridge outputs at a location would be an accurate indication of the average load. Additionally, since there was a possibility of losing one of the bridges at each location, it was considered necessary to have two independent bridges at each location. This approach would provide at least some indication of the load, should one of the bridges become inoperative. The two independent bridges also provided a means of checking the load at each bridge location.

Strain gage cables inside the pile were adhered to the pile wall over substantially the entire length and anchored near each bridge location in order to eliminate cable stretching during driving and data collection. All cables were terminated near the top of the pile into a single multi-pin connector. This cable termination technique was used to eliminate external cables being connected to the pile during hauling and handling and provided a means of easy external connection to the recording equipment.

In addition to the gages in the pile, a specially designed load cell was attached in the field above the head of the pile to verify the recorded loads at the top of the pile. The load cell was constructed from the same type of material as the piles and was instrumented in basically the same manner as the piles. The load cell's theoretical bridge output agreed very closely with the actual output during several calibrations both before and after driving the piles. It was also found that close agreement was obtained between the load cell and the top bridges in the piles throughout the tests.

Therefore, the bridge outputs in the piles were used directly to compute loads at the specified points along the pile.

Strains in each of the bridges were taken using a standard switch and balance unit and a battery-operated strain indicator. The switch and balance unit was used in order to take several readings in a short period of time. The strains were converted to load in the same manner as discussed previously in this study on in-situ load recording equipment.

SOIL DATA AT TEST SITES

Soil Profile

Initial borings made by the Texas Highway Department were used to determine the location of the strain gage bridges on the two instrumented piles as indicated previously. To supplement the initial investigation, soil samples were taken after each in-situ test. The soil properties and the soil profile shown in Fig. 9 were determined from data obtained from soil samples taken during in-situ testing near test pile 1. Fig. 10 shows the soil properties and a soil profile determined from data obtained from soil samples taken during in-situ testing near test pile 2.

The soil of primary interest for test site 1 near test pile 1 was the highly plastic clay 20 ft. below the ground surface. The soil of primary interest for test site 2 near test pile 2 was the highly plastic clay 30 ft. below the ground surface. The top 20 ft.

FIGURE 9 BORING LOG - SITE ONE

DEPTH (FT.)	SOIL SYMBOL	SITE ONE DESCRIPTION OF STRATUM	DRY DENSITY LB/CU FT.	MOISTURE CONTENT %	LIQUID LIMIT	PLASTIC LIMIT	UNCONFINED SHEAR STR. (PSF)
10 19.5		DARK ORGANIC CLAY		* FIELD VANE SHEAR VALUES			* 300 * 240 * 360 * 1000
20 27		WET T&G SILTY CLAY	92.6	22.6	78.9	18.2	2820
30 37		WET T&G CLAYEY SILT	90.5	30.5	78.4	31.3	1480
40 48		WET GRAY CLAY WITH SOME SHELL	84.5	42.6	85.5	30.7	2510
50 60 62		WET T&G SILTY CLAY	105.4	18.4	67.8	18.8	1675
		PLASTIC GRAY CLAY	112.1	16.2	50	17.9	1870

FIGURE 10 BORING LOG - SITE TWO

DEPTH (FT.)	SOIL SYMBOL	SITE TWO DESCRIPTION OF STRATUM	DRY DENSITY LB/CU FT.	MOISTURE CONTENT %	LIQUID LIMIT	PLASTIC LIMIT	UNCONFINED SHEAR STR. (PSF)
10		WET DK. GRAY		* FIELD VANE SHEAR VALUES			* 240 * 360
20		ORGANIC CLAY					* 200 300
30			88	32.9	63	21.2	748
40		WET T&G SILTY CLAY WITH SOME SHELL					
50			103.6	22.5	67.0	21.5	1582
60		WET T&G SILTY CLAY					
67			98	23.9	69.5	24.7	1860
70		WET T&G SANDY CLAY					
73			105.3	20.6	37.5	17.1	2020
		WET T&G CLAY	100	23.8	65.3	21.9	1720

near test pile 1 and the top 30 ft. near test pile 2 were too soft to enable in-situ test of the type conducted in this study to be run.

Soil Properties

The following physical properties were determined from data obtained from soil samples taken during in-situ testing:

1. Dry Density
2. Moisture Content
3. Liquid Limit
4. Plastic Limit.

It was necessary to obtain these physical properties so that a correlation might be attempted between the soil properties and the in-situ test data. The shear strength, an engineering property of the soil, was also determined for each soil sample taken. The shear strength could be used when using Eq. 1 to determine the ultimate bearing capacity, Qu, of a pile. The physical and engineering properties for test site 1 are shown in Fig. 9 and for test site 2 in Fig. 10.

The top 20 ft. of soil near test pile 1 was too soft for unconfined compression tests to be run; therefore, field vane shear tests were run to determine the vane shear strength of the soil. For test site 2 where the top 30 ft. was too soft to enable unconfined compression tests to be run, the field vane shear test was used to determine the vane shear strength of the soil. The physical soil properties and the engineering properties were determined using accepted standard procedures as presented in the laboratory manual by Lambe (7).

TEST PROCEDURE

General

Rehmet (9) and Korb (6) used small instrumented piles to simultaneously measure unit skin friction, f, and unit point bearing, q. Their studies were restricted to testing in shallow soil depths. This restriction was due mainly to the limitations imposed by wires extending from the recording equipment at the ground surface to the strain gages located inside their small instrumented piles. This shallow depth of penetration into the ground is impractical because in-situ measurements of skin friction and point bearing are needed at much greater depths in order to estimate the bearing capacity of a pile. Rehmet (9) tested to depths of 10 ft. and made recommendations for conducting deeper tests. These recommendations were followed in this study.

In-Situ Test Procedure

The test procedure used in this study was as follows:

(1) Using standard drilling rig equipment, advance the test hole to the desired depth.

(2) Retract and remove drilling equipment and insert drill pipe to the bottom of the test hole.

(3) Zero balance the in-situ measuring load cell with the Budd P-350 Strain Indicator, as shown in Fig. 5.

(4) Take an initial reading on the Budd P-350 Strain Indicator of the in-situ measuring load cell to be used during the test.

(5) Connect the load cell to the drill pipe and to the Kelley of the drill rig.

(6) Conduct a tip-only test by measuring the total surface load required to move the drill pipe downward one inch.

(7) Push the drill pipe assembly downward so that the pipe will be embedded eleven inches in the bottom of the test hole.

(8) Conduct an embedded test by measuring the total surface load required to move the drill pipe downward one inch.

(9) Remove the load cell at the surface and retract the drill pipe. Attach drilling equipment and advance the test hole to the next desired depth.

(10) Repeat this procedure until measurements are made at any number of desired depths.

<u>In-Situ Test Series</u>

For identification purposes, series 1 was conducted near test pile 1 and series 2 and 3 were conducted near test pile 2. Each test series involved a number of in-situ tip-only and embedded tests. Test designation and descriptions are found in Table 1 for test series 1, Table 2 for test series 2 and Table 3 for test series 3.

After a test site was selected near test pile 1 for test series 1 and near test pile 2 for test series 2 and 3, a bore hole was drilled down to a soil strata that was stiff enough to enable tip-only and embedded tests to be run. For test series 1, the bore hole was drilled to a depth of 20 ft. and for test series 2 and 3 the bore holes were drilled to a depth of 30 ft. The bore holes were then cleaned out with drilling mud and a 2 ft. Shelby tube sample was taken in order that laboratory determinations of the soil parameters could be made. The bore hole was again washed out and a tip-only and embedded test conducted in accordance with the procedure discussed previously. For test series 1 and test series 2, the blunt end tip shown in Fig. 11 was used for the tip-only and embedded tests. For test series 3 the cone tip shown in Fig. 11 was used for the tip-only and embedded tests. Sampling, washing and in-situ tests conducted at each test depth are shown in Tables 1, 2, and 3.

Instrumented Pile Test Procedure

Test pile 1 was driven 61 ft. into the ground on November 3, 1969 and test pile 2 was driven 72 ft. into the ground on November 20, 1969. After the test piles were embedded to these respective depths, load recording equipment was installed on the test piles.

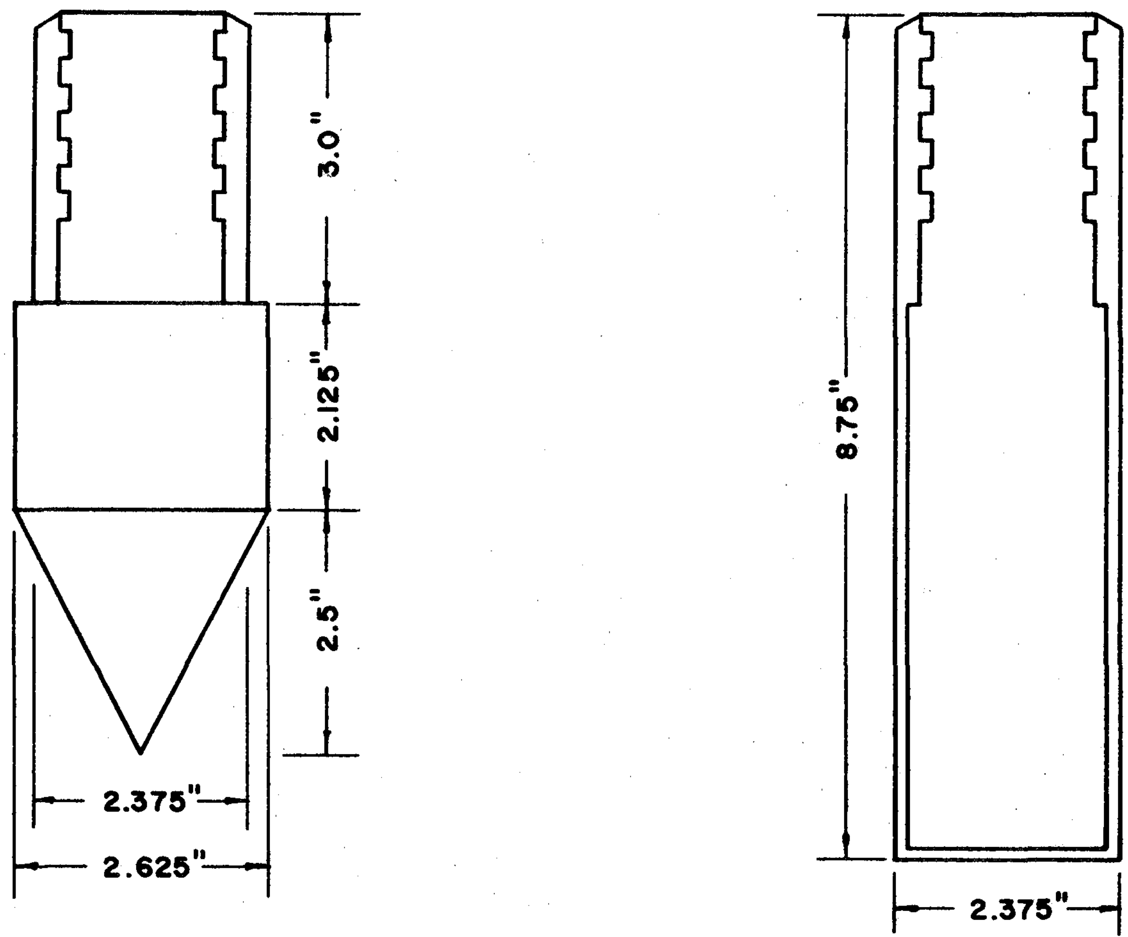

FIGURE 11 CONE AND BLUNT END TIP

TABLE 1.--TEST DESIGNATION AND DESCRIPTION--TEST SERIES I

Series	Hole	Test	Depth, in feet	Description of Test*
I	I	1	21.5	Tip only
I	I	2	22.5	Total embedded test
I	I	3	30.0	Tip only
I	I	4	32.0	Total embedded test
I	I	5	40.5	Tip only
I	I	6	41.5	Total embedded test
I	I	7	60.5	Tip only
I	I	8	61.5	Total embedded test
I	I	9	65.5	Tip only
I	I	10	66.5	Total embedded test

*This test series consisted of all tip only and total surface load tests near instrumented pipe pile No. 1. Instrumented pipe pile No. 1 tests consisted of three full-scale static load tests: immediate, 4-day, and 11-day.

TABLE 2.--TEST DESIGNATION AND DESCRIPTION--TEST SERIES II

Series	Hole	Test	Depth, in feet	Description of Test*
II	II	1	30	Tip only
II	II	2	31	Total embedded test
II	II	3	40	Tip only
II	II	4	41	Total embedded test
II	II	5	50	Tip only
II	II	6	51	Total embedded test
II	II	7	60	Tip only
II	II	8	61	Total embedded test
II	II	9	69	Tip only
II	II	10	69**	Total embedded test
II	II	11	80	Tip only
II	II	12	81	Total embedded test

*This test series consisted of all tip only and total surface load tests near instrumented pipe pile No. 2. Instrumented pipe pile No. 2 consisted of three full-scale static load tests: immediate, 4-day, and 11-day.

**Soil was too stiff to enable embedded test to be run.

TABLE 3.--TEST DESIGNATION AND DESCRIPTION--TEST SERIES III

Series	Hole	Test	Depth, in feet	Description of Test*
III	III	1	31	Tip only
III	III	2	32	Total embedded test
III	III	3	40	Tip only
III	III	4	41	Total embedded test
III	III	5	50	Tip only
III	III	6	51	Total embedded test
III	III	7	65	Tip only
III	III	8	66	Total embedded test
III	III	9	76	Tip only
III	III	10	77	Total embedded test

*This test series consisted of all tip only and total surface load tests near instrumented pipe pile No. 2. Instrumented pipe pile No. 2 consisted of three full-scale static load tests: immediate, 4-day, and 11-day.

Each test pile was then driven three additional feet into the ground. This enabled dynamic loading effects to be recorded during the final 3 feet of driving.

After the test piles were driven, static load tests were conducted to determine the ultimate bearing capacity, Qu, of the test pile. Procedures and load test equipment were in keeping with standard Texas Highway Department practice. The static load tests were conducted immediately, 4 days and 11 days after driving the test piles in order to determine the soil set-up factor around each test pile.

Upon completion of the 11 day static load test, the test piles were redriven one or two feet to record dynamic loading effects after the 11 day set-up of the soil around each of the test piles. It should be noted that dynamic data recorded during driving of the test piles was not reported in this study, but it is reported in a research report by Bartoskewitz (2).

ANALYSIS OF TEST RESULTS

General

The test results for this study were analyzed by comparing measured values of unit skin friction and point bearing from the instrumented test piles with the corresponding values measured from the in-situ measuring device. Since simultaneous measurements of friction and bearing were not made with the in-situ measuring device, it was necessary to develop adjustment factors for the in-situ measurements. After developing the adjustment factors, it was possible to correlate these factors with the soil parameter, plasticity index.

Instrumented Pile Test Data

One of the measured values obtained from the test pile data was the average unit skin friction. It was measured between the strain gage bridges on the instrumented test piles. The actual location of these strain gage bridges is shown in Figs. 7 and 8. The value for the average unit skin friction, f, was obtained by determining how much of the load was transferred to the soil between the strain gage bridges and dividing this load by the circumferential area between the bridges. An example of how the unit skin friction was calculated between two strain gage bridges is shown in Fig. 12.

The unit point bearing, q, was obtained by dividing the tip load by the tip area. It should be noted that the closest strain gage bridge to the tip of the pile was actually located 1 ft. up from the tip as shown in Figs. 7 and 8. To find the correct tip load, the average unit skin friction computed in the segment of the test pile between bridges 4 and 5 was assumed to be the same in the last linear foot of the pile. The circumferential area of one linear foot of the test pile was 4.19 sq. ft. Therefore, the average unit skin friction between bridges 4 and 5 was multiplied by 4.19

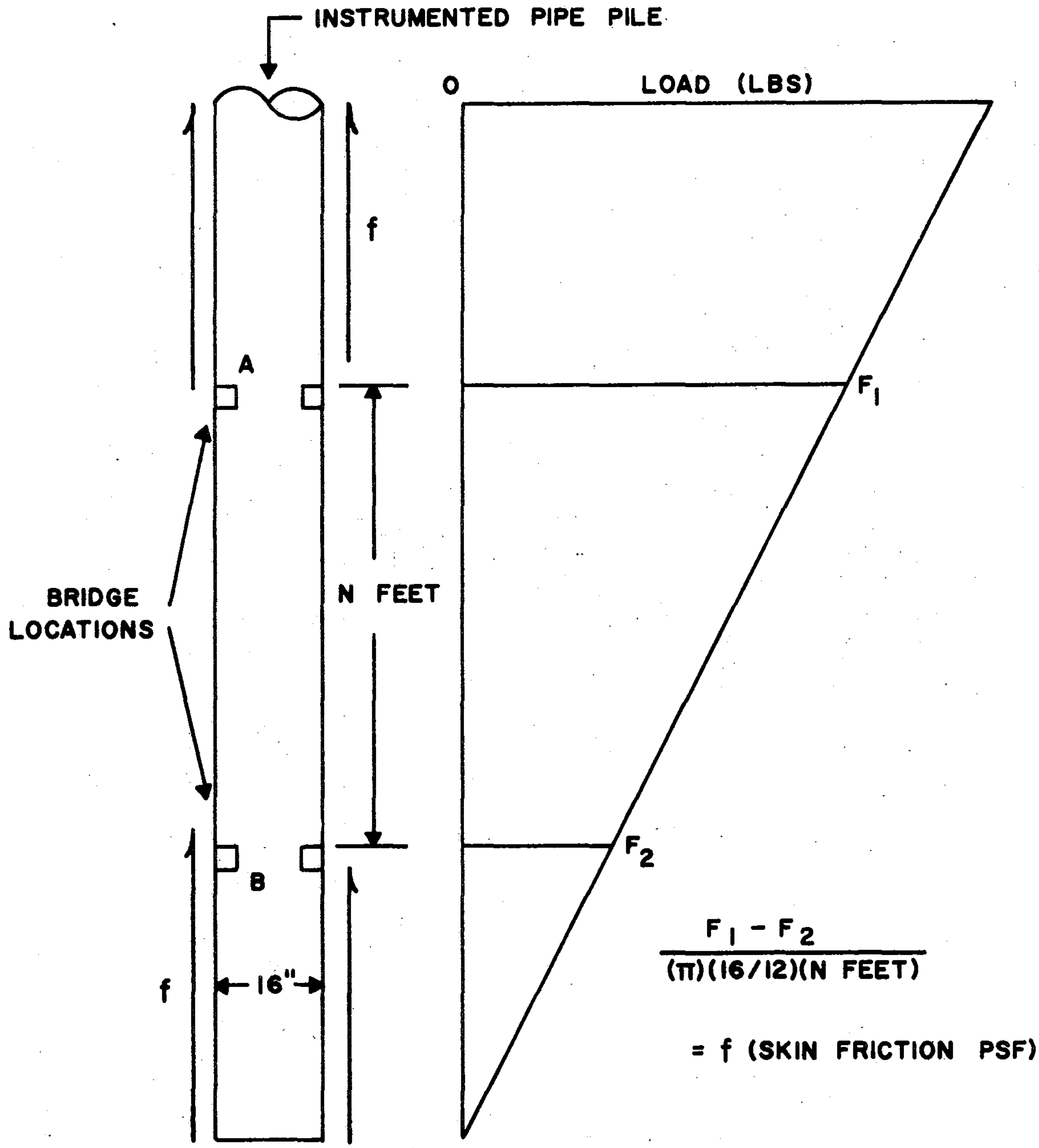

FIGURE 12 CALCULATION OF SKIN FRICTION ON INSTRUMENTED PIPE PILE

sq. ft. and this value was subtracted from the load recorded in bridge 5 to give the corrected load for the tip of the test pile. This corrected load was then divided by the area of the tip of the test pile to find the unit point bearing value.

All of the measured static data from the field load tests on the the two instrumented test piles are given in Appendix III. The Load Distribution versus Depth curves for the test piles are given in Appendix IV. The Load-Settlement curves for the two test piles are found in Appendix V. All loading values given in this study were developed from load data taken from the 5 strain gage bridges in each of the test piles. The failure loads are considered to be those loads corresponding to plunging failure of the test piles.

In-Situ Friction Test Data

From the studies conducted by Korb (6) and Rehmet (9), it was found that the point bearing values determined from tip-only tests changed during the embedded test. Therefore, an adjustment factor was used in this study to correct the point bearing value for the embedded test so that correct friction values would be obtained. The adjustment factor also includes a correction for shape and size effects because in-situ measurments were compared with measured friction from the full-scale piles.

Since the average unit skin friction as determined from the immediate, 4-day and 11-day static load tests on the instrumented test piles and the in-situ measurements were known, the measured friction values from the test piles were equated to the in-situ measurements. The following equation was used to equate average unit skin friction, F_M, to the in-situ measurements:

$$F_M = \frac{T_E - [T_0(C_T)]}{AREA} \tag{2}$$

where: F_M = average measured unit skin friction obtained from the instrumented test piles.

T_E = load in lbs. measured from the in-situ embedded tests.

T_0 = load in lbs. measured from the in-situ tip-only tests.

C_T = adjustment factor for the in-situ tip-only tests.

Area = circumferential area for 1 ft. of drill pipe.

Since the F_M value used in Eq. 2 is an average value obtained from the test piles, it was necessary to make several in-situ measurements in each different soil strata so that an average value for C_T could be found. Everything in Eq. 2 was measured and known except the adjustment factor, C_T. Therefore, it was possible to compare a C_T value for each soil strata and for all tests conducted in this study. Tables 4, 5 and 6 contain average measured unit skin friction values, in-situ measurements of tip-only tests, embedded tests, and the corresponding C_T values.

TABLE 4
SERIES I
FRICTION DATA - PILE I - IMMEDIATE STATIC LOAD TEST

DEPTH FEET	SERIES I TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
12.0				41.32	
21.5	1	920			
22.5	2		2239		2.32
30.0				173.5	
31.0	3	790			
32.0	4		1625		1.92
40.5	5	748			
41.5	6		1711		2.14
50.0				352.3	
60.5	7	1135			
61.5	8		2218		1.76
60.5	7	1135			
61.5	8		2218		1.30
62.5				1202.78	
65.5	9	1189			
66.5	10		1868		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ. FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 4A
SERIES I
FRICTION DATA - PILE I - 4 DAY STATIC LOAD TEST

DEPTH FEET	SERIES I TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
12.0				16.79	
21.5	1	920			
22.5	2		2239		1.75
30.0				722.82	
31.0	3	790			
32.0	4		1625		1.49
40.5	5	748			
41.5	6		1711		1.71
50.0				702.26	
60.5	7	1135			
61.5	8		2218		1.57
60.5	7	1135			
61.5	8		2218		1.22
62.5				1337.39	
65.5	9	1189			
66.5	10		1868		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O(C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ.FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 4B SERIES I FRICTION DATA – PILE I – 11 DAY STATIC LOAD TEST					
DEPTH FEET	SERIES I TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
12.0				40.02	
21.5	1	920			
22.5	2		2239		1.69
30.0				1101.19	
31.0	3	790			
32.0	4		1625		1.19
40.5	5	748			
41.5	6		1711		1.55
50.0				894.52	
60.5	7	1135			
61.5	8		2218		1.47
60.5	7	1135			
61.5	8		2218		1.35
62.5				1111.59	
65.5	9	1189			
66.5	10		1868		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ. FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 5
SERIES 2
FRICTION DATA - PILE 2 - IMMEDIATE STATIC LOAD TEST

DEPTH FEET	SERIES 2 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				91.78	
30.0	1	580			
31.0	2		1200		1.95
40.0				111.96	
40.0	3	695			
41.0	4		1215		1.65
50.0	5	890			
51.0	6		1440		1.26
56.2				515.51	
60.0	7	753			
61.0	8		1635		1.56
66.0				743.42	
79.0	9	680			
80.0	10		2150		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ. FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 5A
SERIES 2
FRICTION DATA - PILE 2 - 4 DAY STATIC LOAD TEST

DEPTH FEET	SERIES 2 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				188.07	
30.0	1	580			
31.0	2		1200		1.46
40.0				571.73	
40.0	3	695			
41.0	4		1215		1.24
50.0	5	890			
51.0	6		1440		.97
56.2				931.53	
60.0	7	753			
61.0	8		1635		1.20
66.0				1174.55	
79.0	9	680			
80.0	10		2150		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O\,(C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ.FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 5B
SERIES 2
FRICTION DATA - PILE 2 - 11 DAY STATIC LOAD TEST

DEPTH FEET	SERIES 2 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				220.64	
30.0	1	580			
31.0	2		1200		1.13
40.0				882.07	
40.0	3	695			
41.0	4		1215		.96
50.0	5	890			
51.0	6		1440		.83
56.2				1125.0	
60.0	7	753			
61.0	8		1635		.99
66.0				1438.83	
79.0	9	680			
80.0	10		2150		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \text{ AND } A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ.FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 6

SERIES 3

FRICTION DATA - PILE 2 - IMMEDIATE STATIC LOAD TEST

DEPTH FEET	SERIES 3 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				91.78	
31.0	1	758			
32.0	2		1596		2.01
40.0				111.96	
40.0	3	1033			
41.0	4		1990		1.86
50.0	5	807			
51.0	6		1778		1.81
56.2				515.51	
65.0	7	1105			
66.0	8		2335		1.82
66.0				743.42	
76.0	9	1307			
77.0	10		2630		1.66
76.0	9	1307			
77.0	10		2630		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O\ (C_T)]}{A^{***}} \text{ AND } A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ. FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 6A
SERIES 3
FRICTION DATA - PILE 2 - 4 DAY STATIC LOAD TEST

DEPTH FEET	SERIES 3 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				188.07	
31.0	1	758			
32.0	2		1596		1.63
40.0				571.73	
40.0	3	1033			
41.0	4		1990		1.58
50.0	5	807			
51.0	6		1778		1.49
56.2				931.53	
65.0	7	1105			
66.0	8		2335		1.46
66.0				1174.55	
76.0	9	1307			
77.0	10		2630		1.46
76.0	9	1307			
77.0	10		2630		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \text{ AND } A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ.FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

TABLE 6B
SERIES 3
FRICTION DATA - PILE 2 - 11 DAY STATIC LOAD TEST

DEPTH FEET	SERIES 3 TEST NO.	TIP-ONLY TEST T_O (LBS)	EMBEDDED TEST T_E (LBS)	* MEASURED FRICTION F_M (PSF)	** C_T
16.4				220.64	
31.0	1	758			
32.0	2		1596		1.38
40.0				882.07	
40.0	3	1033			
41.0	4		1990		1.40
50.0	5	807			
51.0	6		1778		1.33
56.2				1125.0	
65.0	7	1105			
66.0	8		2335		1.31
66.0				1438.83	
76.0	9	1307			
77.0	10		2630		1.33
76.0	9	1307			
77.0	10		2630		

* F_M IS THE AVERAGE MEASURED FRICTION BETWEEN STRAIN GAGE BRIDGES ON THE INSTRUMENTED PIPE PILE, WHERE

$$F_M = \frac{T_E - [T_O (C_T)]}{A^{***}} \quad \text{AND} \quad A^{***} = \frac{(\pi)(2.375)(12)}{144} = .621 \text{ SQ. FT.}$$

** C_T = CORRECTION FOR TIP LOAD ON EMBEDDED TEST

In-Situ Bearing Test Data

From in-situ test series 1, 2 and 3, the tip-only tests were compared directly with the measured point bearing obtained from the instrumented test piles. After the test piles were driven and the depth of the tip was located, the in-situ tip-only tests were conducted at these depths. This enabled comparisons to be made between in-situ point bearing measurements and the actual measured point bearing.

The unit point bearing, q_t, for the in-situ tip-only test was obtained using the following equation:

$$q_t = \frac{T_o}{\text{tip area}} \tag{3}$$

where:

q_t = in-situ unit point bearing

T_o = load in pounds measured from in-situ tip-only tests

Tip Area = area of the blunt or cone tip.

The adjustment factor, C_p, for the in-situ measurements of the point bearing was obtained using the following equation:

$$C_p = \frac{q_t}{q_p} \tag{4}$$

where:

C_p = adjustment factor for point bearing

q_t = in-situ measured point bearing

q_p = test pile measured point bearing.

Thus, C_p is the ratio of the in-situ measurements to the test pile measurements of unit point bearing.

For test series 3, the tip area varied in diameter as shown in Fig. 11. The diameter of the base of the cone was used to compute the cross-sectional area of the cone tip in accordance with the procedure recommended by Begemann (3). The cross-sectional area of the tip in test series 3 was computed to be 0.0376 sq. ft. The unit point bearing, q_t, for series 1, 2, and 3 and the computed C_p values for the immediate, 4-day and 11-day static load tests are given in Table 7.

Correlation with Soil Properties

If in-situ measurements are made without having instrumented test pile data available, the adjustment factors, C_t and C_p, must be known in order to determine unit skin friction and point bearing. Correlation of C_t and C_p to some soil property which is obtainable by disturbed soil sampling methods is most desirable.

It was found in this study that the plasticity index, I_p, could be correlated to the adjustment factors, C_p and C_t. Plots of C_p and C_t versus the plasticity index of the soil for these test series are given in Figs. 13 and 14. Fig. 13 shows the relationship between the C_p values obtained from correlation with the immediate, 4-day and 11-day static load test on the two instrumented test piles. Fig. 14 shows the relationship between

TABLE 7
POINT BEARING DATA

DEPTH FEET	TEST PILE	SERIES AND TEST NO.	TIP-ONLY TEST (LBS)	* POINT BEARING FROM TIP-ONLY TEST (PSF) q_T	** POINT BEARING IMMEDIATE TEST (PSF) q_P	*** C_P	** POINT BEARING 4-DAY TEST (PSF) q_P	*** C_P	** POINT BEARING 11-DAY TEST (PSF) q_P	*** C_P
65.5	1	1-9	1189	38,604	9,329	4.13	1143	33.77	2887	13.37
80	2	2-11	680	22,078	9,329	2.37	16,415	1.34	10,614	2.07
76	2	3-9	1307	34,761	9,329	3.73	16,415	2.12	10,660	3.26

* POINT BEARING FROM TIP-ONLY TEST (q_T) WHERE $q_T = \dfrac{\text{LOAD (LBS)}}{\dfrac{(3.14)\left(\frac{2.375}{12}\right)^2}{4}} = \dfrac{\text{LOAD (LBS)}}{.0308 \text{ SQ. FT.}}$

FOR SERIES 3 $q_T = \dfrac{\text{LOAD (LBS)}}{.0376 \text{ SQ. FT.}}$

** POINT BEARING DETERMINED FROM FULL SCALE STATIC LOAD TEST ON INSTRUMENTED PIPE PILES (q_P) WHERE $q_P = \dfrac{\text{LOAD (LBS)}}{\dfrac{(3.14)\left(\frac{16}{12}\right)^2}{4}} = \dfrac{\text{LOAD (LBS)}}{1.40 \text{ SQ. FT.}}$

*** C_P IS FOUND BY DIVIDING TIP-ONLY VALUE FOR POINT BEARING q_T BY THE INSTRUMENTED PILE POINT BEARING q_P WHERE $\dfrac{q_T}{q_P} = C_P$

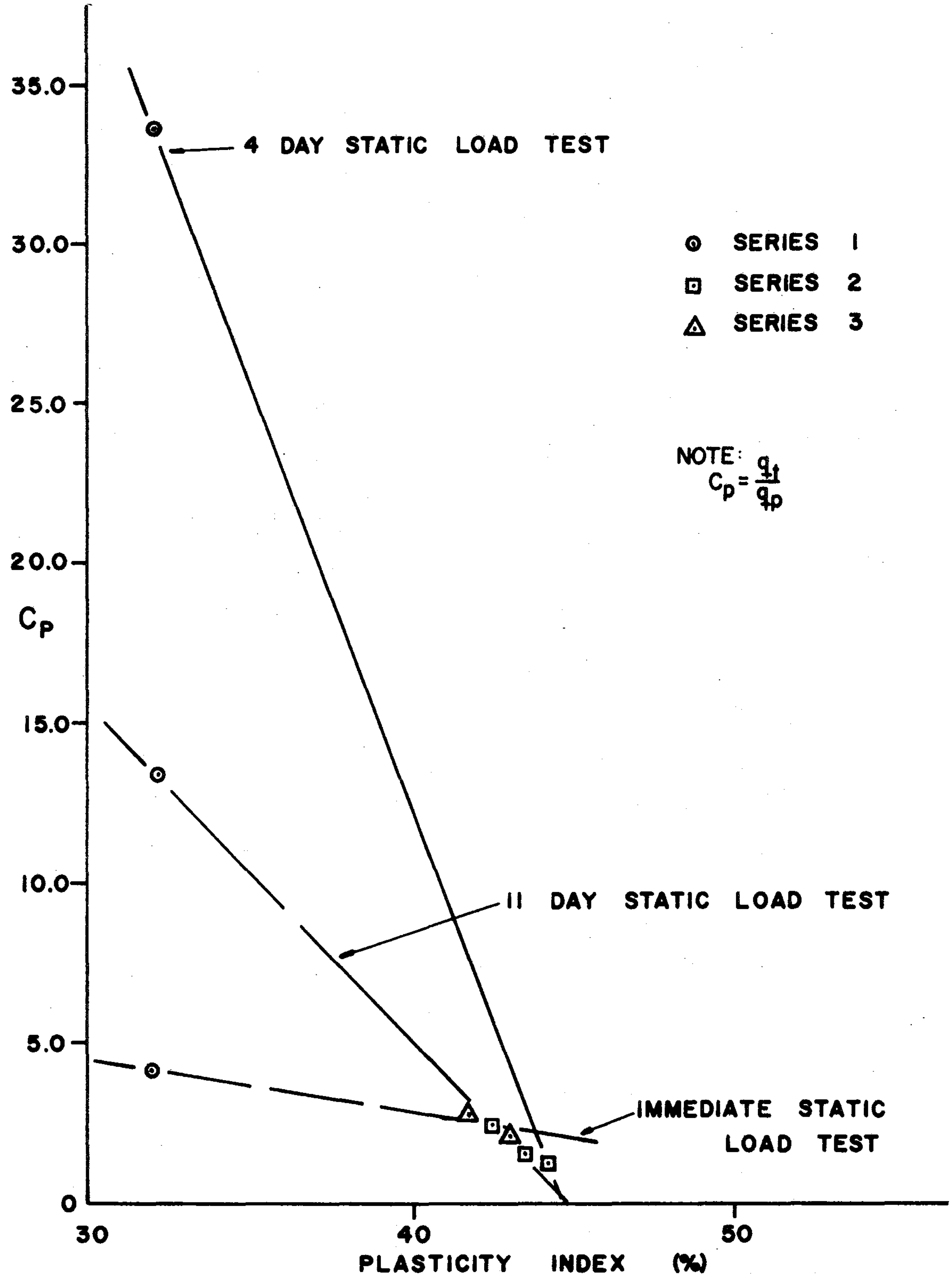

FIGURE 13 C_P VS. PLASTICITY INDEX

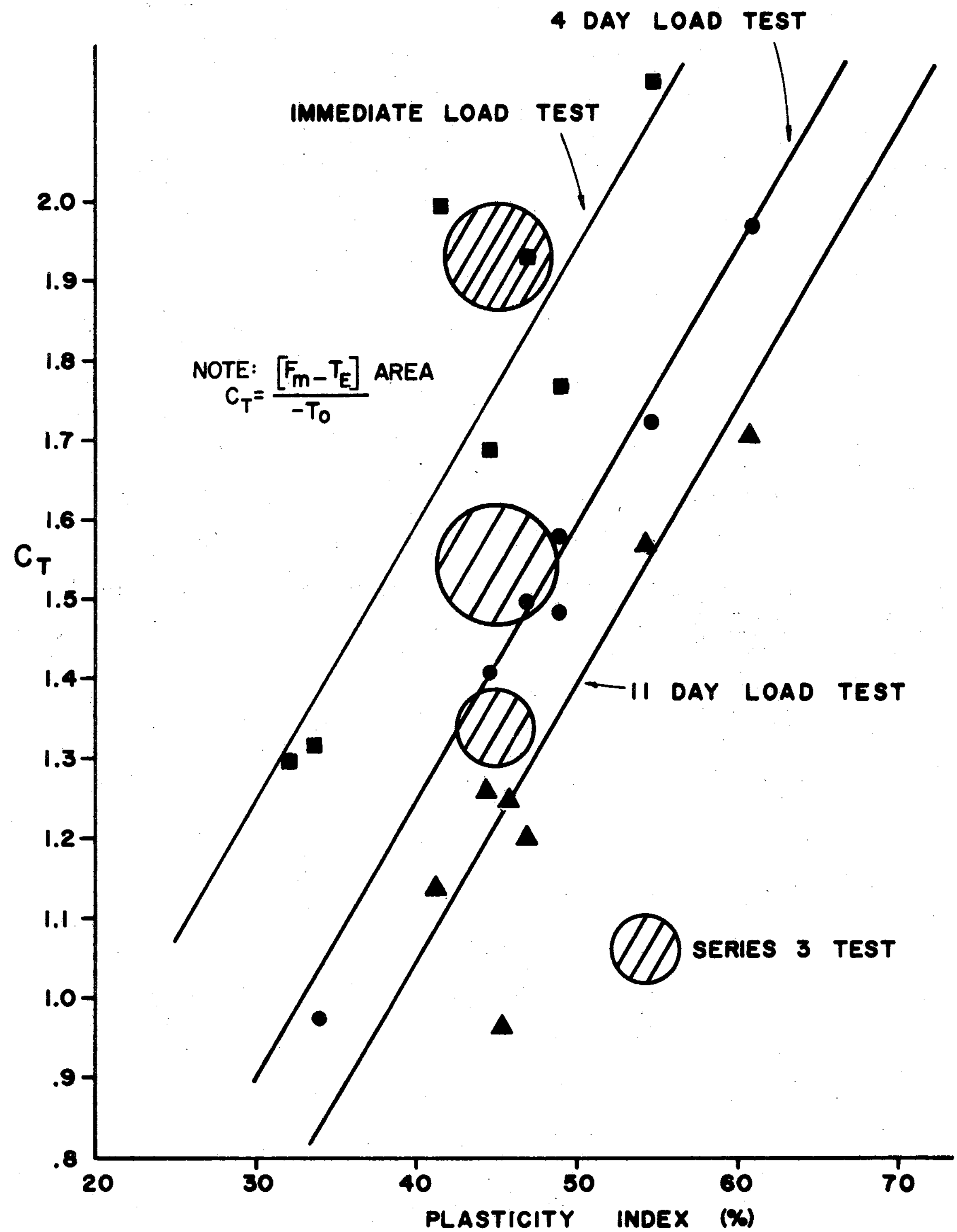

FIGURE 14 C_T VS. PLASTICITY INDEX

the C_t values obtained from correlation with the immediate, 4-day and 11-day static load tests. It should be noted that in test series 3 the C_t values and the plasticity index of the soil at each depth tested were approximately equal in value. Therefore, the shaded circular areas in Fig. 14 show the relationship between the C_t and the plasticity index of the soil for test series 3.

Since a correlation was obtained between the C_t and C_p values with the plasticity index of the soil, it is possible to use these values of C_t and C_p with in-situ measurements of the type presented in this study to determine the bearing capacity of a pile. The recommended procedure for estimating the ultimate bearing capacity, Q_u, of a pile and an example problem are given in Appendix VI.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The objectives of this research study consisted of two parts:

(1) to conduct a field test program at a test site which involves measurements of in-situ skin friction and point bearing during sampling operations, and

(2) to correlate the measured values of skin friction and point bearing taken during sampling operations with data obtained from full-scale instrumented test piles.

Both of these objectives were accomplished, but it is difficult to develop specific conclusions because only one test site in one soil is involved.

The following general conclusions can be made concerning the in-situ load measuring device and the test procedure used in this study:

1. The in-situ load measuring device can be used with standard drilling rig equipment to measure friction and bearing in medium to firm clays.
2. The test procedure used in this study with the load measuring device located at the ground surface can be used to measure friction and bearing in medium to firm clays.
3. The in-situ measurements of friction and bearing can be correlated with the corresponding measured values from an instrumented test pile. However, adjustment factors must be used to compensate for size and shape effects when using this device and procedure.
4. The adjustment factor developed in this study can be related to the plasticity index of the soil. (See Figs. 13 and 14). However, these relationships are only valid for the soils tested during this study.

Recommendations

It has been shown as a result of this study that the load measuring device cannot be used either in very soft or very stiff soils. Future in-situ tests should include a different method of testing in very soft and in very stiff soils. Testing in very soft soils could be accomplished by utilizing a field vane device. Testing in very stiff soils could be accomplished by using the standard penetration test.

It has also been shown in this study that the adjustment factors developed can be related to plasticity index. However, future in-situ tests and full-scale instrumented pile tests should be conducted in a variety of test sites in order to verify these relationships. Much additional data would be necessary before the procedure outlined in Appendix VI could be used to estimate the bearing capacity of a pile.

APPENDIX I.--REFERENCES

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APPENDIX II.--NOTATION

Q_u = ultimate bearing capacity (tons)

f = unit skin friction (lbs./sq. ft.)

A_s = embedded surface area (sq. ft.)

q = unit point bearing (lbs./sq. ft.)

A_p = pile end area (sq. ft.)

F_M = average measured unit skin friction obtained from the instrumented test piles (lbs./sq. ft.)

T_E = load measured from the in-situ embedded tests (lbs.)

T_O = load measured from the in-situ tip-only tests (lbs.)

C_T = adjustment factor for the in-situ tip-only tests (dimensionless)

q_t = in-situ measured point bearing (lbs./sq. ft.)

q_p = test pile measured point bearing (lbs./sq. ft.)

C_p = adjustment factor for point bearing (dimensionless)

I_p = plasticity index (%)

APPENDIX III

STATIC LOAD TEST DATA

FOR INSTRUMENTED TEST PILES 1 & 2

TABLE III-1.--TEST PILE 1, IMMEDIATE STATIC LOAD TEST

Average Load 1A & 1B*, in kips	Average Load 2A & 2B*, in kips	Average Load 3A & 3B*, in kips	Average Load 4A & 4B*, in kips	Average Load 5A & 5B*, in kips
20.6	18.0	11.9	4.3	1.7
34.6	32.5	23.1	10.3	3.5
49.9	48.1	36.5	18.5	5.8
69.4	67.1	54.7	30.2	9.3
83.8	82.4	68.2	41.7	13.7
92.3	89.1	75.3	45.8	18.1

*See Fig. 7 for location of bridges with respect to Ground Surface.

TABLE III-2.--TEST PILE 1, 4-DAY STATIC LOAD TEST

Average Load 1A & 1B*, in kips	Average Load 2A & 2B*, in kips	Average Load 3A & 3B*, in kips	Average Load 4A & 4B*, in kips	Average Load 5A & 5B*, in kips
18.6	17.4	7.0	1.9	0.3
37.0	33.3	14.5	3.7	0.8
55.0	54.0	24.3	6.2	1.2
71.8	67.5	33.3	9.2	2.0
85.5	81.7	43.7	12.1	2.8
101.3	99.6	54.5	16.4	3.6
117.0	117.0	69.5	21.8	4.4
134.7	131.1	79.2	26.7	5.3
142.0	141.1	85.9	30.3	5.6
149.0	146.3	91.9	32.4	6.2
155.6	154.3	96.8	38.0	7.2

*See Fig. 7 for location of bridges with respect to Ground Surface.

TABLE III-3.--TEST PILE 1, 11-DAY STATIC LOAD TEST

Average Load 1A & 1B*, in kips	Average Load 2A & 2B*, in kips	Average Load 3A & 3B*, in kips	Average Load 4A & 4B*, in kips	Average Load 5A & 5B*, in kips
17.8	16.0	6.3	1.6	0.3
34.3	32.9	14.2	3.7	0.6
50.5	47.3	21.3	5.5	1.5
68.2	64.8	29.7	7.5	1.8
85.8	81.8	37.1	9.8	1.6
102.1	99.4	48.9	15.0	2.9
119.3	117.4	61.2	15.6	2.9
134.4	133.5	70.3	18.7	4.6
142.7	142.2	76.0	20.5	4.8
152.1	151.5	82.6	22.5	5.3
159.7	158.9	87.4	24.3	5.5
167.6	159.3	92.4	25.5	5.7
174.0	173.3	96.8	26.1	6.3
182.8	182.8	102.4	29.4	6.7
189.7	188.2	105.3	31.4	7.3
199.9	196.8	109.2	34.3	8.7

*See Fig. 7 for location of bridges with respect to Ground Surface.

TABLE III-4.--TEST PILE 2, IMMEDIATE STATIC LOAD TEST

Average Load 1A & 1B*, in kips	Average Load 2A & 2B*, in kips	Average Load 3A & 3B*, in kips	Average Load 4A & 4B*, in kips	Average Load 5A & 5B*, in kips
18.9	14.7	11.8	8.6	0.9
39.0	34.2	27.3	12.3	2.0
56.6	46.4	42.4	20.3	3.7
76.1	64.8	59.4	32.0	6.9
67.8	72.7	66.7	39.0	8.2
92.4	80.0	74.4	40.0	10.7
100.2	88.0	80.5	48.1	16.2

*See Fig. 8 for location of bridges with respect to Ground Surface.

TABLE III-5.--TEST PILE 2, 4-DAY STATIC LOAD TEST

Average Value 1A & 1B*, in kips	Average Value 2A & 2B*, in kips	Average Value 3A & 3B*, in kips	Average Value 4A & 4B*, in kips	Average Value 5A & 5B*, in kips
19.2	13.6	6.4		0.8
41.4	28.0	14.3	6.0	1.4
56.4	43.4	23.0	9.6	1.4
75.7	60.0	34.4	15.1	2.7
92.9	76.6	43.4	19.8	3.4
112.7	92.4	56.1	26.2	5.3
129.6	110.1	69.2	33.5	5.2
148.4	125.9	87.2	43.1	6.5
165.6	144.4	104.3	53.7	9.1
183.4	158.6	120.9	64.3	11.3
200.1	175.1	136.8	78.3	27.9

*See Fig. 8 for location of bridges with respect to Ground Surface.

TABLE III-6.--TEST PILE 2, 11-DAY STATIC LOAD TEST

Average Load 1A & 1B*, in kips	Average Load 2A & 2B*, in kips	Average Load 3A & 3B*, in kips	Average Load 4A & 4B*, in kips	Average Load 5A & 5B*, in kips
11.97	8.53	3.19	1.34	0.12
34.42	25.42	11.21	5.13	0.67
55.06	43.00	19.89	8.13	1.23
75.26	58.68	28.35	12.25	1.91
91.94	73.06	36.15	16.03	2.47
111.72	90.05	45.71	21.05	3.48
131.39	107.48	57.14	26.06	4.38
148.60	122.87	67.24	31.18	5.05
167.42	140.04	80.10	37.09	6.06
175.76	148.46	87.35	41.06	6.74
184.20	156.38	93.94	44.50	7.41
193.50	164.98	101.53	48.66	8.20
200.88	171.89	108.99	52.89	8.98
208.68	179.01	117.45	57.58	9.88
217.66	187.73	126.79	62.69	11.23
226.53	196.18	135.81	67.81	12.92
235.74	204.82	145.48	73.94	15.27
242.90	212.37	153.28	82.63	20.89

*See Fig. 8 for location of bridges with respect to Ground Surface.

APPENDIX IV

LOAD DISTRIBUTION VERSUS DEPTH CURVES

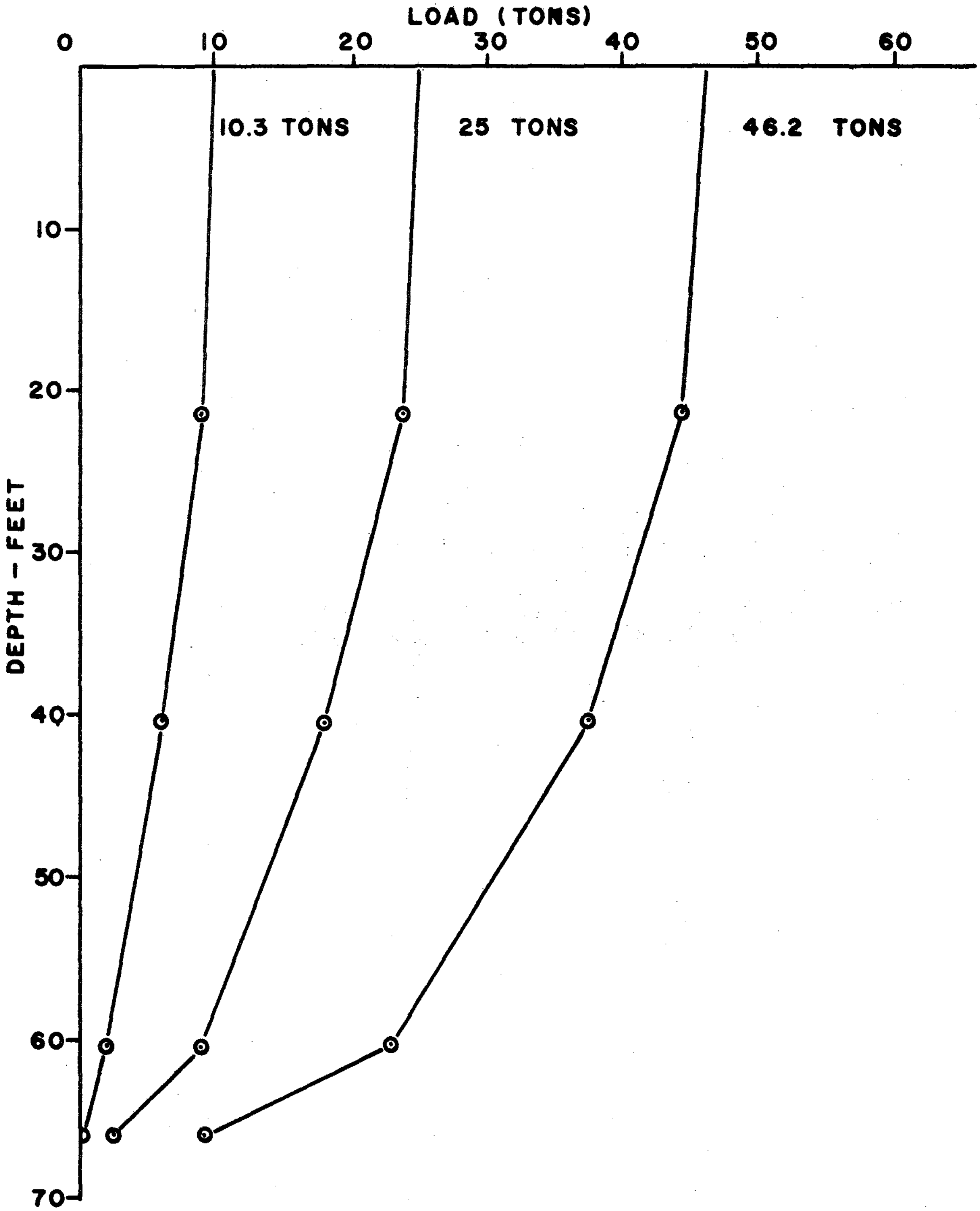

FIGURE IV-1 LOAD VS. DEPTH
PILE 1 "IMMEDIATE STATIC LOAD TEST"

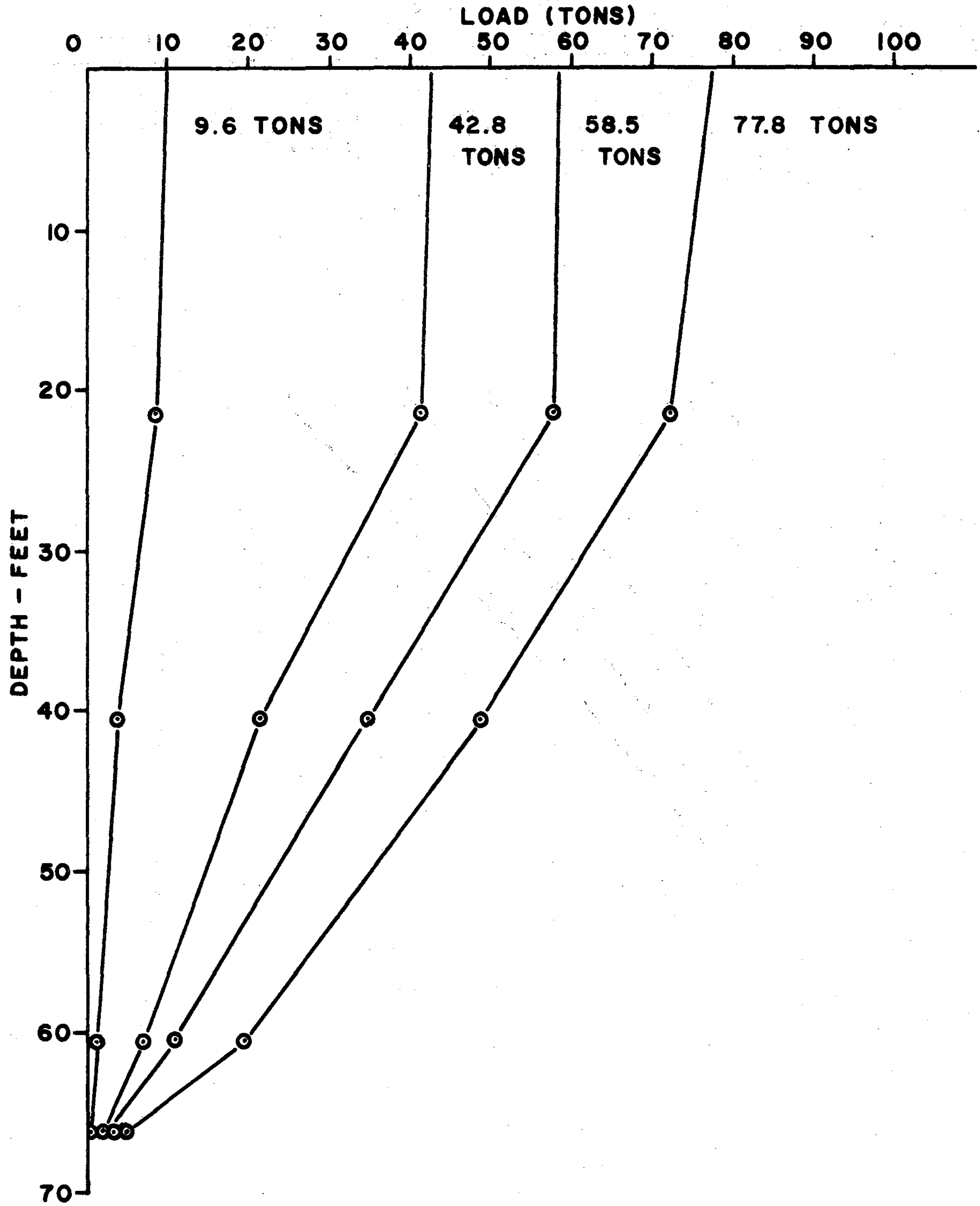

FIGURE IV-2 LOAD VS. DEPTH
PILE I "4 DAY STATIC LOAD TEST"

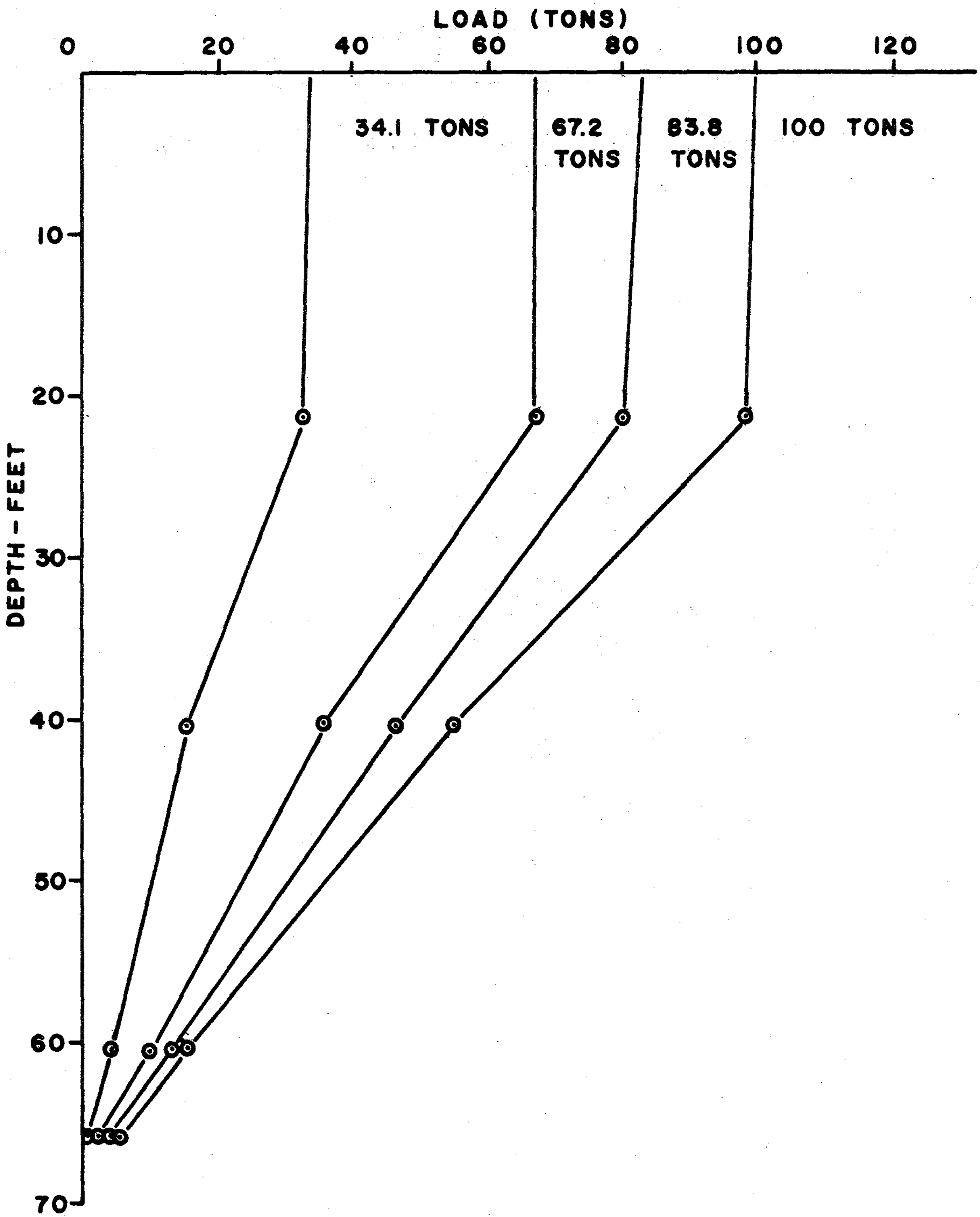

FIGURE IV-3 LOAD VS. DEPTH
PILE 1 "11 DAY STATIC LOAD TEST"

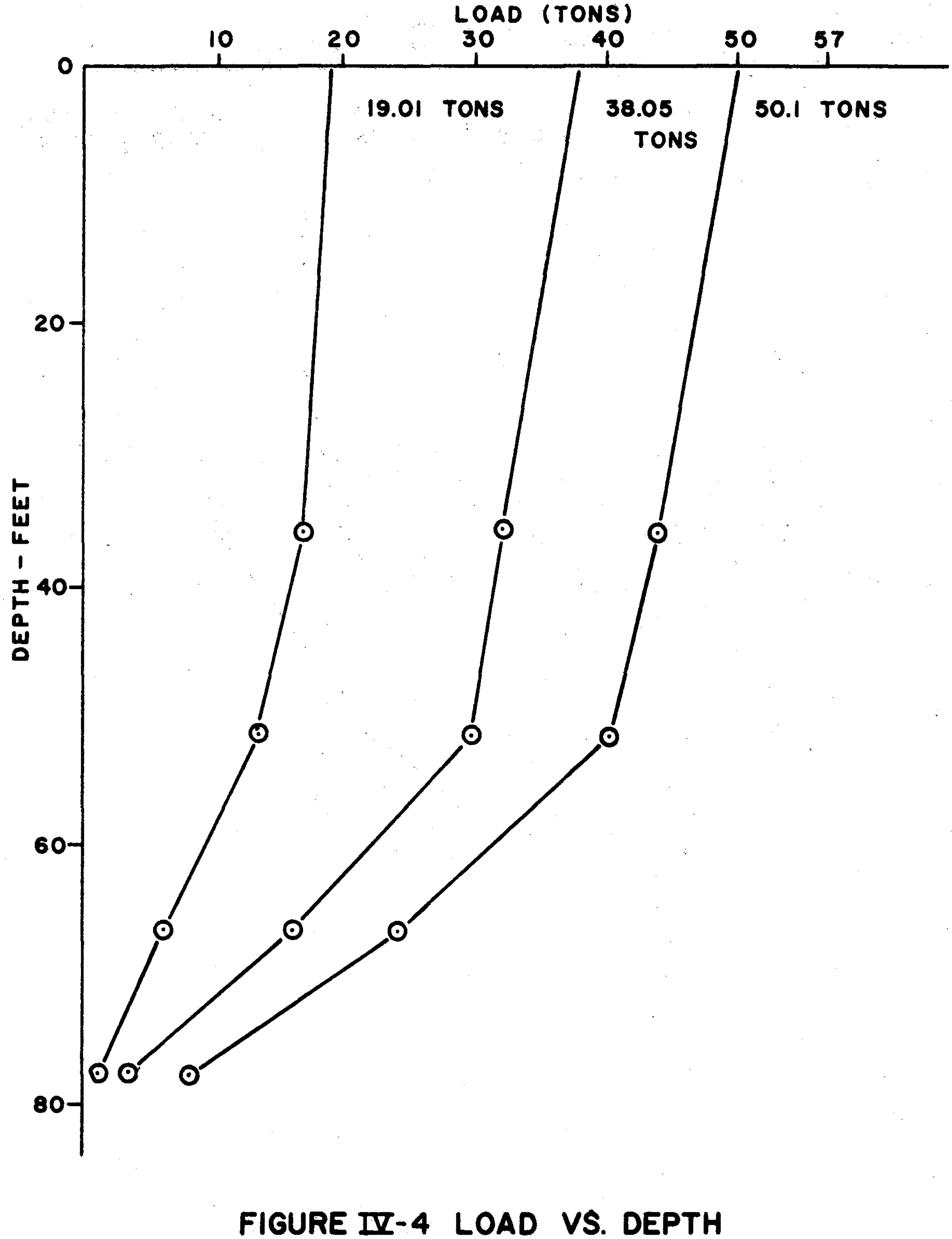

FIGURE IV-4 LOAD VS. DEPTH
PILE 2 "IMMEDIATE STATIC LOAD TEST"

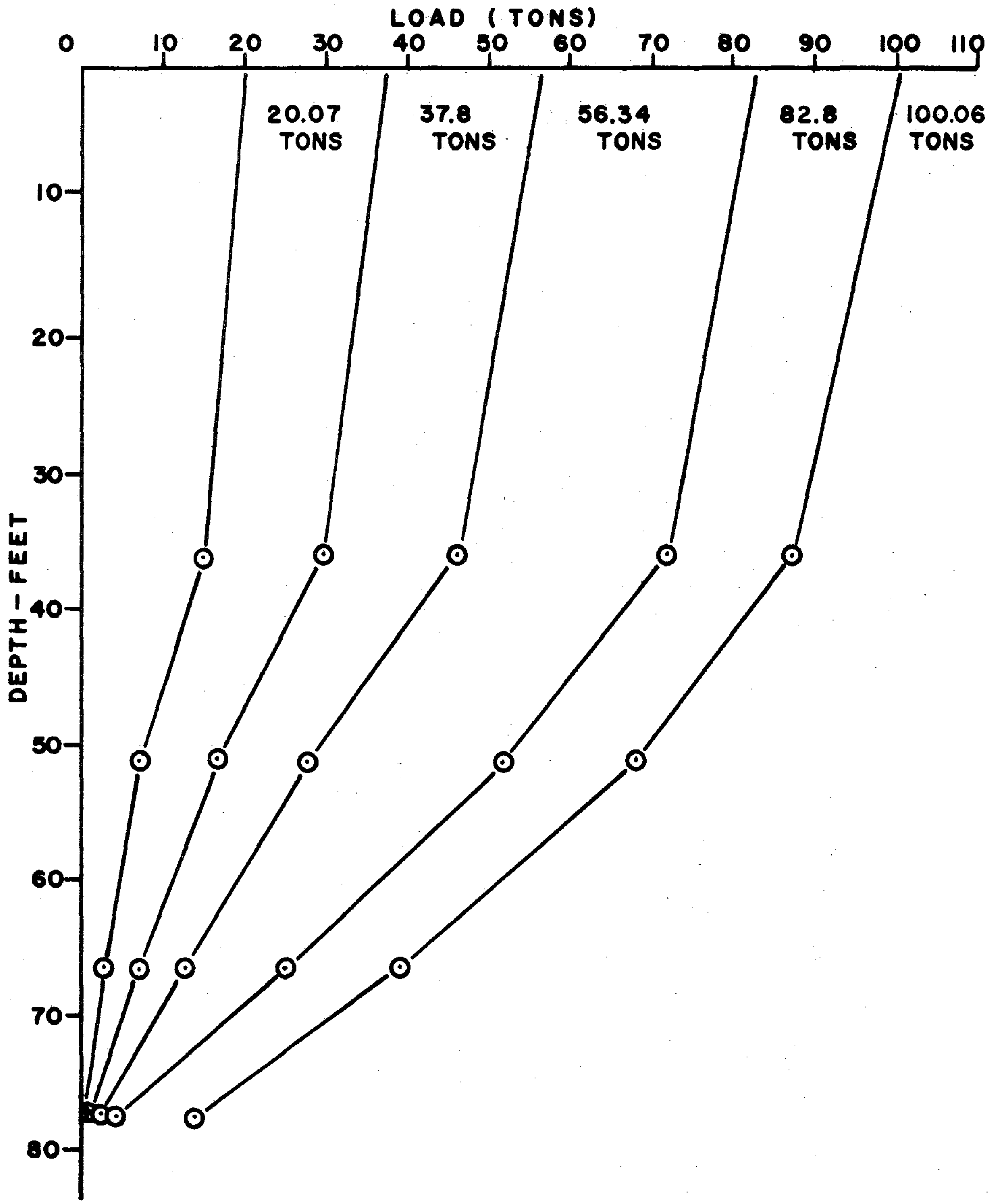

FIGURE IV-5 LOAD VS. DEPTH
PILE 2 "4 DAY STATIC LOAD TEST"

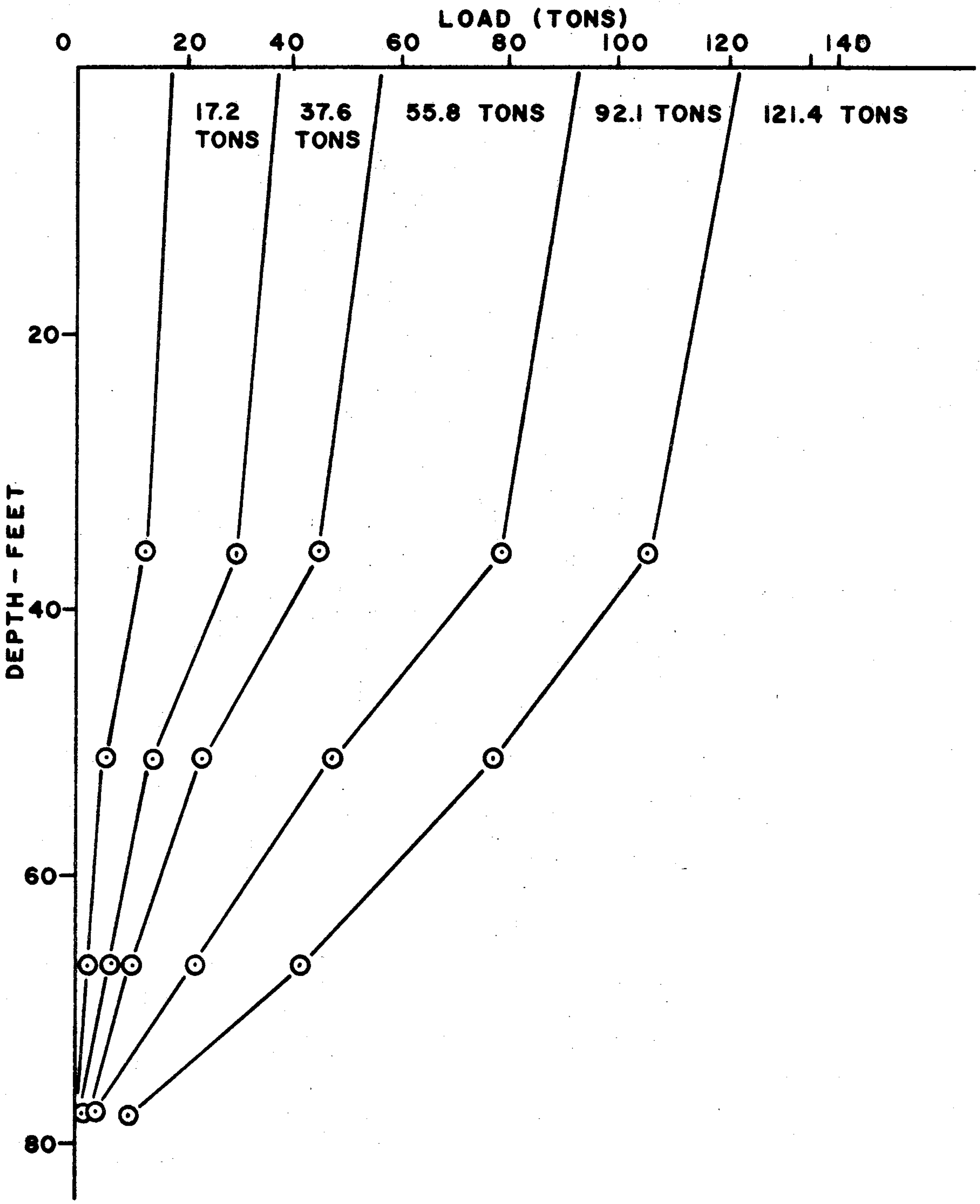

FIGURE IV-6 LOAD VS. DEPTH
PILE 2 "11 DAY STATIC LOAD TEST"

APPENDIX V

LOAD-SETTLEMENT CURVES

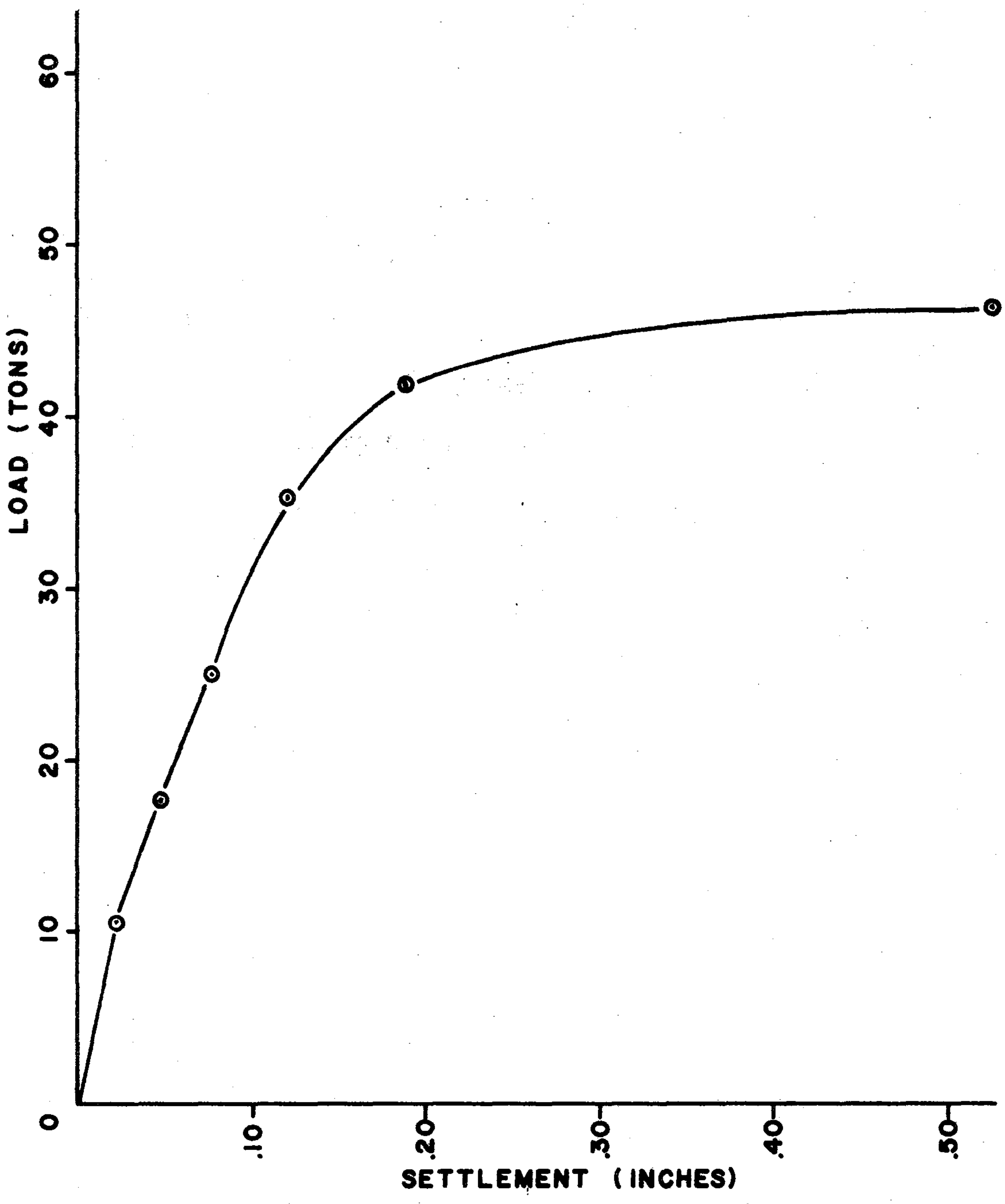

FIGURE V-1 LOAD VS. SETTLEMENT CURVE
PILE 1 "IMMEDIATE STATIC LOAD TEST"

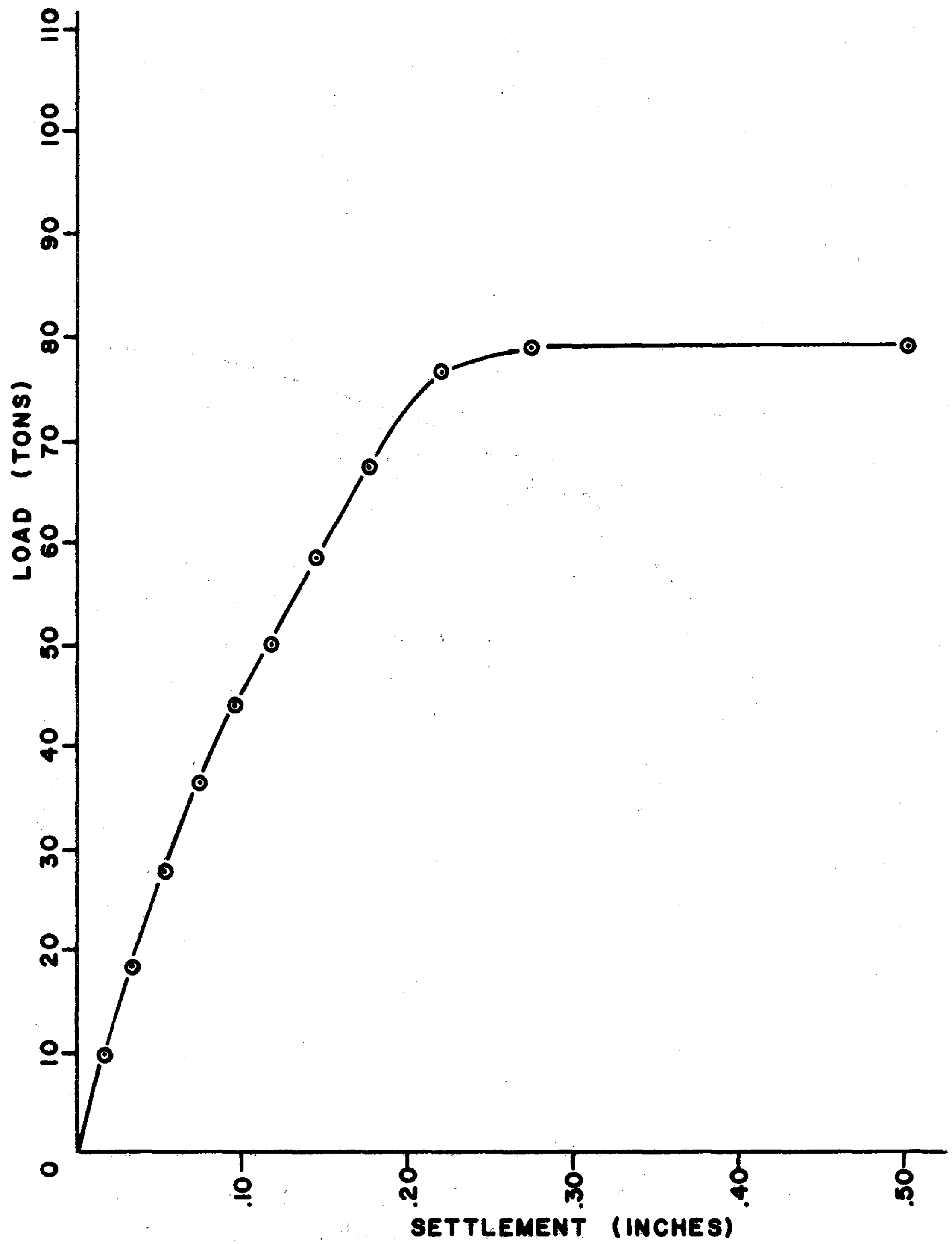

FIGURE V-2 LOAD VS. SETTLEMENT CURVE
PILE 1 "4 DAY STATIC LOAD TEST"

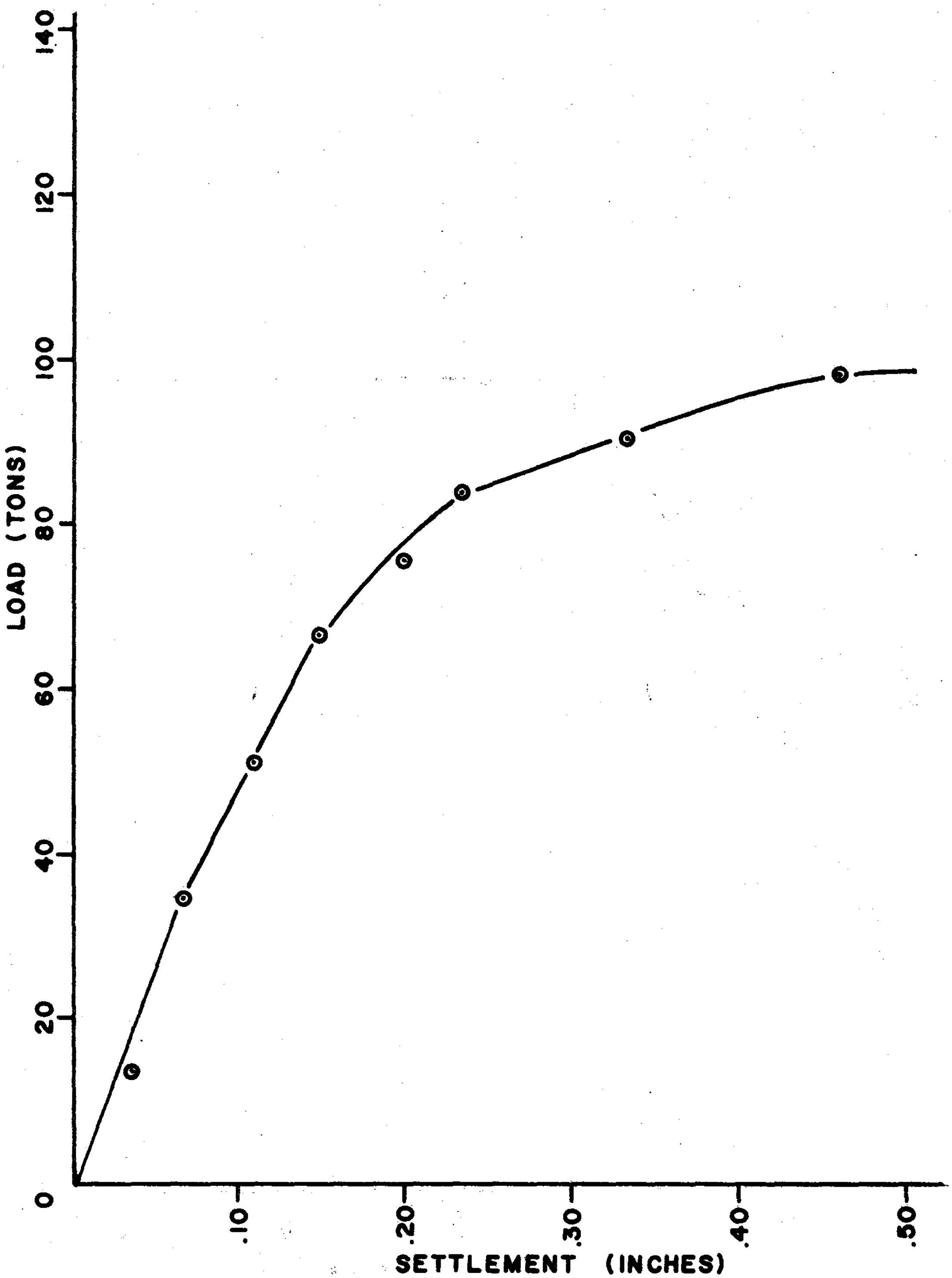

FIGURE V-3 LOAD VS. SETTLEMENT CURVE
PILE I "11 DAY STATIC LOAD TEST"

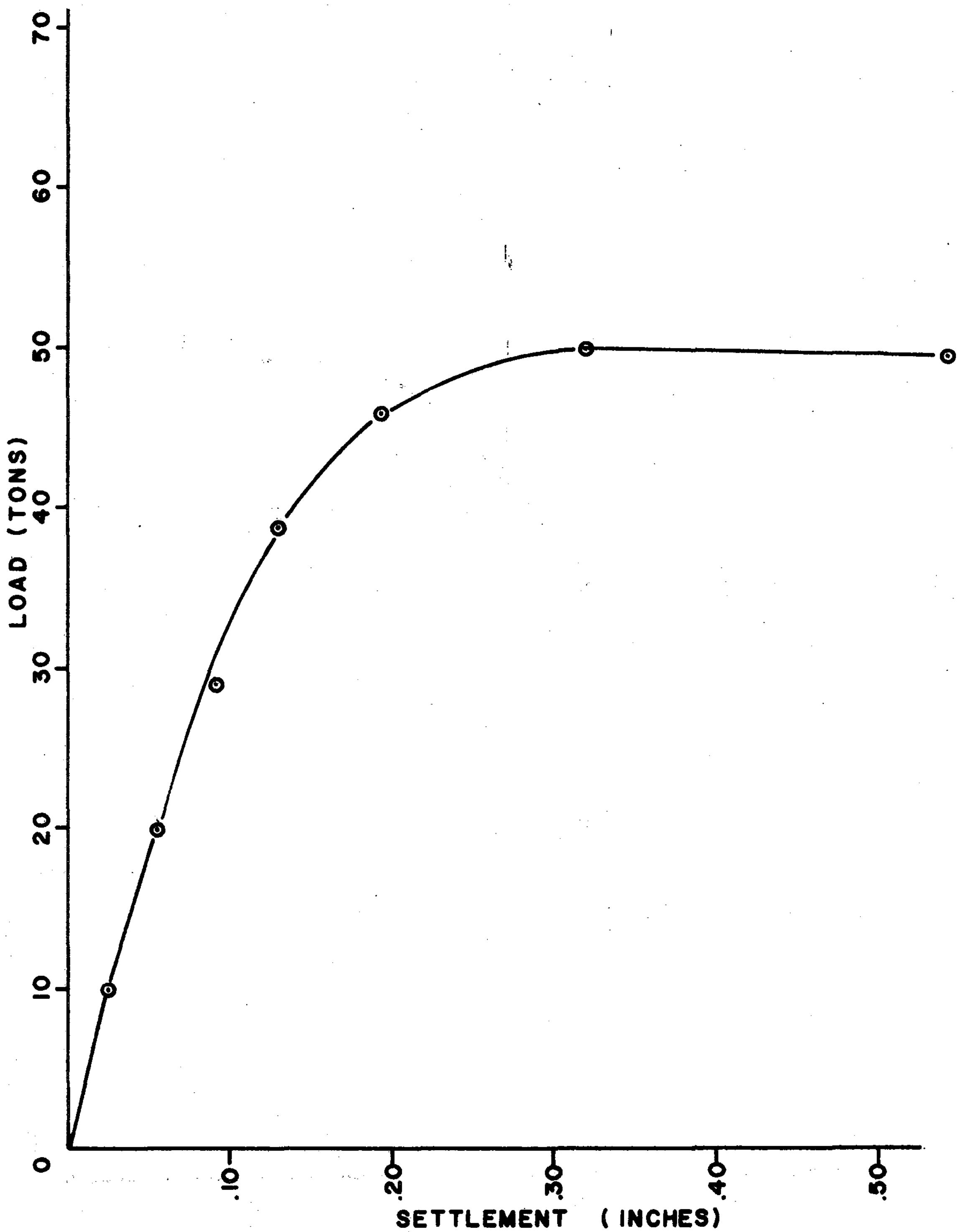

FIGURE V-4 LOAD VS. SETTLEMENT CURVE
PILE 2 "IMMEDIATE STATIC LOAD TEST"

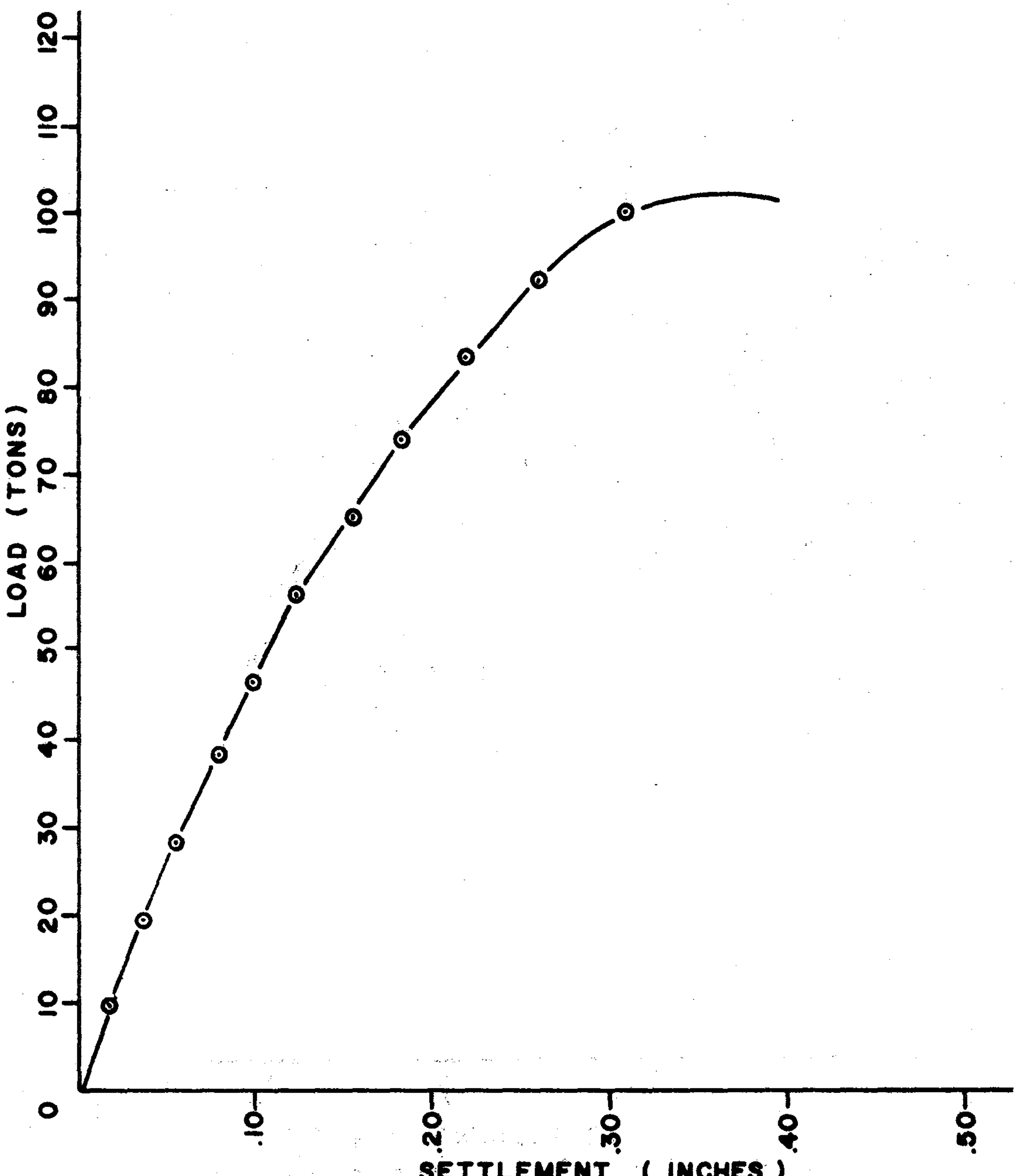

FIGURE V-5 LOAD VS. SETTLEMENT CURVE
PILE 2 "4 DAY STATIC LOAD TEST"

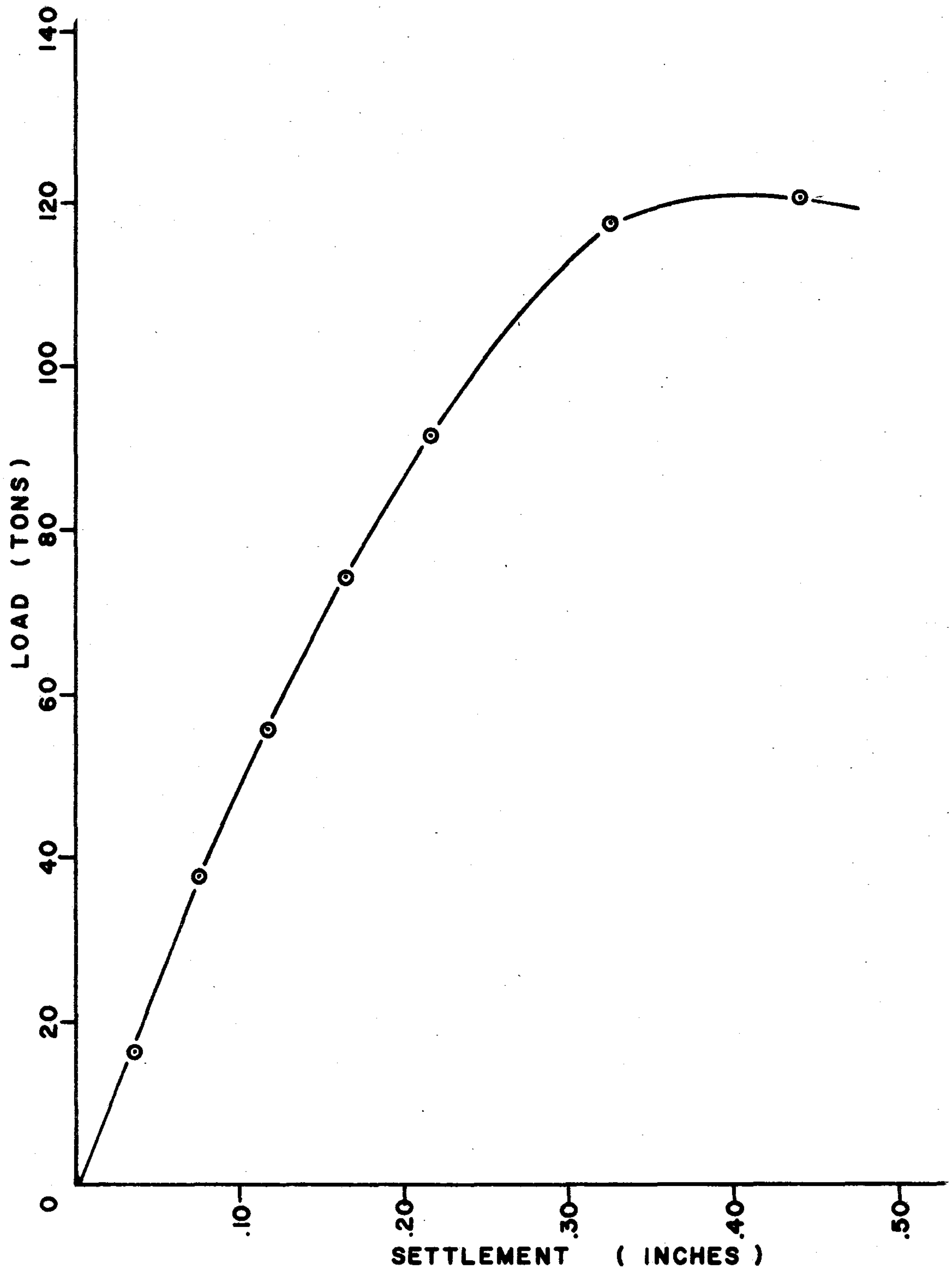

FIGURE V-6 LOAD VS. SETTLEMENT CURVE
PILE 2 "11 DAY STATIC LOAD TEST"

APPENDIX VI.--PROCEDURE AND EXAMPLE PROBLEM FOR ESTIMATING THE BEARING CAPACITY OF A PILE

Procedure

The following method is recommended for estimating the bearing capacity of a pile:

(1) Conduct several in-situ tip-only and embedded tests in each different soil strata.

(2) Take an average value for the tip-only and embedded tests in each soil strata.

(3) From graphs of C_p and C_T versus the plasticity index as shown in Figs. 13 and 14, and knowing the P_I of the soil, determine the C_T and C_p values needed.

(4) Compute the unit skin friction, f, for the pile using equation 2 and the in-situ measurements.

(5) Compute the load carried in each different soil strata by multiplying the length of the pile in each different soil strata by the corresponding unit skin friction, f.

(6) Sum up the load carried by the pile in friction.

(7) Determine the unit point bearing, q, of the pile by using the appropriate tip-only tests and its corresponding C_p value.

(8) Determine the area of the tip of the pile and multiply this area times the unit point bearing, q, to obtain the load carried by the tip of the pile.

(9) Add the load carried by the tip of the pile and the load carried by skin friction to determine the bearing capacity of the pile.

Example Problem

The following is an example problem for determine bearing capacity of Test Pile 2 with a 16 in. diameter and embedded 74 ft. The bearing capacity is computed using C_T and C_p values obtained in Test Series 2.

Depth	Load Carried by Skin Friction	lbs./sq. ft.
0'-32'	No tests were run.	
32'-48'	C_T values of 1.95 and 1.65. (Found in Table 5) Average C_T value 1.83.	
	Friction = $\frac{(1200 \text{ lbs.}) - (580 \text{ lbs.})(1.83)}{0.621 \text{ sq. ft.}}$ =	$\frac{266 \text{ lbs.}}{\text{sq. ft.}}$
48'-63'	C_T value of 1.31	
	Friction = $\frac{(1400 \text{ lbs.}) - (890 \text{ lbs.})(1.26)}{0.621 \text{ sq. ft.}}$ =	$\frac{516 \text{ lbs.}}{\text{sq. ft.}}$
63'-74'	C_T value of 1.68.	
	Friction = $\frac{(1635 \text{ lbs.}) - (753 \text{ lbs.})(1.56)}{0.621 \text{ sq. ft.}}$ =	$\frac{745 \text{ lbs.}}{\text{sq. ft.}}$

The next step is to compute the load carried in each segment of the test pile.

Depth	Total Load Carried in Each Segment		Load
0'-32'	From Load vs. Depth Distribution Curve for Pile 2, it was found that the first 32 ft. carried a load of 12,000 lbs.		12,000 lbs.
32'-48'	$\frac{266 \text{ lbs.}}{\text{sq. ft.}}$ X $(16')(3.14)(\frac{16''}{12''})$	=	15,150 lbs.
48'-63'	$\frac{516 \text{ lbs.}}{\text{sq. ft.}}$ X $(15')(3.14)(\frac{16''}{12''})$	=	32,400 lbs.
63'-74'	$\frac{745 \text{ lbs.}}{\text{sq. ft.}}$ X $(11')(3.14)(\frac{16''}{12''})$	=	34,400 lbs.
		TOTAL	93,950 lbs.

To find the point bearing, use series 2 test 9 tip-only test value of 680 lbs. and determine the in-situ unit point bearing, q_t.

$$q_t = \frac{680 \text{ lbs.}}{\frac{(3.14)(\frac{2.375''}{12''})^2}{4}} = \frac{22{,}078 \text{ lbs.}}{\text{sq. ft.}}$$

Taking the value of $\frac{22{,}078 \text{ lbs.}}{\text{sq. ft.}}$ and dividing this value by C_p=2.37 to obtain a unit point bearing, q_p for the test pile gives:

$$q_p = \frac{\frac{22{,}078 \text{ lbs.}}{\text{sq. ft.}}}{2.37} = \frac{9330 \text{ lbs.}}{\text{sq. ft.}}$$

Now, multiply $\frac{9330 \text{ lbs.}}{\text{sq. ft.}}$ by the area of the tip of the pile, 1.4 sq. ft. to obtain the point bearing carried by the test pile.

$$q = \left(\frac{9330 \text{ lbs.}}{\text{sq. ft.}} \right)(1.4 \text{ sq. ft.}) = 13{,}020 \text{ lbs.}$$

Adding the load carried by skin friction and point bearing to give an estimated bearing capacity, the result is:

$$\begin{array}{r} 93{,}950 \text{ lbs.} \\ \underline{13{,}020 \text{ lbs.}} \\ 106{,}970 \text{ lbs.} = 53.485 \text{ tons.} \end{array}$$

The bearing capacity of test pile 2 obtained from the immediate static load test data was equal to 50.1 tons. It should be noted that test pile 2 was loaded until a plunging type failure was obtained.