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CASE HISTORY OF A TEST PILE PROGRAM FOR
THE REDEVELOPMENT OF NEWARK AIRPORT

BY

PATRICK DE MICHELE

A THESIS

PRESENTED IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE

OF

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AT

NEWARK COLLEGE OF ENGINEERING

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Newark, New Jersey
1967

APPROVAL OF THESIS

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THE REDEVELOPMENT OF NEWARK AIRPORT

BY

PATRICK DE MICHELE

FOR

DEPARTMENT OF CIVIL ENGINEERING

NEWARK COLLEGE OF ENGINEERING

BY

FACULTY COMMITTEE

APPROVED: _____

NEWARK, NEW JERSEY

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ABSTRACT

A pile load test program was conducted at Newark Airport to determine the most economical piles to be utilized to support the structure in the Redevelopment of Newark Airport. The controlling criteria were the pre-determined design capacity of 50 to 100 tons for the piles and conformance to the Newark Building Code regarding the maximum allowable loads on the Airport's bearing strata and the allowable settlements for test loaded piles. Thirty-four piles, ten timber, four steel H-piles, four pre-stressed concrete, seven cobo shell and nine steel pipe piles, were driven to determine driving characteristics of the piles and the reaction of the supporting soils strata. Six piles, test loaded in conformance with the City Building Code, were used to test van Weele's theory of separating the bearing capacity of a pile into frictional and tip resistance. Three of these piles, a single concreted cobo shell, and two steel pipe piles, one of which was concreted, were instrumented along the length of the pile to determine the portion of the applied load which was assumed by the frictional resistance. All the piles tested, except for the timber piles, exceeded the design loads and the criteria of the Building Code. Van Weele's method of determining the load transferred to friction by calculating the elastic recovery of the pile was compared to the load transferred to friction by instrumenting the test piles. The frictional forces from the instrumented tests exceeded the results obtained by van Weele's method. Pile

movement and peak frictional resistances for the tests were compared to shear strengths of the soil. Residual forces resulting from the pile driving and forces exerted on the instrumentation from the concrete encasement were in evidence but the values were not calculated.

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INTRODUCTION

A pile load test program was conducted at Newark Airport to determine the piles to be utilized in the Redevelopment of Newark Airport. The Structures to be supported in the Redevelopment consist of three three level Terminal Buildings, approximately 600' long by 160' wide, with nine connecting satellite terminals, bridges, bi-level roadways, a heating and refrigeration plant and other associated structures.

There were four objectives for conducting the test pile program.

First, based on column loads for the Terminal Buildings, piles capable of supporting 50 to 100 tons would be required. To attain this capacity in the subject soils strata, a pile would have to be point bearing on bedrock. The Newark City Building Code limited piles founded in the shale bedrock at the Airport to an 80 ton capacity. Thus, it was anticipated that if the pile tests exceeded the 80 ton capacity established by the Building Code, the governing criteria for the local conditions might be relaxed.

Second, the evaluation of the driving performance of the various types of piles would aid the design engineer in establishing the contract requirement. Piles driven under previous contracts were unable to penetrate through some of the soils strata requiring spudding, jetting, pile modification and tip elevation re-evaluation which caused costly delays to the projects.

The compilation of research data was attained. Certain piles were instrumented to determine stresses under various test loads for determining the proportion of load distributed in friction and that reaching the pile tip. These results were compared to van Weele's method of separating the bearing capacity of a pile into skin friction and point resistance. The driving characteristics and tip elevations of steel pipe piles of different wall thicknesses were observed.

Most important, the premise for performing the pile tests was to determine the pile that meets the design load criteria and provides the greatest economy.

SOIL CONDITIONS

Figures 1 and 2 show the soil profile obtained from borings at Sites "A" and "B" prior to driving.

An early phase of the Redevelopment Program called for the placing of hydraulic fill within the complex to overlay the tidal marsh providing a suitable fill and a working elevation for construction. It would also surcharge the area to provide primary settlement by consolidation of the organic silt.

The general geologic condition of the Airport as determined in a Preliminary Report on Soil Studies - 1963, showed that the Airport had been developed by filling over a tidal marsh deposit varying from 2' to 20' in thickness. This deposit is composed of unusually soft compressible organic silts, with moisture contents ranging from 60% to 100% and variable degrees of peaty soils sometimes attaining a moisture content of 600%. These organic and peaty soils are characteristic of the tidal marsh deposit. The upper portions contain extensive amount of dead vegetation interposed with fine-grained silts deposited by streams which emptied into the marsh. The lower portions of this deposit are frequently coarse grained, varying from sandy organic silts to fine-grained sands.

Underlying the tidal marsh are variable and not often distinct layers of glacial outwash and lake deposits, residual soil and basement rock in this sequence. The glacial lake deposits which underlie the

Airport are usually interposed with outwash material. These coarse-grained soils were probably washed down from the more highly-elevated outwash deposits that are located to the north and west of the Airport. These lake deposits which consist of reddish brown silts and clays, frequently varved, are pre-consolidated to pressures of 3 to 4 tons per square foot, while present overburden-pressure is about $1\frac{1}{2}$ tons per square foot. Exactly how these soils became overconsolidated is not known; possibly the terminal moraine was breeched and the glacial lake drained when the ocean was much lower than its present level. The loss of buoyancy resulting from a lowering of the water level of 50 feet would have subjected the lake deposits to an overburden load of more than 3 tons per square foot.

Bedrock lies from approximately elevation 260' at the east of the field to elevation 186' at the extreme west end. (Newark Airport Datum - Elevation 297.347 is equivalent to U.S. Coast and Geodetic Elevation of 0.000 which is mean sea level at Sandy Hook). It is a soft red shale, known as Brunswick shale, from the Triassic period. Overlying the bedrock is the weathered upper surface, a dense clayey silt with shale fragments.

Site "A" was 13' above Site "B". The quality of stone at Site "B", where there was 5' of decomposed rock due to weathering and only 17% of core recovery as compared to Site "A" where there was no

measurable amount of weathering and a 63% of core recovery, was inferior. The piles driven at Site "A" would be more economical, due to the length, and would probably be seated into the bedrock more readily.

INSTRUMENTATION

Three of the test piles to be loaded were instrumented to measure the loads distributed to skin friction and point bearing. To calculate the stresses along the length of pile 5b, a 12-3/4" diameter steel pipe pile with 3/8" thick walls, strain rods were installed. Test piles 9A, a 12-3/4" diameter steel pipe pile with 1/4" thick walls, and 17A, a 12" diameter "Cobi" pile, both concreted, utilized Carlson strain meters.⁸

In conjunction with these aforementioned measuring devices a Wild precise N-3 level was utilized to record the gross settlement of the subject piles. Permanent bench marks, two of the untested piles which were driven to bedrock, were utilized as back sights. In a similar fashion, a permanent foresight was utilized at the pile as shown in photos 1 and 2.

The strain rods, which were used to measure the elastic shortening of the pile due to loading were anchored at several locations inside a hollow steel pipe pile, (figure 5), thus dividing the pile into several segments as recommended by Coyle and Reese.¹ Dial indicators, as shown on photo 1, were attached to the strain rods to measure the elastic shortening of each pile segment. Knowing the modulus of elasticity and the dimensions of the pile, the stress at each strain indicator in the pile was readily calculated.

The Carlson Strain Meter⁸ is an unbonded, electric resistivity strain measuring instrument which is enclosed in a completely sealed brass tube, one inch in diameter and ten inches long. The meter not only reads to an accuracy of a few micro inches, but it also is an accurate thermometer.

Installation of the strain meter (figure 6) was accomplished with care since being unbonded it is sensitive to shock. Prior to installation of the meter, it was centered to a positioning bracket which aligned it to the axis of the pile. Concrete was first placed to the bottom of the meter location, then the positioning bracket and meter were lowered into place. Concrete was carefully placed around and one foot over the meter using a long cylindrical bucket with a flap bottom. The pile was then concreted to just below the next meter installation and the procedure was repeated until all the meters were placed. The meters were pre-set prior to installation, thus when the piles were concreted the strain meters were subject to a force equivalent to the hydrostatic pressure of the full head of concrete. Except for one strain meter, #5, which was installed ten feet from the top of Pile 17A, all responded satisfactorily, indicating that the pressure of the fluid concrete produced a continuous column of dense concrete.

The testing system consisted of four strain meters for pile load measurement and one meter at the top of the pile for calibration. The load at the top of the pile was measured with a 350 ton load cell manufactured by the Brewer Engineering Laboratories, which has an accuracy of .002%. The upper strain meter, because it is in the portion

of the pile where the soil has been removed by the pipe cylinder or by the mud slurry, does not show any dissipation of the applied axial load. When cycling, the upper strain meter was calibrated with the load cell and a calibration curve for the upper meter was derived. This was then utilized to determine loads at the other meters.

The instrumentation used in the test pile program demonstrated advantages and disadvantages for their use in future pile programs.

The strain rods are more durable than the Carlson strain meters but because of improper installation, insufficient results were obtained. The Rods, if they are placed properly, are more resistive to shock than the meters and should be more reliable. They are limited in their use since they could only be utilized in a homogenous pile such as a pile where the strain rods would not be affected by confining stresses.

The meters were perhaps the more sophisticated of the two but because of their sensitivity to shock, their value was slightly diminished. The main advantage of the meters was their use in a composite pile of steel and concrete. They could be set within the pile showing only a minor affect from the hydrostatic pressure of the concrete. The problems arising from the use of the composite piles probably discounted some of its distinct advantages and flexibility.

Meter #2 in pile 17A at a depth of approximately 52' was improperly pre-set prior to installation and submitted doubtful data.

During the cycling program, Meter #1 for test pile 9A, at a depth of 57', began to act erratically, as will be explained in the Test Results.

DRIVING RECORDS

A contract for the driving of 34 piles of various types, including 10 timber piles, 9 steel pipe piles, 3 10-3/4" O.D. with 1/4" wall, 2 10-3/4" O.D. with 3/8" wall, 2 12-3/4" O.D. with 1/4" wall, 2 12-3/4" O.D. with 3/8" wall, 4 steel H piles (10 BP 57) 4 fourteen inch octagonal prestressed concrete piles and 7 twelve inch "Cobi" piles was awarded to the Cayuga Construction Corp. Six Piles of which three were instrumented at several locations along its length, were load tested. In addition, there was a provision for seven pipe cylinders, 18 feet in length, to act as casings through the sand fill and tidal marsh deposit in order to remove the frictional support of these strata.⁹ Due to installation problems with the 14" octagonal prestressed concrete and the "Cobi" pile it was not feasible to provide a pipe cylinder for every test load.

The piles were driven with a Lima 803 crane equipped with 100 foot leads to permit driving to the underlying shale without having to splice in the leads. The plan view drawing of the piles and their locations in each area is shown on figures 3 and 4.

The pile driving results are summarized on figures 3 and 4. Drawings of the pile record reports for each individual test pile are shown on figures 1 and 2. The records show easy driving to the shale with only a slight build-up of driving resistance through the dense sands

and silts below the tidal marsh deposit. All the other piles yielded similar results and only differentiated at the bearing strata.

All of the steel piles, which were driven to a final driving resistance of 20 blows per inch with a Vulcan "O" hammer developing 24,375 foot pounds per blow, penetrated into the bedrock. There was no apparent difference in pile penetration in Area "A" where the shale was of a sounder quality, as discussed in the soil conditions, but in Area "B", the more weathered bedrock section, there was a marked delineation. The H piles penetrated to approximately 235.0'. The pipe piles with a wall thickness 3/8 inch penetrated 1.5 to 2.5 feet lower than the 1/4 inch thick pipe. There was no indication at Area "A" that the thickness of the pipe had any effect on the penetration of the shale. In Area "B" the 3/8" walled pipes, being stiffer, did not lose as much energy in driving as the 1/4" walled pipe, thus they penetrated the shale further. The dissipation of driving energy through friction because of the greater surface area was the probable cause of the 12-3/4 inch diameter pipe piles not driving as far as the 10-3/4" diameter pipe. Also, a consideration that the tops of the 12-3/4 inch diameter pipe piles (6B & 8A) were rolled during driving because the driving head did not fit the piles properly.

Seven "Cobi" cast-in-place piles were driven, four in area "A" and three in area "B". These piles were 12 inch outside diameter, 16 gauge, hel-cor steel shell driven with an expanding type mandrel. These piles were driven with a Vulcan 06 hammer producing 19,000 foot

pounds per blow. A number of construction problems related to the utilization of these piles was encountered.

In driving "Cobi" piles, 13B and 14B, the mandrel became stuck within the shell and had to be withdrawn by utilizing the pile hammer as a pile extractor. Both these piles were driven to a final driving resistance of 15 blows per inch, while all the other "Cobi" piles were driven to a final resistance of ten blows per inch with the mandrel being extracted easily. The problem of overdriving these piles would only increase the construction problems and not improve the capacity of the pile.

All the piles, except for pile 15B, were able to penetrate through the decomposed rock to the shale. Test pile 15B had a tip elevation 3.7' above the tip elevations of the two other "Cobi" piles in its grouping. Because of the proximity of the two deeper piles the variation in the depth of bedrock was discounted. An attempt was made to test load this pile, however, in driving, the pipe cylinder collapsed the pipe shell, and the test loading of this pile could not be accomplished.

Since the test piles in Area "B" were driven first, it was decided to devise a different method for test loading a "Cobi" pile. For test pile 16A the pipe cylinder was driven and cleaned out prior to driving the test pile. However, in driving 16A, its shell caught on the tip immediate area to pour into the pile.

Test pile 17A was driven and concreted for load testing and instrumentation. An annular cylinder of mud slurry was constructed around the test pile, through the upper fill and marsh deposit as a field expedient substitute for the pipe cylinder. A "mud rig" used for test borings was utilized for the work.

Four prestressed concrete piles were driven, two in Area "A" and two in Area "B". The piles were octagonal in cross-section, measuring 14 inches between opposite sides.

The piles were to be driven with the Vulcan No. 0 Hammer to resistances of 20 blows per inch and 14 blows per 1/2 inch. In Area "B", driving difficulties occurred. Because the driving head used for the piles was too large for the pile, it moved off center during the driving. It slipped off the cushion, striking the head of test pile 9B and eventually cracking the upper three feet. However, with necessary interruptions to re-position the driving head, test pile 9B was driven without any other damage to elevation 241.6 with a resistance of 15 blows per inch. Test pile 12B, with a 9" cushion compared to the 4½", cushion for pile 9B, was driven to a tip elevation of 237.8' with a final resistance of 15 blows per inch.

The two prestressed concrete piles in Area "A" resembled similar driving qualities to the 10" inch steel pipe piles and the "Cobi" piles. These piles were driven to the designed driving resistance of 20 blows per inch and 14 blows per 1/2 inch.

The 10 timber piles, 5 in each area, were driven with a Vulcan No. 1 hammer, developing 15,000 foot lbs. per blow. Each pile was spudded through the overlying sand fill to overcome the initial resistance to driving. In Area "B", test piles 1B, 3B and 4B were driven into the decomposed shale to resistances varying from 3.7 to 4.8 blows/inch.

Pile 2B broke at elevation 259, after attaining a resistance of 36 blows per foot, while pile 16B could not be driven any lower than elevation 277, possibly because of losses in driving energy resulting from a blow on the pile which caused it to whip in the leads. These piles in penetrating the very fine sandy silt stratum between elevations 263 and 285 attained a driving resistance of 2.9 to 3.7 blows/inch.

The driving results for the timber piles at Site A demonstrate the requirement of an empirical approach to the pile design. Test pile 3A was the only pile to penetrate to the shale. Test piles 1A and 2A resisted at 44 blows/inch in the reddish brown silt at tip elevations of 272 and 275. Test pile 4a attained a resistance of 11 blows/3 inches at Elevation 260.8. Test pile 5a attained a resistance of 25 blows/inch at Elevation 269.

The differentiation in the driving results between the two areas is a common occurrence. The engineering properties of each stratum of soil are variable at diverse areas of the field, sometimes at only a few hundred feet.

Experience from previous pile driving contracts gave evidence of inconsistency in driving characteristics and tip elevations at times within 20 to 30 feet.

Inspection of the "Cobi" pile and steel pipe pile for alignment and dryness showed satisfactory results. By dropping a light down the pile it was noted that only pile 16A, as previously mentioned, had a torn shell; all others had no visible damage.

Piles 7B, 7A, 8A and 18A had slight dog legs, but not much more than the diameter of the pile, since the light was still visible.

The steel pipe piles remained dry but the "Cobi's" had a tendency to take on some water. The water level in pile 17A during the 41 day period, subsequent to driving, varied from 2" to 4" to 1/2". This was not considered either abnormal or detrimental.

LOAD TEST PROCEDURE

The basic criterion for performing the load tests was to conform to the requirements of the Building Code of the City of Newark since two of the three Terminal Buildings and most of the primary pile supported structures would lie within the City of Newark's limits.

The Newark Code states that "the allowable load on any pile when determined by the application of an approved driving formula shall not exceed forty (40) tons." Thus to utilize an approved test load "the resulting allowable load shall be not more than one-half (1/2) of that test load which produces a permanent net settlement per ton of test load of not more than one-hundredth (0.01) inch." With no restriction on gross settlement, the allowable net settlement on an 80 ton pile is 1.6 inches. The net settlement does not seem critical. Thus the provision in the code regarding the rate of settlement, whereby the test pile is loaded to 200% of the proposed design load in eight equal increments, was the critical factor. This test load was to be maintained until the settlement rate did not exceed 0.01 inches in eight hours and at 150% of the proposed design load the settlement rate did not exceed 0.01 inches in twenty-four hours. A provision of the New York City Code was also adhered to during the tests whereby 200% of the proposed design load was maintained until the rate of settlement did not exceed 0.01 inches in 24 hours.

The program also checked the code restrictions regarding the maximum allowable loads on different bearing strata. For sedimentary deposits, or hard shale, the maximum allowable load was 80 tons. For piles bearing on "soft, broken shale" the allowable load was 60 tons. Also, if the pile receives lateral support from the soil, the allowable load must not exceed the capacity of the pile designed as a short column.

In addition to conforming to the previously mentioned codes, a method of performing load-tests first introduced by van Weele and suggested by Ireland³ was utilized.

Van Weele suggests that after applying each load increment, the test pile is reset to a "zero" load, then re-cycled again to the test increment two or three more times before applying the next load increment. The purpose of the re-cycling of the pile load after each increment is to relieve the residual frictional forces in order to determine the residual settlement of the pile tip in addition to the total settlement.

As the loading occurs, in increments, the maximum resistance in skin friction is attained, the resistance is then mobilized, and all loads applied thereafter will be transmitted to the tip. If only the displacements of the pile top are measured the elastic compression of the pile, together with that of the sub-soil, can be obtained by means of the recovery of the pile top during unloading. The elastic

compression of both the pile and the material supporting the pile tip will have a linear relationship to the pile load after the skin friction is fully mobilized, thus the same should apply to the elastic displacements of the pile butt.

The apparatus utilized in applying the load tests is illustrated in photos 1 and 2. The loads were applied by jacking against a loading platform which supported the dead weight. For the piles to be test loaded, the load cell, as described in the instrumentation and the swivel plate as suggested by Davisson² were utilized.

Pile 5B which was equipped with the strain rods as shown on figure 5, was load tested in conformance with the Newark City Building Code in increments of 40 Kips each testing day. The initial load was applied in approximately 5 minutes, remained for 30 minutes, and unloaded in 5 minutes. This load was not re-applied until 30 minutes had elapsed. This sequence of operations was repeated two more times. The fourth and final load applied for the day was the test load for the following test day; an additional 40 Kips. This load remained on overnight and the sequence continued until termination of the test.

Measurements were taken at every break in the loading sequence. Level readings, checks on the load cell strains, strain dial readings, and temperature variations were maintained.

Prior to performing the load tests on pile 9A and 17A, which were equipped with the Carlson Strain Meters, as shown on figure 6, readings

were taken to determine the effect the shrinkage of the concrete developed upon the meters. Testing of these piles proceeded when the compressive strain exerted by the concrete on the meters varied linearly with time.

The test load for the day, which had been applied on the previous test day was initially reduced to a "zero" load for calibration of the strain meters. The load was then applied in increments of 1/8 of the test load every 3-5 minutes until the load had been attained. It was allowed to stand for 30 minutes and was re-cycled back to "0" load. Only the upper strain meter was read during this procedure to obtain data for calibration. After a 30 minute duration the test load was applied within 5 minutes and this procedure was repeated once again, reading all meters. The test load for the following day was then applied and allowed to remain overnight. This sequence was repeated each test day.

Measurements were taken at distinct intervals. Level readings, load cell checks, direct and reverse resistivity readings and temperature readings from the Carlson Strain Meters were performed.

Piles 4B, 8B and 11A were not instrumented and the load tests only consisted of the load application as prescribed by the City of Newark Building Code.

LOAD TESTS

Cycling Theory and Method

Van Weele^{II} states that by cycling the load in a heavy loading test the residual settlement can be determined. The settlement of the pile top, which is the total settlement is composed of (a) the elastic compression of the pile, (b) the elastic compression of the sub-soil below the pile tip, (c) and the residual settlement of the sub-soil. Thus, by obtaining the displacement of the pile top during unloading the elastic compression of the pile and sub-soil are determined. The elastic compressions of both these components have a linear relation to the pile load, thus when the skin friction is mobilized the elastic displacement of the pile top and the applied load will have a linear relationship. Thus, it is possible to prove that the skin friction remains constant beyond a certain pile load, that is, after a certain settlement has occurred. At the point the skin friction is mobilized any increase in the pile load will be transmitted directly to the point without any further increase or decrease in load carried in skin friction.

To test van Weele's premise, incremented loads were cycled to obtain the combined elastic recovery of the pile and the sub-grade; from this linear relationship the constant frictional force is derived.

Using test pile 5B for an example, in measuring the replacement of the pile top during cycling for test pile 5B, the data as shown on figure 7 is obtained. This graph represents the incremental loading of 280 kips for December 19, 1966. Level readings were taken at the start and completion of each phase of operation in loading and unloading.

(A) is the uncorrected total settlement, which was taken prior to cycling in the morning after the load had been held since at least the previous afternoon.

The uncorrected net settlement for each cycled load is the average of (C) and (D). The uncorrected elastic deformation of the pile, the sub-soil under the pile toe, and the residual settlement of the pile toe is the difference between the total and net settlements.

To obtain the corrected total settlement, the difference of (B) minus (A) is taken and added to (B). This is the increment so each incremented load has the same settlement for each final load as illustrated by the sample calculations in Table I.

TABLE I

TEST PILE 8B - CYCLING DATA

	<u>50</u>	<u>100</u>	<u>150</u>	<u>200</u>
(A) Total Settlement - ft	- 0.0012'	0.0048'	0.0089	0.0135
50 Ton	+ 0.0020'	+0.0020'	0.0020	0.0020
100 "		-0.0012'	-0.0012	0.0012
150 "			-0.0006	-0.0006
200 "				+0.0016
(B) - (B-A) Corrected Set.	0.0008'	0.0056	0.0091	0.0153
Elastic Deformation	-0.0024'	-0.0050	-0.0085	-0.0130
Point Settlement	-0.0016'	0.0006'	0.0006	0.0023

The corrected elastic deformation is derived from

$$\frac{(A) + (B)}{2} - \frac{(D) + (E)}{2}$$

which is the difference between the average of the total settlement prior to first and final unloading cycles and of the incremented load and the average at the first and final loading cycles. The corrected point or net settlement is then the difference between the corrected total settlement and elastic deformation.

Effects of Cycling

Three piles 5B (10-3/4" - Ø.D. pipe pile), 9A (10-3/4" - O.D. pipe pile) and 17A (12" Cobi pile) were instrumented to check van Weele's thesis in separating the bearing capacity of a pile into skin friction and point resistance.

Pile 5B was equipped with strain rods. The rods were anchored at several locations inside the hollow pipe pile to measure the elastic shortening of different segments. From the dial readings, the strains for the segments could be determined and using 29×10^6 psi Modulus of Elasticity of the pile, the stress could be determined. Thus, for each segment of the pile, the load transferred to the skin friction could be calculated.

Three plates and five strain rods were installed in the pile as shown on figure 5.

With this location system the readings could be readily ascertained for accuracy in determining the load at set positions of the pile. Expectations from this pile though were not forthcoming. Rods #2 and #3

did not move since they probably became jammed and obstructed. Also, the results from Rod #5 were not applicable since the strain readings approximated the sum of Rods #1 and #4, probably because the rod slipped and was caught immediately below plate #2.

By cycling each increment of test load, the residual stresses being exerted on the pile tend to be relieved. Figure 8 shows the effect between the initial reading prior to cycling and the decreased readings during cycling. Both figures 8 and 9 shows that the relief of the residual load increases gradually with the increased applied load. As the applied load is increased, the elastic deformation increases causing a greater relative movement between the pile surface and the adjoining soil. The continual movement from the cycling tends to release the effective pressure exerted on the pile by the soil thereby relieving any negative frictional or residual forces. 10 Kips of residual load was measured to have been relieved at the 280 Kip loading. If the strain rods could have carried an applied load greater than 280 Kips, there would probably have been a greater relieved residual load. This result, which was due to the increased movement of the pile face where the bond of the pile and soil yielded, also caused a decrease in the mobilization of skin friction.

The graph on figure 9 illustrates the load carried by skin friction above elevation 268.5. It shows a loss of skin friction from the initial unloading cycle for the 80 Kip, 120 Kip and 160 Kip applied loads, because when the peak skin friction is mobilized, soil creep

occurs, causing a loss in the skin friction. Continual load cycling appears to stabilize the load carried by skin friction. Using only the final readings after load cycling, the load carried by skin friction attained a maximum of 27 Kips for the 80 Kip applied load and then decreased gradually with increasing load until at the 280 Kip load it was 17 Kips. This curve, which illustrates the approximate shear strength of the soil, at the 80 Kip load, showed a similar trend to results that will be discussed later. These results show that there is a decrease in skin friction for soil strains beyond the frictional resistance.

Piles 9A (10-3/4" - O.D. pipe pile with a 0.25" wall) and 17A (12" ϕ Cobi) both concrete filled, were each equipped with five Carlson strain meters. The meters were placed as previously mentioned, figure 6, with the #1 meter at the tip of the pile and increasing in equidistant segments to meter #5 at the base of the casing, which was provided to void any friction loads in the compressible material.

Pile loads at each strain meter were determined by calibrating the upper meter with a known load at the outset of the day's cycling.

However, meter No. 5, the upper strain meter, for test pile 17A recorded strains that were too large, probably because the concrete at the meter was not properly densified. This meter was only 10 feet from the pile top so that the hydrostatic pressure was small and the upper portion of the concrete fill was not rodded as should have been required. After the readings from 17A were deemed doubtful, it was

necessary to use the modulus of elasticity of the concrete from the measurement for 9A to calculate the pile loads. In effect, meter #4 was used for calibration. This method, though not desirable, yielded consistent results.

The calibration for the load strain relationship of pile 9A, (figure 10), was constant until January 16 when the 320 Kip load was applied and cycled. The elasticity of the pile or testing apparatus increased approximately 10% and an additional 10% on January 19, 1967 when the applied load was increased to 360 Kips. All meters were assumed to be effected and further development of the test data was similarly affected.

The Analytical method of determining the modulus of elasticity of a steel pipe with a concrete core is to assume that the percentage of the total applied load carried by the member is proportional to the cross-sectional area and the modulus of elasticity of each element. Applying a modulus of elasticity of concrete of 5.0×10^6 psi, as averaged from breaks on test cylinders which ranged from 4.5×10^6 psi to 5.4×10^6 psi, and 29×10^6 psi for steel, the theoretical modulus of elasticity of the pile should have been approximately 7.0×10^6 psi. From the load strain curve of figure 10 the modulus of elasticity of the pile was actually 26×10^6 psi, thereby showing that the steel pipe was carrying a greater share of the applied load.

This disparity is due to the exhibited physical properties that concrete cannot exceed its tensile strength. As concrete cures, the

volume decreases, thereby shrinking the concrete core. This shrinkage induces a force along the inner wall of the steel pipe which does not move in relation to the concrete. The steel pipe takes this force in compression and because of an equal and opposite reaction, the concrete core takes the stress in tension. Cooling causes a thermal contraction in the core which is only minor but is increased because the pile is held in friction and the steel pipe cannot contract because of the adherence of the concrete on the inside and the soil pressure on the outside. Concrete, under sustained loading, will creep, exhibiting an effect similar to the shrinkage of the concrete.

Because of these physical properties the stresses in the steel pipe could have increased two to four times of that obtained under initial loading, if at that time, the stresses in the pipe were determined.⁶

Under initial loading of the pile, when most of the pile load is absorbed in friction, most, if not all of the load, is carried by the steel pipe with a negligible amount, if any, carried by the concrete core. When the pile is cycled from load to unload, the tendency is for the steel pipe to expand to its original length, but both the concrete core which does not expand or contract as uniformly as the pipe and the negative skin friction now exerted on the outside of the pipe prevent a full elastic recovery.

Because of this extreme cycling effect and the pressure exerted by the steel pipe the concrete core could not withstand the tension load and on January 16, with an applied load of 320 Kips, the concrete core ruptured. This is then the probable cause of the calibration shift.

Figure 11 shows the inelastic strain at a "0" load condition vs. time for pile 9A. The temperature strains for meters #2 and #4 were based on the cooling temperatures taken from the readings on the strain meters.

The total strains due to creep and shrinkage prior to loading reached a linear stage 9 days after the concrete was poured and continued until the initial loading. During cycling it can be noted that the meters showed that the pile was restricted from rebounding to its original "0" setting prior to loading. This was due to the twofold effect of the negative skin friction and the restrictive pressure of the concrete core.

With the application of the 376 Kip loading the inelastic strain curve started to show a relief in the compressive forces in the pile. Similarly, as with pile 5B, the greater the load cycled, the greater the relief of any residual forces in the pile. Meters #2 and #5 show relief of the forces constricting the pile from rebounding after unloading. Meters #3 and #4 show not only relief of the aforementioned forces but also show relief of the forces due to concrete shrinkage and creep. Possibly with time these forces would have been relieved.

With greater loads relieving the residual forces, these meters would possibly react similarly to meter #1, whose concrete encasement of the meter could not withstand the tension from the complete recovery of all the residual forces in the pile. These stresses were established in the pile due to driving. The pile is contracted at each application of the driving hammer, and when this impact load is withdrawn, the pile tends to expand back to its original length, but the negative skin friction exerted by the soil constricts the complete elastic recovery, thereby inducing a compressive strain on the pile. Thus, when this compressive force is relieved the tension load is transmitted to the concrete core, which, of course, was poured after driving, thereby straining the concrete to its yield point.

The inelastic strain at a "0" loading for test pile 17A, figure 12, seemed as if it would follow a pattern similar to 9A except the residual forces were not as readily relieved with increased loading.

Results and Analysis

Two graphs, one of the butt load versus the gross and net settlements and the other of butt load versus the elastic recovery of the pile top were derived for each of the six piles test loaded using van Weele's analysis. These results are shown on figures 13 to 24.

Using the load-settlement graphs, failure conditions can be ascertained. Test pile 4B (Timber) failed at approximately 40 Kips (figure 13). The other piles to reach failure were 5B (10-3/4" O.D. pipe pile with a 0.365" wall) at 360 Kips (figure 14) and 8B (12-3/4"

O.D. pipe pile with a 0.25" wall) at 400 Kips (figure 15). None of the piles tested in Area "A" failed though pile 9A (10-3/4" O.D. pipe pile with a 0.25" wall) appeared close to failure at 400 Kips (figure 16). Pile 17A (14" prestressed concrete) was tested to 600 Kips without any indication of failure (figure 17). Pile 17A (12" ϕ Cobi shell) had exceeded a capacity of 320 Kips when the test data for net settlement became doubtful (figure 18).

In Area "A" the 400 Kip loading showed net settlements of 0.042' for 9A, 0.033 for 17A and only 0.010 for 11A. The lesser proportion of settlement for the prestressed concrete pile could be attributed to development of added friction resistance from the compressible deposit and overlying sand surcharge and to the added frictional factor of concrete over steel. In addition, the load at the tip was distributed over a greater surface area.

Figures 19 to 24 exhibit the elastic recovery of the test piles 4B, (figure 19), 5B, (figure 20), 8B, (figure 21), 9A, (figure 22), 11A (figure 23), and 17A (figure 24). The mobilized skin friction, obtained with van Weele's method is listed and compared in Table II.

In averaging the axial load of the pile, utilizing the elasticity of steel as 29×10^6 psi and of the concrete as 5.0×10^6 psi, as was previously discussed, the theoretical elasticity of the pile was derived. Comparing these results to the actual pile and sub-grade recovery a similarity in results was noted. Actually, these results are misleading, because if the load at the pile tip were actually that

excessive, as say 266^k for pile 17A, and using a sub-grade modulus of 13^k/in³ (figure 29), the sub-grade would develop an elastic recovery of .015' which signifies that the mobilized skin frictions for all the test loaded piles was not attained by this method.

TABLE II

TEST PILES - CHECK ON VAN WEELE METHOD

Pile	Mobilized Friction	Arbitrary Butt Load	Theo. Pile Elasticity	Actual Pile and Sub-Grade Recovery
4B - Timber	23 ^k	120 ^k	.039'	.024'
5B - Steel pipe	38 ^k	200 ^k	.033'	.033'
8B - Steel pipe	83 ^k	250 ^k	.018'	.017'
9A - Steel pipe	43 ^k	300 ^k	.030'	.028'
11A - Prestress. Conc.	73 ^k	300 ^k	.019'	.019'
17A - Cobi shell	34 ^k	300 ^k	.030'	.032'

The load distribution curves for test piles 9A and 17A are illustrated on figures 25 and 26. The load transferred to skin friction at each meter location increases until the maximum possible skin friction is attained. The mobilization along the pile continues in a downward direction until the pile is completely mobilized and all of the additional applied load is transmitted to the tip. The illustrations do not show a definite trail because as the friction forces at the lower end of the pile are being mobilized the upper portions which have already been mobilized, tend to lose any support to maintain the mobilized frictional strength, because their excessive relative movement

with the adjoining soil tends to cause a shear failure and disrupt the soil bond. Thus, a load greater than the applied additional load is transmitted to the pile tip.

Data from these results with the results obtained using van Weele's analysis from figures 22 and 24 for separating the load carried by point bearing and skin friction, did not agree. Van Weele's method showed that the mobilized friction loads for 9A and 17A should have been 43 Kip and 34 Kips respectively. With only an 80 Kip applied loading the piles each have a load carried by skin friction of 60 Kips and 78 Kips. At a 300 Kip loading the skin friction loads increase to 130 Kips and 170 Kips. This difference is attributable to the differing soil conditions at the test sites. Van Weele's tests were conducted in a soft soil whereas the soil at Newark Airport is medium stiff and dense with differing stress strain characteristics.

That portion of the load distribution curves showing the tip load increasing at the same rate as the applied load compares favorably with results obtained by Mohan, Jain and Kumar⁵, whose soil structure was similar to the Airport. Mansur and Kaufmann⁴, and Seed and Reese⁹, did not compare favorably with the results of the subject tests for the load applied to the tip. Their soil conditions were similar to van Weele's but their results did not agree with his since their load, carried by friction, kept increasing with each increasing applied load. The results of these others are illustrated on figure 27.

Figure 28 shows graphs of the Tip Load vs Tip Settlement for 9A and 17A. The tip load for each applied loading was taken from the strain meter reading at the bottom meter or from the projected reading from the load distribution curve. The tip settlement is the total settlement minus the elastic recovery of the pile. Figure 29 shows graphs of the elastic recovery of the pile tip vs the tip load for 9A and 17A. The elastic recovery of the sub-grade is determined from the elastic recovery at the pile top minus the elastic recovery of the pile.

The results show that for a tip settlement of .02', Pile 9A, the lesser cross sectional bearing area, had a load of 164 Kips and 17A a load of 128 Kips.

Both sub-grade moduli of 9A were approximately twice as large as 17A. These variations could be attributed to the differing quality of shale at the pile tip. Pile 9A, the stiffer pile,^{7,10} was driven 1.6 further into the bedrock than 17A where it possibly rested on a sounder quality stone, thereby it required a greater load to attain a tip settlement equal to 17A. The variations in the sub-grade moduli based on settlement and plastic recovery could be accounted for with the same premise.

Actually, the basis for these curves is questionable since there is no certainty that the pile was able to recover elastically as the loads were being cycled.

Figures 30, 31, 32 and 33 show the Load Transfer vs Pile Movement for both net settlement and elastic movement at the meter segments for piles 9A and 17A.

The load transfer is the difference of load between two meters. The net settlement, again, is the difference between the total settlement and the elastic recovery. The elastic movement is the difference at the "zero" loading prior to cycling the incremental load and immediately after.

The movement for each segment of the pile is that movement that the centroid of the segment travelled relative to a fixed position. Both sets of curves resemble each other except that elastic movement curves at the bottom of the pile do not show a peaking effect.

The relation between the peaks from the Load Transfer vs Pile Movement curves for 9A and 17A, especially for net settlement, figures 30 and 32, is defined. For the lower two segments, the peak strength for 17A is approximately 40% higher than 9A. This is attributable to the effect of friction on the pile surface. Because of the corrugations on pile 17A the interface of the frictional force is almost soil against soil. Pile 9A is a smooth steel pipe where the interface is soil against steel, thus the soil will yield more readily against this smooth surface.

These curves resemble, in structure alone, the laboratory findings of Coyle and Reese^I, figure 34.

For the lower depths, where the lateral pressure on the pile is greater, the load required to yield the soil from the interface of the pile is much greater. The effect is that at the upper section of the pile where the lateral pressures are less, less movement is required to shear the soil. The soil reaches a peak, similarly to remoulded soils, mobilizes and then decreases.

These segments of the pile lower down did not exhibit the decreasing stresses but did show a peaking where mobilization of the skin friction does occur. The non-uniformity of the soil structure is illustrated by the overlapping curves of the slightly organic material.

Field tests have been connected by others and studied by Coyle and Reese^I which tend to peak, mobilize and not decrease the load transferred to friction, but these tests were run in a clay softer and looser than at Newark Airport.

EVALUATIONS

The pile test results validated the opinion that the Building Code for Newark, as it pertains to the Airport, is too restrictive. The code pile capacities of 80 tons for the shale and 60 tons for the broken shale could be safely increased to 100 tons. Failure in test areas "A" and "B" did not occur until the loading capacity was almost 50% over the code's limiting capacities. In addition, the code does not assign any size or type of pile for specific load carrying capacities.

The timber piles showed an inconsistency in driving and refusal depth. This was expected since the previous contracts in which timber piles were utilized were delayed and costly because some piles could not be driven to their minimum tip elevation. The sturdier piles drove with less consistent results at Site "B" than at Site "A" because of the broken, weathered shale. The steel pipe piles drove more uniformly than the others and reached the bedrock at consistent elevations.

Van Weele's theory for separating the bearing capacity of a pile load into point resistance and skin friction varied with the results from this test. The test pile van Weele studied was driven 40' in a soft, loose peat and sandy clays. The shearing strengths do not exhibit a peak as did the shear strengths obtained from this test, which was conducted in medium stiff and medium dense soils. Van Weele's

frictional load also resembled his shear strength curves, in that the load carried by skin friction increased until maximum skin friction was reached and then remained constant, whereas, this test showed a decrease in strength after peaking.

The relief of residual stresses due to the load cycling were more pronounced after maximum skin friction had been attained.

The instrumentation was relatively successful even though a good percentage of the meters did not function as anticipated. Both the strain rods and strain meters emitted comparable results. The strain rods are more expensive to install and more durable, though not proven by this test program. The strain meters, though economical and more sophisticated, had a distinct disadvantage in their installation. The physical properties of the concrete affected their durability and subsequently made some inoperable.

The effect of energy loss on the steel pipe piles with variable wall thicknesses was inconclusive since a number of the pile tips were pointed. However, it was noted that at Site B, when the stiffer, thicker walled pipes were tipped, the penetration of bedrock was almost 2' further than the flat bottomed thinner walled piles. At Site A, where the thinner walled pipe piles were tipped, the penetration was approximately the same.

In evaluating the economics of the piles tested, it was determined that the timber piles could only be justified for loads to 25 tons. Their use would be limited since the driving problems encountered would not be alleviated.

The prestressed concrete piles were over designed and could not be utilized unless the load requirements of the City Code were relaxed. Their cost, which is approximately triple the steel H or the steel pipe piles, is an economic disadvantage.

The use of steel H-piles was discounted because it was doubted that the pile tips could be firmly founded in the bedrock.

The concrete filled pipe piles were considered the most suitable of all the piles tested. They surpassed the requirements of the Building Code and were able to withstand loads in excess of 80 tons. The "Cobi" piles were less expensive but were not considered as economical as the pipe piles. The allowable load on the "Cobi" pile is limited to the allowable stress of the concrete, whereas, in the pipe pile, the casing absorbs a greater share of the applied load, thus decreasing the stresses on the concrete core. Also, the "Cobi" piles, which met the design loads, required greater construction inspection due to their vulnerability to damage during driving.

Phase II of the Test Pile Program at Newark Airport, which is pending, will be performed at different sites to validate design for structures in other areas. Only steel pipe piles will be utilized. Additional results will be studied to determine the phenomenon of the load bearing capacity of piles.

REFERENCES

1. Coyle, H.M. and Reese, L.C., "Load Transfer for Axially Loaded Piles in Clay", Proceedings, ASCE, Vol. 92, S.M. 2, 1966.
2. Davisson, M.T., "Summary of Knowledge Gained from Tests on Instrumented Driven Piles", Seminar, SM&FE, United Eng. Center, N.Y., May, 1966.
3. Ireland, H.O., "The Conduct and Interpretation of Pile Loading Tests", Seminar, Met. Section, SM&FE, United Eng. Center, N.Y., May, 1966.
4. Mansur C.I. and Kaufmann, R.I., "Pile Tests, Low Sill Structure, Old River, Louisiana", Transactions, ASCE, Vol. 123, 1958.
5. Mohan, Jain, Kumar, "Load-Bearing Capacity of Piles", Geo-technique, Vol. 1, March, 1963.
6. Proceedings, American Concrete Institute, Vol. 27, 1931; Vol. 28, 1932.
7. Raamot, T. "Analysis of Pile Driving by the Wave Equation", Foundation Facts, Raymond International, Vol. III, No. 1, 1967.
8. Raphael, J.M. and Carlson, R.W., "Measurement of Structural Action in Dams", J.J. Gillick & Co., Berkley, California, 1956.
9. Seed, H.B. and Reese, L.E., "The Action of Soft Clay Along Friction Piles", Transactions, ASCE, Vol. 122, 1957.
10. Smith, E.A.L., "Pile Driving Analysis by the Wave Equation", Proceedings, ASCE, Vol. 86, S.M. 4, August, 1960.
11. Van Weele, A.F., "A Method of Separating the Bearing Capacity of a Test Pile into Skin Friction and Point Resistance", Proceedings, 4th International Conference on S.M. and F.E., Vol. 2, London, 1957.

SOIL PROFILE AND DRIVING RESISTANCE OF
TEST PILES AT SITE "A"
(Surface Elevation 307.0)

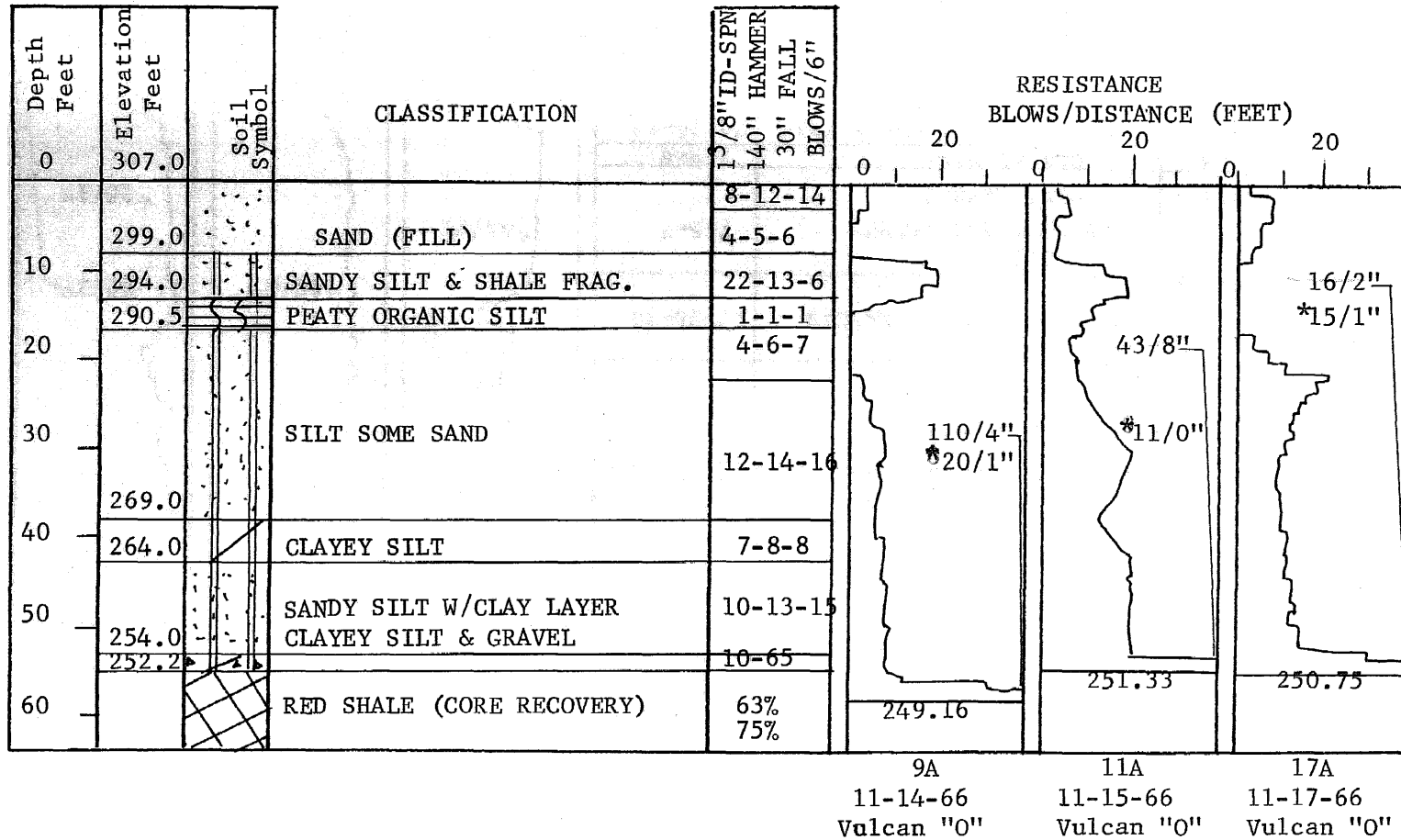


FIGURE I

(refer to fig. 3 for pile type)
*(final driving resistance)

SOIL PROFILE AND DRIVING RECORDS OF
TEST PILES AT SITE "B"
(Surface Elevation 303.2)

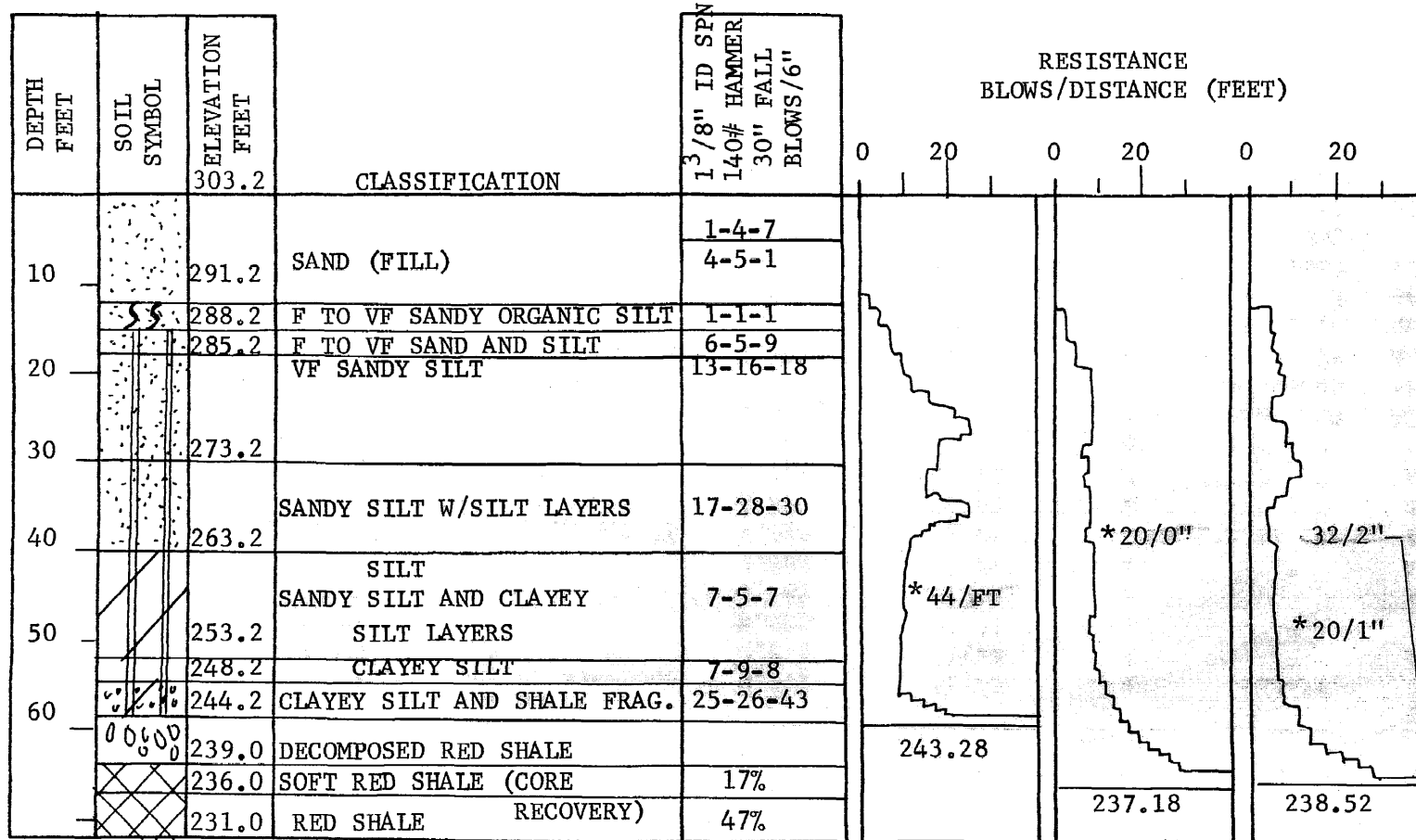


FIGURE 2

4B 5B 8B
10-31-66 10-31-66 10-31-66
Vulcan "1" Vulcan "0" Vulcan "1"
(refer to fig. 4 for pile type)
*(final driving resistance)

SITE "A"
 PLAN AND DATA
 (Surface Elevation 307.0)

PILE NO.	TYPE PILE	TYPE HAMMER VULCAN	DRIVING RESISTANCE BLOWS/DIST.	TIP ELEV.	DESCRIPTION
1A	Timber	1	44/1'	272.0	Southern Yellow Pine 7" Tip
2A	"	1	44/1'	275.0	" " " 6½" "
3A	"	1	11/3"	250.8	" " " 7" "
4A	"	1	11/3"	260.8	" " " 7" "
5A	"	1	25/1'	269.0	" " " 9" "
6A	St. Pipe	0	19/1"	249.4	10 3/4 O.D. x 1/4" wall with point
7A	" "	0	20/1"	250.0	10 3/4 O.D. x 3/8" wall
8A	" "	0	Rolled Top	249.5	12 3/4 O.D. x 1/4" wall with 1" Bot. Plate
9A	" "	0	20/1"	249.2	10 3/4 O.D. x 1/4" wall with 1" Bot. Plate
10A	Concrete	0	20/1"	250.1	14" Octagonal prestressed concrete
11A	Concrete	0	14/½"	251.2	14" Octagonal prestressed concrete
12A	St. Pipe	0	20/1"	251.2	12 3/4 O.D. x 3/8" wall with point
13A	H. Pile	0	14/0"	250.8	10 BP 57 with Pruyne point
14A	H. Pile	0	20/½"	248.9	10 BP 57 with Pruyne point
15A	Cobi	06	10/½"	251.0	12" O.D. Hel Cor Shell, 16 Gauge
16A	Cobi	06	10/1"	250.8	12" O.D. Hel Cor Shell, 16 Gauge
17A	Cobi	06	15/1"	250.8	12" O.D. Hel Cor Shell, 16 Gauge
18A	Cobi	06	10/1"	246.0	12" O.D. Hel Cor Shell, 16 Gauge

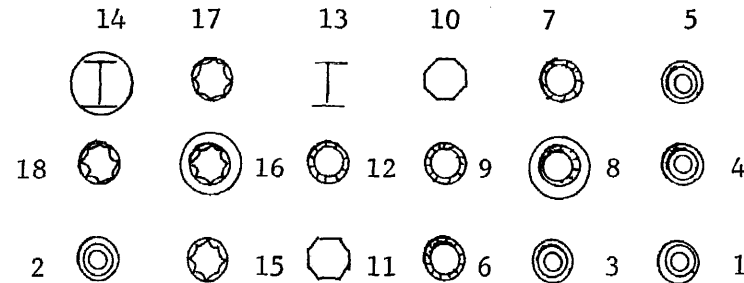


FIGURE 3

SITE "B"
 PLAN AND DATA
 (Surface Elevation 303.2)

PILE NO.	TYPE PILE	TYPE HAMMER VULCAN	DRIVING RESISTANCE BLOWS/DIST.	TIP ELEV.	DESCRIPTION
1B	Timber	1	29/6"	244.2	Southern Yellow Pine, 7½" Tip
2B	"	1	-	Broke	" " " 7" "
3B	"	1	11/3"	243.1	" " " 7½" "
4B	"	1	44/1'	243.3	" " " 8" "
5B	St. Pipe	0	20/0"	237.2	10 3/4 O.D. x 0.365" wall with point
6B	St. Pipe	0	60/3"	236.1	12 3/4 O.D. x 3/8" wall with point
7B	St. Pipe	0	20/0"	238.5	10 3/4 O.D. x 1/4" wall
8B	St. Pipe	0	20/1"	238.6	12 3/4 O.D. x 1/4" wall
9B	Concrete	0	Head Spalled	244.2	14" Octagonal prestressed concrete
10B	H. Pile	0	20/½"	234.1	10 BP 57 Pruyun Point
11B	H. Pile	0	20/1"	235.5	10 BP 57 Pruyun Point
12B	Concrete	0	15/1"	237.8	14" Octagonal prestressed concrete
13B	Cobi	06	15/1"	238.4	12" O.D. Hel Cor Shell, 16 Gauge
14B	Cobi	06	15/1"	238.1	12" O.D. Hel Cor Shell, 16 Gauge
15B	Cobi	06	10/1"	242.1	12" O.D. Hel Cor Shell, 16 Gauge
16B	Timber	0	44/1'	277.0	Southern Yellow Pine, 6" Tip

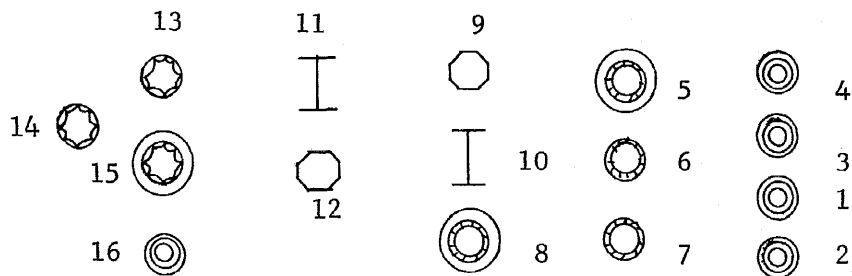
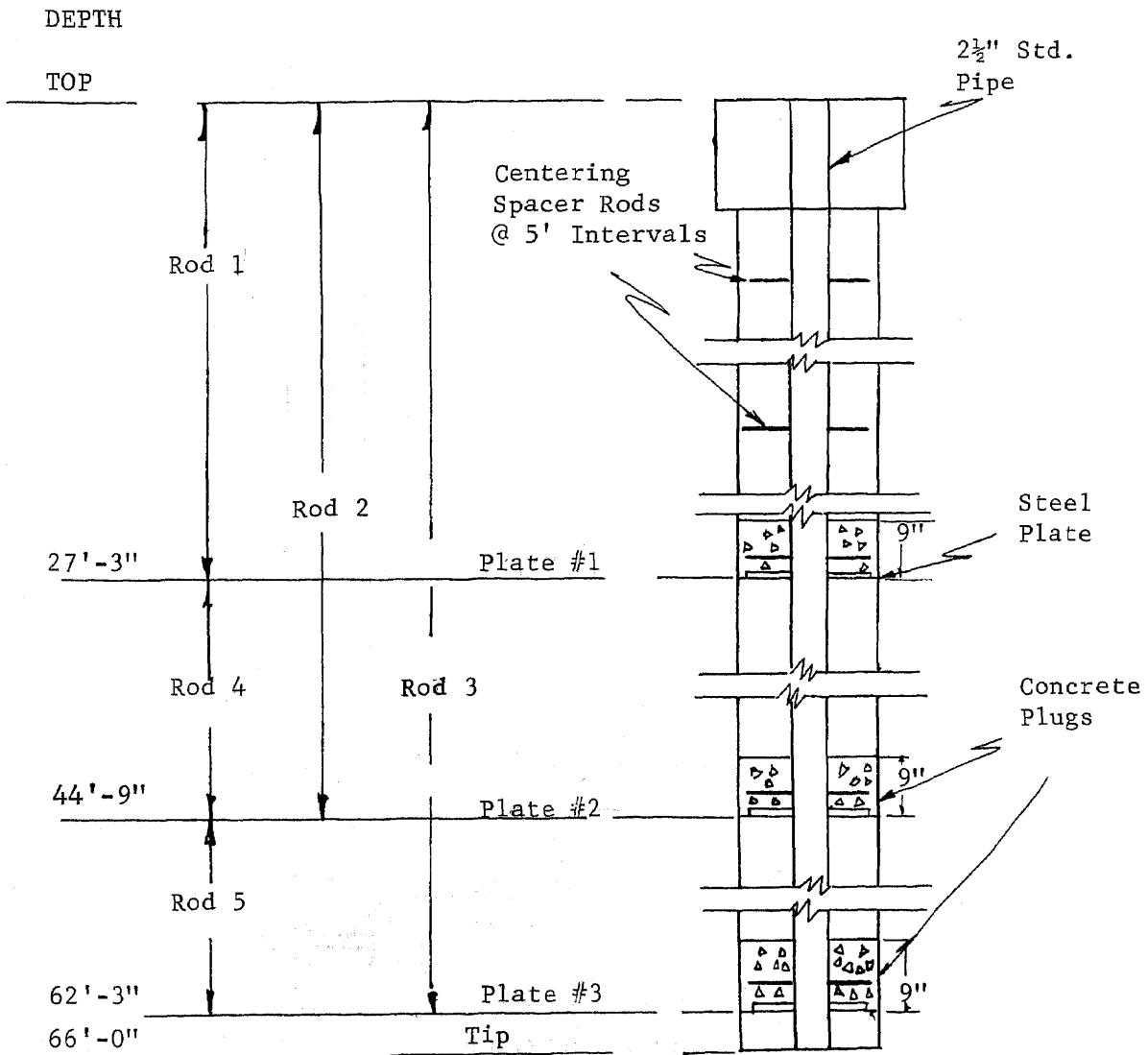
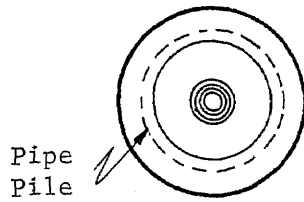


FIGURE 4



PILE 5B
STRAIN ROD INSTALLATION

FIGURE 5

PILES 9A & 17A
STRAIN METER INSTALLATION

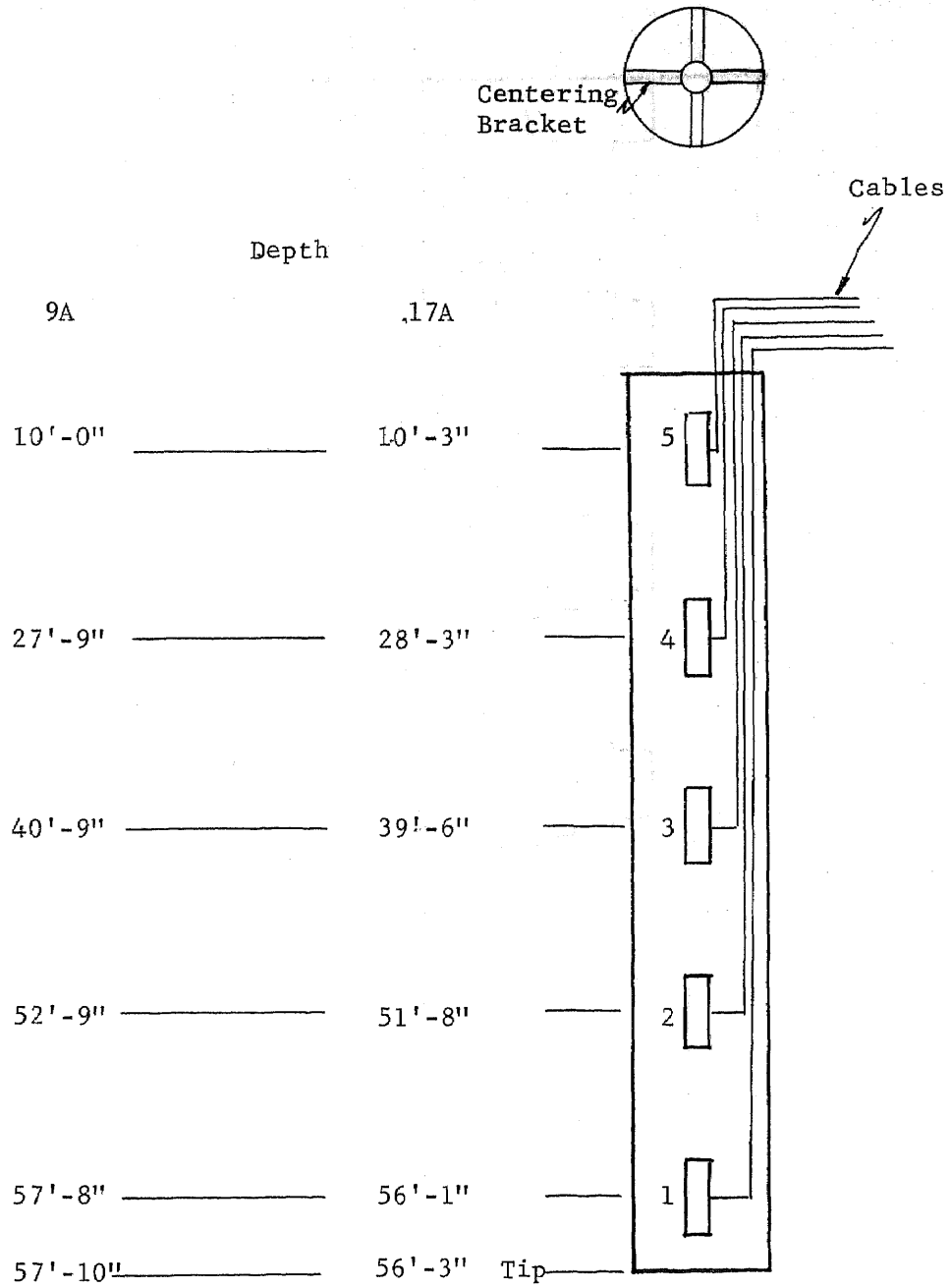


FIGURE 6

PILE 5B
CYCLING GRAPH

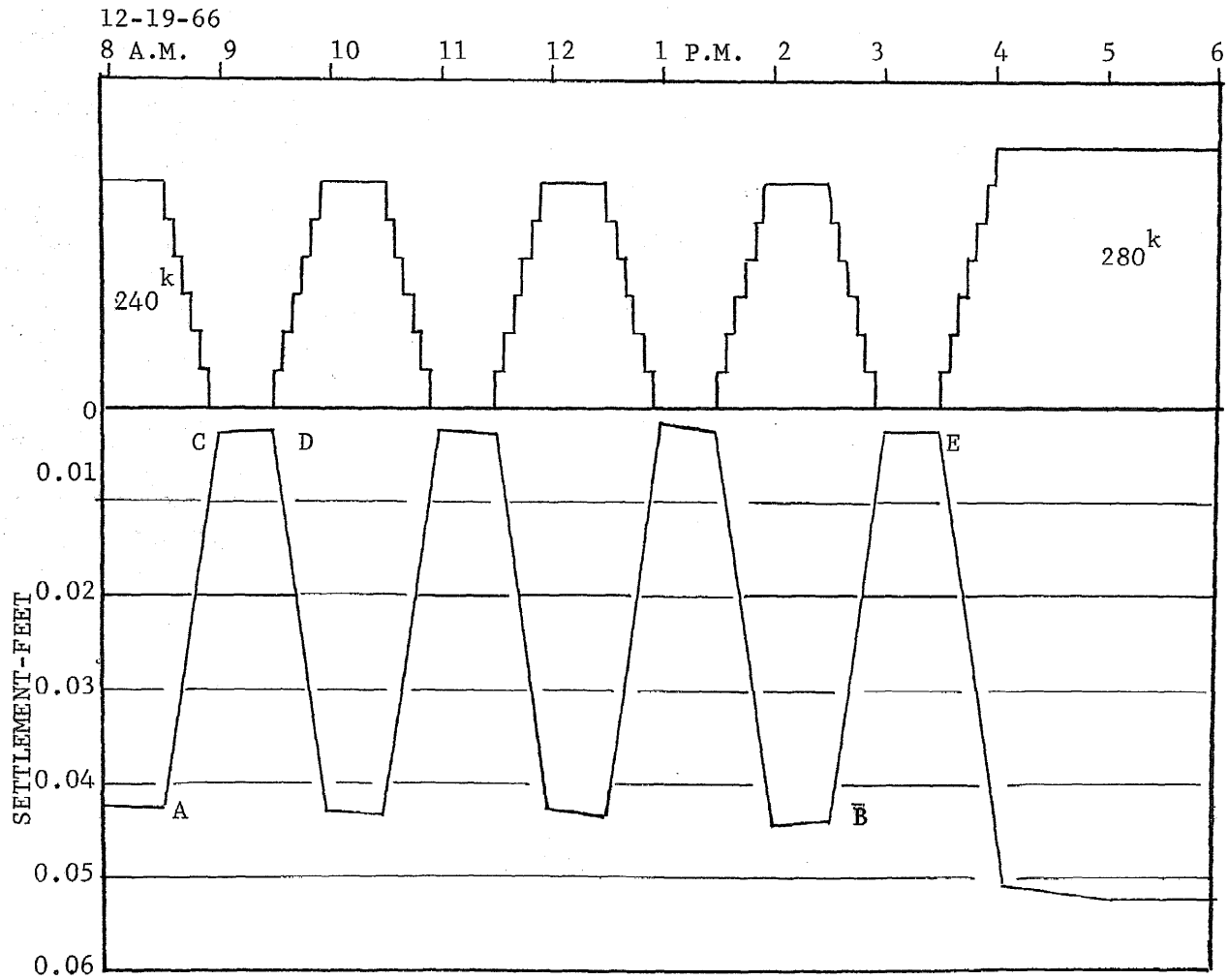


FIGURE 7

PILE 5B
EFFECTS OF CYCLING

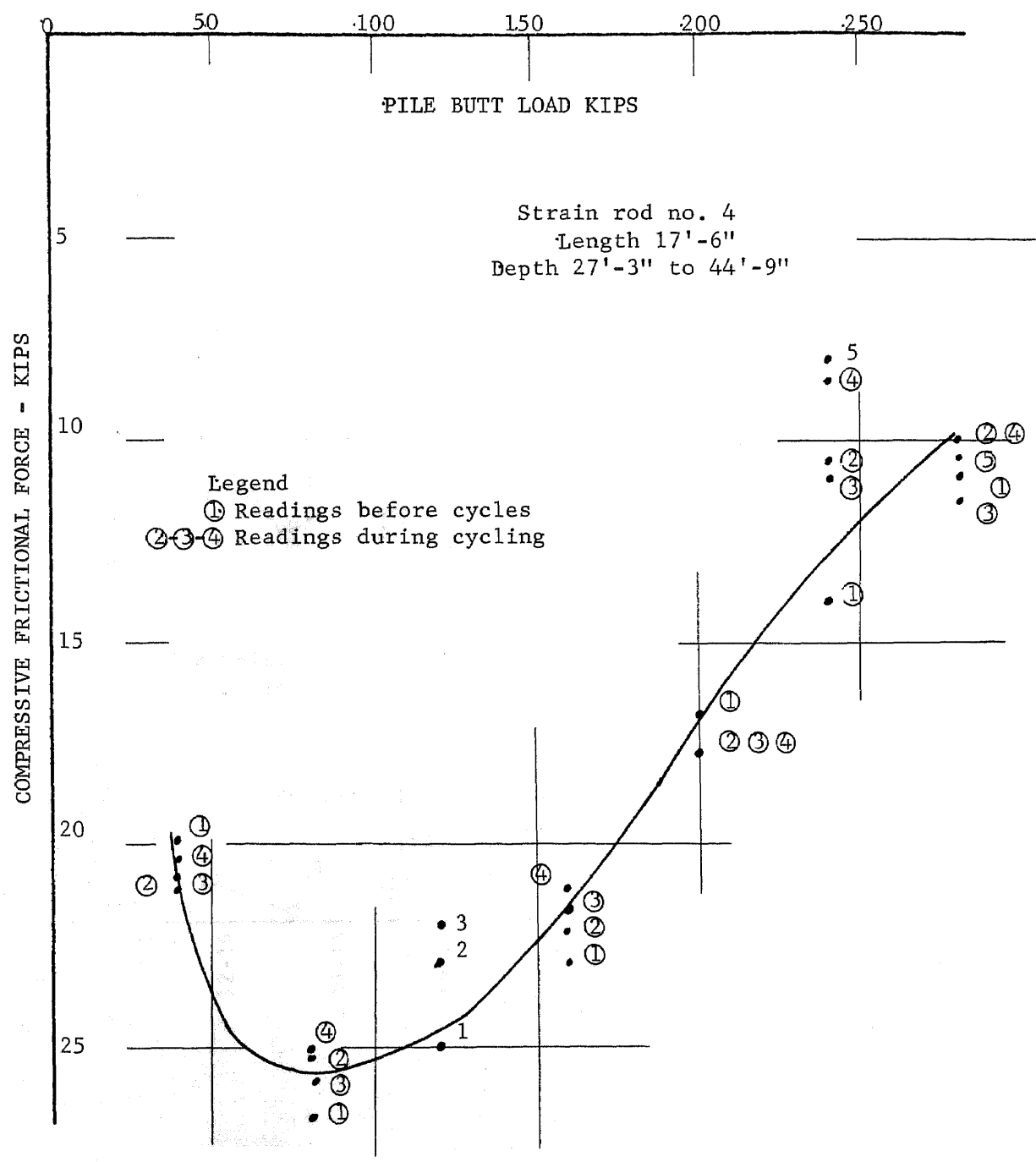


FIGURE 8

PILE 5B
LOAD DISTRIBUTION

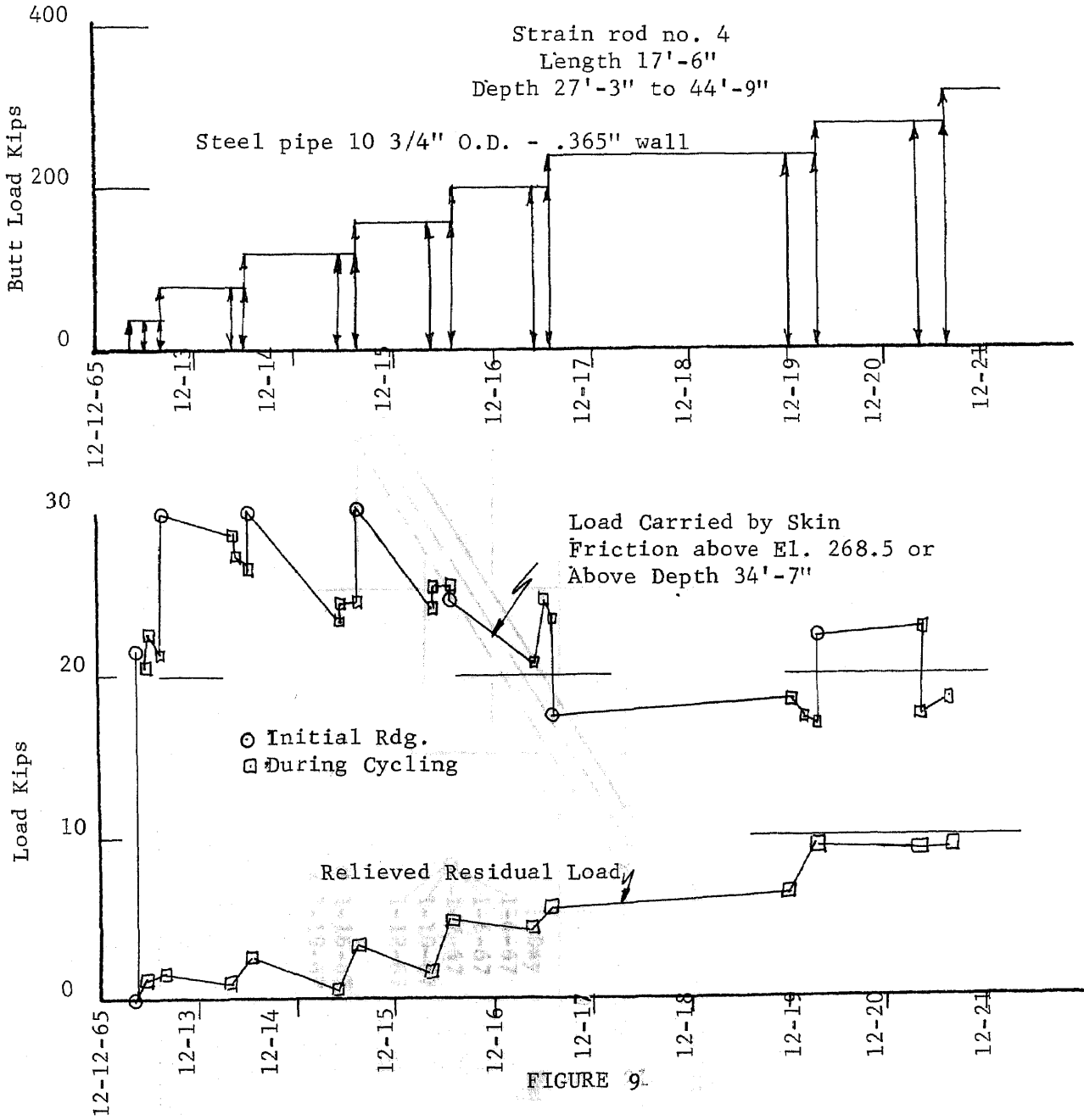


FIGURE 9.

PILE 9A
LOAD - STRAIN

Steel Pipe Pile
10 3/4" O.D. - 1/4" Wall

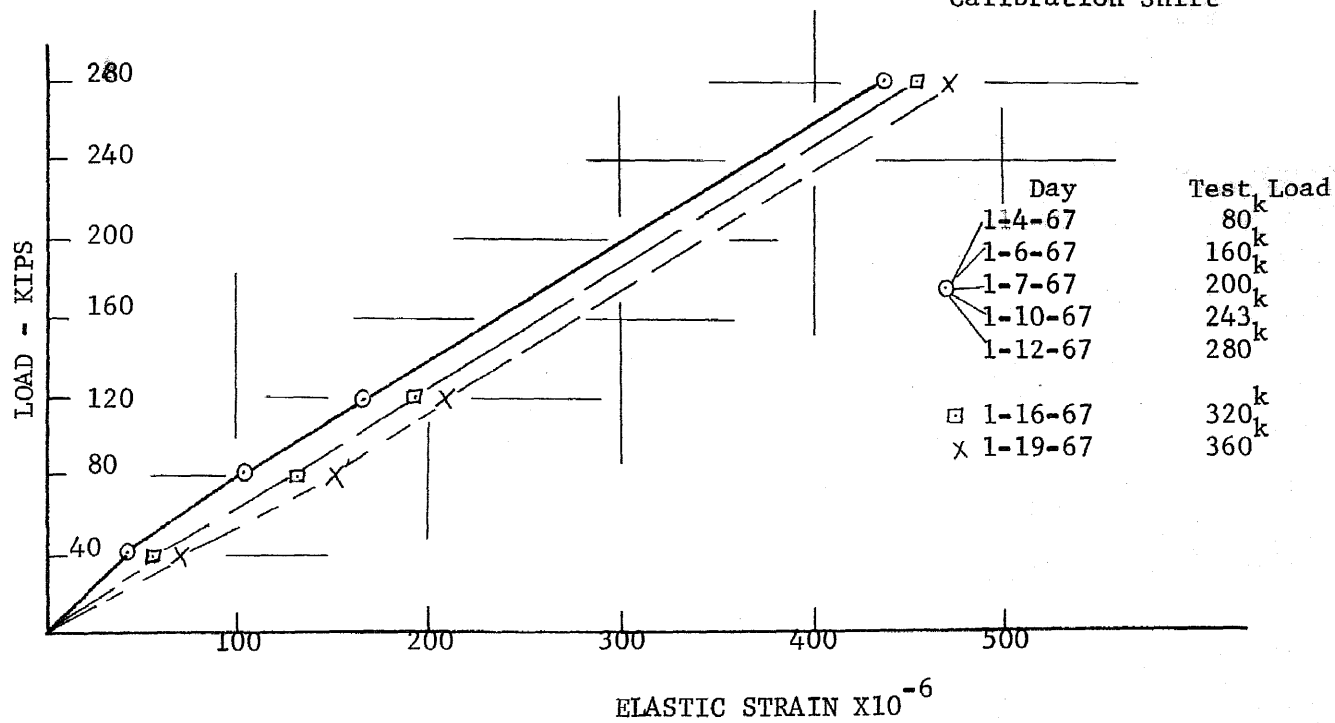


FIGURE 10

PILE 9A
 INELASTIC STRAIN - "ZERO" LOAD VS. TIME

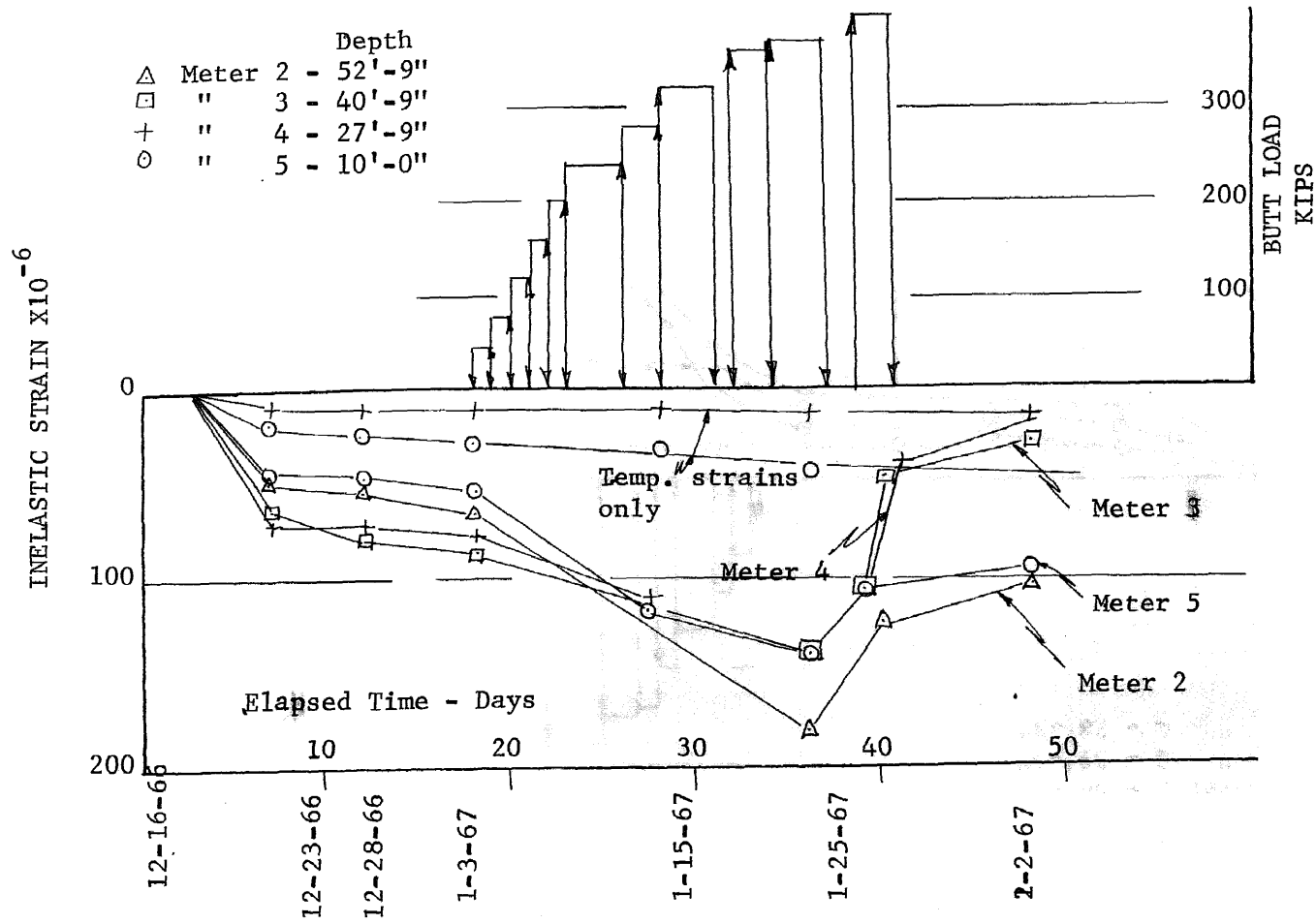


FIGURE 11

PILE 17A
 INELASTIC STRAIN - "ZERO" LOAD VS. TIME

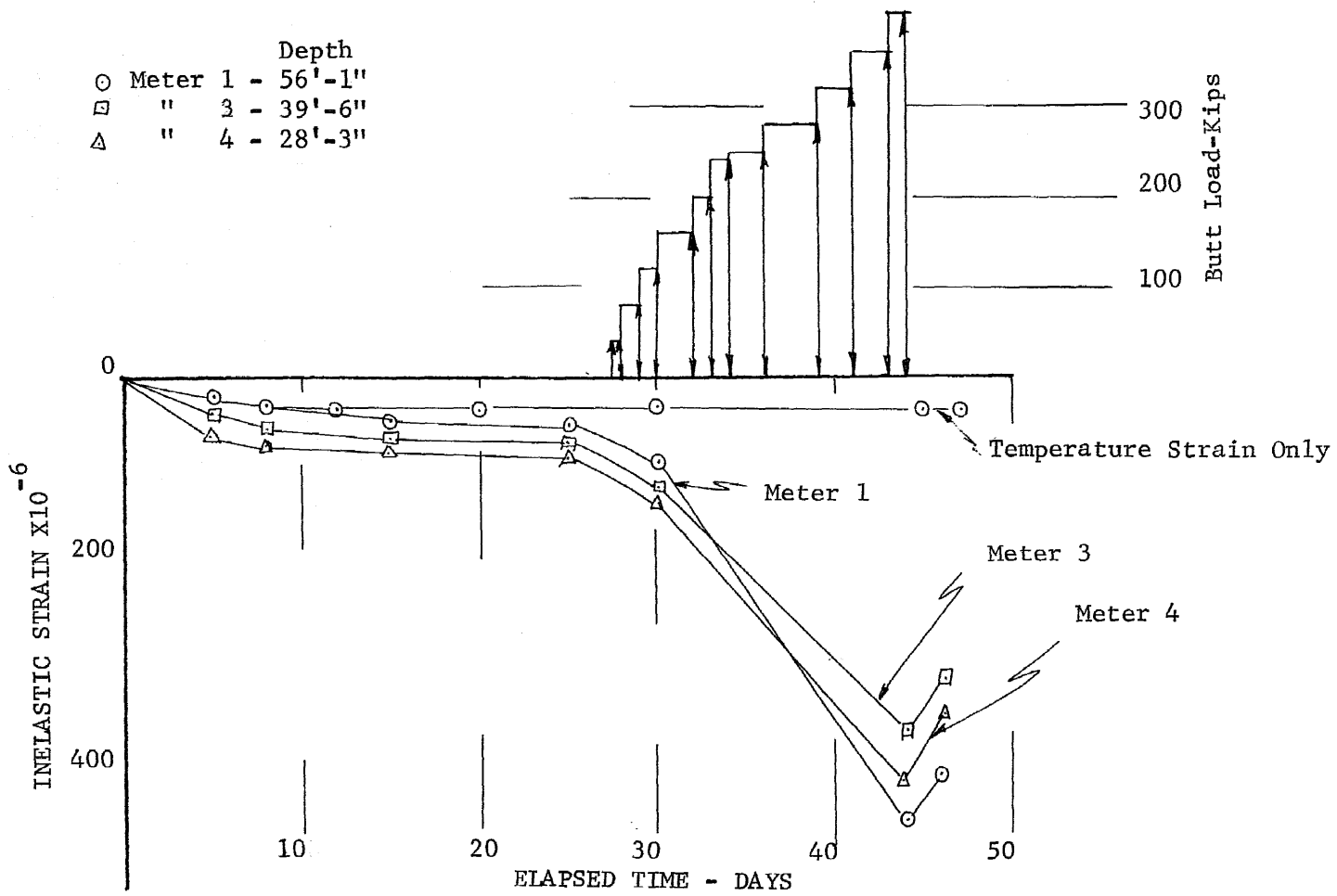
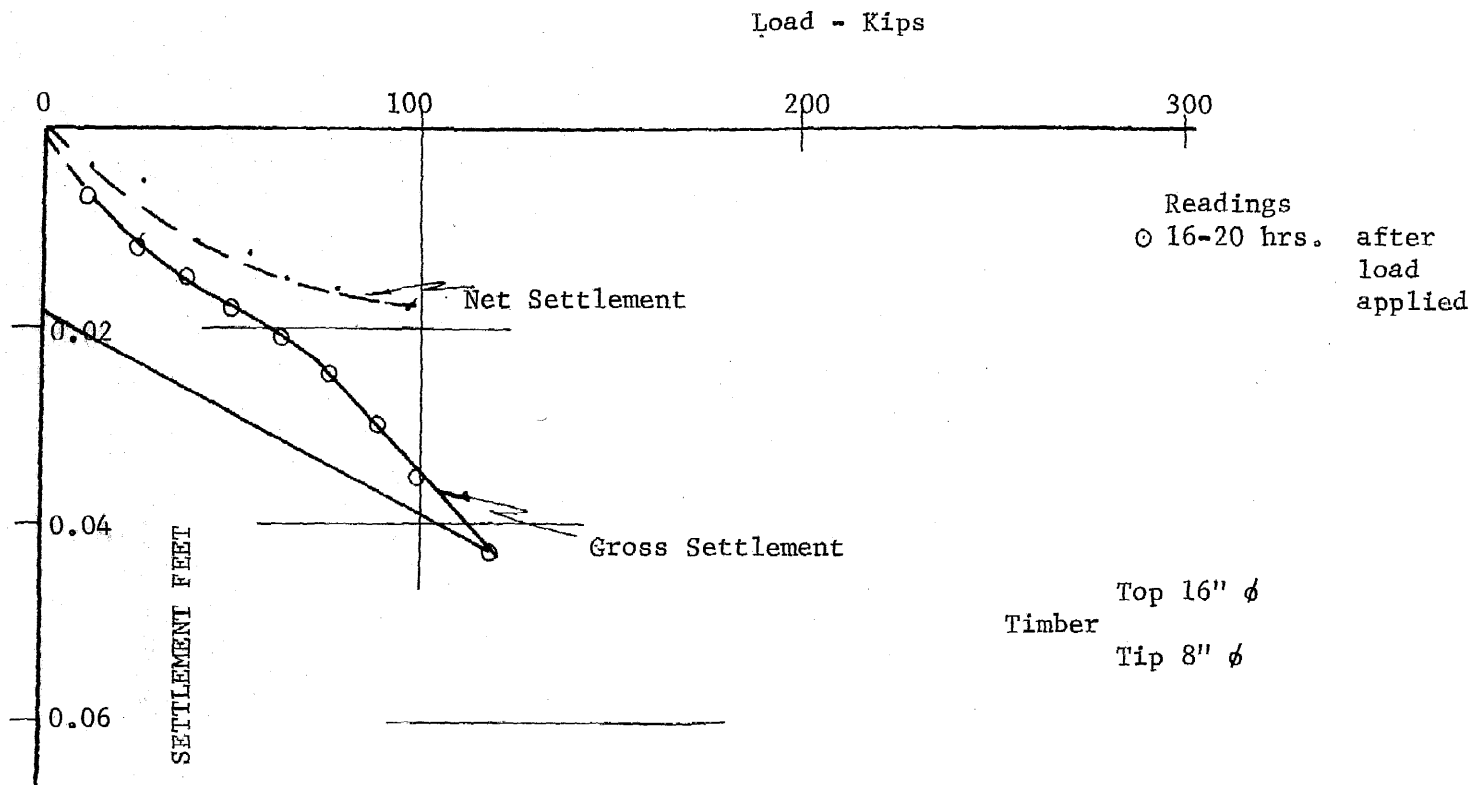


FIGURE 12



PILE 4B
SETTLEMENT OF PILE BUTT VS. BUTT LOAD

FIGURE 13

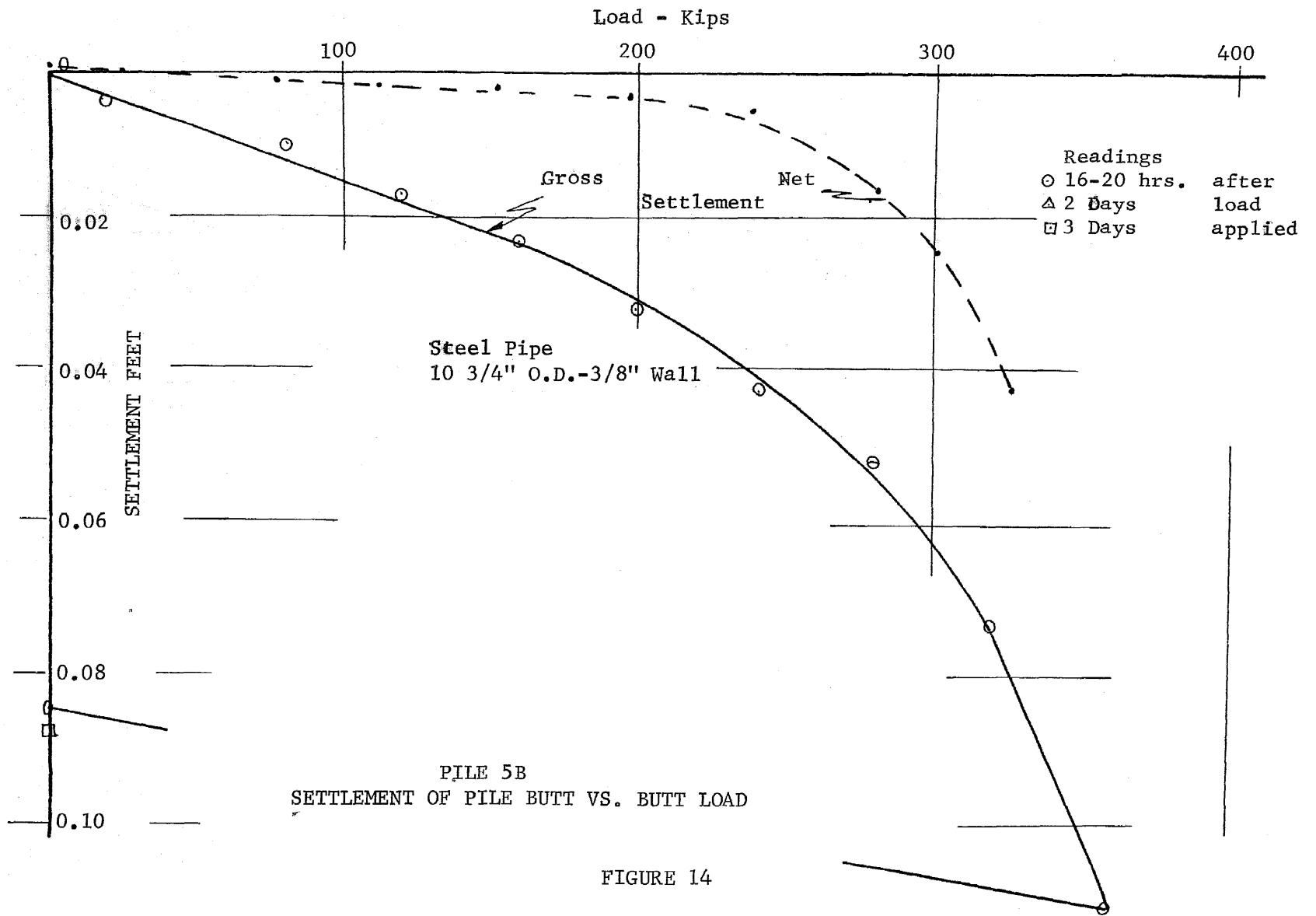


FIGURE 14

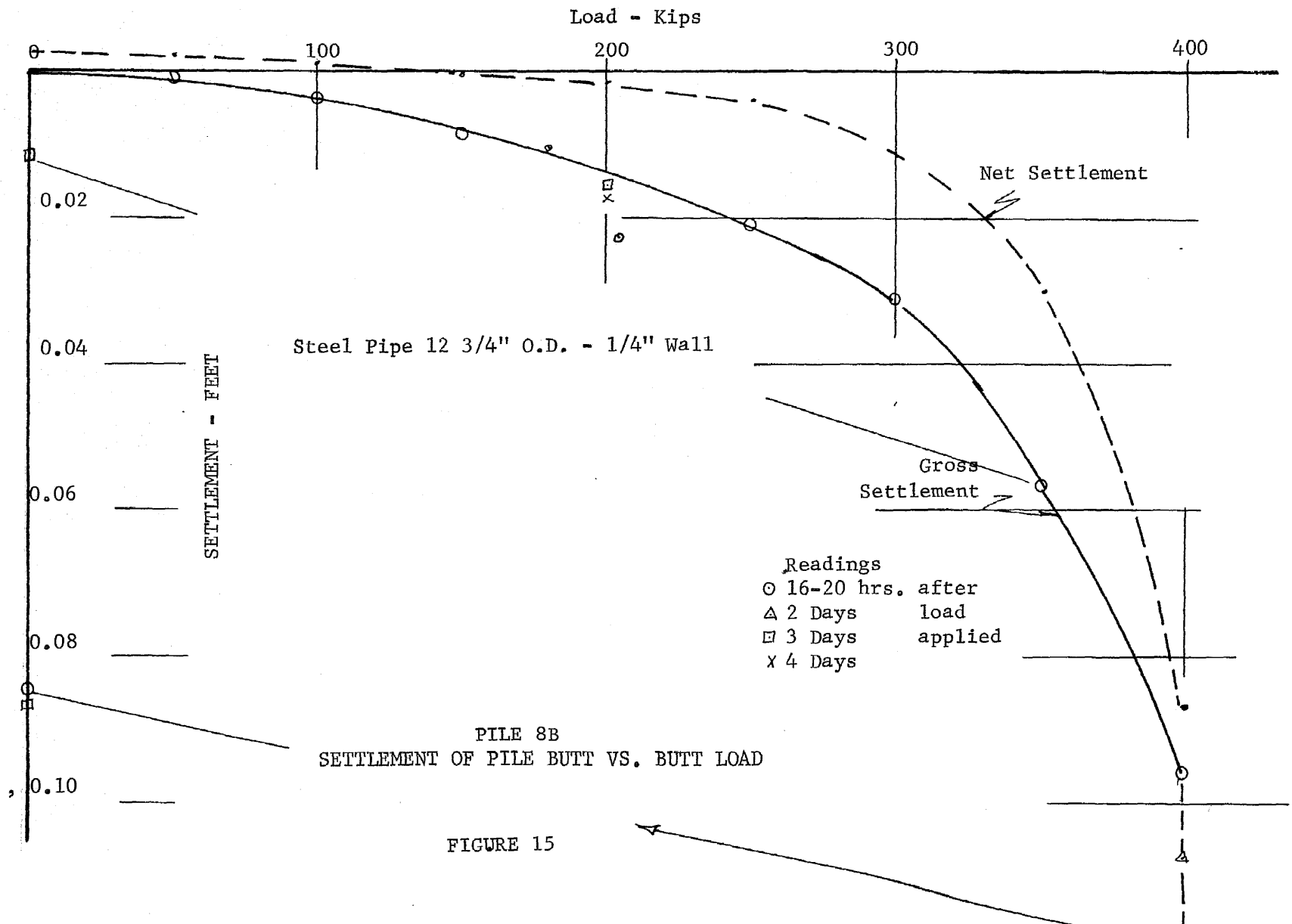
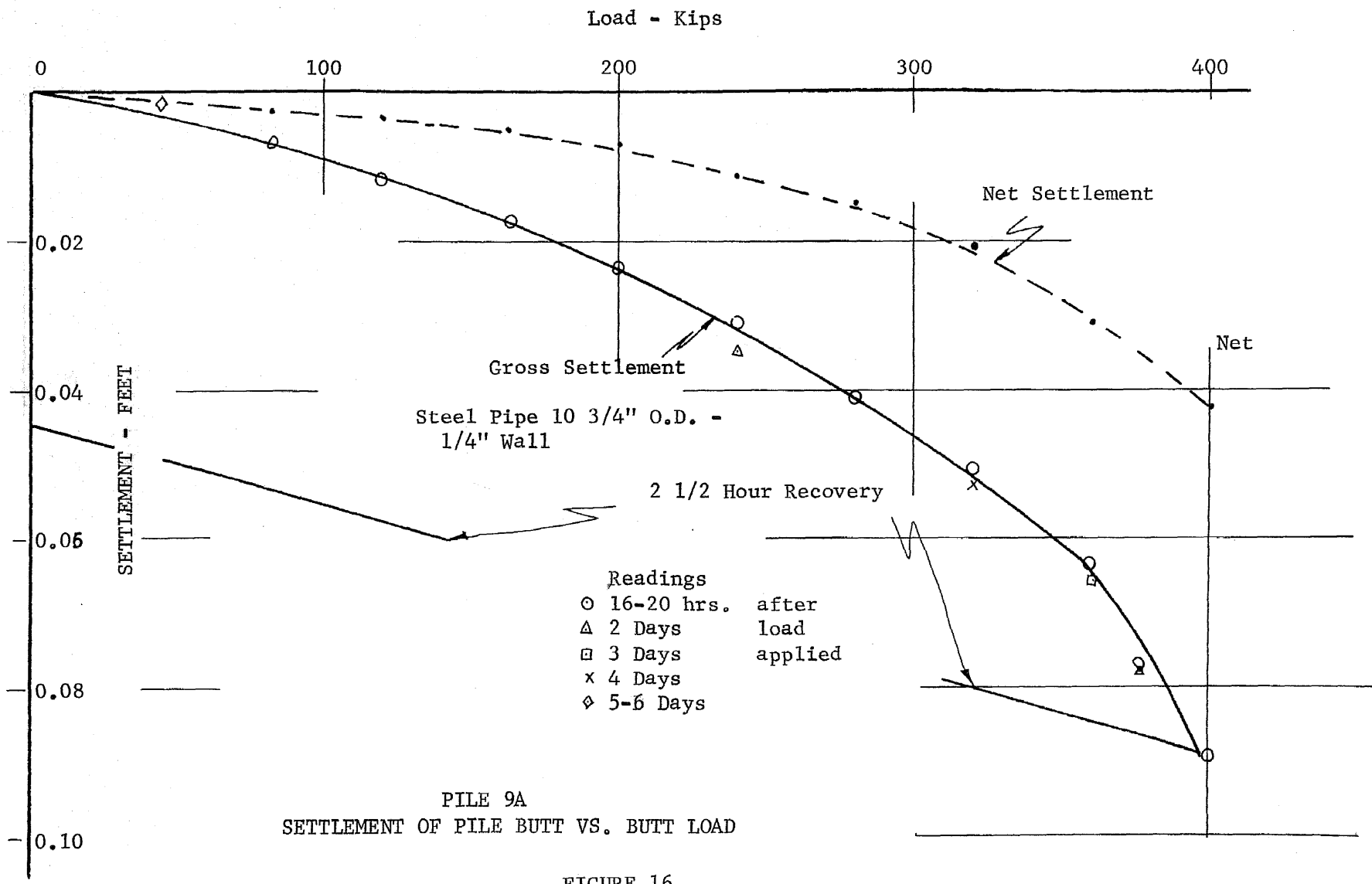
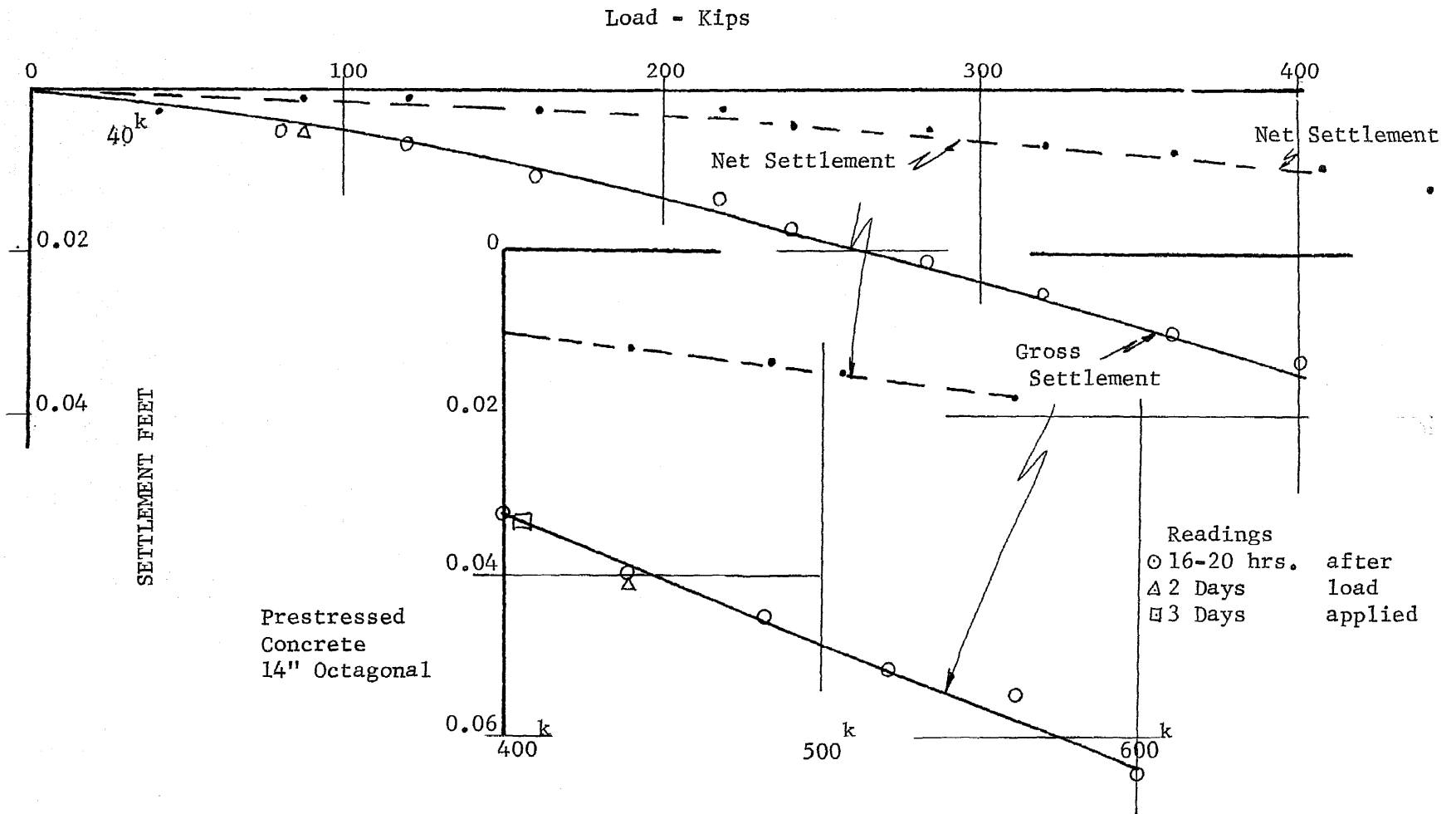


FIGURE 15



PILE 9A
 SETTLEMENT OF PILE BUTT VS. BUTT LOAD

FIGURE 16



PILE 11A
SETTLEMENT OF PILE BUTT VS. BUTT LOAD

FIGURE 17

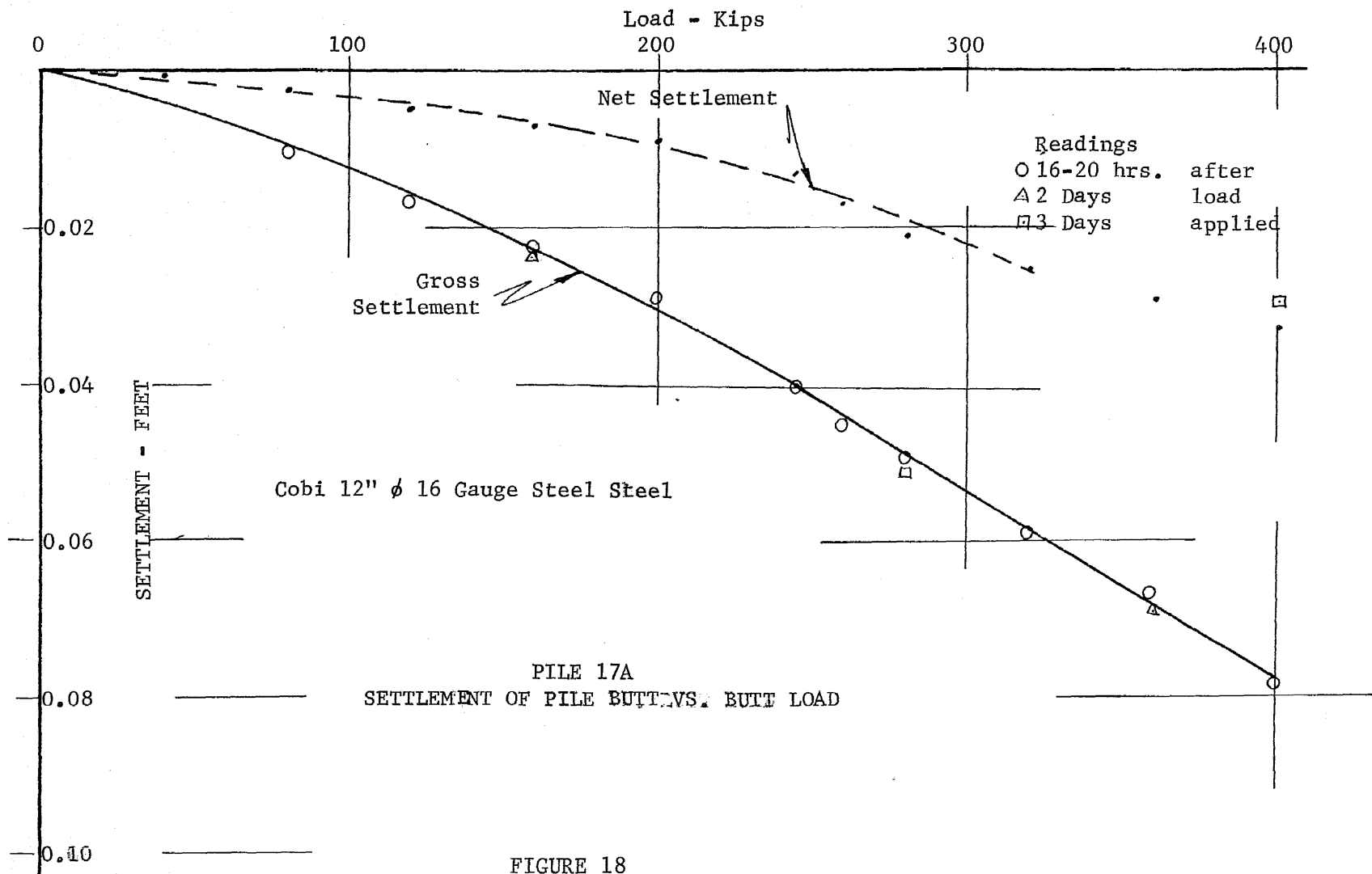


FIGURE 18

PILE 4B
ELASTIC RECOVERY OF PILE TOP

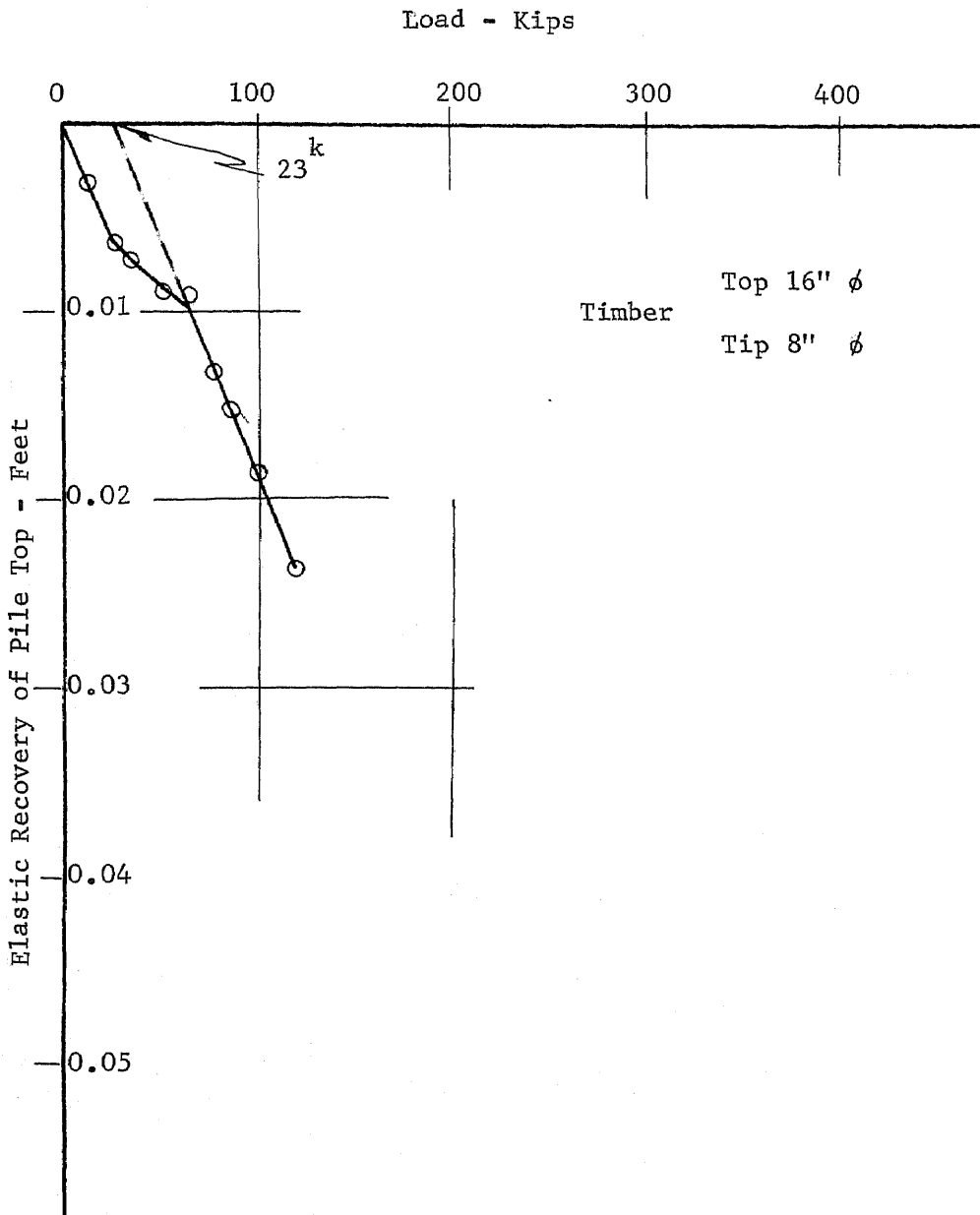


FIGURE 19

PILE 5B
ELASTIC RECOVERY OF PILE TOP

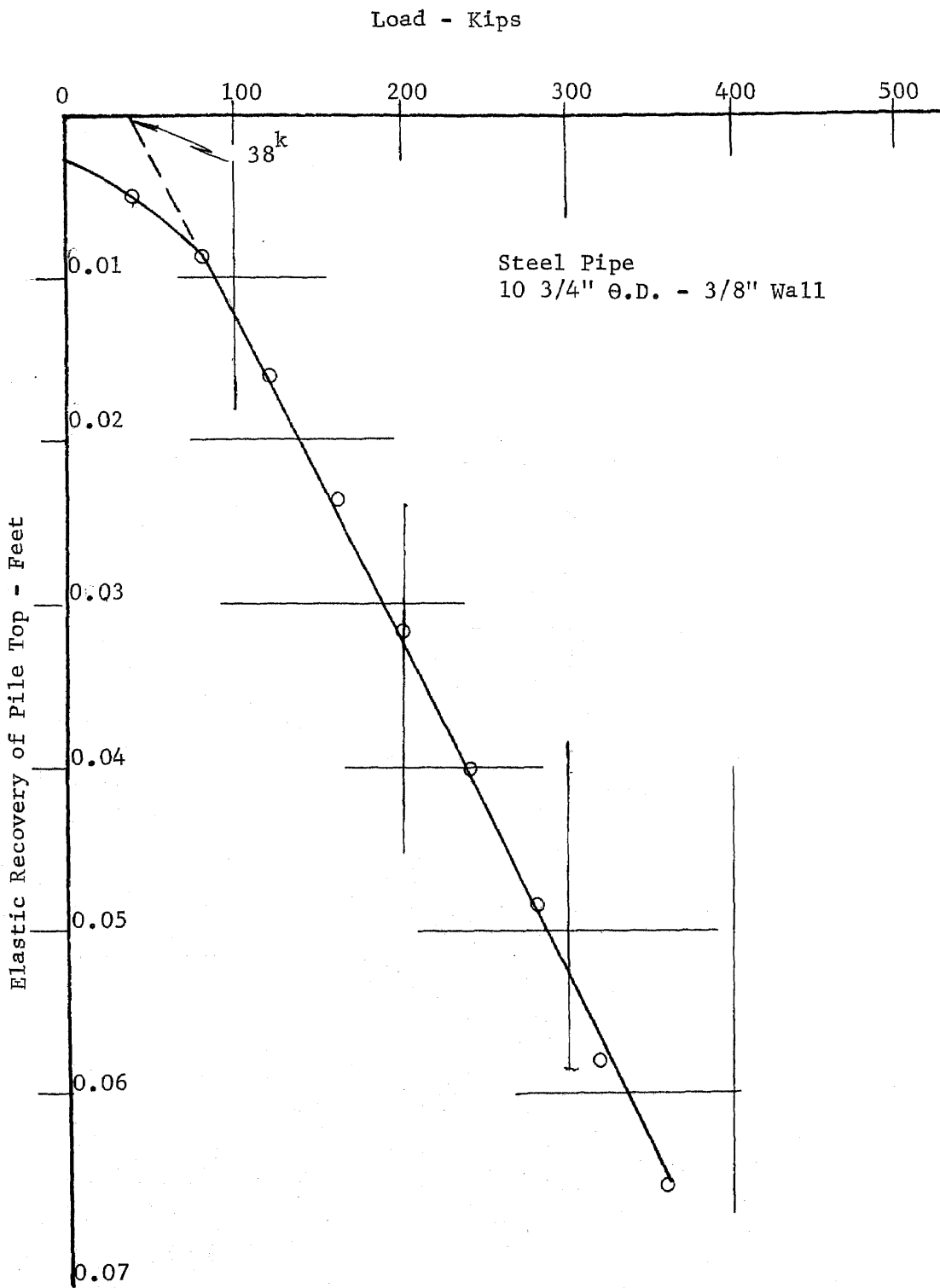


FIGURE 20

FILE 8B
ELASTIC RECOVERY OF PILE TOP

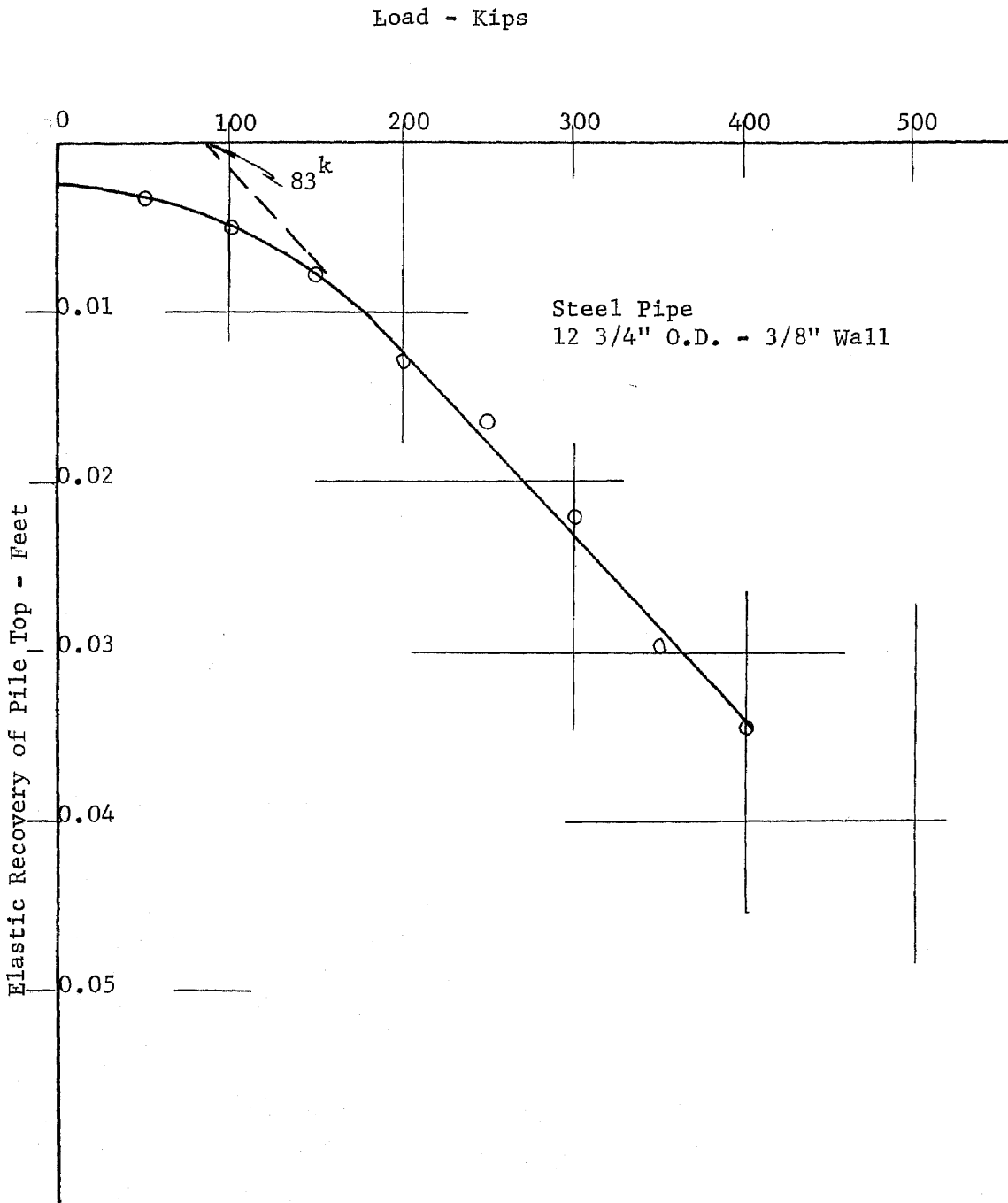


FIGURE 21

PILE 9A
ELASTIC RECOVERY OF PILE TOP

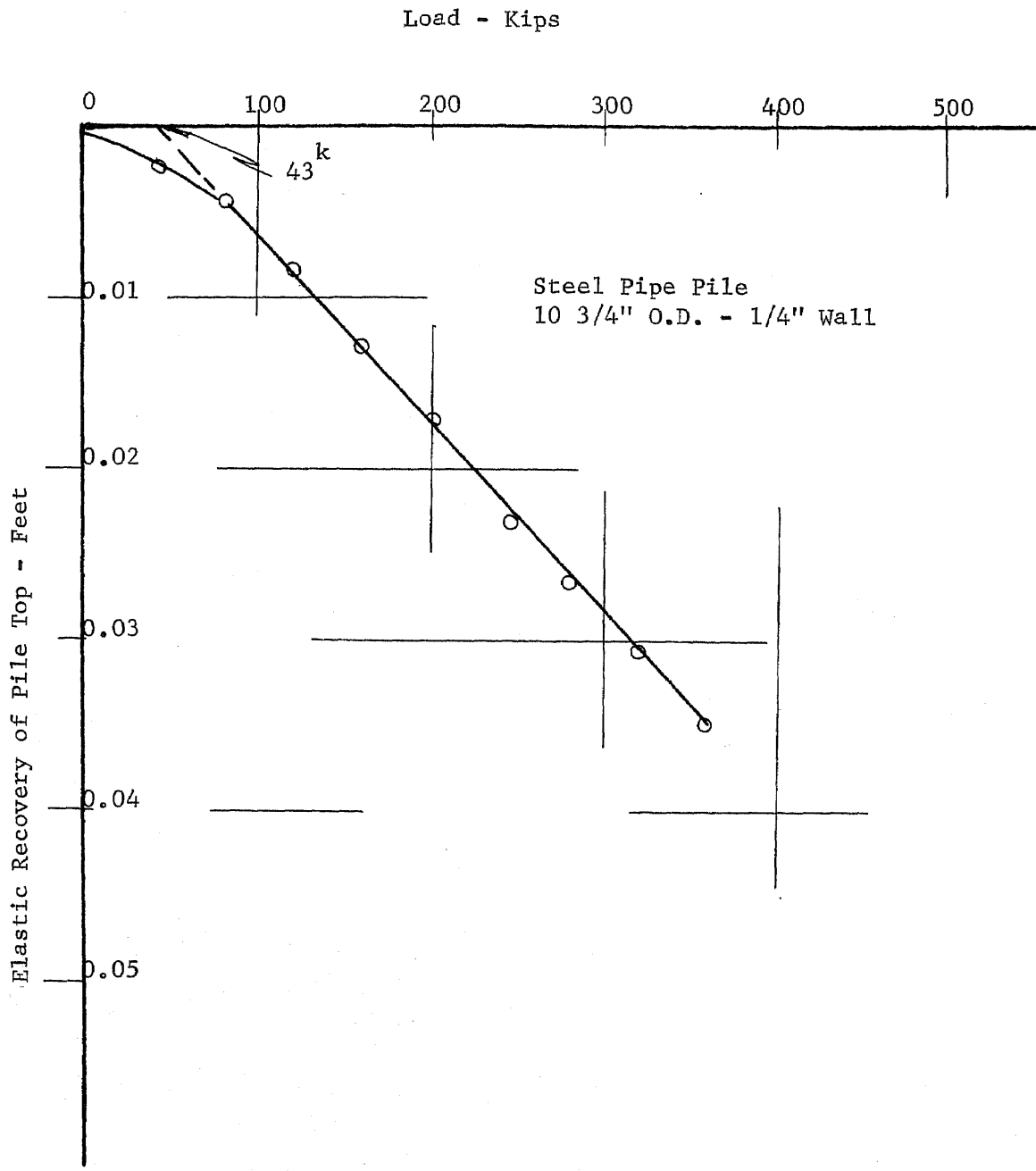


FIGURE 22

PILE 11A
ELASTIC RECOVERY OF PILE TOP

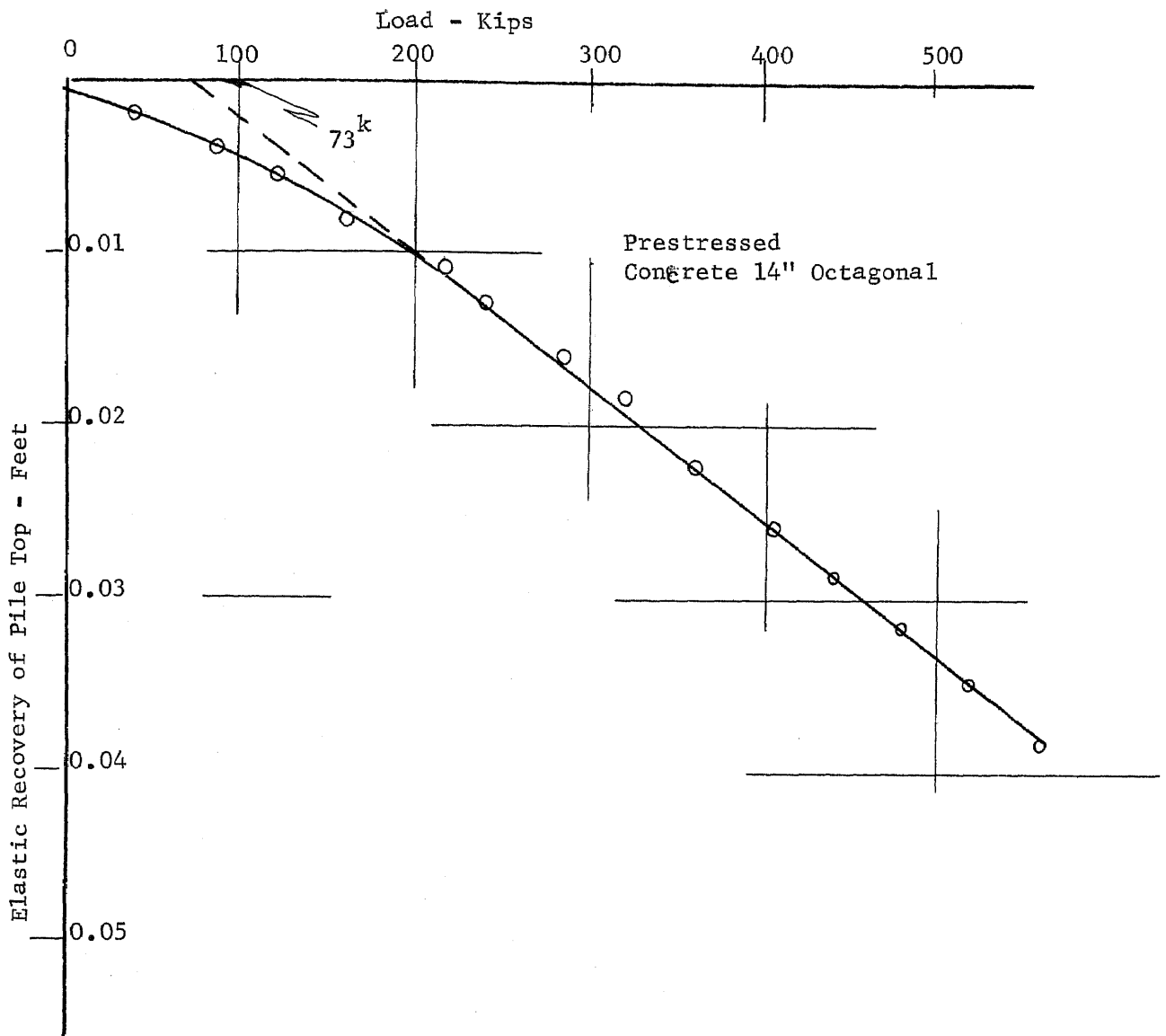


FIGURE 23

PILE 17A
ELASTIC RECOVERY OF PILE TOP

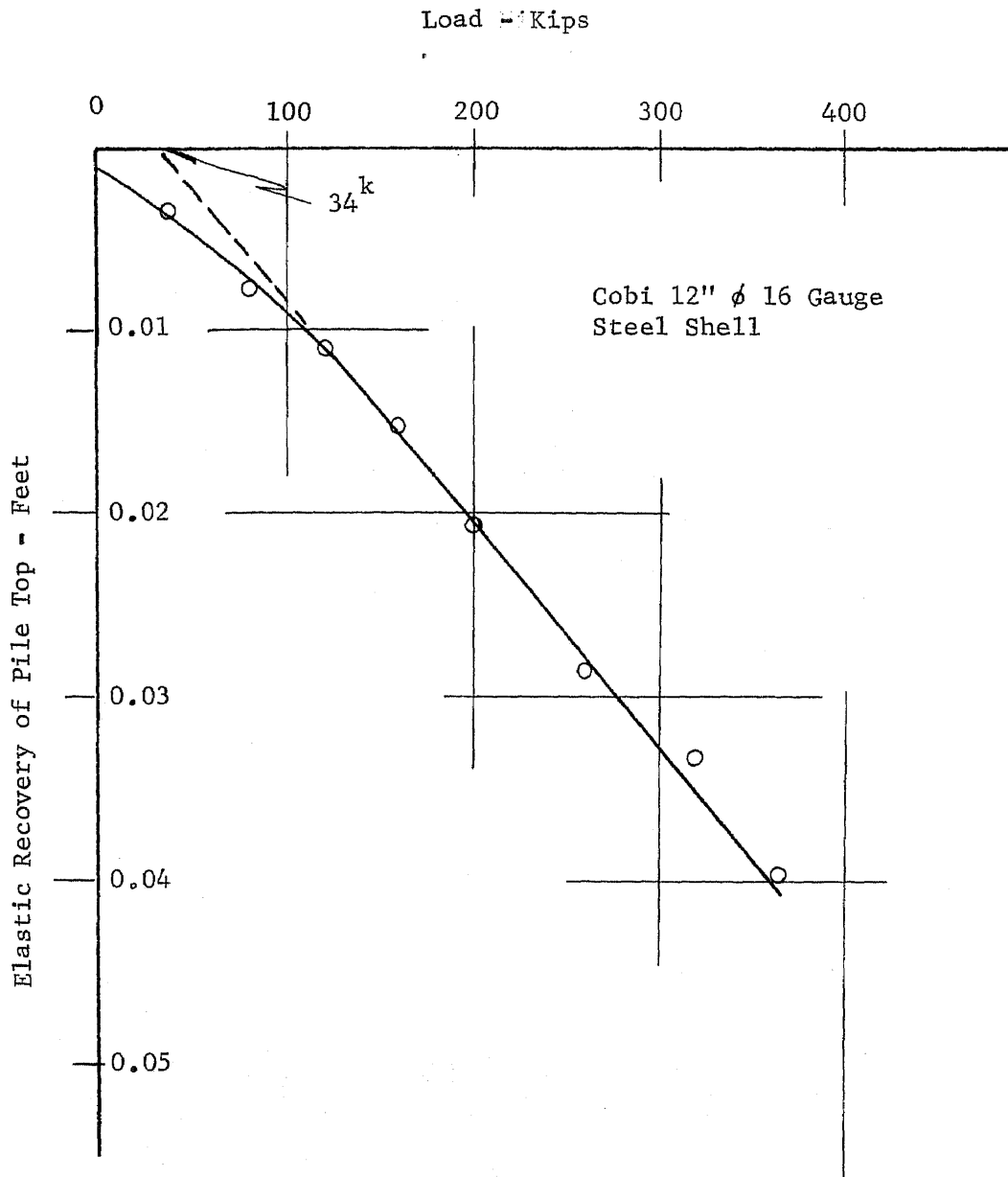


FIGURE 24

PILE 9A
LOAD DISTRIBUTION

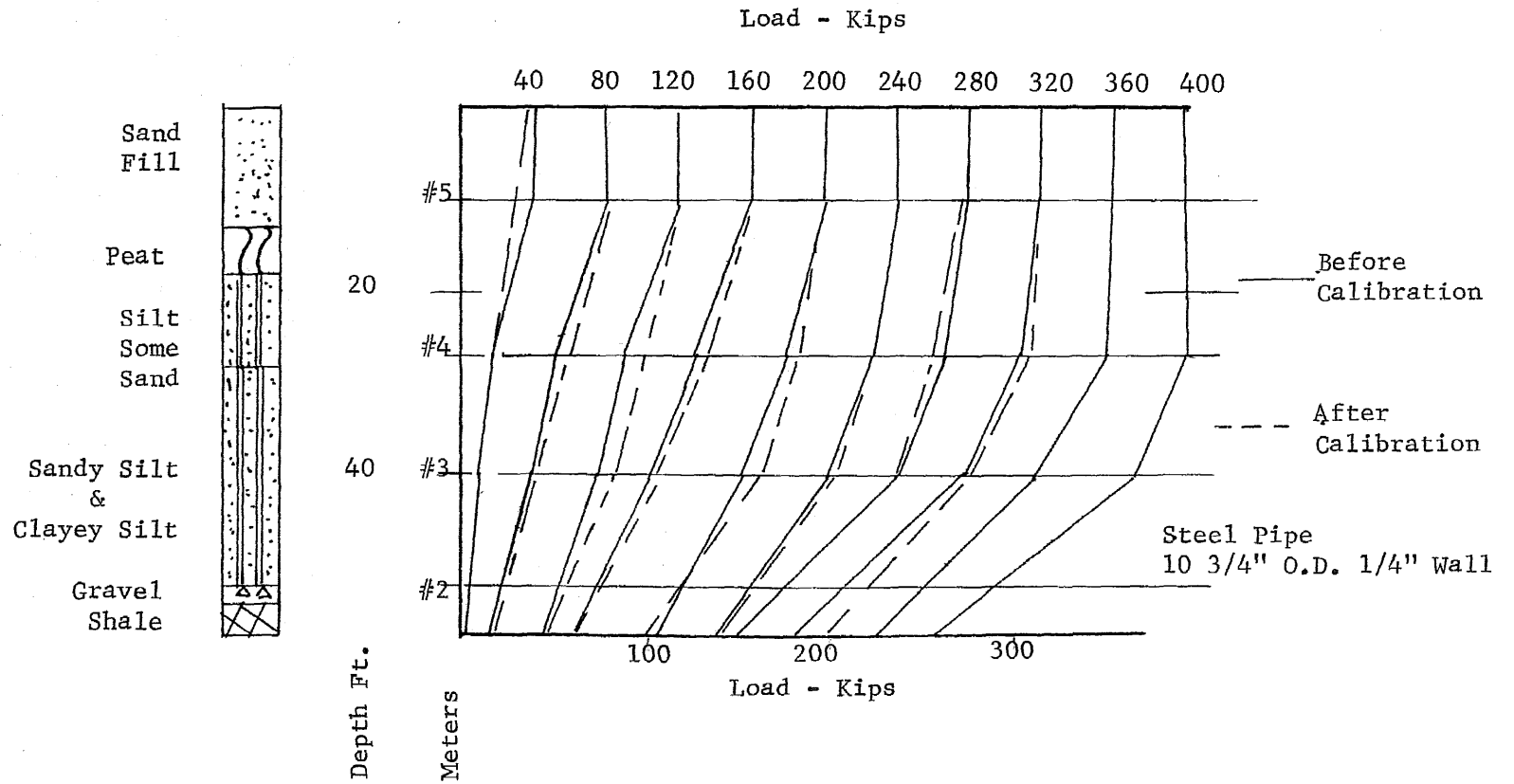


FIGURE 25

PILE 17A
LOAD DISTRIBUTION

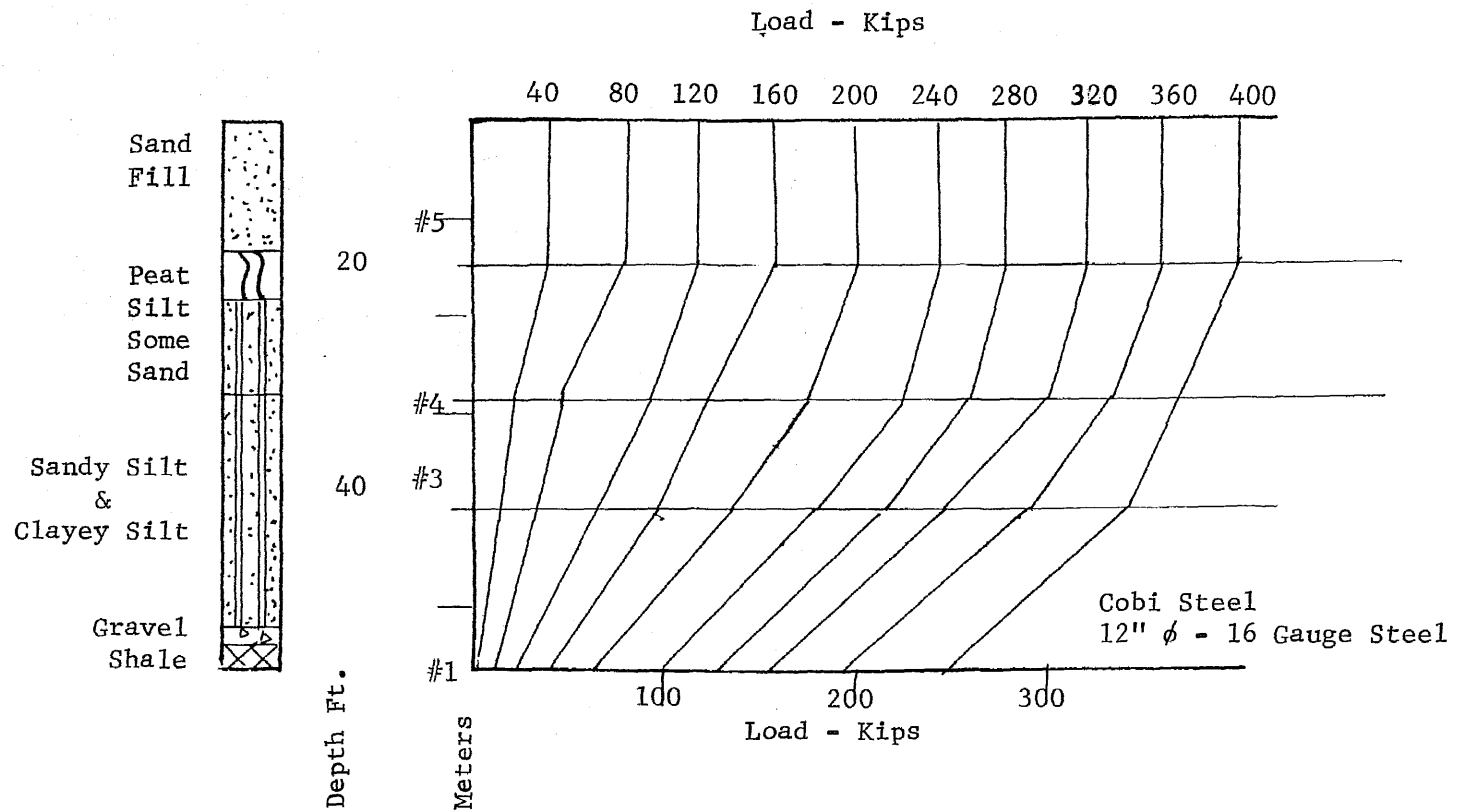


FIGURE 26

LOAD DISTRIBUTION

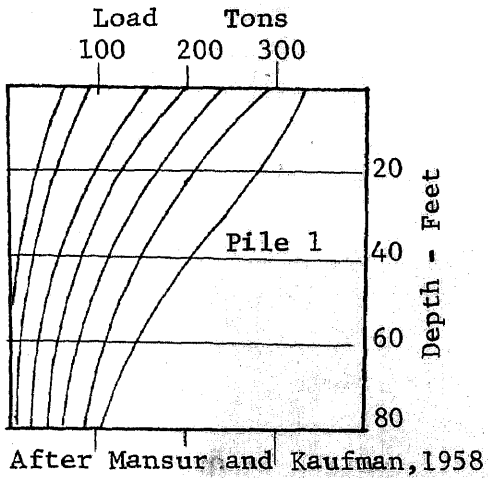
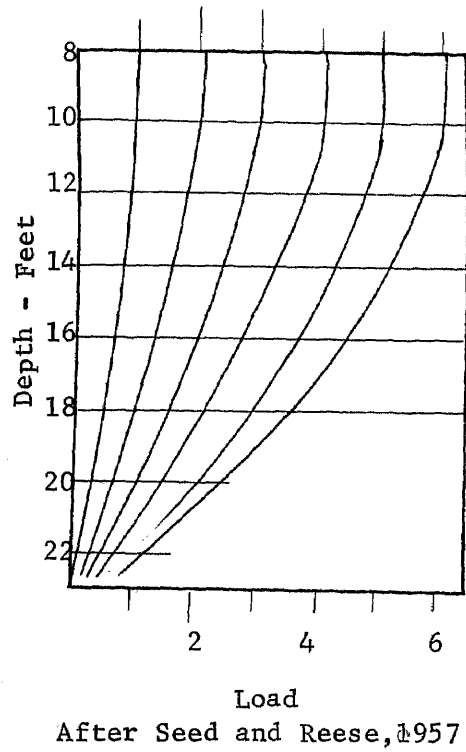
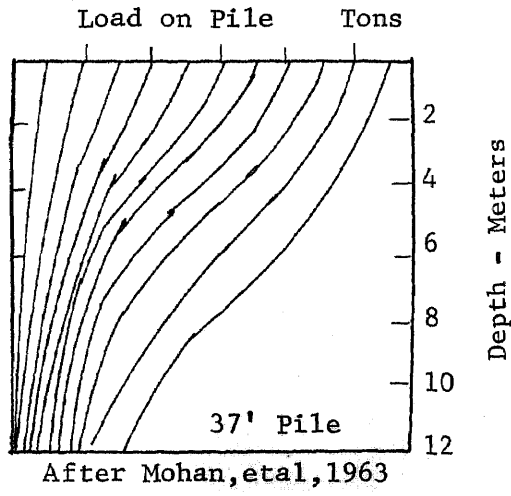


FIGURE 27

TIP LOAD VS. TIP SETTLEMENT
TEST PILES 9A & 17A

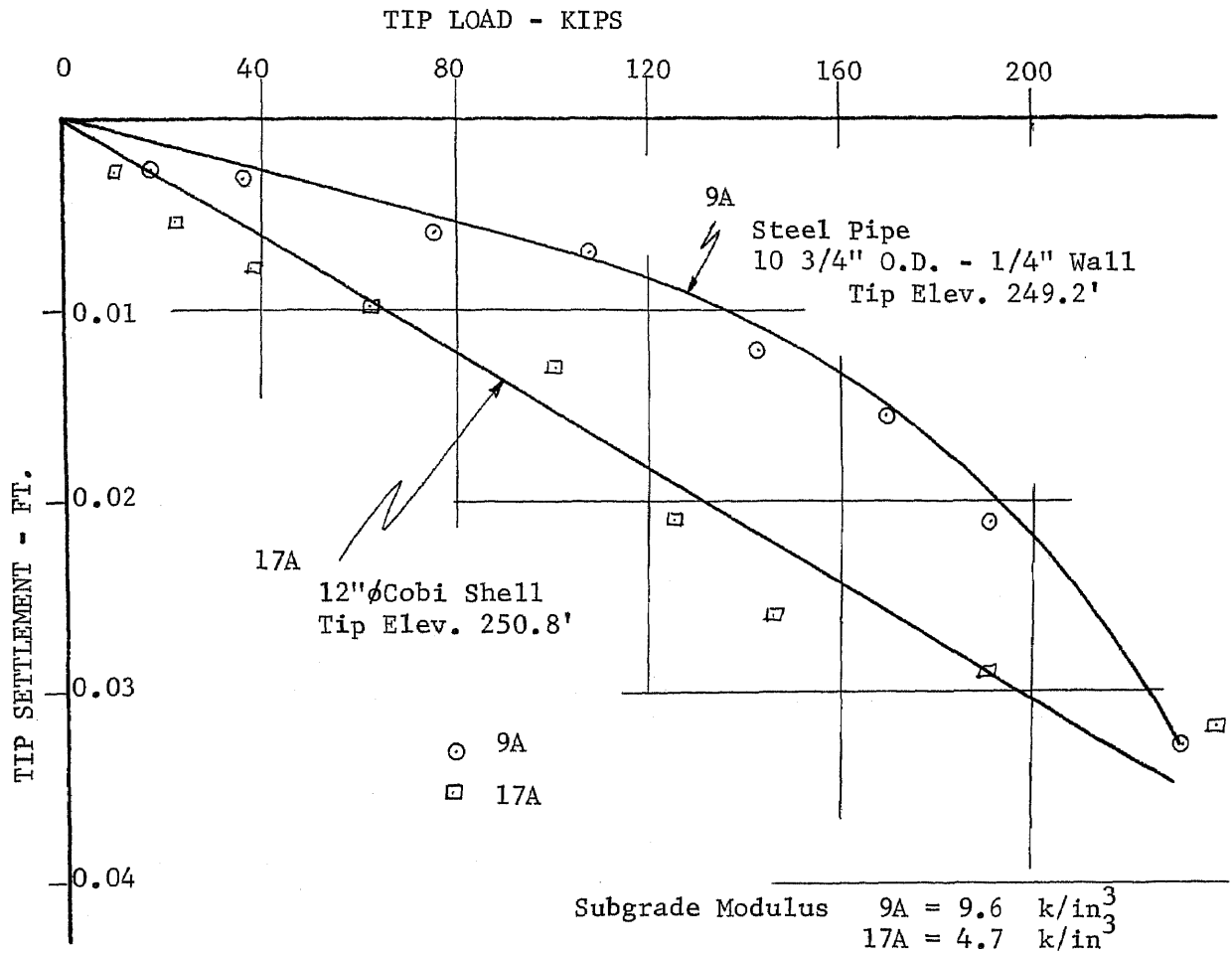


FIGURE 28

ELASTIC RECOVERY OF PILE TIP
VS. TIP LOAD

TEST PILES 9A & 17A

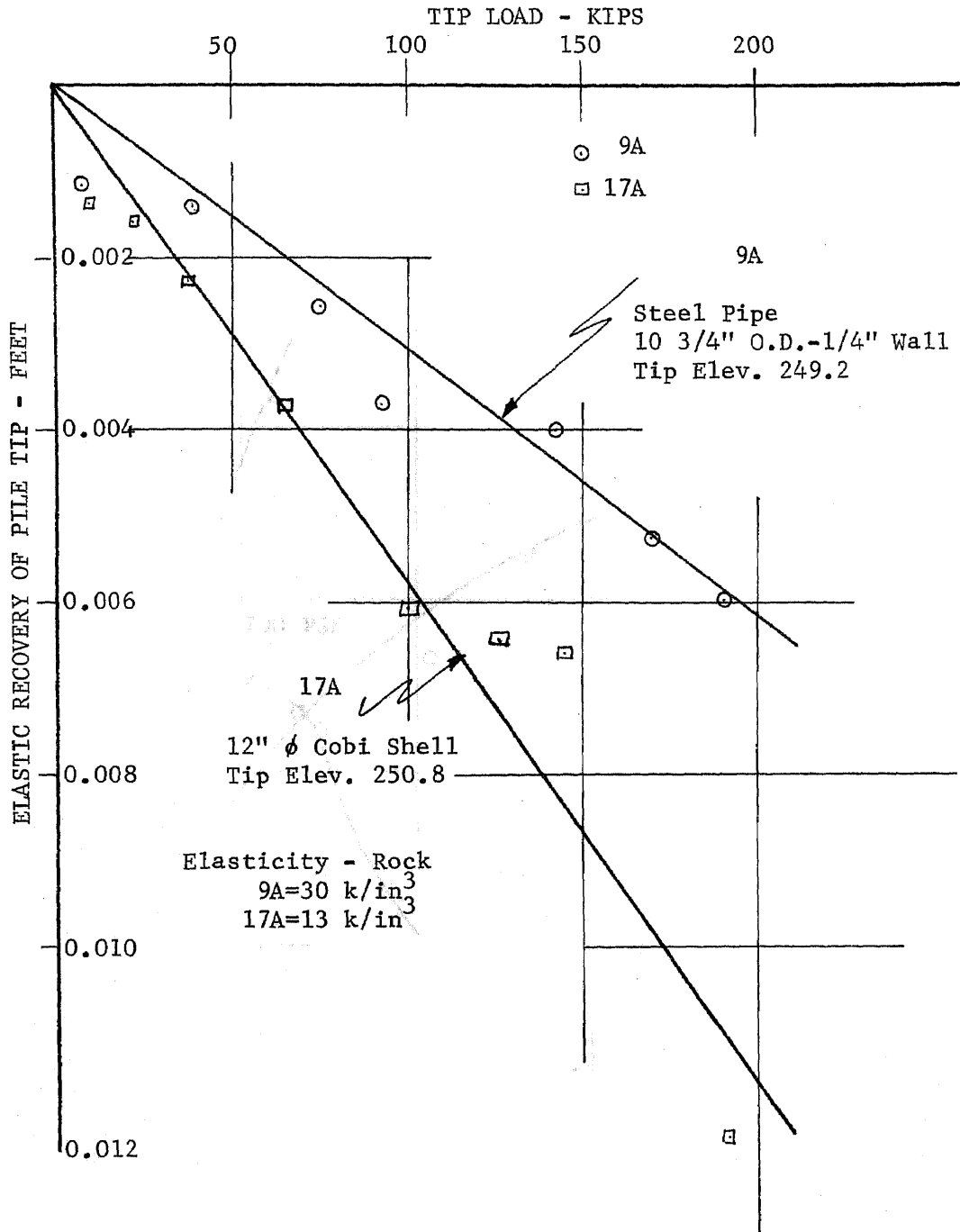


FIGURE 29

PILE 9A
 LOAD TRANSFER VS. PILE MOVEMENT
 (Net Settlement)

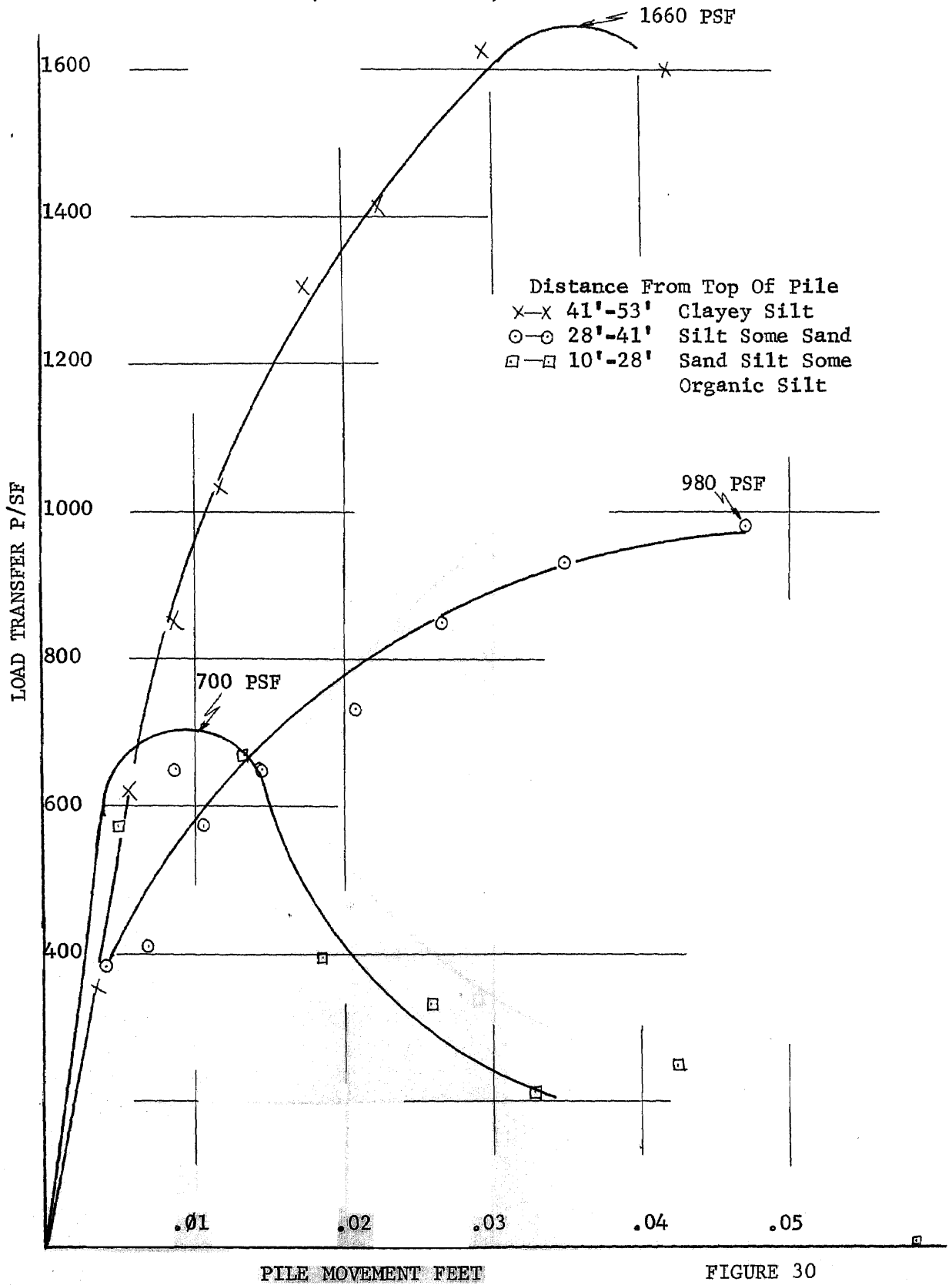
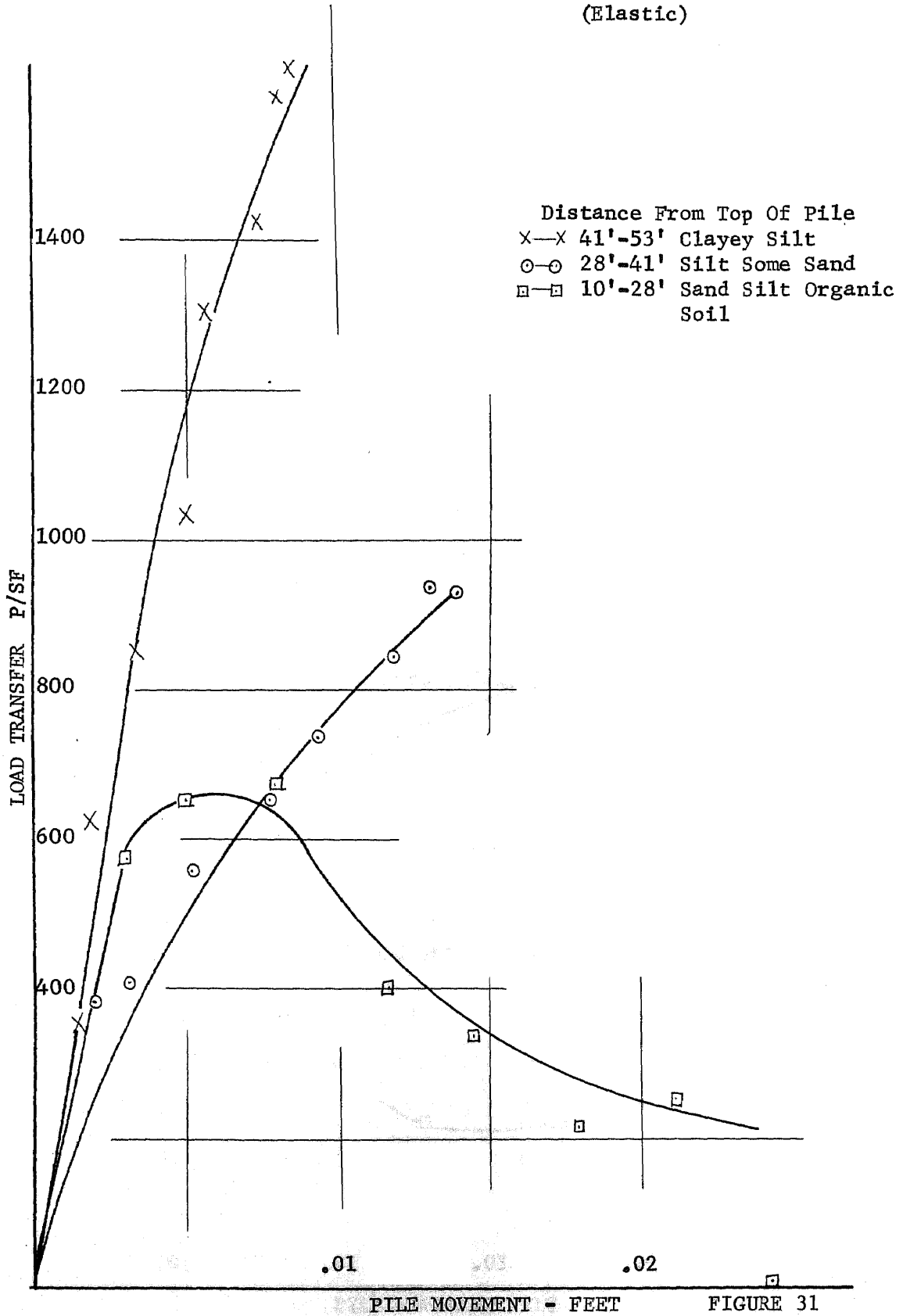


FIGURE 30

PILE 9A
 LOAD TRANSFER VS. PILE MOVEMENT
 (Elastic)



TEST PILE 17A
 LOAD TRANSFER VS. PILE MOVEMENT
 K/LF VS. FEET
 (Net Settlement)

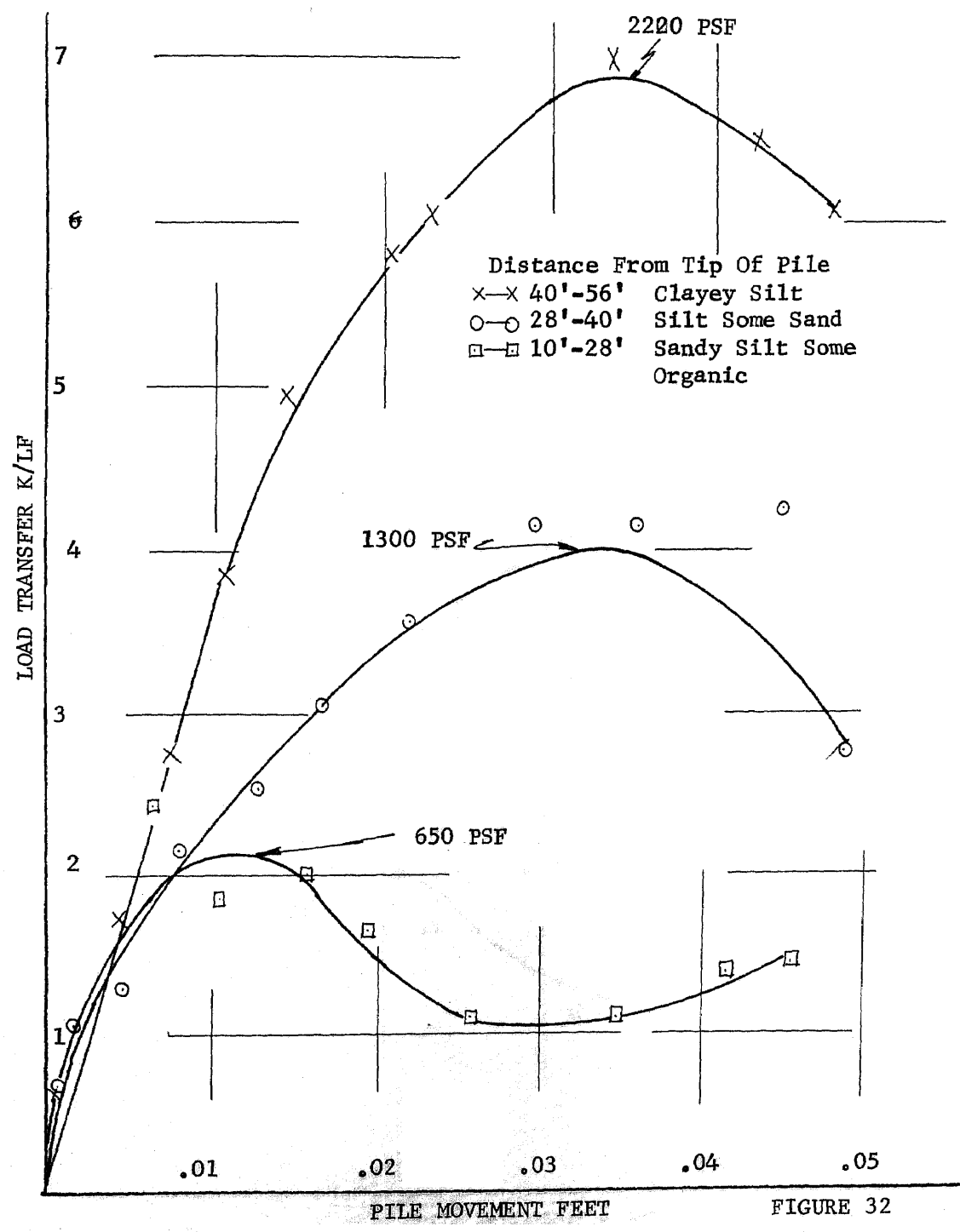


FIGURE 32

TEST PILE 17A
 LOAD TRANSFER VS. PILE MOVEMENT
 (Elastic)

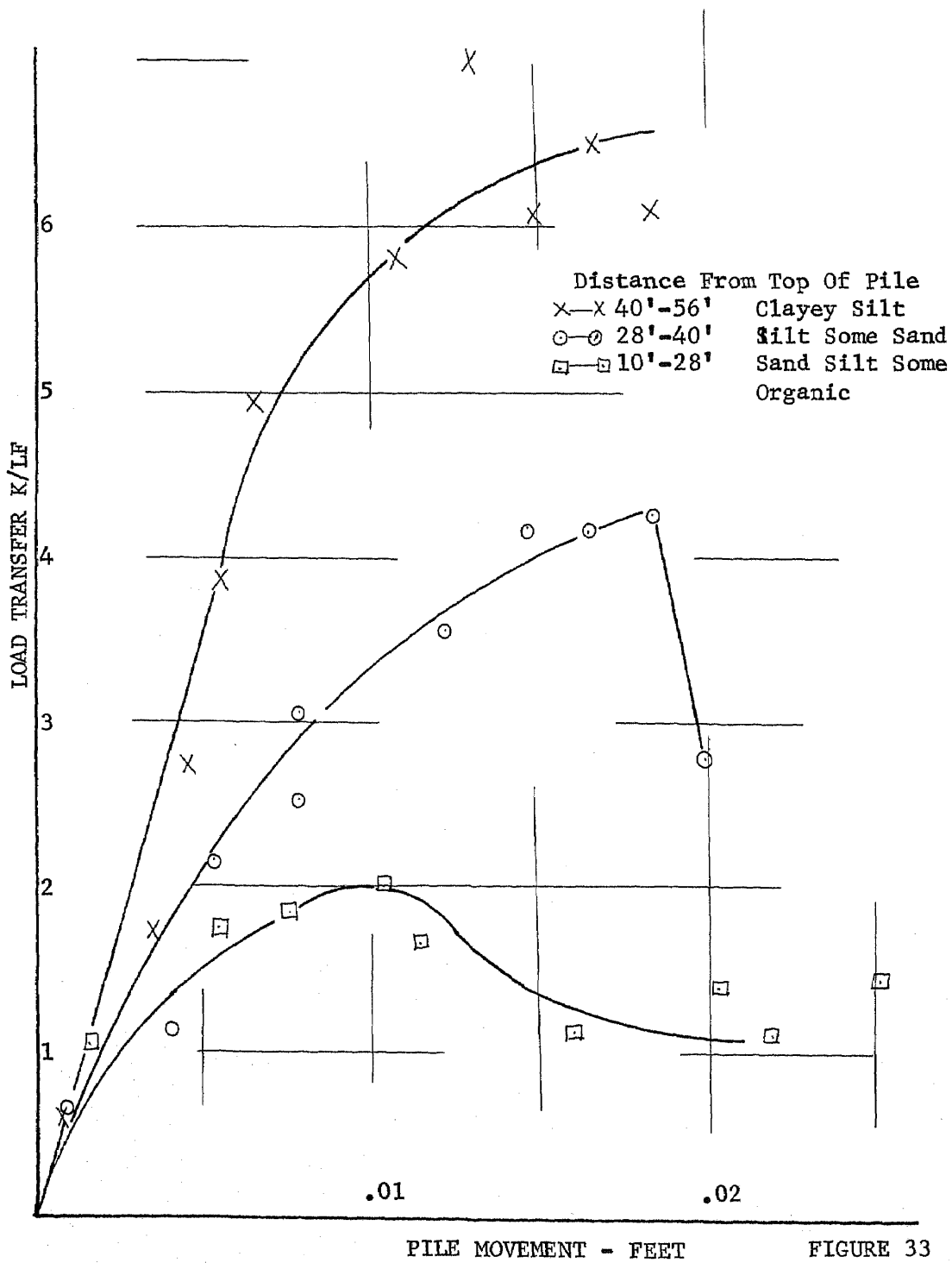


FIGURE 33

LABORATORY CURVE OF SOIL STRENGTH
AFTER COYLE AND REESE, 1966

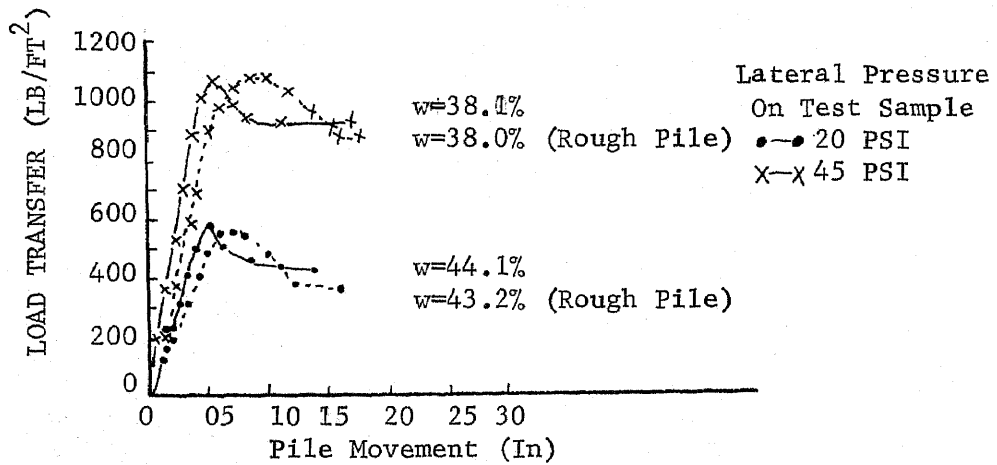
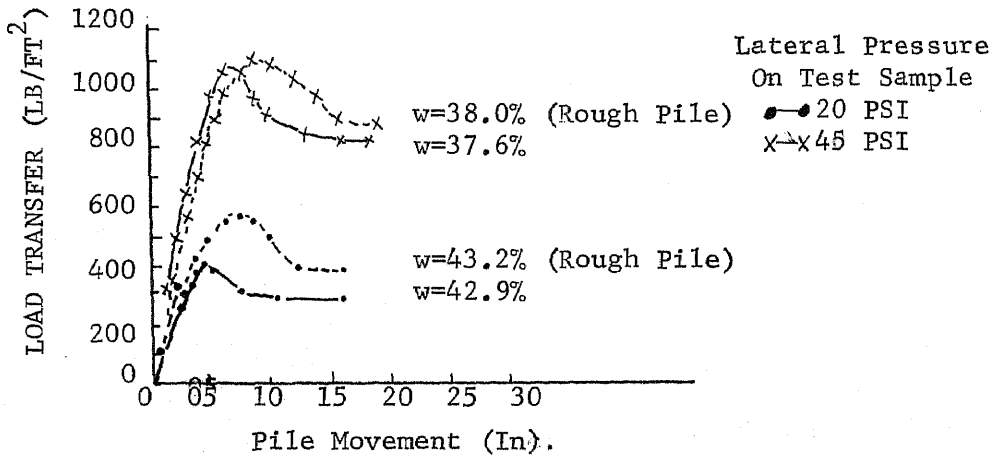


FIGURE 34

PHOTO 1

TEST PILE 5B - DIAL INDICATOR
FOR STRAIN ROD MEASUREMENTS

