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LATERAL PRESSURES AND TIP LOAD ON A CYLINDRICAL PILE DRIVEN IN A DRY COHESIONLESS SOIL

A THESIS

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Roger Alan Brown

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LATERAL PRESSURES AND TIP LOAD ON A CYLINDRICAL PILE DRIVEN IN A DRY COHESIONLESS SOIL

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Chairman

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I would like to thank Mr. Charles Pavey for his time and help in the construction of the equipment. Also I would like to thank Mr. Roland Brown for the many hours of work he spent in helping me in the instrumentation and testing of the piles.

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SUMMARY

The purpose of the pile tests presented in this thesis was to determine by actual measurement the lateral pressures exerted on a rough, 4.5 inch diameter, cylindrical, steel pipe pile after being driven into a dry, relatively loose, cohesionless sand.

A 15-foot pile was driven into an 8-foot diameter, 22 feet deep pit filled with Chattahoochee River sand. To attain the most uniform density possible, the pit was filled by passing the sand through a No. 10 sieve that was eight feet in diameter. The sieve was raised with the sand level, always keeping the sieve two feet above the sand level. A total of three piles were driven; one at an average relative density of 21 per cent; one at an average relative density of 30 per cent; and one at an average relative density of 55 per cent. Each pile was loaded to failure at depths of 8 feet and 13 feet. The pile was instrumented with four load cells to measure the lateral pressures against the side of the pile and one load cell at the pile tip to measure the end bearing. After driving, the pile was loaded to failure and readings were made from all load cells at failure. The end bearing force was subtracted from the total pile failure load to determine the skin resistance.

For these tests, the theoretical end loads that were calculated from the general bearing capacity formula based on Meyerhof's N_q for deep cylindrical foundations and the N_q determined by Berezantsev in 1961, were higher than the actual measured end loads. This indicates

that for a loose sand, their bearing capacity factors give unsafe results. End bearing calculations using Meyerhof's factor for shallow foundations produced safe side results. Therefore the N_q for a loose sand is between Berezantsev's 1961 prediction as an upper limit and Meyerhof's prediction for shallow foundations for a lower limit.

The measured skin friction indicated that from 30 to 50 per cent of the total pile load was skin friction. The theoretical values calculated using Norlund's proposed method were as much as twice the measured skin friction.

The lateral pressures, when plotted as a function of the vertical stress, show a linear relationship to a relative depth of approximately 22 pile diameters. Below the depth of 22 diameters, the lateral pressures appear to be constant.

CHAPTER I

INTRODUCTION

Piles are relatively thin shafts usually of a diameter less than 24 inches that are used to either transfer a load to the soil surrounding it or to carry the load to end bearing at a deeper depth. The use of piles is by no means new as there is evidence of piles being used to support prehistoric dwellings. Usually as many piles as possible were used under these early structures apparently to densify the soil. However, in the mid 1800's as industry progressed, heavier structures were being built, sometimes on soft soil. New materials were introduced, such as concrete and steel, and the necessity of determining the load-carrying ability of a single pile became apparent. In today's world of ever increasing skyscrapers and large complex structures, the need is even more acute.

Fortunately, buckling is not one of the many problems critical to the analysis of piles since the soil surrounding a pile generally gives sufficient lateral support to prevent buckling. A pile's bearing capacity depends on its ability to transfer its load to the surrounding soil and indicates the load at which the pile settlement increases continuously without a load increase. Piles transfer their load by end bearing, skin friction along the sides of the pile, or by a combination of both. Piles driven in sands are usually referred to as friction piles even though a large amount of their capacity may be due to end bearing.

The soil stress distribution on a pile is very complicated and encompasses a vast field of variables, such as the pile shape, the pile size, the pile material, and the many varying soil properties. In developing any theory, one must start with the simplest case, even though it may not exist in practicality, and proceed from there. Therefore the tests in this study were conducted with a friction pile in a dry cohesionless sand of which the properties were known. The sand was placed carefully in an artificial pit in an attempt to creat a homogeneous stratum. A well-instrumented pile was then driven into the pit with a drop hammer and direct measurements were made of the vertical load on the pile tip and the lateral pressures which the sand exerted on the sides of the pile. The measured vertical loads were then compared to the values calculated using Terzaghi's general bearing capacity formula and the bearing capacity factors proposed by Meyerhof and Berezantsev. The measured lateral pressures were used to determine the average skin friction of the pile and were then compared to the skin friction values determined by subtracting the pile end bearing load from the pile failure load.

CHAPTER II

LITERATURE REVIEW

The total bearing capacity of a pile is usually expressed as follows:

$$Q_u = Q_o + Q_f \tag{2.1}$$

where Q_u is the ultimate bearing capacity of the pile

Q_o is the end bearing of the tip of the pile

Q_f is the resistance due to friction of the surface of the pile.

A common assumption is that Q_o and Q_f act independently and are calculated separately. They are then simply added together.

According to Sowers and Sowers (1), the point of a pile can be considered a small shallow footing that punches into the soil and creates successive bearing capacity failures during driving. Therefore ultimate end bearing is computed by using the general bearing capacity equation. Based on these assumptions (see Figure 2-1) Terzaghi (2) developed the general bearing capacity formula as follows:

$$Q_o = \frac{b\gamma}{2} N_\gamma + cN_c + q'N_q \tag{2.2}$$

where Q_o denotes the end bearing of the pile

γ denotes the unit weight of the soil

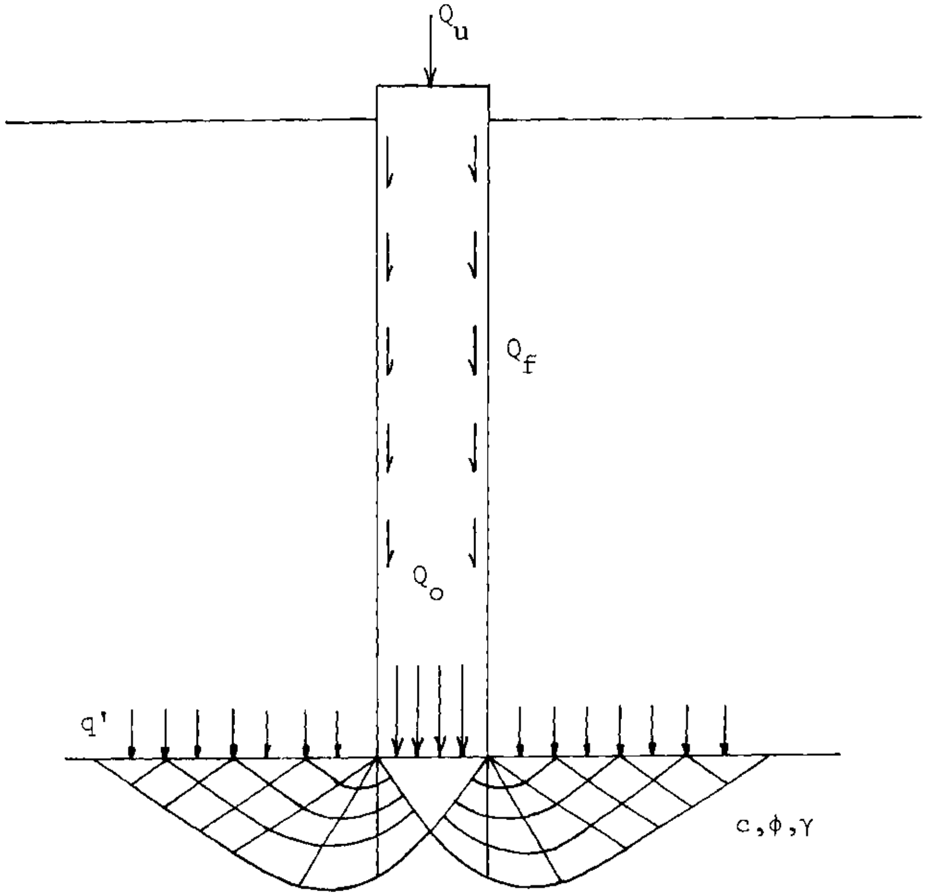

Figure 2-1. Terzaghi's Assumed Failure Pattern Under a Deep Foundation (2)

b is 0.9 times the diameter of the pile

c denotes the cohesion of the soil

q' denotes the surcharge above the pile tip

N_γ is the influence of the soil weight and the foundation width

N_c is the influence of the cohesion

N_q is the influence of the surcharge.

N_γ, N_c, and N_q are called bearing capacity factors and are functions of the angle of internal friction, ϕ.

For a cohesionless material, such as sand, $c \cong 0$. Therefore the equation can be reduced to the following form:

$$Q_o = \frac{\gamma b}{2} N_\gamma + q'N_q \qquad (2.3)$$

Since the diameter of a pile is usually small, b is small which makes the term $\frac{\gamma b}{2} N_\gamma$ very small as compared to $q'N_q$. Now the equation for end bearing is simply

$$Q_o = q'N_q \qquad (2.4)$$

The resistance due to friction on the side of the pile depends on many variables. Basically it can be computed from the formula

$$Q_f = fA \qquad (2.5)$$

where f is the skin friction

A is the surface area of the pile

For a cylindrical pile of diameter D and length L, the area A is

$$A = \int_0^L \pi D \, dz \tag{2.6}$$

The factor f is a complicated parameter and very hard to determine accurately. Sowers and Sowers (1) suggest expressing it as follows:

$$f = a + \bar{p} \tan \delta \tag{2.7}$$

where a is the adhesion

$\bar{p}$ is the effective horizontal stress

δ is the angle of sliding friction of the pile surface.

For a cohesionless material like sand, a = 0, which makes the skin friction equation

$$f = \bar{p} \tan \delta \tag{2.8}$$

Assuming $\bar{p}$ to be directly proportional to the average overburden pressure, it can be expressed as

$$\bar{p} = K \bar{q} \tag{2.9}$$

where K is the coefficient of lateral earth pressure

$\bar{q}$ is the average overburden pressure.

The average overburden pressure, $\bar{q}$, can be expressed as γZ for a dry sand where γ is the unit weight of the sand and Z is the depth.

Therefore

$$f = K\gamma Z \tan \delta \tag{2.10}$$

and

$$Q_f = \int_0^L \pi K\gamma ZD \tan \delta \, dz \tag{2.11}$$

$$Q_f = \frac{1}{2} \pi K\gamma DL^2 \tan \delta \tag{2.12}$$

Wu (3) and Silburman (4) both give Equation (2.12) as the equation for finding the total skin resistance. However, recently Coyle and Sulaiman (5) have shown that Equation (2.12) is invalid, stating that both K and δ vary with depth and are not constant as assumed. Vesic (6), on the other hand, in his 1967 report has stated that for very long piles, both the end bearing and side resistances reach a constant value. He has found that up to a relative depth of 10 pile diameters for a relatively loose sand, and 30 pile diameters for a relatively dense sand, the lateral pressure distribution of a pile increased linearly with depth and then remained constant for any further increase in depth. To support this, Vesic hypothesized a constant K and a variable vertical pressure, σ_z. He has explained the variable vertical pressure by arching in the sand around the pile at and below a relative depth of from 10 pile diameters for a loose sand to 30 pile diameters for a dense sand, causing the σ_z to remain constant with K below the relative depth where the arching began. Healy and Meitzler (7) state that the soil stresses causing skin friction are not directly related to the overburden pressures but primarily to the sand density and method of pile placement. According to Szechy (8), the frictional resistance, Q_f, should be multiplied by a function of the soil displaced by the pile, V_k. He has developed a method to determine a V_k which varies with the void ratio of the sand, the relative density, the grain distribution, and the length and diameter of the pile. The effects of the function V_k seem to be very small so the conventional assumption of the two factors acting independently is made here for simplicity.

As a pile is driven into a dry cohesionless soil, such as sand, the soil is compacted in the vicinity of the pile. The amount of this compaction depends on the size of the pile being driven or the volume of soil being displaced. The change in the ϕ angle which results from this increase in density then not only depends on the ϕ of the soil in the undisturbed state but also depends on the pile size. Meyerhof (9), to account for this change in compaction influence, developed a method in which he calculated the ϕ of the compacted soil and used it in a static bearing capacity formula. Meyerhof hypothesized a zone of failure in the shape of an inverted heart (see Figure 2.2). Several years later, Norlund (10) introduced another method to account for the change in ϕ due to the compaction from impact driving. Norlund included in his work the effect of pile taper and roughness of the pile surface which Meyerhoff did not include.

Norlund (10) expressed the bearing capacity of a pile in a sand as

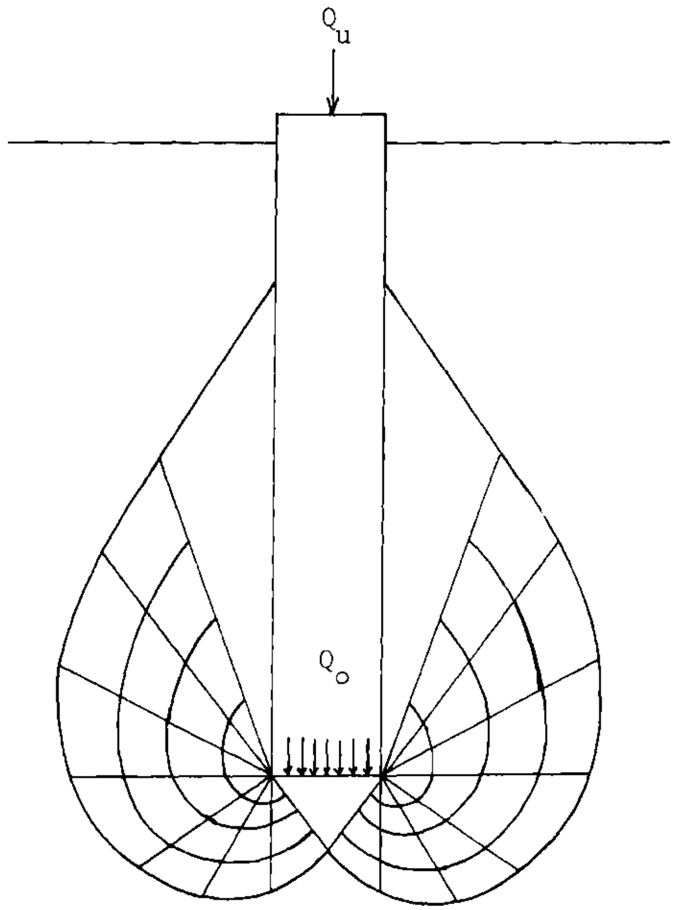

Figure 2-2. Meyerhof's Failure Pattern Under a Driven Circular Pile. From Meyerhof (9) (Based on Jaky)

$$Q_u = N_q A p_d + \sum_{d=0}^{d=D} K_\delta p_d \sin(\omega+\delta) \sec(\omega) C_d \Delta d \qquad (2.13)$$

where Q_u is the ultimate bearing capacity.

N_q is the dimensionless bearing capacity factor of surcharge influence.

A is the area of the pile tip.

p_d is the effective overburden pressure.

D is the depth of embedment of the pile.

K_δ is the ratio of the resultant of the effective normal and shear stresses on an incipient failure plane passing through a point and the effective overburden pressure at that point.

ω is the pile taper in degrees.

δ is the friction angle on the surface of sliding.

C_d is the minimum perimeter of the pile.

A pile with no taper reduces Equation (2.13) to

$$Q_u = N_q A p_d + \sum_{d=0}^{d=D} K_\delta p_d \tan(\delta)\, C_d \Delta d \qquad (2.14)$$

As stated previously, N_q is the bearing capacity factor which gives the influence of the surcharge. Many investigators have used various approaches to obtain N_q. Figure 2.3 gives a result of several of these investigators. In 1961 Berezantsev, et al. (11) presented a solution where N_q was a function not only of ϕ but also of the ratio D/B where D was the depth of embedment and B was the diameter of the pile tip.

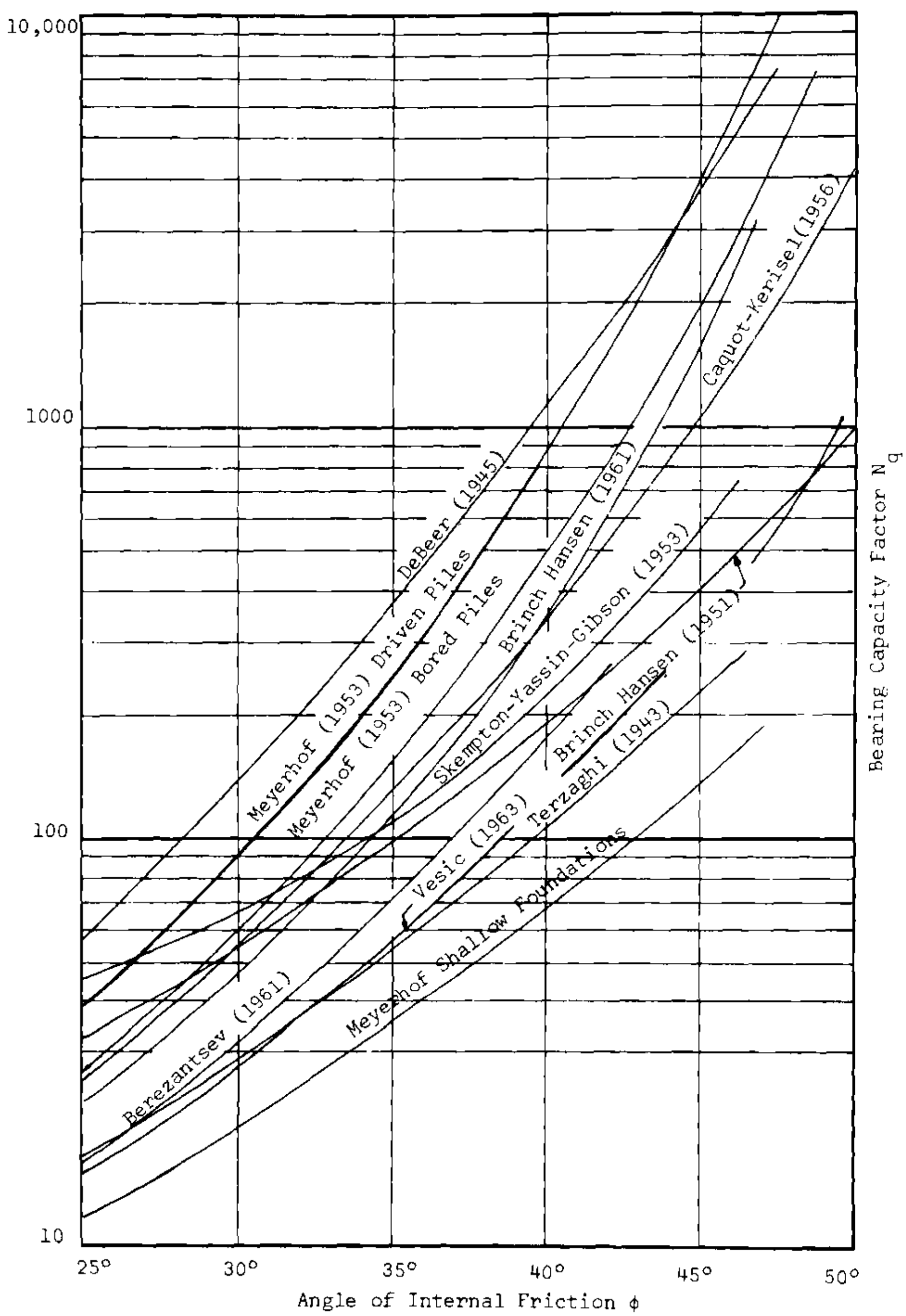

Figure 2-3. Bearing Capacity Factors for Circular Deep Foundations

It was upon this value of N_q that Norlund based his calculations. Norlund (10) considers the side friction to be a function of (a) ϕ, the friction angle of the soil; (b) δ, the angle of surface sliding friction; (c) ω, the taper of the pile; (d) γ, the effective unit weight of the soil; (e) D, the length of the pile; (f) C, the minimum pile diameter; and (g) V, the volume of displaced soil per unit length. His term K_δ included the variances of ϕ, δ, and ω; γ appears in the p_d term; and the term $C_d \Delta d$ is the area over which the side resistance acts. Norlund developed his term K_δ by first considering the hypothetical case of an infinitely long, nondisplacement wall. He hinged the wall at the bottom and rotated it about the hinge an angle θ. (See Figure 2-4.)

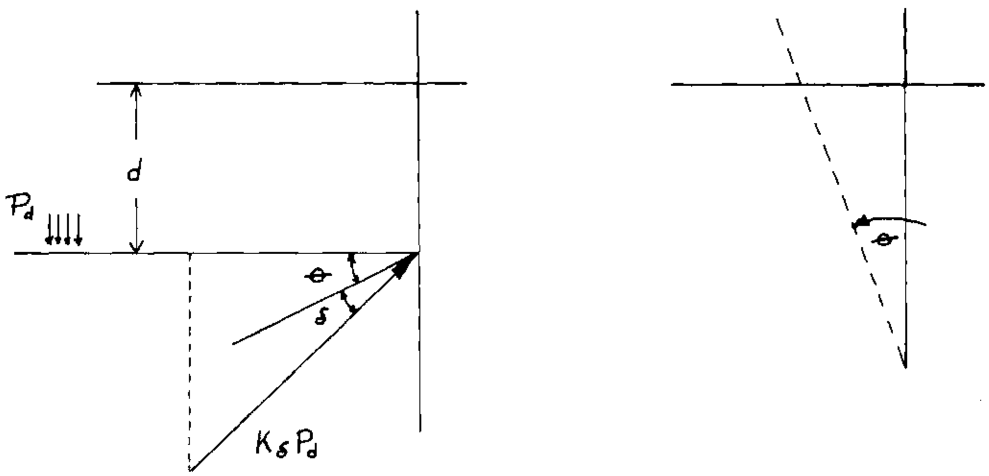

Figure 2-4. Relation Between Wall Movement and Horizontal Earth Pressure. From Norlund (10)

The K_δ had as a lower limit, the value when the wall underwent no rotation. For simplification he assumed $\delta = \phi$. The resulting value of K_δ was $K_o \sec(\delta)$ where $K_o = 0.5$. The upper limit of K_δ is the ultimate passive pressure and is obtained by rotating the wall θ degrees until the passive condition is reached. Values for K_δ for the upper limit have been calculated by Caquot and Kerisel (12). They are exact values but are good only for the condition $\phi = \delta$. Values of the relation of ϕ and K_δ were necessary. Using the results of tests by Terzaghi (13), Norlund approximated such a relationship as in Figure 2-5. Norlund extended his values of K_δ by determining correction factors for the cases when $\phi \neq \delta$. He then empirically extended his values of K_δ by adding more corrections which allowed for an increase of δ due to roughness of the pile surface, an increase of soil compaction due to the increase of the volume of the displaced soil per unit length, and finally a decrease in δ due to the increased soil compaction. His final results are in the form of a set of curves for different ϕ angles.

Sowers and Sowers (1) give in Table 2-1 typical values for the coefficient of lateral earth pressure, K, and indicate that K varies with the relative density of the sand and the method of placement of the pile. Model tests from Cornell by Silberman (4) also indicate that K is highly sensitive to the same two variables. According to Broms (14), Norlund's proposed method assumes K to be independent of the relative density. Broms believes that the sensitivity of K with respect to the relative density can be attributed to arching in the soil and further states that since Norlund uses 0.5 as the lower limit for K, then his method should produce safe side results.

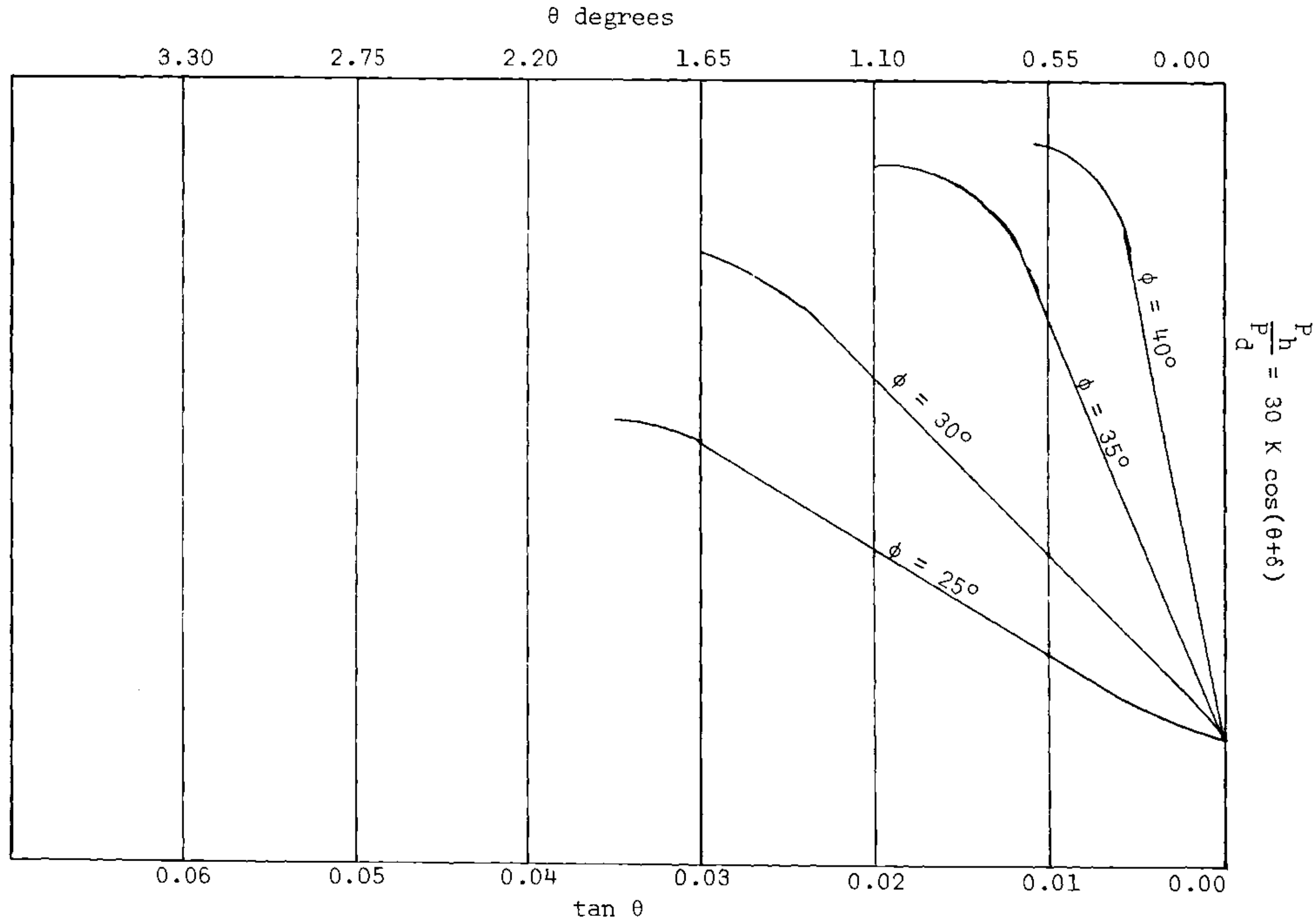

Figure 2-5. Terzaghi Relationship Between Wall Movement and Horizontal Earth Pressure. From Norlund (10)

Table 2-1. Typical Lateral Pressure Coefficients for Piles. From Sowers and Sowers (1)

Condition	K
Bored or jetted pile	0.5
Pile jetted, then driven a few feet	1
Pile driven in loose soil	1
Pile driven in dense soil	2-4

Summarizing, the capacity of a pile consists of two components--the end bearing and the side resistance. The conventional assumption of the two components acting independently was made for simplicity. The end bearing, Q_o, was obtained by reducing the general bearing capacity equation to simply

$$Q_o = q'N_q$$

Since the skin friction factor is very complicated and is a function of numerous variables, Norlund's development of Equation (2.14) was used as it allowed for the greatest number of these variables. Norlund used Berezantsev's 1961 N_q value to find the end bearing. The latest work by Broms (14) and Vesic (6) indicate that from actual tests, the skin friction is sensitive to the relative density because of the arching of the sand. They also point out that Norlund's proposed method assumes the skin friction to be independent of the relative density.

CHAPTER III

APPARATUS

Since the main purpose of this investigation was to study the lateral pressures a cohesionless soil exerts on a pile, a special pile was instrumented and used. The pile had to withstand the driving forces yet still have loading cells sensitive enough to record the small lateral pressures which the soil would exert on the sides. The pile used was the same as the one Shiver (15) used in a similar test at Georgia Tech in 1967. A modification was made to the base loading cell.

The test pile was a 4.5-inch outside diameter pipe pile with a 0.75-inch wall thickness. The pile consisted of three 4-foot sections and one 1.5-foot section at the foot of the pile. Each section was coated with epoxy and rolled in sand giving a rough surface to the pile and also making $\phi \approx \delta$. Special connectors containing the load cells for the lateral pressures were each 5.25 inches high giving a total pile length of 15 feet upon assembly. See Figure 3-1 for a diagram of the pile.

To measure the pressures on the pile, five independent acting load cells were used. Four cells were used to measure the lateral pressures and one cell was used for measuring the pressure at the base of the pile. The lateral pressure cells used were based on the design used by Shiver. See Figure 3-2 for a detailed cross-section of one of

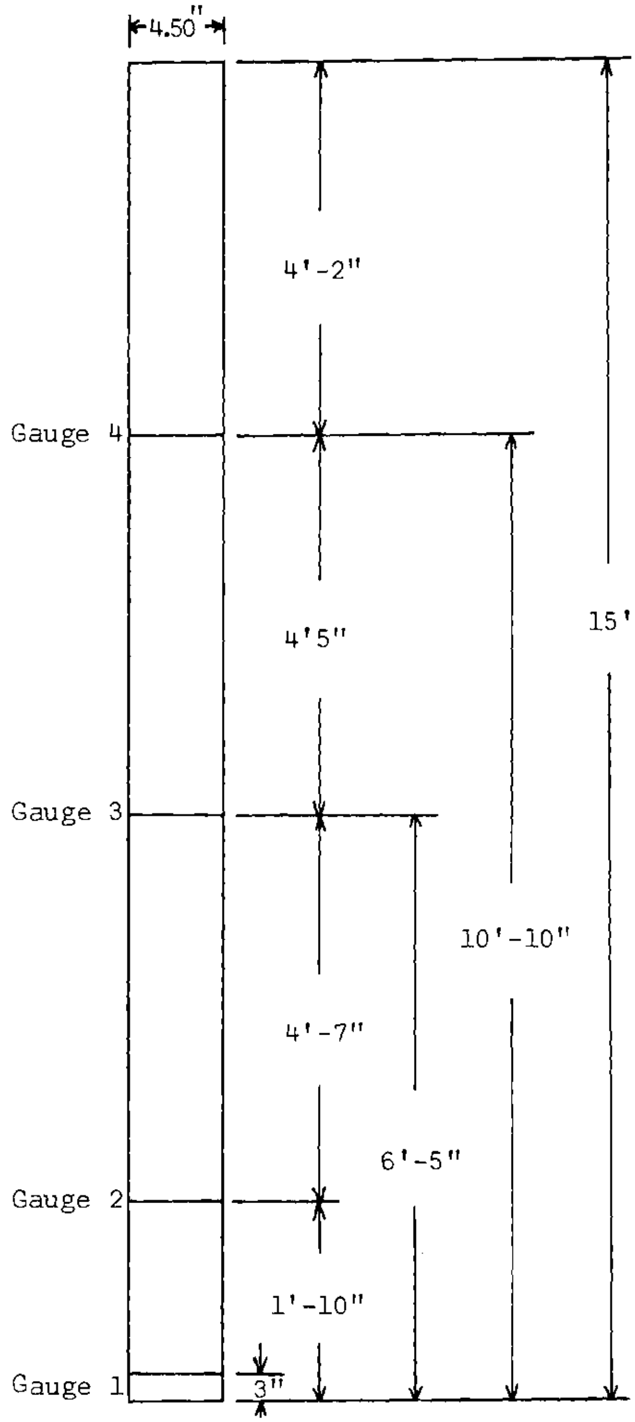

Figure 3-1. Pile Shaft Dimensions

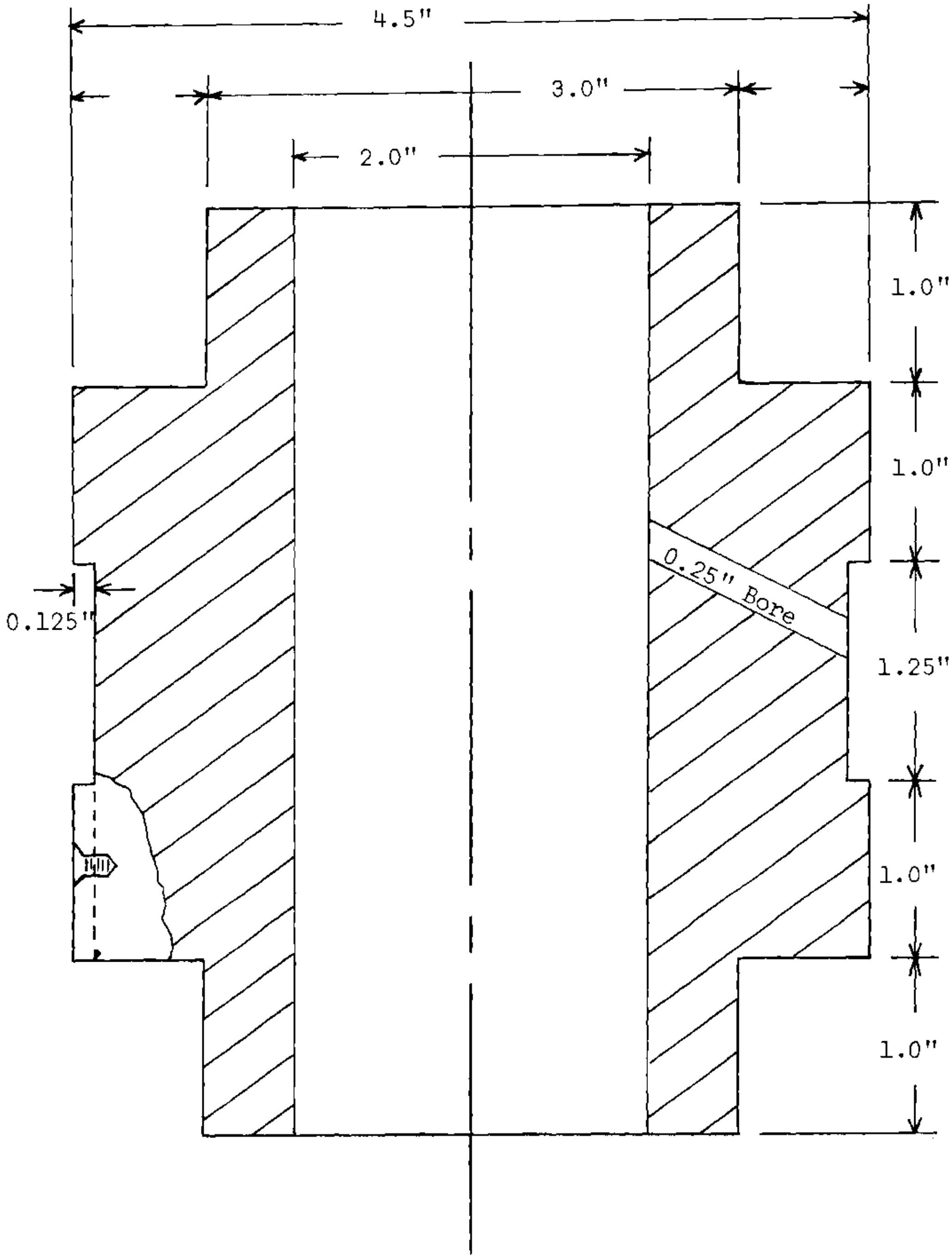

Figure 3-2. Lateral Pressure Load Cell and Connector

the lateral load cells and connectors. The cells were located on the section connectors of the pile. The actual pressure measuring device consisted of a thin, tubular aluminum ring with 0.8" Universal Precision strain gages mounted on the inside of it. The ring was machined to 4.5 inches outside diameter, an average wall thickness of 0.012 inches, and a height of 1.25 inches. Rings thicker than 0.012 inches proved to be too thick and not sensitive enough to measure the small lateral pressures. Four strain gages were cemented at 90 degrees and wired in series. See Figures 3-3 and 3-4. Each aluminum ring was then mounted on the load cell as in Figure 3-5. Since the cell also had to act as a connector for the pile sections, and since the pile was to be pulled out after driving, the connections had to withstand tension as well as large repetitions or compressive loads during driving. Allen set screws were used to connect the load cells and pile sections. Since the cells had to transmit the large driving force, the fragile, flexible aluminum rings had to be isolated from this large force. The aluminum rings, therefore, were floated on "O" rings to prevent the transmission of the driving force through the aluminum rings. The wires from the strain gages were threaded through a hole in the cell to the center of the pile. They were then run up and out the top of the pile to the strain indicator box. To prevent the wires from pulling loose due to whipping caused by the driving, the small holes in the cells were sealed with silastic 892-RTV adhesive sealant and then tied tightly to a special cap on the top of each load cell.

In order to measure the pressures on the base of the pile, Shiver's original load cell was modified. The same cell was used

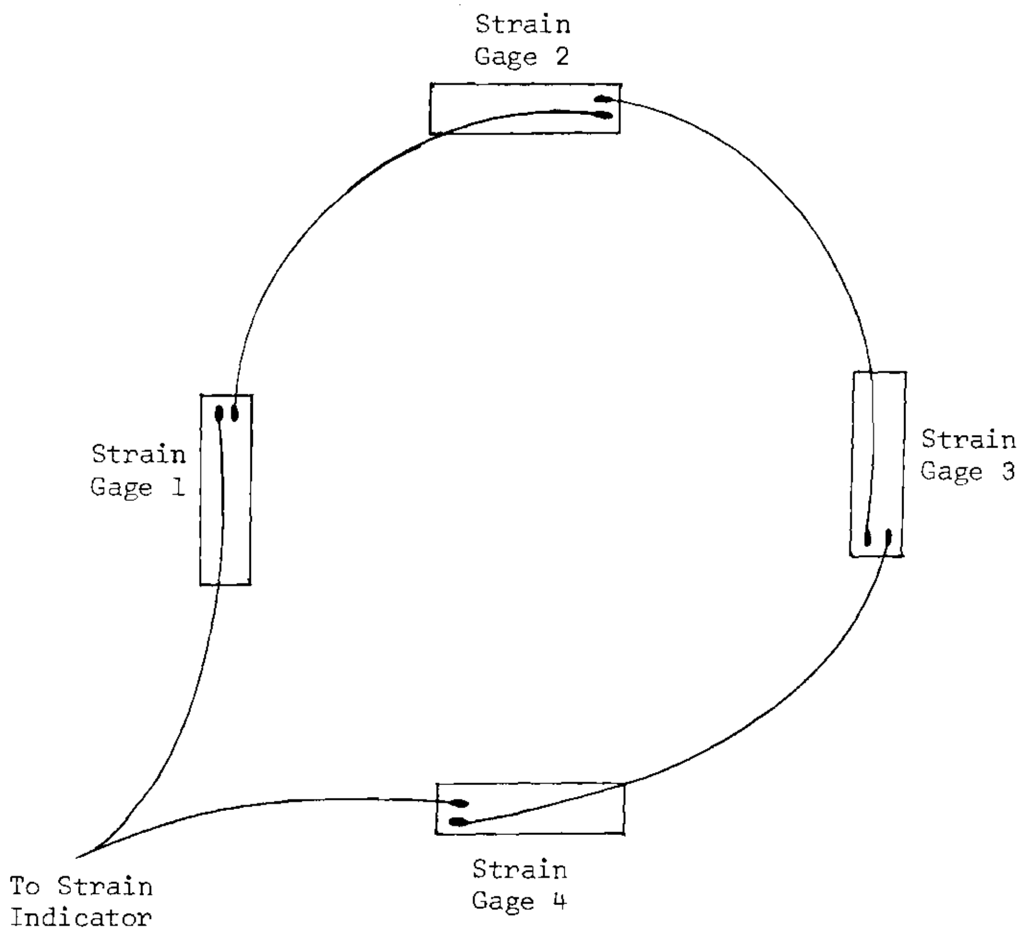

Figure 3-3. Wiring Diagram for Lateral Load Cells

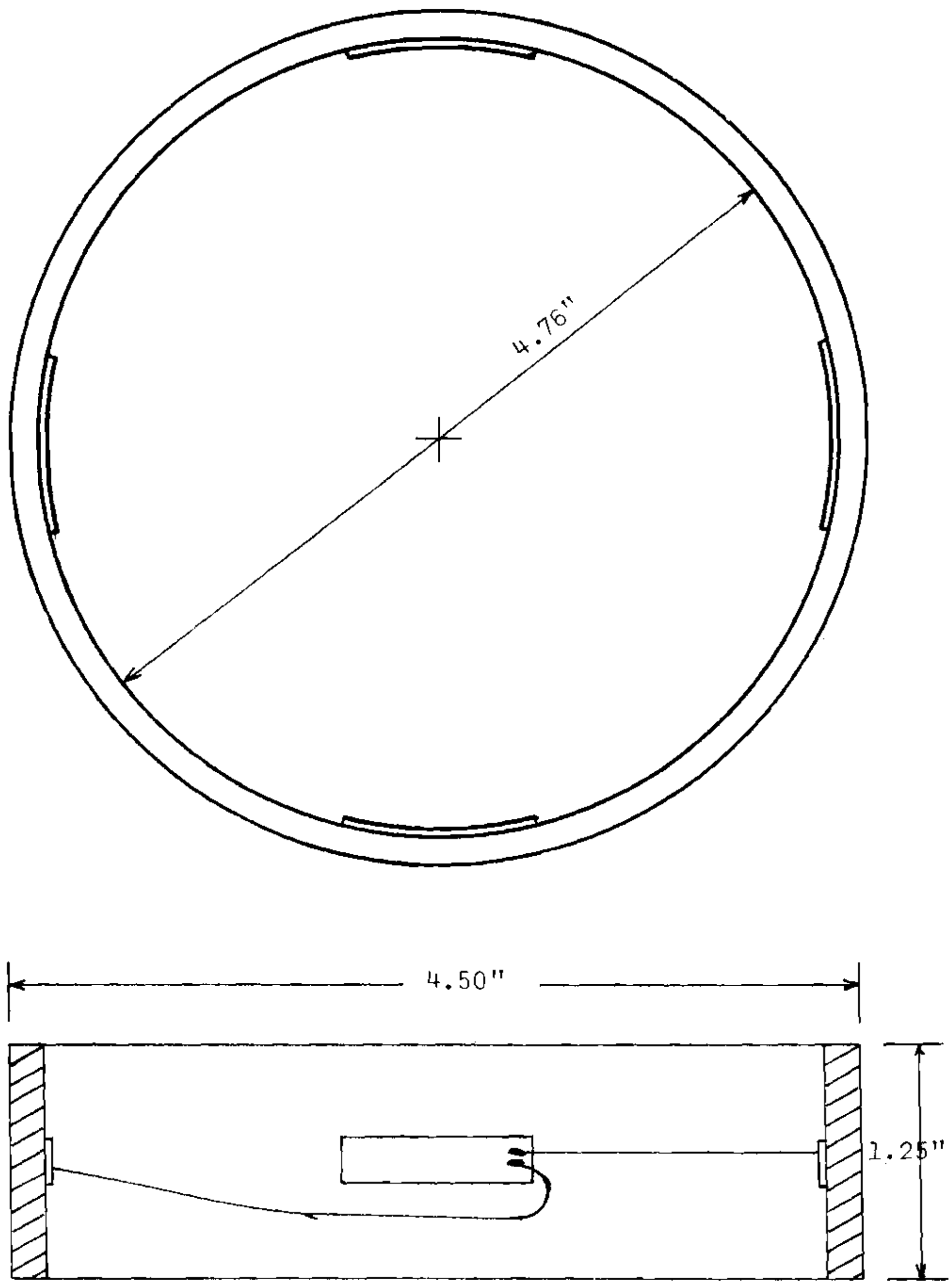

Figure 3-4. Strain Gage Locations on the Lateral Pressure Gages

except that the rods used to transmit the load around the proving ring at the bottom were omitted. Instead, a proving ring was designed strong enough to withstand the large dynamic driving forces, yet sensitive enough to measure the pressures on the base. After several unsuccessful pilot tests, a circular proving ring with an outside diameter of 3 inches, a width of 1.125 inches, and a wall thickness of 0.1875 inches was found to work satisfactorily. The ring was machined from a solid, 3-inch diameter cylinder of 150,000 psi yield steel. Lower yield steels were tried but all unsuccessfully. Two strain gages were cemented to the inside of the ring at 180 degrees from each other and wired in series. The wires were run up the center of the pile and out the top to a strain indicator. The proving ring was fastened to both parts of the load cell and acted as a connector when the pile was pulled out. See Figure 3-5 for a detailed drawing of the base load cell. The base load cell also contained a lateral pressure ring like the others described.

The test pile was driven into a dry cohesionless sand in a special pit that was 8 feet and 4 inches in diameter and 22 feet deep. The sand was originally obtained from the Chattahoochee River near Atlanta, Georgia. It was the same sand used by Shiver in his 1967 tests (15) and Vesic (16) in his 1963 tests at Georgia Tech. Vesic made a study of the properties of the sand. Basically the sand was composed of sub-angular quartz particles with some mica. It was classified as a medium uniform sand. Tables 3-1 and 3-2 give the maximum and minimum densities as well as the shear strength values determined by Vesic (16).

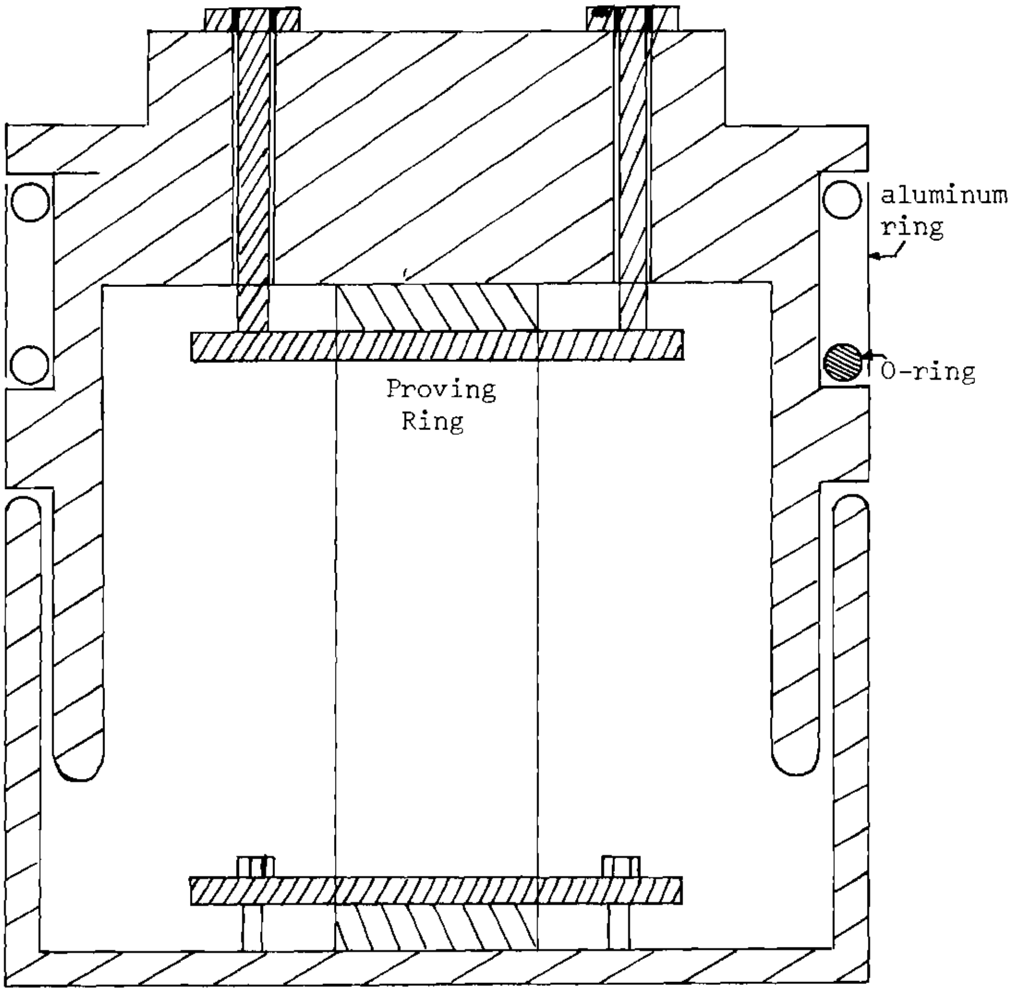

Figure 3-5. Base Plate Load Cell

Table 3-1. Maximum and Minimum Densities of Chattahoochee River Sand. From Vesic (16)

Density	Dry Unit Weight pcf	Void Ratio e	Porosity n
Minimum	79.0	1.10	52.4%
Maximum	102.5	0.615	38.1%

Table 3-2. Shear Strength Characteristics of Chattahoochee River Sand. From Vesic (16)

Angle of Internal Friction ϕ, in Degrees		Void Ratio e	Relative Density D_R in Per Cent
σ_3<10psi	10<σ_3<80psi		
-	44.3	0.6	103.0
44.3	40.0	0.7	82.5
40.5	36.2	0.8	61.8
36.8	33.5	0.9	41.2
34.0	31.0	1.0	20.6
31.9	-	1.1	0.0

The density of the sand in the pit was determined with a penetrometer. The penetrometer consisted of a 0.375-inch diameter steel rod with a pointed end of 0.5 square inches in area. A 3000-pound hydraulic jack and proving ring was used to push the penetrometer into the sand and determine its resistance. To move the sand, a mechanical digger and conveyor belt was used. An 8-foot diameter #10 sieve was used to pass the sand through on refilling the pit in an attempt to get the most homogeneous and uniform density as possible.

The pile hammer was a simple free-drop hammer consisting of 500 pounds of lead weights. The hammer was guided by 20-foot metal leads which were bolted to a large A-frame. A winch was used to raise the hammer. A special head for the pile was made that enabled the gage wires to be brought out the top without damage during the driving.

CHAPTER IV

PROCEDURE

Since Shiver had run similar tests on a full scale pile, the construction of equipment was minimized. However, since the loading cell at the base of the pile was modified, several pilot tests were necessary. The same proving ring that Shiver used was tried first. The dynamic forces of the driving exceeded the elastic range of the proving ring. No usable data were acquired. Several different pilot tests were made varying the yield strength of the steel, the height of the hammer drop, and the size of the ring. Being limited to a small space, the size of the ring could be altered very little without losing the needed sensitivity. The final proving ring used was of 150,000 psi steel, 3 inches outside diameter, 1.125 inches wide, and 0.1875 inches thick. A hammer of 500 pounds of stacked lead weights was dropped 30 inches, exerting 1250 foot-pounds of energy per blow on the pile.

New aluminum rings for measuring the lateral pressures were constructed and each instrumented with four strain gages wired in series. During the calibration of the rings, it was discovered that they were too thick and consequently too insensitive to measure the small lateral pressures. Therefore, the rings were remachined, new gages were attached, and the system calibrated again. The inside of each ring was first cleaned with acetone to assure a good bond of the gages to the ring. The four strain gages were then cemented to the ring with Duco cement. The longitudinal axes of the gages formed a horizontal plane. The gages were wired in series so any deflections would be additive and increase the sensitivity of the rings. The gages were next covered with a rubber sealant, as were all bare wire connections in order to prevent the gages from shorting out. Each ring was floated on "O" rings on its respective loading cell. Both the bottom edge and the top edge of the ring was filled with the rubber sealant. The sealant had the dual purpose of protecting the gages by precluding the entry of sand into the ring during driving and to make an airtight seal to allow calibration by air pressure. For this latter purpose, the hole for the gage wires was also sealed. After the pressure cells were assembled, they were calibrated in a steel cylinder two feet in diameter and one foot high. Two rubber gaskets were placed around the top edge between the cover and the cylinder to prevent air leakage and loss of pressure. Air pressure was applied in one psi increments through a regulator and Bourdon Gage. The readings were recorded from a strain indicator box. Each of the four lateral pressure rings were calibrated separately in this manner. A minimum of three calibrations were made for each ring to minimize error and assure more consistent results. Figures 4-1, 4-2, 4-3, and 4-4 are the calibration curves of indicated strain versus pressure for the lateral load cells.

Next the ring that was to be used to measure the vertical pressures on the base of the pile was calibrated. After cleaning the ring, two strain gages were cemented to the inside of the ring and wired in

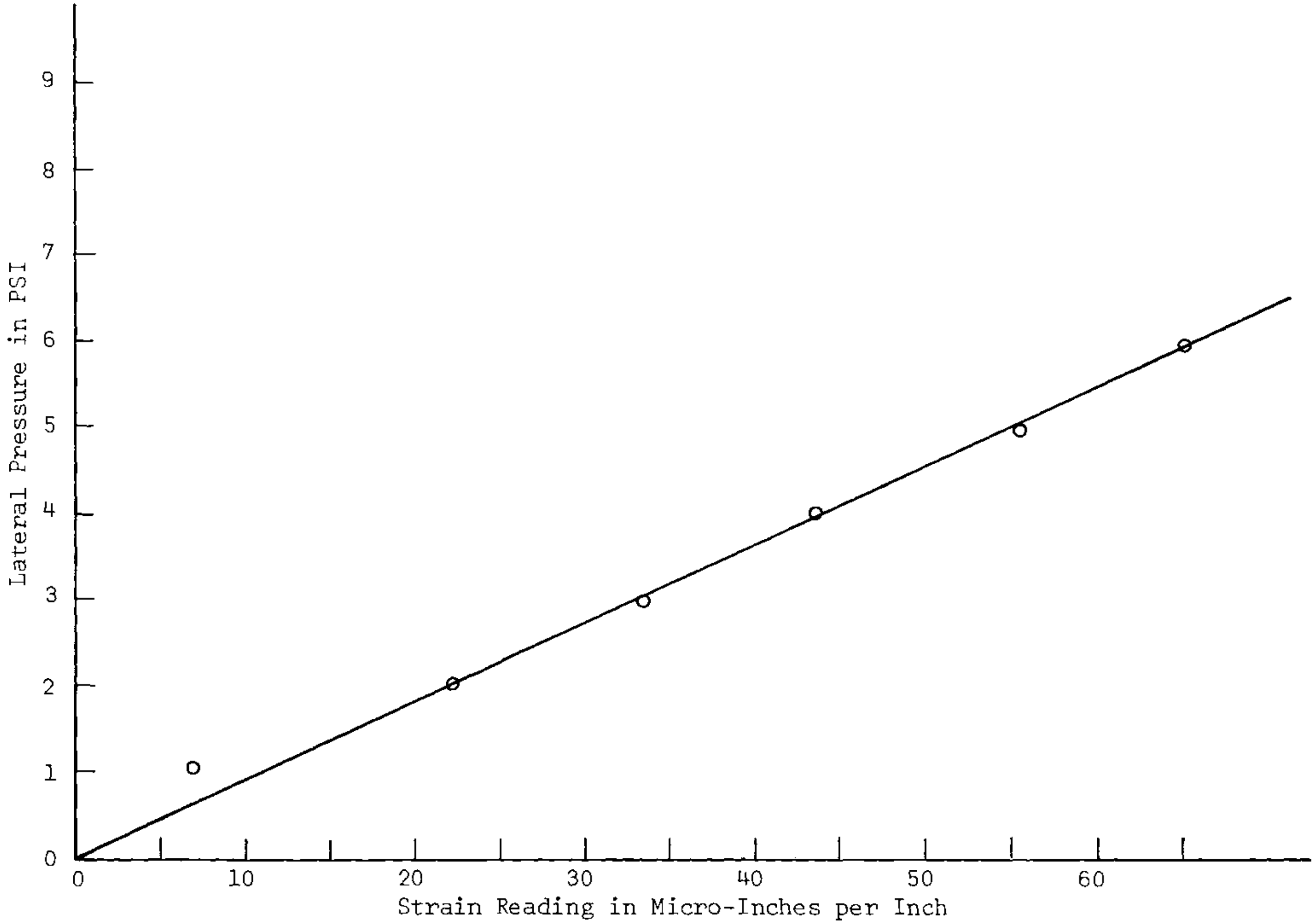

Figure 4-1. Gage 1--Lateral Load Cell Calibration

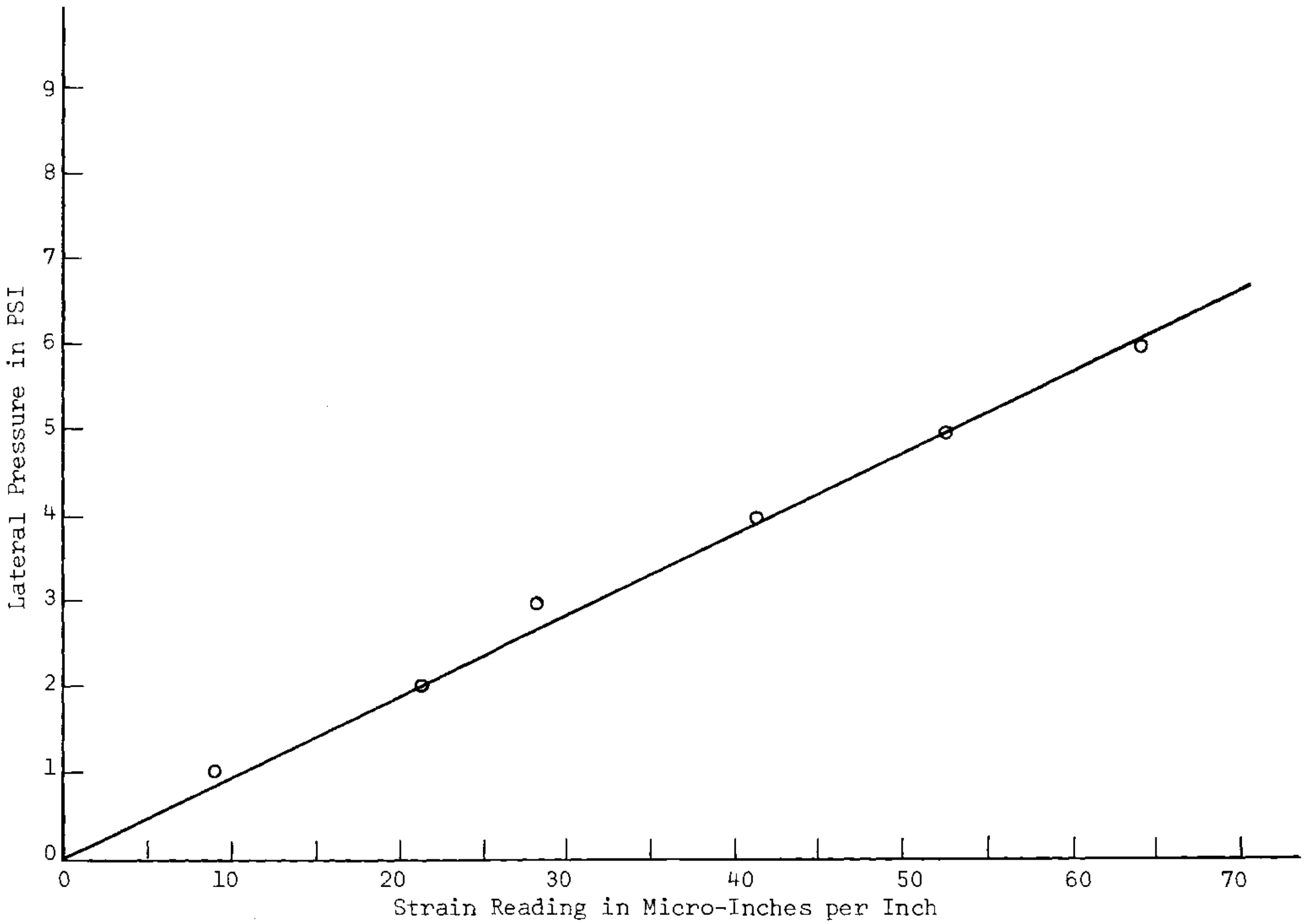

Figure 4-2. Gage 2--Lateral Load Cell Calibration

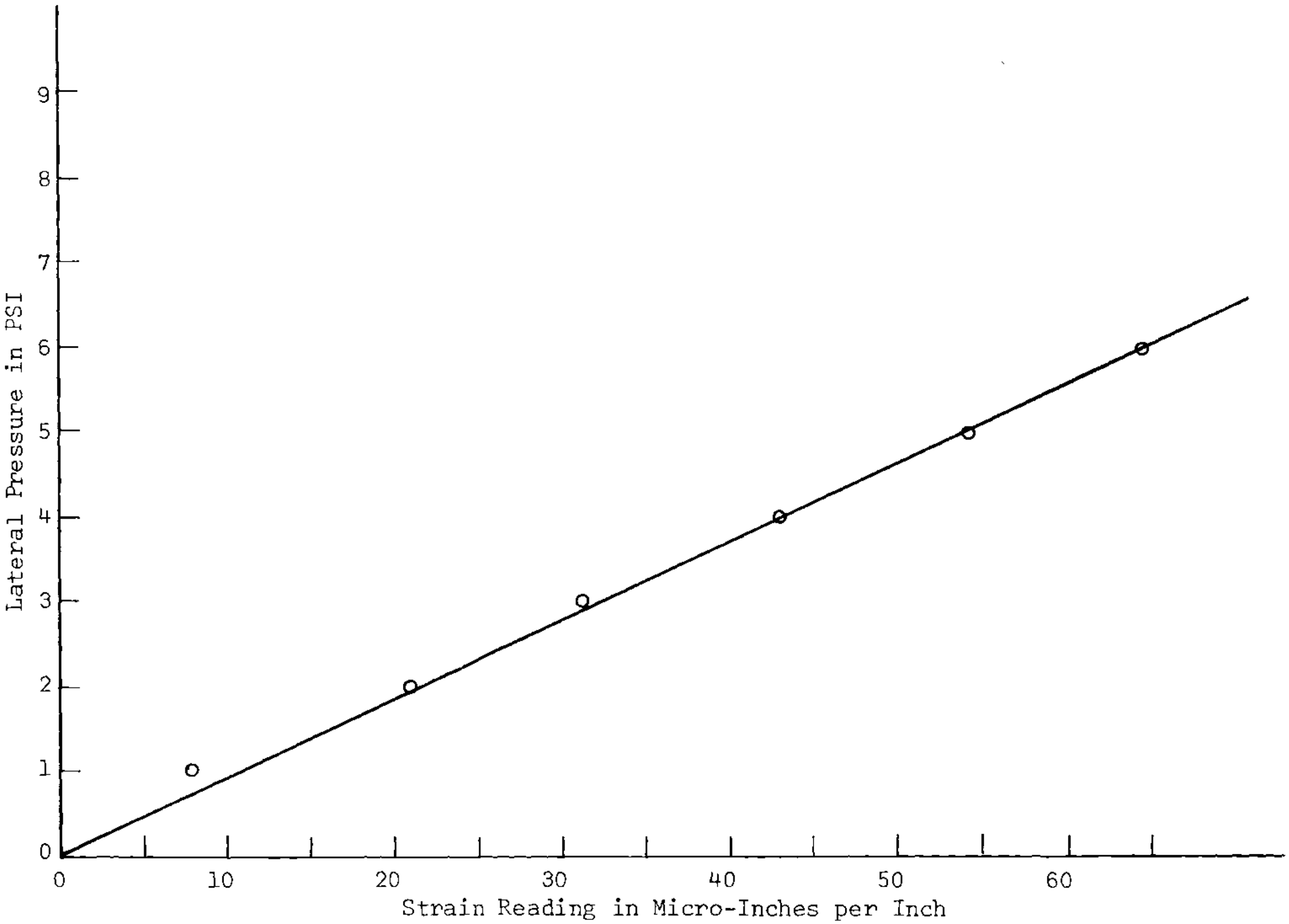

Figure 4-3. Gage 3--Lateral Load Cell Calibration

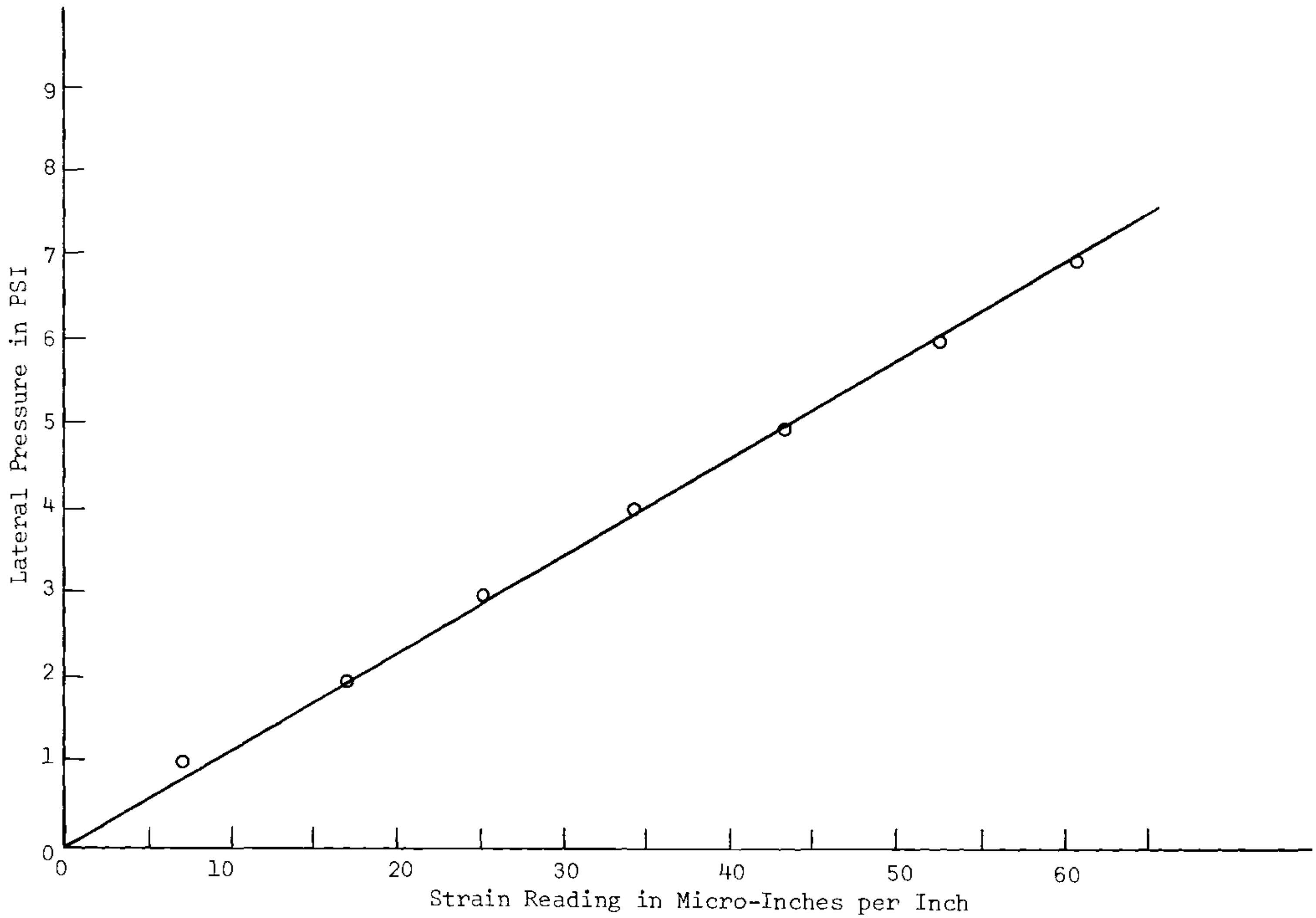

Figure 4-4. Gage 4--Lateral Load Cell Calibration

series. The gages were located 180 degrees from each other, half way up each side, with the gages having their longitudinal axes perpendicular to a horizontal plane. Load was placed on the ring in 2000 pound increments with a 20,000 pound controlled displacement rate testing machine. The curve of load versus strain reading for the ring is in Figure 4-5.

The last ring to be calibrated was the loading ring for the penetrometer tests. The loading ring consisted of a steel ring with a 0.001 inch micrometer fixed on the inside to measure deflections. Calibration was made by placing dead loads in ten-pound increments until a load of 200 pounds was achieved. Results are presented in Figure 4-6.

Before driving the pile, the pit had to be emptied and the sand replaced in a way so that it would be homogeneous and of uniform density. The leads of the drop hammer were removed so a mechanical digger could be lowered into the pit. The leads were pulled up with the winch and tied off with chains to the large A-frame. The mechanical digger, held up by the winch, was moved over the pit and lowered as it removed the sand. A conveyor belt was rigged so that the sand from the digger would be caught and transported to a storage bin behind the pit. The digger was able to remove the sand to a depth of 15 feet. A minimum of five hours digging time was required to empty the pit.

To fill the pit so that the sand would be as homogeneous and uniform density as possible, an eight-foot diameter, No. 10 sieve was lowered into the pit. The sand was moved back to the pit with the

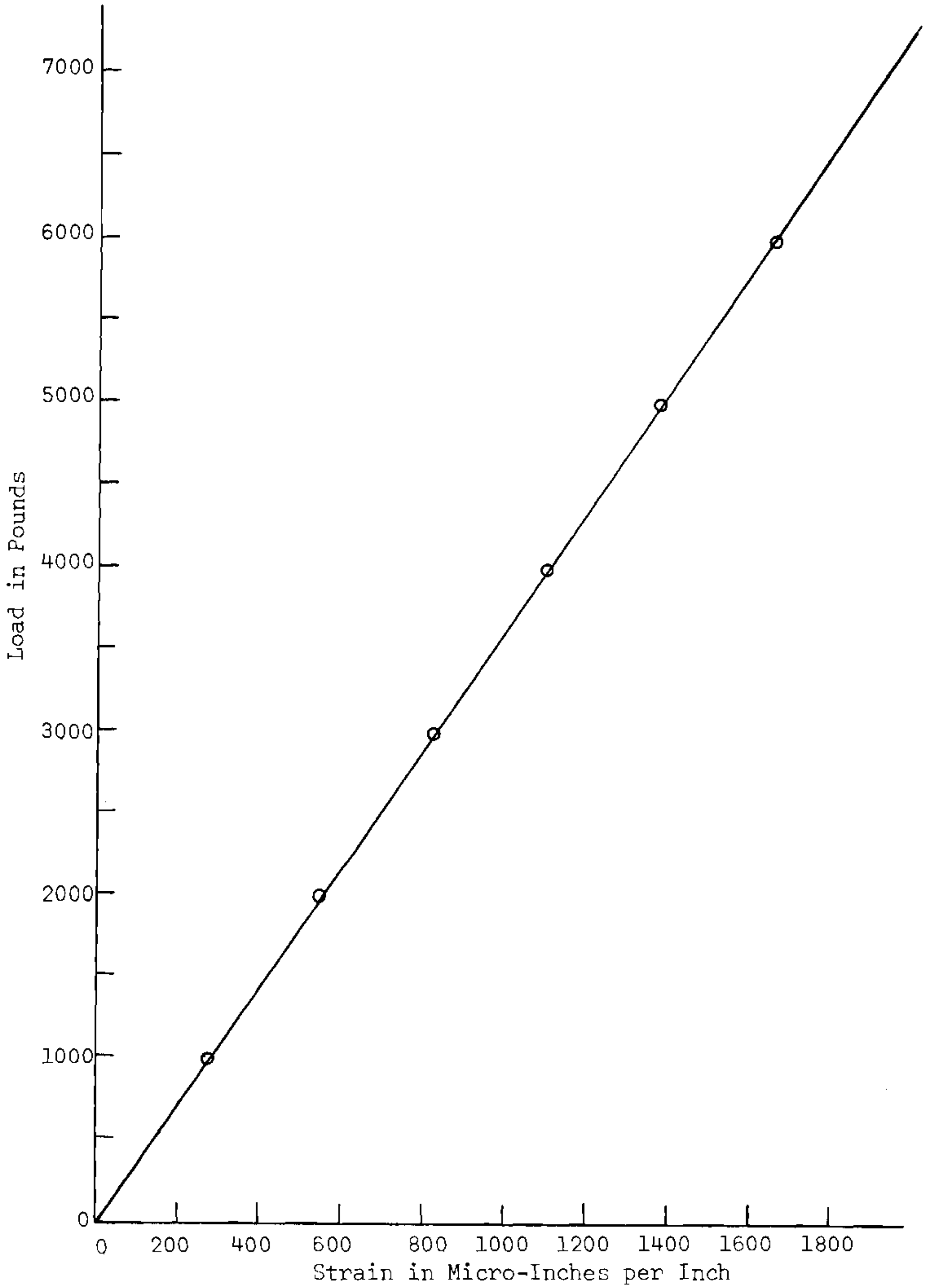

Figure 4-5. Base Plate Proving Ring Calibration

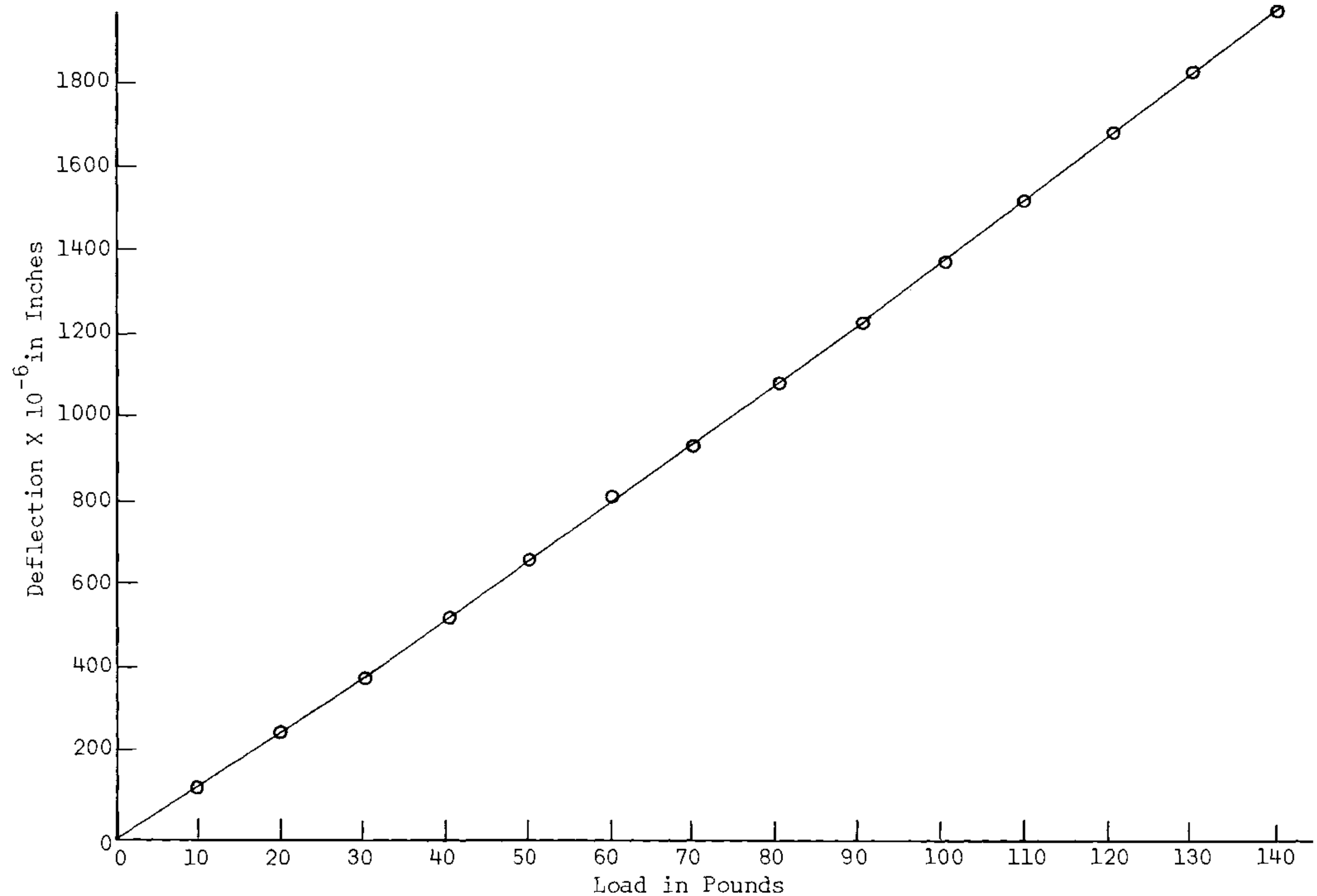

Figure 4-6. Penetrometer Proving Ring Calibration

conveyor belt. The belt was placed under a chute in the side of the bin. A plastic hose was attached to the other end of the conveyor belt and was used to spread the sand onto the sieve and disperse it evenly, producing a more uniform density. The sieve was raised as the sand level raised, always keeping a two-foot difference between the two. By placing the sand carefully in this way, a reasonably homogeneous medium of uniform density was obtained. Filling the pit correctly was very important and required three days to complete.

Cone penetrometer tests were then made to determine the density of the sand. The 1000 pound crane was moved over the pit and a steel frame was attached such that it could swing a three-foot radius circle. A 12-foot long, 3000-pound hydraulic ram was connected to the frame. Between the piston of the ram and the penetrometer rod was placed the calibrated proving ring. Air pressure was applied to the bottom of the ram in order to keep the rod up. A greater air pressure was applied to the top of the ram, forcing the penetrometer rod down into the sand after overcoming the initial starting friction. Using the air pressure allowed close control over the rate of movement of the penetrometer rod into the sand. The maximum reading on the proving ring was recorded and used to determine the actual pressure on the tip of the penetrometer rod. The rod was marked at one-foot intervals and readings were made at these one-foot intervals. Penetrometer readings were made in four different areas and then the average of these values was used in determining the density from the curves of Vesic (16) shown in Figure 4-7. As reported later in Table 5-2, the densities varied with depth since the sand was not homogeneous. However, an

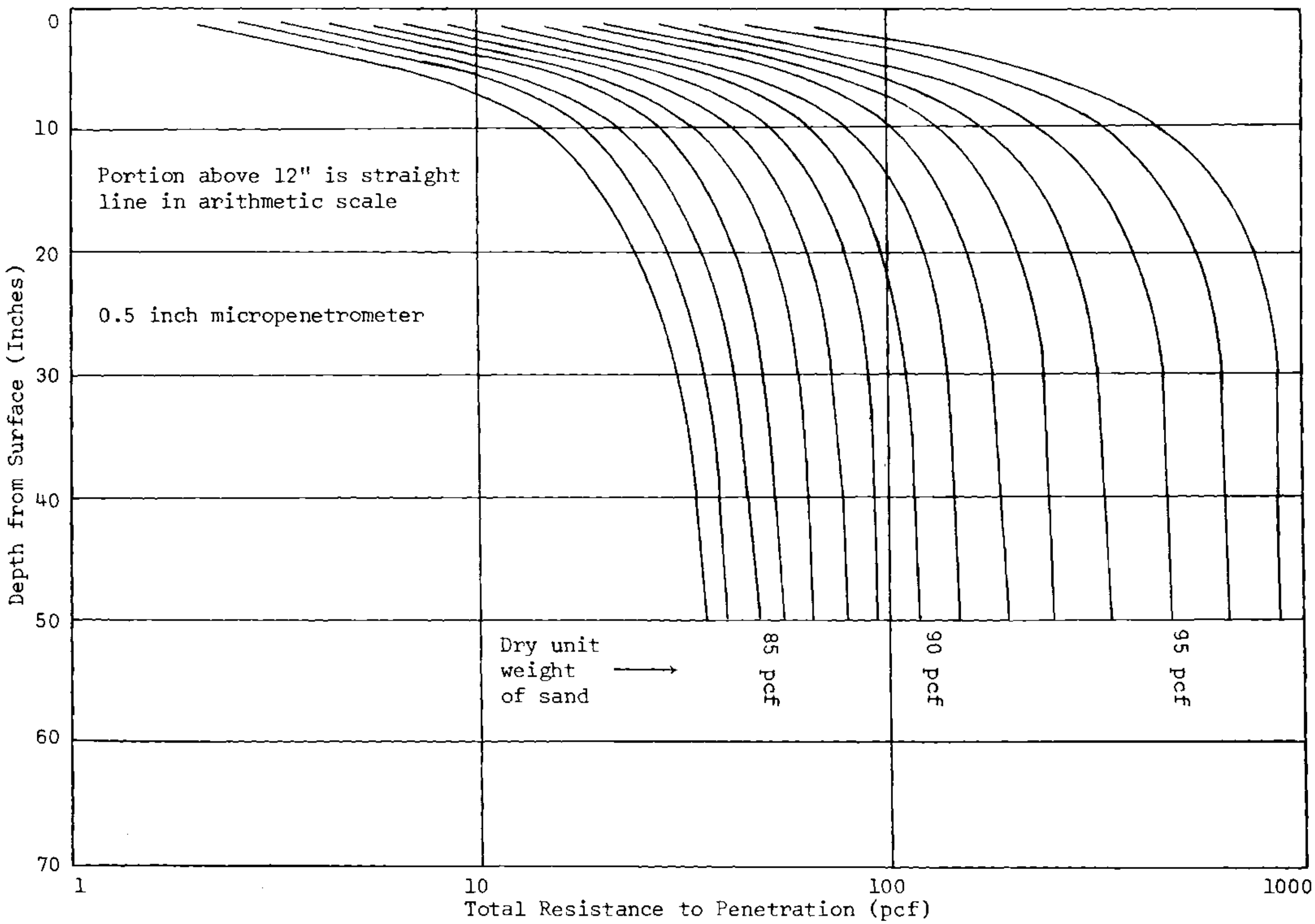

Figure 4-7. Relationship Between Depth and Total Penetration Resistance for Different Sand Densities. From Vesic (16)

average density was determined from these varying densities and used for all calculations. The average density was then assumed to exist uniformly throughout the medium. The average density found for these tests were all low, with the relative densities varying from 21 percent to 55 percent.

After the pit was filled properly and the average density determined, the pile was ready to be driven. The first two sections were connected and centered between the leads with the winch. The problem of the gage wires being whipped around during driving and being pulled loose from the strain gages was alleviated by tying them to the top of each pile connector with string. The weight of the wires running up the center of the pile was also removed from the thin connection wires by this tying. Zero readings were taken on both the lateral cells and the cell measuring the force on the bottom of the pile before the pile was lowered into the sand. These sections of the pile were then driven into the sand with 30-inch drops of the hammer. The third section of the pile was added; the zero reading was made on the lateral ring; and then the pile was driven. The fourth and last section was added and treated similarly.

When the pile tip was 8 feet deep, the driving was stopped. The hammer was raised above the pile and chained tightly to the metal leads. A 20-ton hydraulic ram was placed on top of the pile and jacked against the hammer until the pile failed. At the time of failure, readings were taken on all gages and recorded. A Bourdon Gage registered the pressure in the ram and consequently the load being applied to the pile by the ram. The same procedure was repeated after the pile had been driven to a depth of 13 feet.

After the final driving and after all readings were made, the pile was immediately pulled out using the winch. Each section was carefully removed as it was pulled out and rezero readings were made to check the gages.

CHAPTER V

TEST RESULTS

The purpose of these pile tests was to determine the pressures exerted on a rough, 4.5-inch diameter, cylindrical, steel pipe pile by a dry cohesionless sand after driving. The common way of determining the pressures on a pile is to load the pile to failure measuring the pressure at the tip and the force required to fail the pile. The difference between these two loads is considered to be the skin friction force or side resistance of the pile. These tests were an attempt to determine, by direct measurement, the lateral forces on the pile and hence allow a direct determination of the skin resistance. Since instrumentation for such a test requires a full-scale pile test, which has proved to be very costly both in time and money, little work has been done in trying to make such direct measurements. Most research concerning piles has been model studies. As a result, very little data is available and very little is known about the true lateral pressures on a pile.

Three piles were driven and tested, each in a soil of a different density and at depths of 8 feet and 13 feet. The sand properties for each test are given in Table 5-1. Since the sand was the same sand that Vesic used in his 1963 tests (16), the soil properties of the sand that he determined were used and are listed in Tables 3-1 and 3-2. Figure 4-7 shows the relationship between depth and total

Table 5-1. Sand Properties

Test Number	Average Density, in PCF	Average Relative Density, D_R, in Per Cent	Angle of Internal Friction, ϕ, in Degrees
1	92	55	39.2
2	84	21	34.2
3	86	30	35.5

penetration resistance for different sand densities. Vesic obtained this relationship by sounding sand models in a 24 by 16 by 60-inch box placed on a scale and filled the same way the large pit was filled. These curves were used to determine the density of the sand.

After determining the shear strength characteristics of the sand by a total of 54 standard triaxial tests on air-dry, 2.8-inch diameter, 6-inch high samples, Vesic derived the following approximation for the angle of internal friction, ϕ, as a function of the void ratio, e, when the confining pressure was less than 10 psi:

$$\tan \phi = \frac{0.68}{e} \tag{5.1}$$

He assumed the Mohr-Coulomb criterion of failure to be valid in his derivation. Since the lateral pressures measured on the pile were all less than 10 psi, Equation 5.1 was used to determine the values of ϕ for the tests in this report.

In tests such as these, the determination of the in situ sand density and the ϕ angle was very important. Figure 2-3 shows the sensitivity of the bearing capacity factor, N_q, with respect to ϕ. After the average in situ density was approximated from the penetrometer tests, the void ratio was determined by interpolation of Table 3-1. The ϕ angle was then approximated from Equation (5.1). Measuring in situ density by the penetrometer is unfortunately at best an approximation but was the best method available for these tests. Technical Report R-571 of the Naval Civil Engineering Laboratory (17) gives an empirical correlation between ϕ and the vane resistance of sand and uses this relationship to determine ϕ. It, too, is only an approximation of the ϕ value, however. Part of the disagreement between the calculated values and the measured test results may be due to errors in the determination of the sand density and hence the ϕ angle. For example, using Meyerhof's constants in Test Series No. 1, an error of $\pm$ 1 degree in the true ϕ would result in the theoretical pile tip bearing capacity at a depth of 13 feet to be from 63,000 pounds to 119,000 pounds. Assuming the ϕ angle was measured correctly, the value would be 91,000 pounds. Berezantsev's constants are more conservative but are similar in sensitivity to changes in ϕ.

Test Series No. 1

Eight-Foot Pile

Using the penetrometer results from Table 5-2 and Figure 4-7, the average in situ density for Test Series No. 1 was found to be 92 pcf. The densities varied from 91.7 pcf to 92.3 pcf.

Table 5-2. Penetrometer Test Data

Depth (ft)	Test Series No. 1 Resistance (psi)	Test Series No. 1 Density (pcf)	Test Series No. 1 D_R (%)	Test Series No. 2 Resistance (psi)	Test Series No. 2 Density (pcf)	Test Series No. 2 D_R (%)	Test Series No. 3 Resistance (psi)	Test Series No. 3 Density (pcf)	Test Series No. 3 D_R (%)
1	112	92.0	55.3	-	-	-	-	-	-
2	158	92.0	55.3	-	-	-	40	85.5	27.6
3	158	91.7	54.0	30	82.7	15.7	45	85.3	26.8
4	217	92.3	56.3	42	84.0	21.2	60	86.3	31.0
5	230	92.3	56.3	59	86.0	29.7	70	87.0	34.0

The average relative density, D_R, was found to be 55 per cent and the angle of internal friction, ϕ, was determined to be 39.2 degrees from Equation (5.1). The measured and theoretical end bearing forces and side resistances are given in Tables 5-3 and 5-4.

Table 5-3. Theoretical Values of End Bearing and Side Resistance

Test:		Series 1		Series 2		Series 3	
Average Relative Density		55%		21%		30%	
Pile Length in Feet		8	13	8	13	8	13
Norlund	Pile Capacity in pounds	16600	32200	5080	10280	6040	12110
Meyerhof (deep)	End Bearing in pounds	56000	91000	14800	24000	21200	34400
Berezantsev		11350	18400	3120	5050	3780	6150
Meyerhof (shallow)		5030	7650	2370	3840	2700	4430
Norlund	Side Resistance in pounds	5250	13800	1960	5180	2260	5960
Equation 2.12 with K=1		2830	7460	2150	5680	2320	6120

The force on the tip of the pile was measured to be 2,900 pounds. Using Meyerhof's N_q value for deep circular foundations given in Figure 2-3, a base force of 56,000 pounds was calculated. This calculated base force was much larger than the measured base force of 2900 pounds. Using the N_q Berezantsev proposed in 1961, Norlund's Equation (2.14) yields a lower but still high value of 11,350 pounds for the base force.

Table 5-4. Measured Test Results of End Bearing and Side Resistance

	Test Series No. 1		Test Series No. 2		Test Series No. 3	
Average Relative Density	55%		21%		30%	
Pile Length in Feet	8	13	8	13	8	13
Pile Capacity in Pounds	5200	7630	4330	6920	3810	10050
End Bearing in Pounds	2900	4550	2700(?)	3600(?)	2710	6020
Side Resistance in Pounds	2300	3080	1630(?)	3320(?)	1100	4030

For failure of the pile, a total load of 5,200 pounds was required. By subtracting the base force from the pile failure load, the total side resistance of 2,300 pounds was obtained which means that 44.3 per cent of the pile capacity was carried by skin friction. For comparison, two theoretical calculations of the side resistance were made. Using Equation (2.12) which uses a triangular lateral pressure distribution, and assuming $\delta = \phi$ and K = 1.0, a Q_f of 5,680 pounds was calculated. The angle δ was assumed equal to ϕ since the pile was coated with epoxied sand. For a loose sand, Sowers and Sowers (1) give 1.0 as a typical K value. On this basis, a K value of 1.0 was assumed in the theoretical calculations. Using the second half of Norlund's Equation (2.14), again assuming $\delta = \phi$, a Q_f of 5,250 pounds was obtained. These theoretical values of skin friction were high as were the end bearing values, but not to the same extreme.

Utilizing a triangular distribution of the lateral pressures on the side of the pile, the average K value necessary to give the measured side resistance of 2,300 pounds was calculated to be 0.813. The measured pressures and all K values for all tests are given in Tables 5-5 and 5.6.

Table 5-5. Measured Lateral Pressures

8 Foot Pile				13 Foot Pile			
Depth Ft.	Lateral Pressure PSI			Depth Ft.	Lateral Pressure PSI		
	Test Series No. 1	Test Series No. 2	Test Series No. 3		Test Series No. 1	Test Series No. 2	Test Series No. 3
	D_R=55%	D_R=21%	D_R=30%		D_R=55%	D_R=21%	D_R=30%
1.5	-	-	-	2.0	5.80 (?)	1.80	-
6.0	-	2.90	5.53	6.5	-	-	-
8.0	5.15	7.60	2.90	11.0	-	6.30	-
				13.0	6.44	8.20	5.27(?)

Because of equipment failure, only two lateral pressures were measured. These pressures were measured as 5.53 psi at a depth of 6 feet and 2.9 psi at 8 feet. Assuming that the lateral pressure and the overburden pressure, γz, were related by K during loading, the K for the lateral pressure of 5.53 psi was 1.08. The 2.9 psi lateral pressure seemed too low and is of questionable validity since it is equivalent to a K of only 0.566.

Table 5-6. Comparison of Theoretical and Calculated Lateral Pressure Coefficients

	Pile Length Ft.	Gage Depth Ft.	Test Series No. 1 D_R=55%	Test Series No. 2 D_R=21%	Test Series No. 3 D_R=30%
Sowers and Sowers (1)			1	1	1
Norlund (10)			1.1	1.2	2.0
Average K from Equation (2.12) and Measured Side Resistance	8		0.813	0.758	0.475
	13		0.414	0.584	0.660
K Calculated from Measured Lateral Pressures	8	6	1.08	-	0.81
	8	8	0.566	1.1	1.59
	13	2	-	5.04(?)	1.51
	13	11	-	-	0.96
	13	13	0.635	0.848	1.06

Thirteen-Foot Pile

The end bearing force for the 13-foot pile was measured as 4,550 pounds. The load for failure was 7,630 pounds which means 3,080 pounds or 40.4 per cent of the total load was due to side resistance. Again the tip load was compared to the theoretical values using Meyerhof's and Berezantsev's N_q values from Figure 2-3. Meyerhof's value was 91,000 pounds and Equation (2.14) of Norlund using Berezantsev's N_q produced a theoretical end bearing of 18,400 pounds. As before, these theoretical end bearing values were all high as compared to the measured end bearing values and thus on the unsafe side.

Using Equation (2.12) which assumed a triangular lateral pressure distribution and the second half of Norlund's Equation (2.14), Q_f values were computed to be 7,460 pounds for Equation (2.12) and 13,800 pounds for Norlund's Equation (2.14). The Q_f from Norlund's equation was about twice the Q_f value found from Equation (2.12) since K was assumed to be 2.0 by Norlund's method and only 1.0 in Equation (2.12). Regardless of these differences, both Q_f values were far in excess of the 3,080 pounds measured.

In driving to the 13-foot depth, another of the remaining lateral pressure gages was damaged. A lateral pressure of 5.27 psi was measured at a depth of 13 feet. The average K value for this pressure, assuming again it to be related to the overburden stress, was 0.635. Assuming a triangular lateral pressure distribution, the average K value necessary to give the measured side resistance of 3,080 pounds was calculated as 0.414. Possibly the measured side resistance was too low. There was a possibility that the pile was not loaded to failure and if such was the case, the side resistance would be too low since it was considered to be the difference between the end bearing of the pile and the total pile capacity. This possibility of the pile not failing was further supported by the relatively low end bearing and failure forces that were measured when compared to the results of Test Series No. 2 and 3 in Tables 5-4 and 5-5.

Test Series No. 2

Eight-Foot Pile

The density of the sand for Test Series No. 2 was determined to be 84 pcf from the combined use of the penetrometer results in Table 5-2 and the curves in Figure 4-7. The sand had a relative density of 21 per cent and a ϕ angle of 34.2 degrees as calculated from Equation (5.1). In measuring the failure load and end bearing, the equipment failed to work properly since a larger force was measured on the pile tip than was applied at the top to cause failure. The failure load at the pile top of 4,330 pounds was considered to be correct and only the pressure at the tip was considered invalid. The average densities and failure loads of Test Series No. 2 and Test Series No. 3 were approximately the same. Therefore, in order to make their base forces approximately the same also, 2000 pounds was subtracted from the invalid base force of Test Series No. 2. Using this reduced base force of 2700 pounds, a side friction value was computed. It was these speculated values based on the 2000 pound reduction that are compared to the other theoretical and measured results. Using 2700 pounds as the base load, the side resistance was 1,630 pounds. Meyerhof's N_q value for a deep cylindrical foundation yielded an end bearing force of 14,800 pounds which was extremely high when compared to the measured value. From Berezantsev's 1961 N_q, a value of 3,120 pounds was calculated for the tip capacity.

The side resistance, dependent on the assumed tip value in the test, was 1,630 pounds or 37.7 per cent of the total pile capacity. The theoretical side resistance from Equation (2.12) with a K = 1.0 was 2,150 pounds and from Norlund's Equation (2.14) using a K = 1.1 was 1,960 pounds. A K of 1.1 was used in Norlund's Equation (2.14) since his method proposed a way to determine a K from curves. As in the other tests, the theoretical values of Q_f were all high.

Assuming a triangular lateral pressure distribution on the side of the pile the average K value necessary to give the 1,630 pound side resistance force was 0.758. Again, trouble with the lateral pressure rings allowed only one direct lateral pressure measurement and this at a depth of 8 feet. The lateral pressure measured was 5.15 psi and its corresponding K value was 1.1.

Thirteen-Foot Pile

The failure load for the 13-foot pile was 6,920 pounds. Since the base tip was reduced by 2,000 pounds for the 8-foot pile, 2,000 pounds was also subtracted from the measured tip capacity for the 13-foot pile reducing it to 3,600 pounds. Therefore, 3,320 pounds or 52 per cent of the total load was carried by the side resistance. Theoretically computed, a base load of 24,000 pounds was calculated using Meyerhof's N_q and 5,050 pounds using the N_q Berezantsev determined in 1961. As before, all theoretical values were higher than the measured values with Meyerhof's N_q yielding an extreme value.

Equation (2.12), which assumed a traingular distribution, yielded a Q_f value of 5,680 pounds with a K of 1.0 and Norlund's Equation (2.14) yielded a Q_f of 5,180 pounds for a K of 1.1. Both theoretical values were almost double the value of 3,320 pounds found by subtracting the speculated base load from the failure load.

After driving the pile to 13 feet, the gage at the bottom registered a lateral pressure of 6.44 psi. A seemingly very large pressure of 5.8 psi was measured at a depth of 2 feet and is of questionable validity. Relating the pressure to the overburden by K, the K was 0.848 for a pressure of 6.44 psi and 5.04 for the 5.8 psi pressure. Assuming a triangular lateral pressure distribution on the side of the pile, the average K necessary to give the measured side resistance of 3,320 pounds was 0.584. The K of 0.584 was below the typical K of 1.0 for a loose sand as suggested by Sowers and Sowers (1). However, the side resistance was from the estimated end bearing value and if the value estimated was too large, the side resistance would be too small and consequently a low K value would result. The K from the measured pressure at a 30-foot depth was reasonably close to 1.0, being 0.848.

Test Series No. 3

Eight-Foot Pile

Because of the questionable validity of Test Series No. 2, another test series was conducted. The sand had an average density of

86 pcf and a ϕ of 35.5 degrees. The relative density was 30 per cent, low as in Test Series No. 2. The pile failed under a load of 3,810 pounds with 2,710 pounds being end bearing and 1,100 pounds being side resistance. The side resistance comprised 28.9 per cent of the total load. A force of 500 pounds due to the driving was on the pile tip after driving and before loading. After loading and unloading, the 500-pound force was still present. Comparing the theoretical end bearing values in Table 5-3 of Meyerhof and Berezantsev, Meyerhof's value of 21,200 pounds was extremely high and Berezantsev's value of 3,780 pounds was the closer though still high and on the unsafe side when compared to the 2,710 pound measured force.

Assuming a K of 1.0 and $\delta = \phi$, the skin friction Equation (2.12) yielded a Q_f value of 2,320 pounds, whereas Norlund's Equation (2.14) yielded a Q_f of 2,260 pounds when assuming a K of 1.2 and $\delta = \phi$. These two theoretical values were remarkably close to each other and were just over twice the measured skin resistance as were the end bearing values. Assuming a triangular lateral pressure distribution, the average K value necessary to give the measured side resistance of 1,100 pounds was calculated to be 0.475. As before, this K was below the typical K value of 1.0 suggested by Sowers and Sowers (1). The possibility of no pile failure as in Test Series No. 1 can be eliminated since a load-settlement curve, Figure 5-1, was made to assure that pile failure was reached. The piles were loaded immediately after driving. It has been observed that the end bearing of a pile in a dry sand may reduce with time after driving, in rare cases up to as much as 40 per cent (18). This possible loss of end bearing with time

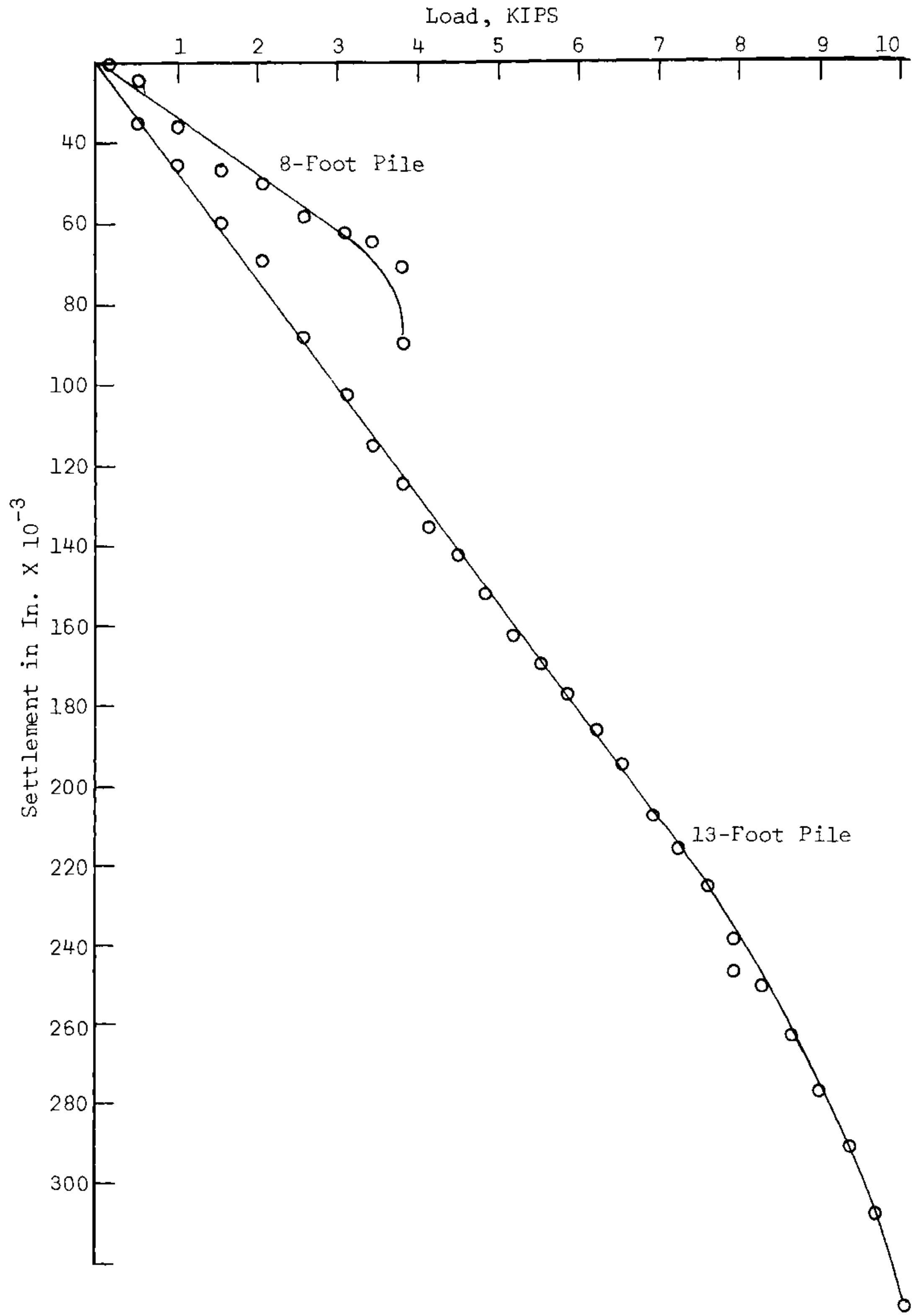

Figure 5-1. Load-Settlement Curves for Test Series No. 3

would be due to the dissipation of a temporary state of stress that formed in the sand around the pile tip during the driving. Since no time was allowed for this possible state of stress to dissipate, a high end bearing may have been obtained which reduced the side resistance and consequently the K value.

The lateral pressure gages and rings were closely inspected before this test in an attempt to get good readings from all gages. Two of the rings were found not to be completely free to float on the "O" rings and as a result were damaged in Test Series No. 1 and No. 2 due to the large driving stresses being transmitted through them. They were repaired and fixed so they would be floating freely. One gage was still lost during the test but the other three apparently worked properly throughout the test. The gage that was lost was due to a loose wire connection and not due to overstressing. At 6 feet deep, a lateral pressure of 2.9 psi was measured. At 8 feet the pressure was 7.6 psi. However, in this gage a small amount of the driving forces were being transmitted through the ring. The value of 7.6 psi may be too large as a result. The K for the 2.9 psi lateral pressure was 0.81 and the K for the 7.6 psi lateral pressure was 1.59. These lateral pressures and K values are tabulated in Tables 5-5 and 5-6. Again, the K being larger than 1.0 was probably due to a small increase in pressure reading due to some driving stresses or possibly to an increase in the sand density caused by the driving.

Thirteen-Foot Pile

The end bearing of the 13-foot pile was measured to be 6020 pounds. The failure load was 10,050 pounds. The pile skin friction consisted of 38.4 per cent of the pile capacity or 4,030 pounds. The theoretical end bearing values, calculated using the N_q values of Meyerhof and Berezantsev, were 34,400 pounds and 6,150 pounds, respectively. The Meyerhof value was too high and on the unsafe side when compared to the measured value and Berezantsev's value of end bearing was very close to the actual value of 6,020 pounds. Again the force on the pile tip was measured after the driving but before loading. The force was 750 pounds. After unloading, the pile still had a force of 810 pounds being exerted on its tip.

The theoretical side resistance calculated using Equation (2.12) was computed to be 6120 pounds for a K of 1.0. Norlund's skin resistance value was 5960 pounds using a K of 1.1. Both were too high and on the unsafe side when compared to the measured value of 4030 pounds but agreed closer than any of the other test values. Again assuming a triangular lateral pressure distribution, the average K necessary to give the measured side resistance of 4030 pounds was calculated to be 0.66 which again was below 1.0.

Three lateral pressures were measured directly. The deepest ring was 13 feet deep and showed a pressure of 8.2 psi. This was the same gage used for the 8-foot depth for the 8-foot pile in the first half of Test Series No. 3 and for the same reasons, may be too high. If the sand densified due to the driving of the pile, this 8.2 psi lateral pressure would be reasonable. The gage at an 11-foot depth measured a pressure of 6.3 psi and the third gage measured a 1.8 psi lateral pressure at a 2-foot depth. To agree with the typical K value of 1.0 as suggested by Sowers and Sowers (1), all these pressures should have a corresponding average K value of 1.0. The pressure at 13 feet had an average K of 1.06 and the pressure at the 11-foot depth had an average K value of 0.96. The lateral pressure at 2 feet had a K of 1.51.

Since the lateral pressure gages apparently worked properly, lateral pressures only were recorded for a 4-foot pile, a 6-foot pile, and a 10-foot pile as well as the 8- and 13-foot piles that were also tested for bearing capacity. These additional lateral pressures are given in Table 5-7 with their corresponding calculated K values. All but one of these K values were greater than 1.0 and varied from 1.08 to 1.38.

Table 5-7. Additional Lateral Pressures and Lateral Pressure Coefficients from Test Series No. 3

4-Foot Pile			6-Foot Pile			10-Foot Pile		
Depth Ft.	Lateral Pressure PSI	K	Depth Ft.	Lateral Pressure PSI	K	Depth Ft.	Lateral Pressure PSI	K
2	0.5	0.43	4	3.2	1.38	8	5.0	1.08
4	2.9	1.20	6	4.1	1.18	10	-	-

CHAPTER VI

DISCUSSION OF TEST RESULTS

Each test has been examined briefly on an individual basis. Now all six will be compared together. Tables 5-3 through 5-6 present the data for easy comparison. Looking first at the actual failure loads measured, in all cases they are lower than the theoretical values found using Norlund's Equation (2.14). The tests show that the failure load increases both with the pile length and the sand density with the exception of the 13-foot pile in Test Series No. 1. Evidently the pile was not loaded to failure as it has a lower capacity than the 13-foot pile in Test Series No. 2. Norlund's equation which used Berezantsev's N_q was used for the theoretical pile capacity and as stated before, produced results on the unsafe side, varying from the measured values by as much as 60 per cent in one case and as little as 17 per cent in another.

In comparing the measured loads at the pile tip, only Test Series No. 3 can be used with any certainty, as Test Series No. 1 results were too low due to not loading the pile to complete failure and Test Series No. 2 values were estimated. Theoretical values found using Meyerhod's N_q value for deep circular foundations, when compared to test values, showed that all theoretical values were extremely high. Since Test Series No. 1 was not loaded to failure, only Test Series No. 2 and No. 3 were compared. Berezantsev's N_q of 1961 produced results much closer to measured values than did Meyerhof's N_q. Using Berezantsev's N_q, the theoretical and measured values of the 13-foot pile in Test Series No. 3 differed by only 130 pounds. The maximum variance for all tests using Berezantsev's N_q was 1,450 pounds or 28 per cent and this was for one of the estimated values. Considering the sensitivity of the tip capacity with the ϕ angle, Berezantsev's N_q value yielded theoretical values that were remarkably close to the measured values.

Comparison was also made to the values found by using Meyerhof's N_q for shallow foundations which should act as a lower bounds on the tip capacity. With the exception of one of the speculated values, all theoretical results were on the safe side. Therefore the N_q value necessary to give the measured values can be said to lie between Berezantsev's values as an upper limit and Meyerhof's values for a shallow foundation as a lower limit. These tests show that for the lower values of ϕ, Meyerhof's N_q for shallow foundations is best and for higher values of ϕ, Berezantsev's N_q gives the best results. Norlund's equation also gives good theoretical values for the tip capacity.

Since the side resistance of the pile was found indirectly by subtracting the tip load from the failure load, any errors in either will also affect the side resistance values. Again Test Series No. 1 results were invalid and omitted from the comparative discussion since the pile was not loaded to failure. Also Test Series No. 2 results were from the speculated values of the tip load. Test Series No. 3 were assumed to be reasonably correct. As before, all theoretical results were higher than the measured values. It can be seen that the side resistances do increase with both an increase in pile length and an increase in density. This was also true for the tip resistances and pile capacities.

Two values of Q_f were found theoretically which agreed very closely with each other, with the values found using Norlund's method being the more conservative. The theoretical value for the 8-foot pile in Test Series No. 3 was twice the measured value, being 1100 pounds higher. For the 13-foot pile in Test Series No. 3, the values differed by as much as 2000 pounds but the theoretical value in this case was only 1.5 times the measured value. Again the sensitivity due to ϕ made this seemingly large difference be quite reasonable. In percentage, the Q_f value was from 30 to 50 per cent of the total pile capacity load. Also, it seemed that the longer the pile, the greater the percentage of the total load carried by the skin friction. The measured values of skin friction showed an increase of between 11 and 13 per cent for the increase of pile length from 8 to 13 feet. Norlund's equation gave an increase of 12 per cent and allowed from 31 to 51 per cent of the total load to be carried by skin friction. Terzaghi and Peck (19) state that the side friction of a pile in sand can be as much as 50 per cent of the total pile capacity. However, Low (20) states that the skin friction comprises only 10 to 20 per cent of the pile capacity. The data from these tests tend to support the statement of Terzaghi and Peck more than Low's. The results of Shiver's two tests in 1967 (15) showed the skin friction to consist of 53 per cent and 66 per cent for a 10-foot and 15-foot pile, respectively. Shiver used the same pile that was used in the tests in this report. Vesic (6), using a similar pile, also found the skin friction to consist of greater than 50 per cent of the total pile capacity in a loose, dry sand at a depth greater than 8 feet.

After failing to work satisfactorily in Test Series No. 1 and No. 2, the lateral pressure rings were closely inspected. The rings that failed to give valid readings were found to be transmitting some of the driving stresses since they were not completely free to float on the "O" rings. The rings were reworked to fit properly and worked very satisfactorily for Test Series No. 3. Table 5-5 gives a summary of all directly measured lateral pressures with their corresponding depth and density. Generally the lateral pressures showed an increase with an increase in depth and density. For Test Series No. 3 additional pressures were measured since the lateral rings were working properly. They are reported in Table 5-7.

In Figure 6-1 and Figure 6-2, plots of all measured lateral pressures versus the overburden, γz, and the relative depth, z, expressed in pile diameters, were made. Figure 6-1 consists of only the data from these three tests and Figure 6-2 includes the pressures of Shiver (15) in addition. In Figure 6-1, treating all points equally, the relationship appears to be linear to a relative depth of 22 pile diameters and then breaks downward with a steeper slope. Considering only the pressures from Test Series No. 3, this same change occurs at the same relative depth. If it is taken into account that the two pressures from the deepest gages in Test Series No. 3 may be high due to erratic behavior during driving, the break downward is more definite. In Figure 6-2 Shiver's pressures were added and this nonlinear interpre-

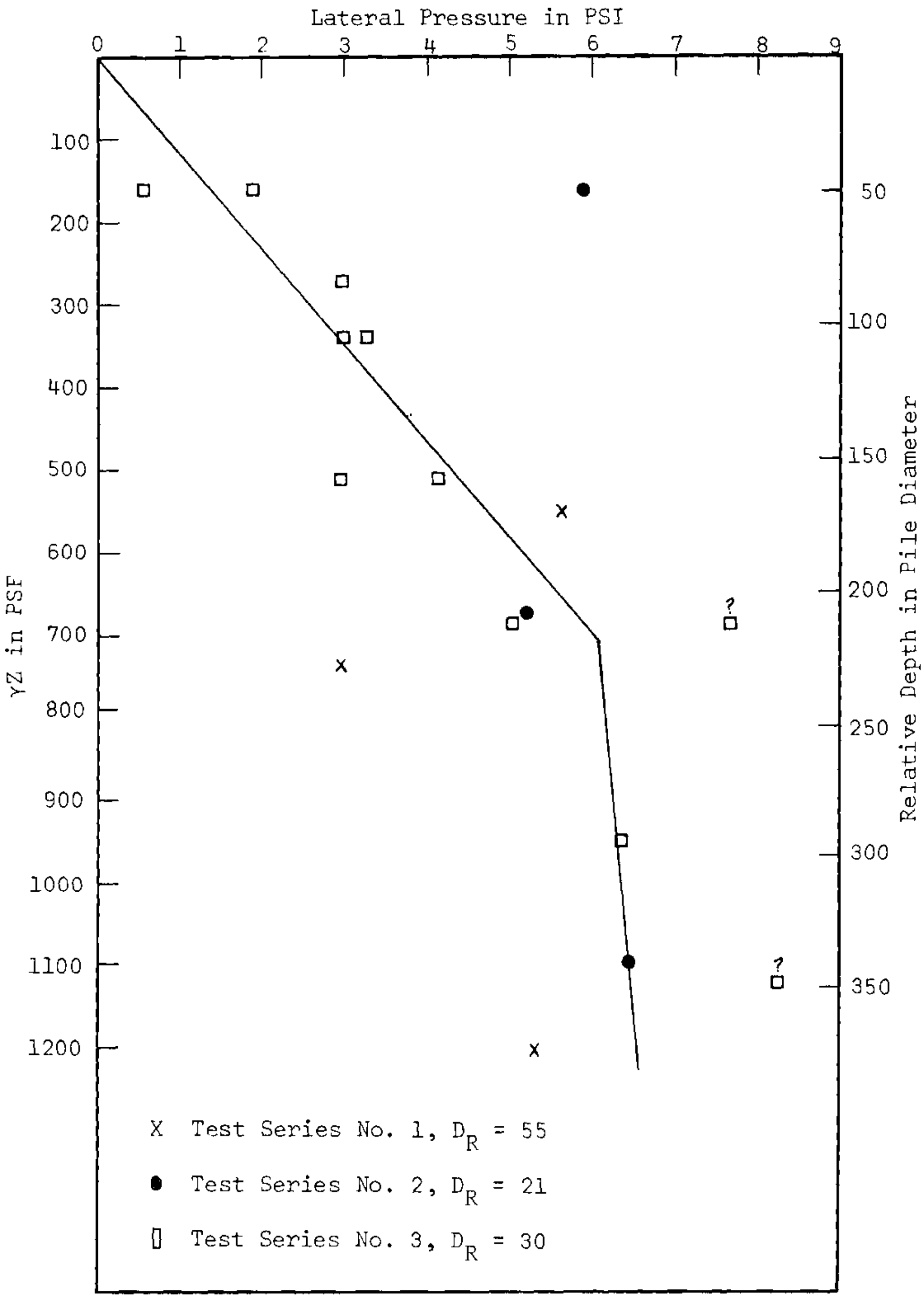

Figure 6-1. Lateral Pressure versus γZ and Z

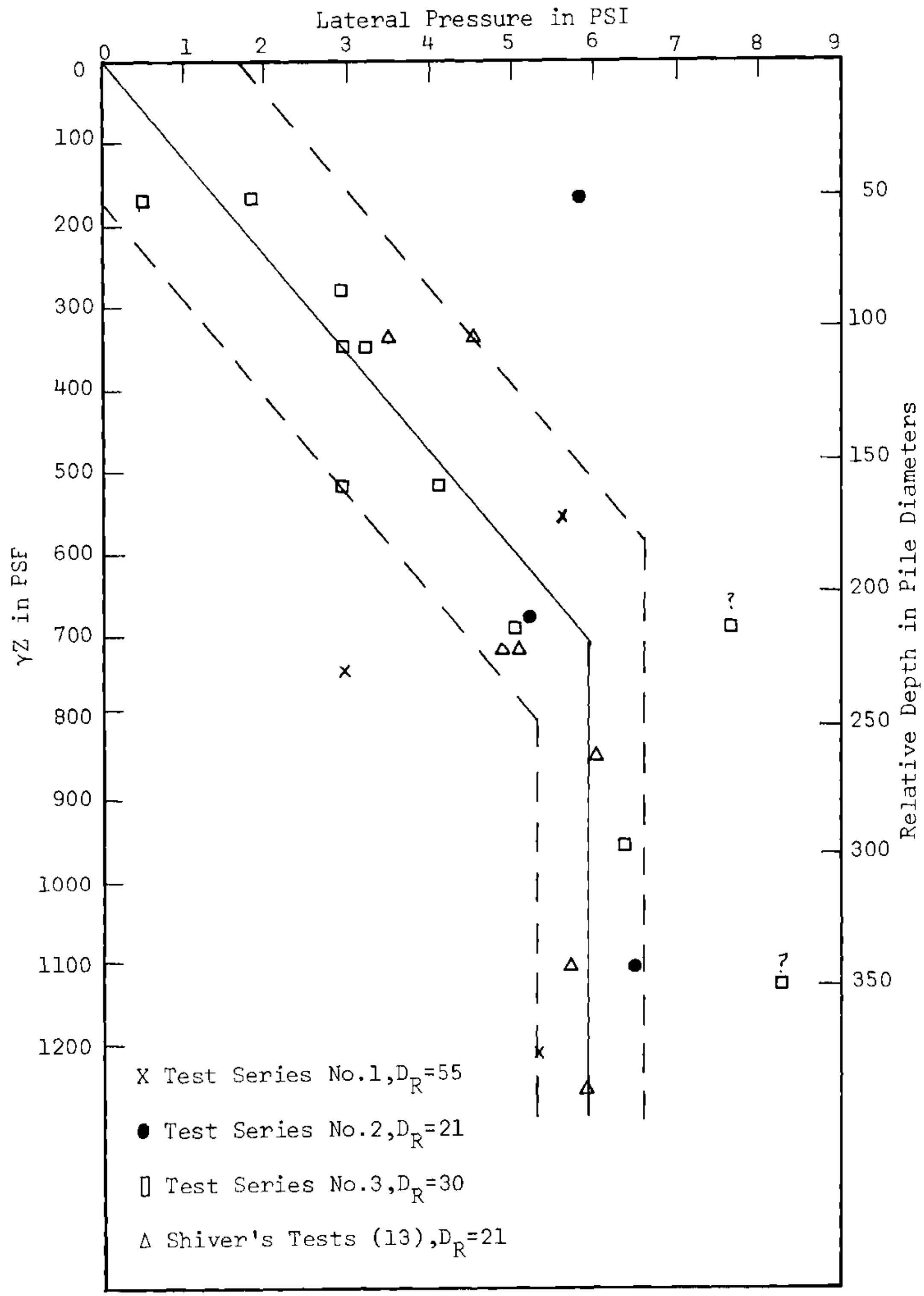

Figure 6-2. Lateral Pressure versus γZ and Z

tation became even more prominent. Actually a relatively narrow band was drawn such that all but four of the pressures were included and two of these four pressures were questionable. Again the break downward occurs at the relative depth of 22 pile diameters. Nothing conclusive can be stated since the data are limited but after more tests are run and more data made available, especially at the deeper depths, perhaps this band can be narrowed even more into a more definite relationship. From the data available to date as presented in Figure 6-2, the relationship can be plainly seen to be nonlinear and not linear as once postulated.

As stated previously, K was assumed to be 1.0 for the theoretical calculations. In his Equation (2.14), Norlund proposed a method which included many variables, such as the pile taper, size and roughness, in his determination of a K value. Using the curves presented in his paper (10), a K was calculated to use in his equation. For Test Series No. 1, a K of 2.0 was found. For Test Series No. 2 and No. 3, K's of 1.1 and 1.3 were determined. Table 5-6 presents the K values in tabular form that were found from the side resistances measured and from the directly measured lateral pressures. The K values calculated from the measured side resistances were all less than 1.0. The low K values from Test Series No. 1 can be explained because the pile was not loaded to complete failure. Also Test Series No. 2 values were from speculated forces on the pile. However, Test Series No. 3 can be explained by the possibility that the side resistance increases with time accompanied by a loss of end bearing due to settlement of the disturbed sand around and under the pile. More reasonable perhaps is the possibility that the tan δ was in actuality lower than the value of tan ϕ instead of being equal to it.

In both of the skin friction Equations (2.12) and (2.14), the angle of the sliding friction, δ, was assumed to be the same as ϕ since the pile was coated with a sand and epoxy covering. However, a crude test was made in an attempt to measure the tan δ. A piece of metal was coated with epoxied sand as was the outside of the pile. A small plexiglass open-ended cylinder was filled with sand and placed on the coated piece of metal. The metal strip was then raised and δ was measured as the angle the metal strip made with the horizontal when the sand and cylinder started sliding. The results of this crude test gave a tan δ of 0.6. The tan ϕ for the loose sand varied from 0.68 to 0.816 in these tests; therefore, the possibility of tan δ being lower than the tan ϕ was sustained. Since lateral pressures were measured along with a side resistance for each test, a tan δ was computed for each lateral pressure measured using Equation (2.12) which assumed a triangular lateral pressure distribution in determining Q_f. See Table 6-1 for the results. In all cases the calculated tan δ was less than the tan ϕ. Four of the calculated tan δ values produced angles from 9 to 11 degrees lower than their corresponding ϕ values. Therefore it seems that perhaps the δ angle should be measured and used in the theoretical calculations instead of assuming that δ was equal to ϕ. The comparison of the true δ angle and ϕ to see if and how they do differ would be a good area for future investigation. Perhaps the δ angle actually varies with the techniques of driving the pile and depth. The low K could be reasonably explained then by having used too

Table 6-1. Angle of Sliding Friction Calculated from Measured Side Resistances and Lateral Pressures

Pile Length (Feet)	Test Series No. 1				Test Series No. 2				Test Series No. 3			
	D_R = 55%				D_R = 21%				D_R = 30%			
	Side Resistance	Lateran Pressure	tan δ	tan ϕ	Side Resistance	Lateral Pressure	tan δ	tan ϕ	Side Resistance	Lateral Pressure	tan δ	tan ϕ
8	2300	2.90	0.677	0.816	1630	5.15	0.467	0.68	1100	7.6 (?)	0.213	0.714
13	3080	5.27	0.532	0.816	3320	6.44	0.467	0.68	4030	8.2 (?)	0.447	0.714

great an angle for δ. Also the sensitivity to the ϕ angle affected these K values so perhaps they agree satisfactorily with the typical value of 1.0 considering this sensitivity. Using the measured lateral pressures and the overburden stresses, K was computed. All values computed were close to 1.0. For Test Series No. 2 and No. 3, the K's from the measured lateral pressures were averaged and they agreed exactly with the value of the average K determined from Norlund's curves. See Table 5-6.

Using the equivalent K found from the measured side resistance forces and by integrating the lateral pressure distribution over the surface of the pile and setting it equal to the measured skin friction, Figure 6-3 was obtained. Actually two different lateral pressure distributions were assumed, each based on a constant K. The first was that the vertical pressure increased linearly with depth and the second was that the vertical pressure increased linearly to a relative depth of 22 pile diameters and then remained constant as Figure 6-2 indicates. According to Vesic (6), for very long piles both the point resistance and side resistance reach a final constant value somewhere below the relative depth of 10 pile diameters--depending on the relative density of the sand. Since the lateral pressure is a function of K, γz, and tan δ, one of the three must vary, and possibly all three may, to cause the nonlinearity Vesic has suggested. Tan δ is usually considered to be constant. Therefore on this assumption, either K or the vertical stress σ_z must vary. Vesic has hypothesized a constant K and a variable σ_z. He has explained the variable σ_z as being due to

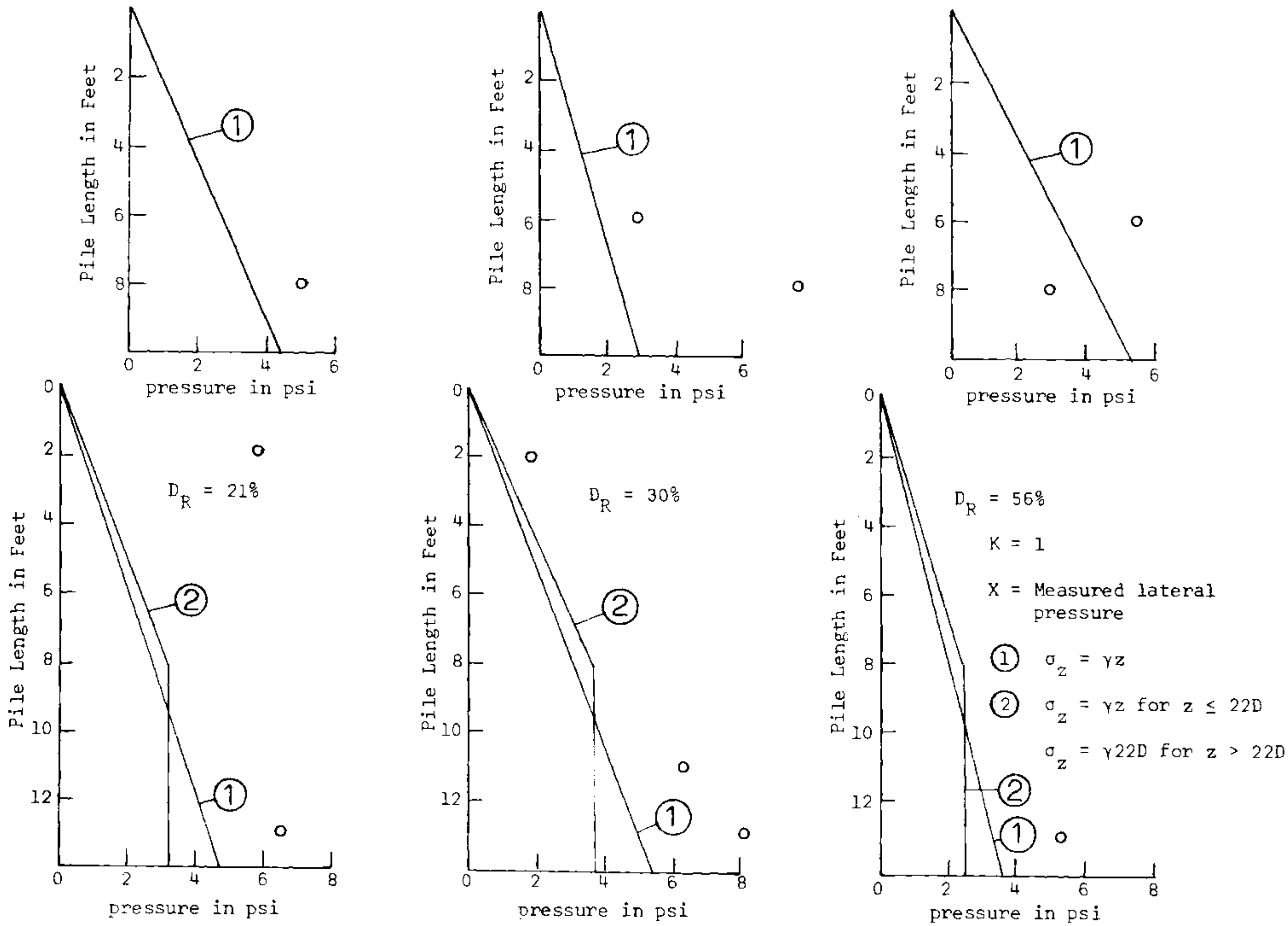

Figure 6-3. Comparison of Measured Lateral Pressures and Those Calculated from the Measured Side Resistance

arching in the sand around the pile at and below a relative depth of from 10 pile diameters for a loose sand to 30 pile diameters for a dense sand, causing the σ_z to remain constant with K below the relative depth where the arching began. For the 8-foot pile then the measured pressures should be linear but for the longer 13-foot pile, they should follow the nonlinear diagram. In Figure 6-3 all the pressures are shown to be too high, being approximately twice the pressures in the diagrams. The reason is that the K values are about one half of what they should have been. As explained previously, this is probably due to the tan δ being lower than the tan ϕ instead of being the same as assumed. The piles in Test Series No. 1 were not completely failed, further explaining the low K and Test Series No. 2 values resulted from speculation. The most reasonable explanation for Test Series No. 3 is that the δ angle was less than the ϕ angle.

A K value of 1.0 was assumed and the same type of distributions were made in Figure 6-4. Again the measured lateral pressures were plotted on each diagram. Seemingly a triangular distribution fitted best but by adding the pressures found by Shiver (15) and those in Table 5-7, the distribution tends to break down away from linearity especially near the relative depth of 22 diameters for the 13-foot pile of Test Series No. 3. This tendency for the curve to reach a constant value supports the findings of Vesic (6). Also important is that regardless of the linearity or nonlinearity, the directly measured lateral pressures agree with the distributions made assuming a constant average K of 1.0. This shows that the K values of Figure 6-3 are low and that a K of 1.0 is a reasonable assumption for the theoretical calculations.

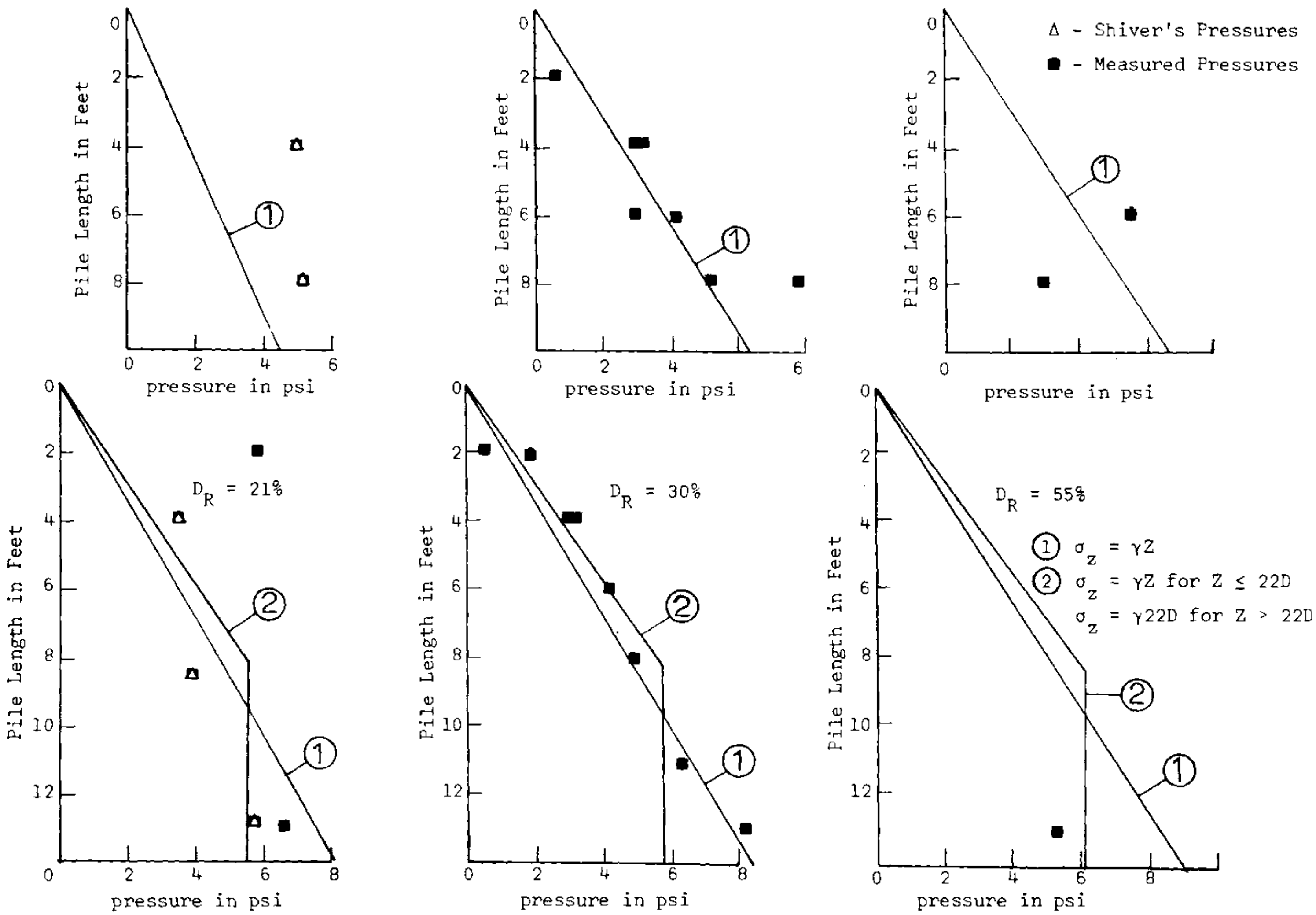

Figure 6-4. Comparison of Measured Lateral Pressures and Those Calculated Using a Constant K of 1.0

Figure 6-5 shows the skin friction variation on a pile in sand as found by Vesic (21) and Kerisel (22). Also included are the skin resistances found from the directly measured lateral pressures of Shiver in 1967 (15), and those reported in Tables 5-5 and 5-7. The resistances determined from the measured lateral pressures of Shiver's tests and the tests in this thesis indicate linearity to a relative depth of 22 diameters and show little increase of resistance below that depth. Omitting Shiver's values, the same relationship appears but not as clearly. Until more data are available, especially at greater depths, nothing conclusive can be stated. However, the present data indicate that a nonlinear relationship exists between relative depth and lateral pressures or skin resistance.

Presently attempts are being made to instrument more full-scale piles. According to Broms (23), a measuring device has been developed in Sweden to separate the point resistance from the total applied load by measuring the compression of the lower part of the pile. Since the device is inserted in the pile after driving, the load distribution before the external loads are put on is impossible to determine. A new, rugged device to withstand the driving pressure is being developed. Reference (24) is a good reference for field data from instrumented piles. It presents details of the instrumentation, results from this instrumentation plus some data on the dynamics of the pile during driving. Reference (17) gives a computer program for the theoretical calculations of bearing capacity and other values used in the report. Most important is that these references show that more work is being done on instrumented full-scale piles furnishing badly needed data.

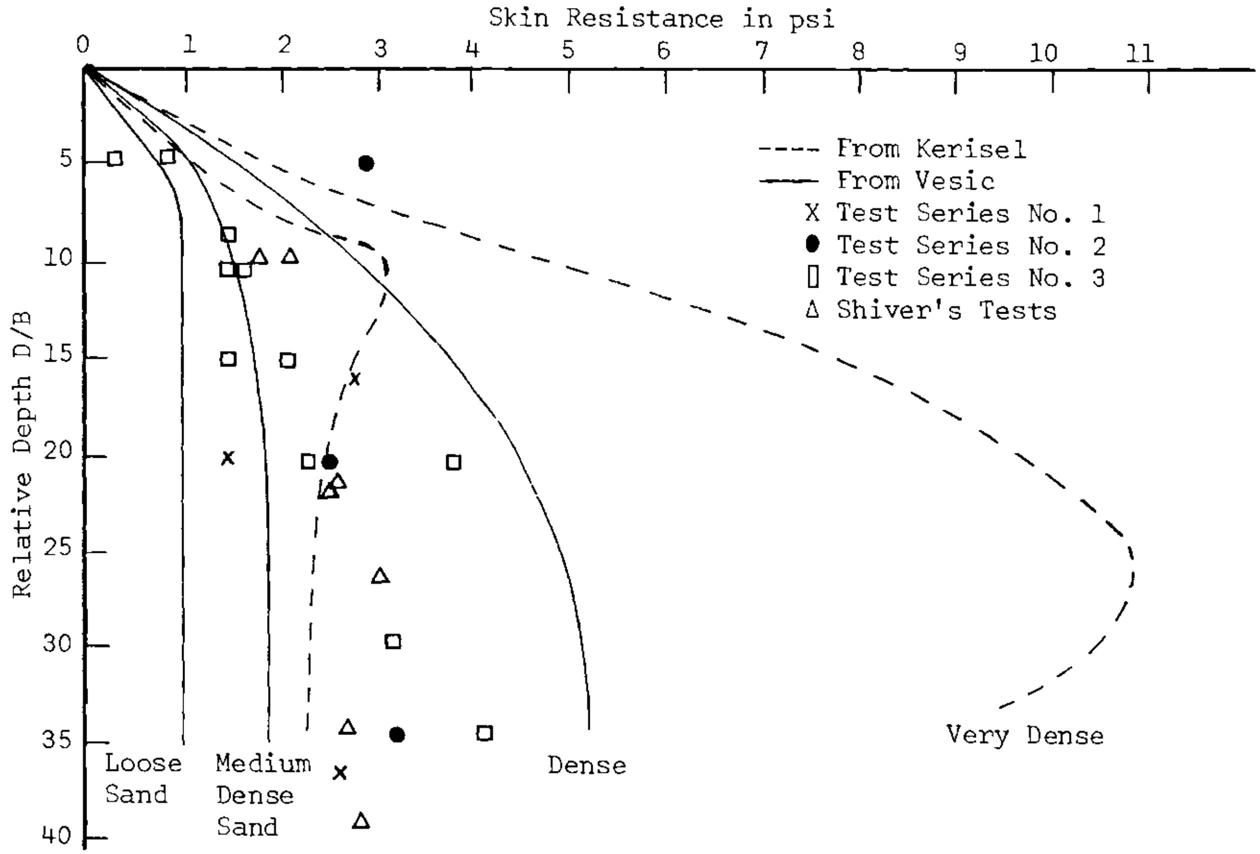

Figure 6-5. Variation of Skin Resistance with Depth

CHAPTER VII

CONCLUSIONS

The three pile tests reported in this thesis were conducted on an instrumented, 4.5-inch diameter, cylindrical, steel, pipe pile with a rough outside coating of epoxied sand. Piles were driven in a dry Chattahoochee River sand at average relative densities of 21 per cent, 30 per cent, and 55 per cent. Each pile was test loaded at depths of 8 feet and 13 feet. At failure, the lateral pressures against the side of the pile and the tip load of the pile were measured in addition to the failure load. From the results of these tests, the following conclusions were made:

1. The skin friction consists of 30-50 per cent of the total pile capacity.

2. The true N_q value lies between Berezantsev's N_q of 1961 as an upper limit and Meyerhof's N_q value for shallow foundations as a lower limit. Meyerhof's shallow N_q is best for the lower values of the angle of internal friction while Berezantsev's N_q is best for the higher values of the angle of internal friction.

3. The skin friction values calculated using Norlund's method could be as much as twice the actual skin resistance.

4. The average coefficient of lateral pressure, K, is approximately 1.0 for a relatively loose sand. The K values determined from the measured lateral pressures varied from 0.566 to 1.59.

5. The end bearing force and the side resistance both increase with an increase of density.

6. The lateral pressure versus γz relationship is linear to the relative depth of approximately 22 pile diameters. Below this depth, the lateral pressures become constant which means that if K is constant as assumed, then γz is constant below this depth also.

7. The tan δ was probably less than the tan ϕ as assumed.

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