

EXPERIMENTS ON MODEL PILE FOUNDATIONS IN SAND

by

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ABSTRACT

Pile foundations are normally constructed with a rigid cap cast on the surface, tying a group of pile heads together. A series of experiments with model pile foundations in sand, have been carried out in which the distribution of load within the piles was measured.

Tests with 25 piles spaced from 2 to 8 pile diameters apart show that the order of driving controls load distribution until the piles fail. The last driven piles carry the most load.

This behaviour is explained in terms of the initial stress conditions beneath individual piles.

Some examples are given which show how the results may be applied to practical design.

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CHAPTER 1

INTRODUCTION AND PREVIOUS WORK

1.1 Introduction

1.2 Previous Work

1.1 INTRODUCTION

The function of piles in sand is to prevent excessive settlement of a structure. One approach to the understanding of this problem is to study the behaviour of single piles. Such an approach is based on the tacit assumption that the environment of the pile is of relatively little significance. Fig.1.1 shows a typical pile foundation, Fig.1.2 a single pile in a semi-infinite mass of sand; it is the first driven pile of the group. Clearly in Fig.1.2, the effect of the remaining piles as well as the top cap is absent.

A complete understanding of the problem requires a knowledge of how the whole foundation distributes load to the sand. A load from the structure will be carried partly by the base and shaft of each pile and partly by the cap (Fig.1.3).

The only suitable means of obtaining this information is by using experiments with models. Such experiments have never been done before.

1.2 PREVIOUS WORK

The effect of driving piles

It has long been realized that driving piles into sand causes an increase in the density of the sand; the term compaction pile, used for piles in loose sand, means an increase in density. Piling contractors note this effect by the greater force required to drive later piles; indeed, they will hesitate to drive the centre pile of a group last, for fear of causing structural damage to the pile.

Plantema and Holet⁽²⁰⁾, using static cone resistance as a measure of the density, found that sand became denser around a 55 cm. dia. pile, even when the static cone resistance before driving (about 160 kg/cm²), indicated a dense sand. In some cases however, particularly near the surface, driving the pile into dense layers caused a slight loosening of the sand. All loose sand became denser due to pile driving. The penetrometer tests were carried out approximately 2 pile diameters away from the driven pile.

There are many other field and model investigations^(7,8,13,14,17) which show the compaction of sand which occurs as a result of driving piles.

Meyerhof⁽¹⁷⁾ has extended his work on the ultimate bearing capacity of a single pile in semi-infinite sand, to include subsequent piles using revised angles of shearing resistance to account for the increase in density. This analysis depends for its validity, on the tacit assumption that piles driven into sand which has been compacted by pile driving, will carry a greater load when part of a group, because of the increase in density. Chapter 8 shows that this is not the case.

When piles are driven into relatively dense sand, a general heave of the surface will occur. If the sand contains piles, they will move upward with the sand. Bondal⁽²⁾ shows that an initial upward movement of a pile 50 cm. dia. occurs when driving another pile at a distance of 2 - 5 metres. The piles were driven to a depth of 14 metres (28 pile diameters) through 12 metres of loam and peat and 2 metres into sand. Although the pile moved upward when another pile was driven, thereafter a time dependant settlement of the pile occurred, undoubtedly caused by the soil in the upper 12 metres.

Load distribution within a group with the cap free of the surface

An extensive series of tests has been carried out by Pepper in which the load distribution within groups with as many as 36 piles was measured. In an interim report⁽¹⁹⁾, a figure is given which shows the centre piles carrying more load than the corner piles, presumably, at failure conditions although this is not definitely stated. The order in which the piles were driven is not given.

On the basis of model experiments, Borodugo⁽³⁾ concludes that initial load distribution within a group depends mainly on the order of driving, the later driven piles carrying more load than the first driven. When failure is approached, there is a redistribution of load and pile position becomes significant.

The research was carried out with wooden and brass piles 0.325 in. dia. and 10 in. long. The piles were driven individually to form a group of 4, 9 or 16 piles in either a random or practical driving pattern. While the patterns used are not the extreme cases⁺, it is interesting to note that later driven piles carry more load than first driven piles, when driving progresses from one side of a foundation to the other. Such a pattern however, makes the foundation unsymmetrical with respect to order of driving and therefore should tilt. Unfortunately, the results were not analysed in terms of settlement. Such an analysis is also required if errors due to eccentric loading are to be considered.[‡]

+ Section 5.1 discusses the patterns necessary to completely separate order of driving from position.

‡ In Appendix C, the possible errors due to eccentric loading are examined.

CHAPTER 2

PROPERTIES OF THE SAND

2.1 Index Properties

2.2 Shear Strength

2.1 INDEX PROPERTIES

Approximately $1\frac{1}{2}$ tons of sand needed for the programme was thoroughly mixed before any group experiments were carried out.

Grading

Ham River sand, nominally 52/100 British Standard Sieve, was used. Fig.2.1 shows the actual grading.

Specific gravity

A specific gravity of 2.68 was obtained using B.S.1377.

Moisture content

All experiments were carried out with the sand in an air dry state, the moisture content being between 0.3 and 0.4 per cent.

Maximum porosity

Maximum porosity using the 'tilting test' ⁽⁹⁾ gave an average value of 49.3 per cent.

Change in index properties with use

It is important in a testing programme to check the index properties during the sequence of testing. The determinations given above represent average values from a sample taken before the programme was begun and near the end. There was no significant difference in any of the measurements. There is no doubt however, that the sand changed; this will be discussed later.

Minimum porosity

Minimum porosity tests were attempted using the Proctor mould procedure suggested by Kolbuszewski ⁽⁹⁾. They could not be reproduced and therefore this technique was abandoned. An estimate sufficiently accurate for this project, can be obtained by using the following statistical relationship between the 'tilting test' and a minimum porosity test given by Alyanak ⁽¹⁾:

$$\text{Minimum porosity} = \text{tilting test} - 9.5 \text{ per cent}$$

The minimum porosity is therefore approximately 39.8 per cent.

Sphericity and roundness

Sphericity determinations using the Rittonhouse method⁽²¹⁾ gave a value of 0.81. The roundness of the sand is 0.96 using the visual method of Krumbein⁽¹²⁾.

2.2 SHEAR STRENGTH

A series of triaxial compression tests were carried out on a sample of sand before the entire $1\frac{1}{2}$ tons had been obtained. The grading was slightly different and the maximum porosity 47 per cent compared with 49.3 per cent. While the shear strength characteristics are given sufficiently accurately for this project, the 2 sands were not identical.

Four inch diameter specimens were tested at a constant cell pressure of 30 psi. The sand was air dry and volume changes were measured under drained conditions. A rain technique was used to deposit the sand. The testing apparatus, being essentially the same as that used by Yassin⁽²⁵⁾, Eldin⁽⁴⁾ and Fraser⁽⁶⁾ need not be described.

Fig.2.2 shows the relation between the maximum angle of shearing resistance and porosity. It will be seen that the angles of shearing resistance are quite high. Consistent with this is the very dilatant behaviour, as shown in the relation between the rate of volume change at failure ($d\frac{\Delta V}{V}/d\sigma$) and porosity (Fig.2.3). It can be seen that the samples dilate at failure for all porosities.

CHAPTER 3

SAND DEPOSITING APPARATUS

- 3.1 Introduction
- 3.2 Factors Controlling the Deposition of Dry Sand
- 3.3 Method of Deposition
- 3.4 Apparatus
- 3.5 Calibration of Apparatus
- 3.6 Variability of Porosity
 - direct measurement
 - single pile tests
- 3.7 Reproducibility of Porosity
 - direct measurement
 - single pile tests

3.1 INTRODUCTION

Model tests carried out in the laboratory have the advantage over field tests in that the experimenter has freedom to control factors which in full scale tests cannot be controlled. For pile experiments in sand, the most important single variable is the sand porosity. In this chapter, an apparatus will be described which deposits a ton of sand homogeneously and reproducibly over a wide range of porosities.

3.2 FACTORS CONTROLLING THE SAND DEPOSITION OF DRY SAND

Kolbuszewski^(10,11) showed experimentally that a wide range of porosity in dry sand could be produced using a rain technique. He found that the factors which controlled the porosity were the intensity of the rain and the height of drop. At a given height, an increase in the intensity increased the porosity; while, for a given intensity, an increase in the height decreased the porosity. The sensitivity of porosity to a change in either one of these factors depended upon the actual value of the other factor.

For any given sand, intensity and height of fall control the resulting porosity sufficiently accurately for most experimental purposes. There is evidence however, that particle elasticity⁽¹⁶⁾ and sphericity⁽¹⁵⁾ are also significant. This makes quantitative correlation for different sands impossible and sets limits within which any one sand can be deposited.

The apparatus developed as part of this project deposits sand at a range of porosity mainly by varying the sand intensity.

3.3 METHOD OF DEPOSITION

Sand is deposited as a thin layer which is formed as a rain of sand passes back and forth over the container, the height of drop always being kept constant (Plate 1). The rain is $5\frac{1}{2}$ feet wide and

moves through a distance of about 4 feet. In filling a 3 ft. dia. container, irregularities in the sand flow, occurring at each end when the direction of motion reverses, fall outside the container in a 6 in. overlap. The actual thickness of the rain on reaching the sand surface at any moment is approximately 1/2 in. for loose sand, when the intensity of deposition is high (Plate 2) and 2 in. for dense sand, where the intensity of deposition is low (Plate 3).

3.4 APPARATUS

Fig. 3.1 shows detailed drawings of the apparatus. A 7 cu.ft. capacity hopper is driven horizontally on 2 rails by a motor driven chain assemblage, at a rate of 0.3 ft/sec. Sand can be fed from the bottom of the hopper through a gap between a 3 in. dia. roller and a steel cutting edge positioned vertically above the centre of the roller. An accurately machined roller and cutting edge are essential to avoid any variation in the gap setting over the full width. Gap settings of up to 0.15 in. are obtained by adjusting the vertical position of the edge. A motor and gear box positioned at one end of the hopper drive the roller at speeds of $2\frac{1}{2}$, 5, 10, 20, 40, 60 or 100 r.p.m. Greater intensities of deposition are obtained using higher speeds of rotation; no sand flows if the roller is stationary.

With the roller turning, an initial horizontal velocity is imparted to the sand, the magnitude of which depends on the roller speed. A plate, positioned about 1/4 in. from the roller, deflects the rain into a vertical plane. The position and angle of the sand deflector must be changed for different porosities.

Thus, there are available, 2 means of controlling the intensity: 1) roller speed and 2) gap setting.

The rails on which the hopper runs are connected horizontally to form a rigid unit. This unit is carried on nuts running on 4 vertical threaded rods ($1\frac{1}{2}$ in. dia. x 8 ft. long, threaded throughout their entire length). The bases of the rods are connected to 4 box sections cast in concrete (Plate 1), the tops being attached by 4 pieces of angle iron. The entire unit may be raised or lowered at speeds infinitely variable from 0 to 2 in/min. by rotating the nuts through a sprocket and chain assemblage, driven by a motor and gear box.

Thus, the height of drop can be maintained constant during deposition by raising the hopper at the given depositing rate.

Sand is placed into the hopper from a storage bin, in a $1\frac{3}{4}$ cu.ft. bucket raised by an electric hoist (Plate 1). A suitably sized screen on the hopper collects extraneous material from the sand. Great care was taken to keep the floor clean where sand fell, but it was impossible to keep the sand entirely free of extraneous material. The atmosphere became contaminated with dust, particularly when dense sand was being deposited. This required the operators to wear dust filters and the 8 ft. length of threaded rod to be protected. No attempt was made to protect the motors; during the 5 months of testing, only one motor was put out of action.

3.5 CALIBRATION OF APPARATUS

The calibration of the apparatus was obtained using 4 cylindrical measuring cylinders to determine the porosity. For 3 gap settings, namely 0.097 in., 0.127 in., and 0.149 in., the porosity was measured at 2 heights and with at least 5 roller speeds. During each test, the rate of deposition was also measured. Fig.3.2 shows the results. As roller speed is the most significant factor, it is plotted directly

against porosity. A total range of 8.7% is possible with this particular sand. With one gap setting (0.149 in.) and one height (19 in.), specific porosities over a range of 8.4 per cent can be achieved, using different roller speeds. Smaller gap settings lower the value of porosity for any given roller speed and height; thus making it possible to deposit over the entire porosity range. For comparison purposes, maximum and minimum⁺ porosities are also shown in the figures.

The minimum height of fall possible is 19 in. as the container is made up of 2 halves, 18 in. high. It can be seen from the figure that the height of fall is a relatively insignificant factor; little accuracy was required therefore, to maintain a constant height during deposition.

In Fig. 3.9, the relation between the rate of deposition and the resulting porosity is shown; this relation is independent of gap setting and roller speed. For any porosity, the rate at which the sand is deposited and therefore the rate at which the hopper must be raised to ensure a constant height, is given by this curve.

3.6 VARIABILITY OF POROSITY

A variation of porosity within the container can be caused by many factors; some of them inherent in this particular method while others are present in any deposition technique involving an appreciable weight of sand. With the scanning method which this apparatus employs, air currents were encountered. A discussion of their occurrence and the corrective measures taken, is given in Appendix F. It is worthwhile noting, however, that 2 thin brass plates positioned within the container as shown in Fig. 3.4, were required to prevent undue disturbance of the rain. The porosity in the areas shown around these plates is known to be variable (denser by approximately 0.7%). The following discussion refers to the remaining area.

+ minimum porosity = maximum porosity - 9.5 per cent (see section 2.1)

There are two independent ways of establishing the uniformity:

1) Direct measurement of porosity and 2) Single pile tests. The overall variation within the container was found by means of direct measurement, which we shall now discuss, and this variation checked with the results of single pile tests.

Direct measurement of porosity

To obtain a direct measure of porosity within the container, calibrated cylinders may be placed in different parts of the container and the individual porosities checked against the average value. This method is however, subject to several criticisms:

1) It is necessary to use measuring cylinders sufficiently large that wall effects become insignificant and, with a height to diameter ratio not exceeding about 1. The smallest cylinders used were 3 in. dia. Porosity measurements therefore represent an average value for this volume.

2) Aside from errors in the actual porosity, the measurement of the volume of a cylinder cannot be obtained very accurately.

The overall variation within the container was found using 4 measuring cylinders placed in the positions shown in Fig. 3.4. The following conditions were tested:

1) Cylinders placed on top of the container, with the hopper (a) full and (b) nearly empty.

2) Cylinders placed at 3 depths within the container. One or two of these tests were repeated with a different weight of sand in the hopper.

At all porosities and for each of the conditions studied, individual porosity values all lay within ± 0.3 per cent of the average (e.g. 44.6 ± 0.3 per cent). In most cases, the range was closer. For two tests, 9 cylinders were used, with the variation being no greater than with 4.

Single pile tests

The ultimate bearing capacity of a single pile⁺ is a sensitive measure of the porosity. For porosities of 41.5, 39.7 and 39.0 per cent, 3 piles, 1/4 in. dia. were tested at positions within the container shown in Fig. 3.5. Each pile is far enough from another pile and the edge of the container to ensure no interference. In each case, the depth was $7\frac{1}{2}$ in. These results are shown in Fig. 3.6. The first driven pile tests in the group experiments have also been included. Over the range of porosity studied, there is an approximate linear relation. The error in this relation on the basis of a direct porosity measurement is indicated by the dotted lines. A range of $\pm .3$ per cent porosity means, that at any porosity, the ultimate bearing capacity may vary by ± 0.8 lb. It can be seen that the measurements of ultimate bearing capacity lie well within this range, with the exception of one test at 39 per cent. It should be noted that the values of ultimate bearing capacity are in themselves subject to error due to the accuracy of the load cell in the model pile (see Section 4.7).

3.7 REPRODUCIBILITY OF SAND DEPOSITION

Direct measurement

A sand depositing apparatus must be able to reproduce the same porosity for a series of experiments.[†] All group experiments were carried out at an initial porosity of 42 per cent. For most of the experiments, direct measurements of the porosity were made after the container had been filled. Throughout the series, no measurement lay outside the limits already established of ± 0.3 per cent porosity.

+ see Section 5.1 for a definition.

† The testing programme lasted 4 months during which 13 group tests were carried out.

Single pile tests

An independent check on the porosity may be obtained from the results of the single pile tests on the first driven pile. These results were included in Fig. 3.6, from which it is seen that they lie within the range established by direct measurement. In Fig. 3.7, the single pile tests have been considered on a larger scale. It can be seen that within the limits set by direct measurement, the ultimate bearing capacity increased as the testing programme proceeded. Although no difference in the grading of the sand could be detected, a slight change undoubtedly occurred; much dust is created during each deposition. Some change in the sand during testing must be accepted. Only a correlation involving such a sensitive indicator as ultimate bearing capacity however, is capable of detecting such a small variation. Fig. 3.6 shows that this error is not significant in the single pile tests. In Appendix C, evidence is presented to show that the group results were not significantly influenced.

CHAPTER 4

MODEL PILE

- 4.1 Introduction
- 4.2 Method of Measuring Load
- 4.3 Load Cells
- 4.4 Model Pile
- 4.5 Heating of Load Cells
- 4.6 Strain Measuring Equipment
- 4.7 Load Cell Calibration

4.1 INTRODUCTION

To enable measurements to be made of the load distribution within the foundation shown in Fig.1.3, it was necessary to design model piles fulfilling the following requirements:

- 1) They must be able to measure base and shaft load separately, with a cap cast on the surface.
- 2) The compressibility of the load cells in the piles must be small compared with the settlement.
- 3) The piles must have a relatively small diameter; anything larger than 1/4 in. requires an unmanageable size of container.
- 4) There must be a number of such piles.

This chapter describes model piles fulfilling these requirements. Twenty-five were built to the same design. Each pile weighs approximately 0.06 lb. and appears on the outside as a smooth brass rod (Plate 4(a)). They were designed to be used at one depth only ($7\frac{1}{4}$ in. = 29 pile diameter).

4.2 METHOD OF MEASURING LOAD

Load is measured as a calibrated strain. Each pile contains two thin-walled cylindrical tubes which strain in compression when under load. The compressive strain of each tube is measured by 4 foil resistance gauges, wired in a Wheatstone bridge. When the compression for any given load is known, the unit becomes a load cell.

4.3 LOAD CELLS

Each pile has one cell which measures base load and one which measures total load; the difference of these two gives the load distributed in shear along the shaft. Both cells consist of a brass tube, 0.003 in. thick for the base and 0.004 in. for the total. The base cell is designed to carry about 25 lbs., while the total load cell carries 30 lbs.

The construction of both cells is the same. Load is applied to one end through a ball seating (Fig. 4.1(A)), the other end being fixed centrally within the pile. Four $1/4$ in. linear strain gauges⁺, two active which measure linear strains and two passive which measure circumferential strains, are located on the tube in such a manner that the effects of a lateral load acting on the free end are minimized. By arranging each pair symmetrically - both in terms of distance from the fixed end and from the centre of the tube - the Wheatstone bridge circuit cancels changes in resistance due to bending. In placing the active gauges closer to the free end where bending strains are smaller, the total change in resistance due to a lateral load is smaller than if the gauge positions were reversed (Plate 4(b)).

4.4 MODEL PILE

The essential functions of the pile are to apply a relatively friction-free load to each cell, while allowing the cells space to compress freely. The cells are fixed centrally at each end of a solid brass abutment (Fig. 4.1 (B)). To this abutment are also attached two outer brass tubes which may easily be removed. As well as forming the exterior of the pile, they guide the load application to each cell. It should be noted that there is a space between the load cells and the interior of these tubes.

Measuring load in terms of strain requires a finite movement and therefore parts must slide relative to each other. A load on the point of the pile is transmitted to the base cell through a hardened steel rod and guiding bush, the sliding surfaces being shown in the figure. It is important to allow as little friction as possible as load cell

+ 38 of the 50 cells have 75 ohm gauges; the remaining 12, 55 ohm.

sensitivity will be reduced. It is more important however, that the value of friction should not change with time, thus changing the cell calibration. To satisfy both these requirements, use was made of the material D.Q.1⁺. Two bushes and therefore two sliding surfaces were used in transmitting load to the cell, in order to further reduce the effect which a lateral load on the point has on the load cell. A compression spring which applies a small zero load to the cell, may conveniently be fitted between these bushes. As well as increasing zero stability, use of a compression spring allows base loads after driving as low as 0.2 lb. to be measured.

With a load applied to the base, the strain of the cell causes a change in length of the pile; length ab shortens to a'b. Under a load of 20 lb. the cell compression (aa') is about 0.0008 in. To allow this movement to occur freely and yet provide protection to the sliding parts, a thin rubber membrane enclosing a sponge rubber washer separates ab.

The top half of the pile, which includes the total load cell, is essentially the same as the bottom. There are however, 4 wires[†] from the base cell as well as 4 wires[†] from the total cell which must pass around the ball seating of the total cell and out the top of the pile. Thus, load must be transmitted to the cell by a tube with the wires on the inside. Any load applied to the top of the tube registers directly on the total load cell. Great care was taken with each pile to make this surface smooth and even, as load must be applied to the pile through this butt seating. The pile heads used for the single pile and group tests will be described later.

+ D.Q.1 is a material manufactured by Glacier Metal Co. which has a low coefficient of friction and may be worked in a lathe.

† Fine nylon, acetate coated wire was used, as it bonds together to form a wire rope when dipped in acetone; 4 wires therefore became 1 wire rope.

As with the base cell, a movement must occur with each applied load; distance cd shortens to c'd. Under a load of 25 lb. the cell compression (cc') is about 0.0008 in.

4.5 HEATING OF LOAD CELLS

In making the load cells of a thin-walled tube, an instrument sensitive to temperature is also created. Changes in measured strain result from two effects.

When the strain measuring instrument (Baldwin) passes a current through the resistance gauges, the load cell is heated and the resulting strain measured on the Baldwin. Two methods were used to avoid this error:

1) Each cell was measured after the Baldwin had been heating it for a given time; about $1\frac{1}{2}$ minutes was sufficient.

2) All cells were preheated to a stable reading by supplying each with the same current supplied by the Baldwin.

The first method was used for the single pile tests; the second for the group tests. Both methods are satisfactory; the second a little more accurate and much easier.

The second heating effect is due to a change in room temperature. The sand itself being a good heat insulator delayed the effect of any change in room temperature on the piles. The time required for a single pile test was only about 10 minutes, during which no change in temperature occurred. For the group experiments, the heating characteristics of the piles for each spacing were measured. Only about 8 of the 50 gauges changed significantly during the 2 hours required for a test. The piles containing these gauges were placed randomly in the group and the corrections become insignificant when the results are averaged in terms of position⁺.

+ In Section 6.3, the method of analysing the results is discussed.

4.6 STRAIN MEASURING EQUIPMENT

As previously noted, strain readings on the 25 piles were taken using a Baldwin strain meter. As it was decided to use a standard instrument for this purpose, a Baldwin was required as four 55 ohm gauges in a Wheatstone bridge will damage a Beckel by overloading the oscillators.

The four leads from each cell were connected to a heating and switching unit from which any one of the 50 cells could be connected either to a Baldwin circuit or a heating circuit. In Plate 10, which represents a group test, the heating and switching unit appears in the bottom right hand corner. Each pile lead - about 15 in. long - was soldered directly to a connection box, which could be placed near the piles. This same assemblage was used for group tests, single pile tests and pile calibration.

4.7 LOAD CELL CALIBRATION

All piles were initially calibrated under constant temperature conditions. Using dead weights, the top cell was loaded to 25 lb. and the base to 20 lb. Each cell has a unique calibration; the total cells, being thicker, are slightly less sensitive than the base. Fig. 4.2 shows a typical pile calibration.

To use any load measuring device in a series of tests, it is necessary to provide the following evidence:

- 1) that the calibration does not change during the series of tests; or if it does, the calibration should be known for all tests;
- 2) that it is valid to use the calibration for the conditions which exist in the test.

Throughout the experimental programme, the piles were recalibrated in the constant temperature room, all 25 at least once and some 2 or 3 times. All calibrations reproduced to within 10 per cent at any load; more than 2/3 of them to within 4 per cent or better. Included in Fig 4.2 is the pile recalibration.

The piles were used in sand with strain measured under 2 conditions:

1) Cells were unheated, strain readings taken after a given time (single pile tests).

2) Piles were preheated for about 12 hours, by which time the cells had become stable. To take readings, each cell was switched from the heating circuit to the Baldwin circuit (group tests).

To check the calibrations under the conditions of the single pile test, 5 of the piles were driven into sand and loaded, using dead weights; strain readings were taken after a given time. The calibration of the total load cell may be checked directly against the loads added; it is impossible to check the base cell with the pile in sand. The calibrations reproduced those obtained under constant temperature conditions.

Each single pile test itself, is a check on the total cell. This evidence is meaningless however, without a full comprehension of the arrangement for the single pile test. It is therefore presented in Appendix B.

It is impossible to check the pile calibrations under exactly the same conditions as exist when they form part of a foundation with the cap on the surface. However, two tests were carried out which closely approximate these conditions. In the first, 25 piles were driven into sand and the piles heated for 12 hours. The temperature of the sand near the piles can rise 5 or 6 deg.C. above room temperature when the spacing is close. Six of the total load cells were individually calibrated using dead loads. Strain readings were taken in the same way as for the group tests. The piles which were measured, reproduced their previous calibrations.

For the second test, a cap was cast onto the 25 piles, but suspended from the surface. The conditions were exactly the same as when the cap was formed on the surface, except that the plaster was not present. With the cap free of the surface, any load applied to the foundation should equal the summation of loads measured by the total load cells. The results are shown in Fig. 4.3. It is seen that the cells read consistently low, the error however, can be neglected.

CHAPTER 5

THE SINGLE PILE

5.1 Definitions

5.2 Procedure and Apparatus

5.3 Single Pile Behaviour

- a) a single pile in semi-infinite sand
- b) a single pile in sand containing driven piles
- c) sand compaction during pile driving
- d) the effect of driving piles on a pile which has already been driven

Summary

5.1 DEFINITIONS

If known loads are applied to a pile and the resulting settlements plotted against applied load, the diagram is known as a 'load-settlement diagram', such a test a 'pile test' and the pile a 'single pile'. It acts independently of any other pile or structure which may be near. Throughout this thesis, pile behaviour will be frequently described in terms of load-settlement diagrams. It is useful therefore, to consider a typical diagram for a single pile (Fig. 5.1). The settlement increases slowly at first, then passes with increasing plastic deformation into a fairly linear relation (AB) known as a 'failure envelope'. Point A, at which the pile ceases to be unique, represents failure of the pile⁺ for that depth. The load at failure is called the 'ultimate bearing capacity'. Fig. 5.2 shows the entire failure envelope which the pile would follow if it were tested from the surface.

Immediate settlement

If loads are applied in increments and each load maintained, a creep settlement will be measured. Fig. 5.3 shows the load-settlement diagram for the first driven pile; each load was maintained about 2 minutes. In order to compare the load-settlement behaviour of piles when incremental loads are maintained for different periods of time, it is necessary to subtract the creep settlement. The resulting curve C_1 represents immediate settlement, while curve C_2 , total settlement. A slightly different failure envelope is generated in each case. All load-settlement diagrams will be considered in terms of immediate settlement.

Pile spacing

This is the centre to centre distance between the piles expressed as a ratio of the pile diameter, when all the piles in the group have been driven.

+ This term implies failure of the soil and not of the pile material.

⊕ Whitaker and Cooke⁽²⁴⁾ discuss this criterion for piles tested at a constant rate of penetration.

Order of driving

The order of driving refers only to the succession of pile driving. Two terms will frequently be used: first and last driven piles. What is intended is clear from the context.

Driving pattern

The sequence in which the foundation positions are occupied by piles is the driving pattern.

The need for the last three terms to be clearly defined is more easily seen by an example. Consider the basic grid used in the experiments: square with 5 piles along each side. Fig. 5.4 shows the two driving patterns employed in the experiments to drive the 25 positions of this grid. Case 1 illustrates a 'centre to corner' driving pattern. The centre pile is driven first; thereafter piles are driven in a symmetrical sequence towards the corners of the grid. Case 2 illustrates a 'corner to centre' pattern in which the corner piles are driven first, the centre pile being the last. If spacing is considered after all piles in the group have been driven, the two cases have the same spacing; they differ only in the pattern used. If the arrangement is considered, for example after the fourth pile has been driven in both cases, the spacing is not the same. Not only is the spacing different as pile driving continues but the environment in the two cases is completely different. Compare the environment into which the 22nd pile is driven in the two cases. Clearly, there are many additional factors involved in the variable driving pattern. These two patterns were chosen because they represent the extreme conditions which are needed to completely separate the effects of pile position from order of driving. Fig. 5.5 shows the two patterns when 16 piles are driven around the periphery of a foundation.

Initial porosity

The only time when the condition of the sand is definitely known, is before any piles have been driven. The porosity at this stage will be called the initial porosity.

Pile driving

Full scale piles in sand are generally driven under a series of impact loads. In this series of experiments, each model pile was pushed by hand at a relatively constant rate of penetration. They will however, be referred to as 'driven' piles and the act of placing the piles as pile 'driving'. *

5.2 PROCEDURE AND APPARATUS

All the piles to be driven were suspended in a guiding unit as shown in Plate 5. A test which included only 16 piles is shown for the sake of simplicity. The guide unit consists of two 1/2 in. steel plates, 1 in. apart with 25 holes, 1/4 in. dia., drilled at 1/2 in., 1 in., 2 in. and 4 in. centres. This one unit could then be used to guide piles for spacings of 2d, 4d, 6d and 8d. Each pile is a sliding fit to ensure that it is driven vertically. The guide unit locates on a rigid metal frame which stands clear of the container (Plate 5 and 6). The frame itself locates on the floor with reference to the container (Plate 10). As the bottom half of the container is fixed in position, the piles are always driven into the centre of the container. The frame is vertically adjustable for levelling the guide unit to 0.6 in. clearance above the surface of the sand (Plate 5).

To guide each pile when clear of the guiding unit and to apply load to the top load cell during testing, a pile head, 3 in. long, must be fitted over the lead, onto the top of the pile. Now the first pile to be driven is wired to the Baldwin and zero readings on the load cells taken. The pile is released from the guide plate and driven into the sand until a predetermined mark on the pile head becomes level with the guide plate. The toe of the pile will be at a depth of $7\frac{1}{4}$ in. below the original level

(5)
+ Fleming carried out a series of model tests with the piles forced to depth at a constant rate of penetration and driven with a hammer. No significant difference was noted in the ultimate bearing capacity of the piles.

of the sand. The load cells are then read again. No attempt is made to measure the failure envelope during driving.

Using the electric hoist, the loading frame is placed on the rails of the sand depositing apparatus and moved into position above the pile (Plate 6).

Application of load

Load is applied manually through a worm reduction gear. A sensitive proving ring measures the load transmitted to the pile through a frictionless ball race. To apply load to the driven pile without interfering with the piles suspended in the guide unit, a ram extension 7 in. long was employed (Fig. 5.6 and Plate 6). This steel rod transmits load by means of a butt seating to the top of the pile head which has remained in position since driving. The pile head was made as a sliding fit over each pile and through each hole of the guide unit, for it must be easily removable from the pile after testing. Any load added to the pile head acts directly on the top load cell, this load being equal to the value measured by the proving ring.

Two 10^{-6} in. settlement dials - two being necessary to compensate for ram clearance in the bush - measure the pile settlement with respect to the rigid frame (Plate 6).

Single pile test

With the pile under no added load, zero readings are taken on the load cells and the settlement dials. Incremental loads are now applied to the pile. Each load is maintained by means of the proving ring⁺, long enough for both load cells to be read. The actual time depends on the heating characteristics of the load cells; the longest time required was $1\frac{1}{2}$ minutes. Settlement was read at the beginning and end of each load

+ Loads were applied and maintained using the proving ring as a measure of load. All single pile tests however, are plotted using load cell measurements. See Appendix B for a further check on the calibrations, which the proving ring provides.

increment in order to obtain immediate settlements. The test was continued until a failure envelope had been established.

5.3 SINGLE PILE BEHAVIOUR

Before considering the effects of driving a number of piles in sand, it is useful to examine the behaviour of a single pile driven into sand unchanged by any previously driven piles.

5.3(a) A Single Pile in Semi-infinite Sand

By semi-infinite, the following conditions are meant:

- 1) The radius of sand around a pile is great enough to ensure no container effect (see Appendix E).
- 2) The sand in the zone to be affected by driving the pile has not been disturbed by any previous pile driving; the sand is therefore at an initial porosity. To satisfy this condition, the zone cannot contain any previously driven piles.

If piles under these conditions are studied with the sand at different initial porosities, the behaviour may be correlated directly with changes in sand porosity. When discussing the variability and reproducibility of sand deposition (Section 3.6 and 3.7), the results of tests carried out at four initial porosities were given as evidence (Fig. 3.6). We shall now consider these tests in greater detail.

Base load after driving

A pile driven into sand at this porosity to a depth of $7\frac{1}{4}$ in. (29d), maintains, when the external pile driving force has been released, a small but significant positive base load. This load must be balanced by an equal and opposite shaft load, leaving the pile under a slight compression. Fig. 5.7 shows an idealized failure envelope of the pile from the surface. The base and shaft load build up as shown. When the depth has been reached and the driving force released, the base load returns to a small positive value, while the shaft load returns first to zero and then becomes negative.

Fig. 5.8 shows the relation between the base load after driving and the initial porosity. It can be seen that a pile driven into denser sand maintains a greater compressive load after driving.

Load-settlement behaviour

Fig. 5.9 shows the load-settlement diagrams. The importance of a porosity change of 3 per cent is clearly shown, not only in the ultimate bearing capacity which is more than doubled but also in the initial slope of the curves. All piles fail at a settlement of approximately 5 per cent d .

Load distribution between base and shaft

When a load is added to a pile, part of that load will be carried by the base and the remainder by the shaft. In order to show clearly how this load is distributed at any settlement, load-settlement diagrams with the base and shaft originating from zero will be considered. These loads are 'added loads'.

The load distribution is essentially the same for all porosities studied. Fig. 5.10 shows a typical test result; the porosity in this case being 42 per cent. It will be seen that the load is carried predominately by the base at all settlements, failure of the pile being brought about by failure of the base.

It will also be noted in the figure that the shaft load is fully mobilized at a smaller settlement than the base. In adding a load to a pile a greater settlement is required to mobilize fully the base load than the shaft. Let us re-examine the existence of a base load after driving in the light of this information. A greater unloading settlement will be required to release the base load than the shaft. Indeed, this entire recovery is never fully realized; for, the shaft load decreases to zero and builds up negative. When this load balances the base load, no further recovery occurs. In Fig. 5.7 the unloading behaviour has been shown.

The load-settlement behaviour of a pile may be considered in terms of 'total loads', in which the original compressive load on the pile is included. The curves are the same as for added loads except that base and shaft load do not originate from zero. Fig. 5.11 represents the same test shown in Fig. 5.10. As loads considered with respect to an absolute zero taken before pile driving have been called total, the curves are shown as base, shaft, and base + shaft. In single pile tests, base + shaft load equals total load, in the group tests this is not the case. For the sake of consistency, the same nomenclature will be used throughout. It should be remembered however that the shaft load is the difference between the other two measurements.

5.3(b) A Single Pile in Sand Containing Driven Piles

All piles were driven to a depth of $7\frac{1}{4}$ in. into sand initially at 42 per cent porosity.

Base load after driving

The first driven pile maintains a base load of about 0.2 lb. Succeeding piles maintain greater loads. In order to compare this effect at different spacings, tests using a centre to corner driving pattern will be considered. Fig. 5.12 shows the base load after driving, as a function of the order of driving for the 4 spacings used. For every spacing, the base load on each succeeding pile increases rapidly until between 12 and 16 piles have been driven; thereafter, the increase is relatively small.

Load-settlement behaviour

Each pile is more difficult to drive than the previous one. The load-settlement behaviour manifests the greater driving force required. The total load-settlement diagrams for 6 piles tested after driving⁺ at a spacing of 2d are shown in Fig. 5.13.

+ All single pile tests were carried out before any other piles had been driven.

The greatest change in behaviour occurs with the first few piles driven. To show this effect for different spacings, it is convenient to represent each curve by one point - ultimate bearing capacity. In Fig. 5.14 the results for the centre to corner driving pattern are shown. It can be seen that the ultimate bearing capacity increases rapidly with the first few piles driven. After about the 16th pile however, little increase occurs. The changes in load-settlement behaviour with the number of piles driven is very similar to those noted for the base load after driving (compare with Fig. 5.12).

5.3(c) Sand Compaction During Pile Driving

We may now compare the behaviour of single piles in sand containing piles with single piles in semi-infinite sand. If Fig. 5.12 and 5.14 are compared with Fig. 5.8 and 3.6 respectively, it will be seen that a decrease in porosity occurs during pile driving. Fig. 5.12 and 5.14 both show that the greatest change in porosity occurs with the first few piles which are driven.

In order to include the results using a corner to centre driving pattern, it is necessary to consider the relation between base load after driving and ultimate bearing capacity on the same pile when tested. Fig. 5.15 shows this relation for piles driven into sand containing piles. It will be seen that the greater the base load after driving, the greater is the ultimate bearing capacity when tested, independent of the driving pattern. In Fig. 5.16 the relation for piles in semi-infinite sand is shown by a solid line. In this case the initial porosity into which the piles were driven is given along the top of the figure. Included in the figure as a dotted line is the previous relation. That the two relations are relatively similar is consistent with the conclusion that pile driving causes sand compaction.

Other changes in the sand due to pile driving

A later driven pile enters sand which has been prestressed and which has been 'reinforced' by previously driven piles. Inherent in Fig. 5.15 is the effect of these secondary factors.

5.3(d) The Effect of Driving Piles on a Pile which has already been Driven

Let us examine this effect with 25 piles driven at 4d spacing using a centre to corner driving pattern. The central 9 piles having been driven, the base cell of pile No.8 is now balanced on the Baldwin until heating effects become negligible. With the base cell continuously measuring load, the remaining 16 piles are driven. As each pile is being driven, any change in the base load on pile No.8 is measured directly. This test is extremely accurate. Changes in load of 0.03 lb. are significant, for the only way that the reading on the cell can be affected is through an alteration in the state of stress around and beneath the pile. The base load on pile No.8 after each succeeding pile has been driven and the driving pattern which was used, are shown in Fig. 5.17. As the effect of driving the 9th pile was not measured, the initial base load after driving (0.54 lb.) will be taken as the starting point. Driving pile 10, reduces this load to 0.31 lb., while pile 11 increases the load but not to its original value. As the remainder of the piles are driven the base load on pile 8, alternately decreases and slightly increases, the factors controlling this behaviour being inherent in the pile driving pattern. Each change in load caused by driving a pile should be correlated with the position of the pile which was driven, since each change is experimentally significant.

After the 18th pile, all remaining piles reduce the load, but the change is relatively small. In the figure, the final value after driving the 25th pile appears negative, which means physically a finite gap between the base of the pile and the sand. As the initial load is subject to some error and the effect due to driving pile 9 was not measured, the

absolute load is subject to a slight positive or negative error. There is no doubt however, that the final base load on pile 8 will be very close to zero.

The 8th driven pile was chosen because the initial base load is relatively large. The effect of driving the first 9 piles on pile 1 was also measured. The same trend occurred. The base load, initially only about 0.2 lb., had decreased to a slight negative value after driving the 9th pile. Indeed, piles 7, 8 and 9 caused little change in the base load on pile 1.

Summary

Single piles have been examined from the point of view of understanding the behaviour of pile foundations. Only factors which are relevant to this understanding have been considered.

Two important points have emerged from this study:

- 1) Piles compact sand.
- 2) Driving piles will reduce the base load on piles already driven.

P I L E F O U N D A T I O N S

C H A P T E R 6

P R E P A R A T I O N , T E S T I N G a n d M E T H O D O F A N A L Y S I S

6.1 Preparation

6.2 Testing

6.3 Method of Analysis

6.1 PREPARATION

In Chapter 5, the method of individually driving the piles was described. After all the piles have been driven and specific ones tested, the guide plate is removed. Plate 7 shows the 16 piles at this stage. For the piles to become part of a foundation, the tops are cast in plaster as part of a rigid cap, which bears on the sand surface.

To cast the piles as part of a rigid cap and recover them afterwards, requires pile heads which may be freed from the piles. These fittings become part of the casting and are removed with it, leaving the piles in the sand. Load is transmitted to each pile through its pile head which bears on the cross-sectional area of the tubes extending from the top of the piles (Plate 7, Fig. 6.1). The pile head must be a sliding fit over the tube to fulfill adequately its two functions⁺.

When the piles are driven, the surface of the sand around each pile follows a curve[‡] as shown by the dashed line in Fig. 6.1. The pile head is then guided over the flexible tube containing the wires and fitted into position, the bottom edge cutting to some extent into the sand. As was emphasized in the description of the model pile (Section 4.4), a relative movement must occur for load to be measured in the pile. Point a will move down relative to b. To ensure that plaster does not seep into the sand and join these points together, sand is placed around the pile head bringing the surface to the full line. When this procedure is followed for all the piles, there will be an annulus of sand, placed by hand, about 1/8 in. wide around each pile head; the remainder of the sand surface has not been disturbed since the piles were driven.

+ Each pile head was individually fitted to one pile.

‡ This curve depends slightly on the spacing and driving pattern.

Wet plaster placed on the surface of dry sand results in a damp zone of sand to a depth of about 2 in. below the plaster. To prevent this water entering the sand and to further ensure that the top load cell is not prevented from straining, the surface of the sand is now sprayed with hair setting spray⁺. An impermeable, flexible skin of sand approximately 0.03 in. thick is formed.

The sand is sprayed to about 6 in. each side of the piles to allow formers and a holding frame to be placed on the sand without disturbance. Plate 8 shows the wooden formers in position and the aluminium frame which prevents them spreading when the cap is lowered. The formers are placed against the pile heads; the foundation width (w) is therefore:

$$w = 4s + d$$

where s(pile spacing) = 2, 4, 6 or 8d

and d = 1/4 in.

The inside and top of each former is coated with silicon grease to facilitate its removal when the plaster has set.

The plaster used is a high strength type. The powder was mixed with poly-vinyl acetate (P.V.A.)[‡] to an easily pourable consistency.

Casting the cap

The metal part of the cap is suspended from the rails of the sand depositing apparatus; it may therefore be raised or lowered at variable speeds (Plate 8). The cap is held at a height such that each pile head is located about 1 in. inside its respective hole in the cap; it is much easier to locate the pile heads before plastering, particularly when there are 25.

+ The hair setting spray used had a shellac base.

‡ The damp zone of sand observed with the unsprayed sand surface was not as deep with P.V.A. as water; the strain under the loads used is negligible.

Plaster is placed inside the formers using an icing set until the level is slightly above the $3/4$ in. width of the formers, care being taken not to trap any air. The cords are removed and the cap lowered into position. Excess plaster is squeezed out, at the edges and up the holes, as the cap is levelled. The level to which the plaster is forced up each hole is shown in Fig. 6.1.

After about an hour, the formers may be safely removed. Foundations were left at least over night however, before testing.

6.2 TESTING

The load cells are wired and the loading frame moved into position (Plate 9). Load application is essentially the same as in the single pile tests. In this case however, the load is transmitted to the foundation through a short, plane-ended ram extension which bears on a half-ball seating. Three 0.001 in. settlement dials at the corners of the foundation measure eccentric settlement in addition to the two 10^{-4} in. dials on the ram. As with the single pile tests, settlement is measured from the metal frame (Plate 10).

The heating unit is turned on approximately 12 hours before a test. All the tests were carried out in the morning, the room being on the west side of the building, temperature changes were small. Plasticine was placed around the openings to the pile heads, to increase the insulation of the load cells (Plate 9). During a test, air and sand temperatures were measured.

With the weight of the proving ring acting on the foundation[†], zero readings are taken on the 50 cells and 5 settlement dials. Incremental loads are then applied and maintained, while the 50 cells are being read. Settlement readings are taken at the beginning and end of each load

[†] Proving ring calibration under these conditions is more accurate than if suspended.

increment. Approximately 11 load increments constitute a test, which takes about 2 hours to complete. The actual loads to be applied were determined before a test, the increments being closer together at low foundation loads.

Change in load distribution with creep

As each load was maintained between 10 and 15 minutes, during which time the foundation continued to settle, change of load distribution with creep had to be considered. To eliminate this possible error, reading of the piles was random. Thus, the 4 corner piles, which were either the first or last driven, were the 11th and 12th, 17th and 18th, 35th and 36th, 49th and 50th cells to be read. In averaging these 4 piles⁺, one averages the effect of creep; similarly with the other pile positions. There is however, only one centre pile. It was always read at the beginning of the sequence and again at the end. Slightly different values were obtained at high loads but the difference is not significant.

Measuring zero stability

Either the morning before or the morning after a test, the zero stability of the gauges was measured. The testing apparatus was in the same position as during the test and the cells had been heated for the same length of time. Load cell and temperature readings were taken about every 20 minutes over the same 2 hour period.

Dismantling the foundation

The cap was raised, using the sand depositing apparatus, once again as a variable speed hoist. Metal cap, plaster and pile heads acted as one unit, leaving the piles in the sand. The plaster was then broken away from the metal cap and pile heads. Grease placed as shown in Fig.6.1 facilitated this process, but it was still quite difficult. Sand which had been sprayed was discarded. The piles were dug out of the sand and carefully checked before beginning another test.

+ This point is more fully discussed under Method of Analysis.

6.3 METHOD OF ANALYSIS

Averaging pile loads

Pile loads have been averaged in terms of position. This method cancels out the effect of any slight eccentric loading on the foundation as well as averaging variations in porosity and the effect of creep. The pile positions were chosen with this method of analysis in mind. Those piles whose calibration only reproduced to 10 per cent and which required zero stability corrections, were placed randomly throughout the groups. Thus, the accuracy of the average values is approximately the same. The pile chosen for the centre position however, was the most accurate pile of the 25.

In the group of 25 piles used, there are 6 positions. These positions and the symbols used to represent them are shown in Fig. 6.2(A). Each position now has an average order of driving, the value of which depends on the driving pattern used. The average orders of driving are also shown in the figure for the 2 patterns used. Orders of driving of $1/2$, though not physically possible are real points in a relation involving order of driving as a variable. Clearly, in averaging pile positions, small ranges of the variable order of driving are also averaged.

In Fig. 6.2(D), 16 piles are considered. The position nearest the corners, in which there are 8 piles, has been divided into 2 symmetrical units of 4 piles each, to show the effect of order of driving on the same position.

Average immediate settlement

All test results have been plotted in terms of average immediate settlement. The average settlement of the corners reproduced the average ram settlement. Eccentric settlement and the reproducibility are considered in Appendix G.

Added loads

Driving a pile into sand reduces the compressive load on piles already driven. While this effect has been established, quantitative

measurement for each spacing and driving pattern was beyond the scope of this project. Consequently, when all the piles have been driven, the absolute values of base and shaft load are not known. In forming a foundation a weight of metal and plaster is added. This load is distributed to the cap and to the base and shaft of the piles. Each test was begun with an additional load (6.8 lb.) on the foundation due to the proving ring and settlement dials. In all cases, zero loads are considered from this condition; all loads are therefore added loads. Extrapolation of the individual pile curves to distribute the zero load confirms the validity of this procedure. The maximum error existing in the last driven piles is 0.3 lb. As the error is positive, the results as considered are, if anything, conservative.

CHAPTER 7

BEHAVIOUR OF PILE FOUNDATIONS

- 7.1 Introduction
- 7.2 The Effect of Order of Driving on Pile Load Distribution
- 7.3 Pile Load Distribution at Different Spacings
- 7.4 Pile Base Load Distribution at Different Spacings
- 7.5 Load-Settlement Behaviour of the Whole Foundation
- 7.6 Load Distribution within a Foundation

7.1 INTRODUCTION

When a load is applied to a foundation the load distribution within the piles is initially dependent on the order of driving; the last driven piles carry more load than the first driven. With increasing loads, the position of the pile becomes important; the centre pile will carry the most load and the corners the least, independent of the driving pattern adopted.

The pile load distribution becomes more uniform as the spacing increases. At close spacings, the last driven piles carry as much as four times the load carried by the first driven, while at wider spacings only about one and one half times; about 75 per cent of this load in both cases is taken by the base of the pile.

Load distribution within the individual piles must be considered in relation to the behaviour of the whole foundation. Piles have the greatest effect on the load-settlement behaviour of a foundation, at settlements where order of driving is the operative factor in load distribution. At greater settlements where position determines the distribution, large foundation behaviour is controlled by the cap while a small foundation has already failed.

7.2 THE EFFECT OF ORDER OF DRIVING ON PILE LOAD DISTRIBUTION

A comparison of two tests using 16 piles, most clearly shows the importance of the last driven piles. As both driving patterns were used, the corner piles become the first driven in one case and the last driven in the other. The results are shown in Fig. 7.1. Load is initially distributed to the last driven piles whether in the corner or side positions. Until a settlement of 0.010 in. is reached, the two tests are remarkably similar. Thereafter, position modifies the behaviour. When in the corner position, the last driven piles fail, exhibiting a prominent peak load and at large settlements, the distribution becomes reversed with the centre piles carrying a greater proportion of the load. In the other case, the

side piles fail but, being in a more favourable position, continue to carry more load at all settlements.

In Fig. 7.2, the pile base load distribution for each test is given. It will be seen that the significance of order of driving lies in the load carried on the base of the pile.

Two tests with 25 piles, using both driving patterns provide further evidence for the effect of order of driving. The piles were spaced at 8d, making the total width of the foundation 33d. This is almost 4 times the width of the previous foundation. Fig. 7.3 shows the results of the tests. As with the small foundation, the distribution at small settlements is dependent on the order of driving while at large settlements, pile position is the significant factor.

The base load distribution shows the same pattern (Fig. 7.4).

7.3 PILE LOAD DISTRIBUTION AT DIFFERENT SPACINGS

Using a centre to corner driving pattern, foundations containing 25 piles were tested at spacings of 2, 4, 6, and 8d, the size of each foundation being 9, 17, 25, and 33d respectively. As in all tests carried out, the pile cap was rigid (Appendix D).

The results of the four tests are given in Fig. 7.5, 7.6, 7.7 and 7.3(A) respectively. In each case the pattern of load distribution is the same. It is useful to study the curves on the basis of the failure envelope criterion used for single piles. At close spacings, only the outside, last driven piles and particularly the corner piles follow this pattern; the load-settlement relation for the central 9 piles is practically linear. At wide spacings, all of the piles generate a failure envelope.

In the cases where the piles fail, this failure occurs progressively and less catastrophically from the last driven to the first driven. Thus, when the corner piles fail the load on the other piles, although smaller, is still increasing. The load distribution at this settlement is entirely dependent on the order of driving.

The slope of each failure envelope depends on the pile position, the centre piles having the steepest, while the corner piles the flattest which, at close spacings is negative. With further increases in load, the differing failure slopes cause the distribution to be reversed with the centre piles becoming predominant. It is worth emphasising that, where pile position determines the load distribution, some, if not all of the piles are behaving under failure conditions.

As a measure of the effect of order of driving on load distribution at different spacings, the loads on the centre and corner piles at which the difference in load is the greatest, may be chosen. The relations between these loads and the spacing are shown in Fig. 7.8. It can be seen that the distribution becomes more uniform as spacing increases; at a spacing of approximately $13d$, the distribution would be uniform⁺. It is interesting to note that the settlement at which the greatest difference in load occurs is independent of the spacing (i.e. independent of the foundation size). It is worth mentioning in passing that this evidence and the fact that the last driven piles fail at approximately the same settlement, suggests negligible scale effect. The settlement for each spacing is indicated in the figure; they lie between 0.008 in. and 0.013 in. (slightly less than 5 per cent d).

7.4 PILE BASE LOAD DISTRIBUTION AT DIFFERENT SPACINGS

At all spacings (2, 4, 6 and $8d$), it is the load carried by the base of the piles which determines the load distribution. The base load-settlement curves, Fig. 7.9, 7.10, 7.11 and 7.4(A) respectively, follow the same pattern as previously discussed. Order of driving distributes load to the base of the last driven piles; piles fail when the base fails; progressive pile failure reflects the behaviour of the pile base.

+ Schiff⁽²²⁾ found that 25 mm. wide piles driven at $12d$ were still acting in sand influenced by previous pile driving; while Beredugo (3) noted the effects of order of driving on load distribution within a group, to spacings of $9d$ in both loose and dense sand.

As the last driven piles are the most significant, their load-settlement curves in terms of base and shaft, for spacings of $4d$ and $8d$, are given separately (Fig. 7.12).

We may now re-examine the effect of spacing on load distribution at small settlements (Fig. 7.8) in terms of base and shaft load (Fig. 7.13). The figure shows that the pile base load dominates load distribution within the piles (c.f. Fig. 7.8, 7.13). The relation extrapolates to a uniform distribution at approximately $13d$, similar to that shown in Fig. 7.8. It is interesting that there is a slight variation of shaft load within the group which becomes uniform at wide spacings. This is however, insignificant when compared with the base contribution. Fig. 7.13 also shows that the proportion of load carried by the base of the last driven piles is the same, independent of spacing. The centre pile at close spacings however, carries as much in friction as on the base.

7.5 LOAD-SETTLEMENT BEHAVIOUR OF THE WHOLE FOUNDATION

The piles constitute only a part of the foundation. To establish that progressive failure of the piles dominates the behaviour of the foundation, let us examine the load-settlement behaviour of the whole foundation. All pile foundations studied, regardless of spacing (i.e. size of foundation), driving pattern or number of piles, follow the same general load-settlement pattern (Fig. 7.14). The load increases initially steeply, then follows a curve of changing slope until a new relation is established. The curve of changing slope constitutes a partial foundation failure. Clearly it is impossible to pick out unique points on these curves as it was in the analogous case of the single pile. However, the zone of partial failure for all foundations lies approximately between a settlement of 0.005 in. and 0.030 in., as shown. In the larger foundations - particularly the largest - further increases in load give rise to a relation which indicates the importance of sand compressibility. Indeed, the foundations having widths of $33d$ and $25d$ are very far from catastrophic

failure at loads of 1000 lb. and 750 lb. respectively. For the smallest foundation, the zone of partial failure, when considered on a failure envelope criterion, is complete failure.

7.6 LOAD DISTRIBUTION WITHIN A FOUNDATION

Partial failure of the large foundations and complete failure of the small foundation is a manifestation of progressive failure of the piles. That only a partial failure occurs in the large foundations is due to the influence of the cap.

Consider, as the first example, the largest foundation studied. The load distribution within this foundation in terms of the summation of pile loads and cap load is shown in Fig. 7.15. Clearly the piles fail. The effect of this failure on the whole foundation behaviour may be seen. It is useful to refer to Figs. 7.3(A) and 7.4(A) to see the progressive nature of the pile failure. It will be seen that failure of the last driven piles, more specifically, the base of the last driven piles, initiates partial failure of the whole foundation.

Let us now examine the smallest foundation, when there are only 16 piles. The test using a centre to corner driving pattern is shown in Fig. 7.16. The behaviour is similar to the large foundation, but the piles exert a much greater influence on the whole foundation. The failure envelope for the piles is reflected in a foundation failure envelope. In fact, the whole foundation may be considered as one shallow pile. Using a failure envelope criterion, failure occurs at a settlement of approximately 5 per cent d . Most significant in the foundation failure, is the load carried by the base of the last driven piles (c.f. Fig. 7.16 with 7.1(A) and 7.2(A)).

The foundation load distribution for an intermediate spacing ($4d$) is shown in Fig. 7.17. In this case, the behaviour of the piles causes a partial foundation failure. The progressive nature of the pile failure which is again a reflection of the pile base failure may be seen by referring to Fig. 7.6 and 7.10.

So far only examples using a centre to corner pattern have been given. Fig. 7.18 shows the distribution in the largest foundation when a corner to centre pattern is used. In this case, the zone of partial failure is more confined. Comparison with Fig. 7.3(B) and 7.4(B) shows that there is no progressive pile failure. They all fail at approximately the same settlement.

CHAPTER 8

AN EXAMINATION OF TWO POSSIBLE EXPLANATIONS

For the two driving patterns which represent the extreme conditions, it has been shown that load distribution within a group of piles is initially controlled by the order of driving and independent of position. To seek an explanation for this behaviour we must therefore consider effects which may be correlated with the sequence of driving piles.

HYPOTHESIS I : 'Later driven piles carry more load initially because the sand into which they are driven is denser.'

Let us consider single pile behaviour as evidence. The increase in ultimate bearing capacity and compressive load after driving, have been correlated with a decrease in porosity (Fig. 3.6 and 5.8). The greatest change in porosity occurs when the first few piles are driven (Fig. 5.14 and 5.12). If porosity were the controlling factor, one would expect this same pattern to recur when the piles are part of a foundation. The load distribution within the piles for a spacing of $4d$, is shown in Fig. 8.1 as a function of the order of driving. Loads have been taken at a settlement of 0.008 in., where the difference in load between the first and last driven piles is the greatest. It will be seen that the first 12 piles carry approximately the same load; the greatest increase occurs within the last 9 piles. The ultimate bearing capacity of the 12th driven pile (as a single pile) is practically double the first; yet, when part of the group, the load it carries is little greater than the first.

The evidence does not therefore support the above hypothesis and we must seek some other explanation.

HYPOTHESIS II : 'Later driven piles carry more load because the sand under these piles is initially stressed.'

Let us consider once again, piles spaced at $4d$. The 8th driven pile maintains after driving, a compressive load of 0.54 lb. Driving the remaining piles reduces this load to zero (Fig. 5.17); there may in fact be a finite gap below the pile base. It is physically possible for the sand to arch across this distance. The last driven pile however, still maintains a compressive load of 0.78 lb.

Let us now examine the state of stress beneath the base of these two piles. Imagine samples of sand directly beneath the pile bases having the same diameter as the piles and, by analogy with the triaxial test, a height of twice the pile diameter. Fig. 8.2 shows the stresses acting on these samples when all 25 piles have been driven, sample A, directly beneath the 6th driven pile and sample B, directly beneath the 25th. The vertical stress is a contact pressure, which on sample B, requires for equilibrium a lateral stress of certainly more than 4 psi. and probably about 8 psi. The exact values of the stresses are not important when compared with those acting on sample A; for the vertical stress here is zero and the lateral stress approximately 0.2 psi.

Since the foundation is rigid, any given foundation settlement will cause the same strain in both samples. For a settlement of 0.008 in, at which there is the greatest variation in load distribution, both samples must strain approximately $1\frac{1}{2}$ per cent. Clearly, a much greater load is required to strain sample B. Therefore the last driven pile which is compressing this sand, takes much more load.

This hypothesis not only accounts for the load being taken by the last driven piles but also for the low loads taken by the 8th driven as well as the first. When all 25 piles have been driven, the initial stress beneath these piles is practically the same.

The effect of driving a greater number of piles can also be understood. With 16 piles at 2d, the last driven piles carry a load comparable to the last driven of 25 (c.f. Fig. 7.1(A) and 7.5). Fig. 5.12 shows, that when there are only 16 piles, the base load after driving on the 16th is also comparable to the base load of the last driven of 25. That the 16th pile in a group of 25, carries such a small load is a result of the reduction in stress beneath this pile caused by driving the remaining 9 piles.

Although the vertical stress under each pile was not measured after all piles had been driven, it appears that an important relationship exists between this stress and the load-settlement behaviour

of the pile. The initial load carried by each pile is controlled by the stress-strain properties of the sand directly beneath that pile. For a centre to corner driving pattern, settlements which cause the last driven piles to fail are required, before the stress conditions beneath the central piles are greatly influenced by the load carried by the outside piles.

Driving the model piles caused the surface of the sand to heave (Plate 11). Heave is often observed when full scale piles are driven into sand. If the sand contains piles, an upward movement of these piles will occur (Section 1.2). Such a movement could cause a reduction in base load (i.e. reduction in sand stressing).

CHAPTER 9

SUGGESTIONS FOR PRACTICAL DESIGN

Introduction

- Example 1 : Driving Piles in Sand of Uniform Density
- 2 : Driving Piles in Non-uniform Densities
- 3 : The Number of Piles
- 4 : Vertical Stress Distribution
- 5 : Redriving Piles

INTRODUCTION

Terraghi and Peck⁽²³⁾ (P.458) suggest a pile spacing of between $2\frac{1}{2}d$ and $4d$ as the most economical for piles driven in sand. Where piles are driven in clusters, with the structure resting on pile footings, they advocate driving the centre pile of the cluster first (P.469). When the piles are to be spaced uniformly a specific driving pattern is not suggested. Fig. 7.5 and 7.6 reveal that the effect of order of driving is greatest at spacings of from $2d$ to $4d$.

EXAMPLE 1 : DRIVING PILES IN SAND OF UNIFORM DENSITY

Let us consider the foundation shown in Fig. 1.1 and the possible consequences of disregarding the pattern of pile driving. There are many possible driving patterns for these piles. Fig. 9.1 shows one example in which the piles have been driven from one side of the foundation to the other. When the piles are capped and load is applied, there is a likelihood the foundation will tilt, for the last driven piles are concentrated on one side of the foundation.

Piles should be driven from the centre toward the outside of a foundation. Fig. 9.2 represents a pattern which is economically feasible, assuming only one driving rig is available. Moreover, this foundation will be much more stable.

EXAMPLE 2 : DRIVING PILES IN NON-UNIFORM DENSITIES

In practice, deposits of sand are usually variable in density. Using a method of probing, zones of loose sand may be located. Fig. 9.3 shows a possible situation in which a structure is to be located in the position shown. There are many ways in which this foundation may be designed. Let us however, consider the possibilities of using a driving pattern to compensate for the density variation. It has been shown that piles compact sand and that the last driven piles carry more load than the first driven. Using this information the driving pattern shown in Fig. 9.4 is suggested. In leaving the loose zone until the last, not

only will it be somewhat denser than originally, but these piles being the last driven make the most efficient use of what will still be slightly looser sand.

Alternatively, more piles could have been driven or the piles driven to a greater depth. This example has been given to show that the engineer has another, more rational factor at his command in designing foundations on variable sand.

EXAMPLE 3 : THE NUMBER OF PILES

Although one gains in driving an extra pile, it seems also that one reduces the effectiveness of piles already driven. It is useful to compare Fig. 7.1 with Fig. 7.5 from which it can be seen that the contribution of the extra 9 piles was relatively insignificant. In driving 25 piles at $2d$, the first 12 piles were relatively useless. Before a decision is made to drive more piles therefore, one should be aware that in so doing, the potential load carrying capacity of piles already driven is being reduced.

EXAMPLE 4 : VERTICAL STRESS DISTRIBUTION

Different driving patterns can alter the intensity of vertical stress beneath a foundation and therefore the settlement. If the load carried by the top cap and pile shafts is neglected, vertical stress within the sand may be calculated using Mindlin's equation based on the theory of elasticity, for the stress due to a point load in a semi-infinite medium.⁽¹⁸⁾

The largest foundation with the piles spaced at $8d$ provides the best example. Fig. 9.5 shows a sectional view through the corners of this foundation in which the vertical stress contours have been drawn using the actual experimental values. For both driving patterns base loads were taken at a foundation load of 100 lb, at which the settlement for both cases was approximately 0.005 in. It can be seen that the sand is stressed to a greater depth when a corner to centre pattern is used. With a centre

to corner pattern, most of the load is distributed to the sand at the four corners; whereas in the other case, the load is concentrated in the nine central piles making the summation of stress much greater.

With a corner to centre pattern, more load is carried by the pile bases. The following table gives a comparison of the foundation load distribution for the two patterns:

	<u>Centre to Corner</u>	<u>Corner to Centre</u>
Base Load	55 lb.	61 lb.
Shaft Load	16	21
Cap Load	29	18
	$\Sigma = 100 \text{ lb.}$	$\Sigma = 100 \text{ lb.}$

The stress contours in Fig. 9.5 are however, a result of the different load distribution within the piles for the two patterns; that the summation of base load is slightly greater in one case affects the contours very little. The difference in load carried by the cap and pile shafts on the contours is insignificant; indeed, they tend to cancel each other. It should be emphasized however, that Fig. 9.5 does not represent the actual percentage of stress; for, not only have pile shaft and cap load been neglected, but the initial stress due to the base load after driving has not been considered. What is of significance here, is the difference between the two patterns; the latter factor will increase this difference.

Fig. 9.6 shows the stress beneath the centre of the foundation of Fig. 9.5. It can be seen that the sand is more highly stressed at all depths with a corner to centre driving pattern.

Practical application

Although more load is taken by the piles with a corner to centre pattern, Fig. 7.14, reveals little difference in the foundation settlement;

in fact, a slightly greater settlement occurred. The reason lies in the greater stress to which the sand is subjected at any depth⁺.

If uniform sand extends to a great depth, little is gained from the extra force required to drive piles in a corner to centre pattern. Indeed, if a soft clay or silt band exists at depth, (e.g. 60d), much larger settlements may occur, while a reduction in settlement could be achieved if bedrock occurs at the same depth.

EXAMPLE 5 : REDRIVING PILES

The Code of Practice on Foundations (1954, p.67) advises that:

"Piles which have risen as a result of driving adjacent piles should be redriven to the original depth or resistance, unless redriving tests on neighbouring piles have shown this to be unnecessary."

Such a recommendation anticipates order of driving as a significant factor in pile load distribution. The degree of significance of this factor as shown by the present research demands a close examination of the recommendation for piles driven into sand.

The following assumptions are implicit in the recommendation:

- 1) That the load carrying capacity of a pile which has risen on driving adjacent piles will be reduced when part of the foundation;
- 2) That the amount which driving adjacent piles has caused a pile to rise is one valid criterion of the decrease in potential load carrying capacity when part of the foundation;
- 3) That redriving tests on neighbouring piles is another valid criterion of the pile behaviour when part of the foundation;
- 4) That redriving the pile will make it more efficient when part of the foundation;
- 5) That redriving the pile will have little detrimental effect on the behaviour of other piles when part of the foundation.

+ The two tests with 16 piles also show a greater initial settlement with a corner to centre driving pattern for the same reason.

Let us now examine each assumption:

- (1) A decrease in the base load after driving has been related to a reduced pile efficiency when part of the foundation. Fig. 5.17 reveals that driving adjacent piles increased the base load on the 8th driven pile while driving relatively distant piles decreased the base load. Compare the effect of piles 11, 15 and 18 with 10, 13 and 16. The greatest loss in potential load bearing of pile 8 was not caused therefore, by driving adjacent piles.
- (2) An uplift of the pile could account for a decrease in sand stressing beneath the pile and thus, the initial load-settlement behaviour. A correlation involving the actual uplift may not be independent of the order of driving, however.
- (3) It is the state of stress directly beneath each individual pile which has been shown to control pile behaviour. It is doubtful therefore, if redriving tests on neighbouring piles would give a reasonable measure of the behaviour of the risen pile.
- (4) and (5) The results of this research suggest that redriving a pile would increase its efficiency at the expense of other piles. The gain in the load-settlement behaviour of the redriven pile when part of the foundation, may depend on the order of redriving.

While this research neither affirms nor negates any of the above assumptions, the indirect evidence which it provides does not support their validity. In implementing the recommendation in its present form, these assumptions are being accepted. Furthermore, any improvement to the foundation which a particular driving pattern might achieve (as discussed in the previous examples) may well be lost if the piles are redriven.

REFERENCES

1. ALYAHIAK, Z., 1961. Vibration of sands with special reference to the minimum porosity test for sands. Proc. Midland Soil Mech. and Found. Eng. Soc., Vol. 4, p. 37.
2. BENDEL, L., 1948. Movements of a driven in pile when new piles are being driven in (Working of pile groups). Proc. 2nd Int. Conf. Soil Mech. and Found. Eng., Vol.4, p. 190.
3. DEREDUGO, Y.O., 1962. The distribution of load in model piles and pile groups in sand. M.Sc. Thesis, Queens University of Belfast.
4. ELDIH, A.K., 1951. Fundamental factors controlling shear properties of sands. Ph.D. Thesis, University of London.
5. FLEMING, W.G.K., 1958. The bearing capacity of pile groups. Ph.D. Thesis, Queens University of Belfast.
6. FRAZER, A.M., 1957. The influence of stress ratio on compressibility and pore pressure coefficients in compacted soils. Ph.D. Thesis, University of London.
7. HANNA, T.H., 1963. Model studies of foundation groups in sand. Geotechnique, Vol.XIX, n.4, p. 334.
8. KATHEY, B.A., 1960. Contribution to discussion of Meyerhof's paper, 1959. Proc. A.S.G.E. Vol.85, S.H.S. p. 98.
9. KOLBUSZEWSKI, J., 1948. An experimental study of the maximum and minimum porosities of sand. Proc. 2nd Int. Conf. Soil Mech. and Found. Eng., Vol. 1, p.158.

10. KOLBUSZEWSKI, J., 1948. General investigation of the fundamental factors controlling loose packing of sands. Proc. 2nd Int. Conf. Soil Mech. and Found. Eng., Vol.VII, p.47.
11. KOLBUSZEWSKI, J., 1950. Notes on the deposition of sand. Research Vol.3, p.472.
12. KOURDEIN, W.C., 1941. Measurement and geological significance of shape and roundness of sedimentary particles. Sed. Pet. Vol.11, n.2, p.64.
13. LINDICKER, W.Z. Pile bearing increased by close driving. Eng. News Rec., Vol.125, n.5, p.172, and discussions n.11, p.324 and n.21, p.670.
14. LOW, U.I., 1960. Contribution to discussion of Meyerhof's paper, 1959. Proc. A.S.C.E., Vol.86, S.M.3, p.106.
15. MACKAY, R.D., 1962. An investigation of the factors controlling the shear strength of granular materials. Ph.D. Thesis, University of Leeds.
16. MACRAE, J.G. and GRAY, W.A., 1961. Significance of the properties of materials in the packing of real spherical particles. British Journ. of App. Physics, Vol.12, n.4.
17. MEYERHOF, G.G., 1959. Compaction of sand and bearing capacity of piles. Proc. A.S.C.E., Vol.85, S.M.6, p.1.
18. MINDLIN, R.D., 1936. Force at a point in the interior of a semi-infinite solid. Physics, Vol.7, n.5, p.195.

19. PREPPER, A.E.J., 1967. An interim report on a laboratory study of groups of piles in sand. Proc. Indian Soil Mech. and Found. Eng. Soc., Vol.4, p.107.
20. FLANTZIA, G. and KOLET, G.A., 1957. Influence of pile driving on the sounding resistance in a deep sand layer. Proc. 4th Int. Conf. Soil Mech. and Found. Eng., Vol.IX, p.52.
21. RITTENHOUSE, G., A visual method of estimating two-dimensional sphericity. Journ. Sed. Pet., Vol.13, n.2, p.79.
22. SCHIFF, S.G., 1960. Contribution to discussion of Meyerhof's paper, 1959. Proc. A.S.C.E. Vol. 86, S.H.S, p.95.
23. TERZAGHI, K. and PECK, R.D., 1960. Soil mechanics in engineering practice, Wiley.
24. WILTAKER, T. and COOK, R.W. A new approach to pile testing. Proc. 5th Conf. Soil Mech. and Found. Eng. Vol.XI, p.171.
25. YAGGIN, A.A., 1960. Model studies of piles. Ph.D. Thesis, University of London.

APPENDIX AEXPERIMENTAL CONDITIONS AND SUGGESTIONS FOR FURTHER WORK

The pattern of behaviour described is only strictly valid for the conditions of the experiment. These conditions are:

- 1) The initial porosity of the sand was always 42 per cent (medium dense).
- 2) All tests were carried out with $1/4$ in. dia. piles at a depth of $7\frac{1}{4}$ in. (29d).
- 3) All piles were pushed to this depth at a relatively constant rate of penetration.
- 4) The surface of the piles was smooth.
- 5) The results are based on rigid foundations containing 25 and 16 piles.
- 6) Two driving patterns were used.
- 7) No piles were redriven.
- 8) They were model experiments.

Of these conditions, initial porosity, number of piles and driving pattern are considered to warrant study first.

The heave caused by driving piles could account for the low loads which the first driven piles carry, when part of the foundation. In loose sand however, a general subsidence occurs during driving. What effect does this have on previously driven piles and therefore the load distribution within the group?

Fig. 5.17 shows that the driving of some piles slightly increased the base load on pile 8. Is there a driving pattern and possibly a number of piles which would decrease the base load on the first driven piles less than the patterns used in the experiments, therefore making these piles more efficient when part of a group?

A fundamental study of load distribution, in terms of the base load on each pile, after all piles have been driven would give answers to these questions. It would undoubtedly also provide sufficient data for a theoretical approach to the problem. The author would like to see any field or laboratory study on driven piles in sand related to the movement which driving a pile causes to piles already driven. This would provide a link between laboratory and field results.

Much valuable model work can be carried out without piles which measure base and shaft load separately. Fig. 7.14 which shows the load-settlement behaviour of the entire foundation could have been obtained more easily with solid piles. A study of the entire foundation behaviour using the driving patterns suggested, with depth, porosity and number of piles as variables, would be extremely valuable for practical design purposes.

APPENDIX BLOAD CELL BEHAVIOUR DURING SINGLE PILE TESTS

During a single pile test, the load measured by the proving ring is equal to the load recorded by the top load cell in the pile. Fig.B.1 shows the correlation between these two measurements for single pile tests at 2d spacing. Applied load has been plotted in terms of proving ring divisions as accurate calibration is impossible with a zero tensile load of $2\frac{1}{2}$ lb.

Included in the figure are two errors other than the load cell, and therefore not included in any of the single pile results.

1) Some load, the actual amount varying from test to test, is taken in friction between the pile head and the guide plate (Fig.5.6).

2) The proving ring which has a calibration of approximately 0.04 lb/div., is known to be unreliable.

The accuracy of the top cells is therefore somewhat better than this correlation indicates.

APPENDIX C

REPRODUCIBILITY OF GROUP TESTS AND THE ERROR IN AVERAGING CORNER PILES

Of the 13 tests carried out, test 8 was a repeat of test 2 and test 13 a repeat of test 7. Thus, not only is a check on the testing technique provided but the effects of any change in the sand* may also be seen (Fig. 3.7).

Tests at 2d spacing (test 7 and test 13) will be considered in terms of errors in load and errors in settlement. Fig. C.1 and C.2 show added pile load as a function of the added foundation load. Comparison of the two figures reveals little difference in the results. Fig. C.3, which represents the two foundation load-settlement curves also shows little error. In all three figures, it should be noted that readings are more closely spaced at small foundation loads where pile behaviour is significant. The results presented in Chapter 7 for a spacing of 2d are an average of these two tests.

Before considering the tests at 4d spacing (tests 2 and 8), let us examine the errors which arise from averaging the 4 corner piles in one foundation (test 8). As was discussed in Chapter 6, this method of analysis averages errors in porosity, load cell behaviour and eccentric loading. A small range of the variable order of driving is also averaged. Once again, load and settlement will be considered separately. Fig. C.4 shows the individual load on the corner piles as a function of added foundation load. It can be seen that the error from the average is as large as ± 20 per cent in some cases. In Fig. C.5 the error due to settlement

* The correlation shown in Fig. 3.7 is only possible because of the very great sensitivity of ultimate bearing capacity to changes in the sand; no correlation exists using a direct measure of porosity.

is shown. The points in the figure represent the actual settlement of the corner piles at each average load⁴. The curve designated as 'average curve' is the average settlement for the 5 piles. As the settlement of only 3 corners of the foundation was measured, the average of the 3 points may not lie on the curve (e.g. at 7.1 lb.). It can be seen however, that this error is insignificant. The figure also shows a slight but significant difference in the actual settlement of the corners. To see what the total error due to these two factors is, the actual load and settlement of each corner pile has been plotted in Fig. C.6. The error can be seen to be quite small. The 20 per cent error noted in Fig. C.4 is due to eccentric loading; for a corner pile carrying a smaller load than the average at any foundation load, also settled less (Fig C.5). Thus, when individual load and settlement are considered, the relations become very similar (Fig. C.6). Two methods of analysis therefore, are equally valid; either the actual load and settlement of each corner pile (points in the figure) or the average corner pile load and average settlement of the foundation (solid line in the figure). Any one corner pile load considered with respect to an average foundation settlement will however, be subject to the same 20 per cent error shown in Fig. C.4.

Let us now examine the reproducibility of the results; Fig. C.7 shows the load-settlement curves for the two tests. It can be seen that the initial load-settlement behaviour is very similar, the actual failure load being greater however, in test 8. That this error, which was greatest in the corner piles does not invalidate the results may be seen by comparing Fig. C.7 with Fig. 7.6.

Evidence to support the accuracy of any test results may be obtained by considering the error involved in drawing correlations. Consider for example, Fig. 7.8 and 7.13 in which the effect of pile spacing is shown. The small scatter in results tends to confirm the validity of any one test.

⁴ The points do not appear at exactly the same load because of lack of space.

APPENDIX DTOP CAP FLEXIBILITY

The top caps for foundations with piles spaced at 2d and 4d were 1 in. thick mild steel, while the 6d and 8d caps were $1\frac{1}{8}$ in. and $1\frac{1}{2}$ in. thick duralumin. For the repeat tests at 2d and 4d, 2 in. and $1\frac{1}{2}$ in. thick mild steel caps were used. As each cap has 25 holes of $1/8$ in. dia. equally spaced, the deflection under load cannot easily be calculated. The flexibility of each cap was therefore measured, under the loading conditions shown in Fig. D.1. Of significance in the experiments is the deflection under loads whose order of pile driving is the important factor. At a load of 150 lb., the deflection of the largest top cap is 4×10^{-5} in. In a total settlement at this foundation load of 0.010 in., this deflection is negligible. Indeed, the deflection measured under the loading conditions of Fig. D.1 is greater than can occur in the experiment. The deflection of each cap was of the same order of magnitude as the example given. All top caps therefore, may be considered rigid.

APPENDIX ECONTAINER EFFECT

In model pile tests, it is necessary to use a container of sufficient size that the results are not influenced. The diameter of container required, is also of some fundamental and practical importance for it indicates the zone of sand affected by driving a single pile.

As a measure of the container effect, let us first consider the failure envelope for a 1/4 in. dia. pile driven at a constant rate of penetration from surface. The pile was driven into containers of $7\frac{3}{4}$ in., 10 in. 12 in. and 14 in. dia. The sand in each case was dense (37.9 $\pm$ 0.3 per cent porosity). These tests were carried out before the entire $1\frac{1}{2}$ tons of sand had been obtained. Although the grading was 52/100 D.S.S., maximum porosity was 47* per cent as compared with 49.3 per cent for the group tests and the ultimate bearing capacity at any porosity slightly greater than that obtained in the experiments. At any porosity therefore, a container effect will undoubtedly occur at a greater distance with this sand than with the sand used in the experiments. The results of the four tests are shown in Fig. E.1. It can be seen that a container 12 in. dia. is required to ensure no increase in load. This corresponds to a radius of $24d^{\frac{1}{2}}$.

As another measure of the zone of sand affected by driving a pile and therefore the size of container which is required, the lateral extent of the surface heave was measured. The 1/4 in. dia. container was initially deposited leaving about 1/4 in. of sand on the top. The excess

*- This was the same sand used for the triaxial tests.

* Using the same criterion, Yassin obtained a container effect for dense sand, at a radius of $12d$.

was carefully cut off by drawing a straight edge across a predetermined direction. After a pile of $1/4$ in. dia. had been driven at a relatively constant rate of penetration to a depth of 7 in., the straight edge was again slowly drawn across the top of the container in the same direction as before, until the edge of the heave was encountered. The distance from the edge of the heave to the centre of the pile was then measured. The results for three porosities are shown in Fig. E.2. This simple procedure slightly underestimates the effect; for, a radius of $17d$ was measured with dense sand in comparison with about $24d$ in the previous test. Of significance however, is the smaller radius required for looser sand.

The results of these tests indicate that a 3 ft. dia. container is sufficiently large for the medium dense sand (porosity = 42 per cent) used in the experiments.

In driving a pile from surface a rigid base causes an increase in pile load when at a distance of about 1 in. ($4d$) from the toe of the pile.

APPENDIX FAIR CURRENTS WITHIN THE CONTAINER
DURING SAND DEPOSITION

When the rain of sand falling from the hopper is positioned about 12 in. from each end of the container, air currents are formed. These currents flow along each side of the container toward the nearest end; and, if unrestricted, meet at the end reflecting on the central part of the rain itself. The resulting disturbance decreases the intensity of the rain, thus decreasing the porosity.

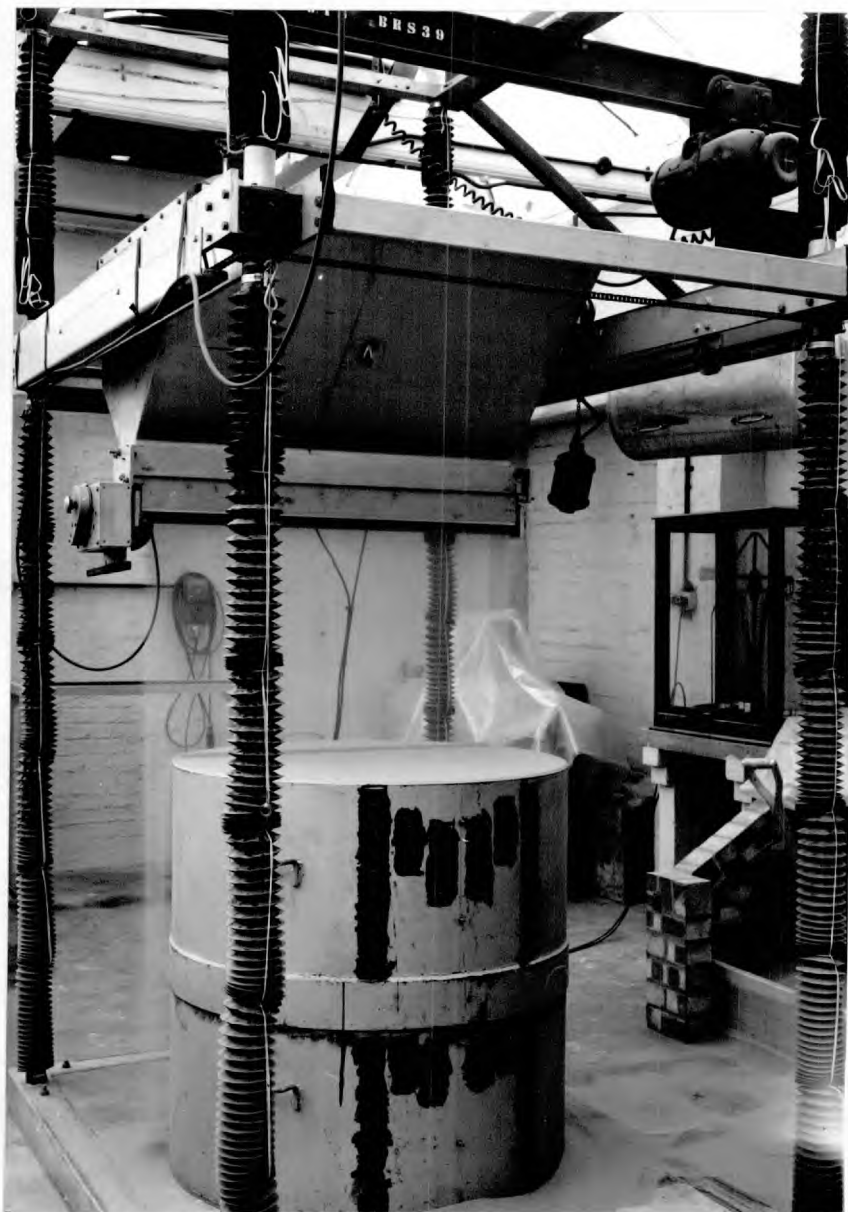
Fig. F.1 shows the corrective measures taken to prevent this disturbance to the rain. Two narrow baffle plates guide some of the air through holes in the container. The air which continues past these holes is prevented from interfering with the depositing sand, by a wide plate positioned parallel to the rain. The air is thus guided out the top of the container. The zone immediately around the plates however, was denser by approximately 0.7 per cent porosity.

APPENDIX GLOAD-SETTLEMENT BEHAVIOUR OF FOUNDATIONS
WITH AND WITHOUT PILES

A series of tests on the four caps was carried out at an initial porosity of 42 per cent. Each cap was formed in exactly the same manner as when piles had been driven (i.e. the sand was sprayed and the metal cap bedded on plaster). The testing procedure was also the same as for the pile foundation tests.

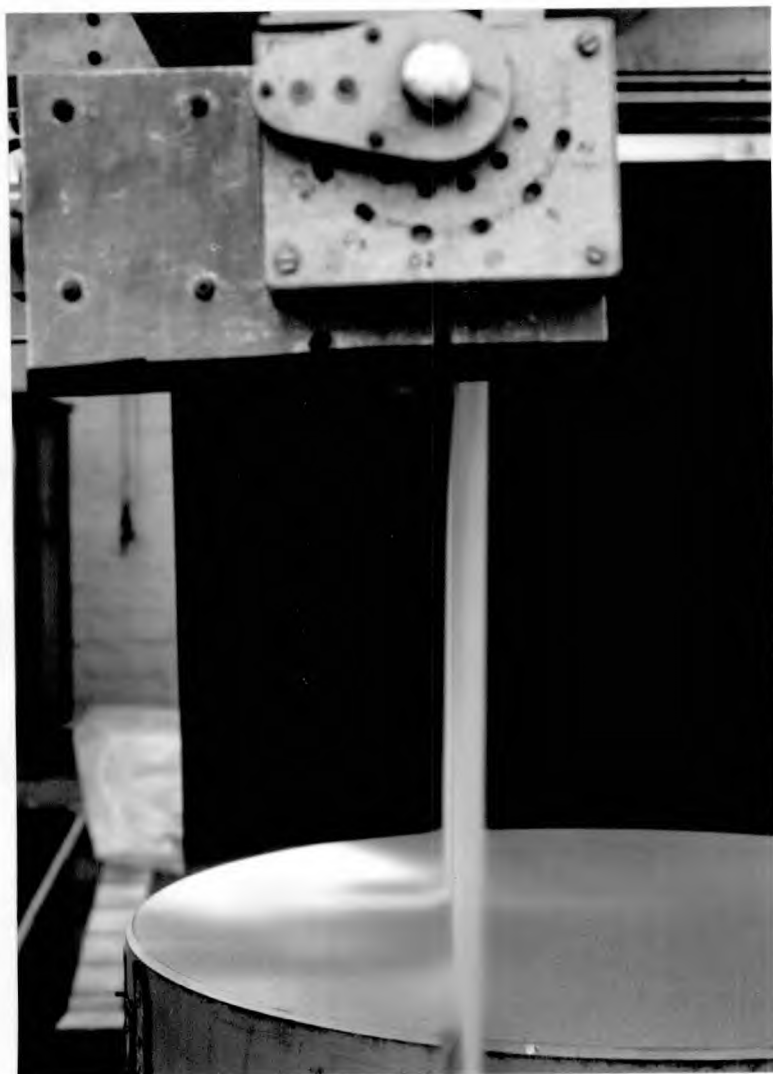
A comparison of the load-settlement curves for the two series of tests (Fig. G.1) shows the influence of the piles. The existence of driven piles alters the load-settlement behaviour of a foundation in two ways: 1) Some load is carried by the piles, 2) the remainder of the foundation load bears on sand which has been changed by pile driving. The effect of the load carried by the piles is clearly seen in the figure. An initial linear load-settlement relation for the cap alone is changed into one in which the settlement increases more slowly before the piles fail. With failure of the piles, the relation for the largest pile foundation becomes comparable with that for the cap without piles; while the smallest foundation continues to carry more load, but the greatest reduction in settlement is achieved before the piles fail.

The influence of the different porosity (probably looser) beneath the remainder of the foundation is of negligible effect in all cases before the piles fail. It may however, account for the slight increase in settlement of the largest pile foundation at settlements when the cap load is important.



Sand depositing apparatus in operation

Plate 1

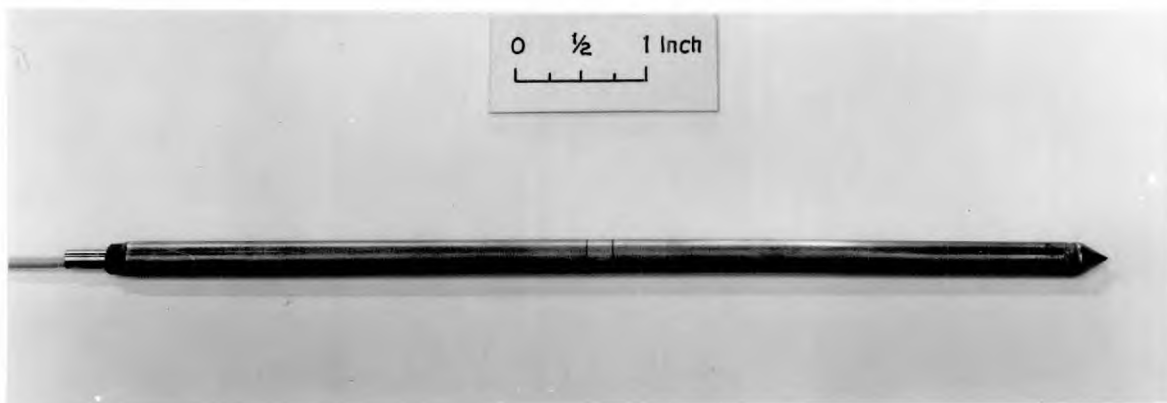


The deposition of loose sand

Plate 2



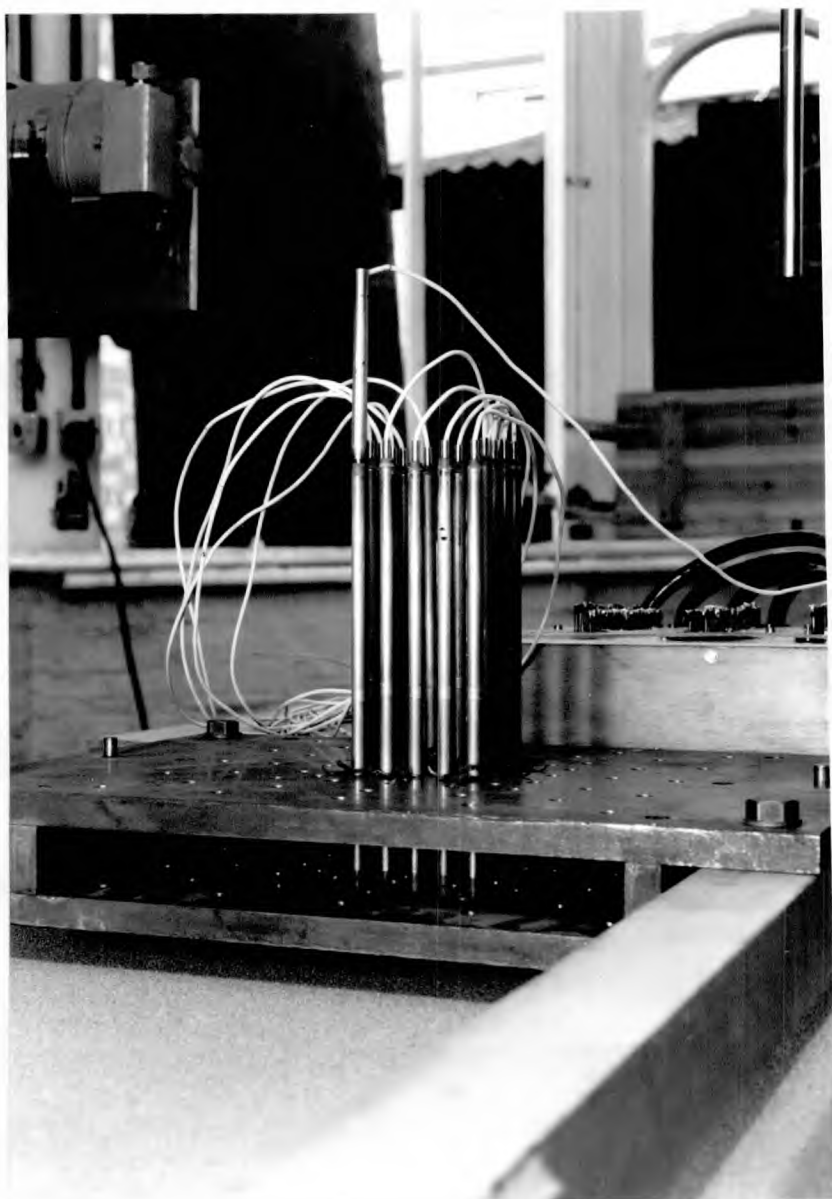
The deposition of dense sand



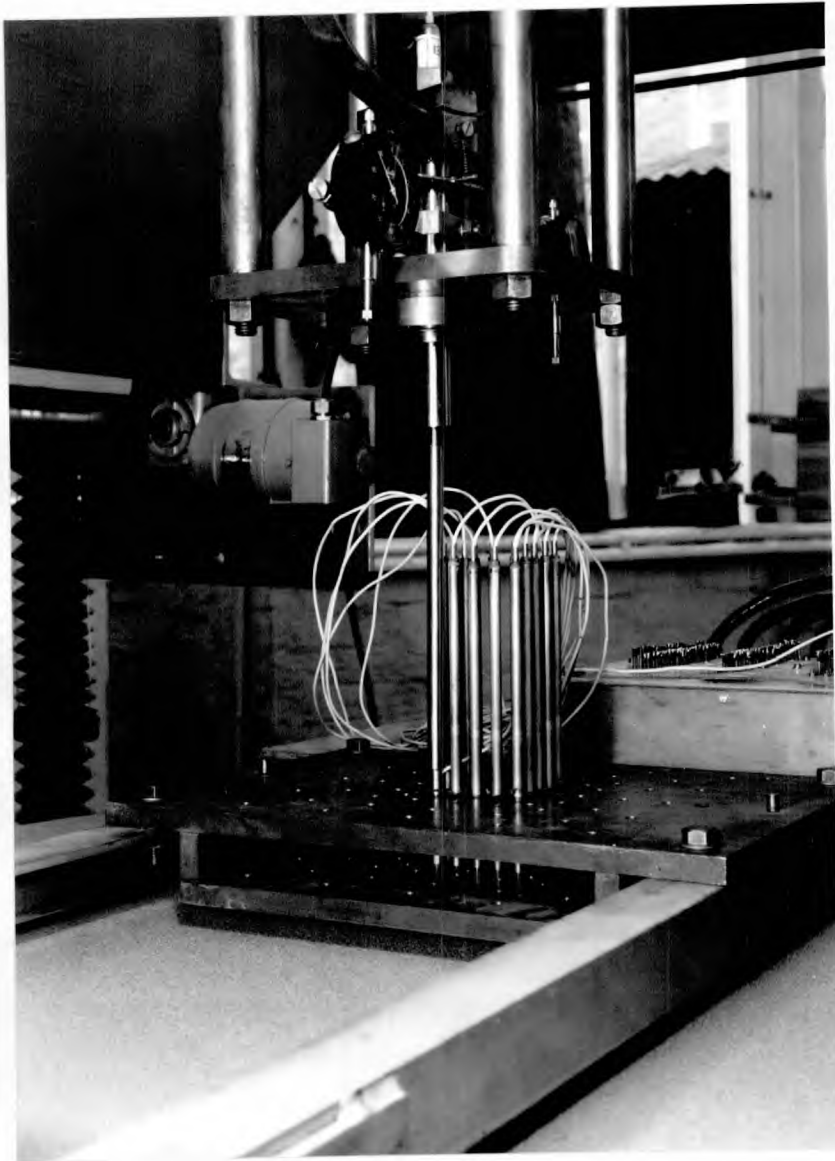
(a) Complete pile



(b) Outer tube removed, showing base load cell
and method of load application

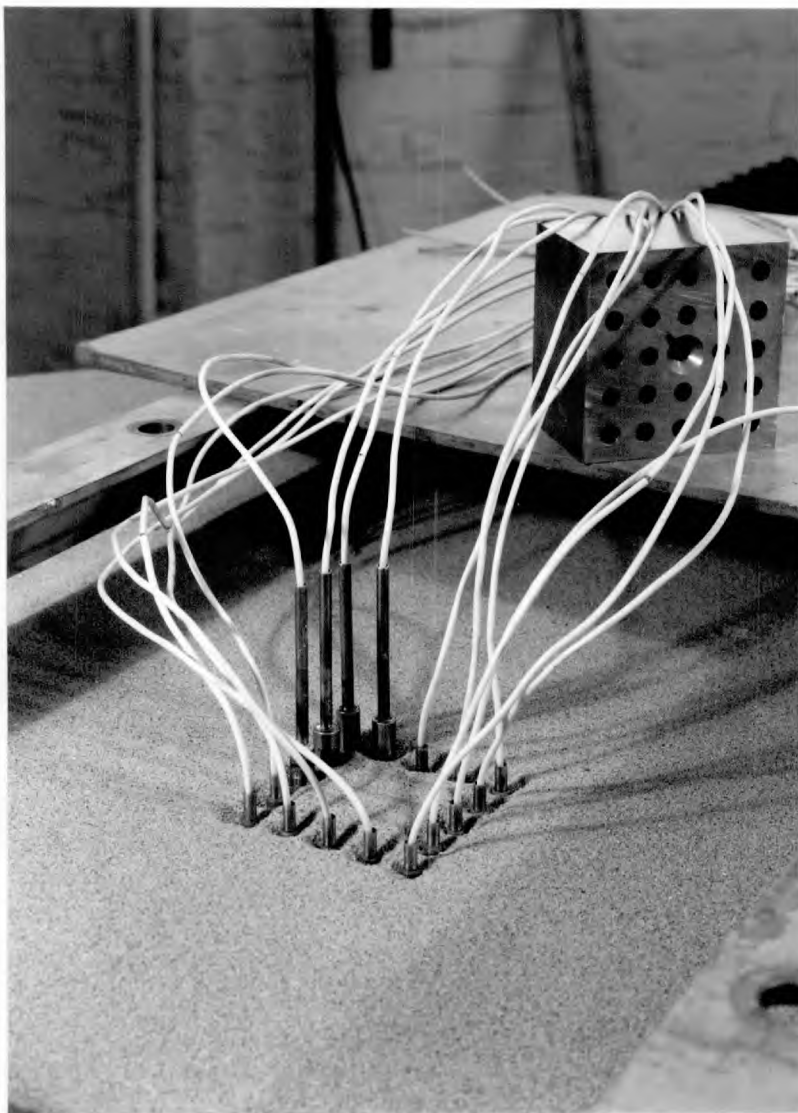


16 piles held in guide unit, free of sand;
corner pile, with driving extension in place
and load cells wired, is ready to be driven.



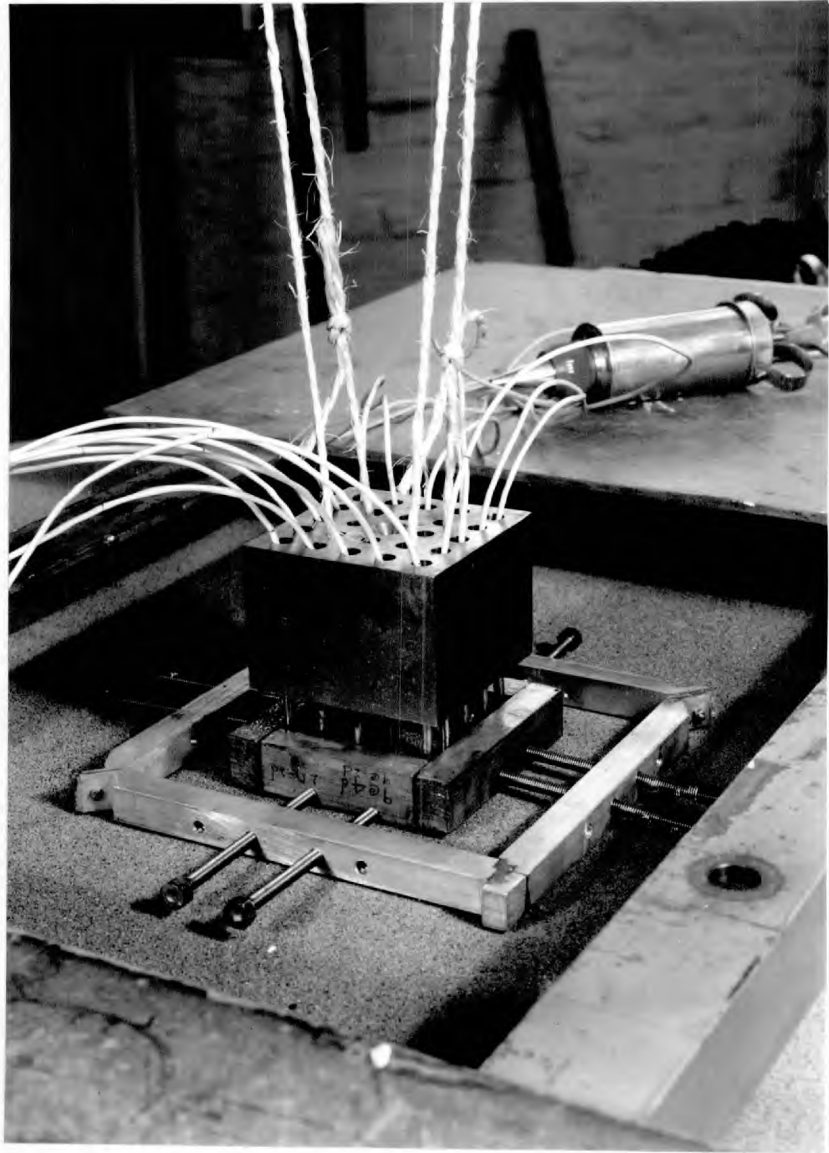
Single pile test on first driven pile

Plate 6



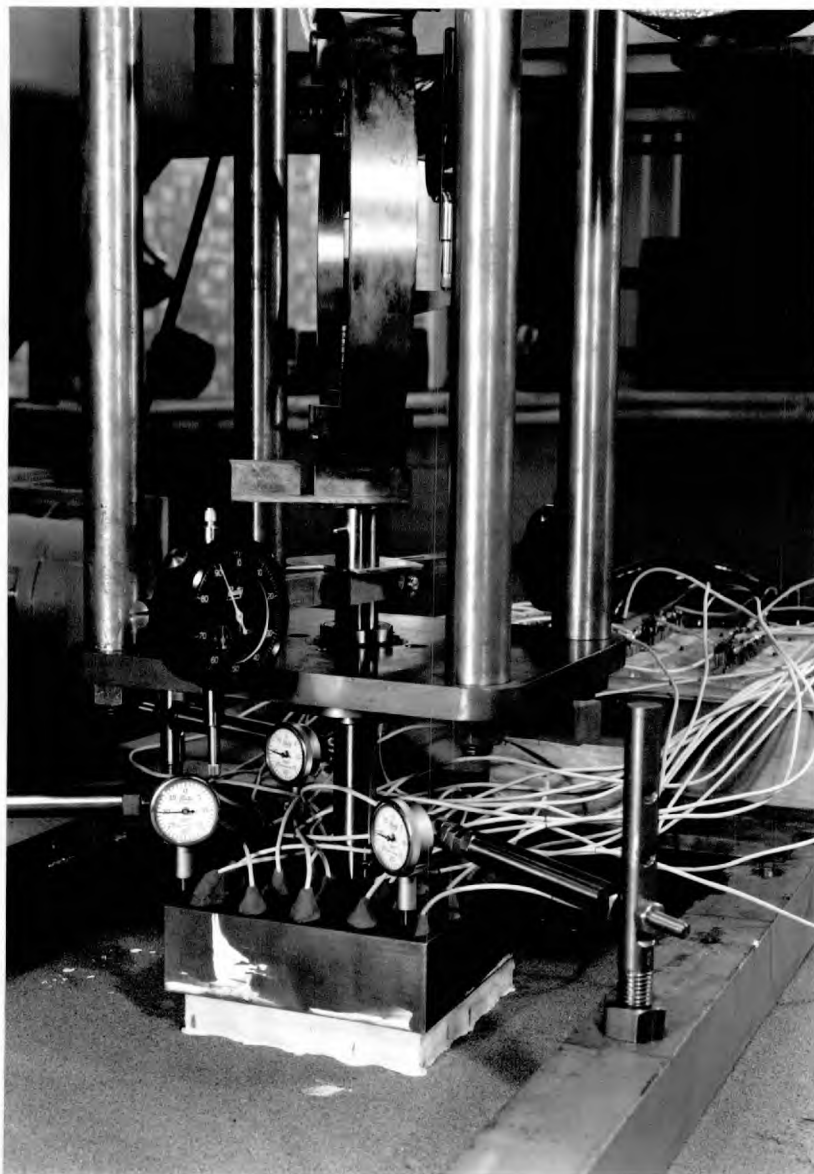
16 piles after driving, with some of
the pile heads placed in position

Plate 7



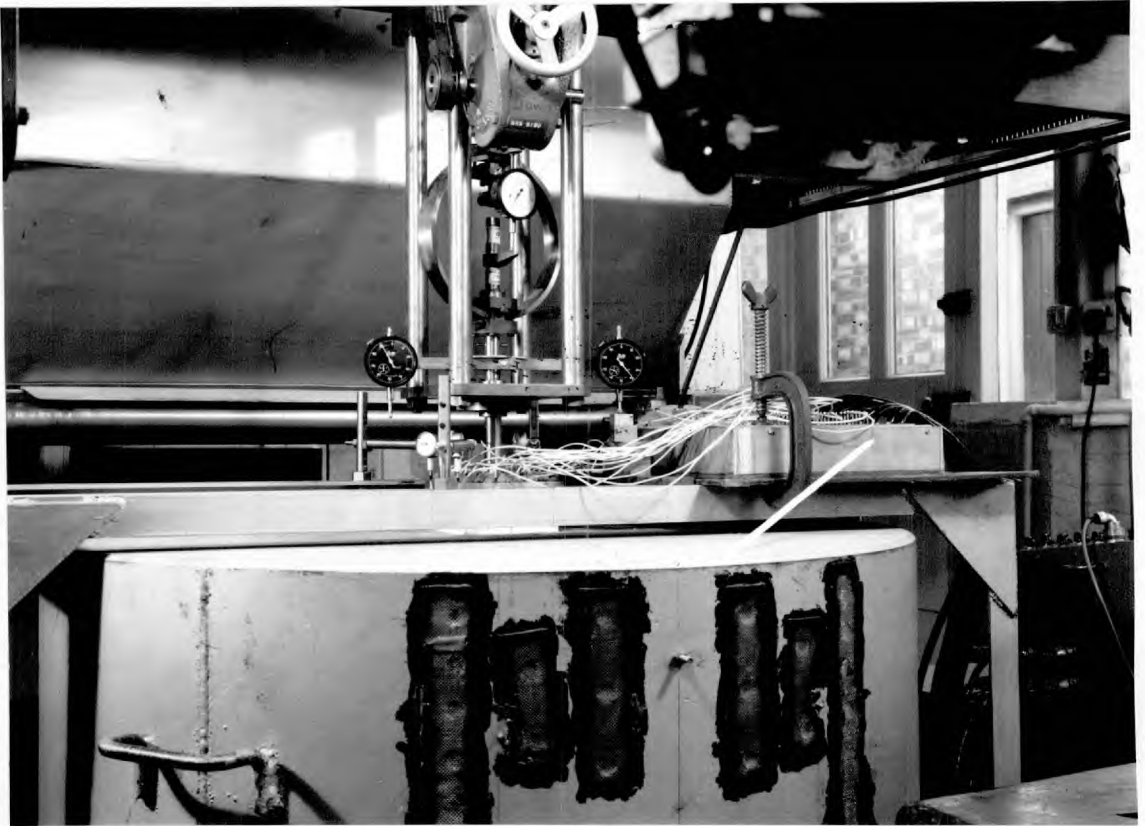
Arrangement for casting the cap

Plate 8



The pile foundation ready for testing

Plate 9



General arrangement for a foundation test

Plate 10



Heave of sand on driving 16 piles, spaced at $2d$;
the sand surface of Plate 7.

Plate 11

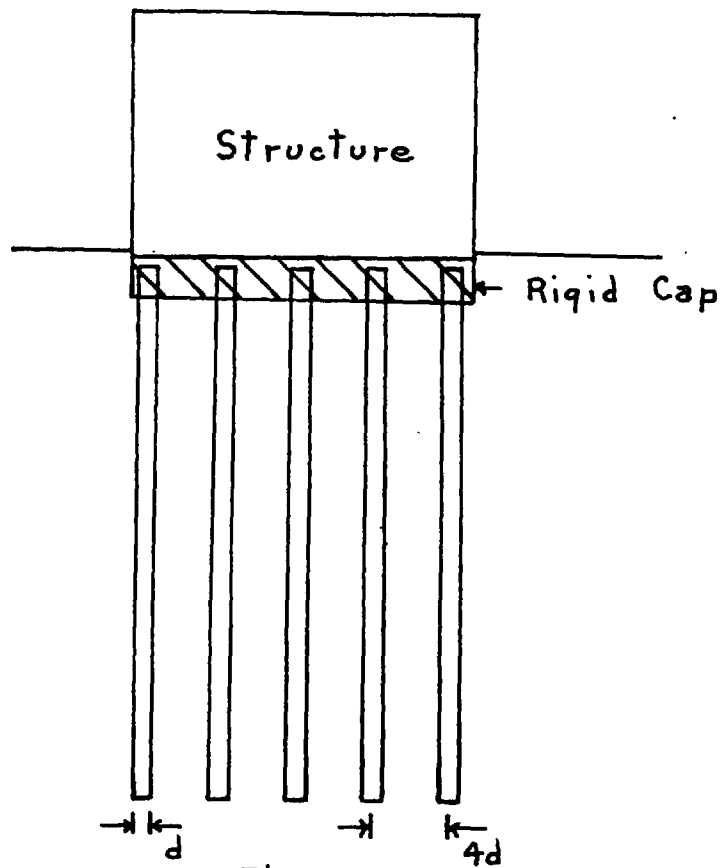


Fig. 1.1

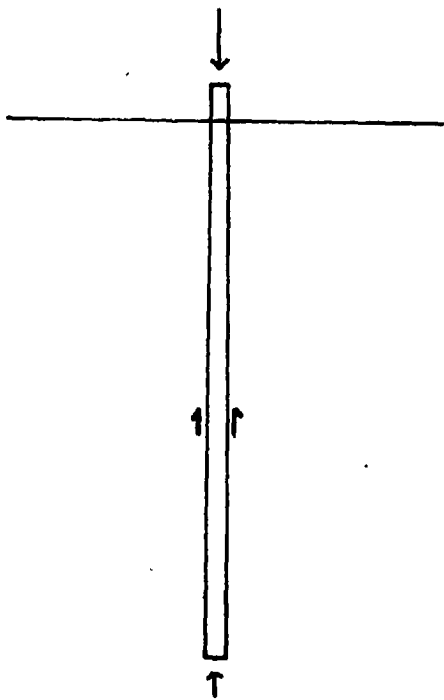


Fig. 1.2

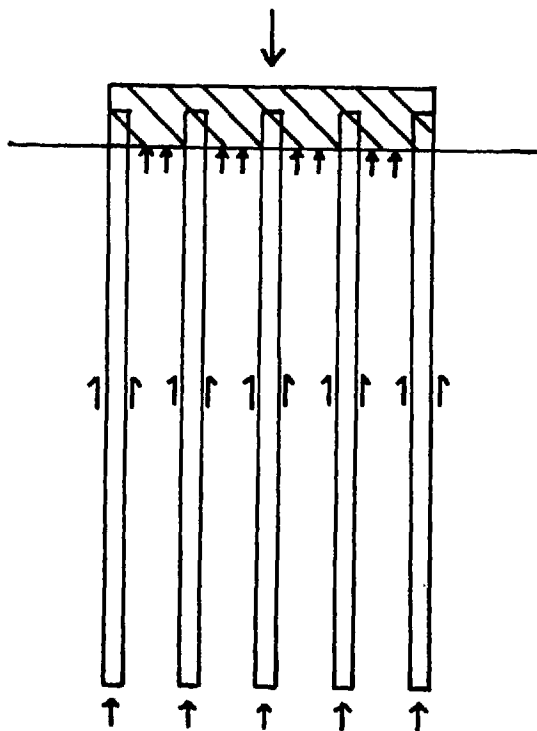
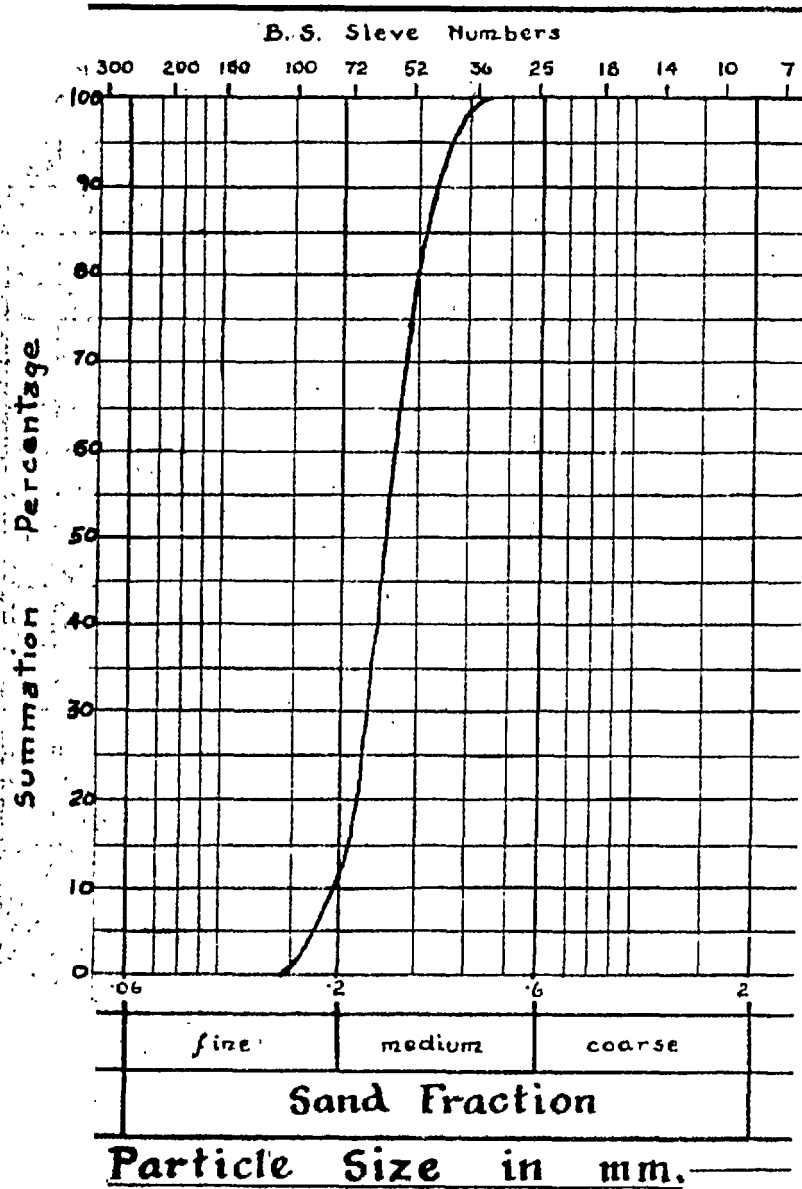


Fig. 1.3

Particle Size Distribution

FIG. 2.1



MAXIMUM ANGLE OF SHEARING RESISTANCE

V_{r0}

INITIAL POROSITY

FIG-2.2

Resistance (degrees)

Maximum Angle of Shearing

— 4 in. Diameter Compression Tests

— $\sigma_3 = 30$ psi

— air dry

— drained

48
46
44
42
40
38
36
34

40

41

42

43

44

45

46

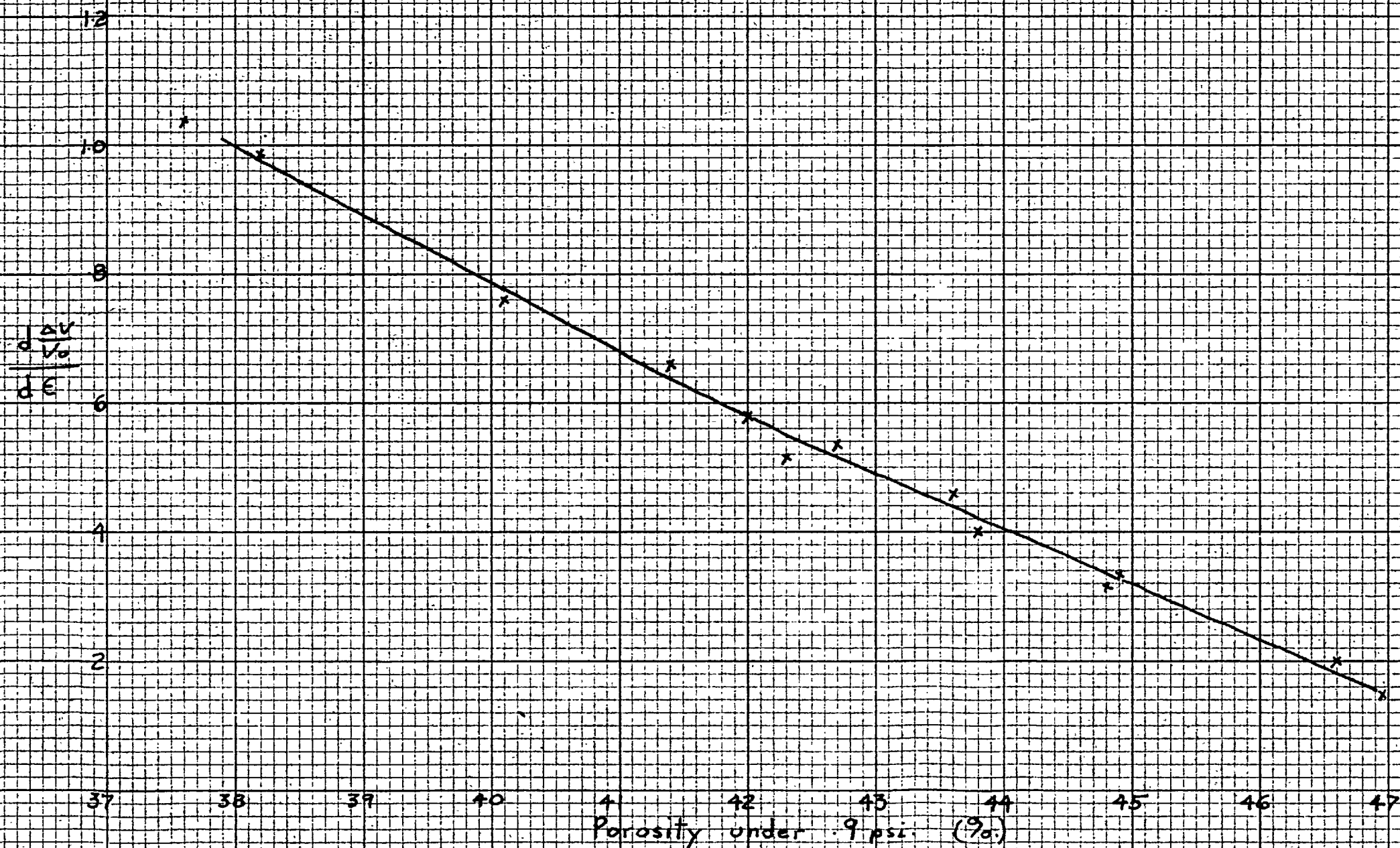
47

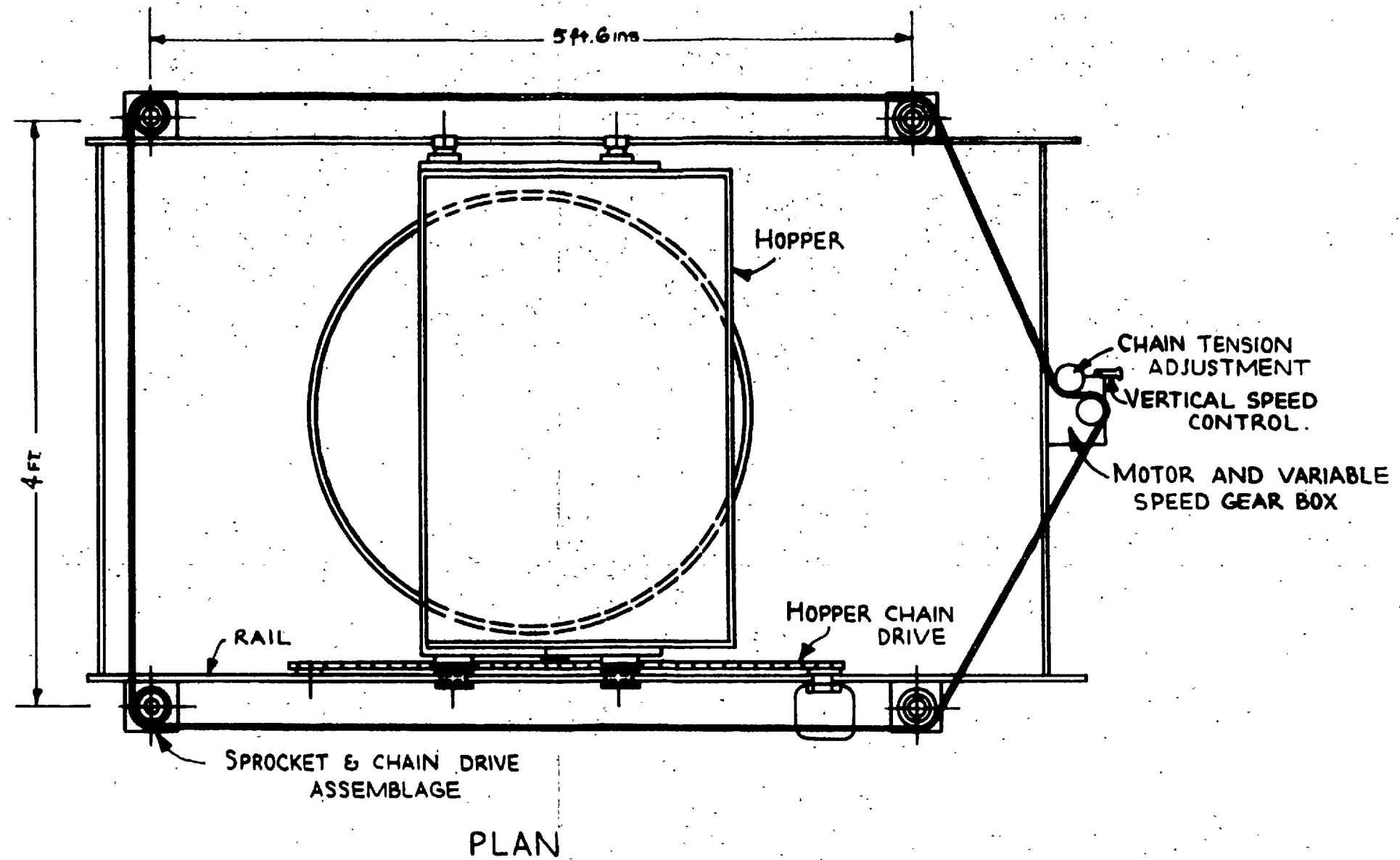
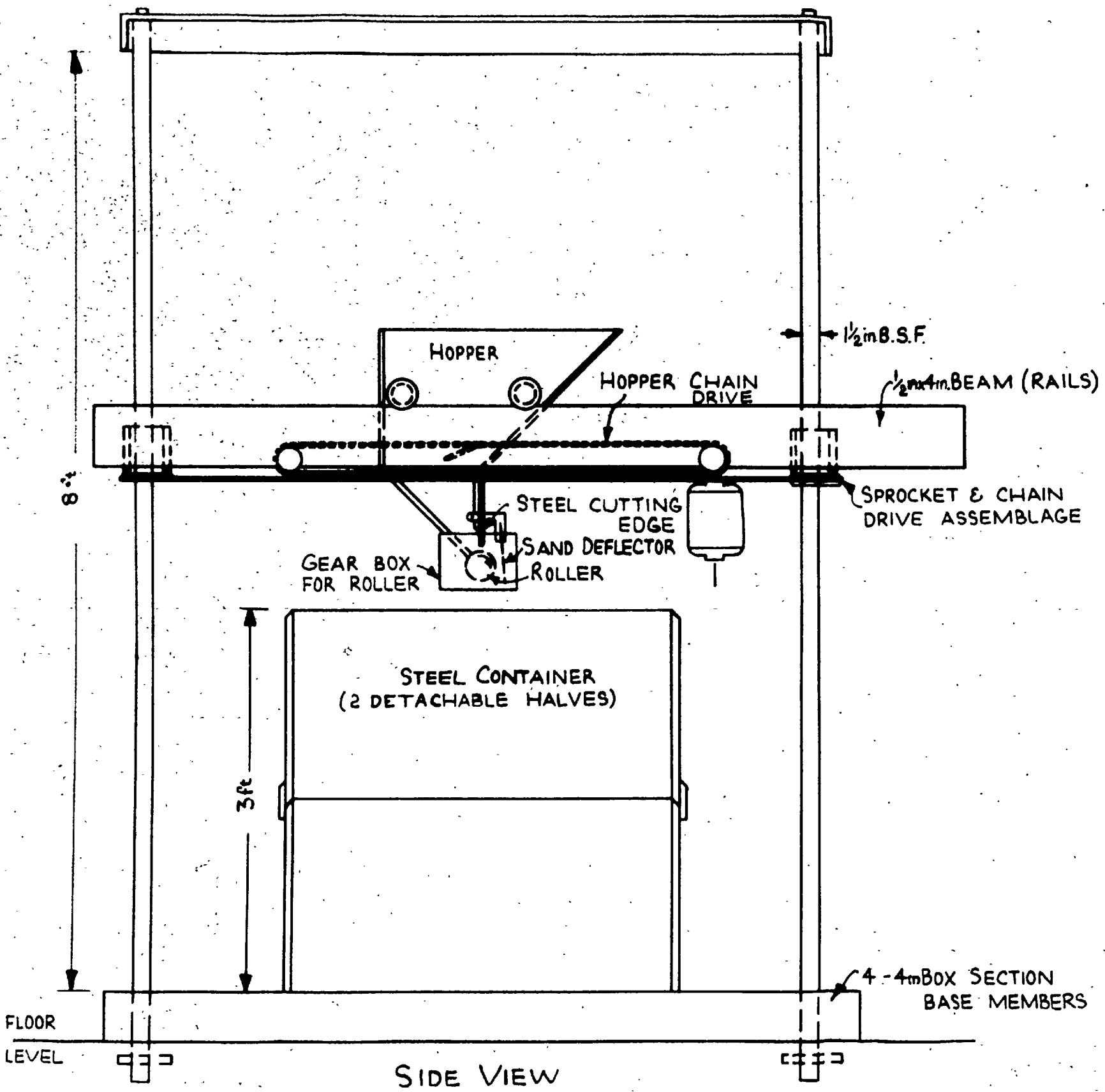
Porosity under .9 psi

(%)

INITIAL POROSITY vs. RATE OF CHANGE OF DILATION AT FAILURE

FIG. 2.3





SAND DEPOSITING APPARATUS

FIG. 3.1

SAND DEPOSITING APPARATUS

FIG. 3.3

Porosity vrs. rate of deposition.

Porosity (%)

50

48

46

44

42

40

38

0

.2

.4

.6

.8

1.0

1.2

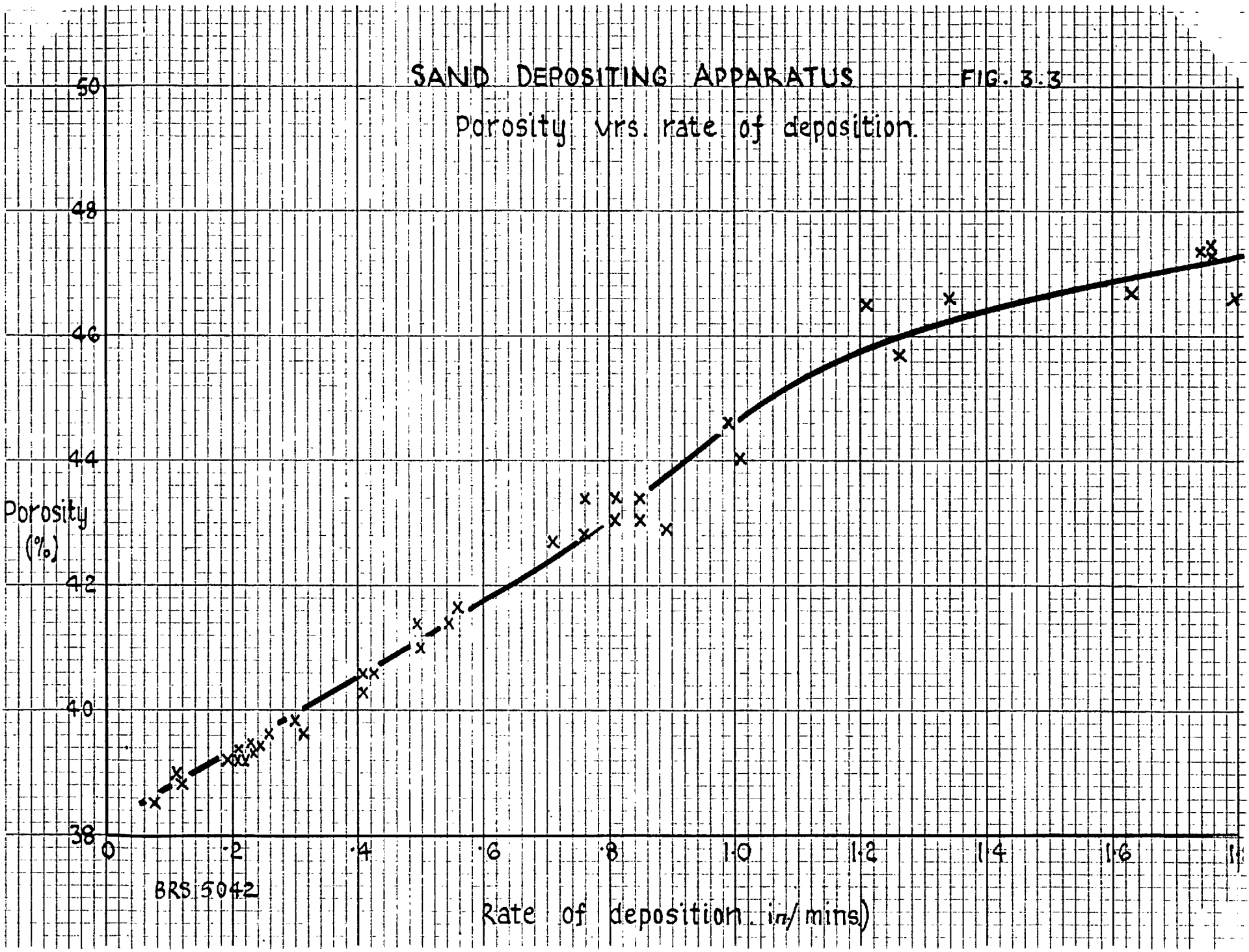
1.4

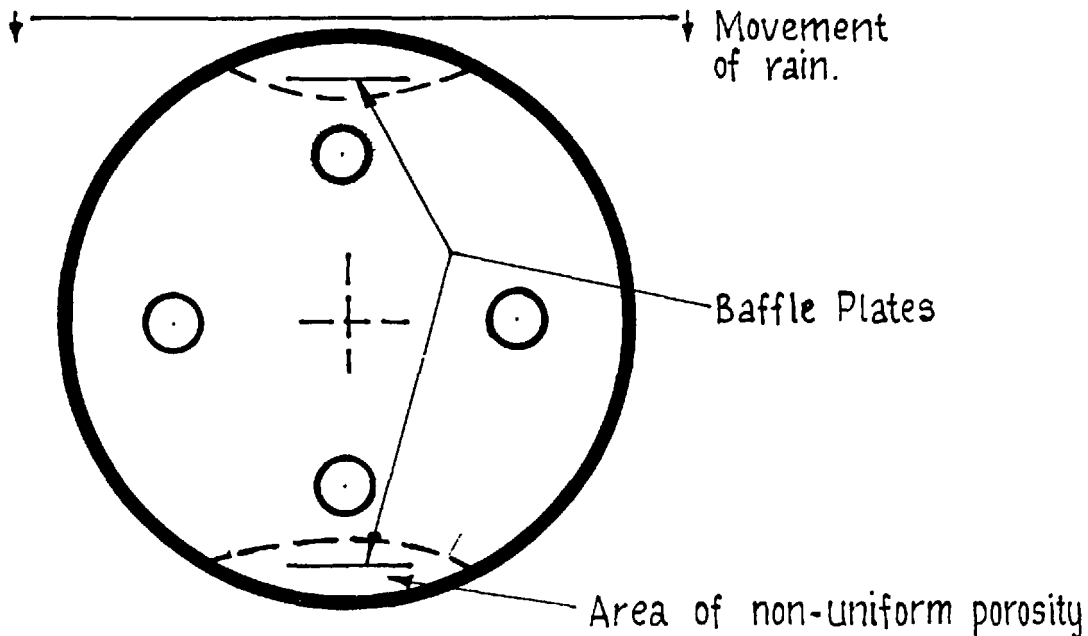
1.6

1.8

BRS 5042

Rate of deposition (in/ mins)

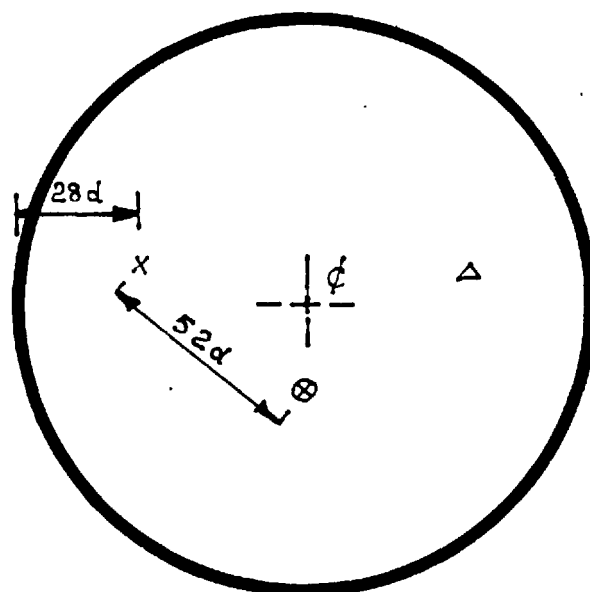




POSITION OF MEASURING CYLINDERS
WITHIN CONTAINER

FIG. 3.4.

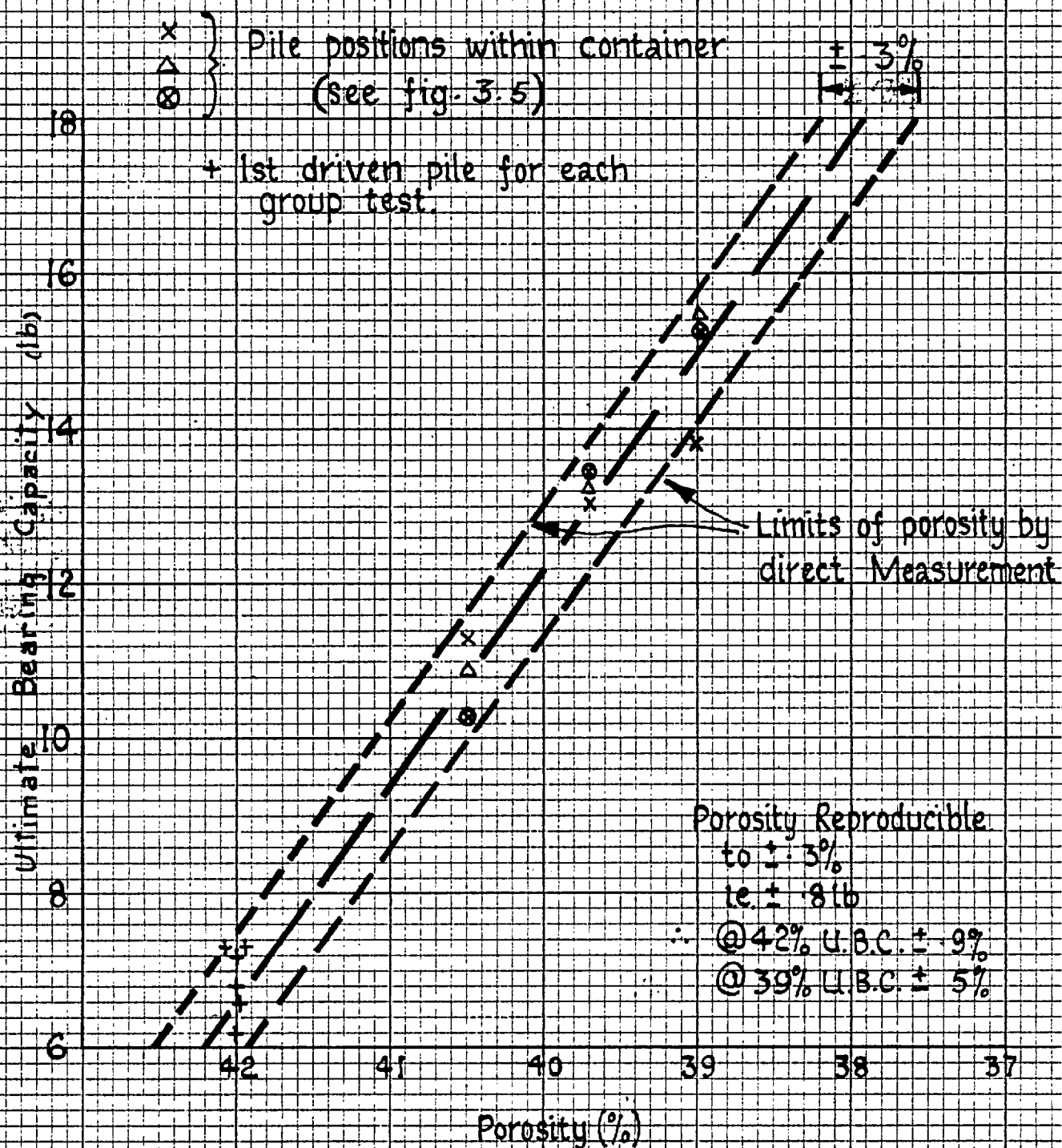
1 ft.



$28d = 7''$

POSITION OF SINGLE PILE
TESTS WITHIN CONTAINER

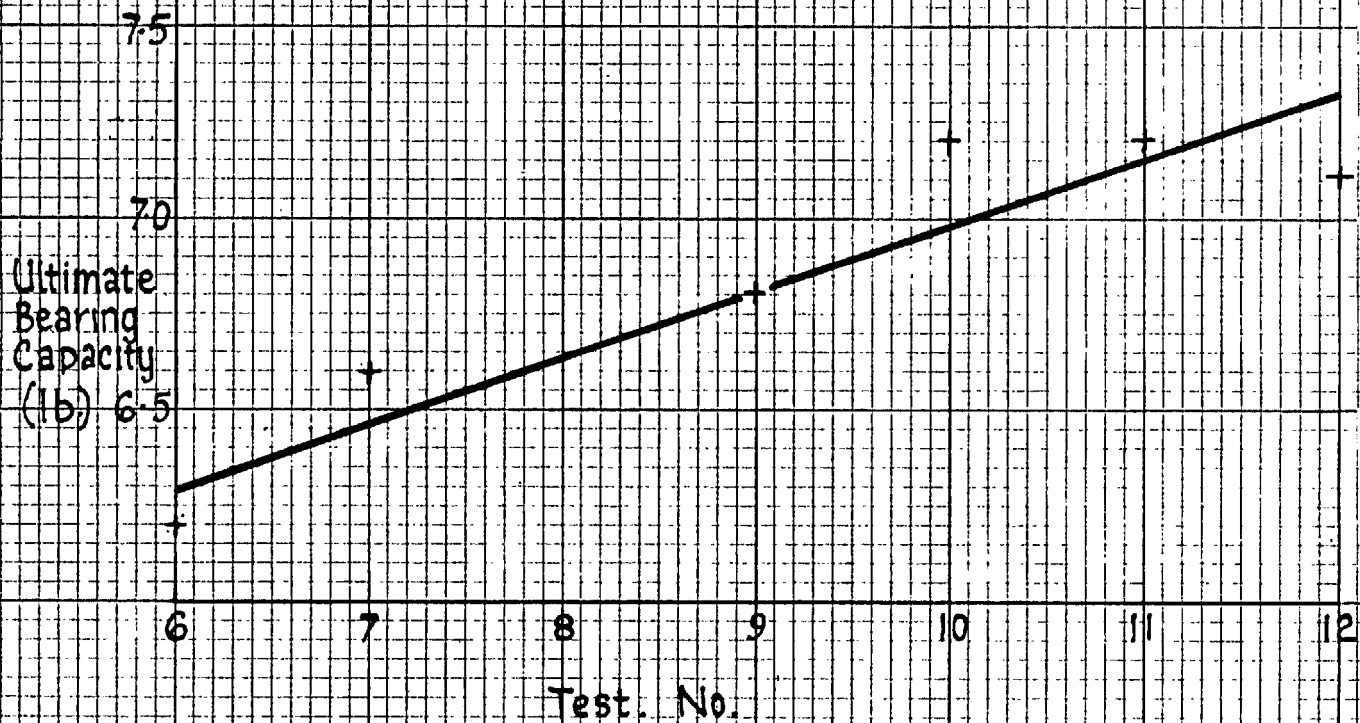
FIG. 3.5

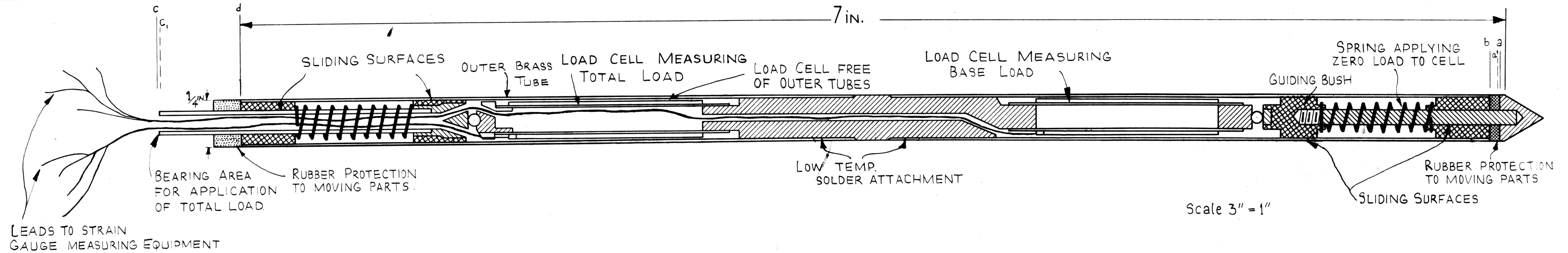


ULTIMATE BEARING CAPACITY OF SINGLE
 PILES IN SEMI-INFINITE SAND VRS. POROSITY
 FIG. 3.6

ULTIMATE BEARING CAPACITY OF
FIRST DRIVEN PILE vrs. TEST No.

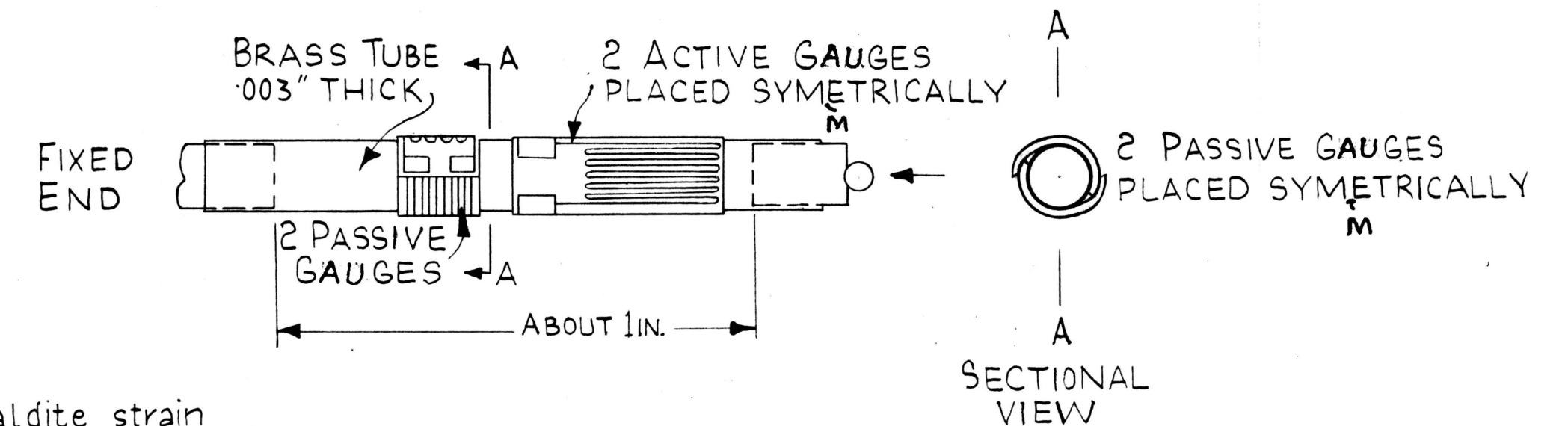
FIG. 3.7





 D.Q.I.
 BRASS
 HARDENED STEEL.

MODEL PILE FIG. 4.1(B)

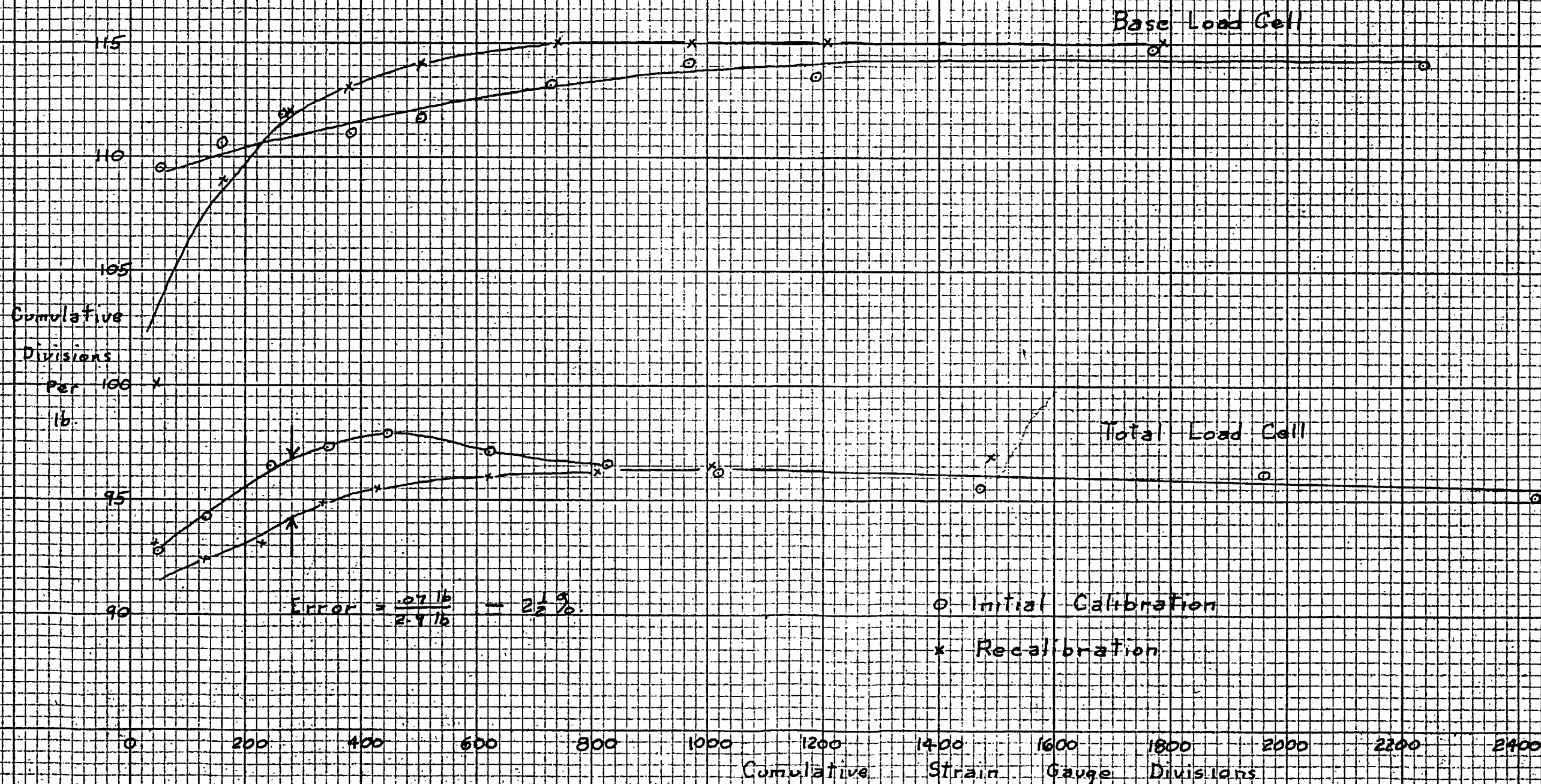


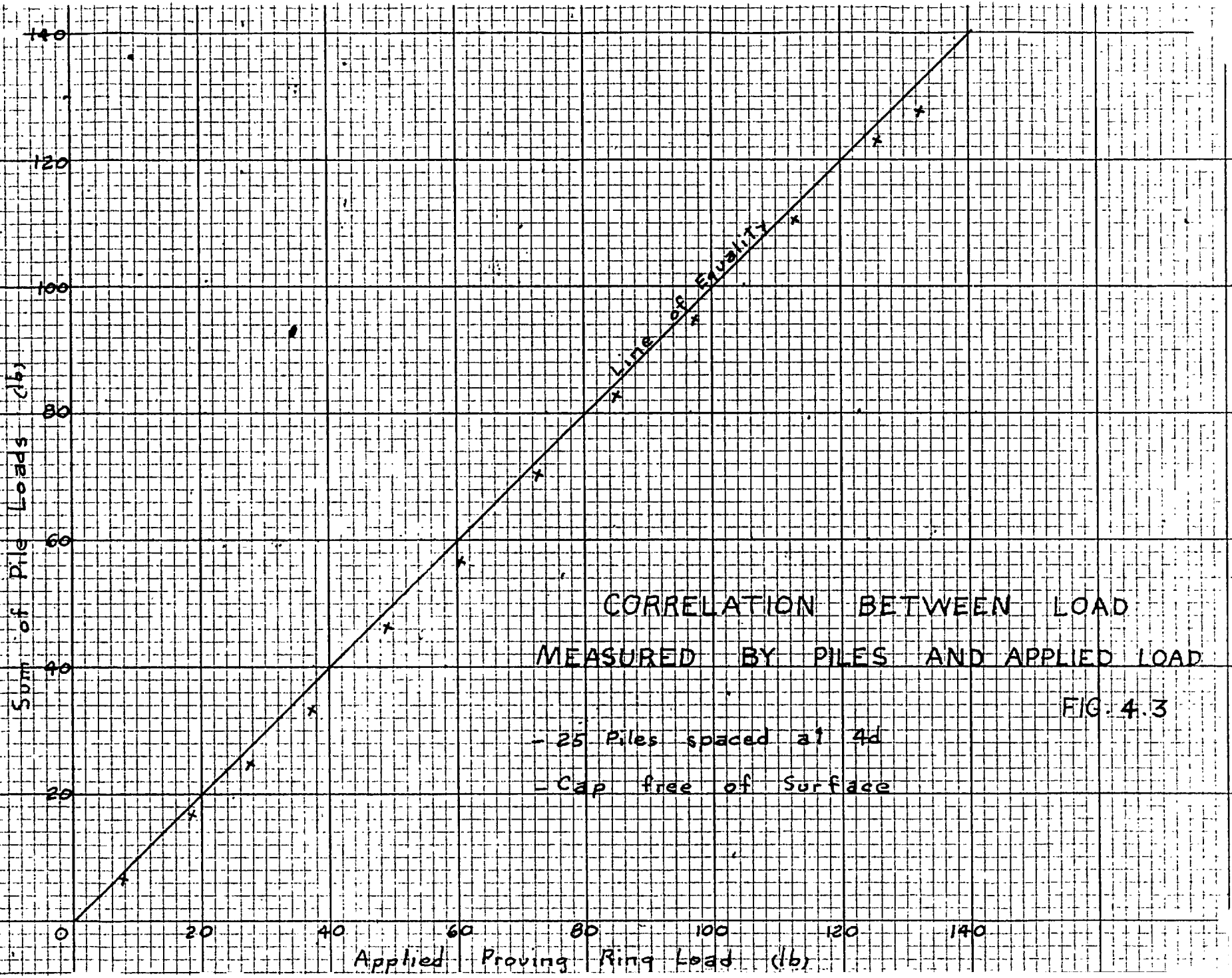
- Araldite strain gauge cement used
- Foil resistance strain gauges - $\frac{1}{4}$ " linear (Saunders Roe)

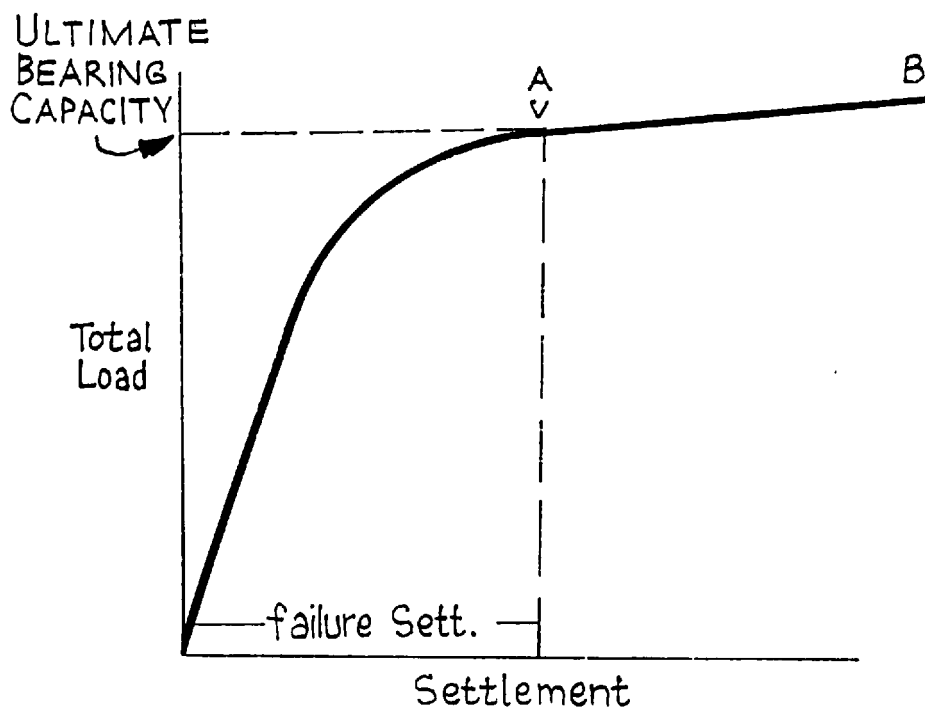
BASE LOAD CELL FIG. 4.1(A)

TYPICAL PILE CALIBRATION

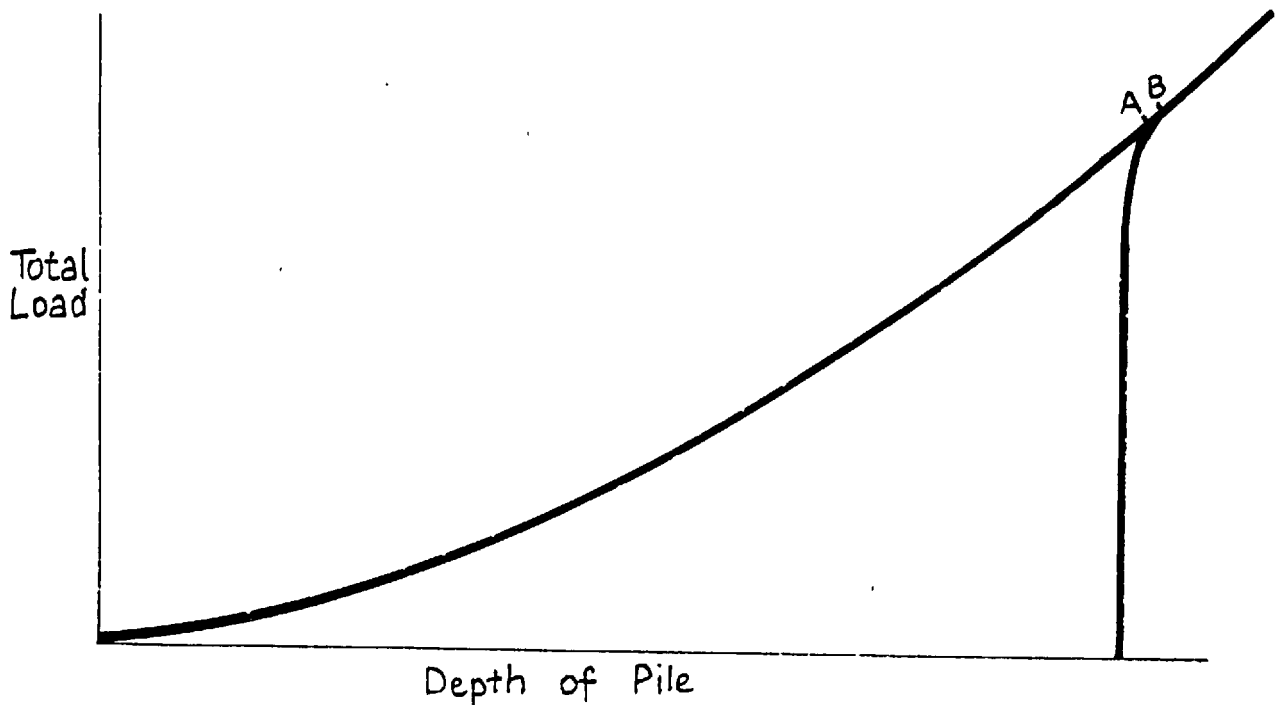
FIG. 4.2



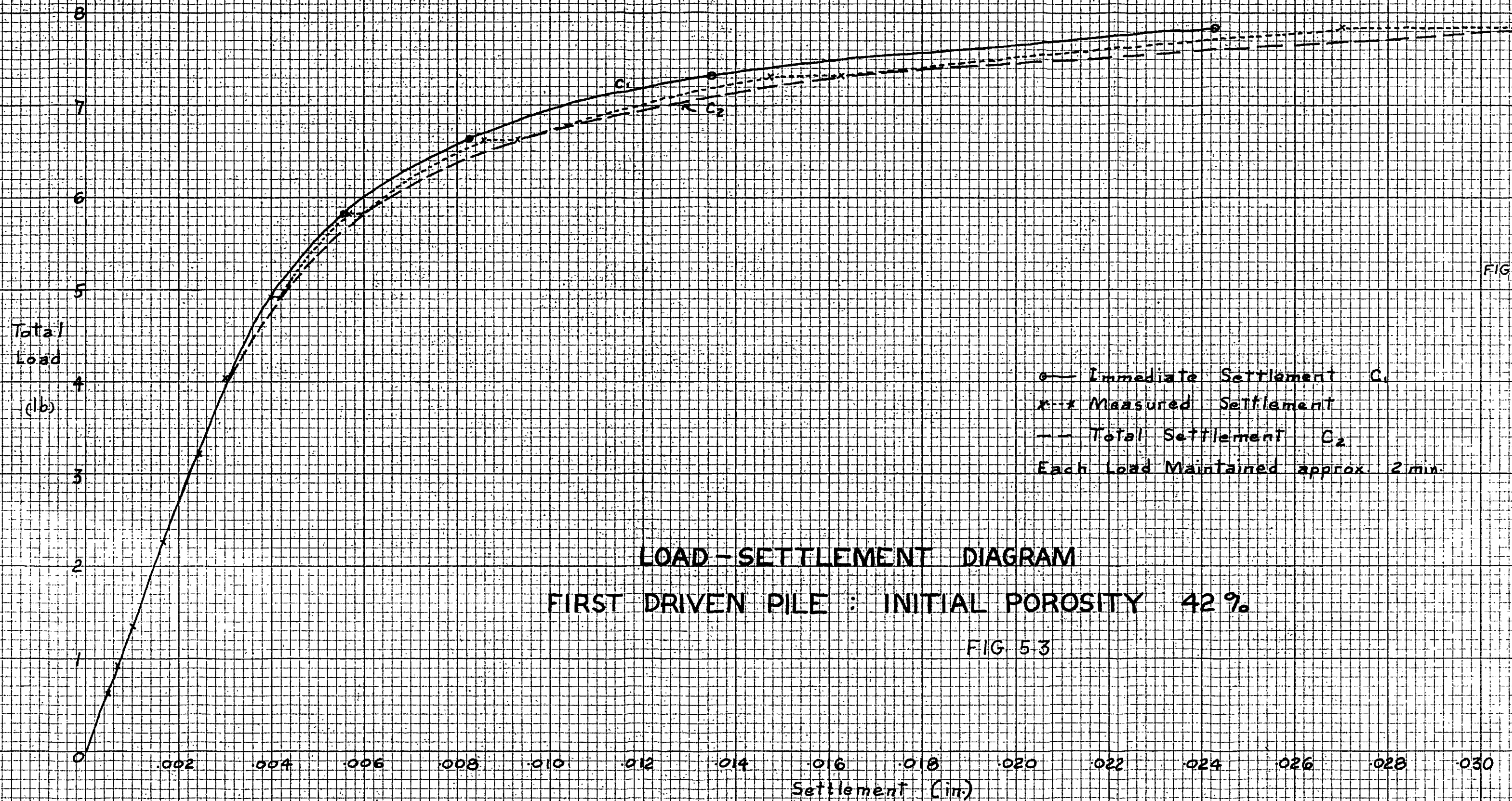


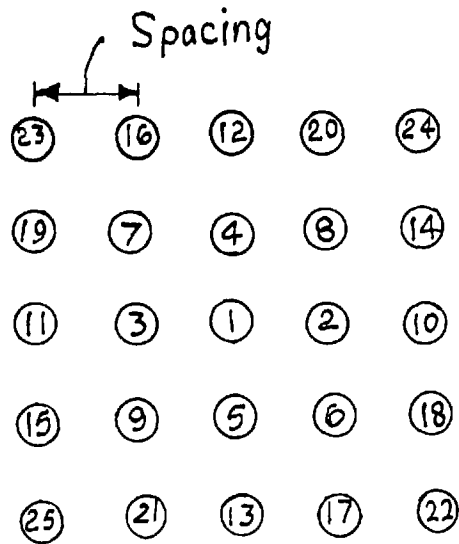


LOAD-SETTLEMENT DIAGRAM
single Pile in Sand
FIG. 5.1

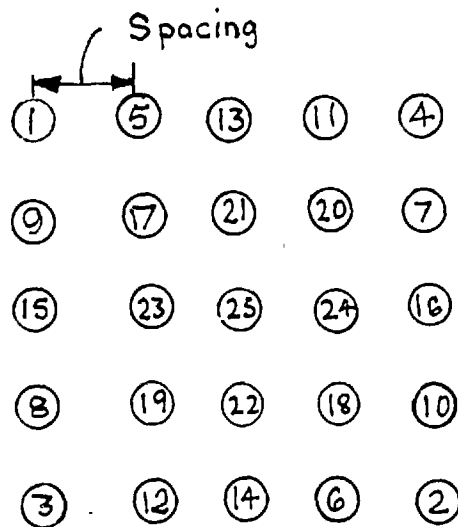


FAILURE ENVELOPE - Single Pile in Sand
FIG. 5.2





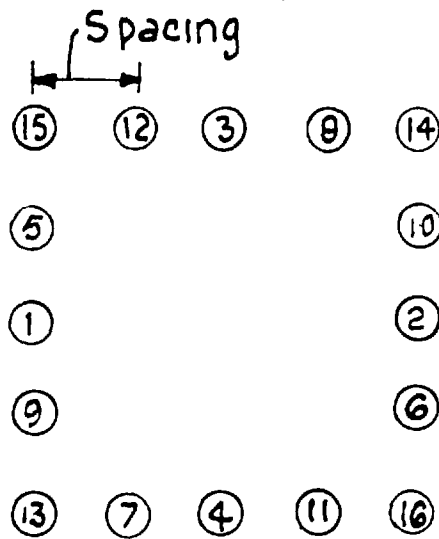
Centre to Corner
Driving Pattern
Case 1.



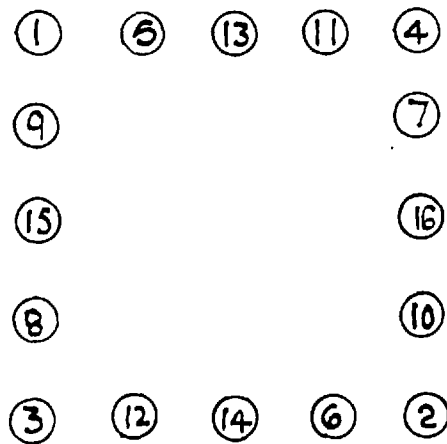
Corner to Centre
Driving Pattern
Case 2.

Spacing - constant;
numbers represent
order of Driving

FIG. 5.4 DRIVING PATTERNS
WITH 25 FILES



Centre to Corner
Driving Pattern
Case 1

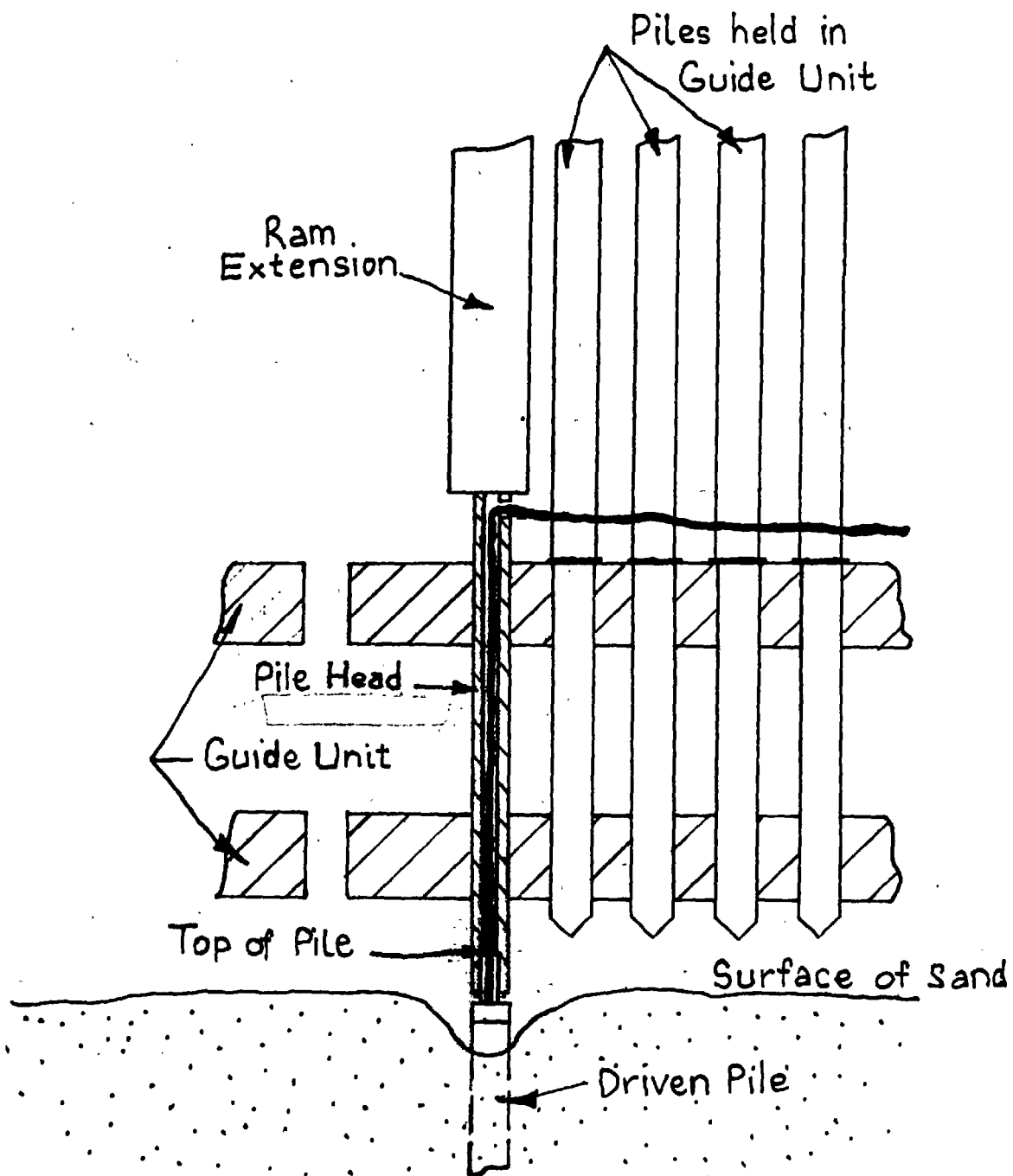


Corner to Centre
Driving Pattern
Case 2

Spacing - constant;
numbers represent
Order of Driving

FIG. 5.5

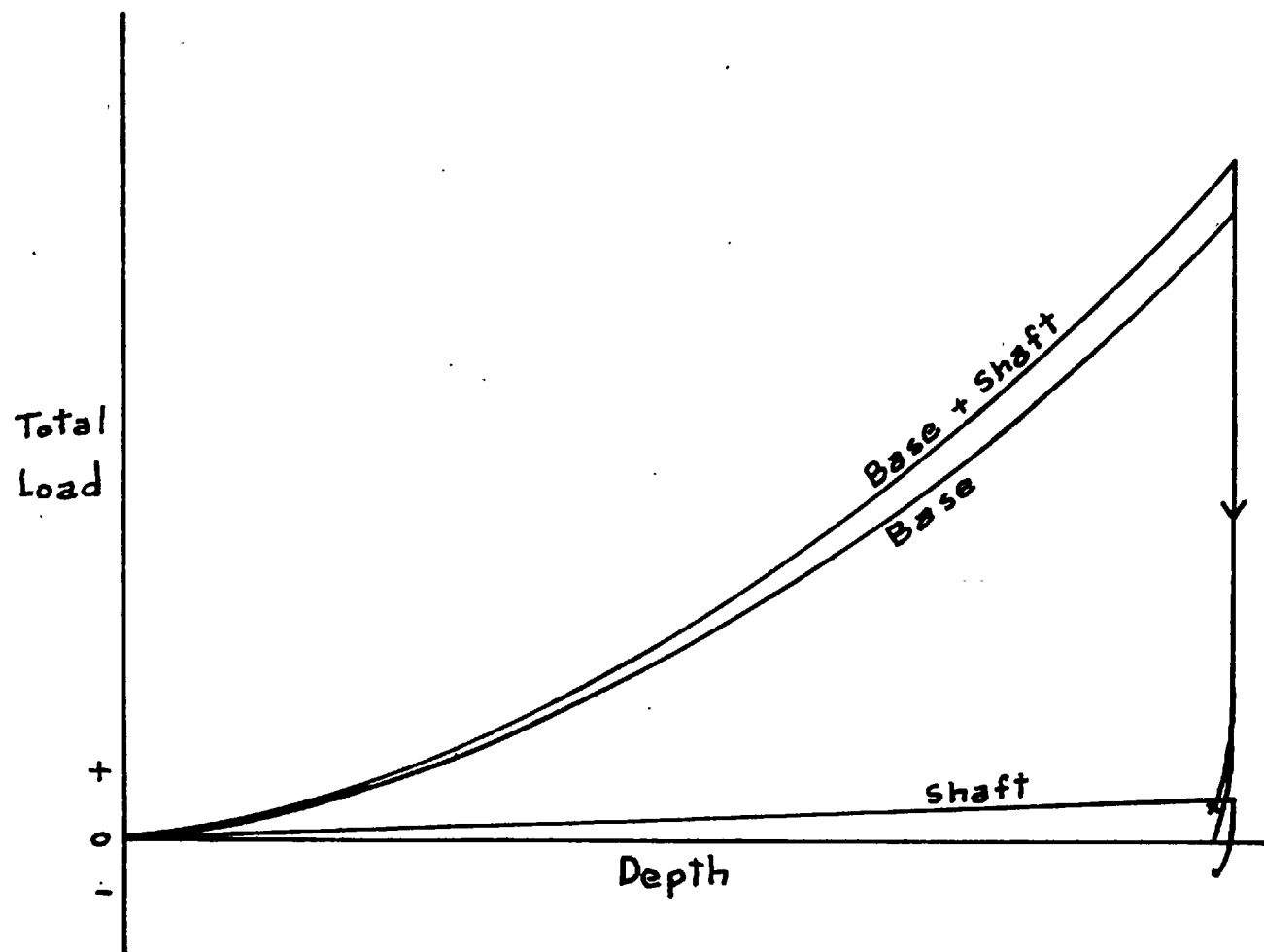
DRIVING PATTERNS
WITH 16 PILES



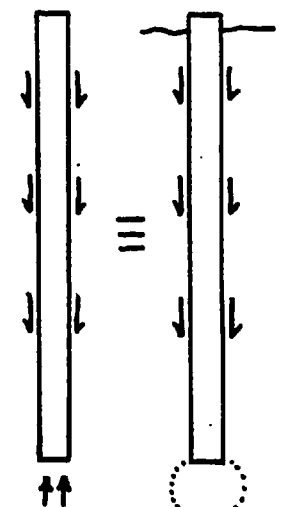
Details of Single Pile
Test

FIG. 5.6

FULL SCALE



After Driving

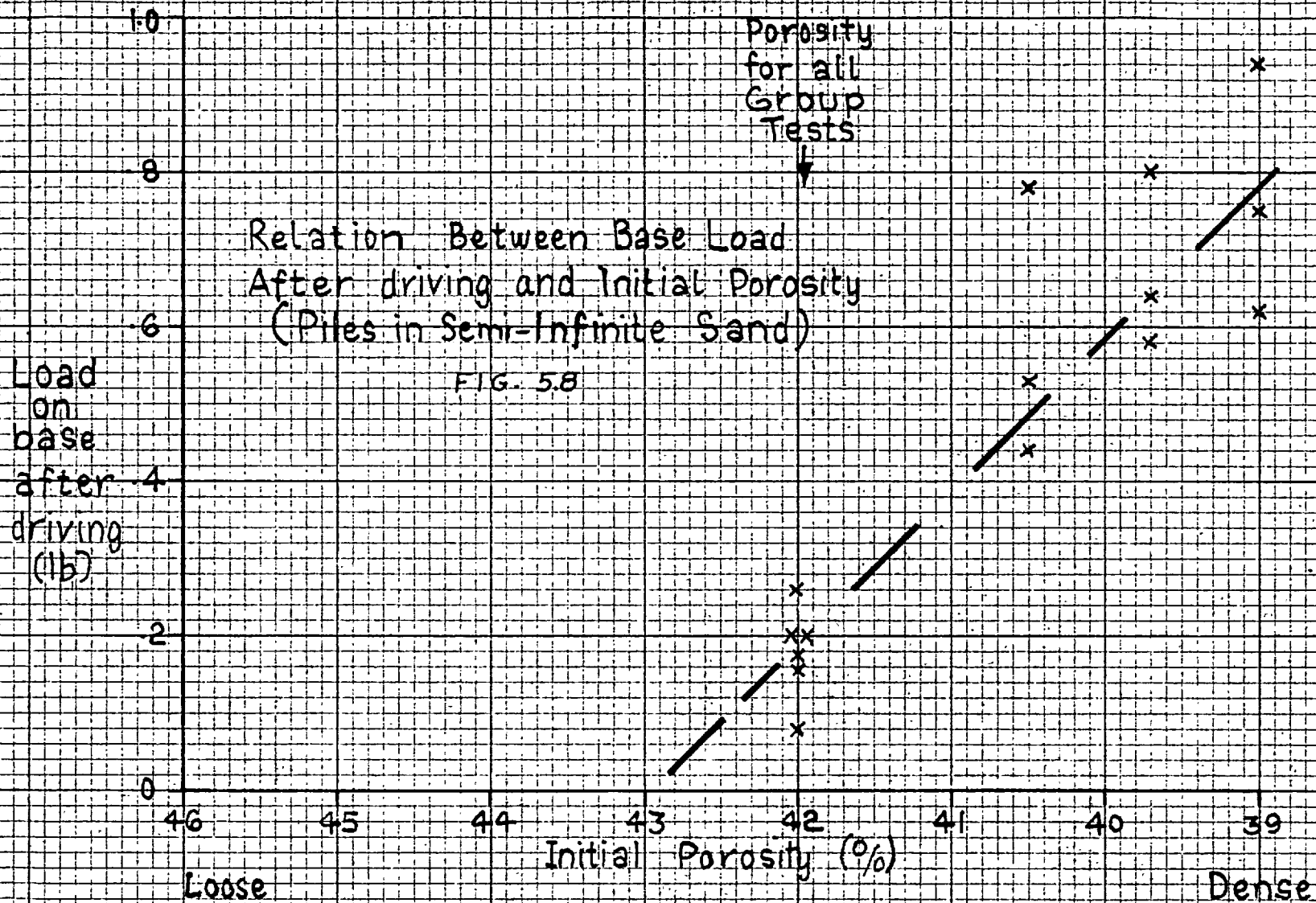


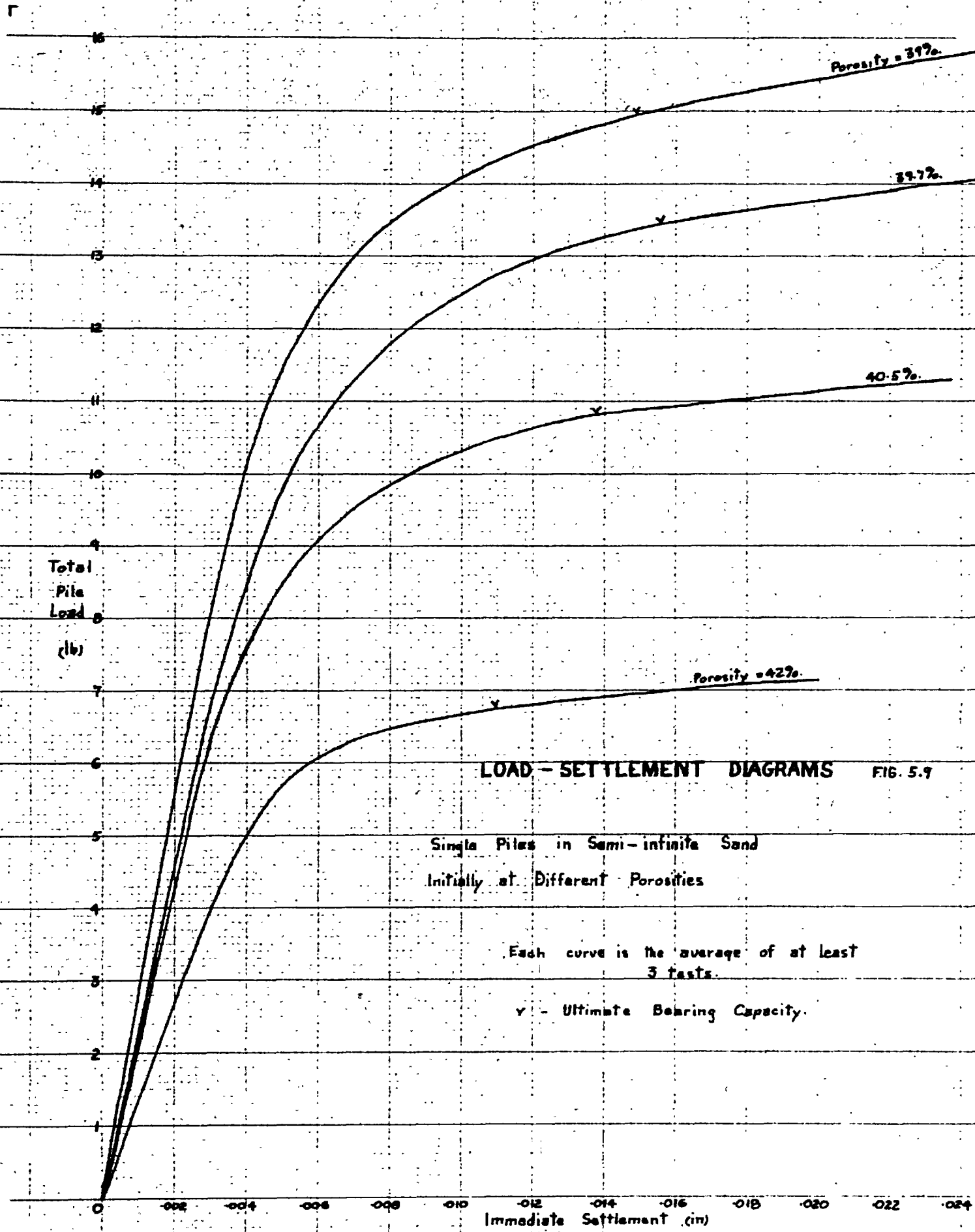
Small Stressed Zone

x Base
• shaft

IDEALIZED BEHAVIOUR DURING DRIVING

FIG. 5.7



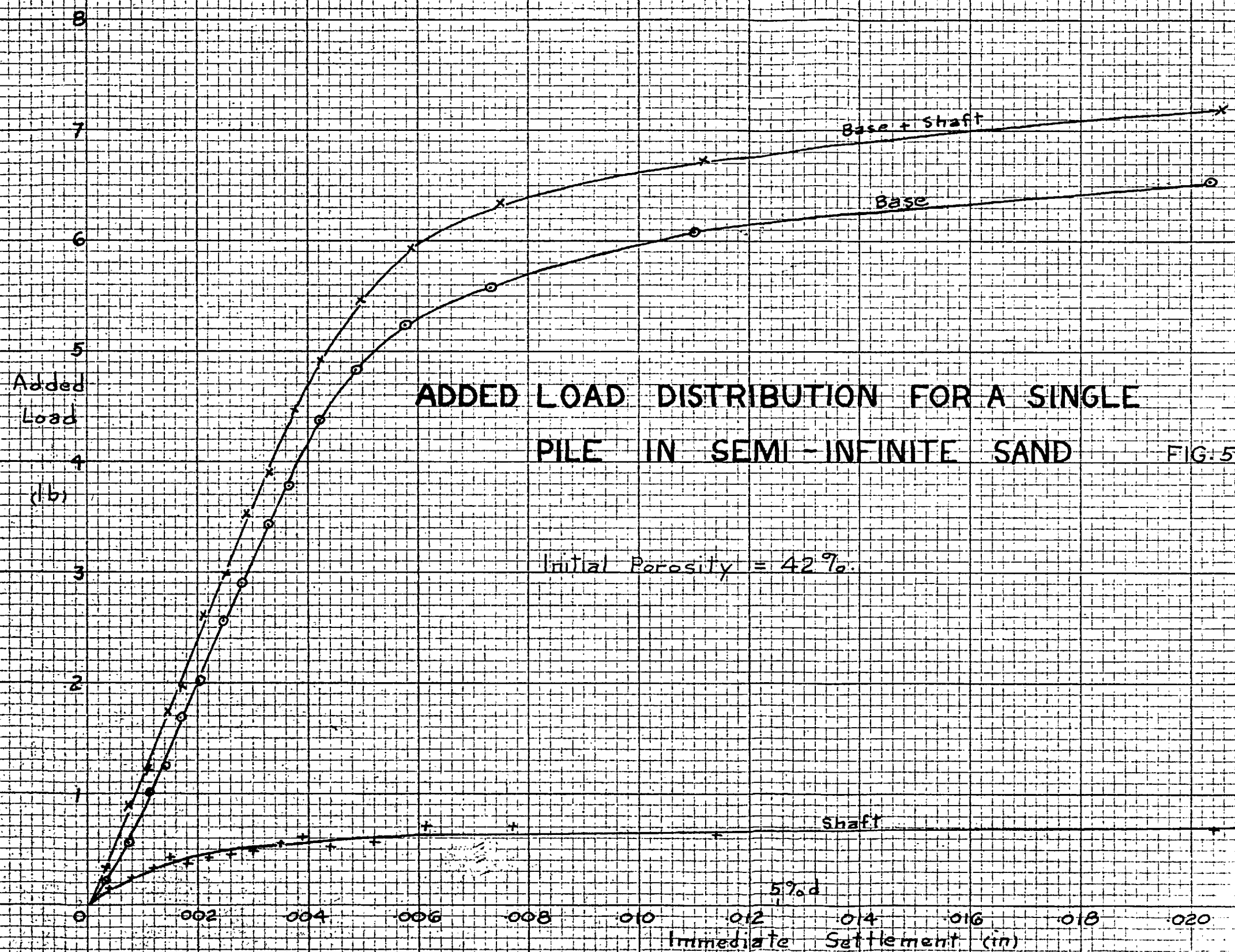


LOAD - SETTLEMENT DIAGRAMS FIG. 5.9

Single Piles in Semi-infinite Sand
Initially at Different Porosities

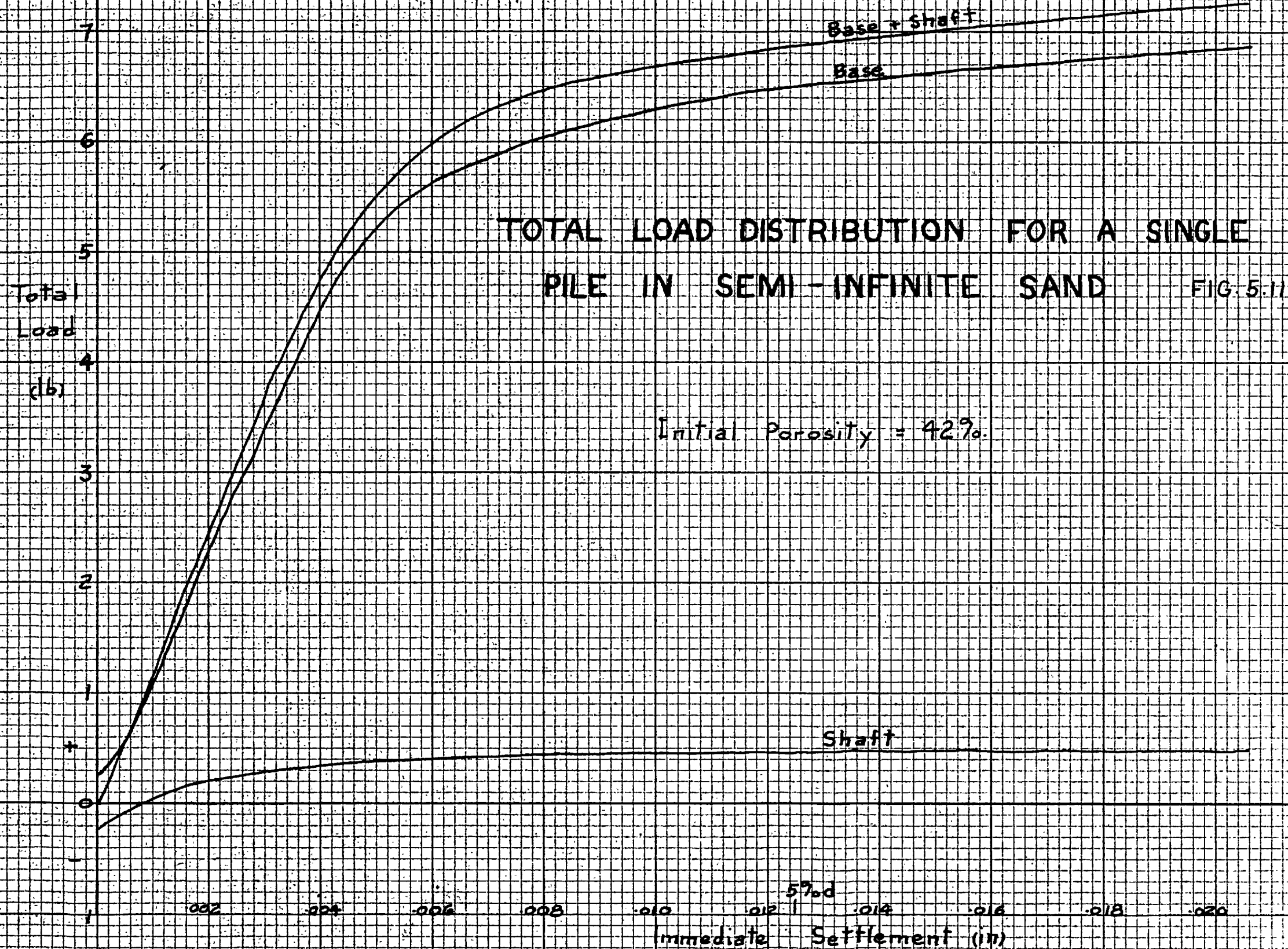
Each curve is the average of at least
3 tests.

y - Ultimate Bearing Capacity.



ADDED LOAD DISTRIBUTION FOR A SINGLE
PILE IN SEMI-INFINITE SAND

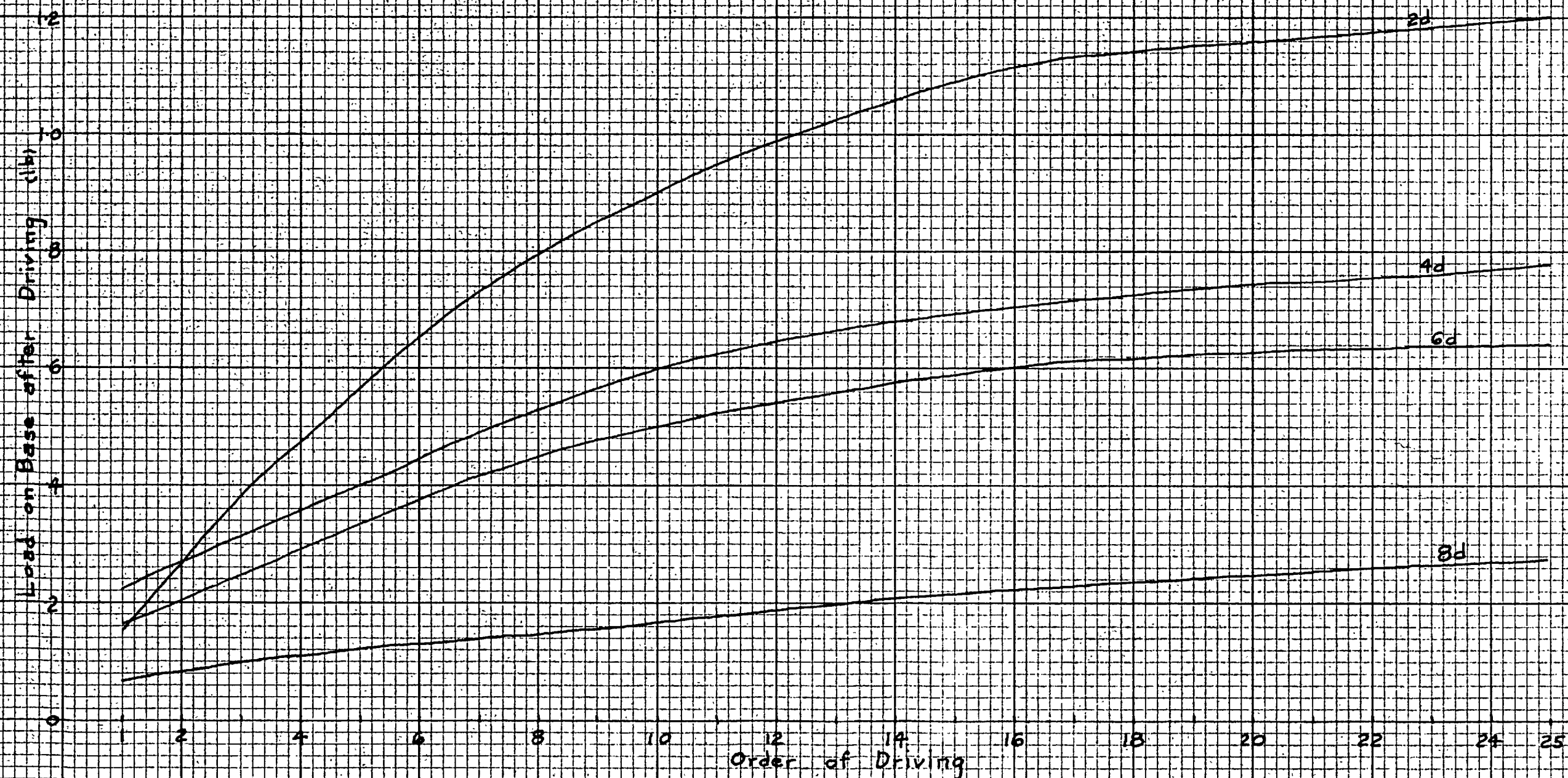
FIG. 5.10

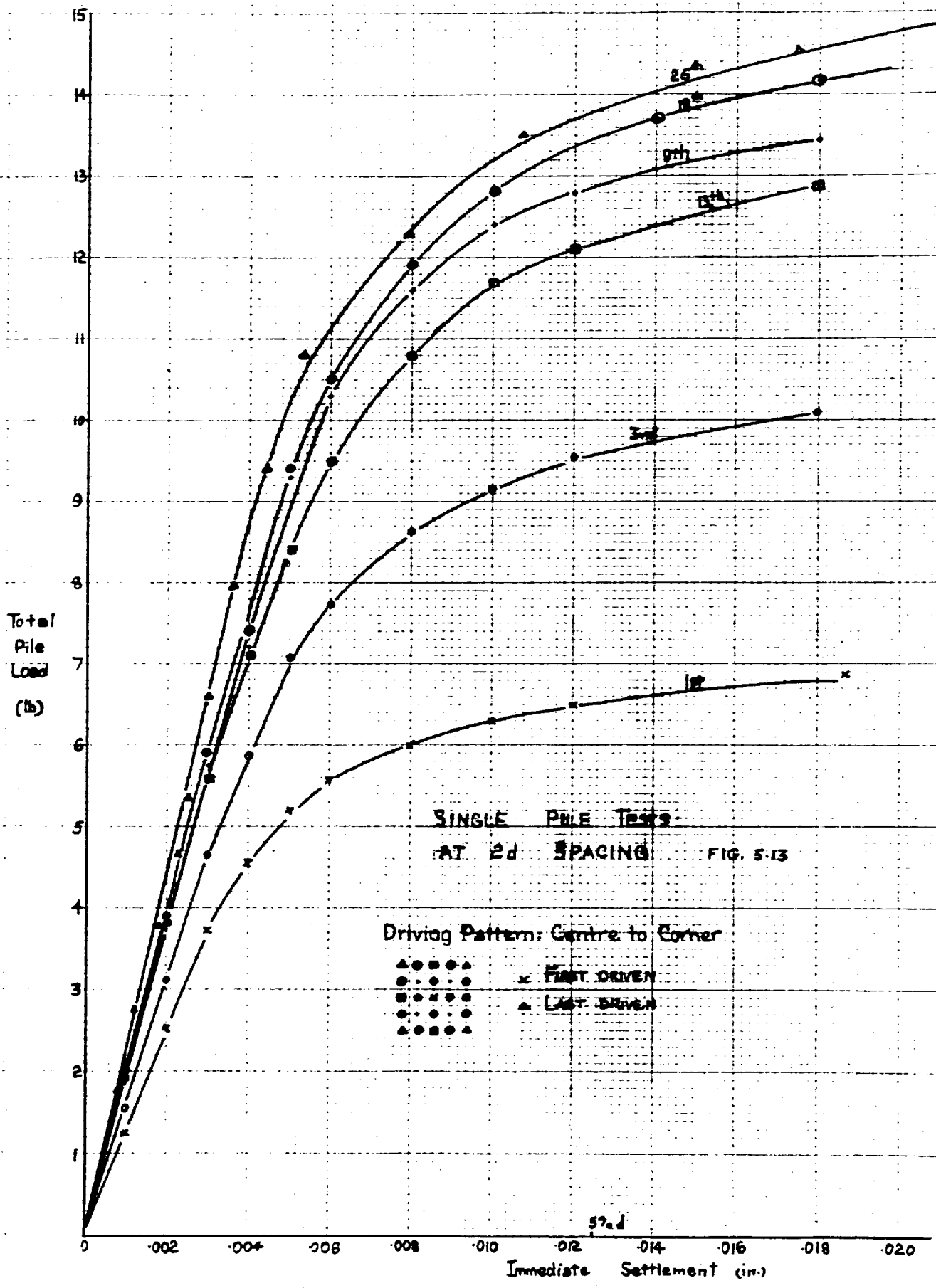


ORDER OF DRIVING
vs.
PILE COMPRESSIVE LOAD AFTER DRIVING

FIG. 5.12

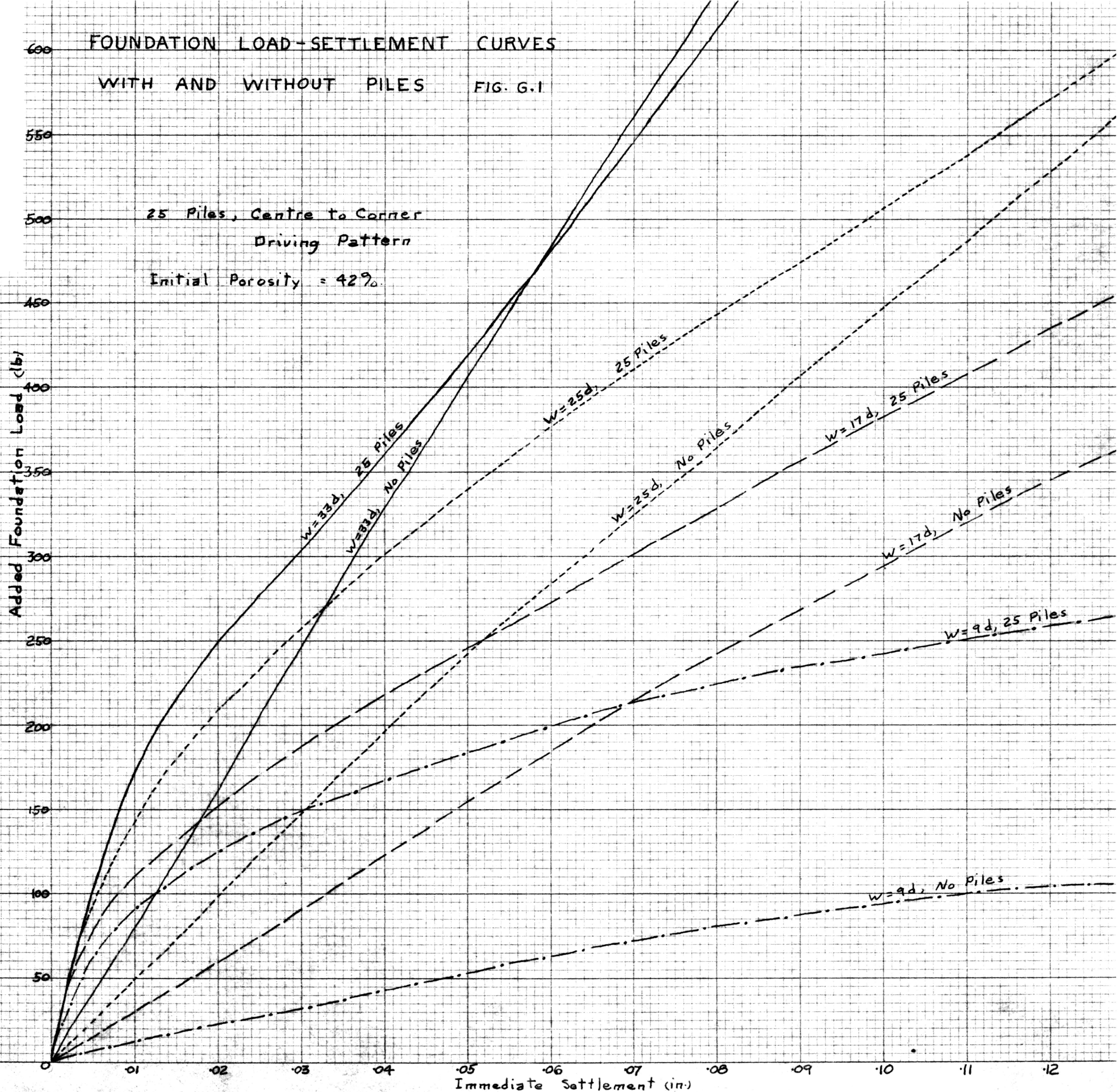
Driving Pattern: Centre to Corner
Piles Spaced at 2, 4, 6 & 8d





FOUNDATION LOAD-SETTLEMENT CURVES

WITH AND WITHOUT PILES FIG. G.1



ORDER OF DRIVING

VTS

ULTIMATE BEARING CAPACITY

FIG. 5.14

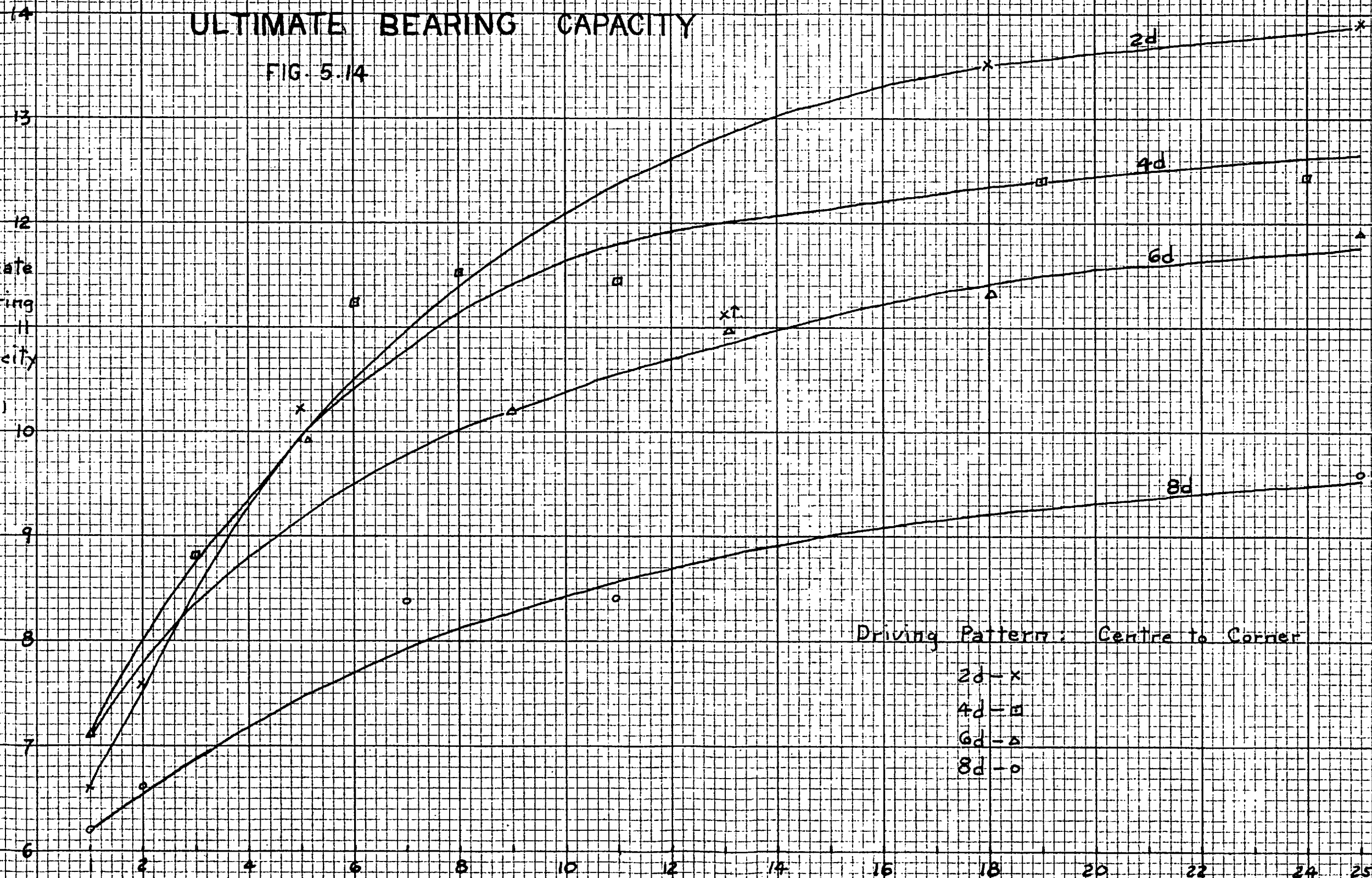
Ultimate
Bearing
Capacity

(lb)

Driving Pattern: Centre to Corner

2d - x
4d - □
6d - △
8d - o

Order of Driving

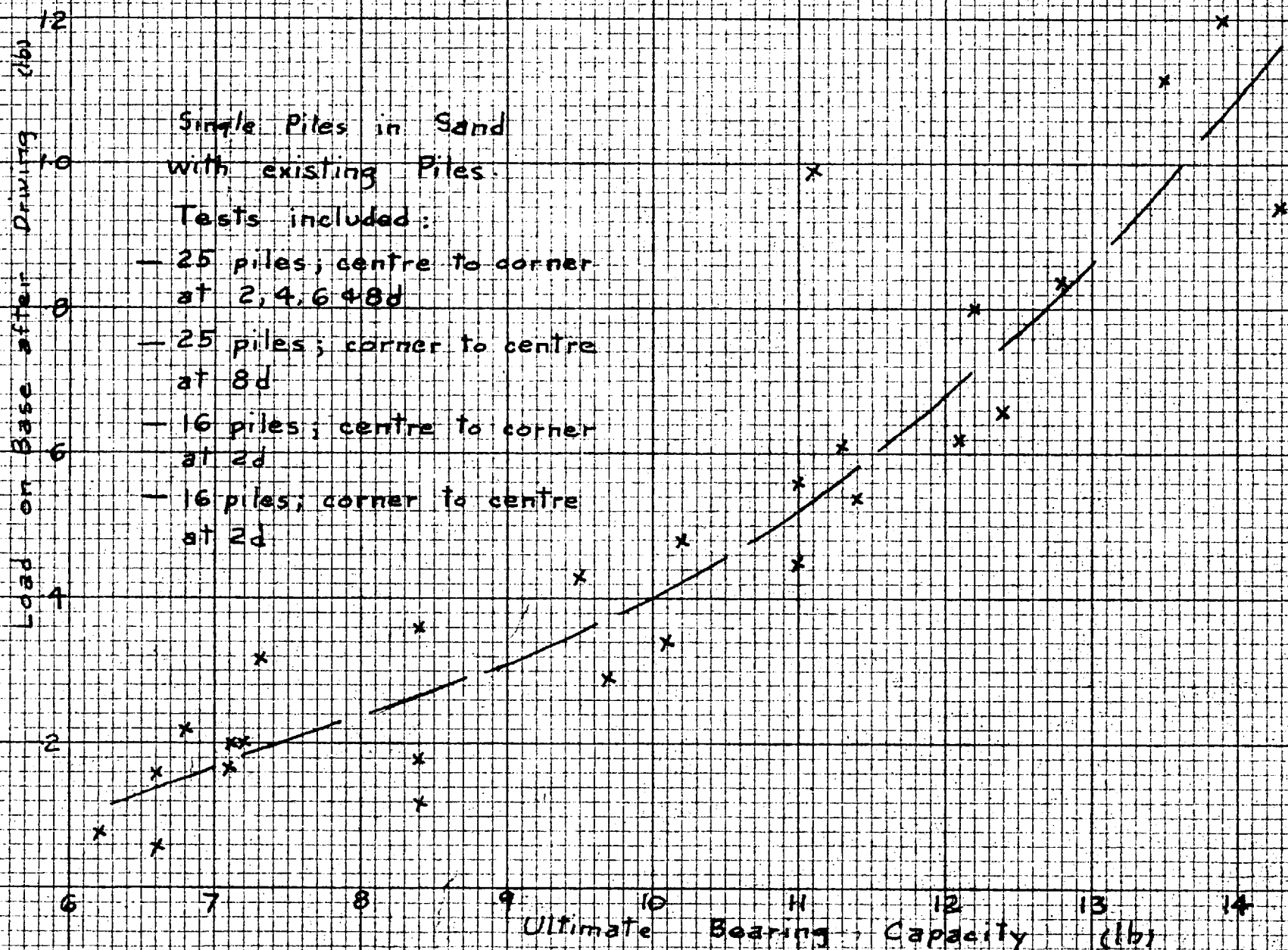


ULTIMATE BEARING CAPACITY

V.F.S.

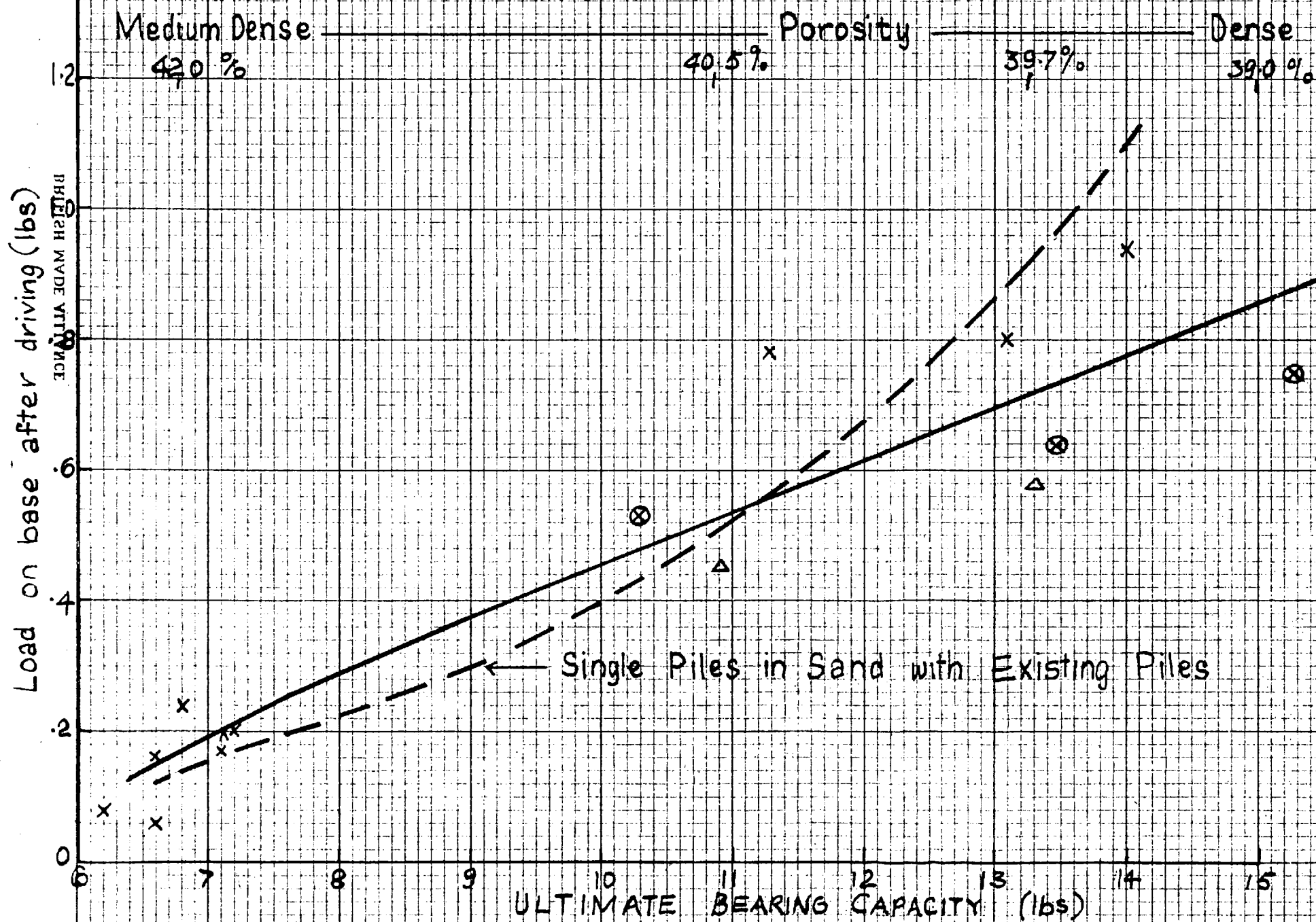
PILE COMPRESSIVE LOAD AFTER DRIVING

FIG. 5.15



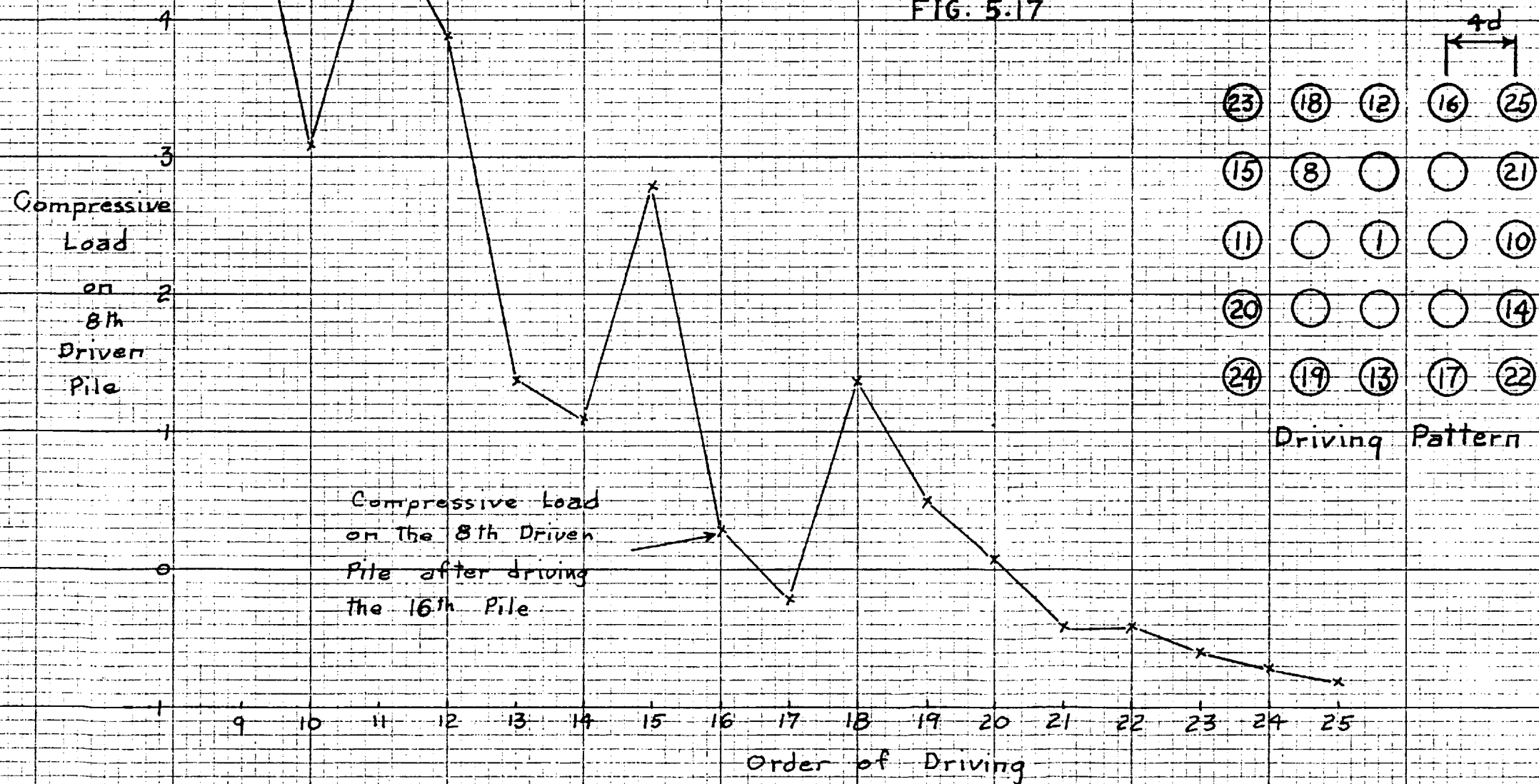
ULTIMATE BEARING CAPACITY VRS BASE LOAD AFTER DRIVING Single Piles in Semi-Infinite Sand

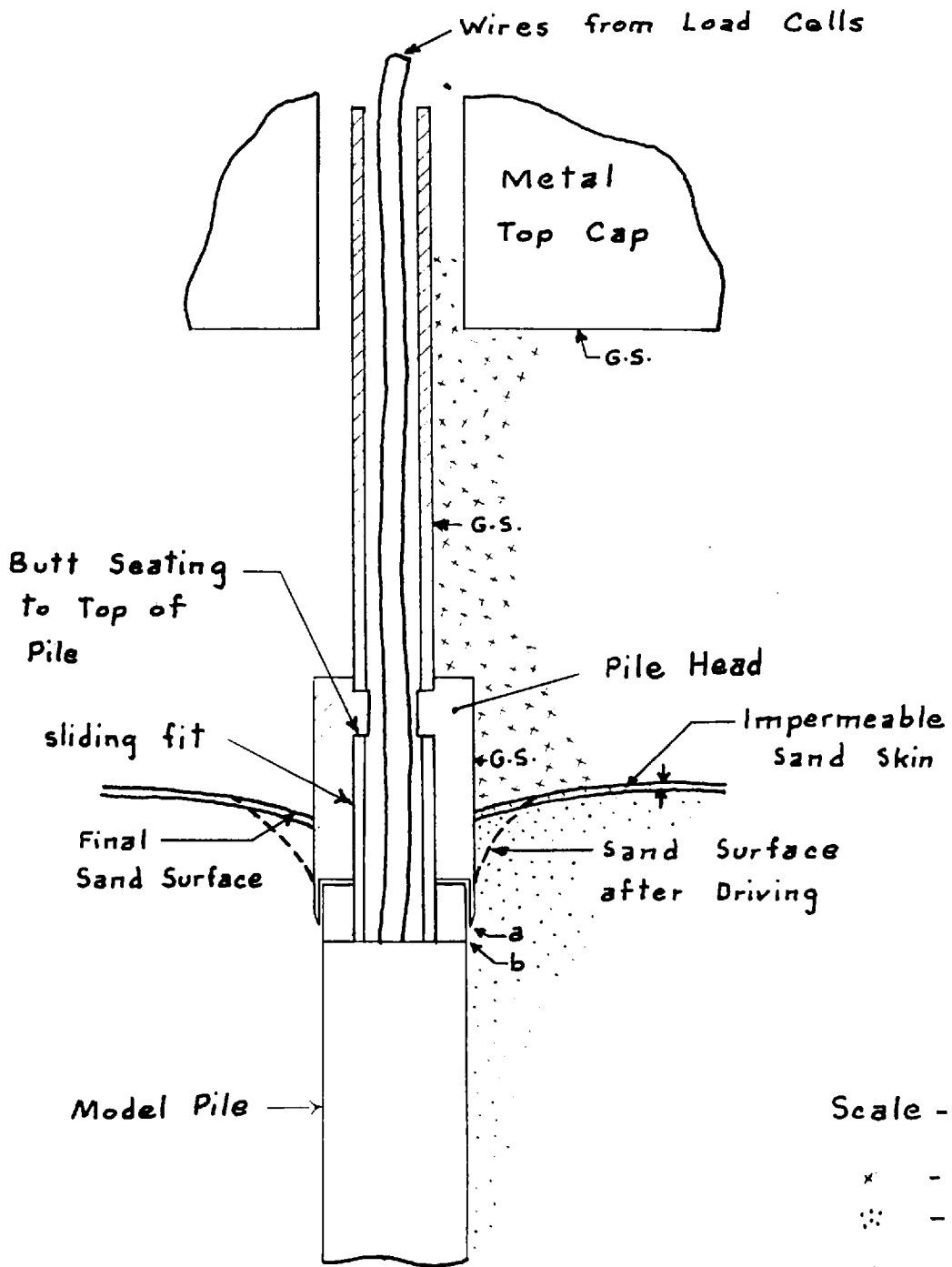
FIG. 5.16



THE EFFECT OF DRIVING PILES ON THE COMPRESSIVE LOAD IN A PREVIOUSLY DRIVEN PILE

FIG. 5.17





Scale - 3in. = 1in.

x - Plaster

· - Sand

/// - Brass

G.S. - Greased Surface

DETAILS OF PILE
FOUNDATION

FIG. 6.1

					Average Order of Driving	
					Centre to Corner	Corner to Centre
△	⊙	⊠	⊗	△	Symbol	
⊙			⊙			
⊠			⊠	⊠	2½	14½
				⊗	6½	10½
⊙			⊙	⊙	10½	6½
△	⊙	⊠	⊙	△	14½	2½

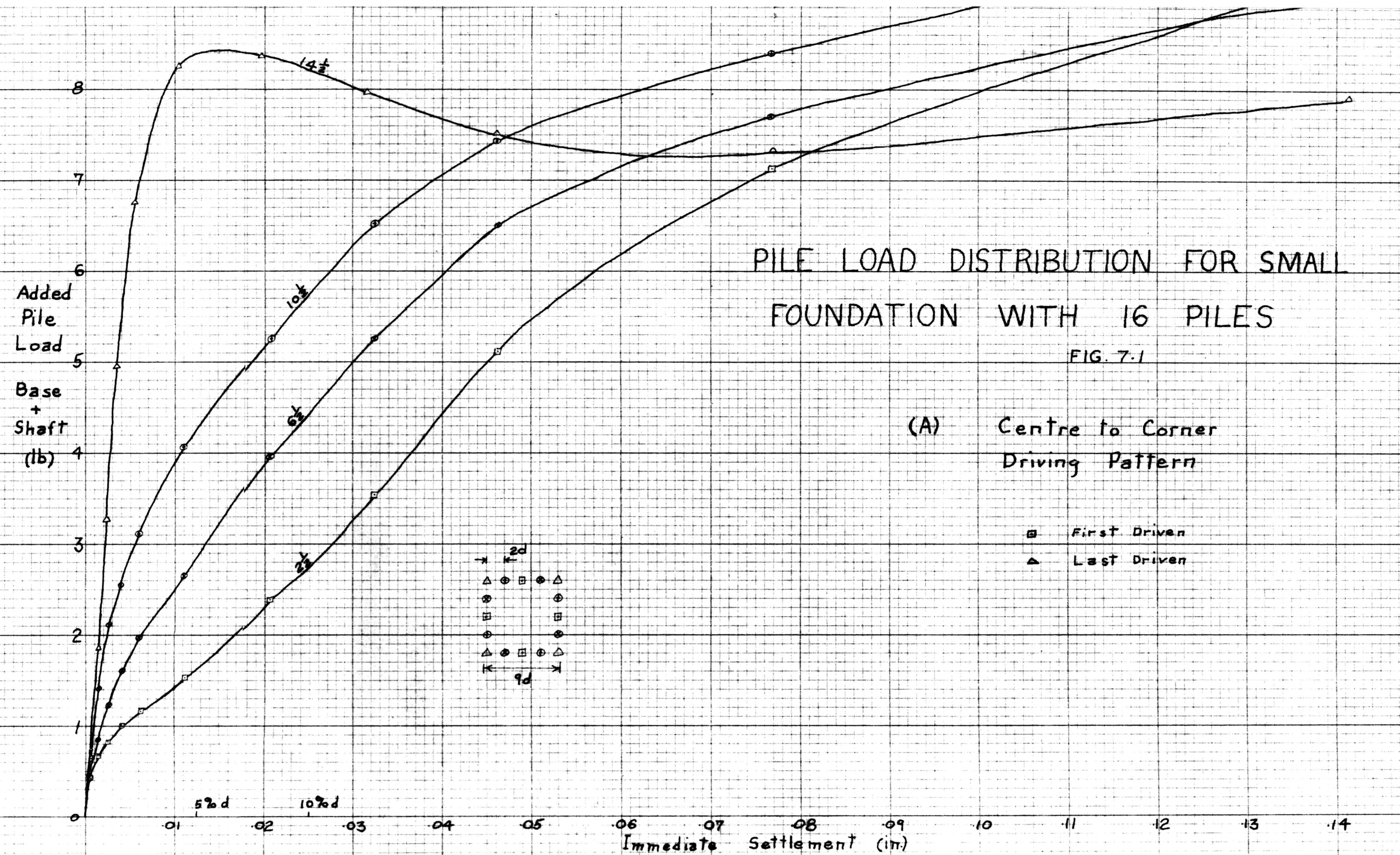
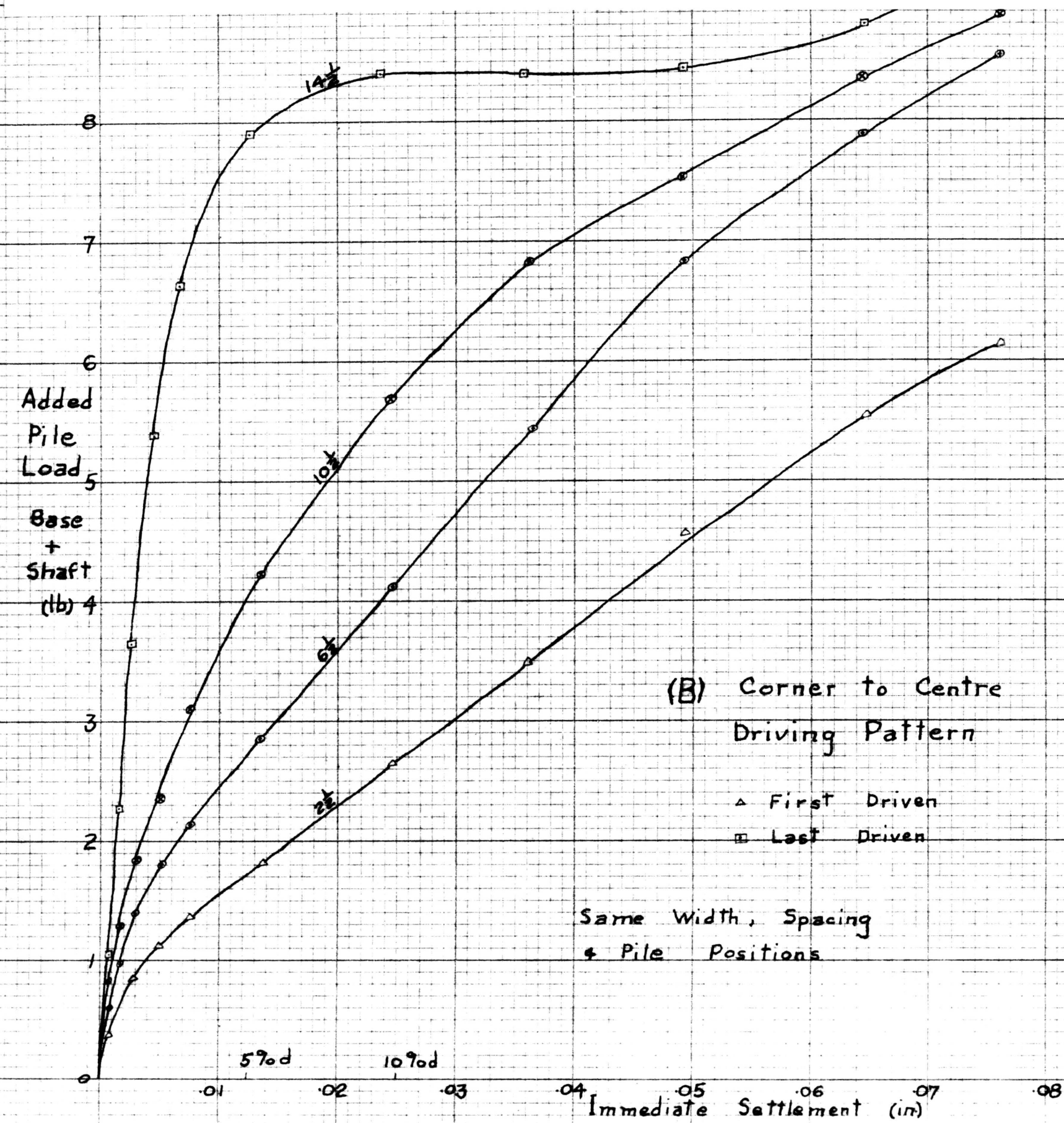
(B) 16 Piles

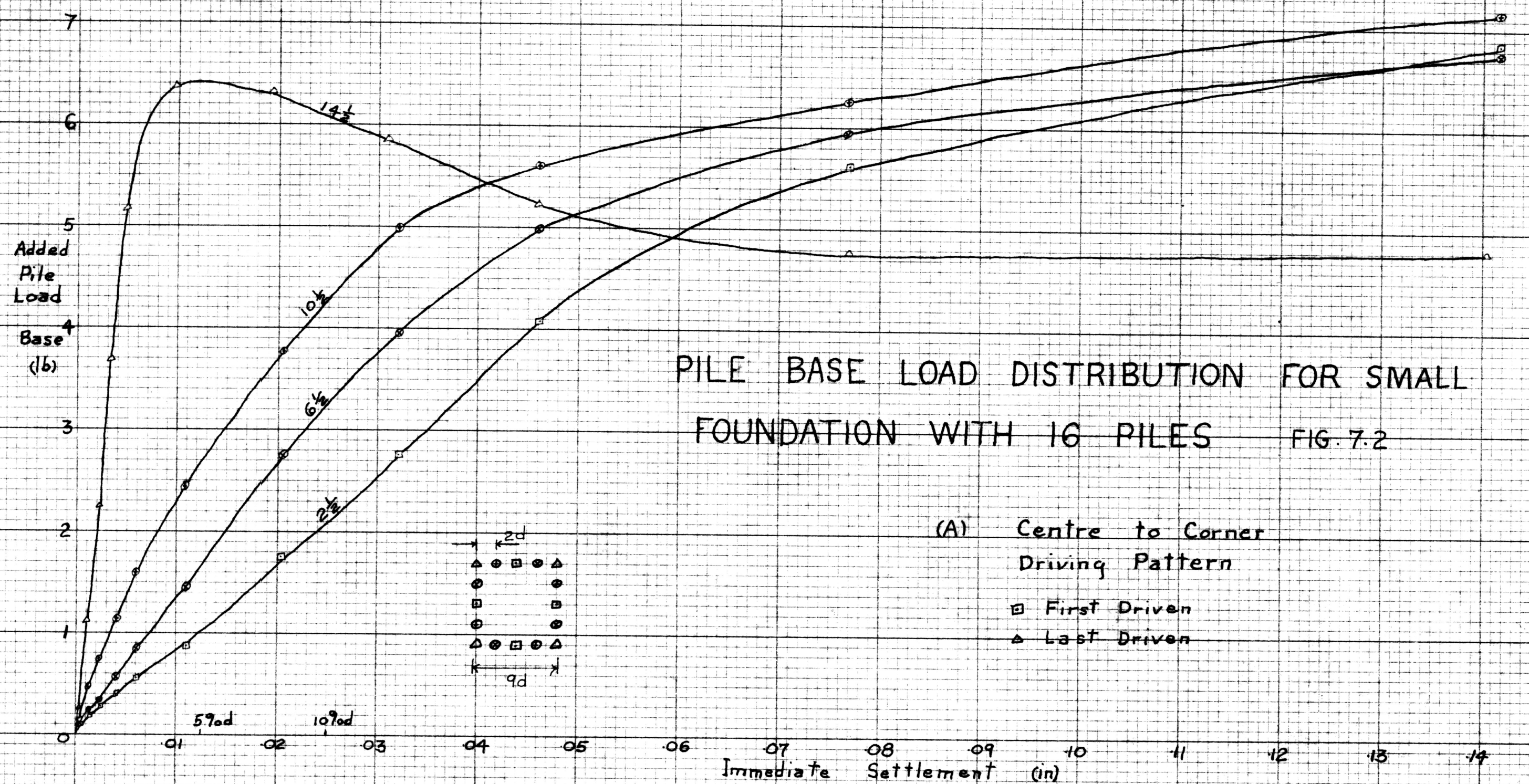
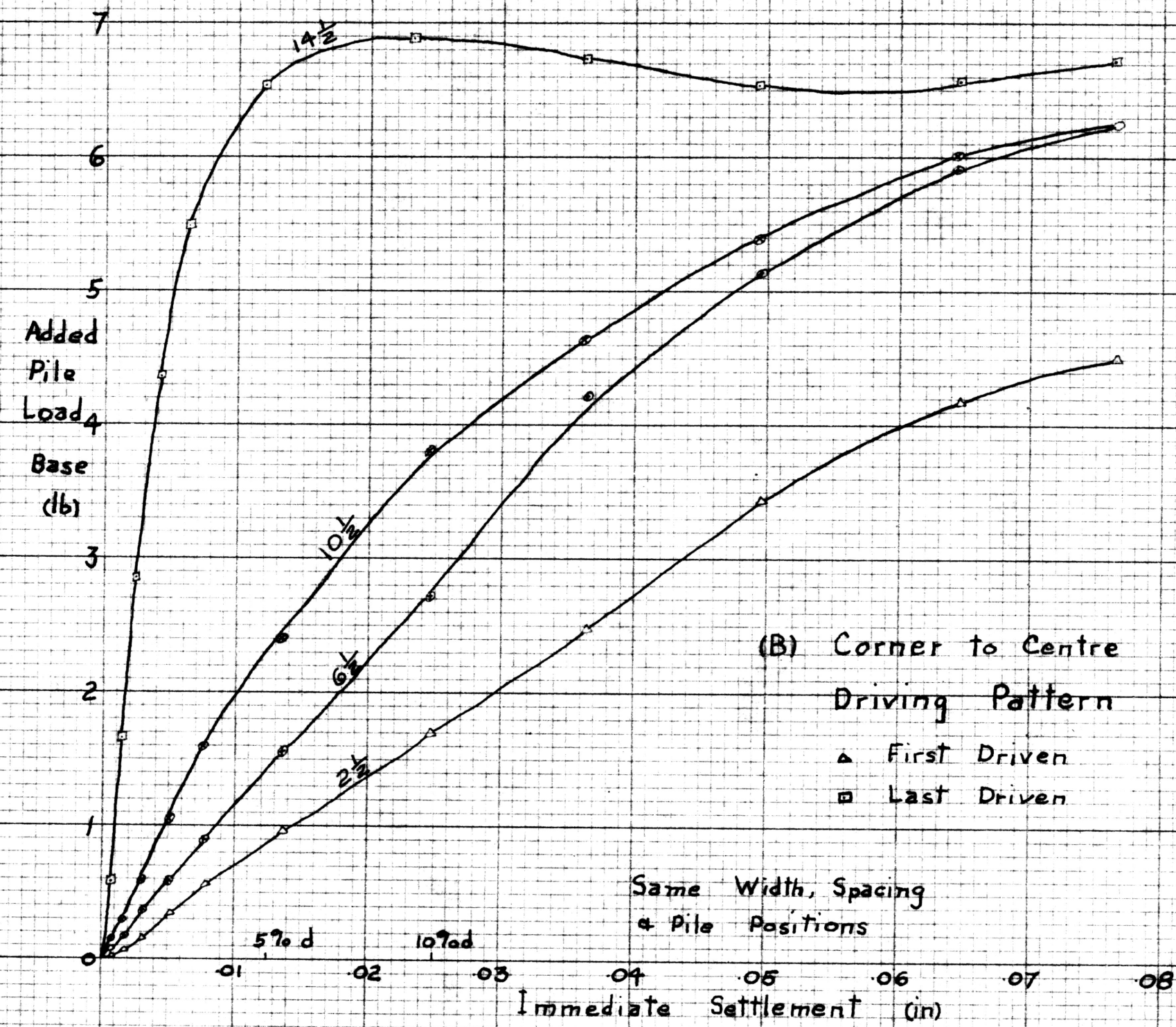
					Average Order of Driving	
					Centre to Corner	Corner to Centre
△	⊙	⊠	⊙	△	Symbol	
⊙	•	○	•	⊙		
⊠	○	x	○	⊠	x	1
				○	3½	23½
⊙	•	○	•	⊙	•	7½
△	⊙	⊠	⊙	△	⊠	11½
				○	17½	8½
				△	23½	2½

(A) 25 Piles

PILE POSITION AND AVERAGE
ORDER OF DRIVING

FIG. 6.2.





PILE BASE LOAD DISTRIBUTION FOR SMALL FOUNDATION WITH 16 PILES FIG. 7.2

Added Pile Load
Base + Shaft
(lb)

Added Pile Load
Base + Shaft
(lb)

PILE LOAD DISTRIBUTION FOR LARGE FOUNDATION WITH 25 PILES

FIG. 7-3

(b) Corner to Centre
Driving Pattern

△ First Driven
x Last Driven

Same width, Spacing
← Pile Positions

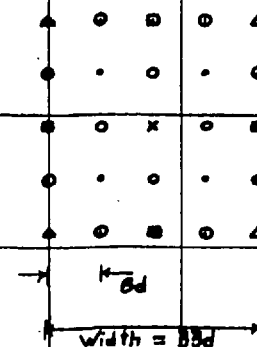
57°d 107°d
Immediate Settlement (in)

(a) Centre to Corner
Driving Pattern

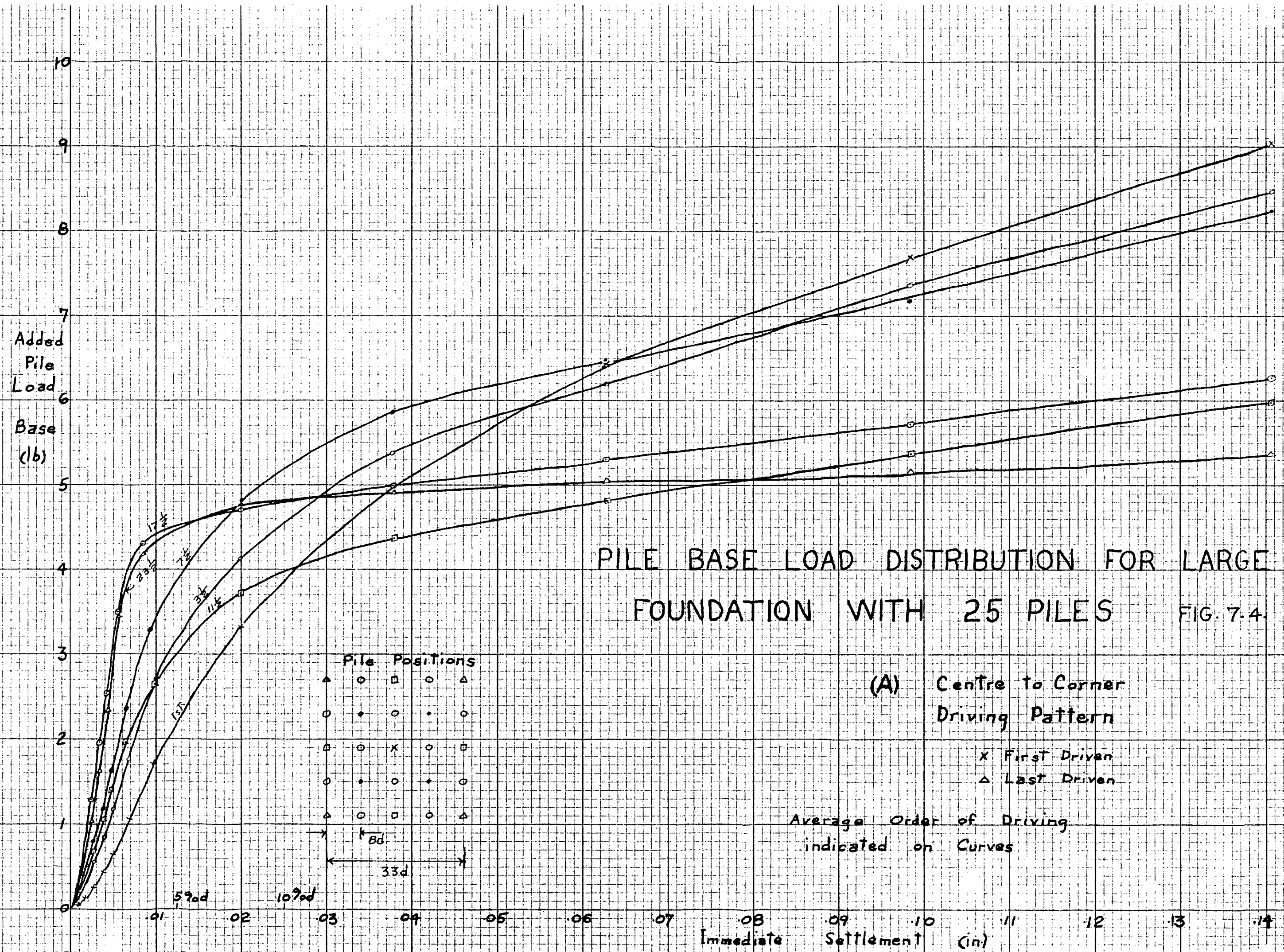
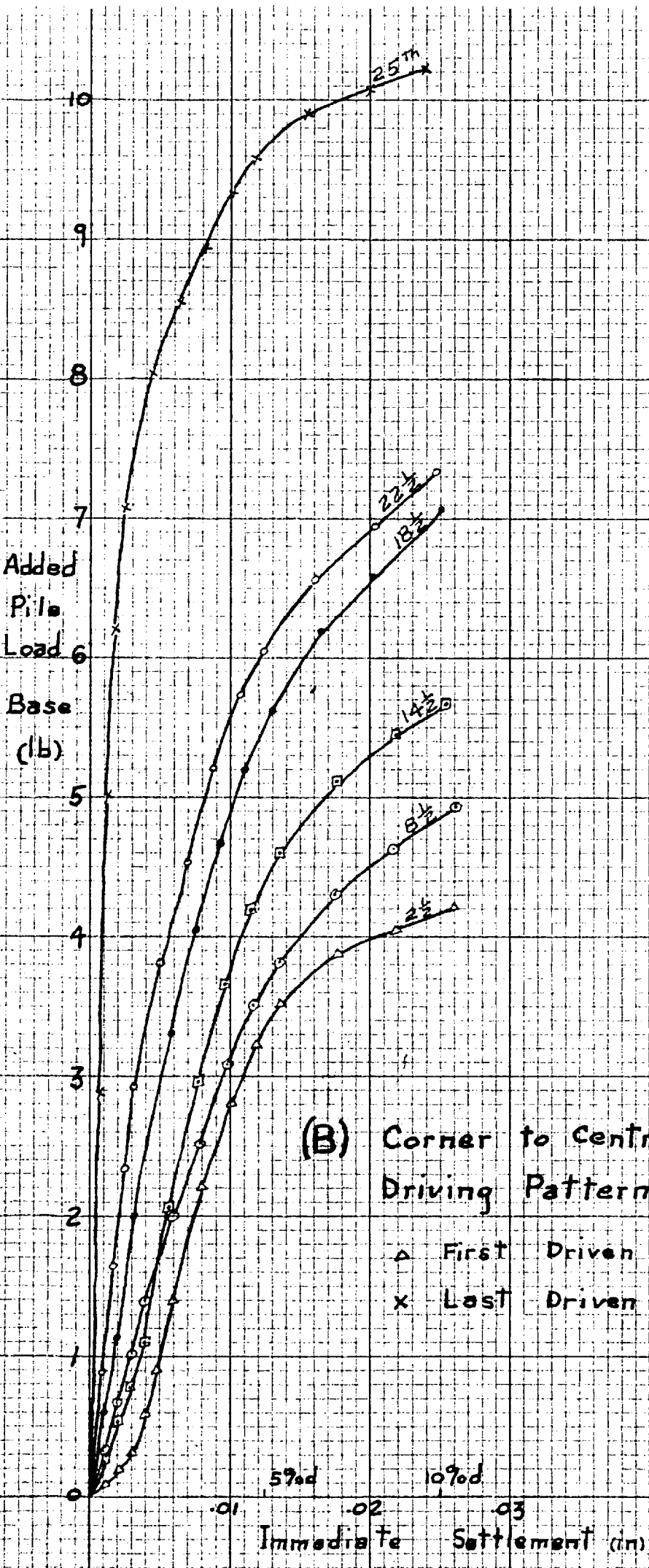
x First Driven
△ Last Driven

Average Order of Driving
indicated on Curves.

Pile Positions



Immediate Settlement (in)

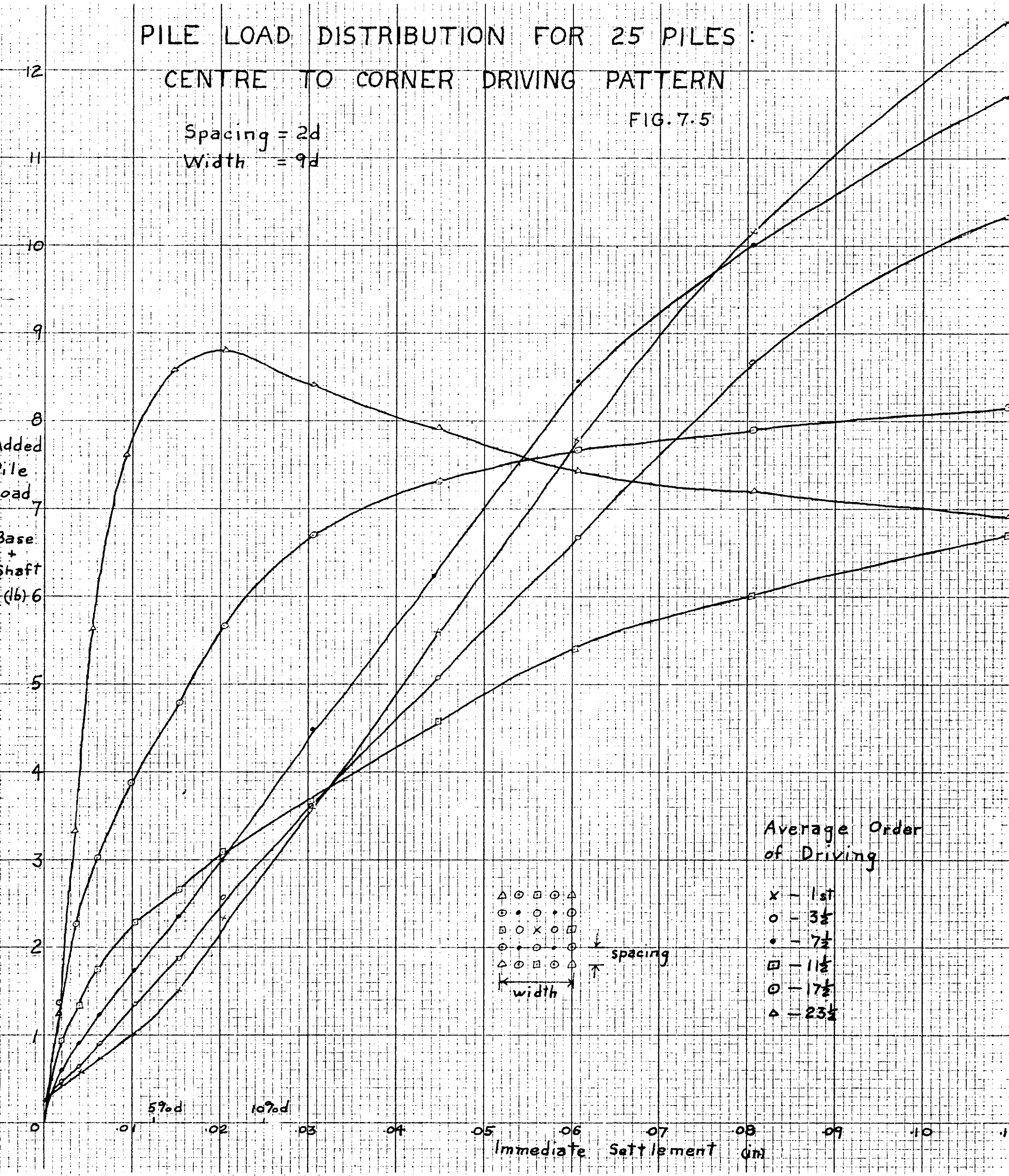


PILE LOAD DISTRIBUTION FOR 25 PILES :

CENTRE TO CORNER DRIVING PATTERN

FIG. 7.5

Spacing = $2d$
Width = $9d$



Added
Pile
Load

Base
+
Shaft
(lb)

PILE LOAD DISTRIBUTION FOR 25 PILES : CENTRE TO CORNER DRIVING PATTERN

FIG. 7-6

Spacing = $4d$

Width = $17d$

Same symbols for average Order
of driving and position as in fig. 7-5

Immediate Settlement (in)

PILE LOAD DISTRIBUTION FOR 25 PILES: CENTRE TO CORNER DRIVING PATTERN

FIG. 7.7

Spacing = $6d$

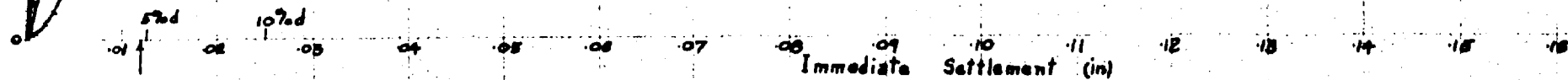
Width = $25d$

Same symbols for average order
of driving and position as in fig. 7.5

Added
Pile
Load
Base
+
Shaft
(lb)

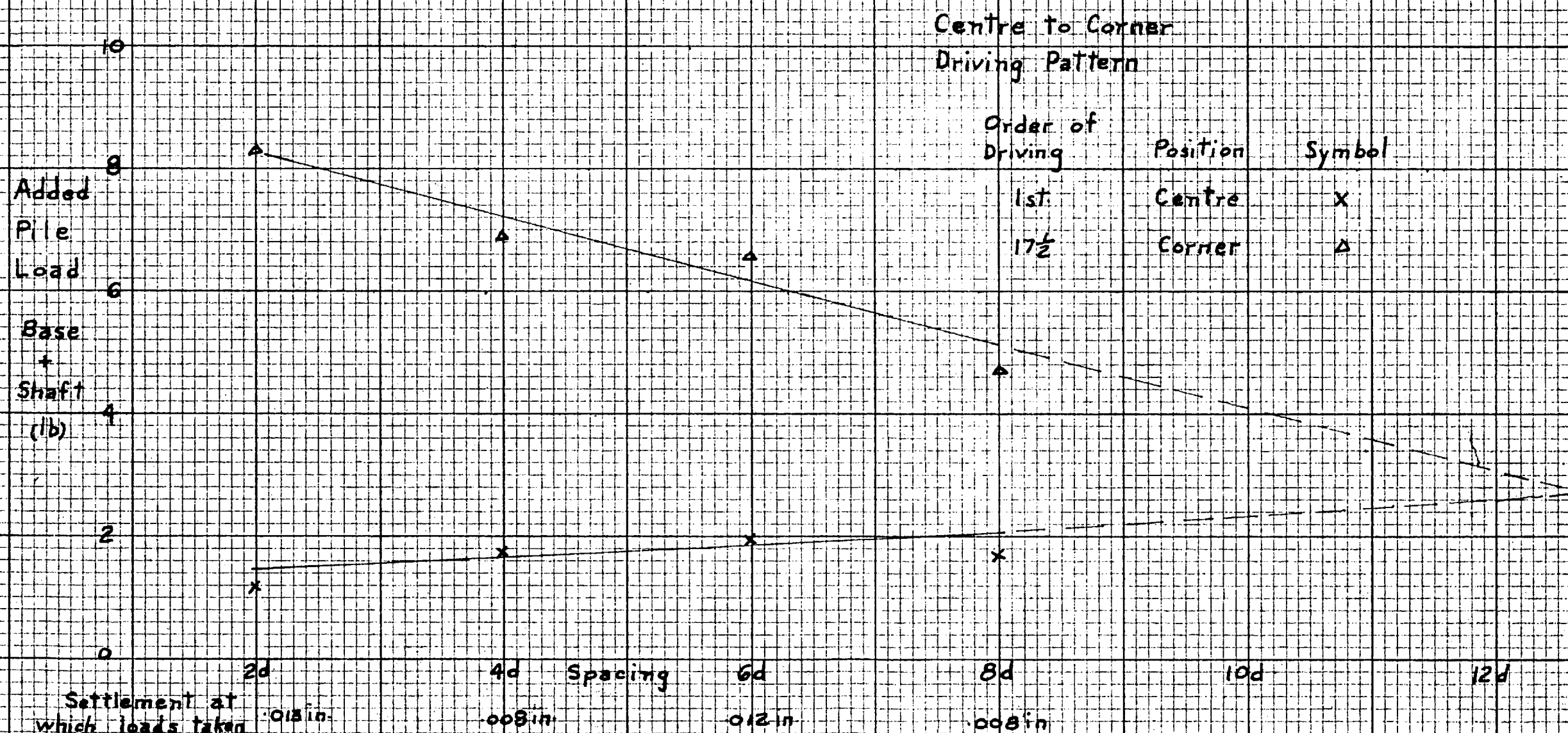
0 .01 .02 .03 .04 .05 .06 .07 .08 .09 .10 .11 .12 .13 .14 .15 .16
Immediate Settlement (in)

5%
10%



THE EFFECT OF PILE SPACING ON LOAD DISTRIBUTION AT SMALL SETTLEMENTS

FIG. 7.8



PILE BASE LOAD DISTRIBUTION CENTRE TO CORNER DRIVING PATTERN

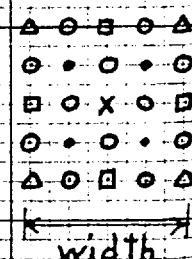
FIG. 7.9

Spacing = $2d$

Width = $9d$

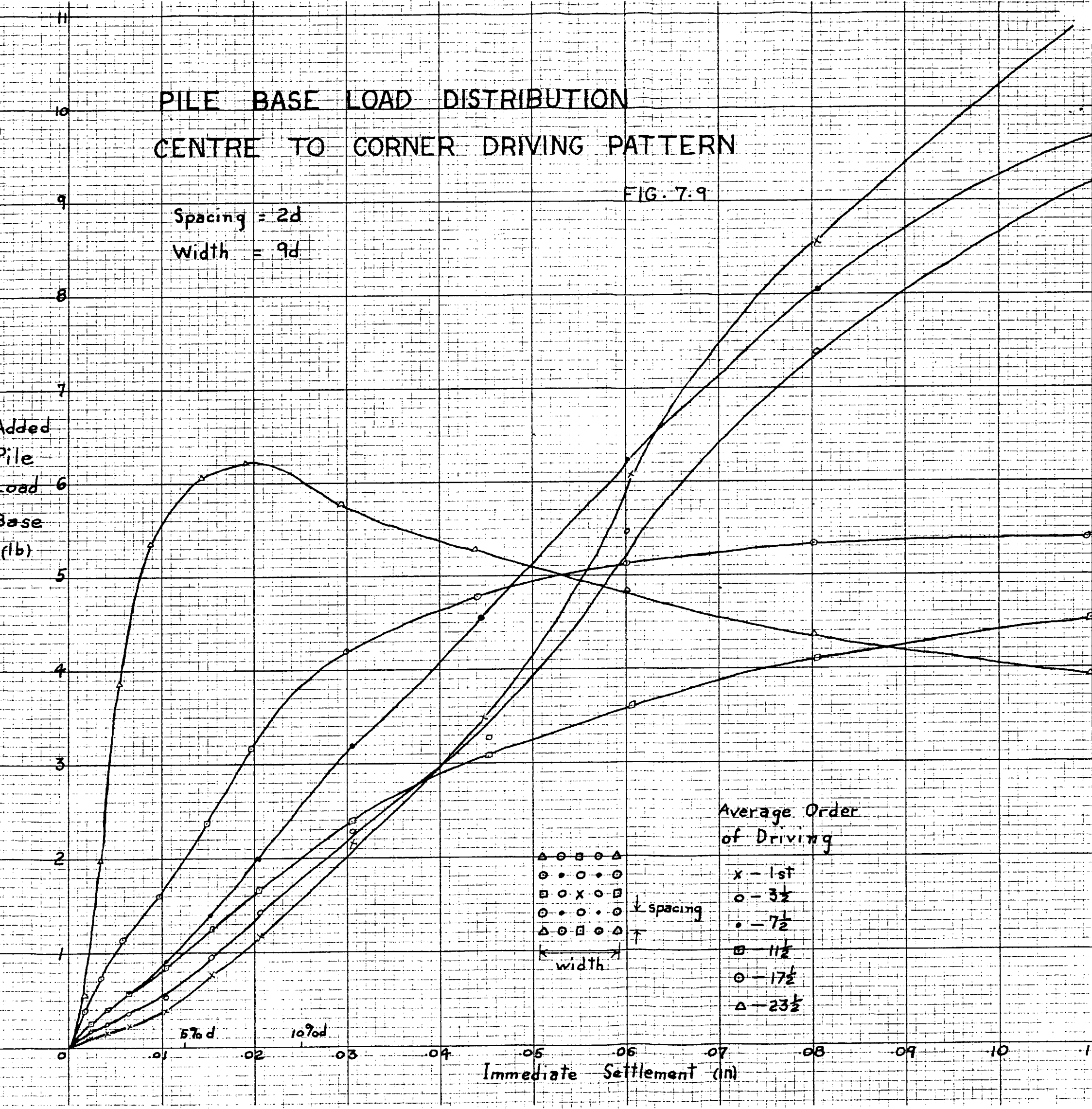
Added
pile
load
base
(lb)

Immediate Settlement (in)



Average Order
of Driving

- x - 1st
- o - $3\frac{1}{2}$
- - $7\frac{1}{2}$
- - $11\frac{1}{2}$
- - $17\frac{1}{2}$
- △ - $23\frac{1}{2}$



PILE BASE LOAD DISTRIBUTION CENTRE TO CORNER DRIVING PATTERN

FIG. 7.10

Spacing = $4d$

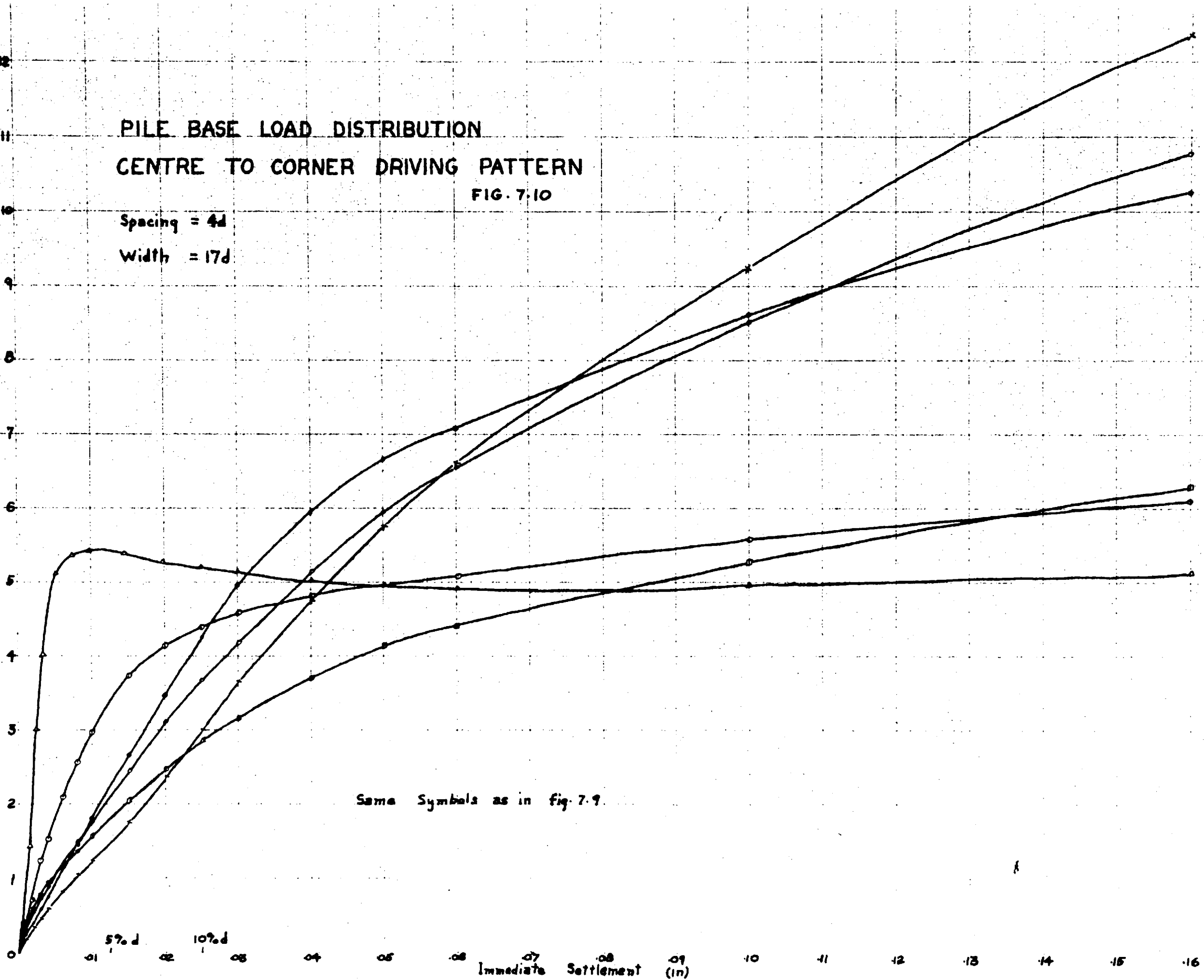
Width = $17d$

Added
Pile
Load

Base
(lb)

Same Symbols as in fig. 7.9.

0 .01 .02 .03 .04 .05 .06 .07 .08 .09 .10 .11 .12 .13 .14 .15 .16
Immediate Settlement (in)



PILE BASE LOAD DISTRIBUTION CENTRE TO CORNER DRIVING PATTERN

FIG. 7.11

Spacing = $6d$
Width = $25d$

Added
Pile
Load
Base
(lb)

11
10
9
8
7
6
5
4
3
2
1
0

5% d
10% d
01
02
03
04
05
06
07
08
09
10
11
12
13
14
15

Immediate Settlement (in)

Same Symbols as in fig. 7.9

0 01 02 03 04 05 06 07 08 09 10 11 12 13 14 15

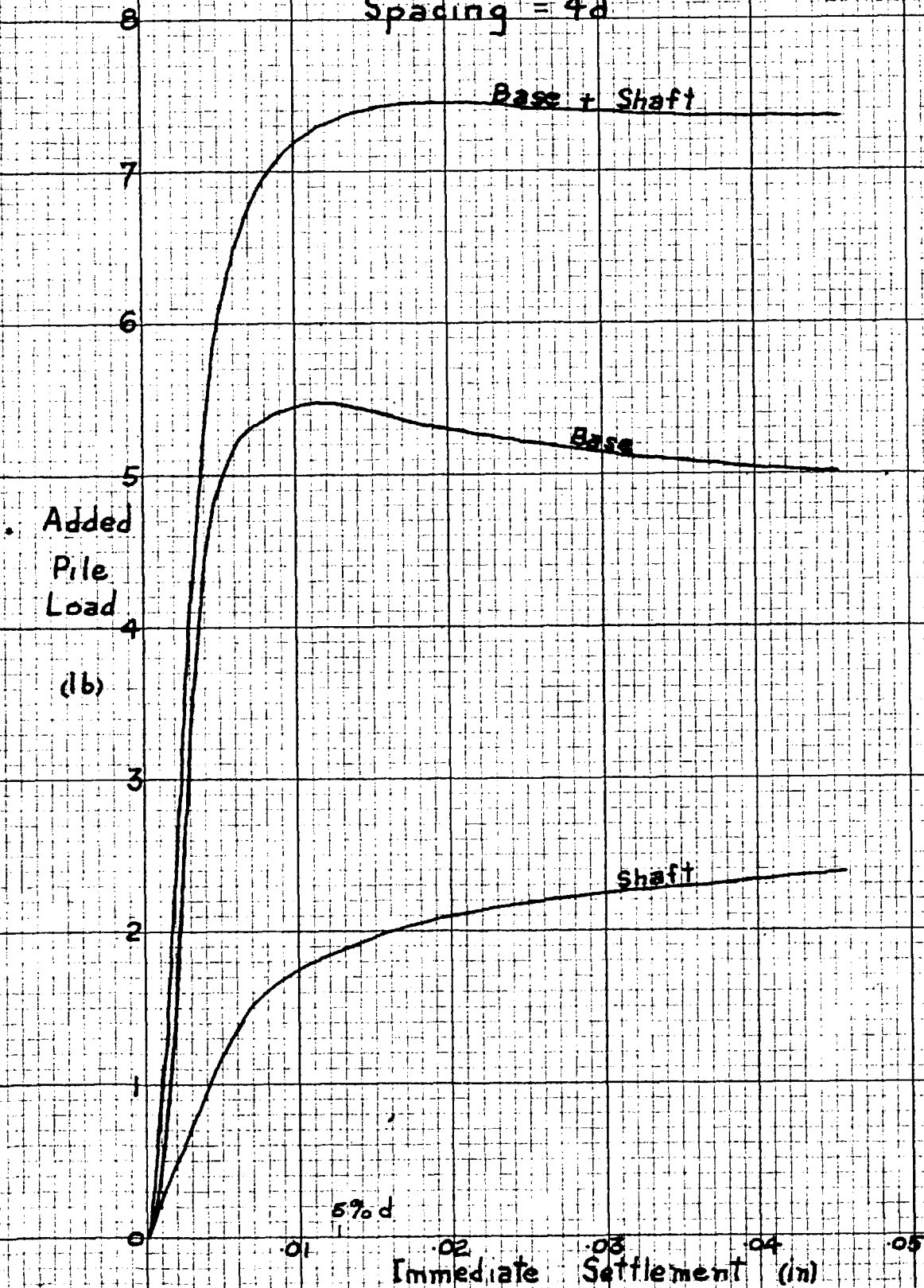
0 01 02 03 04 05 06 07 08 09 10 11 12 13 14 15

LOAD - SETTLEMENT CURVES FOR CORNER PILES:

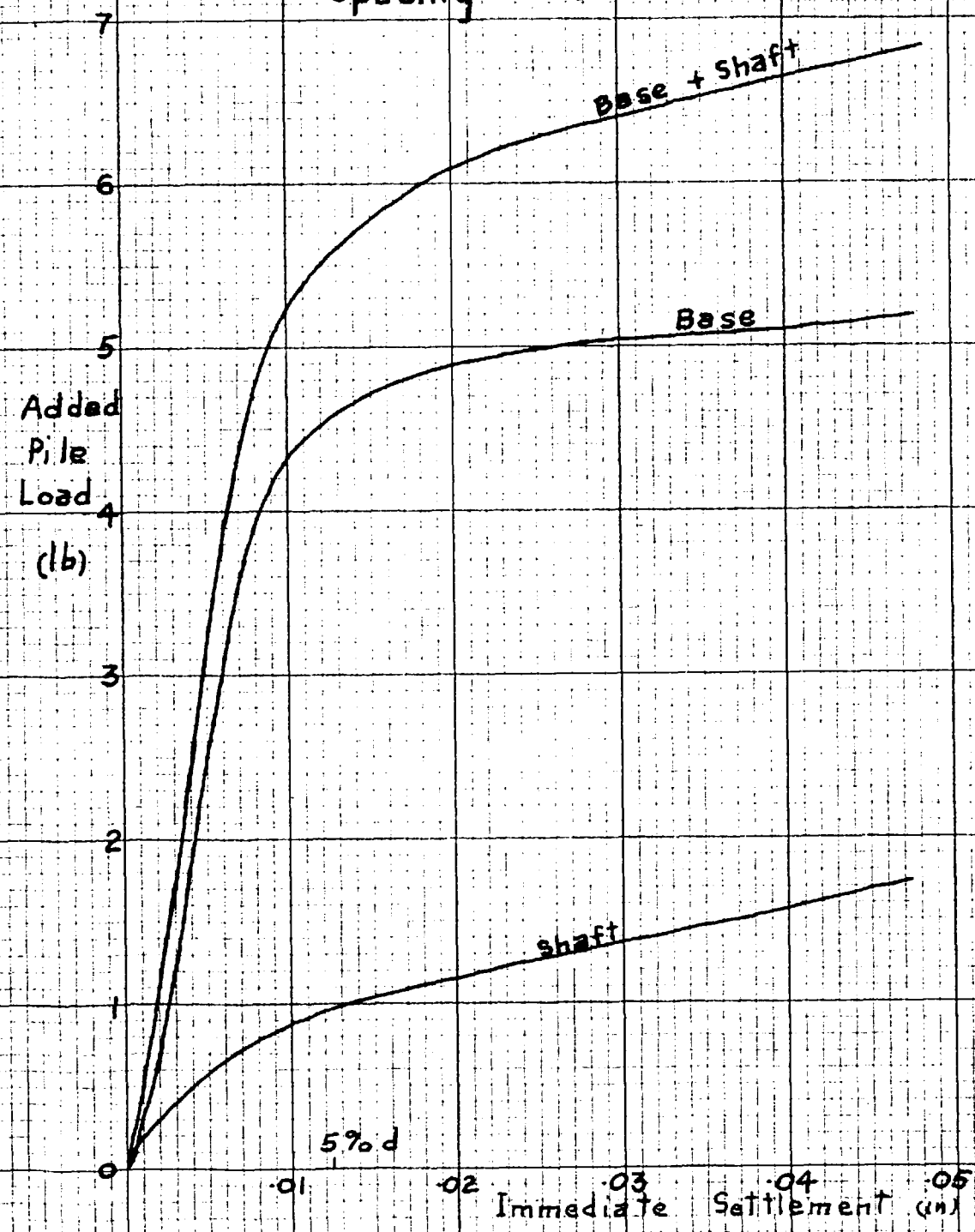
CENTRE TO CORNER DRIVING PATTERN

FIG. 7.12

Spacing = $4d$

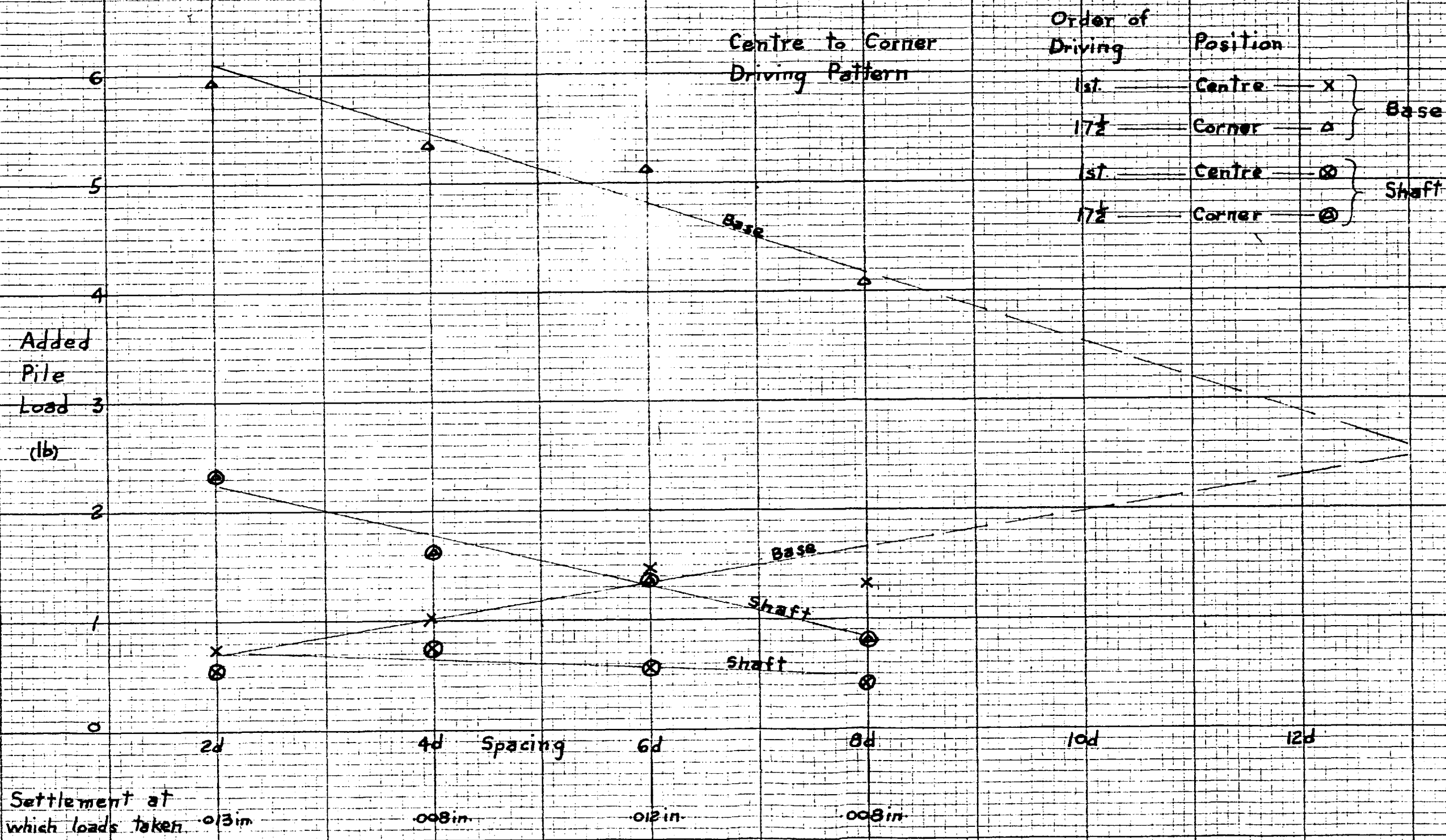


Spacing = $8d$



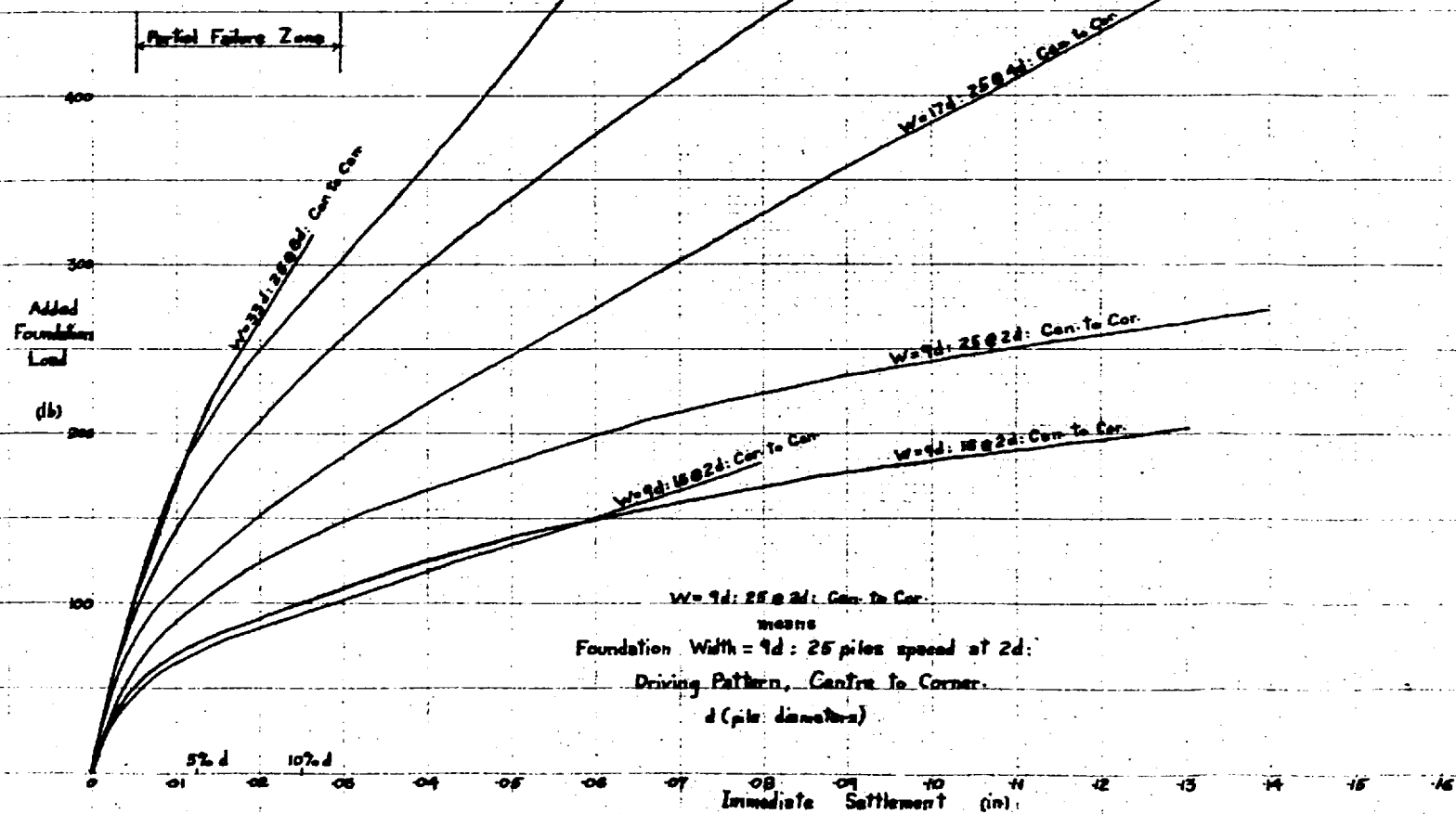
THE EFFECT OF PILE SPACING ON BASE AND SHAFT LOAD DISTRIBUTION AT SMALL SETTLEMENTS

FIG. 7.13



FOUNDATION LOAD - SETTLEMENT CURVES

FIG. 7.19



FOUNDATION LOAD DISTRIBUTION

FIG. 7-15

Foundation Width = 33d
 Pile Spacing = 8d
 25 Piles
 Driving Pattern:
 Centre to Corner

Added
 Load
 (lb)

Foundation Load

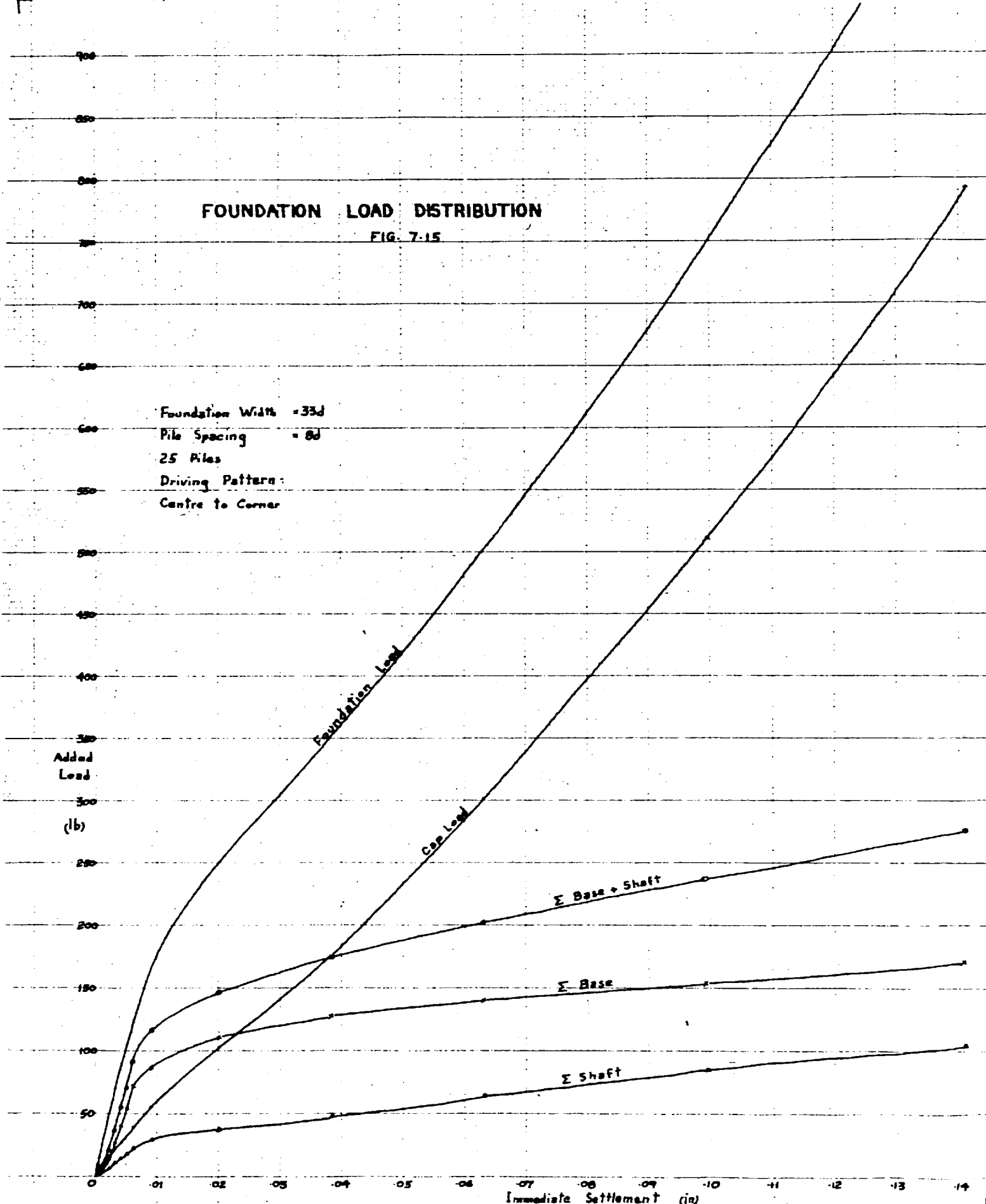
Cap Load

Σ Base + Shaft

Σ Base

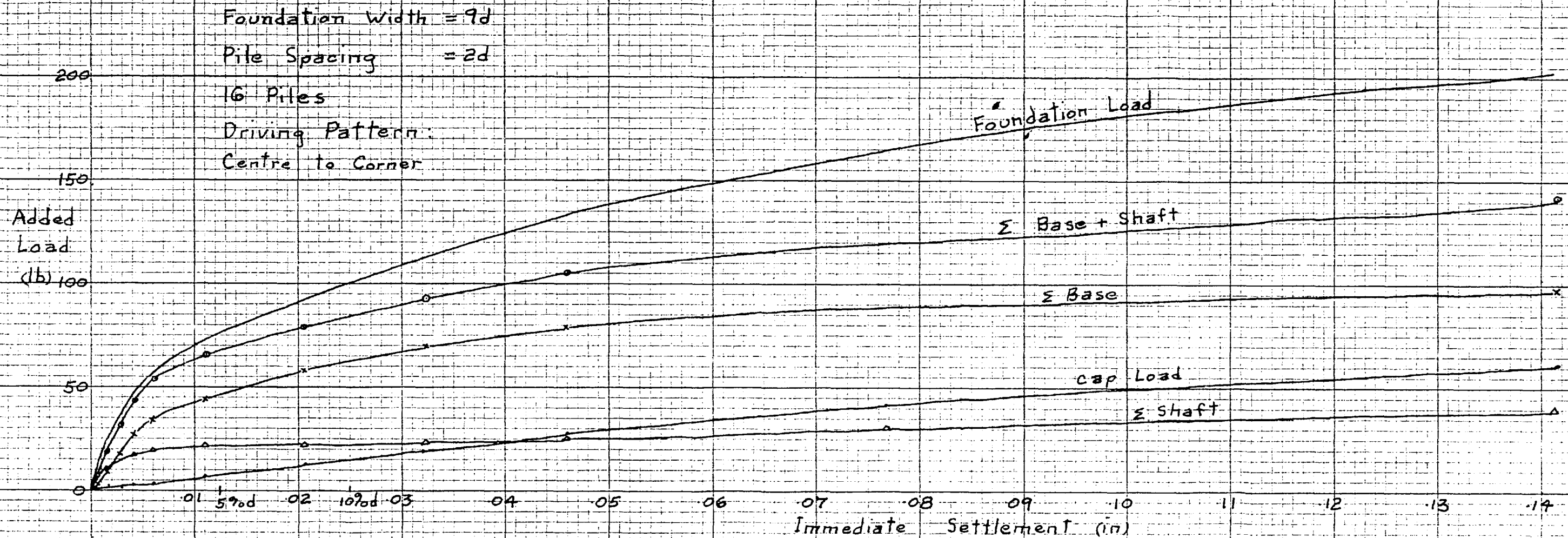
Σ Shaft

Immediate Settlement (in)



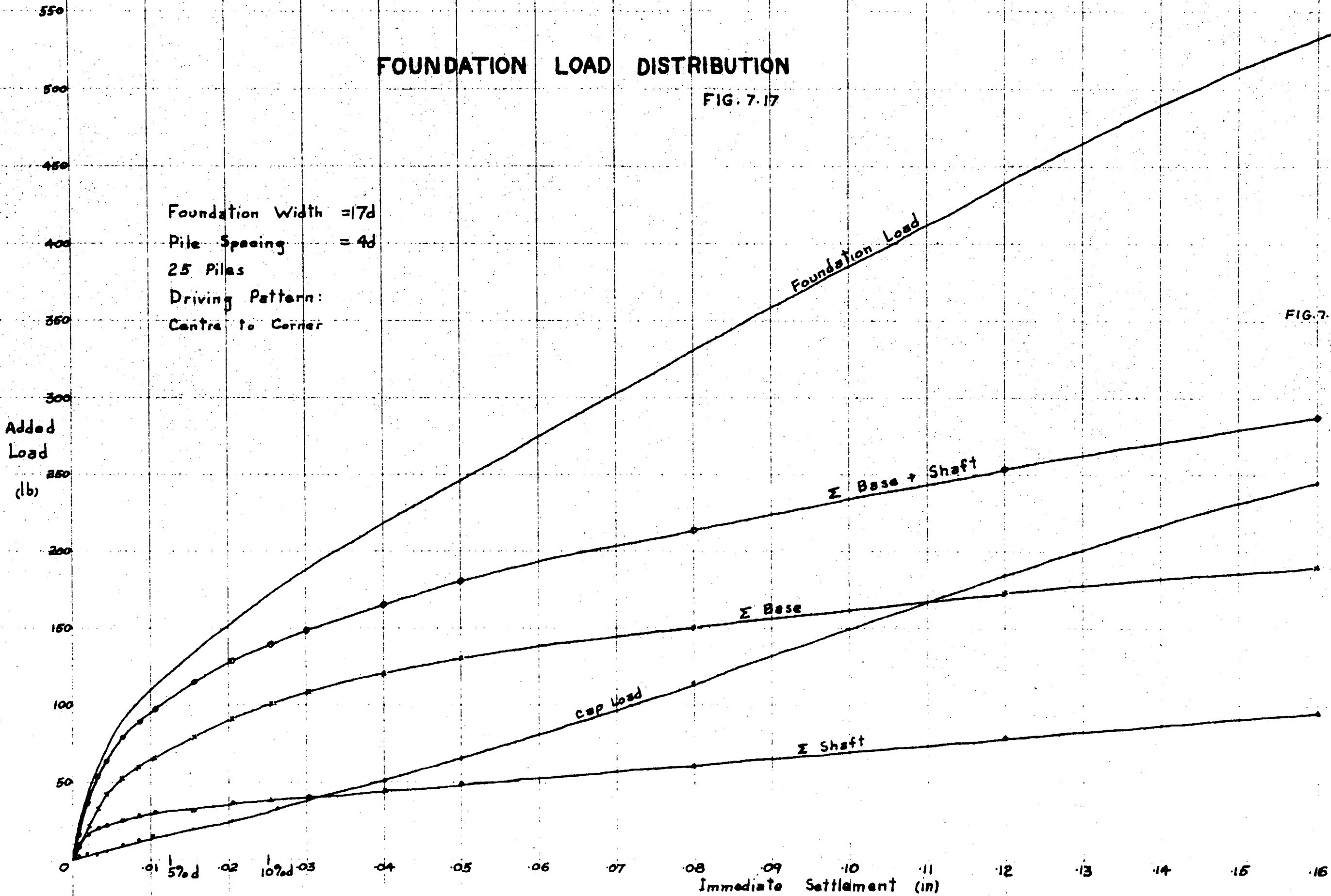
FOUNDATION LOAD DISTRIBUTION

FIG. 7.16



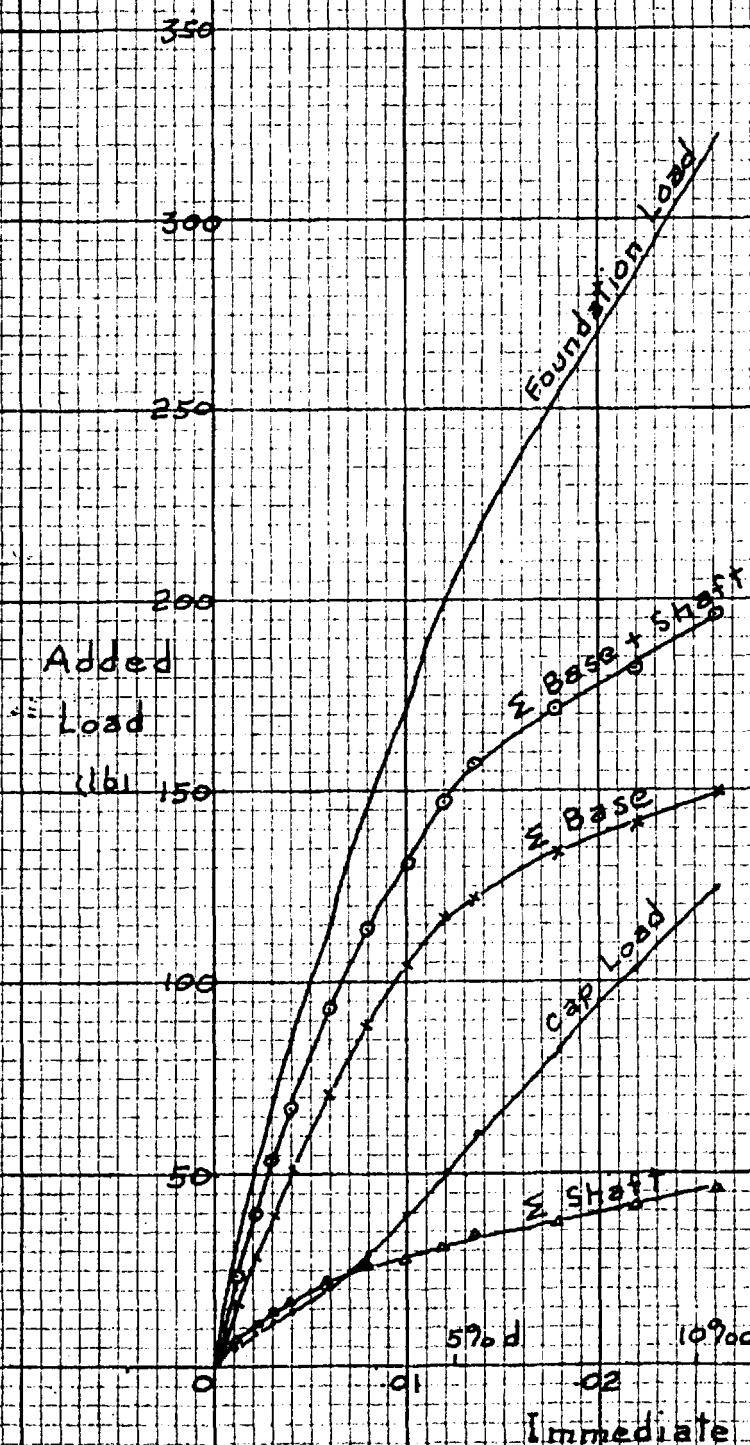
FOUNDATION LOAD DISTRIBUTION

FIG. 7.17



FOUNDATION LOAD DISTRIBUTION

FIG. 7.18



Foundation Width = $33d$

Pile Spacing = $8d$

25 Piles

Driving Pattern:

Corner to Centre

THE EFFECT OF ORDER OF DRIVING ON PILE LOAD

FIG. 8.1

Spacing = $4d$

Driving Pattern: Centre to Corner

Added Load
Base
+
Shaft
(lb)

Settlement
= .008 in

8

7

6

5

4

3

2

1

0

1

2

4

6

8

10

12

14

16

18

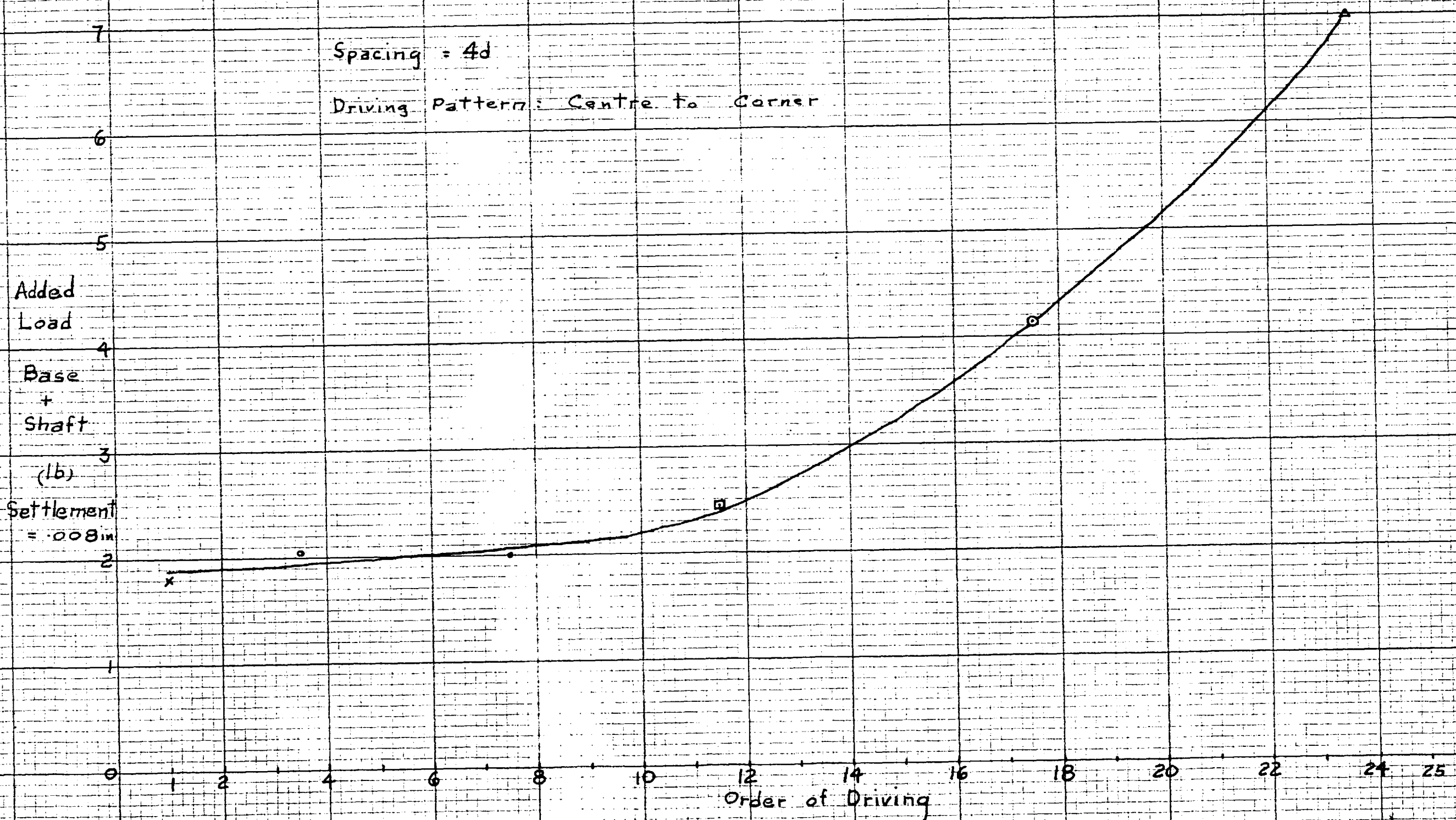
20

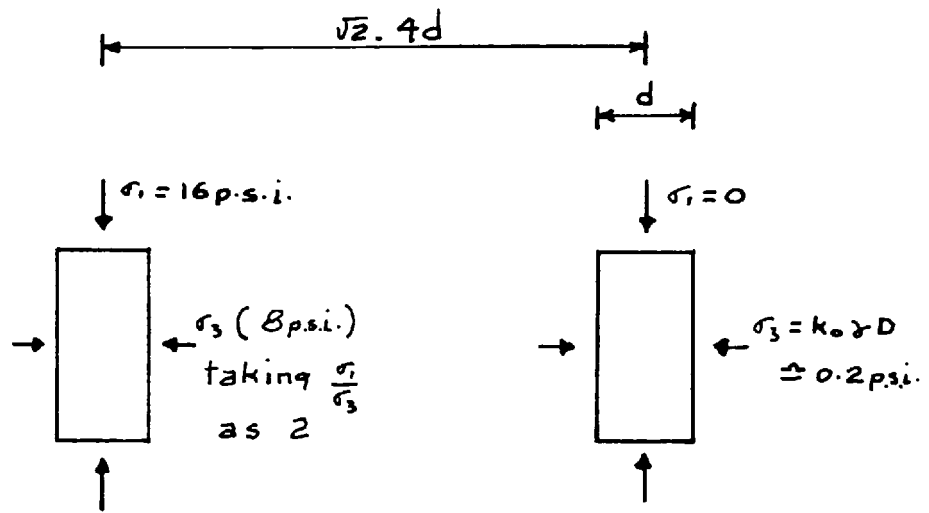
22

24

25

Order of Driving





Sample B

Beneath
25th Driven Pile

Sample A

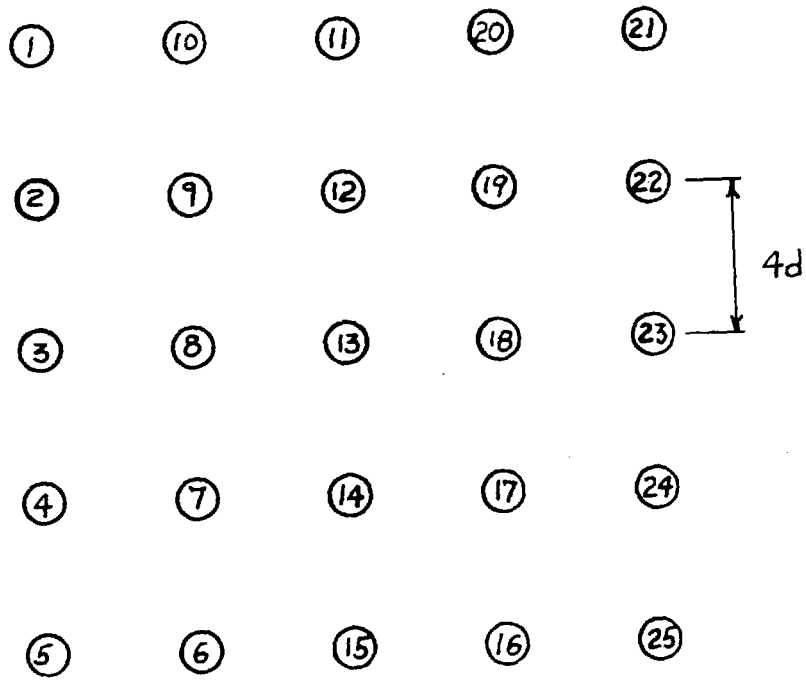
Beneath
8th Driven Pile

At this porosity, $\frac{\sigma_v}{\sigma_h}$ (at failure) $\approx 4\frac{1}{2}$

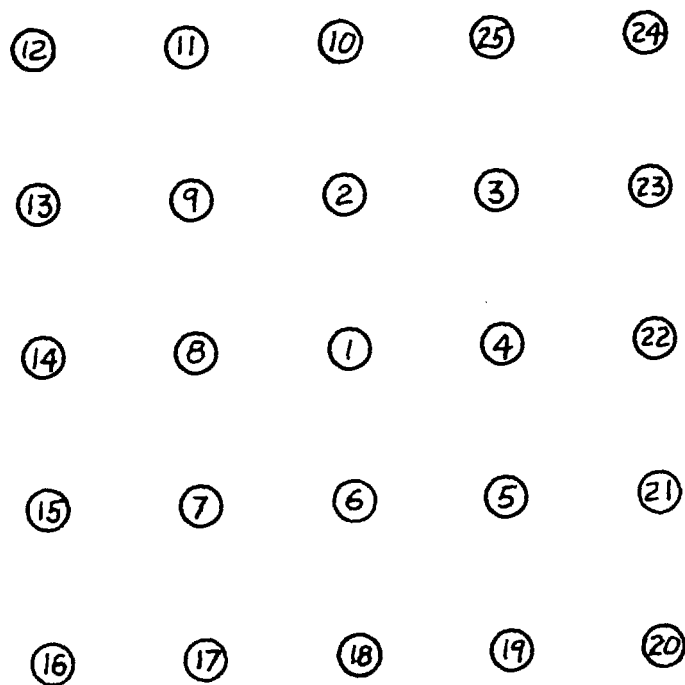
25 Piles spaced at $4d$,

Centre to Corner Driving Pattern

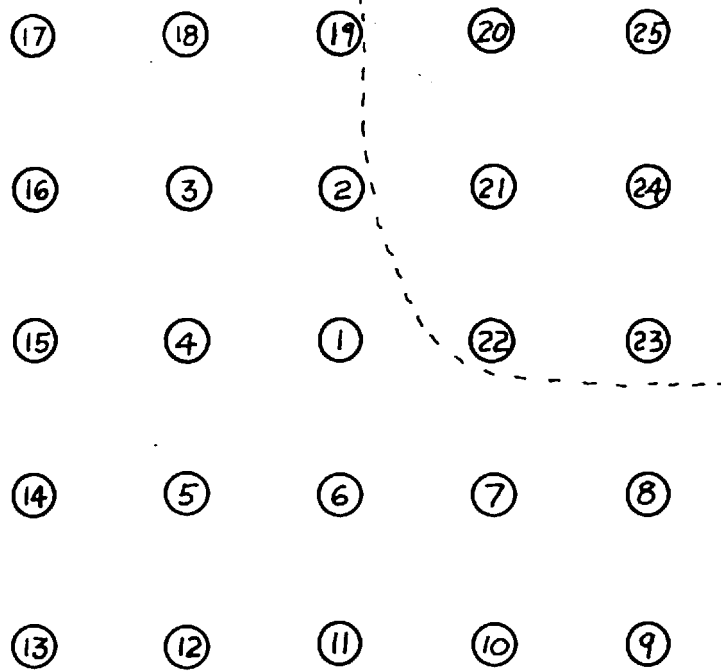
STRESS IN SAND DIRECTLY BENEATH
2 PILES AFTER DRIVING 25 FIG. 8.2



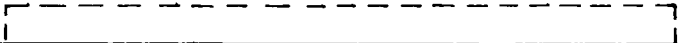
Side to Side Driving Pattern
FIG. 9.1

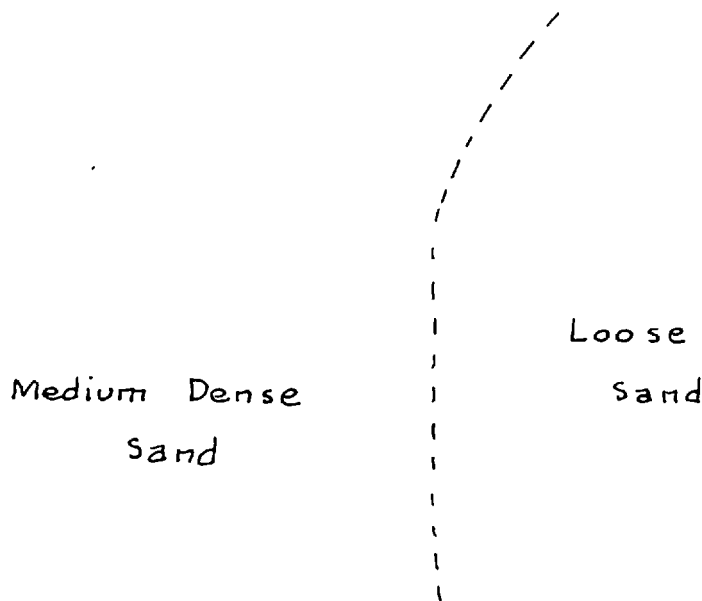


A Practical Centre to
Corner Driving Pattern
FIG. 9.2

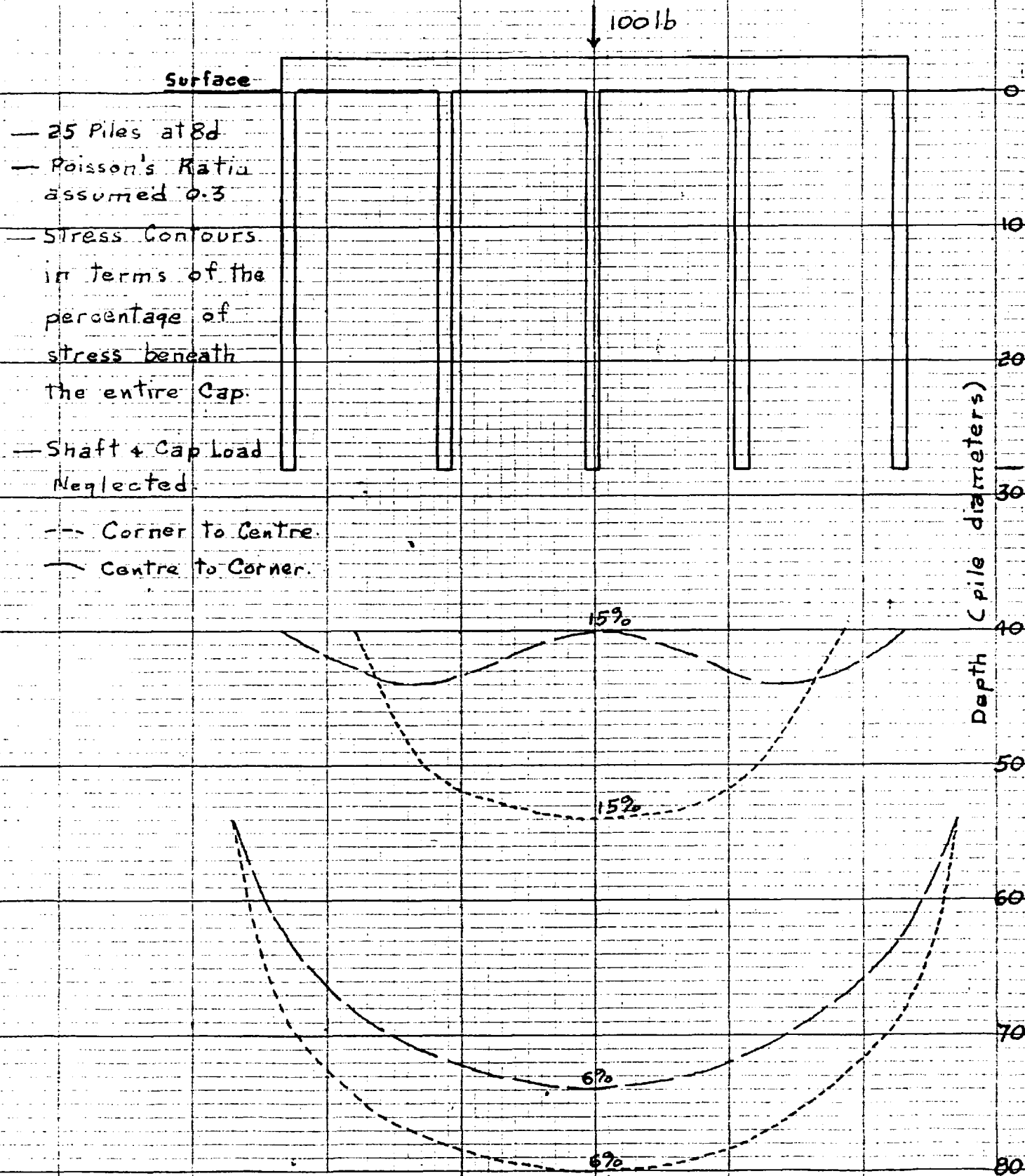


Suggested Driving Pattern FIG. 9.4

Surface 



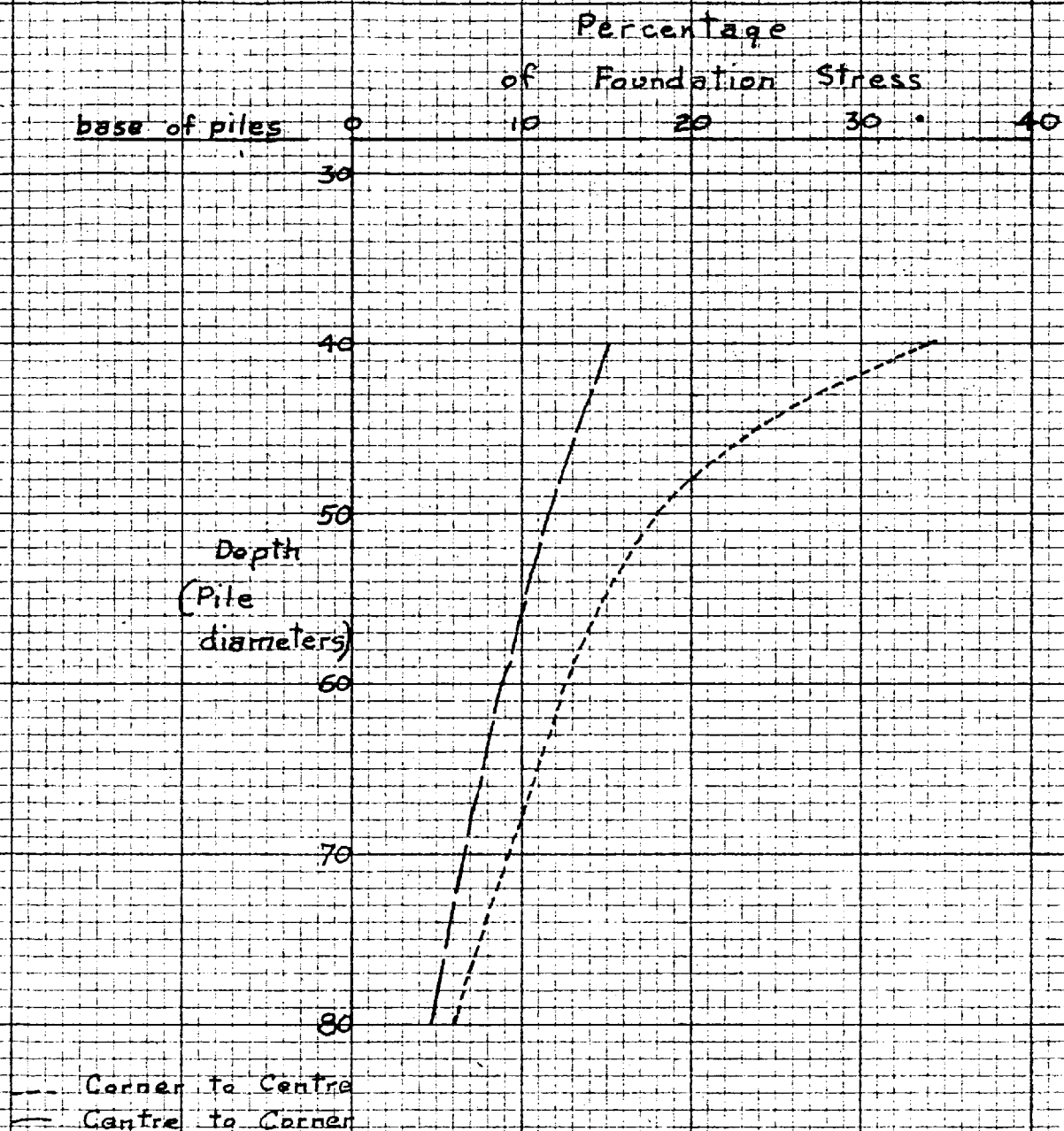
Variable Deposit of Sand on which Pile Foundation to be Constructed FIG. 9.3



VERTICAL STRESS DISTRIBUTION FOR

2 DRIVING PATTERNS

FIG. 9.5

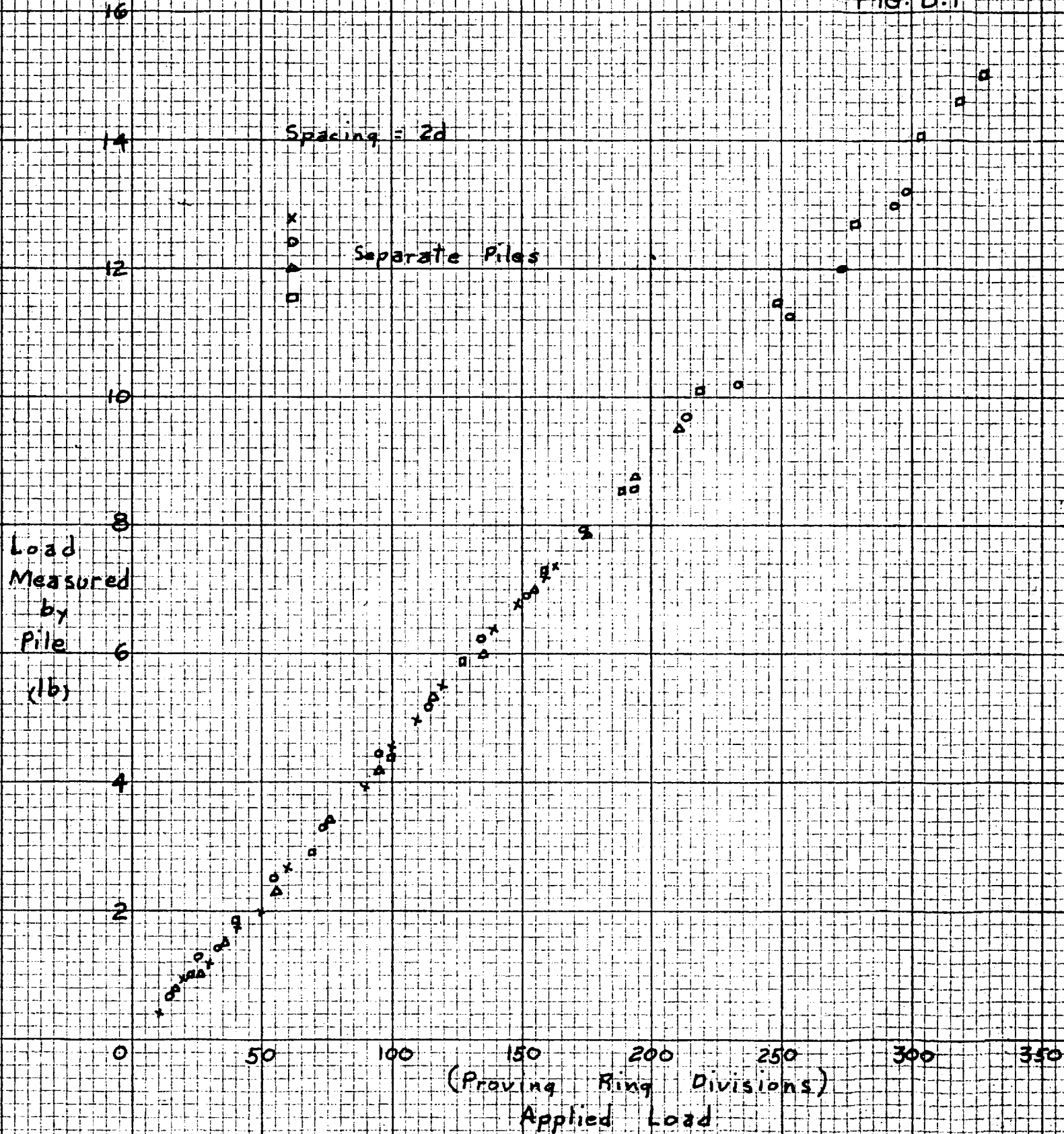


VERTICAL STRESS UNDER
THE CENTRE PILE OF FIG. 9.5

FIG. 9.6

CORRELATION BETWEEN APPLIED LOAD AND MEASURED LOAD DURING SINGLE PILE TESTS

FIG. B.1

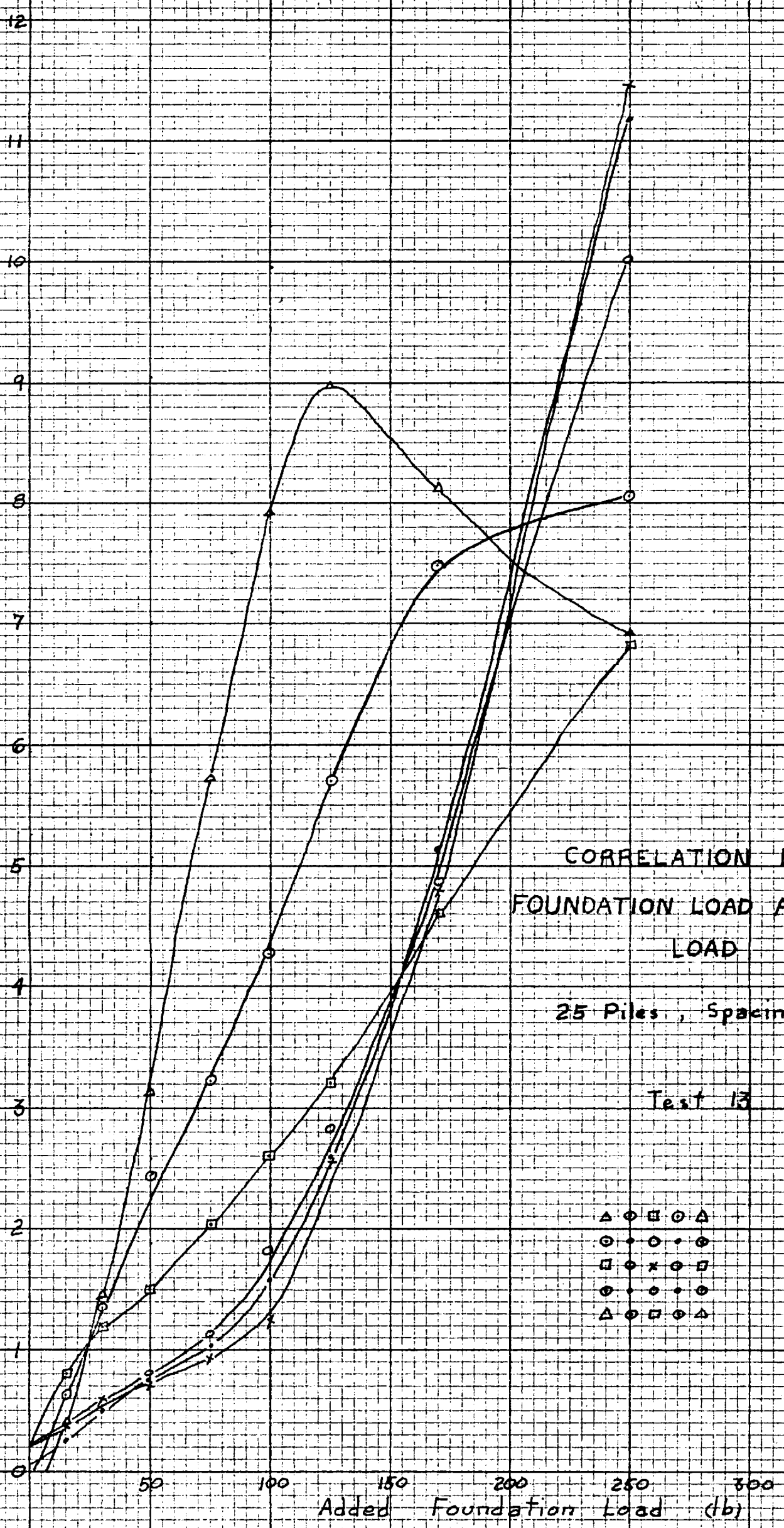
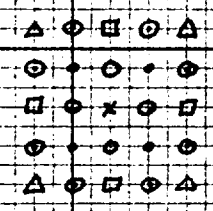


Added
Pile
Load
Base
+
Shaft
(lb)

CORRELATION BETWEEN ADDED
FOUNDATION LOAD AND ADDED PILE
LOAD
FIG. G-1

25 Piles , Spacing = 2d

Test 13



Added
Pile
Load
Base
+
shaft

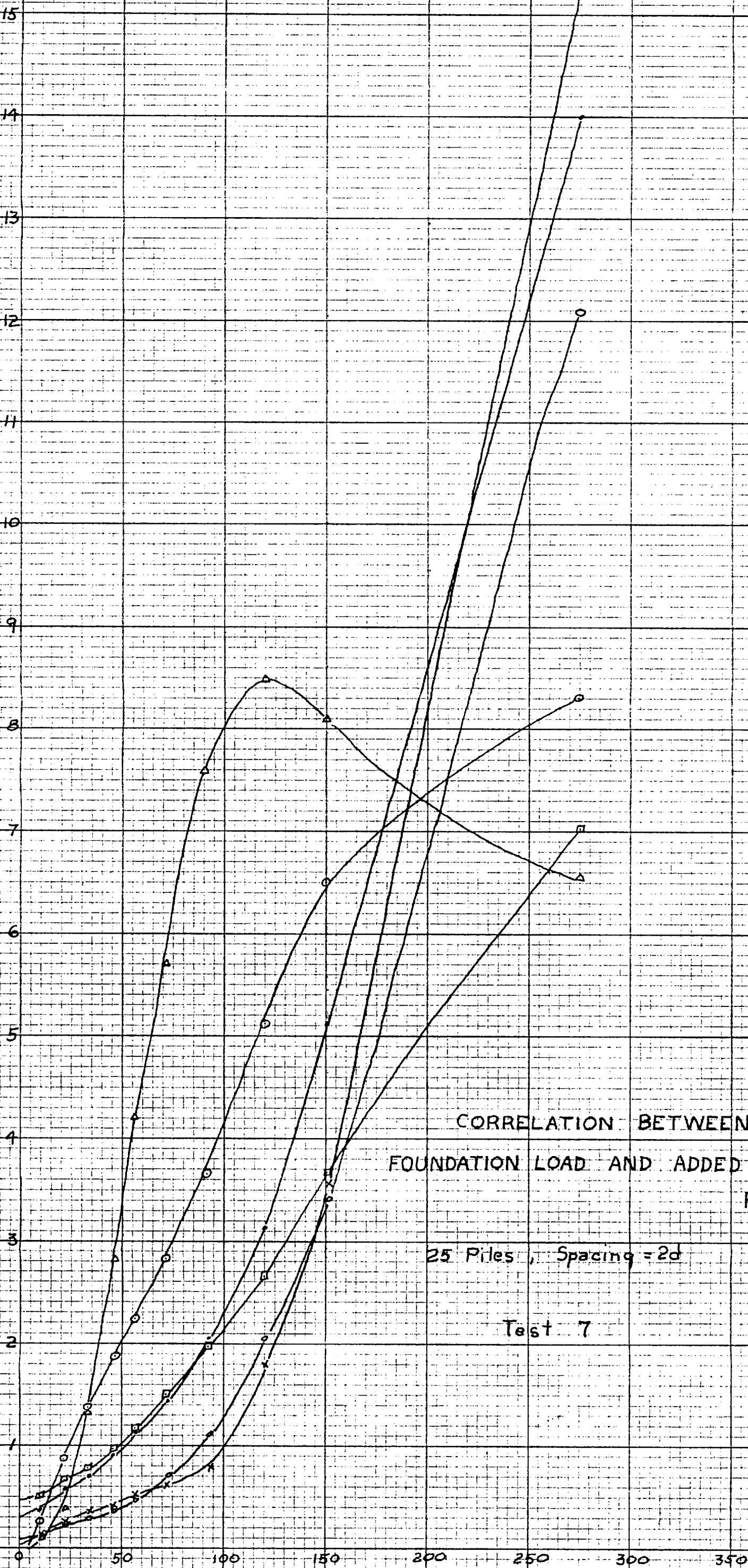
(lb)

CORRELATION BETWEEN ADDED
FOUNDATION LOAD AND ADDED PILE LOAD
FIG-G-2

25 Piles , Spacing = $2d$

Test 7

Added Foundation Load (lb)



FOUNDATION LOAD-SETTLEMENT DIAGRAM

FIG. C-3

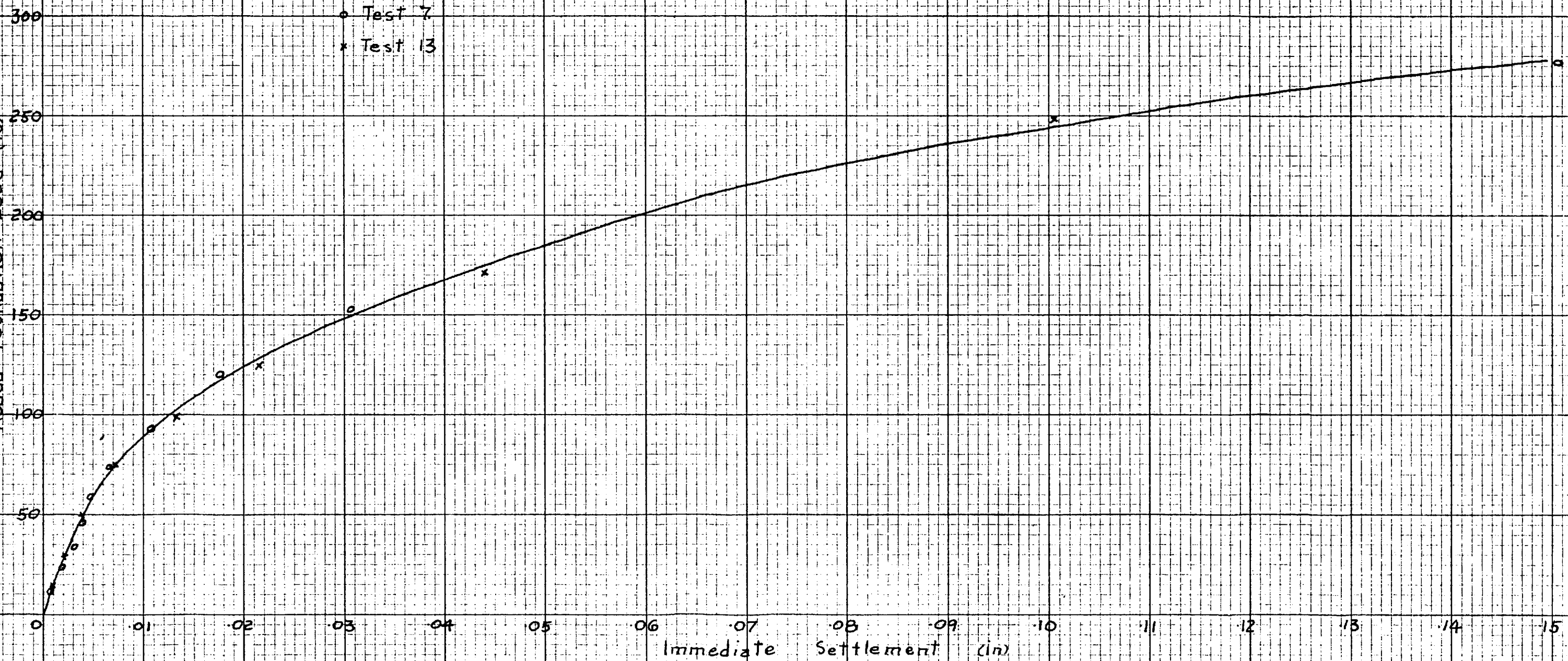


FIG. C-3

LOAD ERROR IN CORNER PILES

FIG. C-4

25 Piles, Spacing 4d

Test 8

Order
of
Driving

Symbol

22

.

23

x

24

o

25

⊕

23 1/2 (Ave) Δ

-- Limits of Error.

Added
Pile
Load 5
Base
+
shaft

(lb)

9

8

7

6

5

4

3

2

1

0

50

100

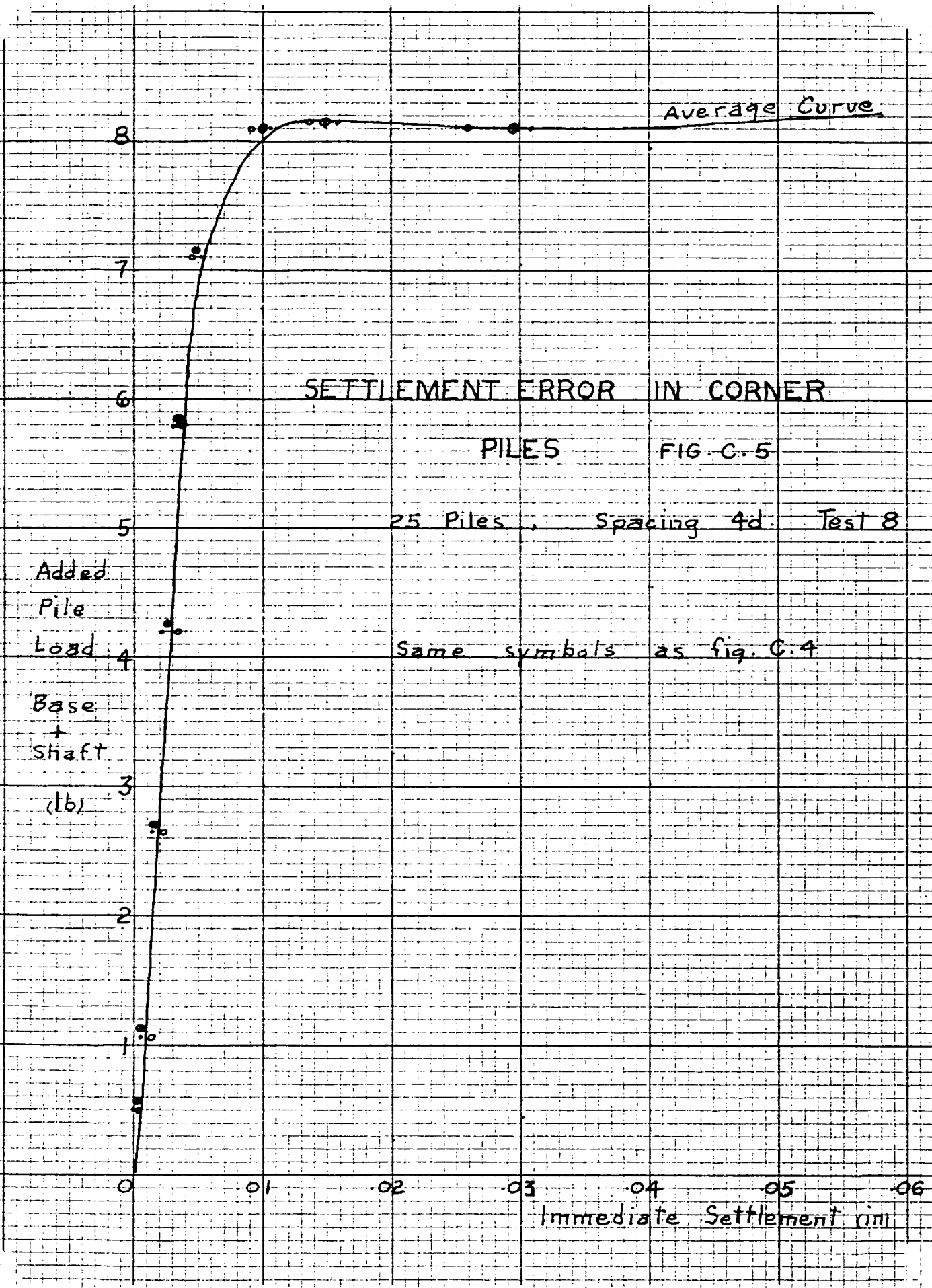
150

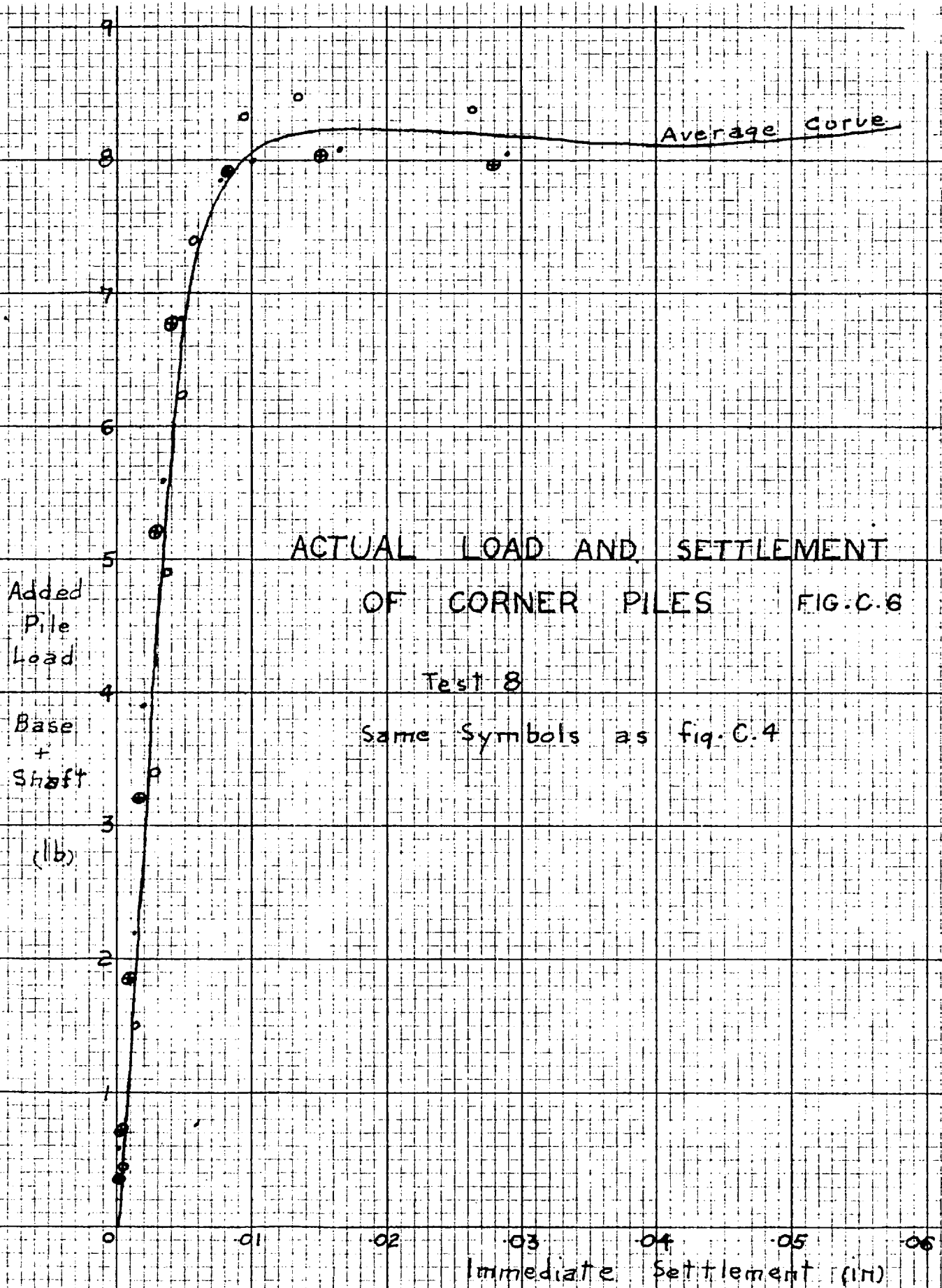
200

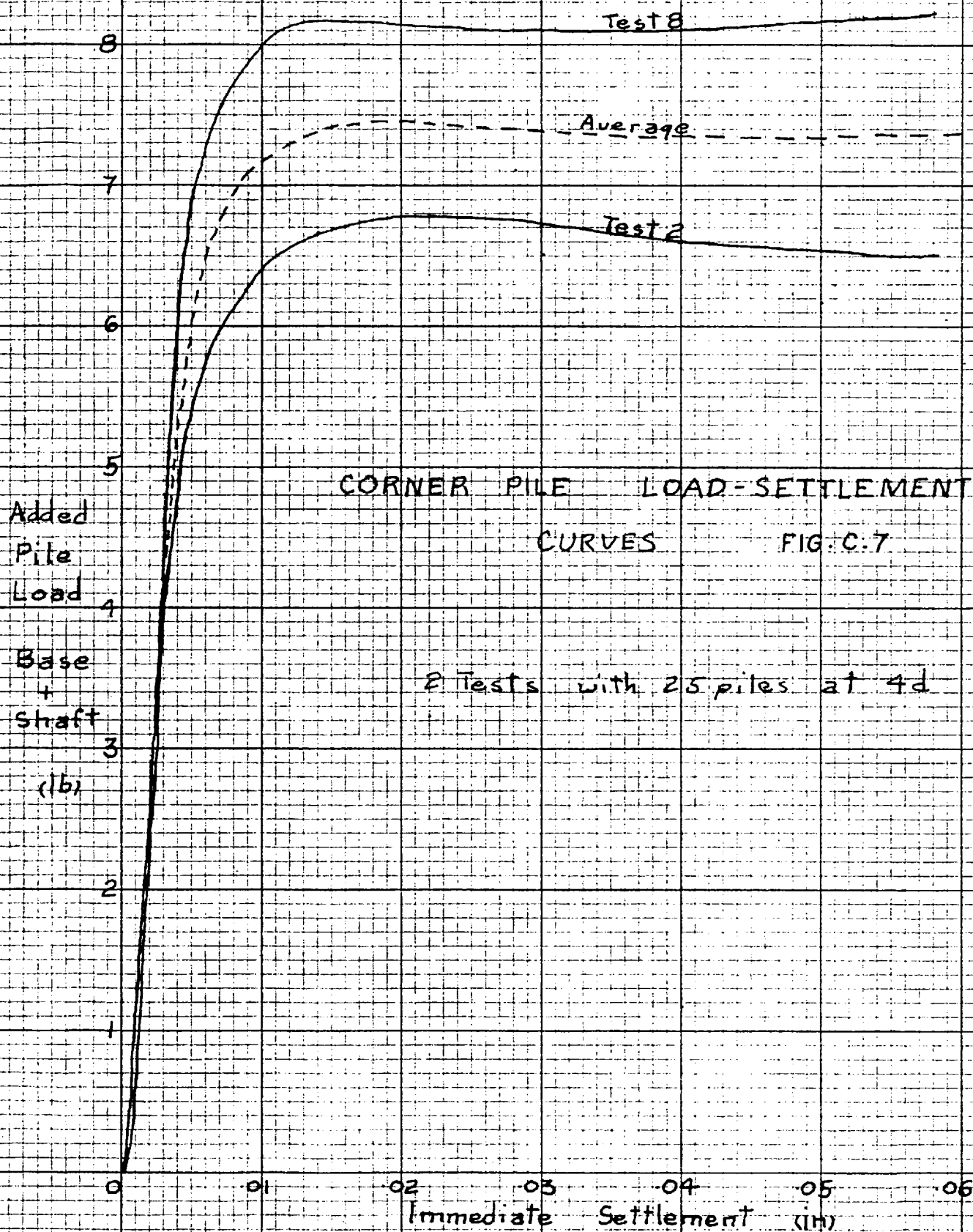
250

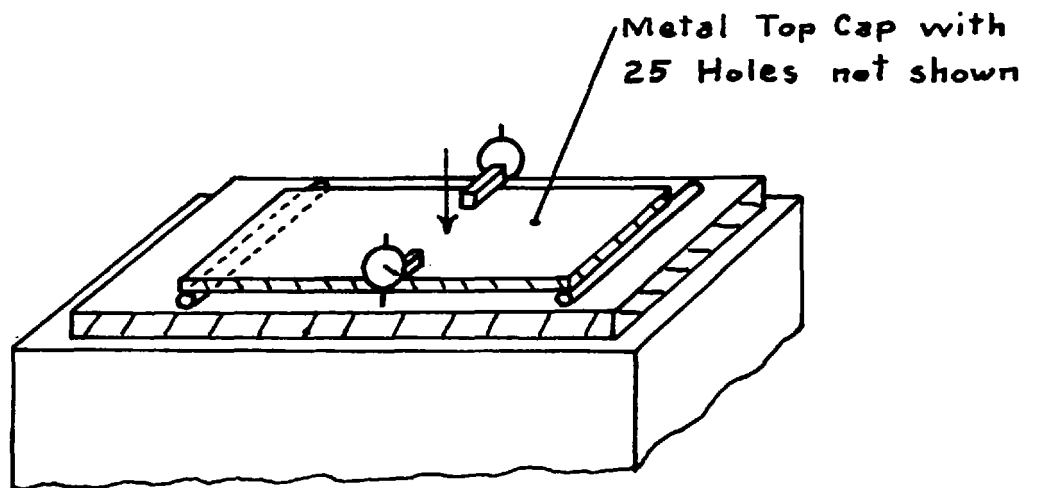
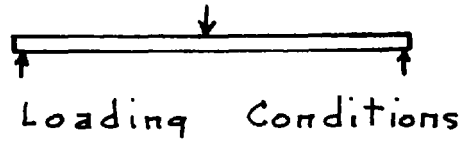
300

Added Foundation Load (l)









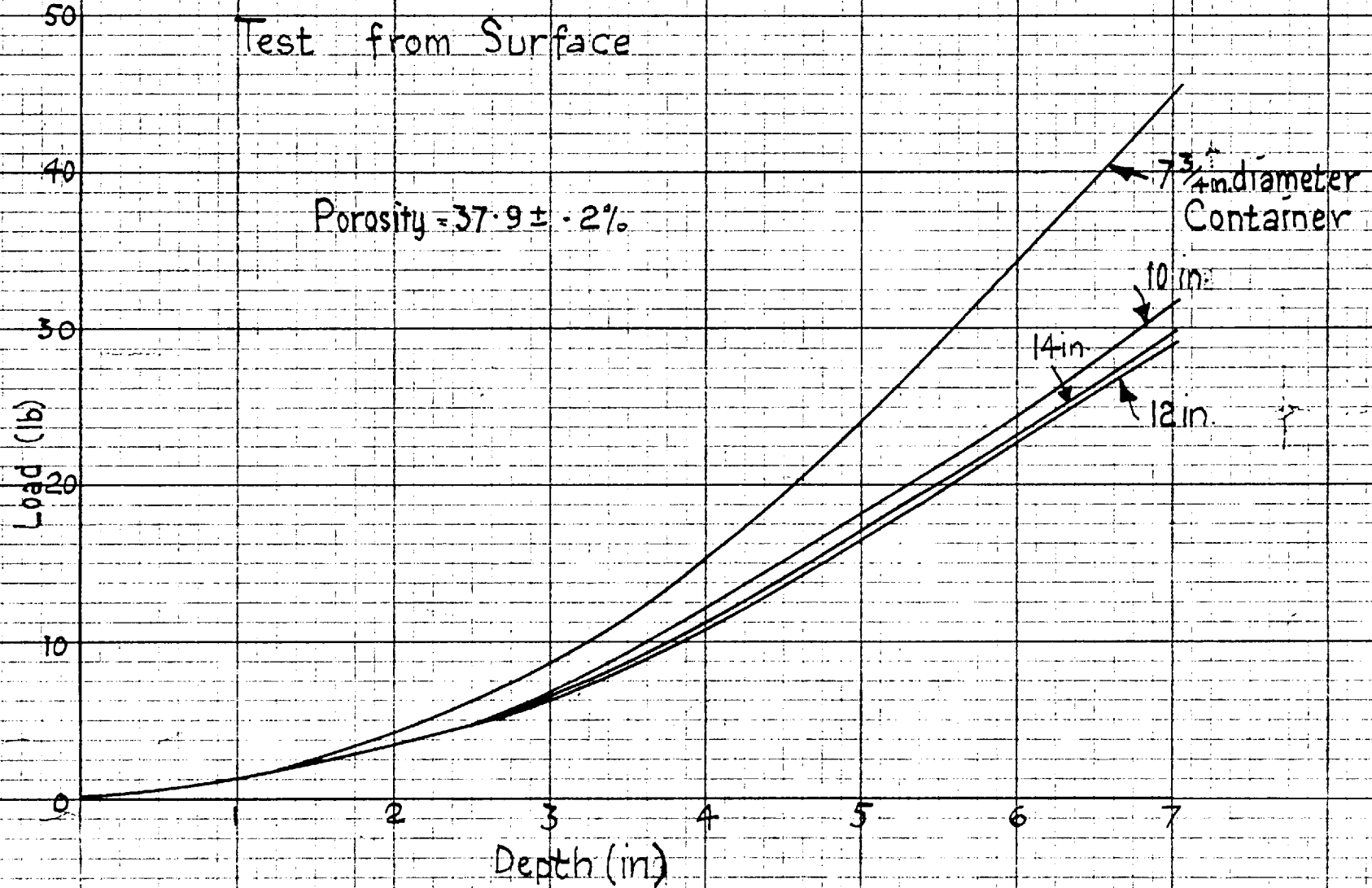
ARRANGEMENT FOR MEASURING
TOP CAP FLEXIBILITY FIG. D.1

CONTAINER EFFECT

FIG. E.1

Constant Rate of Penetration
Test from Surface

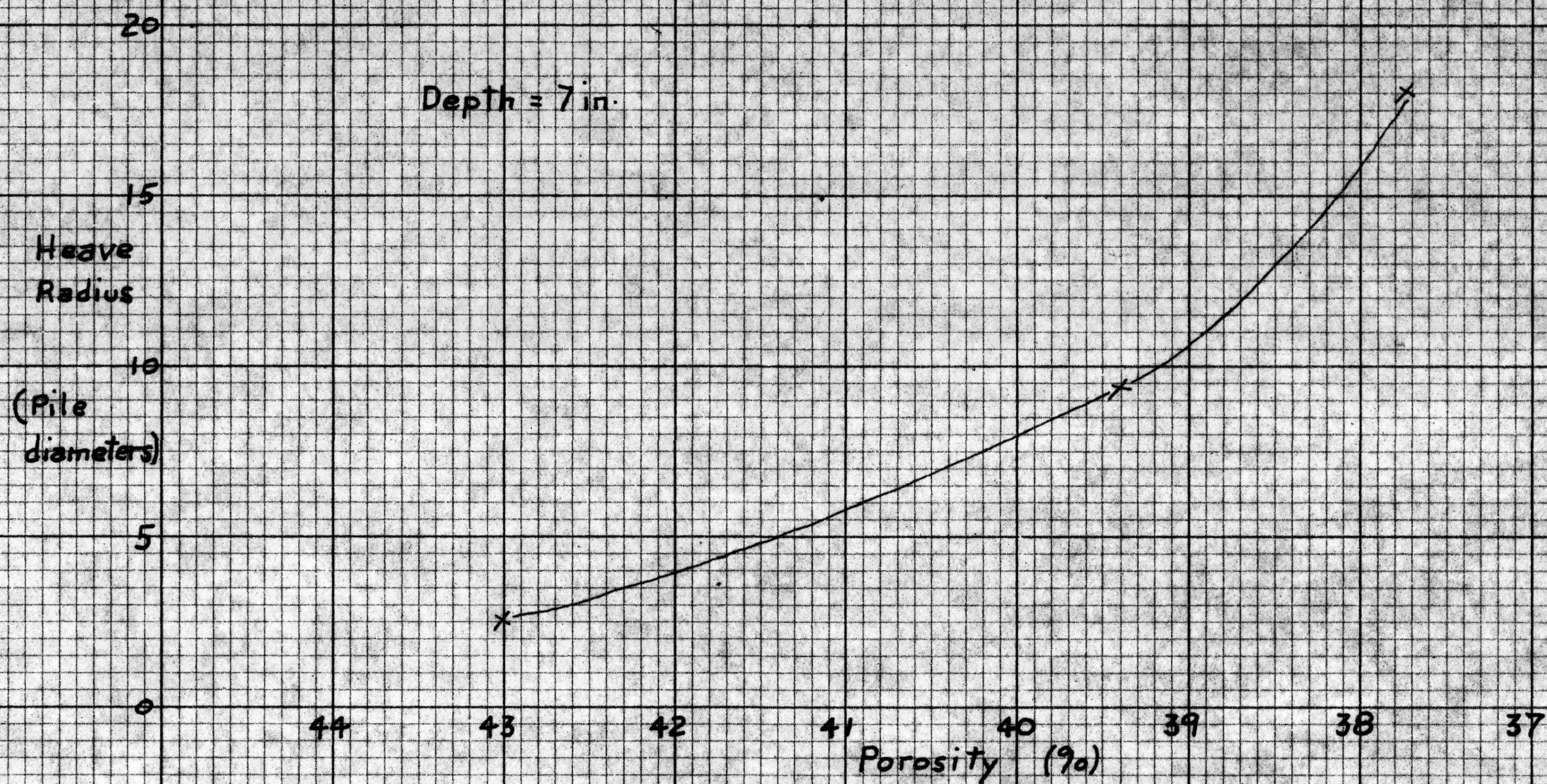
Porosity = $37.9 \pm .2\%$

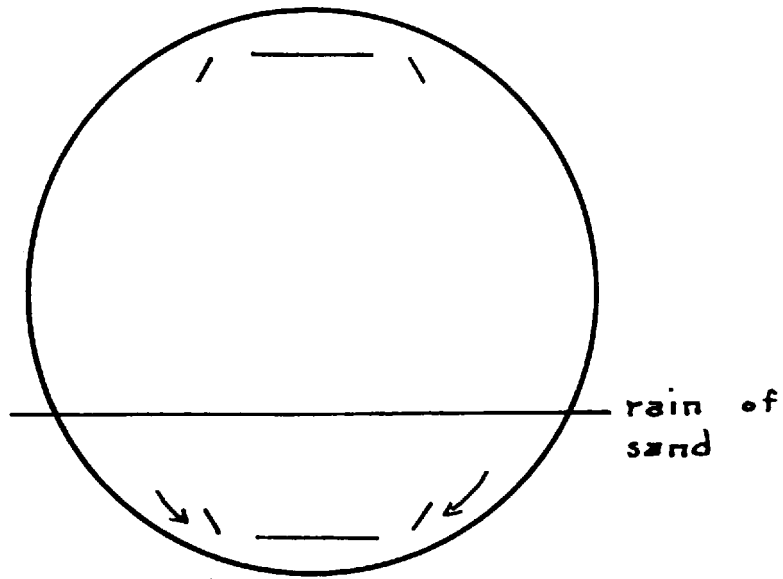


CONTAINER EFFECT

FIG. E.2

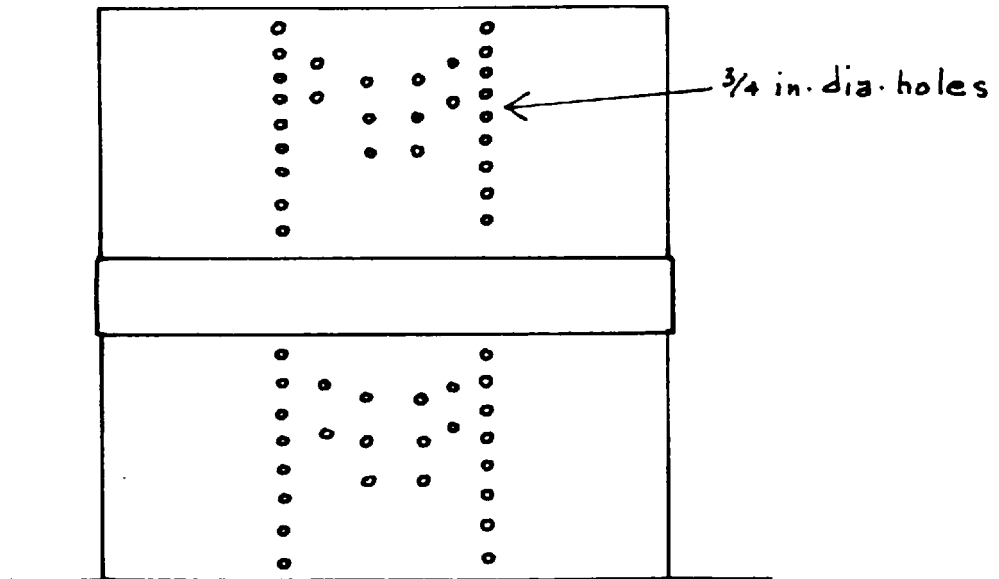
Horizontal Extent of Heave from
a Driven Pile





Position of Baffle Plates

1 ft.



Position of Holes in Container

CORRECTIVE MEASURES FOR AIR CURRENTS
DURING DEPOSITION FIG.F.1