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MODEL STUDIES OF THE LOAD DISTRIBUTION
WITHIN GROUPS OF FRICTION PILES
IN A COHESIVE SOIL

A THESIS

Presented to

the Faculty of the Graduate Division

by

Lyle Lawrence Wilson

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ABSTRACT

MODEL STUDIES OF LOAD DISTRIBUTION
WITHIN GROUPS OF FRICTION PILES
IN A COHESIVE SOIL

(89 pages)

Lyle Lawrence Wilson

Directed by Professor George F. Sowers

The purpose of this investigation was to further the development of a rational approach to the design of friction pile foundations. This objective was accomplished by measuring the distribution of load that occurred within groups of model friction piles in a cohesive soil.

Load tests were conducted on a single pile and on groups of nine piles. SR-4 electric strain gages, mounted inside of the tubular model piles, were used to measure the distributions of load.

The results of the tests show that the bearing capacity of a group became greater as the pile spacings increased. The greatest variation in the group bearing capacities occurred between the pile spacings of one and one-half and two and one-half diameters.

The group test with pile spacings of one and one-half diameters performed with the soil and the piles acting entirely as a unit. At pile spacings of two and two and one-half diameters, the groups behaved in a manner that was a combination of unit action and individual pile action. When the spacings were three and one-half diameters or more, the action of the piles within the group was similar to that of a single pile.

It was found that a rigid cap did not distribute load to the piles in equal amounts, but that piles on the edge of the group received more load than did the center pile. At close spacings, the test results showed that adjacent piles interfered with one another's ability to develop adhesion along their surfaces.

A plastic redistribution of stress within the soil was observed. When the soil reached a maximum stress at any one point, it did not immediately fail. Instead the stress became constant at that point. Additional increases in load were then distributed to other locations, within the soil, that were not as highly stressed.

It is hoped that tests of this nature will be continued. These tests should be extended to include variations in the soil properties, number of piles, length of piles, and the shape of the group. An accumulation of data of this type will be necessary before a comprehensive theory for the behavior of friction piles in a cohesive soil can be developed.

CHAPTER I

INTRODUCTION

The Problem.--It is often necessary to place structures with heavy foundation loads on deep strata of soft soils. At these locations it is usually not economical and sometimes not physically possible to place the foundations on firm materials. Therefore, friction piles, embedded in the soft material, may be utilized to carry the loads of the structure. These piles derive their support from skin friction along their full length which transmits load to the surrounding soil.

The design of the friction pile group is one of the least rational and most empirical operations in foundation engineering. Numerous failures have proven there is great need for an accurate method of determining load carrying capacity of groups of piles that depend mainly on skin friction for support. Improvements in the present methods of design have been restricted by lack of knowledge and test data.

A major cause of pile foundation failures in the past has been the application of single pile, load test results to the design of a pile group. It has been known for sometime that an average pile load from a group of friction piles does not have the same carrying capacity as that of a single pile. It is also apparent that the spacing of friction piles affects the group carrying capacity. The reason for the lack of information concerning these actions is that it is extremely expensive to load-test groups of piles to failure. Furthermore, it is difficult to construct a model pile foundation test that will simulate actual conditions.

The behavior of friction pile groups is a complex problem, involving the distribution of load from the cap to the piles, and the behavior of the pile shafts. It is commonly assumed that in a clay soil exterior piles of a group carry considerably more load than interior piles if all are connected by a rigid cap. That adjacent, closely spaced piles in a group interrupt one another's development of skin friction also becomes apparent in determining group load carrying capacity. Somewhere in the interaction of these variables undoubtedly lies the reason for the behavior of a particular group of friction piles under load.

A final factor, and an important one in the design of friction pile foundations, is the requirement for adequate information concerning the characteristics of the soil in which the piles will be placed. Reliable soil data are absolutely necessary, since they will influence the effectiveness of the analysis regardless of the design approach used.

Brief History.--Evidence exists that pile foundations have been in use for approximately 15,000 years. Methods of design, however, are still based mostly on experience and blind faith. Considerable progress has been made in the past few years, but reliable information is still lacking. The verification of a pile foundation analysis is much more difficult than verifying the action of other component parts of a structure.

Very little data have been recorded and published concerning the performance of actual friction pile foundations. Load testing of a single pile to failure is fairly common, but the useful information obtained as far as group design is concerned, is quite limited. There has been some load testing of both model and full size friction pile groups (1) (2). Although this has added mostly to empirical methods of group design, it has also shown some of the danger areas in the use of friction pile foundations.

In recent years there has been interest shown in the ways a pile transmits its load into the surrounding soil by use of electric strain gages and other forms of instrumentation. Most of these tests have been conducted on isolated single piles. This is a step in the right direction for establishing a rational analysis of the carrying capacity of friction piles; however, the tests must be extended to the critical case, which is load distribution within a pile group.

Purpose of Research.--This work attempts to further the development of a theory of friction pile behavior. A simulated, sensitive clay soil and model piles with strain gages mounted on the inside were utilized in accomplishing the objective. Measurements were made to determine the distribution of load to the pile heads and the transfer of load from the pile shafts to the soil. By varying the pile spacings within a group of nine piles, it was possible to show the effect of these load distributions on the bearing capacity of the group. It is hoped also that these tests will help to promote further investigation in this direction.

CHAPTER II

REVIEW OF THEORY AND LITERATURE

The effect of grouping on the bearing capacity of friction piles appears to be the major complication in the use of pile foundations. Both experience and tests have shown that the load carrying capacity of a group is less than the number of piles in the group multiplied by the load capacity of a single isolated pile. This reduction seems to depend on the size and shape of the group, and the length and spacing of the piles. Experience has also indicated that the spacing of friction piles has by far the greatest effect on group load capacity. The theory and literature that surrounds the subject is quite limited. However, a brief review of it follows.

The load carrying capacity of a pile can be attributed partly to skin friction along the pile's sides, and partly to the resistance of the soil directly beneath the point. The equation representing the ultimate bearing capacity may then be written thus:

$$P_t = P_f + P_p$$

where P_t = total bearing capacity

P_f = portion due to skin friction

P_p = portion due to point resistance

In the case of soft clays, the point resistance is usually negligible compared to the skin friction.

Friction pile groups may fail in two distinctly different manners. A group may fail as a unit by breaking into the ground, or it may fail by individual pile action when the load per pile exceeds its safe carrying capacity. When the piles and the confined mass of soil sink as a unit, the ultimate bearing capacity of the group may be considered analogous to that of a deep foundation such as a pier or caisson. This capacity consists of the shearing resistance of the soil at the periphery of the group between the surface and the embedded depth, and the bearing capacity of the base of the group. The load capacity of a group that fails by individual pile action is dependent only upon the development of skin friction along the sides of the piles. It seems logical that at certain spacings the load distribution action within a group will overlap, resulting in reduced carrying capacity.

According to Solomon (3) the stress around a pile in a homogeneous soil increases quickly from zero at the surface to a fairly constant value, until sufficient resistance has occurred in an upper section of the pile to account for the applied load. With successive increments of loading, this process continues doen the pile shaft until load reaches the point.

Some attempts have been made to utilize the classic Boussinesq equations in predicting the bearing capacity of friction pile groups. Masters (4) proposed a method for predicting group failure loads utilizing this approach. Its application checked closely with the results of tests he conducted. The method consisted of computing the average shear values on the soil around each pile, and then calculating their effects on each of the other piles by integrating Boussinesq's point load formulas. He assumed that all piles within a group received equal amounts of load from the pile cap.

Many soil mechanics experts feel that any resemblance between the actual failure load of a group of friction piles and that predicted by an analysis based on the Boussinesq equations is accidental. The fundamental assumption of the Boussinesq equations is that the load is applied to a plane boundary of a semi-infinite, elastic solid, and therefore a line type load beneath the furface would not apply. Furthermore, the presence of neighboring piles within a group must interrupt load transfers from the piles to the soil. In addition, the driving of piles will certainly, to some extent, destroy elastic continuity of the soil.

Mindlin (5) solved the three-dimensional elasticity equations for the case of a concentrated force acting in the interior of a semi-infinite, elastic solid. The Mindlin equations for stresses and displacements may be extended, by integration, to the case of the line distribution of a force that exists along the surface of a friction pile as long as elasticity is assumed. However, Mindlin (6) states that the integral equation for determining the shear distribution along a pile is difficult to handle. It must also be kept in mind that the assumption of elasticity is debatable.

The application of efficiency equations or factors is a method often used for reducing the load carrying capacity of friction pile groups. This approach provides an efficiency figure in percentage that is multiplied by the load capacity of an isolated single pile. The result is supposedly the average load carrying capacity of a pile within a group. These formulas are completely empirical and apply only to specific experience.

One such approach of accounting for the grouping of piles is termed the Converse-Labarre Method (7), and is expressed in terms of efficiency as follows:

$$\text{Efficiency} = 1 - \phi \left[\frac{(n - 1)m + (m - 1)n}{9\ mn} \right]$$

where

m = number of rows

n = number of piles in a row

ϕ = d/s, in which ϕ is numerically equal to the angle whose tangent is d/s, expressed in degrees

s = spacing center to center of piles

d = pile diameter

This formula has appeared in the Uniform Building Code of the Pacific Coast Building Officials Conference and in the specifications of the American Association of State Highway Officials.

Another way of reducing the load carrying value for a pile within a group is known as the Seiler and Keeney Method (8). This method consists of adjusting the Converse-Labarre efficiency curves to fall within the range of the Master's values for smaller groups. The formula is written as:

$$\text{Efficiency} = \left[1 - \frac{1/s}{7(s^2 - 1)} \times \frac{m + n - 2}{m + n - 1} \right] + \frac{0.3}{m + n}$$

Notation is the same as that for the Converse-Labarre formula.

Mr. Feld (9) uses a rule of thumb method for reducing pile group capacity which is based on his experience. It consists of reducing the load value of each pile in a group by one-sixteenth due to the effect of the nearest pile in each diagonal or straight row of which the pile in question is a member.

Most building codes specify a minimum spacing in the driving of piles. No explanation for these specifications is available; however, it appears that no theoretical considerations are behind them.

As mentioned previously, literature and recorded data concerning pile foundation behavior are very scarce. One of the most worthwhile contributions to the field has been made by Chellis (10). In his text, Pile Foundations, appears a complete compilation of today's knowledge of the theory, design and practice in use of piles. In addition to being a valuable source of information, the book also contains a comprehensive bibliography.

A paper by Morrison (11) discusses the several types of pile foundations and the fundamental principles regarding the interaction between piles and soil. Miller (12) compiled data concerning the behavior of pile foundations in actual use, then correlated pile foundation action with soil conditions and attempted to show the many fallacies in present design methods. A discussion by Conner, Woodruff, Feld, Greulich, Paxson, Mindlin, Solomon and Cummings (13) of Masters' paper on friction pile foundations is especially worthy of notice. These experts cover thoroughly the problems and discrepancies of existing design procedures, and they also indicate what must be done in the way of research to develop a rational approach in the design of friction piles.

Hansen and Kneas (14) reported results of tests in which they measured relative movements of various sections of pile shafts when the piles were under load. These tests were significant in that they were among the first in this country to attempt the measurement of load distribution from a pile to the soil. Swiger (15) gave the results of group load tests which indicated that a rigid pile cap does not evenly distribute load to the friction piles within a group. In view of this it should be noted that all of the methods for group design of friction piles previously discussed assumed that load distribution to the pile heads was equal.

The foregoing paragraphs present briefly the literature available to the designer of friction pile foundations. It can readily be seen that a great deal of work remains to be done in this field.

CHAPTER III

EQUIPMENT AND INSTRUMENTATION

The development of the SR-4 strain gage in recent years has made the measurement of load distribution within a group of friction piles more possible. However, not many tests of this sort have been performed, since economical reasons usually prevent full scale group tests, and model piles have proven difficult to instrument. The entire scope of this investigation depended on the successful installation of SR-4 strain gages inside of tubular, model piles, for the measurement of load distributions.

Three foot sections of extruded aluminum tubing, one and one-quarter inches in diameter, with a wall thickness of thirty-five thousandths of an inch, were used in making the model piles. The tubing was machined down to a diameter of one and twenty-one hundredths of an inch. This was necessary in order that strain readings, due to small individual pile loads, would be of such magnitude that a reasonable degree of accuracy would be obtained. Two holes were drilled opposite one another near the top of the piles for bringing out the lead wires from the gages.

It was found that with some simple equipment and considerable care it was possible to make satisfactory installations of these gages inside the aluminum tubes. The main item of equipment was a five foot aluminum rod, three-eighths of an inch in diameter. A two inch strip of cork was glued on its surface about fourteen inches from one end, and a strip of masking tape with the adhesive surface out was then glued to the cork.

The rod could easily be inserted through a model pile without damaging any of the gages already installed. The cork pad served as a base for applying pressure to the gage during the drying of the cement, and the masking tape held the gage in the correct position during initial stages of the installation. Wood blocking at each end of the rod kept the operation off the working surface. A small bag filled with sand and weighing about three pounds was used to apply pressure during the bonding of the cement.

The installation of the gages was as follows: First, the interior surface of the piles at gage locations was prepared in accordance with the manufacturer's instructions. Different colored lead wires for identification of the gages were then soldered to the gage leads. Short sections of Neoprene tubing were slipped over these connections to prevent their contacting each other or the metal of the pile. The gage was next placed in position with its felt pad adhering to the adhesive surface of the cork, and a liberal amount of glue was applied to the contact surface. The pile was then slid over the gage to the desired location, and weighted with the sand bag.

It was necessary to leave the pile in this position for twenty-four hours to assure proper bonding of the gages to the pile. At the end of this time a slight upward pressure on the pile would break the contact between the felt pad and the masking tape. This operation allowed the installation of only one gage per pile per day.

In this manner a total of eight A-5, SR-4 strain gages were installed in each instrumented pile. Gage locations were at each end and at approximately the one-third points of the pile. At each location two gages were installed opposite each other so that bending effects in the pile could be eliminated. The tests consisted of nine pile groups, but it was necessary to instrument only three piles due to the symmetry of the groups. In addition, two dummy gages were installed in another pile, one for each channel of the strain indicator, to compensate for temperature effects on the strain readings.

When all gages had been installed in a pile, the lead wires were brought out through the holes at the top of the pile. The area around these holes was covered with waterproof tape, and several coats of Petrosene wax were applied to prevent infiltration of moisture inside the tube. The tubes were finally sealed against moisture by forcing tapered, rubber stoppers in each end.

Figs. 1 and 2 show a typical set-up for installing the strain gages, and an interior view of a typically instrumented pile.

A volt-ohm micro-ammeter was used to test for correct gage resistance, and for resistance to leakage from the gage to the metal underneath. These gages had one hundred-twenty ohms resistance each. In addition, they registered infinite resistance to leakage at all times during the test program.

Strain readings were made with a Hathaway Type RS-20 portable strain indicator. Two separate channels were available on this instrument, and readings were possible to one microinch per inch. In conjunction with the indicator, two Baldwin SR-4 switching and balancing units, a six unit and a twenty unit, were utilized in order that readings could be made on all twenty-four gages in the three piles by simply switching from one to another.

Leads from the strain gages were connected to the active side of the switching and balancing units. Each of the latter was connected to one of the channels on the strain indicator. One dummy gage served each switching and balancing unit since all compensating units were connected.

The soil material, in which the pile groups were tested, consisted of commercial, dry bentonite mixed with water to a water content of three hundred per cent. This provided a sensitive cohesive material that would jell to a reasonably stable condition in approximately five days after being disturbed. The proper mixture for this particular reconsolidation ability was determined by Martin (16) through a series of tests in an earlier study at the Georgia Institute of Technology. A portion of the soil in the present investigation was also furnished from the work of Martin. The mixing of the soil was done by hand, since no mechanical mixer on the Georgia Tech campus could perform this task satisfactorily.

The soil container was a piece of culvert pipe. Its size was three feet in diameter and four feet high. A steel plate welded to one end served as a bottom. A raised platform base was constructed for the container in order that it could be conveniently moved.

A steel loading frame, which extended above the container, was attached to the base, and a lever system was used to load the pile groups. The lever was fashioned from one-half inch aluminum flat stock. It had a level ratio of four to one. Lead weights were used to apply load to the lever. An illustration of the container and loading apparatus appears in Fig. 3.

The pile cap was cast of pottery plaster. It consisted of ten parts plaster to seven parts water and was reinforced with a wire mesh.

A micrometer dial gage was used to measure the vertical deflection of the group. A miniature plate load test device and a miniature vane shear device were used to determine the physical properties of the soil, and to keep a check on its consistency during the test program.

CHAPTER IV

PROCEDURE

Calibration.--When the installation of the strain gages in a pile had been completed, it was necessary to calibrate each pair individually. The pile was placed in the testing machine and the gage leads were connected to the strain indicating equipment. Loads, fifty per cent in excess of the maximum estimated pile loads, were applied and then removed. This cycle was repeated approximately ten times to allow the physical properties of the metal to become constant. After this was completed, strain readings were made for each gage at twenty pound load increments until a maximum of one hundred-forty pounds was reached.

It was found that under identical loading conditions the gages would repeat themselves to within three microinches per inch. This indicated good installation of the gages. A plot of average strain readings for a gage location against known load gave the expected straight line variations for gages one and two, three and four, and five and six. It was noticed that if the pile were rotated in the testing machine about its vertical axis, individual gage readings might vary from those observed in the former position of the pile. However, the average of the opposite gages always agreed closely. This illustrates the importance of bending effects in the loaded pile.

Some difficulty was encountered in calibrating gages seven and eight. Reasons for this were the unfavorable end conditions and the fact that these gages were located very close to the end of the pile. This situation was corrected by using a resilient loading head on a ball joint against this end of the pile.

In order to increase the accuracy of all the calibration curves, many strain readings were taken with the pile rotated to various positions about its vertical axis. These readings were then averaged. The calibration curves for each pair of gages in each pile are shown by Figs. 7, 8, and 9.

At various times throughout the testing program the piles were again placed in the testing machine and the calibration of the gages was checked. In all instances the strain readings agreed closely with the original calibrations, indicating good stability characteristics.

Before loading the pile groups it was necessary to determine the force exerted on the pile head by the weight of the lever system. A set of scales was used to measure this force, and it was found to be twenty-two pounds. No attempt was made to counter weight the lever apparatus in the loading of the pile groups.

<u>Tests</u>.--Load tests were conducted on a single pile and variations of a nine pile group. The embedded length of the pile remained the same in all tests, and the only variable was the spacing of the piles within the groups. Pile spacing is defined as the distance in pile diameters from center to center of the piles. Spacings used in this investigation were one and one-half diameters, two diameters, two and one-half diameters, three and one-half diameters and five diameters.

These particular spacings were chosen in an attempt to determine all significant load distribution actions that might occur within the group.

The actual test procedure followed a general pattern. The piles were first placed in a template form to establish the correct spacing. The instrumented piles were placed adjacent to one another at corner, side and center positions. A form for pouring the cap was constructed around the pile heads, so that they would extend about an inch and one-half into the cap. The latter was then poured and allowed to set-up for about two hours. The pottery plaster provided a rigid cap that held the piles firmly at their correct spacings when the forms were removed.

Before placing the piles in the soil, a clear plastic lacquer was sprayed on their surfaces to prevent any corrosive action that might occur between the bentonite and the aluminum. The group was then forced into the soil to an embedded depth of twenty-five diameters, which was about thirty and one-half inches. The soil was allowed to reconsolidate for one week before the load tests were performed.

Before the load tests were begun, all gages were checked for resistance to leakage. They were then connected to the balancing and switching units, which in turn were connected to the strain indicator. The gage bridges were balanced to zero, and the system was ready for making strain measurements.

The loading apparatus was next placed in position, and a micrometer dial gage was attached to measure the vertical deflection of the group. Loads were first applied in even increments to the model foundations, but were decreased in magnitude as the failure load was approached. The lead weights which loaded the lever system were placed gently to avoid any dynamic load effects.

Strain readings were made on each gage, at each load increment, until the failure load was reached. Accuracy checks of these readings were made throughout the test by summing up the strain gage load indications at the one and two gage location of each pile. This figure was then compared to the actual load on the model foundation. Failure load was the point at which the group deflected constantly.

After the load test was completed, the piles were removed and wiped free of the soil material. The cap was easily broken loose from around the heads of the piles with chisel and hammer. Aluminum wool was then used to remove any corrosive effects of the bentonite, or rough spots on the pile surfaces. The piles were then again coated with the clear plastic lacquer, and were ready for the next test.

A miniature vane shear test and a miniature plate load test were next run on the undisturbed soil in the container to determine its physical properties and consistency. The soil was then removed from the container to a depth about six inches below the embedded depth of the piles. It was carefully remixed and replaced to avoid the formation of air pockets.

A poly-ethylene cover prevented moisture loss at all times except during the actual load testing. Fig. 3 shows an actual test in progress.

CHAPTER V

DISCUSSION OF RESULTS

General Comments

Bearing capacities of friction pile groups have previously been studied, at Georgia Tech and correlations have been made between these capacities and various design methods presently in use (17). Considerable variation between the actual and theoretical capacities have been found. It was not the primary intent of this study to make comparisons of this sort. The purpose of this work was to study the distribution of load in friction pile groups, and attempt to obtain an understanding of the manner in which the piles within groups carry their loads.

To accomplish this, the distribution of load was determined along a single pile, and within groups of nine piles. This discussion will consist of two parts: first, a report of the tests; and second, an interpretation of significant test results.

Results of Tests

Single Pile Load Test.--The first load test was conducted on a single pile. The purpose was to establish a guide or control for the pile action in the group tests that were to follow.

The failure load was seventy-nine pounds. Accuracy checks during the tests showed that the applied load agreed closely with the load registered by gages one and two, which were above the surface of the soil. When the pile was removed, it was noticed that no soil adhered to its surface. This indicates that failure did not occur by shearing of the soil adjacent to the pile's surface.

The load distribution curves of Fig. 10 show the action of the soil in removing load from the pile shaft. The invert of the slope of these curves, which is the difference of load that occurs within an increment of length along the pile shaft divided by that length increment, represents the adhesion that is actually developed along the pile surface. Adhesion will hereafter be referred to as friction. A plot of these values showing relative frictions between the top, middle, and bottom sections of the pile appear in Fig. 11. It is noticed from these illustrations that friction along the surface of the pile is not constant, but becomes greater with increasing depth. A sharp increase is observed in the developed friction of the bottom section just prior to failure. This indicates that failure would first commence at the tip of the pile.

Group Load Test Number One.--The pile spacing of one and one-half diameters in this test was selected to assure a unit type action. This action, which occurs when the piles and the soil act as a solid unit, was observed to occur when the group was forced into place, as considerable bulging of the soil was noticed. It was observed again when the group failed, since the piles and the soil within the group behaved as a single large pile. The periphery of the soil shear was along the approximate center line of the exterior piles. This, of course, left a portion of the outer piles' surfaces failing by adhesion of the soil to the pile. An illustration of this "unit type" action appears in Fig. 4.

The total load on the group at failure was three hundred sixty-four pounds. Fig. 14 shows the amount of load that is distributed to the individual piles. The corner piles received approximately sixty-eight per cent, the side pile received less than thirty per cent, and the center pile around three per cent. The load in the center pile appeared to increase slightly at group failure.

The load distribution curves of Fig. 15 show the transfer of load from the piles to the soil. It is noticed that the pattern of these curves vary considerably. The corner pile shows a slight drop-off in load just before the group failed. At this point the shape of the load distribution curve readjusted, and the friction values became more constant along the pile shaft. The dashed line in this illustration indicates that failure first started near the top of the corner pile.

Fig. 16 shows relative frictions between the piles in the group. Fig. 17 shows the variation in friction along the pile shafts. The first curves cited show that greatest values of friction within the group occur around the corner pile, and the least occur around the center pile. The friction along the surface of the side pile is greatest in the top section and lowest in the middle section. Along the center and corner pile, the greatest values of friction are in the bottom sections, and lowest values in the top sections.

Group Load Test Number Two.--This test was run at a pile spacing of two diameters. Bulging of the soil at the surface again occurred when the piles were placed; however, it was considerably less than in the previous test. A unit action occurred when the group failed, but it was not as pronounced as that of Group Number 1. Indications were that this particular group was in a transition zone of failure between unit action and individual pile action. The failure line of the soil was again along the center line of the outer piles.

The bearing capacity of the group was five hundred and eight pounds. The amount of load distributed to each of the piles is seen in Fig. 18. It is noticed that the side piles carried from forty to forty-eight per cent of the nine pile total. With increasing total load, the corner piles decreased from fifty-seven to forty-four per cent of the total, and the center pile increased from two per cent to eight per cent.

The transfer of load from the piles to the soil appears in Fig. 19. A small drop-off in load appeared in the side pile at the level of gages three and four as failure load approached. This condition remained local and did not extend up or down the pile. The corner pile first began to indicate failure at its tip. This action worked its way up the pile shaft. At the same time, the friction along the pile approached a constant. This illustration also shows a definite readjustment of load distributed to the heads of the corner and center piles. In latter increments of group loading, the corner pile reached a maximum constant load, and at the same time the center pile began to pick up load in much larger increments.

Figs. 20 and 21 illustrate relative frictions. These values are greatest in the corner piles, and smallest in the center pile. Along the individual piles surfaces, the friction is greatest in the bottom sections. It is lowest in the top sections for the side and corner piles. The least value of friction along the center pile occurs in its middle section.

Group Load Test Number Three.--The spacing of the piles in this test was two and one-half diameters. The group failed by adhesion of the soil to the individual piles. The failure load consisted of five hundred forty-eight pounds.

From Fig. 22 it is observed that at the start of the test the center pile received only three per cent of the load distributed by the cap to the piles. At this stage the corner pile took sixty-five per cent of this load. However, as the loading continued, the corner pile decreased in its percentage, while the center pile increased. At failure the load in the center pile was almost three-quarters of that in the corner pile. The side piles received approximately one-third of the total group load during the entire test.

A study of the load distribution curves of Fig. 23 indicates that failure of the group first started at the tips of the piles. This illustration indicates that, as in the previous tests, a redistribution of load took place within the group as failure load approached.

The relative friction curves of Fig. 24 show the corner piles to generally have the largest friction values. Smallest frictions are seen to be in the center pile. Relative frictions along the pile shafts may be observed in Fig. 25. These values are largest in the bottom sections in all three piles. The smallest value of friction along the center pile is in its top sections. The other two piles record their smallest friction values in their middle section.

Group Load Test Number Four.--Piles were spaced at three and one-half diameters for this test. As in group number three, failure occurred by individual pile action.

The total load at failure was five hundred seventy-five pounds. The loading of this group was done in even eighty pound increments; therefore, no load distribution was measured at loads close to failure. However, the maximum load on the group was considered to be the full amount necessary to cause failure. This became apparent when it was almost necessary to apply an additional load to cause the group to fail.

The loads delivered to the heads of the piles are illustrated in Fig. 26. The corner piles carried over fifty per cent of the total, and the center pile carried approximately six per cent. The side piles took about forty per cent of the total load. These percentages remained fairly constant during the entire test.

The load distribution curves which appear in Fig. 27, have the same general shape as those of the single pile. Also, the relative frictions along the pile shafts varied as they did for the single pile. These frictions are shown by Fig. 29 to become greater as the depth increases. Fig. 28 compares the friction values between the piles. Largest values occur in the corner piles, and smallest values occur in the center pile.

Group Load Test Number Five.--The spacing of the piles in this test was increased to five diameters. Failure occurred again by individual pile action. The failure load consisted of six hundred pounds.

Fig. 30 shows the loads received by the side and corner piles during the test to be approximately equal. The center pile increased from six to nine per cent of the total load distributed to the piles. Just prior to failure, the loads in the pile heads appeared to be approaching equal amounts.

The load distribution curves of Fig. 31 show failure commencing at the pile tips first. The curves then readjust, with successive load increments, until load distribution along the piles' surfaces becomes more uniform. The corner pile again recorded a constant load.

The friction values along the individual pile surfaces were greatest in their bottom sections. The least values occurred in the top sections. Fig. 33 shows these relative actions. The frictions between the piles in the group are related in Fig. 32. Generally these are greatest in the corner piles and least in the center pile as in all other group tests.

Soil Tests.--Tests were conducted on the physical properties of the soil after each load test had been completed on the model pile foundations. The main purpose in doing this was to determine if the soil properties remained constant during the entire test program.

The shear strength of the soil, as determined by the vane shear tests, remained almost constant at 0.8 of a pound per square inch. Results of these tests are listed in Table 8. They cannot be directly applied to the behavior of the piles, since the latter failed by adhesion of the soil to their surfaces, rather than by shearing of the soil adjacent to their surfaces.

The size of the miniature plate load test was one hundredth of a square foot. Eight pounds were required to produce failure in the first two tests, while only seven were required in the last four tests. The amount of deflection was also greater in the latter tests. This action was attributed to test procedure. The first two tests were conducted on a portion of the soil's surface that had not been disturbed for several weeks. The last four tests were run on soil that had been completely remolded about one week prior to each test. Results of the tests are listed in Table 7 and plotted in Fig. 36.

Interpretation of Results

The various illustrations, referred to in reporting the test results, were designed to describe the distributions of load that occurred. A set of curves for each load test show the load that is distributed to the heads of the instrumented piles. The load distribution curves show in pictorial style the transfer of loads from the pile shafts to the surrounding soil. Other sets of curves relate the frictions that are actually developed along the surfaces of the piles. The data upon which these illustrations are based may be found in the Tables in the Appendix.

The load tests are often referred to in this discussion as single pile, Group Number 1, Group Number 2, Group Number 3, Group Number 4, and Group Number 5. The group tests refer to pile spacings of one and one-half, two, two and one-half, three and one-half, and five diameters respectively.

The load tests were conducted in the following order: (1) Single pile, (2) Group Number 1, (3) Group Number 3, (4) Group Number 4, (5) Group Number 5, and (6) Group Number 2. The decision to run a test at a pile spacing of two diameters, which was last in the order, was made after observing a curve relating group capacities to pile spacing. It was seen that a large variation existed in the bearing capacities of Groups 1 and 3. It was desired to determine the pile action in this critical area.

The average pile efficiencies within the groups were determined and illustrated in Fig. 13. The results of the Single pile tests were the basis for these so called group efficiencies. It is noticed that the group efficiencies become greater as the spacings between the piles are increased. In these group tests they were fifty, seventy-six, seventy-nine, eighty-one, and eighty-three per cent respectively.

Group Load Tests Number 4 and 5 produced load distribution curves that closely resembled those of the single pile test. However, the average pile loads within the group did not reach the load capacity of the single pile test. The upward trend of the efficiency curve in Fig. 13 indicates that one hundred per cent efficiency may be reached at some greater pile spacing.

A factor in the test procedure may also have affected the group efficiencies. The soil was allowed to lie idle for approximately four weeks, and was not disturbed until the Single Pile Test was run. Thereafter, it was remixed after each load test (which occurred on the average of once a week). This additional time for consolidation may have allowed the single pile to develop greater friction values than those developed in the subsequent group tests. The vane shear tests indicated a slight decrease in the shearing strength of the soil during the test program. However, when the load capacity for the Single Pile Test was reduced accordingly, only a small increase was noticed in the group efficiencies.

A significant observation in these tests is that a rigid cap does not deliver load to the pile heads in equal amounts. The center pile received less load than the piles on the edge of the group. The corner pile generally received more load than the side piles except at group loads close to failure.

An explanation of why an exterior pile in the group received more load than an interior pile, can be seen in Fig. 5a. Consider a rigid spread footing resting on a clay soil. The settlement profile of an elastic type soil will assume a dish shape. The rigid footing, however, must settle uniformly. It follows, then, that load must be shifted from the center to the edges. Hence, contact pressure must be greater under the edges than in the center. This analysis may be compared to the pile group, since the piles rest in a clay soil and are connected by a rigid cap.

Another line of reasoning may also explain why greater loads were recorded in the corner piles. Part b of Fig. 5 shows a typical group of nine piles. The stresses in the soil around the piles would be symmetrical and may be represented by circles with the piles as their centers. It then becomes apparent that the corner pile has less interference in the development of friction along its surface.

The distribution of load from the cap to the piles led to another important observation of the pile groups' behavior. This was a plastic redistribution of load that appeared to occur within the soil when it became overstressed at any one point. This action was seen to happen several times during these tests, and will be described in the following paragraphs.

Maximum stresses were first reached in the soil around the corner piles. In the groups with the smaller spacings, this condition occurred when the group load had reached about two-thirds its maximum value. In groups four and five, maximum stresses around the corner piles occurred just prior to group failure.

It was expected beforehand that when the soil had reached its maximum stress around any one pile, failure of the group would take place immediately. This did not happen. The loads reached a maximum in the corner piles, and then became constant. When this condition developed, the center and corner piles began to pick up the entire increases in the group loads. This process continued until the loads in these piles approached that in the corner piles, at which time the group failed. Illustrations of this performance may be seen in both the load distribution curves, and in the relative friction curves.

The test results showed that Group Number 1, which had the smallest spacings, exhibited the least amount of plastic readjustment. Extensive interference between the load distribution curves appeared to be responsible for the pronounced unit action that occurred. Since this action resembled that of a large single pile or pier, the possibility of load redistribution in or around the group was very limited. Consequently, the bearing capacity of the group was very low.

The behavior of the piles in Groups 4 and 5 resembled that of the single pile. Overlapping action of adjacent piles appeared small. This allowed the pile loads to be more nearly equal. Plastic redistribution of load that occurred was uniform throughout group loading.

The adjacent piles of Groups 2 and 3 indicated definite interference with one another's ability to transfer their loads to the soil. The soil within these groups, however, showed a pronounced ability to redistribute its load when it became overstressed around the corner piles. Load distribution curves of Figs. 19 and 23 sharply define this action. It should be pointed out that the corner piles reached a maximum capacity in Groups 1, 2, and 3 at approximately the same total group loads. It follows then, that if the plastic action had not occurred, the bearing capacities of groups two and three would have been much lower than they were.

A second type of plastic action was also noticed in the tests. At certain times the soil along an individual pile shaft would become overstressed. This may have been due to the overlapping actions of adjacent piles or to local conditions in the soil. When this happened the load in the pile at this point would be shifted up or down the pile shaft to a location where the soil was not so highly stressed. Dashed lines in the load distribution curves indicate this action in several instances.

The curves of relative frictions show that these values are greatest in the bottom portions of the piles in almost all cases. The Single Pile, Group 4, and Group 5 load tests show that friction values increased at greater depths along the pile shafts. An explanation for this behavior may be seen in the Mindlin analysis for a point load in the interior of a semi-infinite, elastic mass. Fig. 6 illustrates this situation. Mindlin's equation for the vertical displacement at any point (x, y, z) in this mass is as follows:

$$W = \frac{P}{16\pi G(1-u)} \left[\frac{3-4u}{R_1} + \frac{8(1-u)^2 - (3-4u)}{R_2} + \frac{(z-c)^2}{{R_1}^3} + \frac{(3-4u)\ (z+c)^2 - 2cz}{{R_2}^3} + \frac{6cz(z+c)^2}{{R_2}^5} \right]$$

where

- P = point load
- G = modulus of rigidity
- u = Poisson's ratio
- c, z, R_1, and R_2 = distances shown in Fig. 6

Let the Mindlin equation be applied to determine the vertical deflection in the soil near the surface of the pile. Point loads are applied successively at gage locations along the vertical axis of the pile. When numerical values are substituted for the terms in the equation, it is found that for the same point load vertical displacement becomes less with increasing depth.

Since the pile is rigid compared to the soil, all points along it must settle uniformly. Therefore, it would be necessary for the load applied to the soil to become larger with increasing depth. The Mindlin analysis and the results of these tests verify this action.

The application of Mindlin's analysis is not strictly correct for this situation, since the pile develops a line type load and not point loads. However, the deflection trends with respect to increasing depth would be the same for either case.

The group settlements are compared by the load-settlement curves of Fig. 35. They appeared to be elastic until loads near failure. The point at which the curves departed from the elastic pattern can be correlated with the point at which plastic redistribution of load began. These results also indicate that as the length-width ratio of the pile groups decreased, the settlement of the groups decreased.

Accuracy checks were often made during the load tests. The loads registered by gages one and two were summed up to see if they agreed with the load actually being applied. This procedure always produced agreement within five per cent. Considerable care was also taken in other aspects of the test procedures.

Since the strain readings were small, it should be pointed out that chance for error does exist. A sudden surge of voltage in the power source would cause the strain indicator to vary its readings. A small error of this type would of course cause an error in the indicated load distributions. Local conditions in the soil, such as air pockets, could also cause variations in the test results.

For these reasons no rigorous mathematical analysis was attempted in describing the load distribution patterns. Much more data from tests of this type will be required before a more rigorous analysis could be justified.

CHAPTER VI

CONCLUSIONS

The following conclusions can be drawn from the results of these tests. It should be kept in mind that they apply only to the performance of a single pile and to nine pile groups in a sensitive clay soil.

1. The bearing capacity of a pile group becomes greater as the spacing of the piles is increased.
2. Critical pile spacing occurs between one and one-half diameters and two and one-half diameters. Group bearing capacities vary greatly between these spacings.
3. The rigid pile cap does not distribute load to the pile heads in equal amounts. The corner pile receives the most load and the center pile the least.
4. The pile groups transmit their loads to the soil in one of three ways: unit action, individual pile action, or a combination of these two.
5. The friction per unit area that is developed along a single pile, or along the piles within a group that demonstrates individual pile action, is not uniform. It becomes greater as the depth increases.
6. The frictions per unit area along the piles, in a group possessing a combination type action, may vary greatly. However, they

are always greatest at the tips of the piles.

7. A plastic redistribution of load occurred within the groups in two different manners. This plastic action occurred between pile capacities within a group, and in the friction developed along the surface of an individual pile.
8. Plastic redistribution of load was least in the group with the unit action. It was more noticeable in the groups with individual pile action, and was very pronounced in the groups with the combination actions.
9. Pile Groups 3, 4, and 5 appeared to fail when the individual pile loads approached equal amounts.
10. Failure first started at the tips of the piles and then worked its way up the pile shafts with successive load increments.
11. As pile spacing within a group was increased, its settlement, due to quick loading, decreased.

CHAPTER VII

RECOMMENDATIONS

In order to establish a rational design approach, it is recommended that studies of the load distribution within groups of friction piles be continued. Pile lengths should be considered as well as spacings. The studies should be extended to include groups with greater numbers of piles. Tests should also be conducted to compare the performance of pile groups of different shapes. The latter should consist of square, rectangular and circular groups.

If aluminum tubes are used for model piles in future tests, they should have their surfaces roughened uniformly. This should be done to assure that failure occurs by the shearing of the soil adjacent to the pile surface, rather than by adhension to its surface. Some type of paint may be available that would accomplish this, and would not deteriorate when in contact with the soil mixture.

It is suggested that a tool with expanding surfaces be developed to install strain gages in the interior of additional model piles.

It is recommended that strain indicators used in future model tests be of the automatic, null balancing type that is unaffected by line voltage variations and changing tube characteristics. A time-axis strain recorder for determining loads in the piles at their failure would also be helpful.

The installation of strain gages in full size piles that will actually become a part of a foundation should be encouraged. This would give designers valuable data of load distribution actions over a long period of time.

APPENDIX

Table 1

Single Pile Load Test

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
	1	23	45	30	24	4	23	19	11	2
	2	33	62	39	32	5	33	25	14	3
	3	43	81	52	42	7	43	33	19	4
2	4	53	100	65	51	8	53	41	23	4
	5	63	119	76	61	9	63	48	27	5
	6	73	138	88	71	9	73	56	32	5
	7	77			Failure					

Table 1a

Relative Friction (Adhesion) - Single Pile

Pile	Load Increment	Applied Load (pounds)		Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
	1	23		0.16	0.20	0.23
	2	33		0.25	0.28	0.29
	3	43		0.31	0.35	0.39
2	4	53		0.37	0.45	0.48
	5	63		0.46	0.48	0.56
	6	73		0.53	0.61	0.69
	7	77	Failure			

*Failure Load = Applied Load + 2 Pound Cap Weight

Table 2

Group No. 1 - 1.5 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
	1	40	6	2	1	0	3	1	1	0
	2	80	11	5	4	1	6	3	2	0
	3	120	15	9	7	3	8	5	4	1
	4	160	19	11	8	3	10	6	4	1
Side	5	200	26	14	12	5	14	8	6	2
(1)	6	240	32	19	15	7	17	11	8	2
	7	280	36	20	17	8	19	12	9	3
	8	300	43	24	21	10	23	14	11	4
	9	320	43	26	22	10	23	15	12	4
	10	340	48	30	25	12	26	18	13	4
	11	360				Failure				
	1	40	2	1	2	0	1	1	1	0
	2	80	4	2	3	0	2	2	2	0
	3	120	7	5	4	2	4	3	2	1
	4	160	9	6	7	2	5	4	3	1
Center	5	200	11	9	10	3	6	6	4	2
(2)	6	240	16	12	13	5	8	8	6	2
	7	280	21	15	15	5	11	10	7	3
	8	300	25	18	19	6	13	12	8	3
	9	320	27	20	20	7	14	13	9	4
	10	340	30	23	23	8	16	15	10	4
	11	360				Failure				

*Failure Load = Applied Load + 4 Pound Cap Weight

Table 2 (Continued)

Group No. 1 - 1.5 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
	1	40	15	7	4	0	6	3	2	0
	2	80	33	18	12	4	13	8	5	1
	3	120	49	28	18	4	20	13	7	1
	4	160	72	50	29	5	29	24	12	1
Corner	5	200	86	59	31	6	35	28	13	2
(3)	6	240	100	59	41	7	40	28	18	2
	7	280	117	81	43	9	47	39	19	3
	8	300	122	80	48	10	49	38	21	3
	9	320	135	98	54	11	54	47	23	4
	10	340	130	89	55	13	53	43	24	4
	11	360				Failure				

*Failure Load = Applied Load + 4 Pound Cap Weight

Table 2a

Relative Friction (Adhesion) - Group No. 1

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
Side (1)	1	40	0.06	0	0.03
	2	80	0.09	0.03	0.05
	3	120	0.09	0.03	0.08
	4	160	0.12	0.05	0.08
	5	200	0.19	0.05	0.10
	6	240	0.19	0.08	0.15
	7	280	0.22	0.08	0.15
	8	300	0.28	0.08	0.18
	9	320	0.25	0.08	0.20
	10	340	0.25	0.13	0.23
	11	360	Failure		
Center (2)	1	40	0	0	0.03
	2	80	0	0	0.05
	3	120	0.03	0.03	0.03
	4	160	0.03	0.03	0.05
	5	200	0	0.05	0.05
	6	240	0	0.05	0.10
	7	280	0.03	0.08	0.10
	8	300	0.03	0.10	0.13
	9	320	0.03	0.10	0.13
	10	340	0.03	0.13	0.15
	11	360	Failure		
Corner (3)	1	40	0.09	0.03	0.05
	2	80	0.16	0.08	0.10
	3	120	0.22	0.15	0.15
	4	160	0.16	0.30	0.28
	5	200	0.22	0.38	0.28
	6	240	0.37	0.25	0.40
	7	280	0.25	0.50	0.40
	8	300	0.34	0.43	0.45
	9	320	0.22	0.60	0.48
	10	340	0.31	0.48	0.50
	11	360	Failure		

Table 3

Group No. 2 - 2.0 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Level 1&2	3&4	5&6	7&8
	1	40	9	6	6	0	5	4	3	0
	2	80	15	9	9	1	8	6	4	0
	3	120	23	15	14	2	12	10	6	1
	4	160	30	20	18	3	16	12	8	1
	5	200	39	26	24	4	20	16	11	2
	6	240	48	31	29	5	25	20	13	3
	7	280	56	36	33	6	29	23	15	3
	8	320	62	43	38	6	33	27	17	3
Side	9	340	70	45	38	5	37	28	17	3
(2)	10	360	75	48	41	5	40	30	18	3
	11	380	81	53	44	7	43	34	20	4
	12	400	87	58	48	8	46	37	21	4
	13	420	94	57	48	6	50	36	21	3
	14	440	99	59	49	8	53	37	22	4
	15	460	105	57	50	9	56	36	22	4
	16	480	111	50	49	8	59	32	22	4
	17	500				Failure				
	1	40	3	0	0	0	1	0	0	0
	2	80	5	4	2	2	2	2	1	0
	3	120	9	6	4	2	4	3	2	1
	4	160	13	8	6	2	6	5	3	1
	5	200	17	10	8	4	8	6	4	1
	6	240	20	14	11	6	10	8	6	2
	7	280	23	16	12	7	12	9	6	2
	8	320	28	19	16	7	15	11	8	2

*Failure Load = Applied Load + 8 Pound Cap Weight

Table 3 (Continued)

Group No. 2 - 2.0 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Level 1&2	3&4	5&6	7&8
	9	340	30	19	16	8	16	11	8	3
	10	360	33	24	20	8	17	14	10	3
	11	380	36	25	20	8	19	15	10	3
Center	12	400	40	29	25	9	21	17	13	3
	13	420	43	32	26	9	23	19	14	3
(1)	14	440	49	35	31	9	26	21	17	3
	15	460	58	41	34	10	31	24	18	3
	16	480	70	48	37	9	37	28	20	3
	17	500				Failure				
	1	40	15	9	6	1	6	4	2	0
	2	80	29	19	12	2	11	9	5	0
	3	120	42	28	19	4	17	13	8	1
	4	160	56	38	27	5	22	18	11	2
	5	200	72	48	33	9	29	23	14	3
	6	240	84	59	40	9	34	28	17	3
	7	280	99	67	45	9	40	32	19	3
	8	320	113	77	51	10	46	37	22	3
	9	340	119	78	45	9	48	37	19	3
	10	360	123	80	54	11	50	38	23	3
	11	380	126	82	55	11	51	39	24	3
	12	400	129	84	57	10	52	40	25	3
	13	420	132	84	57	10	53	40	25	3
	14	440	134	88	56	9	54	42	24	3
	15	460	134	83	54	9	54	40	23	3
	16	480	134	86	58	10	54	41	25	3
	17	500				Failure				

*Failure Load = Applied Load + 8 Pound Cap Weight

Table 3a

Relative Friction (Adhesion) - Group No. 2

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq.in.) Top Section	Middle Section	Bottom Section
	1	40	0.03	0.03	0.08
	2	80	0.06	0.05	0.10
	3	120	0.06	0.10	0.13
	4	160	0.12	0.10	0.17
	5	200	0.12	0.13	0.23
	6	240	0.16	0.17	0.25
	7	280	0.19	0.20	0.29
	8	320	0.19	0.25	0.35
Side	9	340	0.28	0.28	0.35
(2)	10	360	0.30	0.29	0.38
	11	380	0.28	0.35	0.40
	12	400	0.28	0.40	0.43
	13	420	0.43	0.38	0.45
	14	440	0.49	0.38	0.45
	15	460	0.62	0.35	0.45
	16	480	0.84	0.25	0.45
	17	500	Failure		
	1	40	0.03	0	0
	2	80	0	0.03	0.03
	3	120	0.03	0.03	0.03
	4	160	0.03	0.05	0.05
	5	200	0.06	0.05	0.08
	6	240	0.06	0.05	0.10
	7	280	0.09	0.08	0.10
	8	320	0.12	0.08	0.15
Center	9	340	0.16	0.08	0.13
(1)	10	360	0.09	0.10	0.17
	11	380	0.12	0.13	0.17
	12	400	0.12	0.10	0.25
	13	420	0.12	0.13	0.28
	14	440	0.16	0.10	0.35
	15	460	0.22	0.15	0.38
	16	480	0.28	0.20	0.43
	17	500	Failure		

Table 3a (Continued)

Relative Friction (Adhesion) - Group No. 2

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
Corner (3)	1	40	0.06	0.05	0.05
	2	80	0.06	0.10	0.13
	3	120	0.12	0.13	0.17
	4	160	0.12	0.17	0.23
	5	200	0.19	0.23	0.28
	6	240	0.19	0.28	0.35
	7	280	0.25	0.33	0.40
	8	320	0.28	0.38	0.48
	9	340	0.34	0.45	0.40
	10	360	0.37	0.38	0.50
	11	380	0.37	0.38	0.53
	12	400	0.37	0.38	0.55
	13	420	0.40	0.38	0.55
	14	440	0.37	0.45	0.53
	15	460	0.43	0.43	0.50
	16	480	0.40	0.40	0.55
	17	500	Failure		

Table 4

Group No. 3 - 2.5 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
	1	80	12	6	6	0	6	3	3	0
	2	160	24	15	13	3	13	9	7	1
	3	240	38	23	19	4	20	14	10	1
	4	280	45	27	22	5	24	16	12	2
	5	320	53	33	26	7	28	19	14	2
Side	6	360	59	37	31	8	31	22	17	3
(1)	7	400	65	41	32	8	35	24	18	3
	8	440	76	47	37	9	42	28	20	3
	9	460	89	49	38	9	47	29	21	3
	10	480	93	52	40	9	50	31	22	3
	11	500	98	57	44	9	52	34	24	3
	12	520	105	65	44	8	56	39	24	3
	13	540				Failure				
	1	80	45	3	3	0	2	2	2	0
	2	160	12	8	8	1	6	5	4	0
	3	240	17	15	15	3	9	9	7	1
	4	280	21	15	16	3	11	10	7	1
	5	320	24	20	20	4	13	13	9	2
	6	360	32	23	24	5	17	15	11	3
	7	400	35	29	30	5	19	18	13	3
	8	440	45	38	38	6	24	24	17	3
	9	460	48	38	39	5	25	24	17	3
	10	480	60	49	48	7	32	31	21	3
	11	500	76	60	50	7	40	38	23	3
	12	520	87	69	45	6	46	44	20	3
	13	540				Failure				

*Failure Load = Applied Load + 8 Pound Cap Weight

Table 4 (Continued)

Group No. 3 - 2.5 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
	1	80	29	18	13	3	12	9	5	1
	2	160	59	38	27	7	24	18	11	2
	3	240	86	56	40	12	35	26	17	4
	4	280	93	57	46	14	38	27	20	5
	5	320	101	59	52	14	41	28	23	5
Corner	6	360	115	67	55	14	46	32	24	5
(3)	7	400	126	73	58	14	51	35	25	5
	8	440	151	88	63	15	61	42	27	5
	9	460	147	81	63	12	60	39	27	4
	10	480	155	97	62	13	63	46	27	4
	11	500	157	94	63	12	63	46	27	4
	12	520	156	92	59	12	63	44	25	4
	13	540				Failure				

*Failure Load = Applied Load + 8 Pound Cap Weight

Table 4a

Relative Friction (Adhesion) - Group No. 3

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
Side (1)	1	80	0.09	0	0.08
	2	160	0.12	0.05	0.15
	3	240	0.19	0.10	0.23
	4	280	0.25	0.10	0.25
	5	320	0.28	0.13	0.30
	6	360	0.28	0.13	0.35
	7	400	0.34	0.15	0.38
	8	440	0.43	0.20	0.43
	9	460	0.55	0.20	0.45
	10	480	0.59	0.23	0.48
	11	500	0.59	0.25	0.53
	12	520	0.53	0.38	0.55
	13	540	Failure		
Center (2)	1	80	0	0	0.05
	2	160	0.03	0.03	0.10
	3	240	0	0.05	0.15
	4	280	0.03	0.08	0.15
	5	320	0	0.10	0.17
	6	360	0.06	0.10	0.20
	7	400	0.03	0.13	0.25
	8	440	0.03	0.17	0.35
	9	460	0.03	0.17	0.35
	10	480	0.03	0.25	0.45
	11	500	0.06	0.38	0.50
	12	520	0.06	0.60	0.43
	13	540	Failure		
Corner (3)	1	80	0.09	0.10	0.10
	2	160	0.19	0.17	0.23
	3	240	0.28	0.23	0.33
	4	280	0.34	0.17	0.38
	5	320	0.40	0.13	0.45
	6	360	0.43	0.13	0.48
	7	400	0.50	0.25	0.50
	8	440	0.59	0.38	0.55
	9	460	0.65	0.29	0.58
	10	480	0.53	0.48	0.58
	11	500	0.53	0.48	0.58
	12	520	0.59	0.48	0.53
	13	540	Failure		

Table 5

Group No. 4 - 3.5 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
Side (2)	1	80	15	11	11	3	8	7	5	2
	2	160	30	21	21	4	16	13	9	2
	3	240	46	33	30	8	25	21	13	4
	4	320	62	45	41	9	33	29	18	5
	5	400	81	56	52	11	43	36	23	6
	6	480	101	72	63	12	54	46	28	6
	7	560	Failure							
Center (1)	1	80	9	6	5	3	6	4	3	1
	2	160	18	13	11	5	10	8	6	2
	3	240	29	20	18	7	15	12	10	2
	4	320	38	28	24	9	20	17	13	3
	5	400	48	36	30	13	25	21	16	5
	6	480	57	46	38	13	30	27	20	5
	7	560	Failure							
Corner (3)	1	80	30	20	15	5	12	9	6	2
	2	160	57	39	29	8	23	18	12	3
	3	240	85	57	40	11	34	27	17	4
	4	320	109	76	52	14	44	36	22	5
	5	400	131	90	65	19	53	43	28	6
	6	480	156	110	87	20	63	53	37	7
	7	560	Failure							

*Failure Load = Applied Load + 15 Pound Cap Weight

Table 5a

Relative Friction (Adhesion) - Group No. 4

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
	1	80	0.03	0.05	0.08
	2	160	0.09	0.10	0.17
Side (2)	3	240	0.12	0.20	0.23
	4	320	0.12	0.23	0.33
	5	400	0.22	0.25	0.43
	6	480	0.31	0.45	0.55
	7	560	Failure		
	1	80	0.06	0.03	0.05
	2	160	0.06	0.05	0.10
Center (1)	3	240	0.09	0.05	0.20
	4	320	0.09	0.10	0.25
	5	400	0.12	0.13	0.28
	6	480	0.09	0.17	0.37
	7	560	Failure		
	1	80	0.09	0.08	0.10
	2	160	0.16	0.15	0.23
Corner (3)	3	240	0.22	0.25	0.28
	4	320	0.25	0.35	0.43
	5	400	0.31	0.38	0.55
	6	480	0.31	0.40	0.75
	7	560	Failure		

Table 6

Group No. 5 - 5.0 Diameter Spacing

Pile	Load Increment	Applied* Load (pounds)	Average Gage Readings 1&2	3&4	5&6	7&8	Pile Load at Gage Levels 1&2	3&4	5&6	7&8
Side (2)	1	80	15	11	9	2	8	7	4	1
	2	160	33	23	19	3	18	15	9	1
	3	240	50	35	30	5	27	22	13	2
	4	320	70	46	41	7	37	29	18	4
	5	400	87	58	51	7	46	37	23	4
	6	480	102	66	53	9	55	42	24	5
	7	520	114	68	50	10	61	43	22	5
	8	560				Failure				
Center (1)	1	80	10	8	6	2	5	4	3	1
	2	160	20	14	11	3	10	8	6	1
	3	240	31	23	18	5	16	13	9	2
	4	320	42	30	25	7	22	18	13	2
	5	400	55	40	31	8	29	24	17	3
	6	480	70	51	37	8	37	30	20	3
	7	520	82	59	31	8	45	35	17	3
	8	560				Failure				
Corner (3)	1	80	23	16	9	2	9	7	3	1
	2	160	46	34	19	5	19	16	8	2
	3	240	69	52	32	6	28	25	14	2
	4	320	88	63	41	8	36	30	18	3
	5	400	108	78	49	9	44	37	21	3
	6	480	130	92	60	9	53	44	26	3
	7	520	132	90	49	8	54	43	21	3
	8	560				Failure				

*Failure Load = Applied Load + 40 Pound Cap Weight

Table 6a

Relative Friction (Adhesion) - Group No. 5

Pile	Load Increment	Applied Load (pounds)	Friction (Lbs. per sq. in.) Top Section	Middle Section	Bottom Section
	1	80	0.03	0.08	0.08
	2	160	0.09	0.15	0.20
	3	240	0.16	0.23	0.28
Side	4	320	0.25	0.28	0.35
(2)	5	400	0.28	0.35	0.48
	6	480	0.40	0.45	0.48
	7	520	0.56	0.53	0.43
	8	560	Failure		
	1	80	0.03	0.03	0.05
	2	160	0.06	0.05	0.13
	3	240	0.09	0.10	0.17
Center	4	320	0.12	0.13	0.28
(1)	5	400	0.16	0.17	0.35
	6	480	0.22	0.25	0.43
	7	520	0.31	0.45	0.35
	8	560	Failure		
	1	80	0.06	0.10	0.05
	2	160	0.09	0.20	0.15
	3	240	0.09	0.28	0.30
Corner	4	320	0.19	0.30	0.38
(3)	5	400	0.22	0.40	0.45
	6	480	0.28	0.45	0.58
	7	520	0.34	0.55	0.45
	8	560	Failure		

Table 7

Plate Load Test Results

Load (pounds)	Deflections per Test (inches) 1	2	3	4	5	6
1	0.003	0.003	0.006	0.0045	0.0045	0.004
2	0.006	0.007	0.012	0.009	0.009	0.008
3	0.0105	0.010	0.019	0.017	0.0135	0.014
4	0.015	0.012	0.027	0.025	0.021	0.021
5	0.020	0.018	0.036	0.034	0.028	0.030
6	0.025	0.030	0.046	0.042	0.039	0.051
7	0.0305	0.037	0.058	0.065	0.070	0.077
8	0.045	0.056				

Table 8

Vane Shear Test Results

Test No.	Critical Load (pounds)	$S = \frac{Pd}{\pi a^2(L + a/3)}$ *
1	2.91	0.82
2	2.91	0.82
3	2.91	0.82
4	2.86	0.805
5	2.86	0.805
6	2.86	0.805

* Shear strength in lbs/sq. in.

where P = critical load
d = Ø of torque disk = 2.0625"
a = diameter of vane = 1.0"
L = height of vane = 2.0"

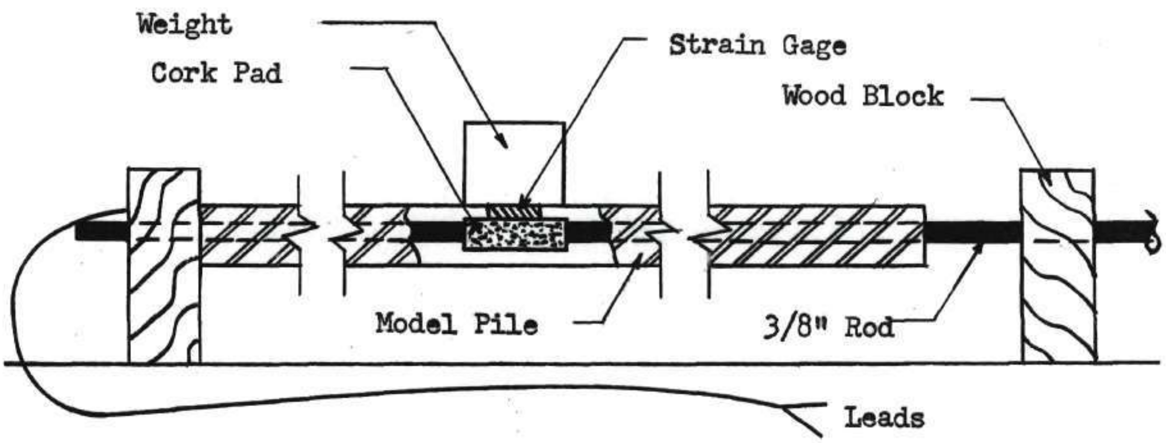


Fig. 1 Detail of Strain Gage Installation

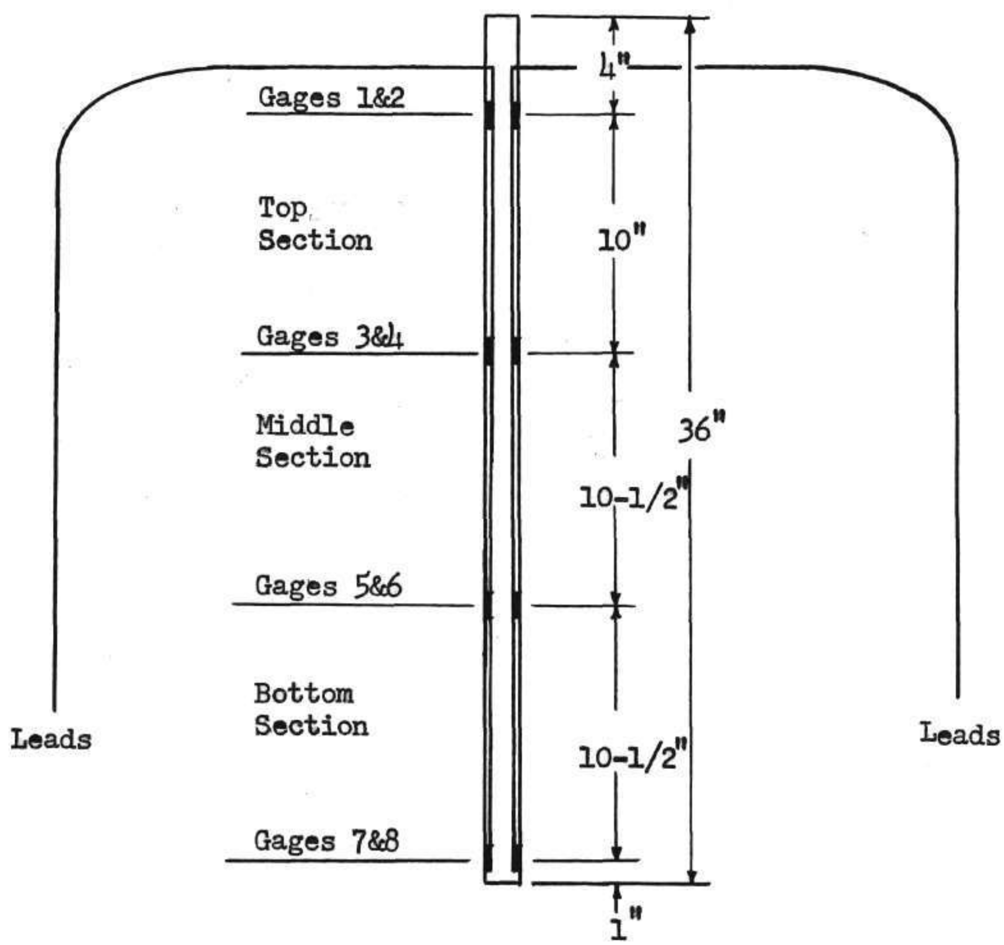


Fig. 2 Interior View of Gage Locations

Fig. 3 Test Apparatus

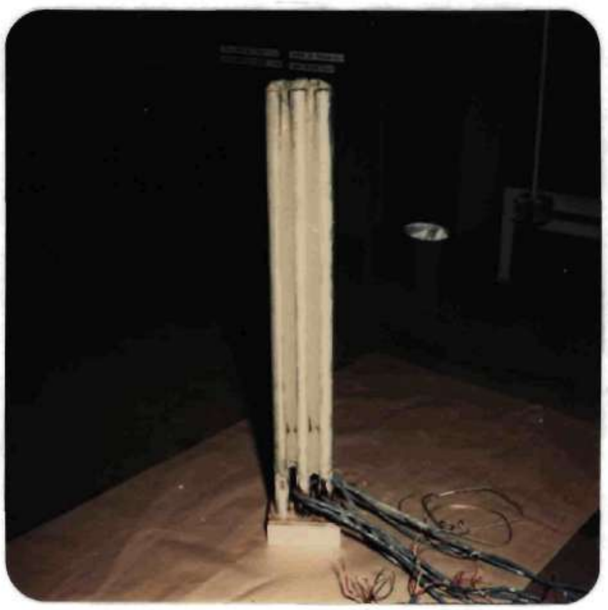
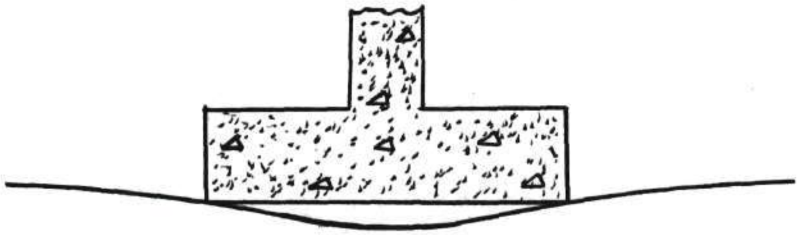
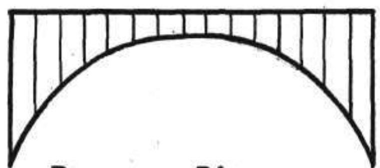


Fig. 4 Illustration of Unit Action



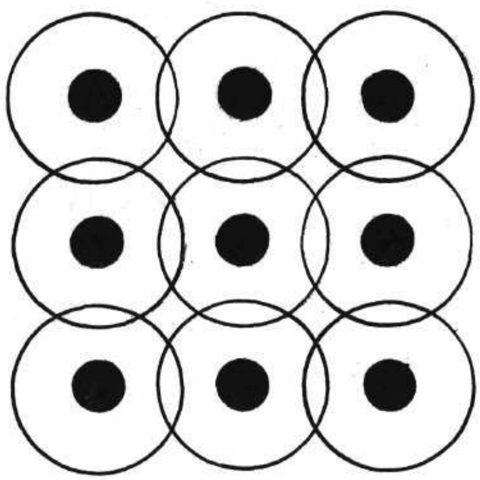
Soil Profile



Pressure Diagram

-a-

Rigid Footing on Clay Soil



-b-

Illustration of Stress Interference

Fig. 5 Analysis of Load Distribution to Pile Heads

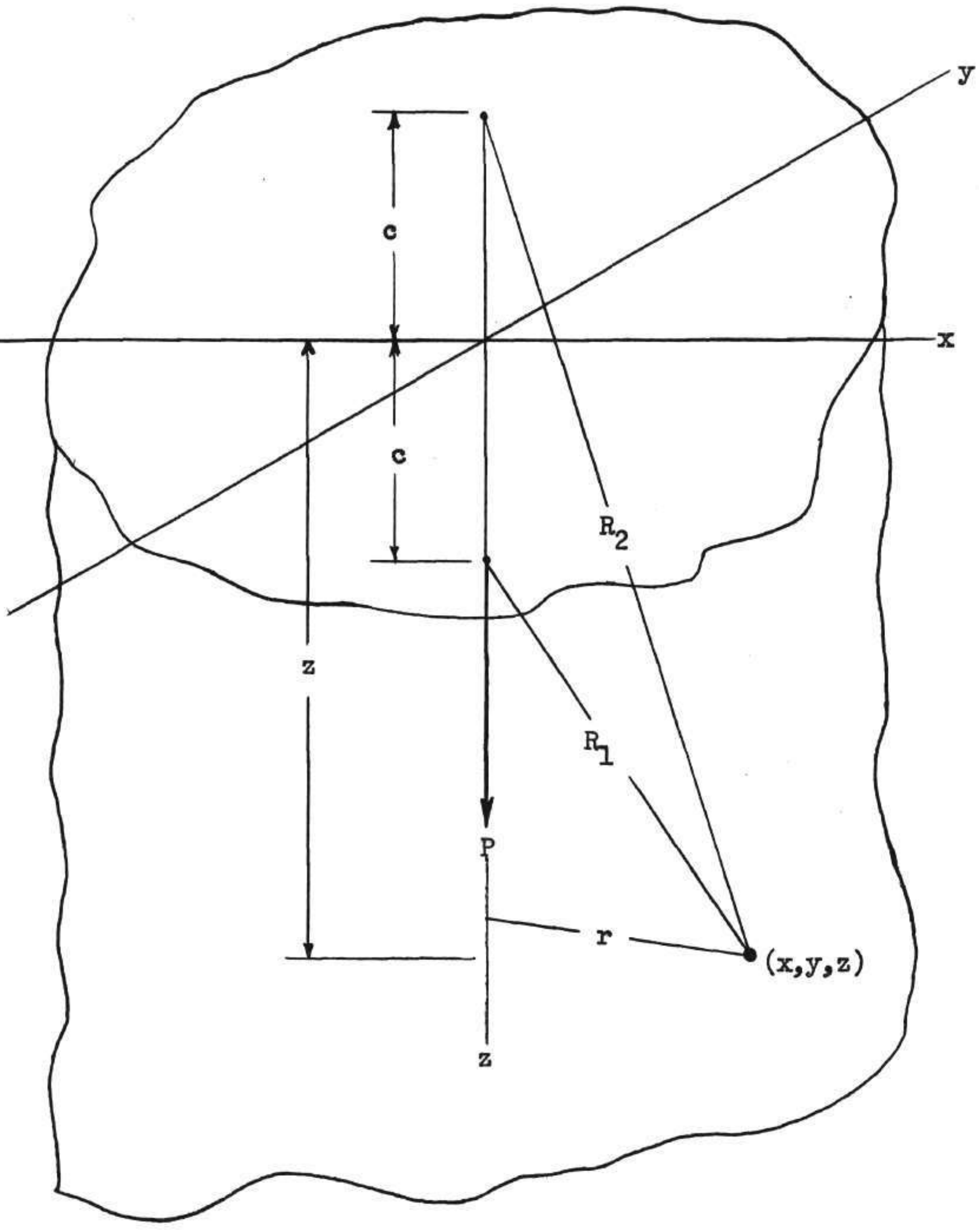


Fig. 6 Point Load in Semi-Infinite Elastic Body

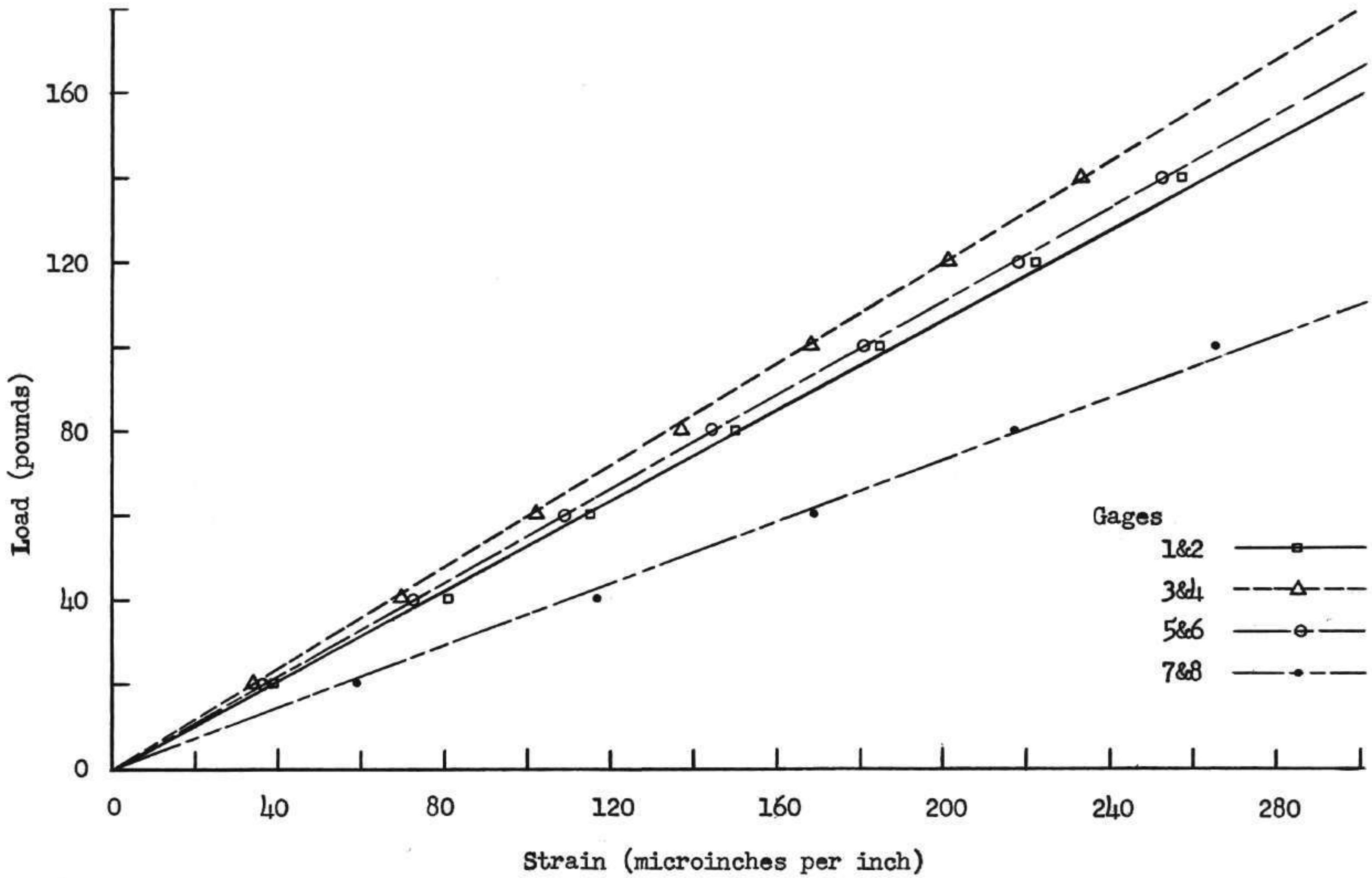


Fig. 7 Calibration - Pile No. 1

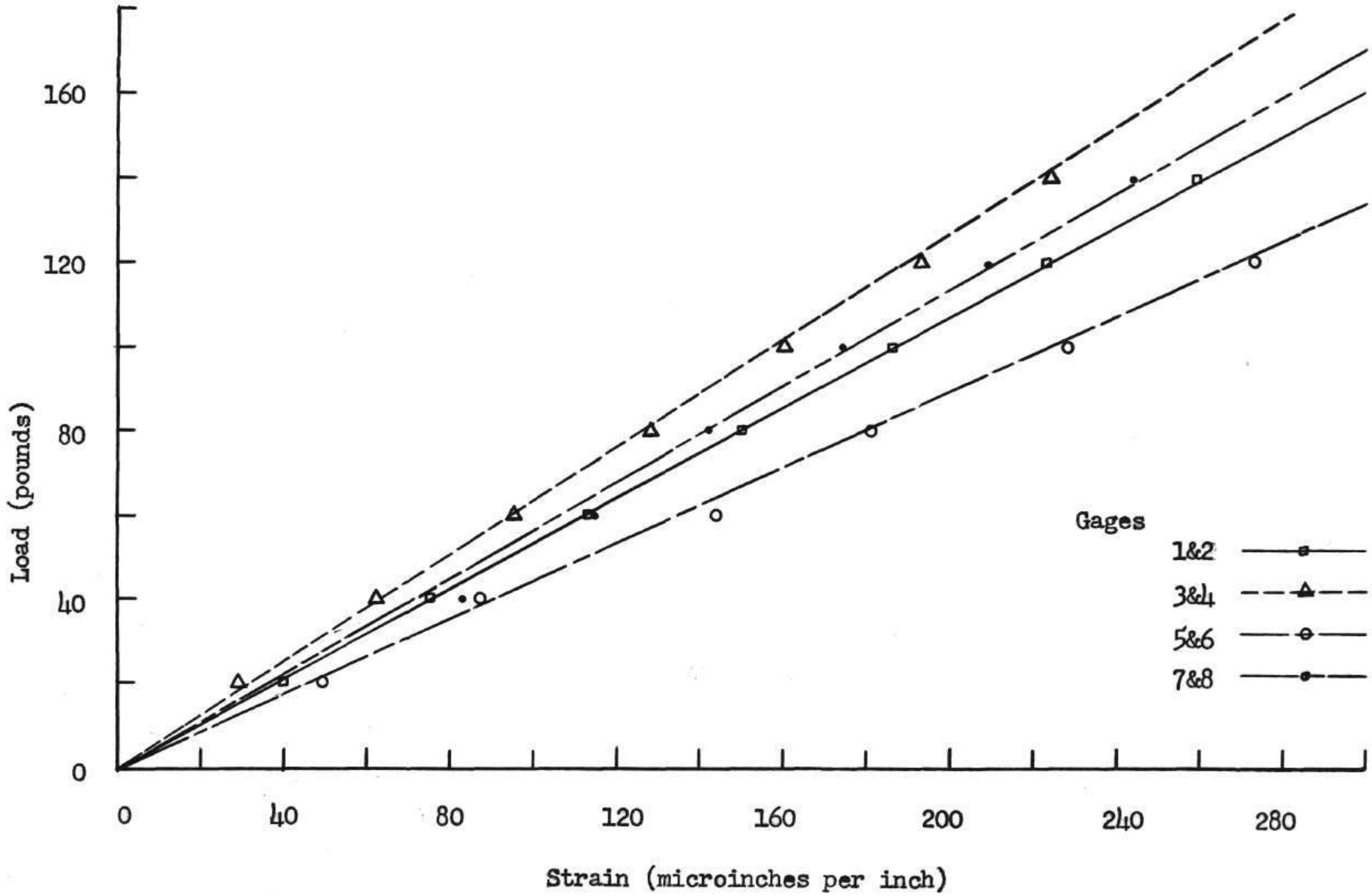


Fig. 8 Calibration - Pile No. 2

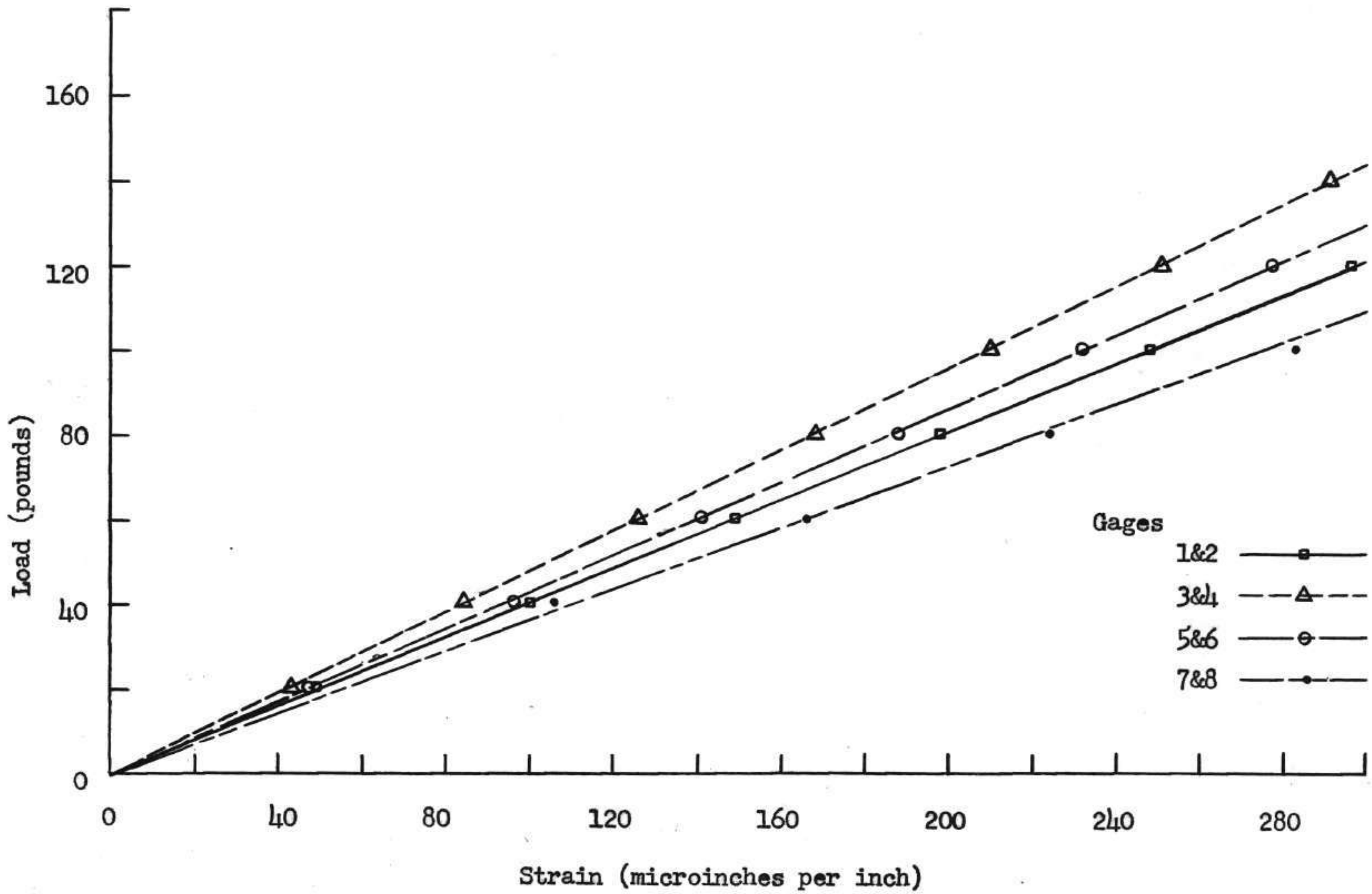


Fig. 9 Calibration - Pile No. 3

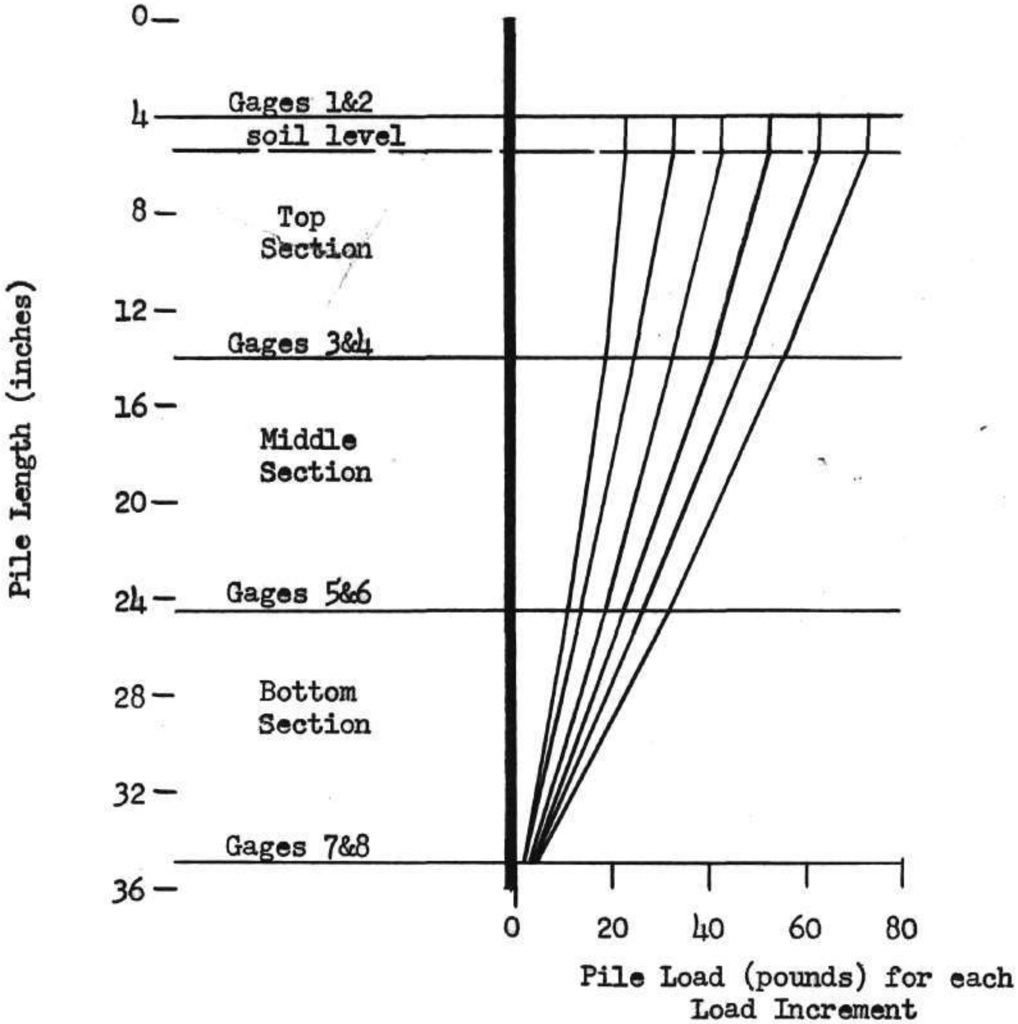


Fig. 10 Load Distribution Curves (single pile)

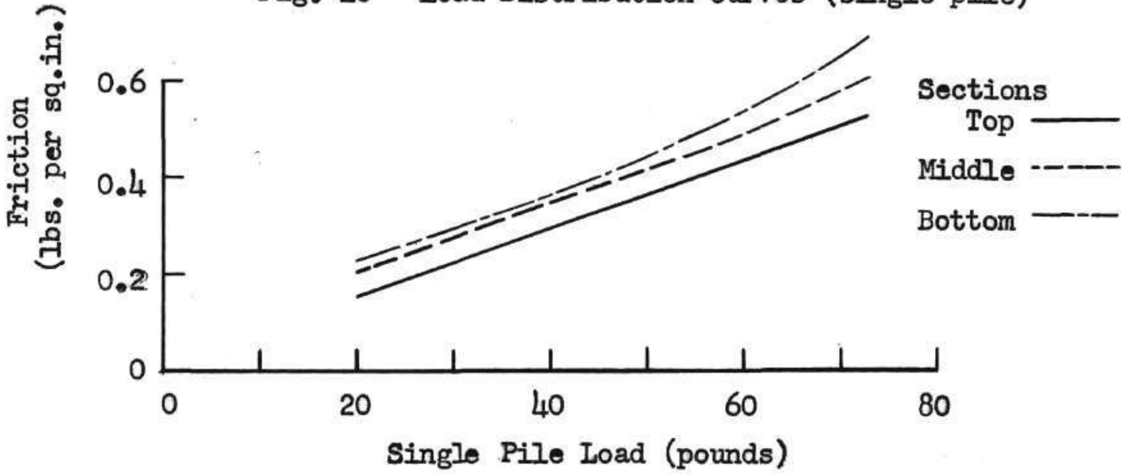


Fig. 11 Relative Frictions Between Sections (Single Pile)

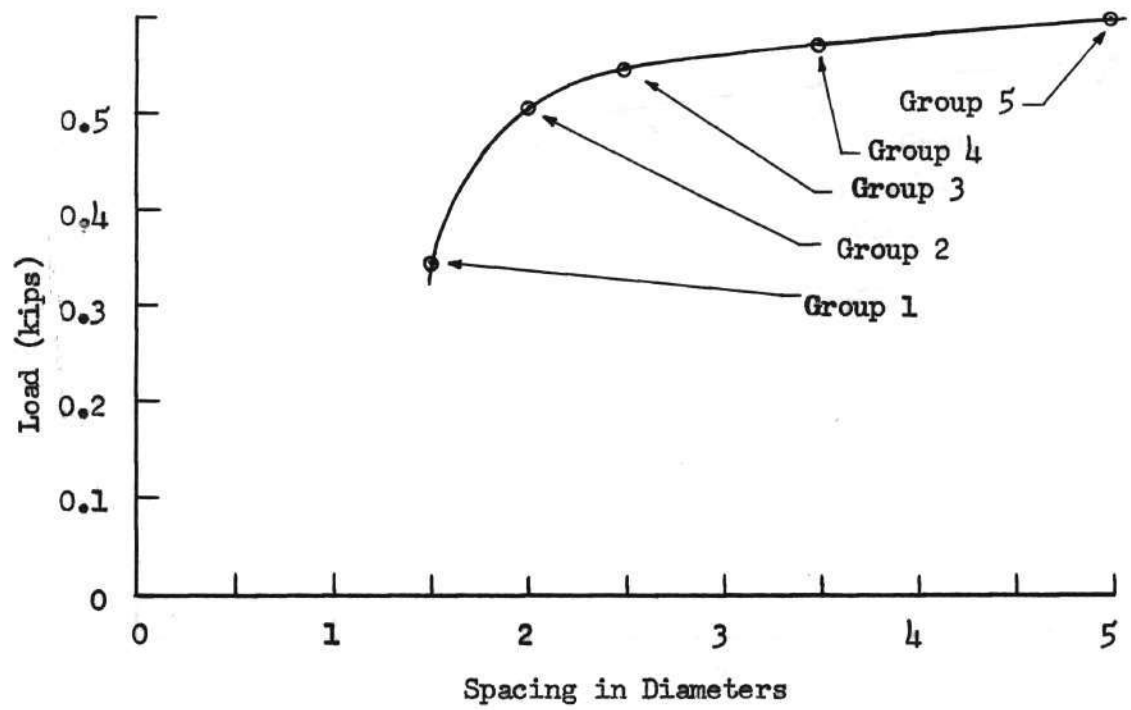


Fig. 12 Group Load vs Spacing

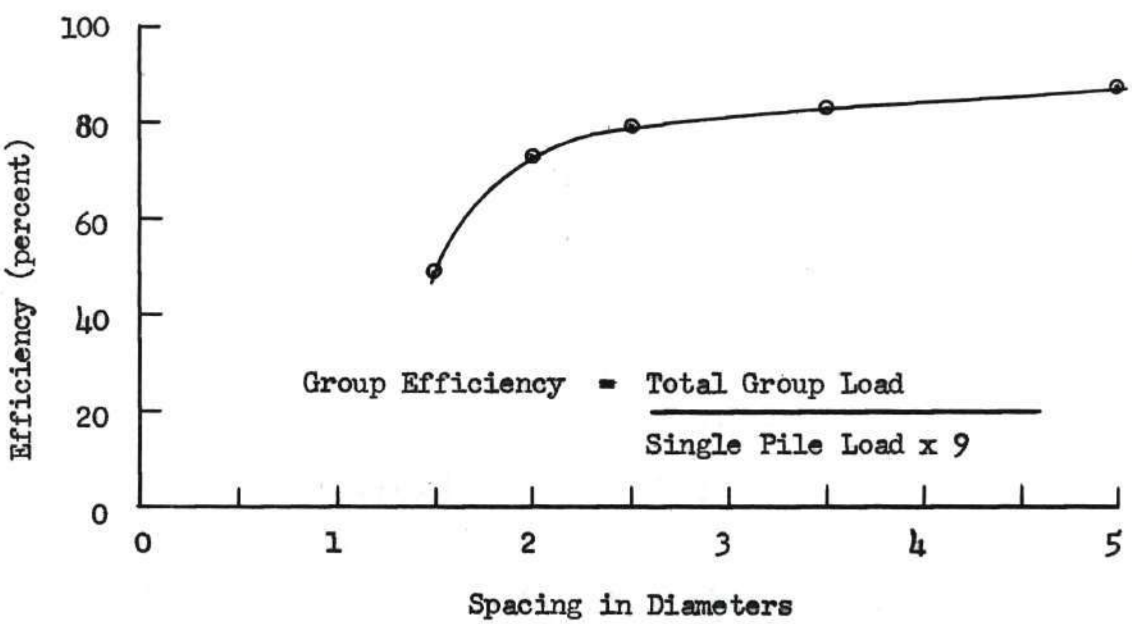


Fig. 13 Group Efficiency vs Spacing

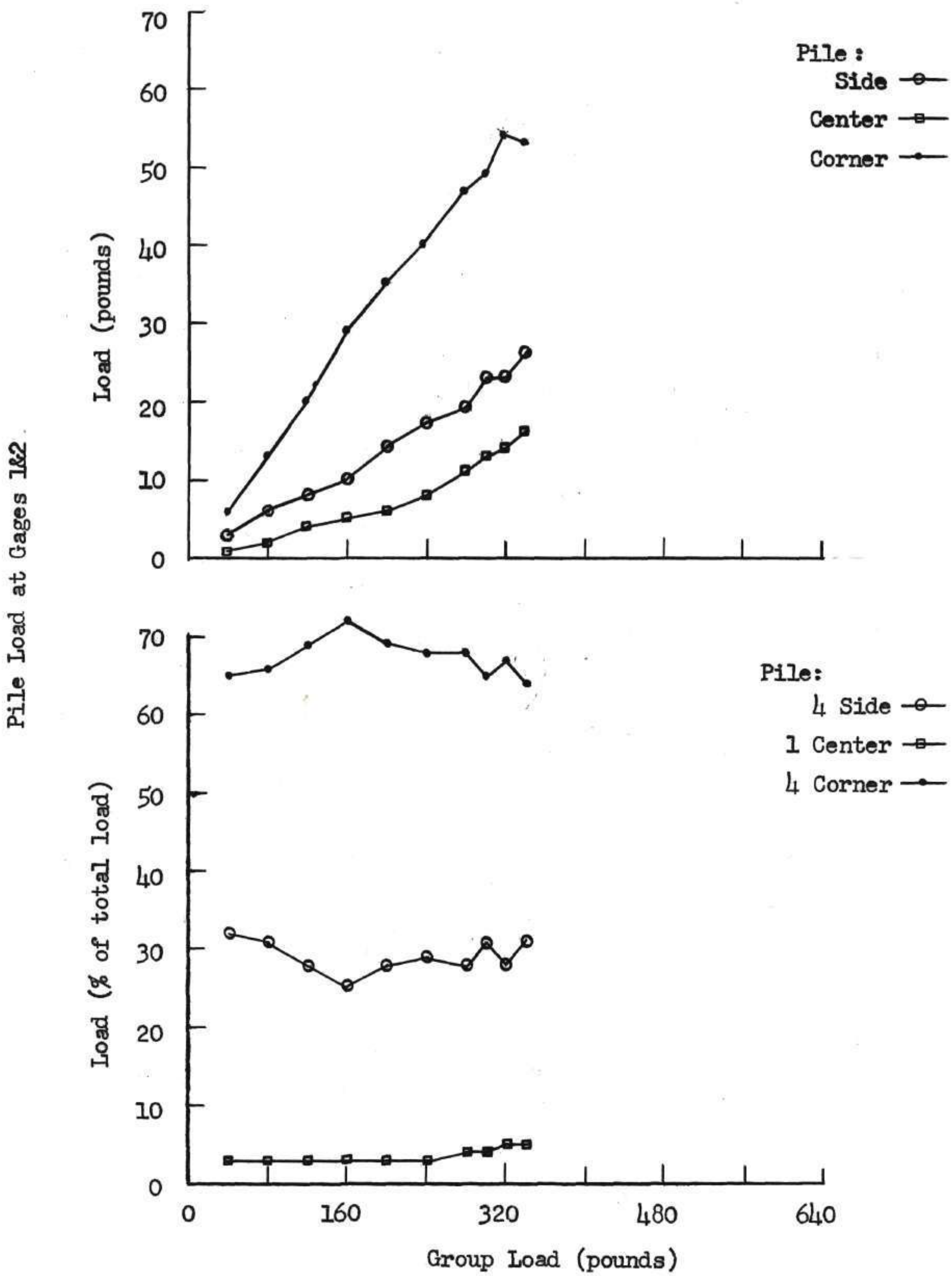
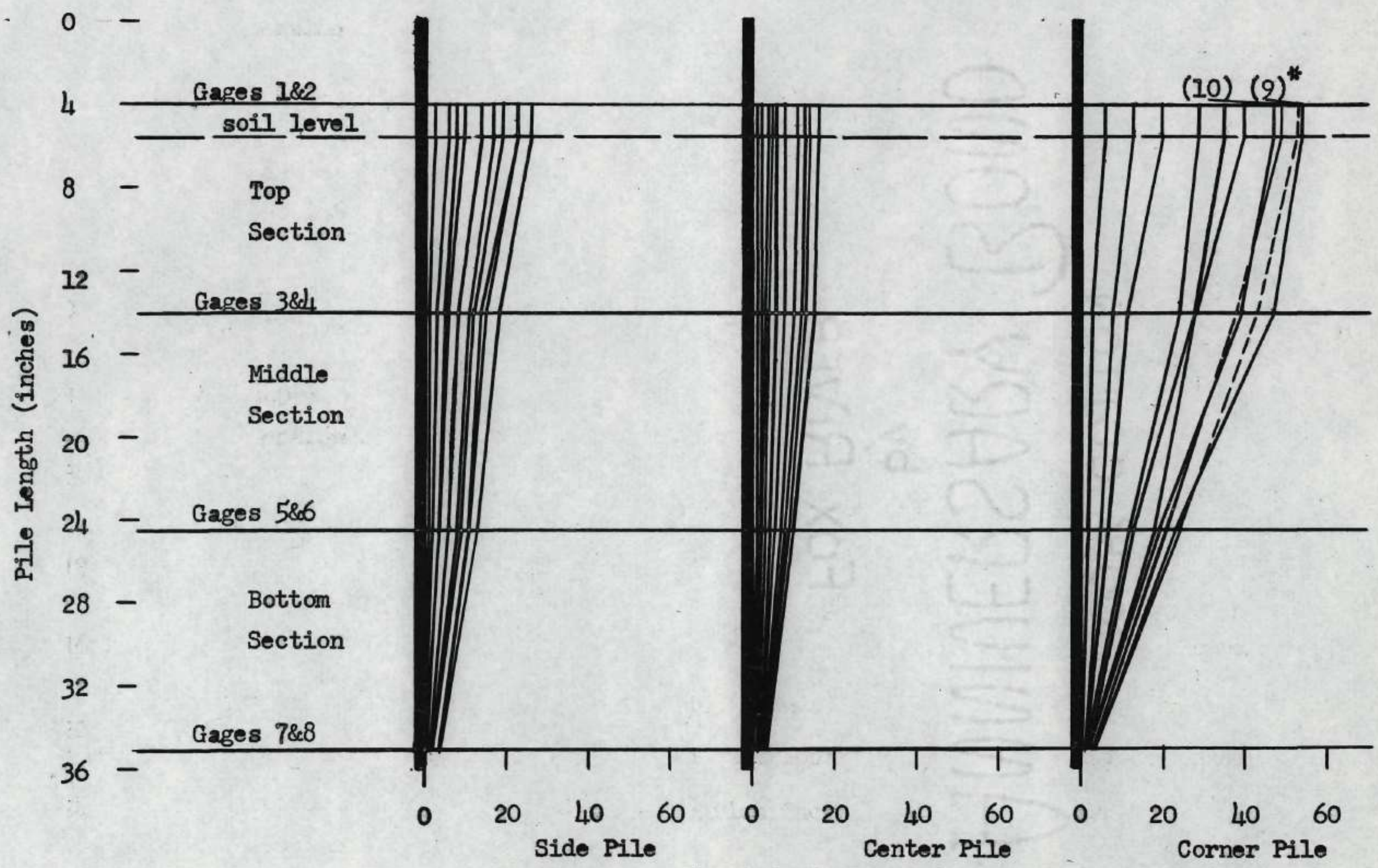


Fig. 14 Load Distributed From Cap to Piles (1.5 Diameter Spacing)



Load in Pile (pounds) for each Increment of Group Load

Fig. 15 Load Distribution Curves (1.5 Diameter Spacing)

*Clarifies increment of group loads

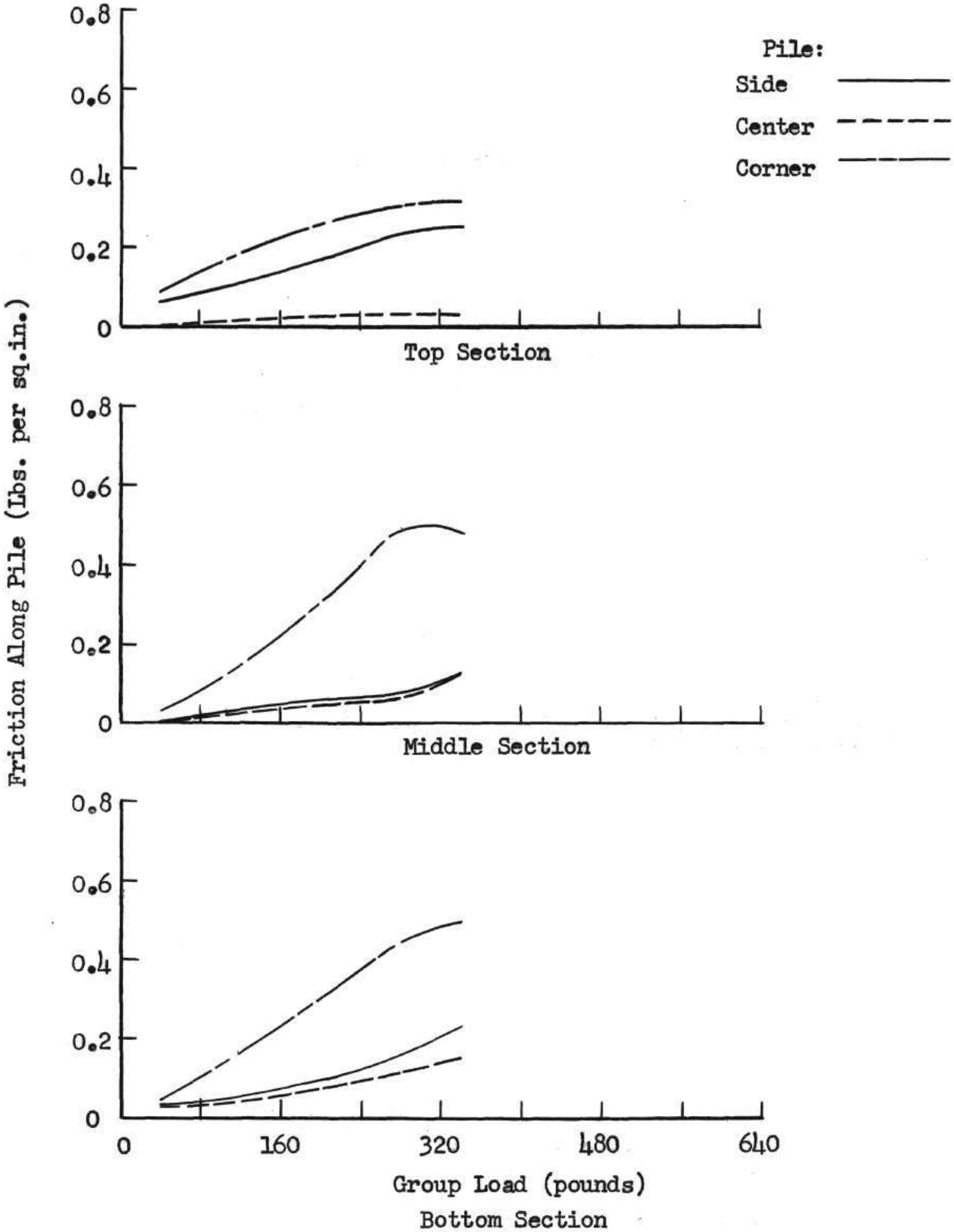


Fig. 16 Relative Frictions Between Piles (1.5 Diameter Spacing)

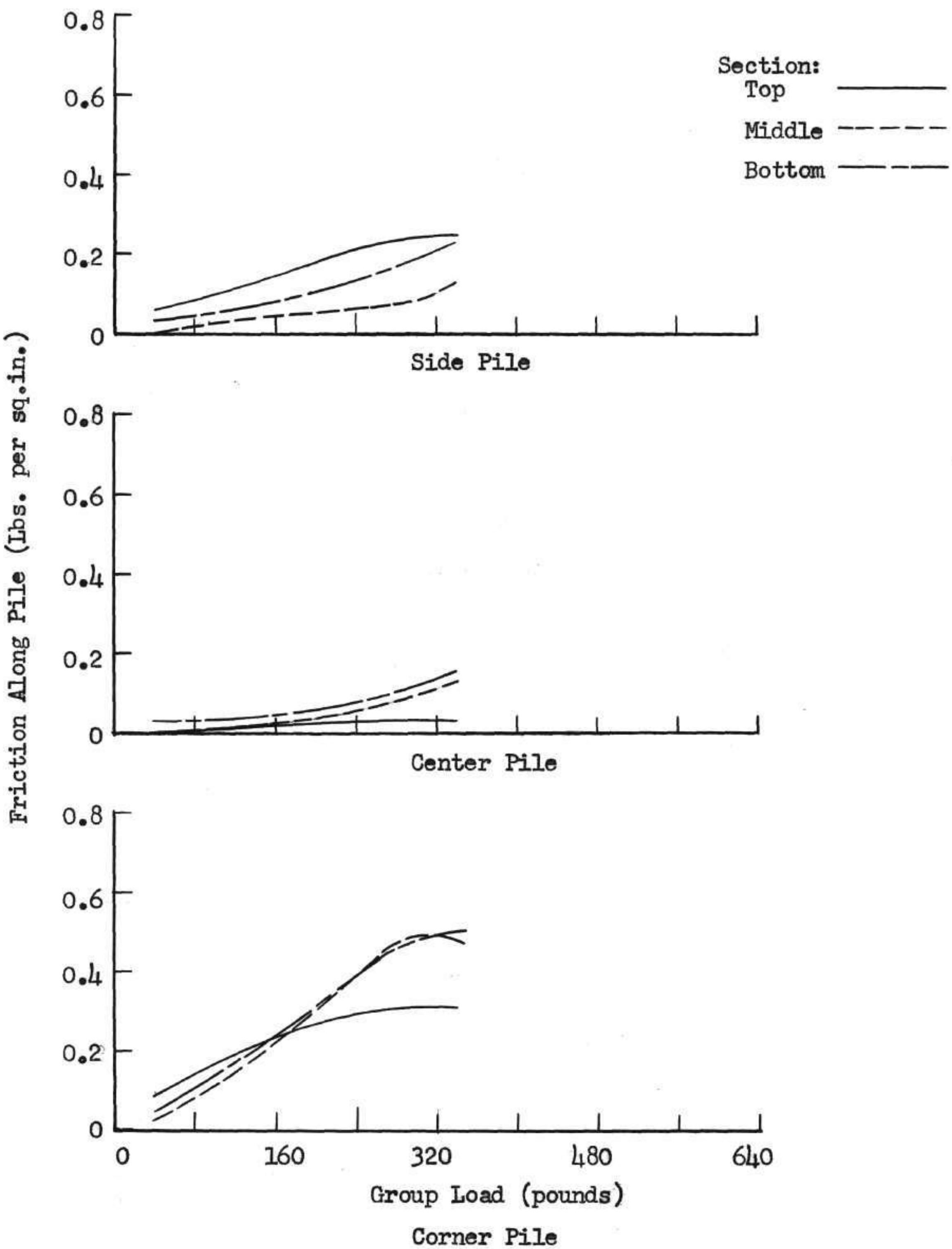


Fig. 17 Relative Frictions Between Sections (1.5 Diameter Spacing)

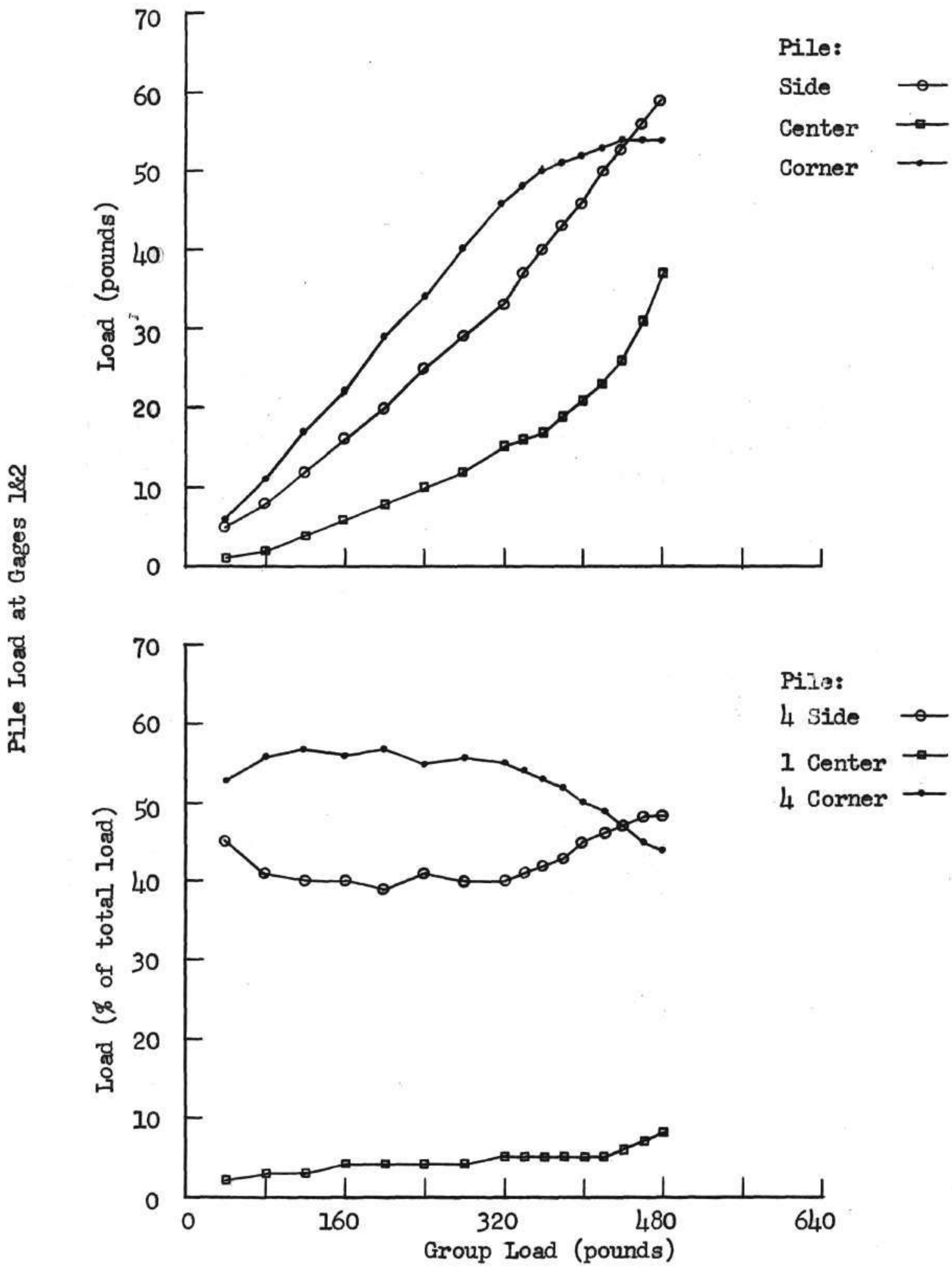


Fig. 18 Load Distributed From Cap to Piles (2.0 Diameter Spacing)

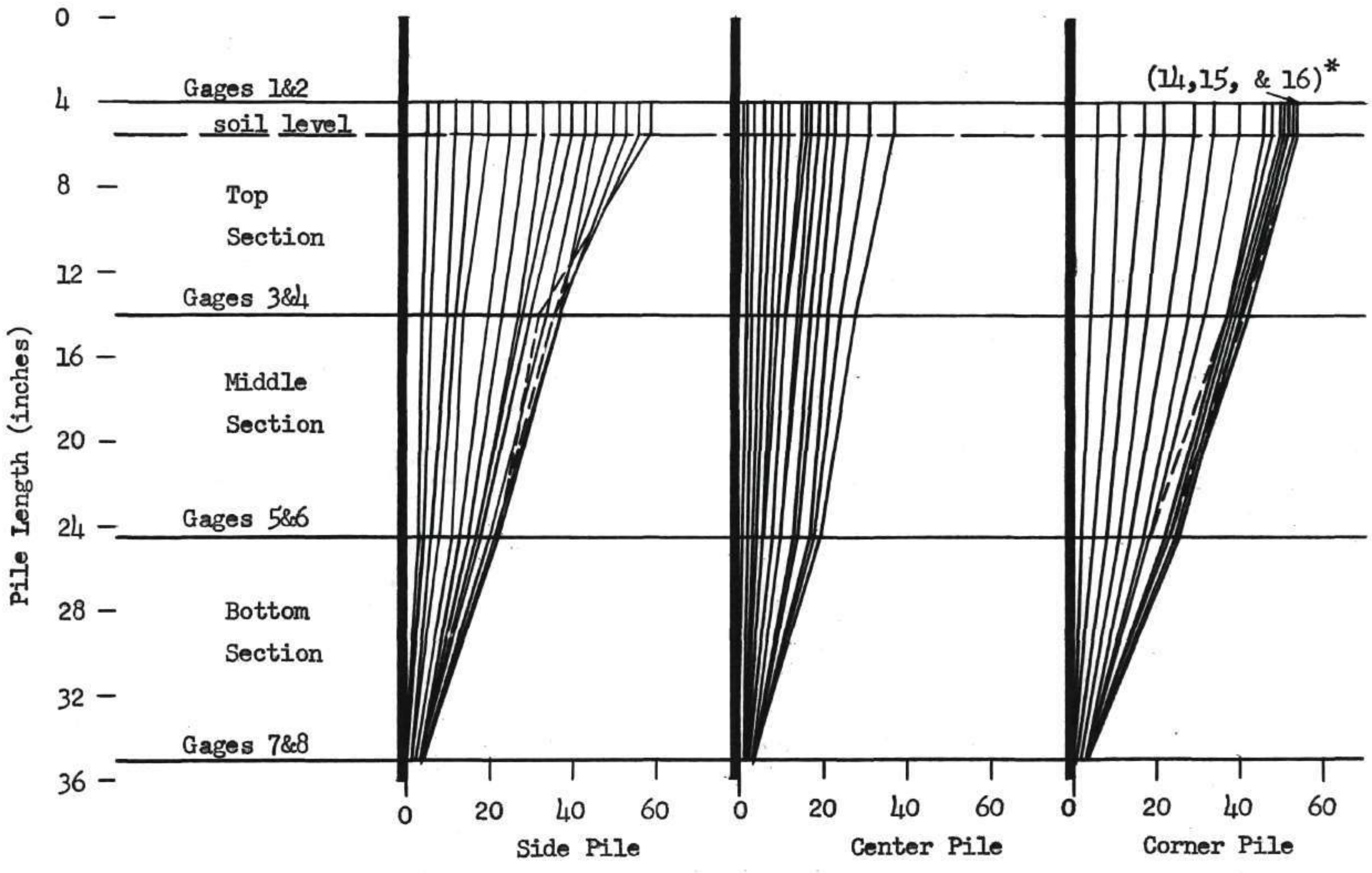


Fig. 19 Load Distribution Curves (2.0 Diameter Spacing)

* Clarifies increments of group loads

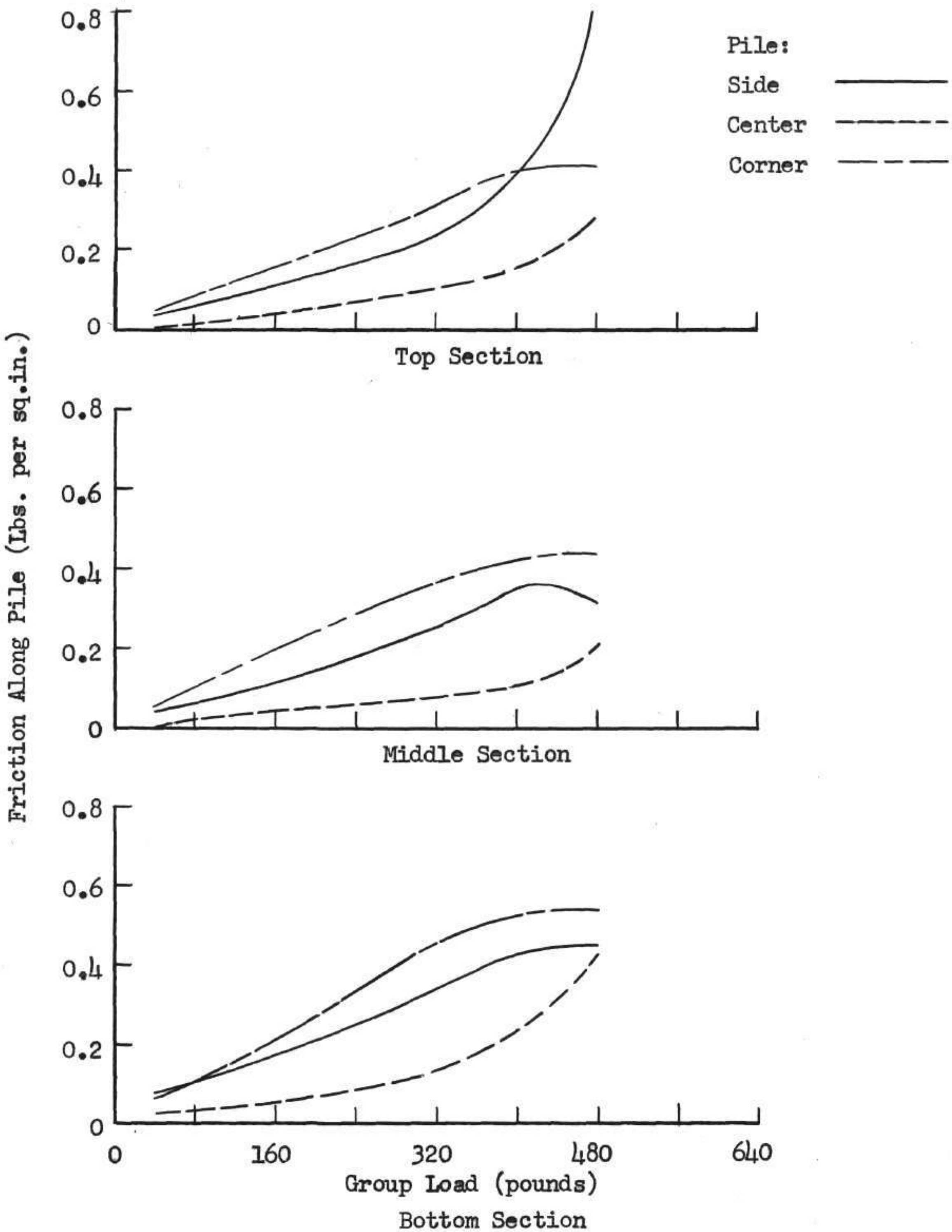


Fig. 20 Relative Frictions Between Piles (2.0 Diameter Spacing)

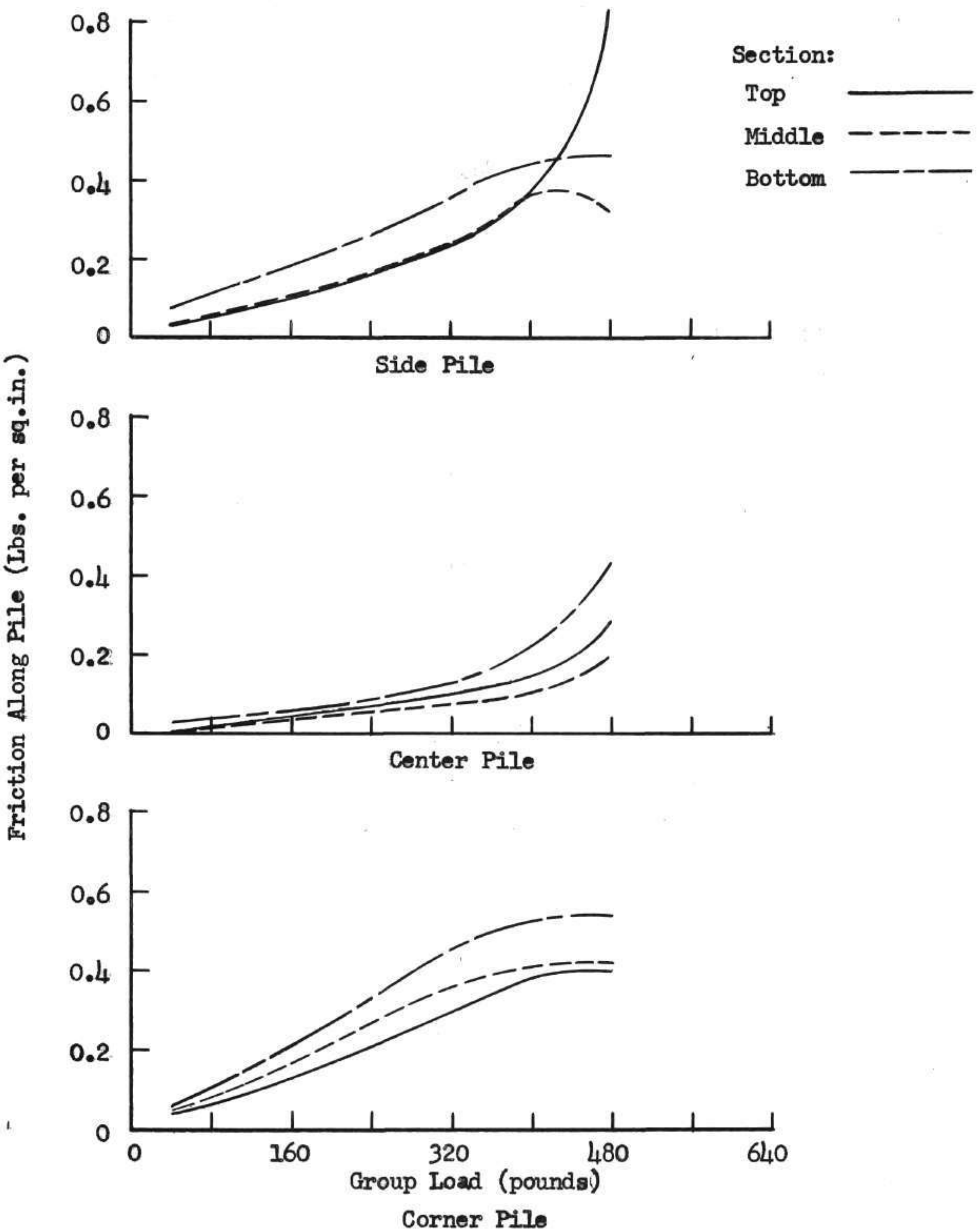


Fig. 21 Relative Frictions Between Sections (2.0 Diameter Spacing)

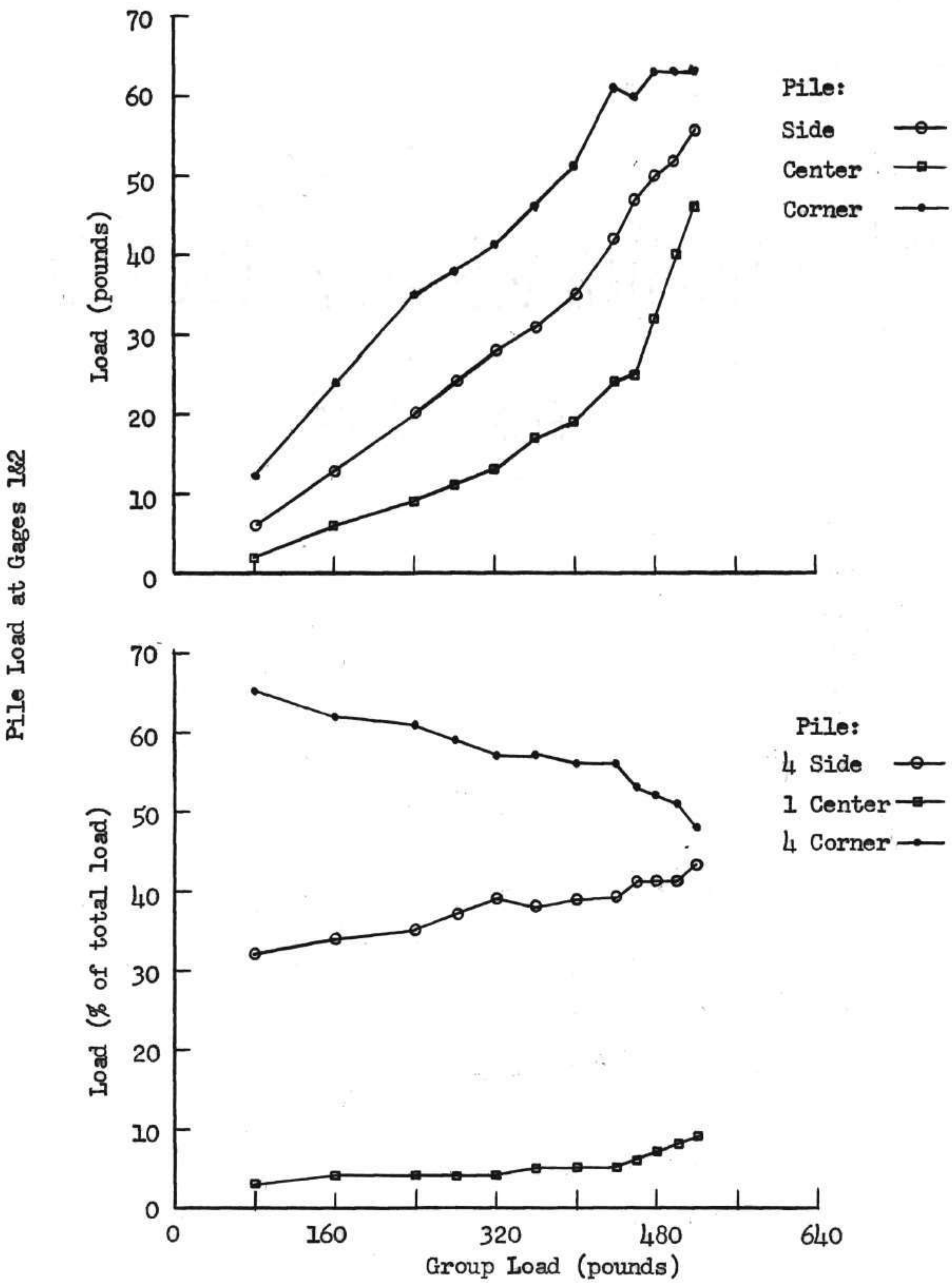
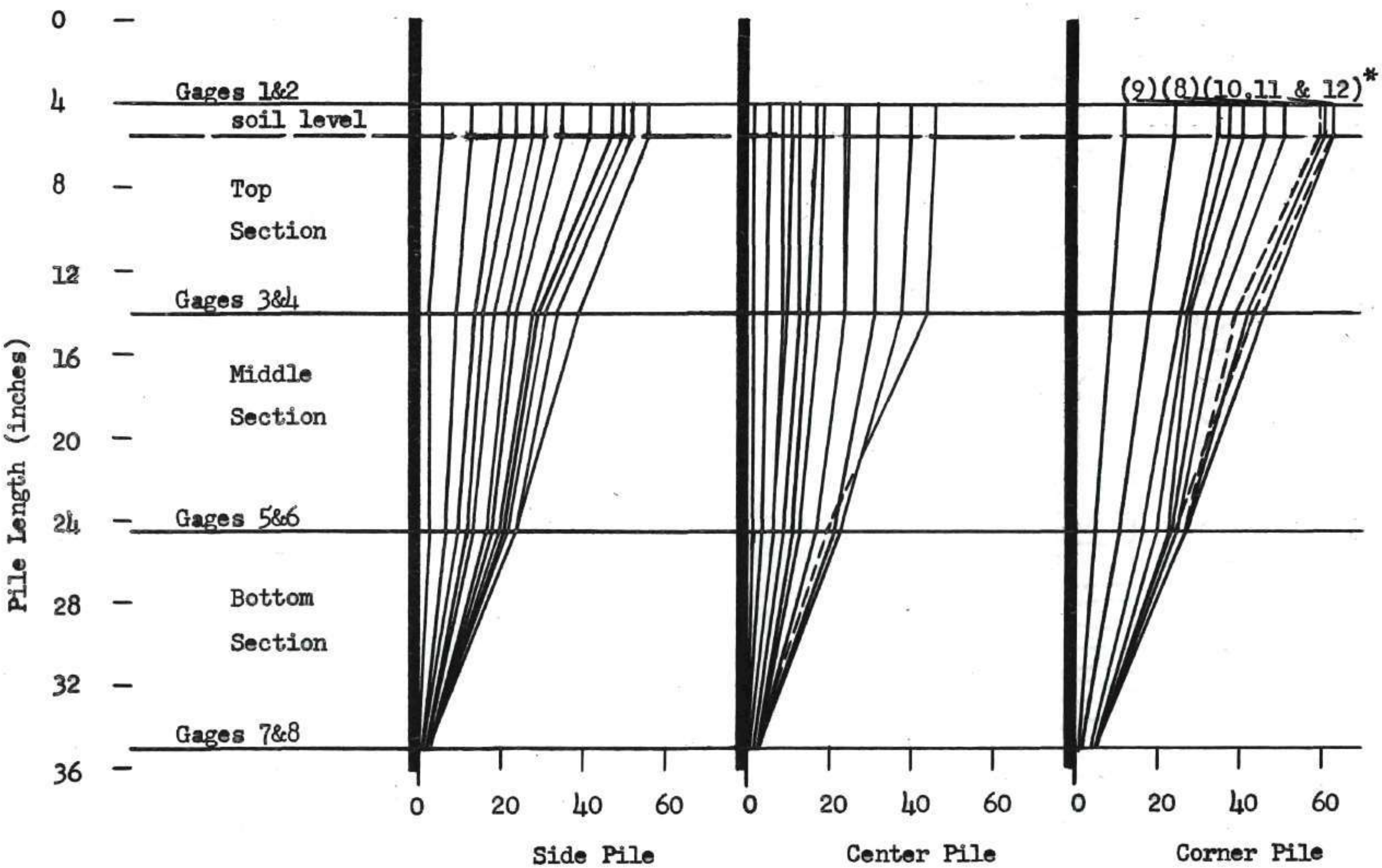


Fig. 22 Load Distributed From Cap to Piles (2,5 Diameter Spacing)



Load in Pile (pounds) for each Increment of Group Load

Fig. 23 Load Distribution Curves (2.5 Diameter Spacing)

*Clarifies increments of group loads

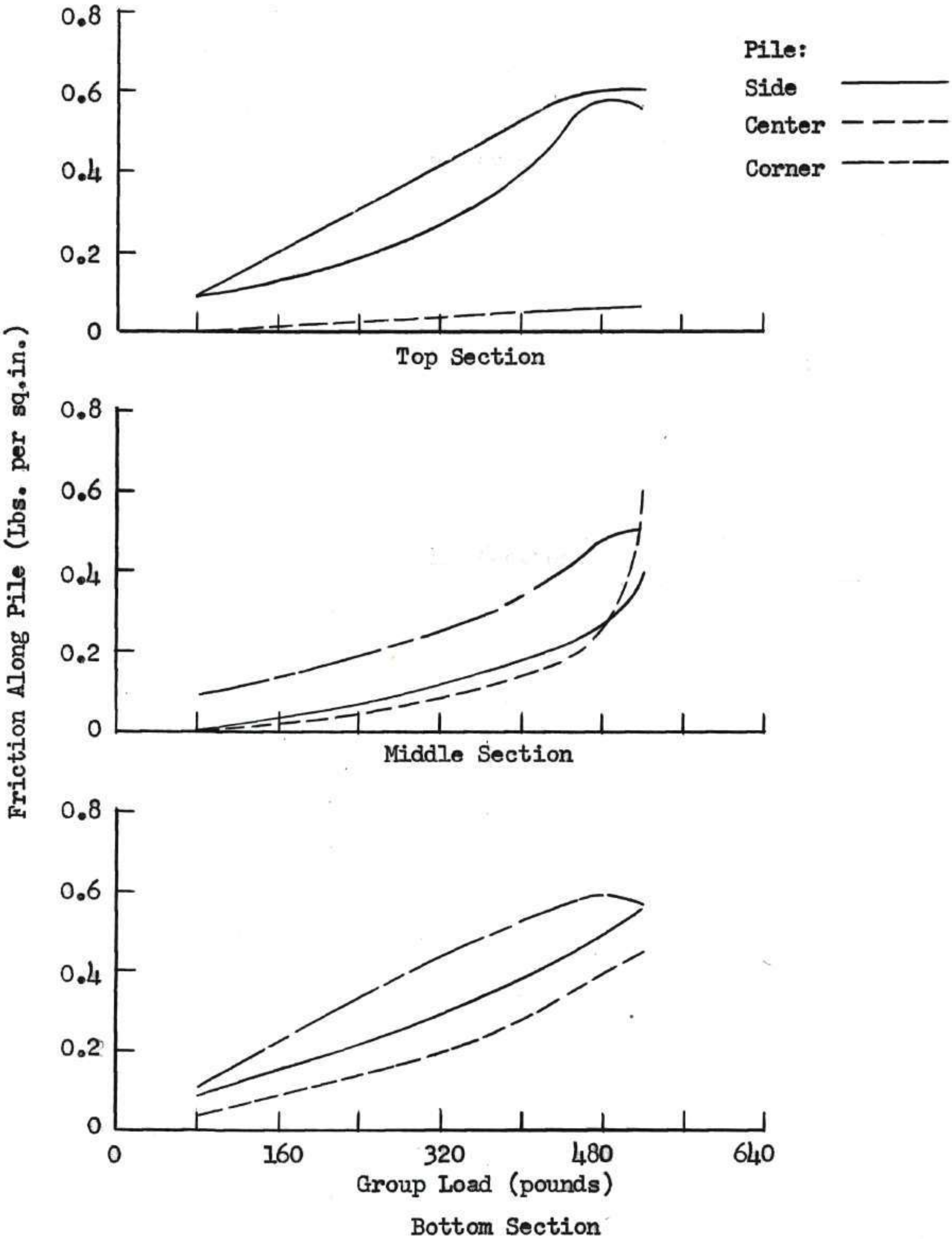


Fig. 24 Relative Frictions Between Piles (2.5 Diameter Spacing)

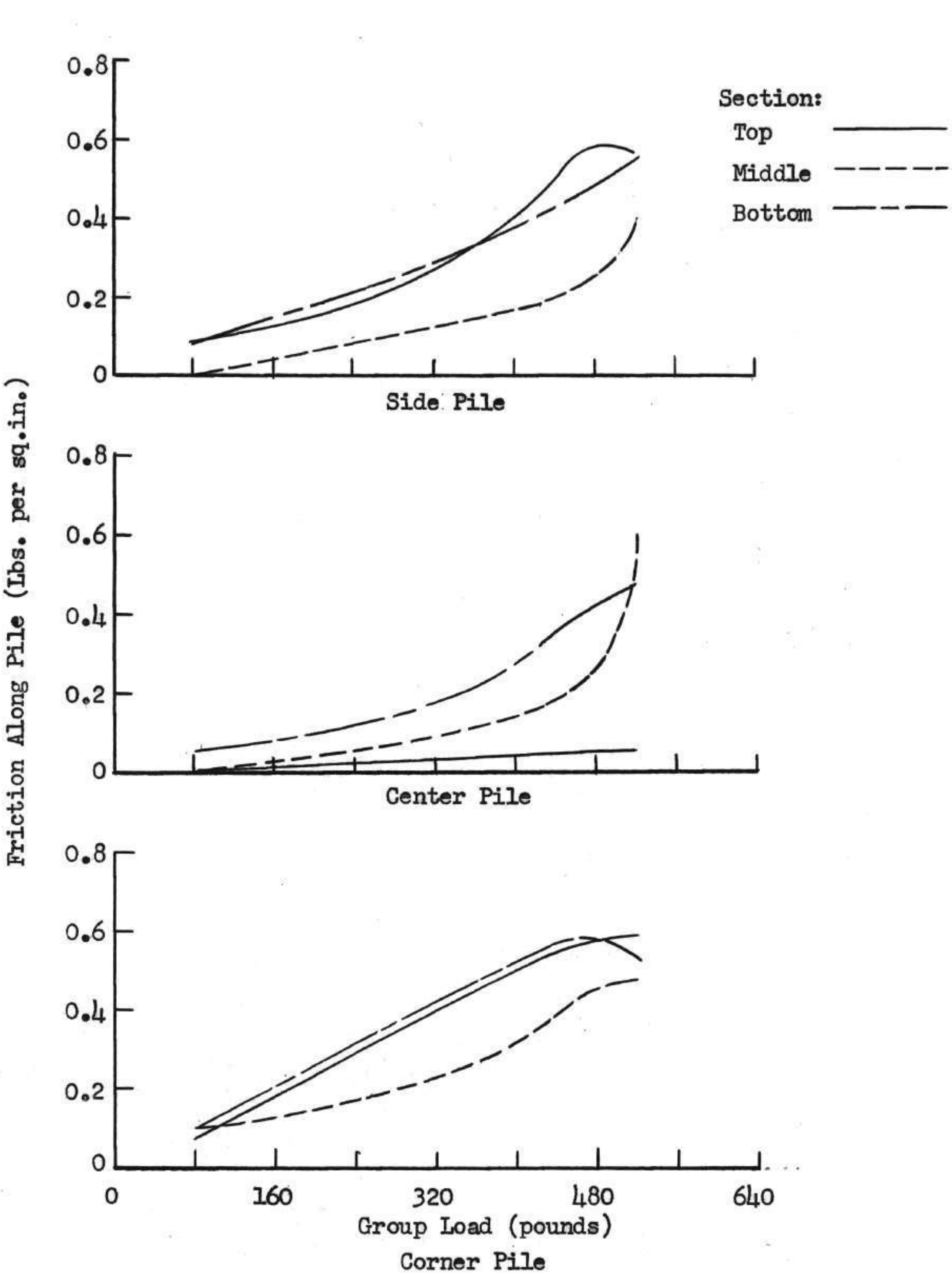


Fig. 25 Relative Frictions Between Sections (2.5 Diameter Spacing)

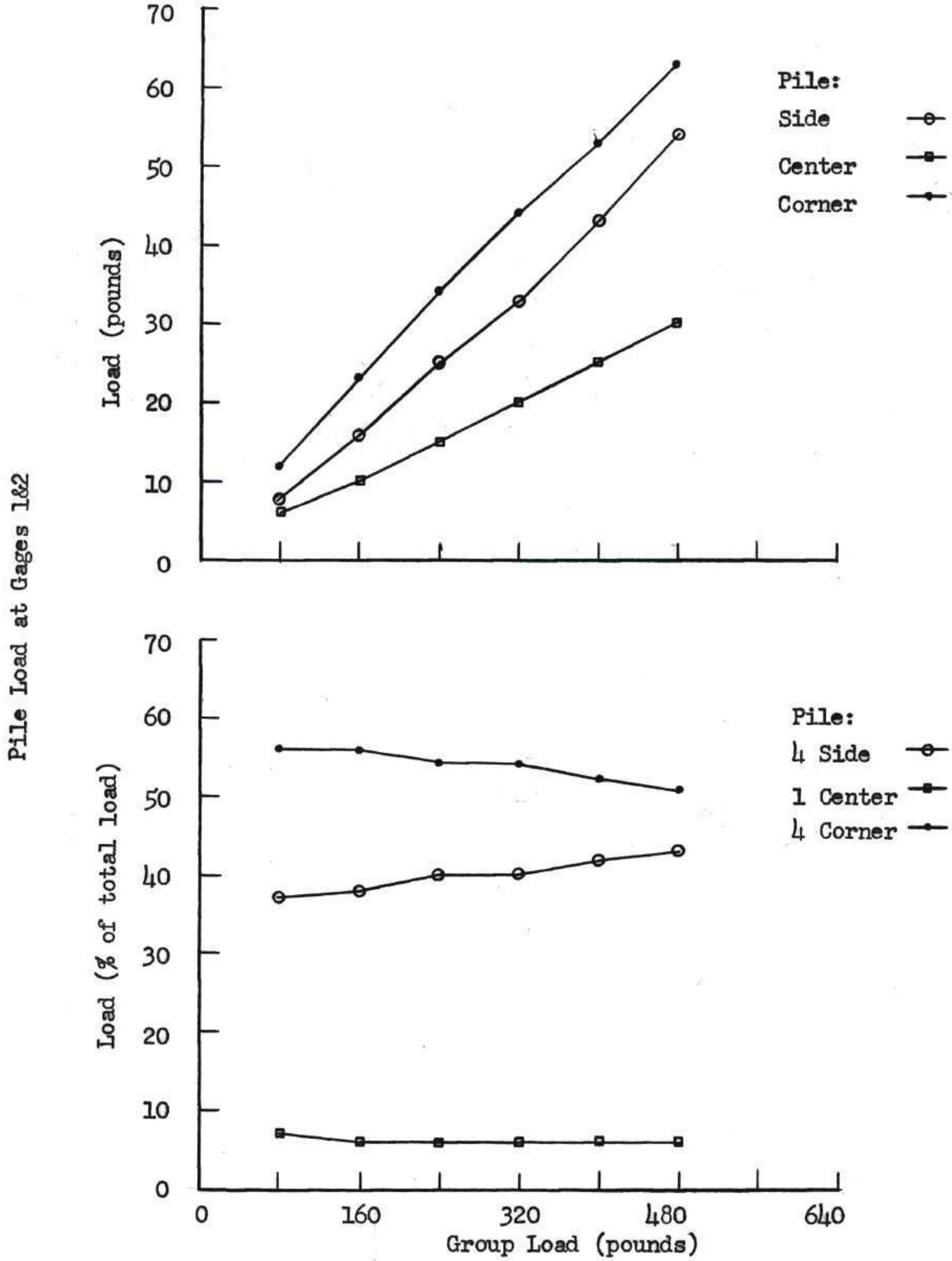


Fig. 26 Load Distributed From Cap to Piles (3.5 Diameter Spacing)

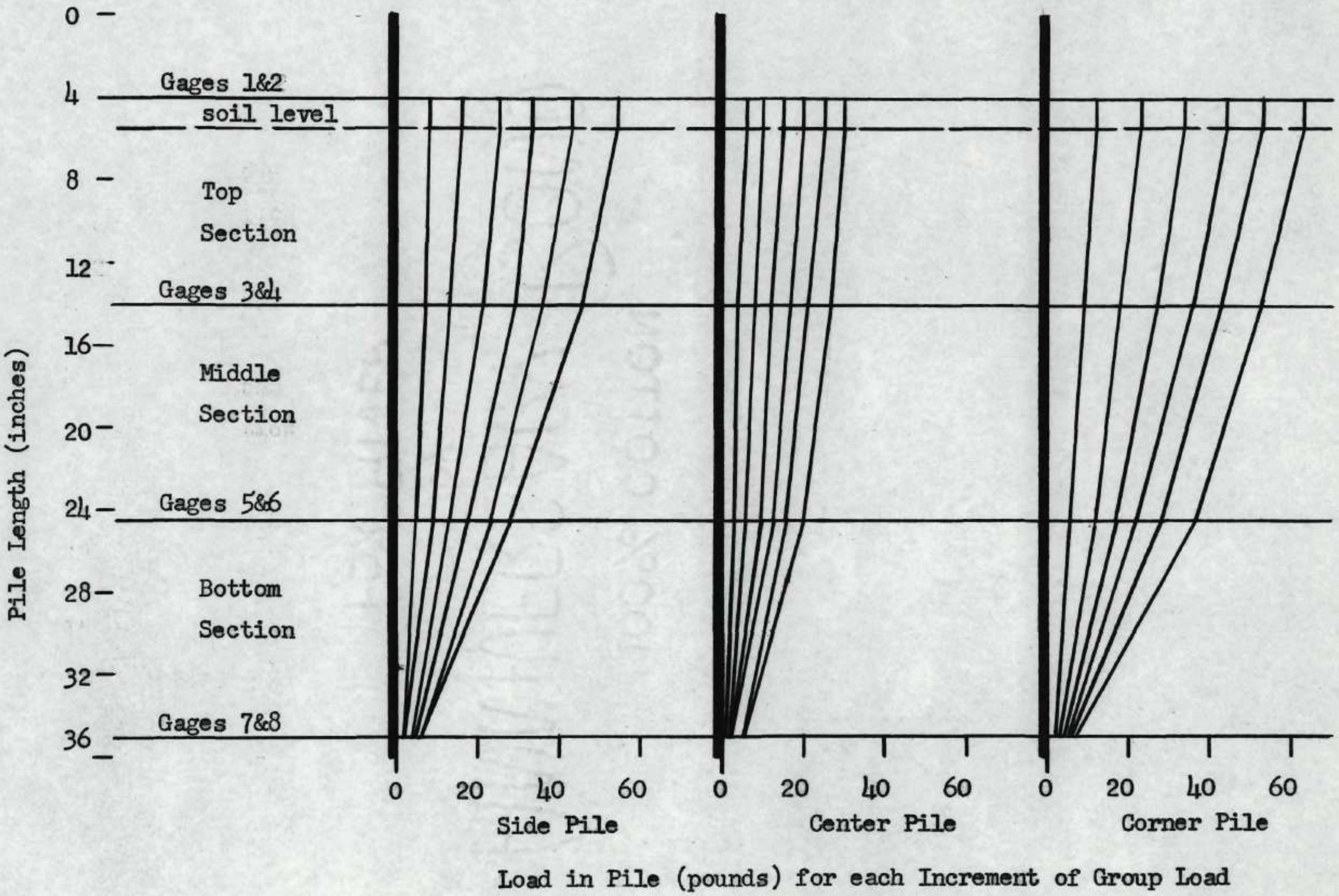


Fig. 27 Load Distribution Curves (3.5 Diameter Spacing)

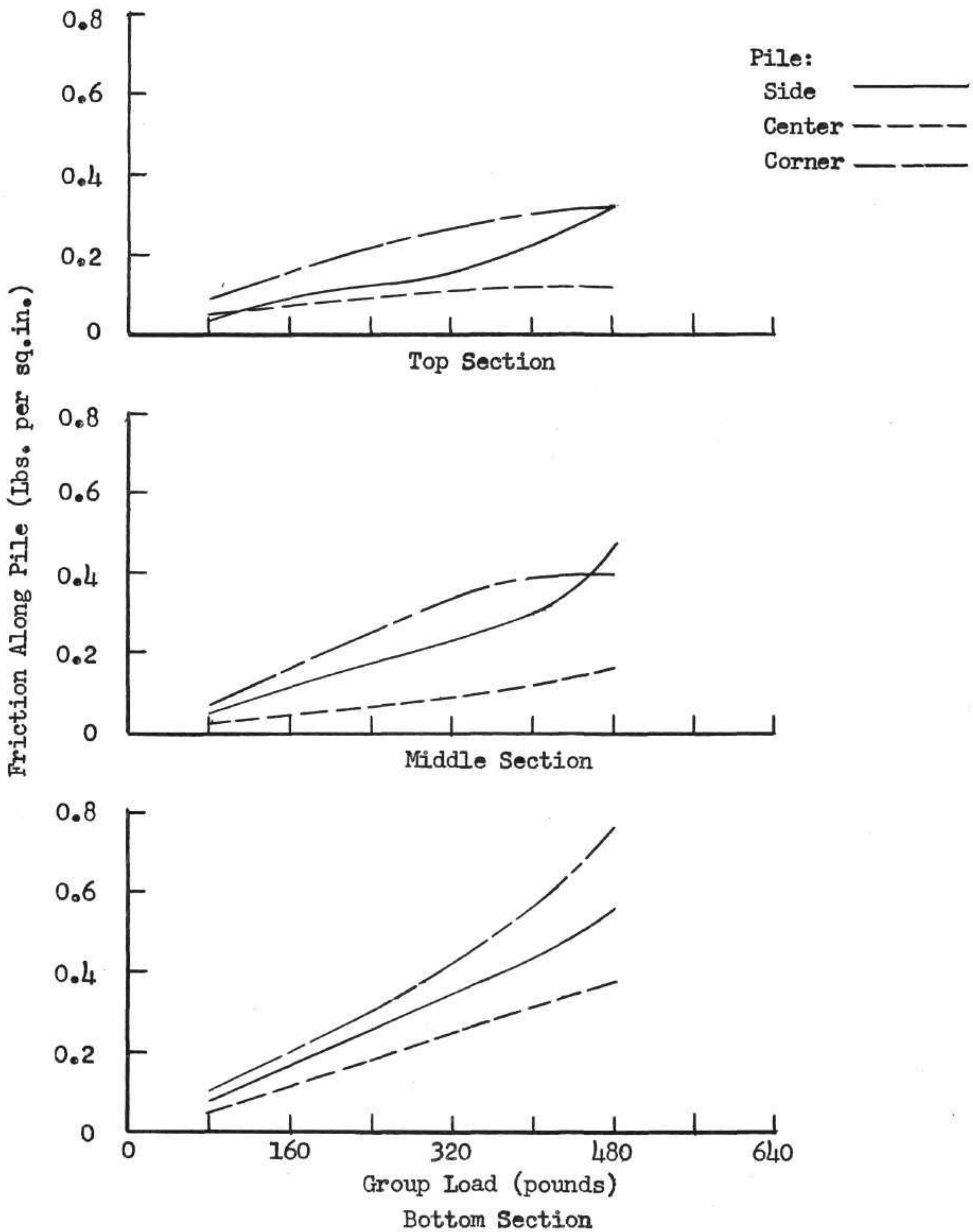


Fig. 28 Relative Frictions Between Piles (3.5 Diameter Spacing)

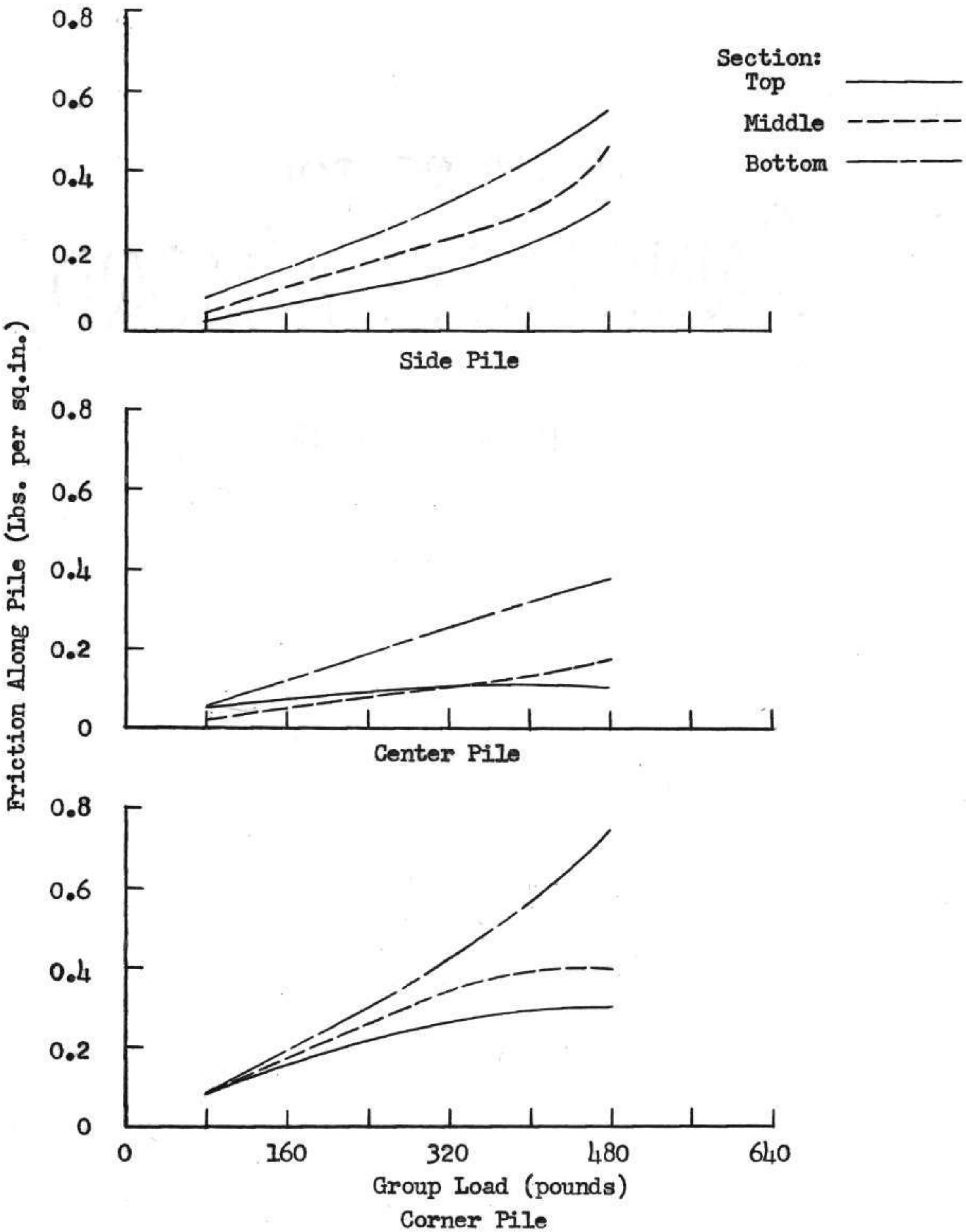


Fig. 29 Relative Frictions Between Sections (3.5 Diameter Spacing)

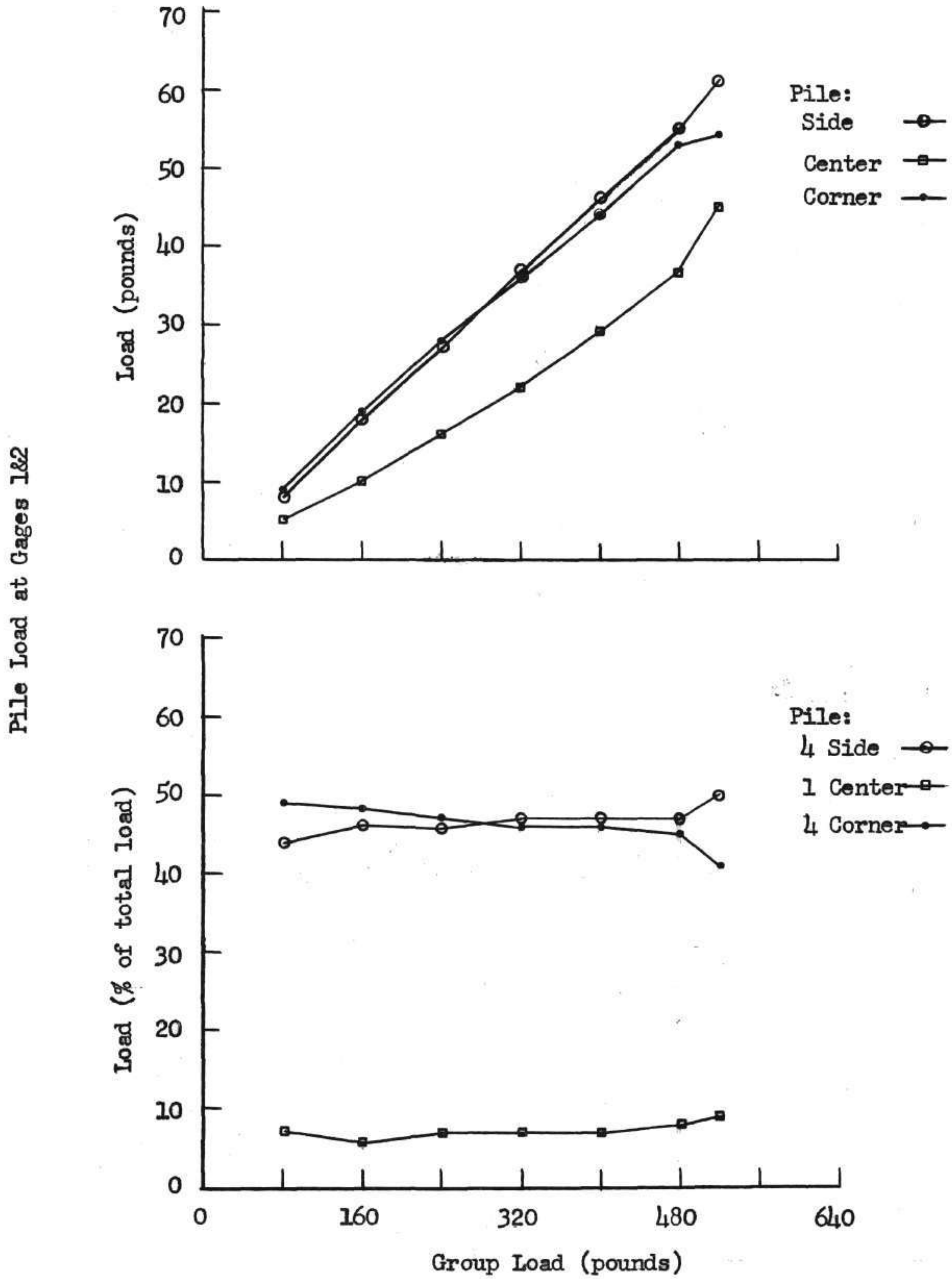


Fig. 30 Load Distributed From Cap to Piles (5.0 Diameter Spacing)

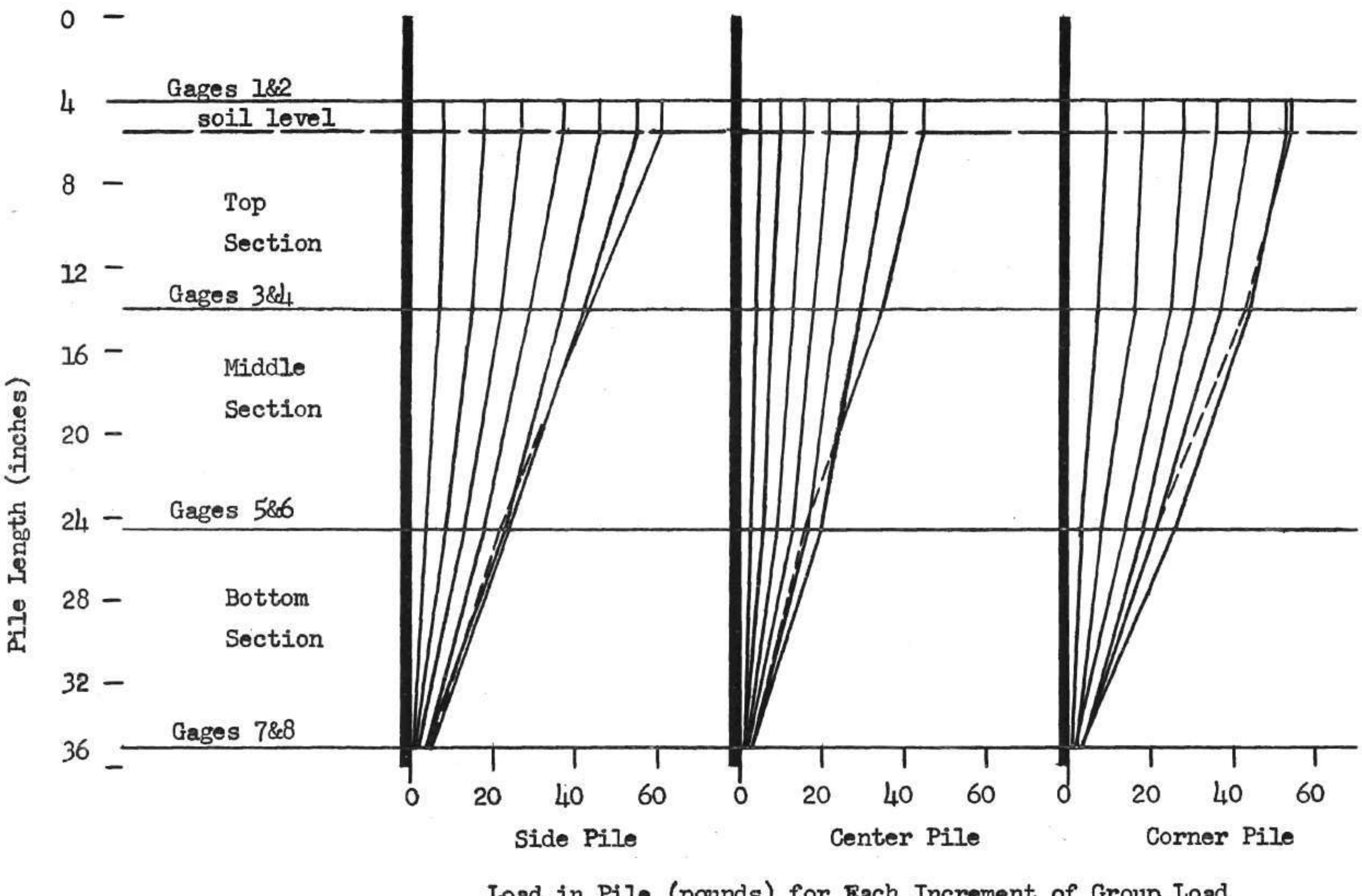


Fig. 31 Load Distribution Curves (5.0 Diameter Spacing)

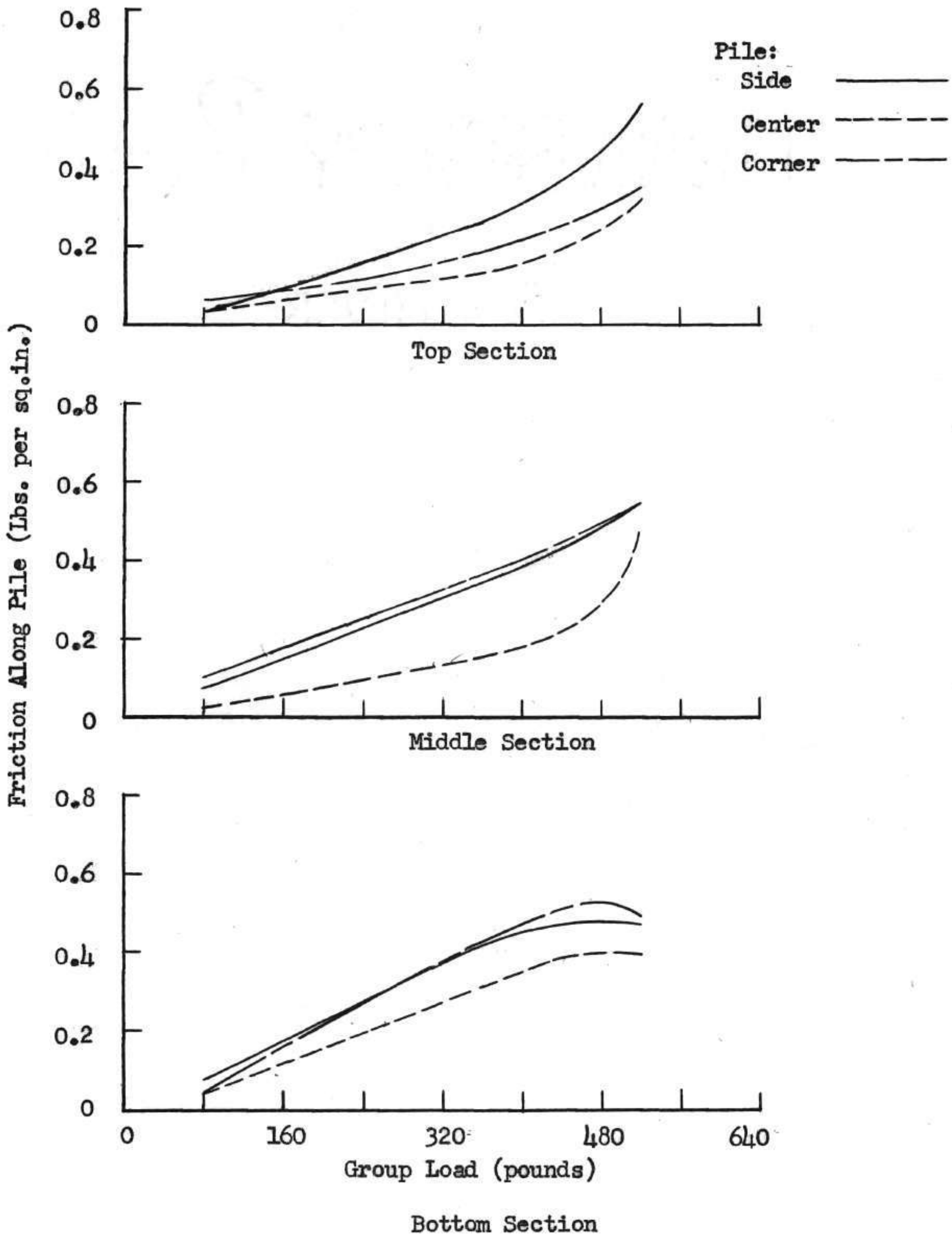


Fig. 32 Relative Frictions Between Piles (5.0 Diameter Spacing)

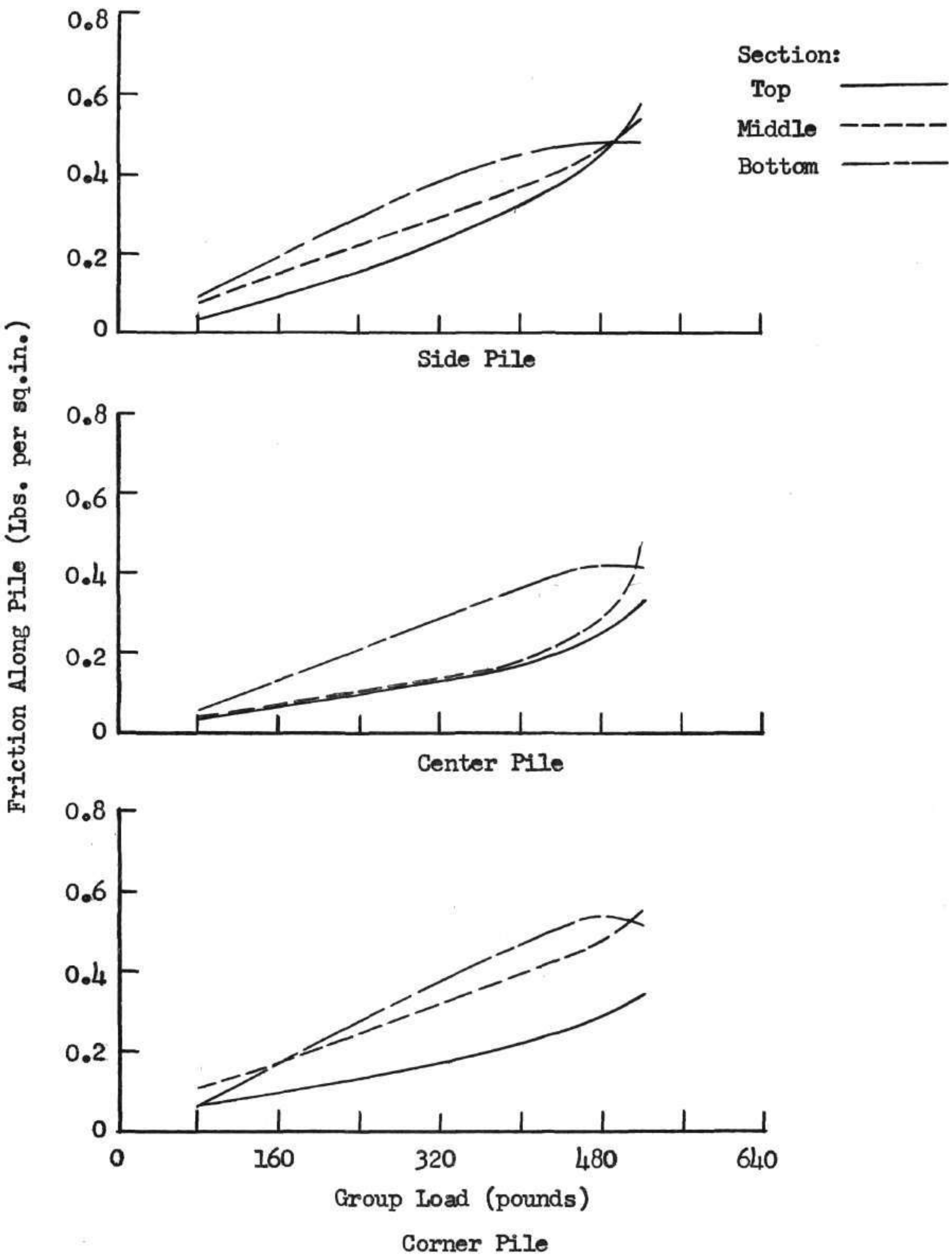


Fig. 33 Relative Frictions Between Sections (5.0 Diameter Spacing)

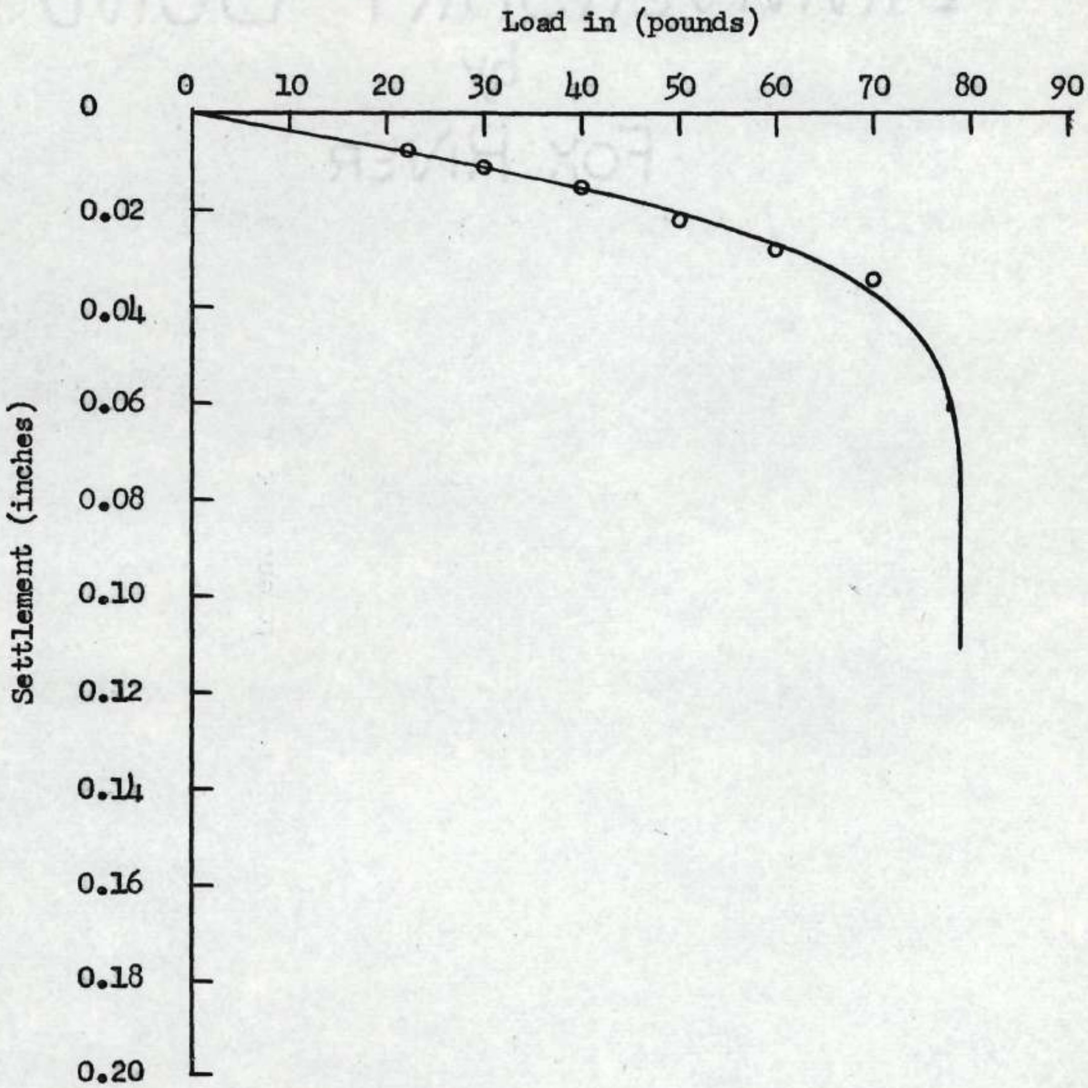


Fig. 34 Load-Settlement (Single Pile)

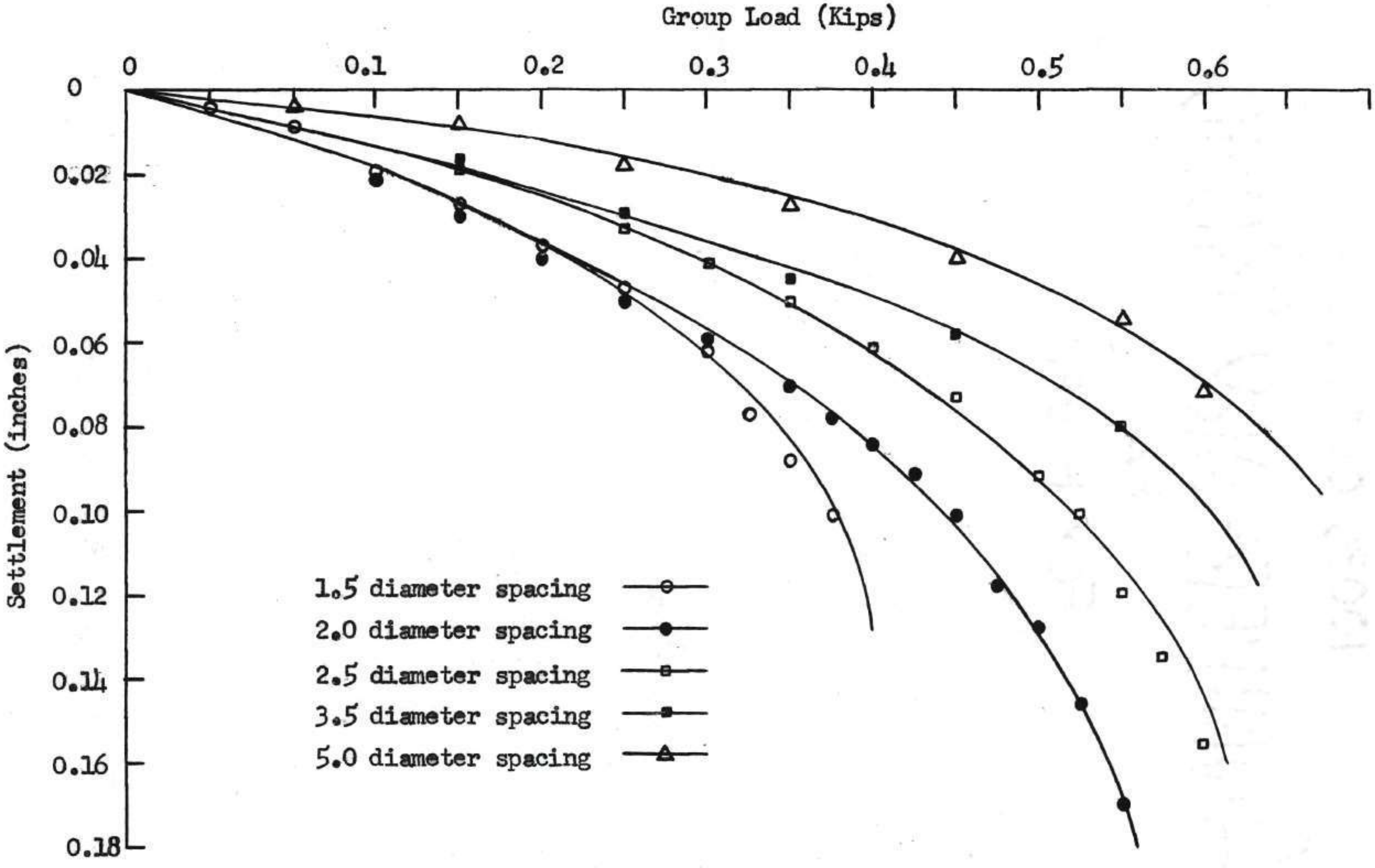


Fig. 35 Load-Settlement Curves (Nine Pile Groups)

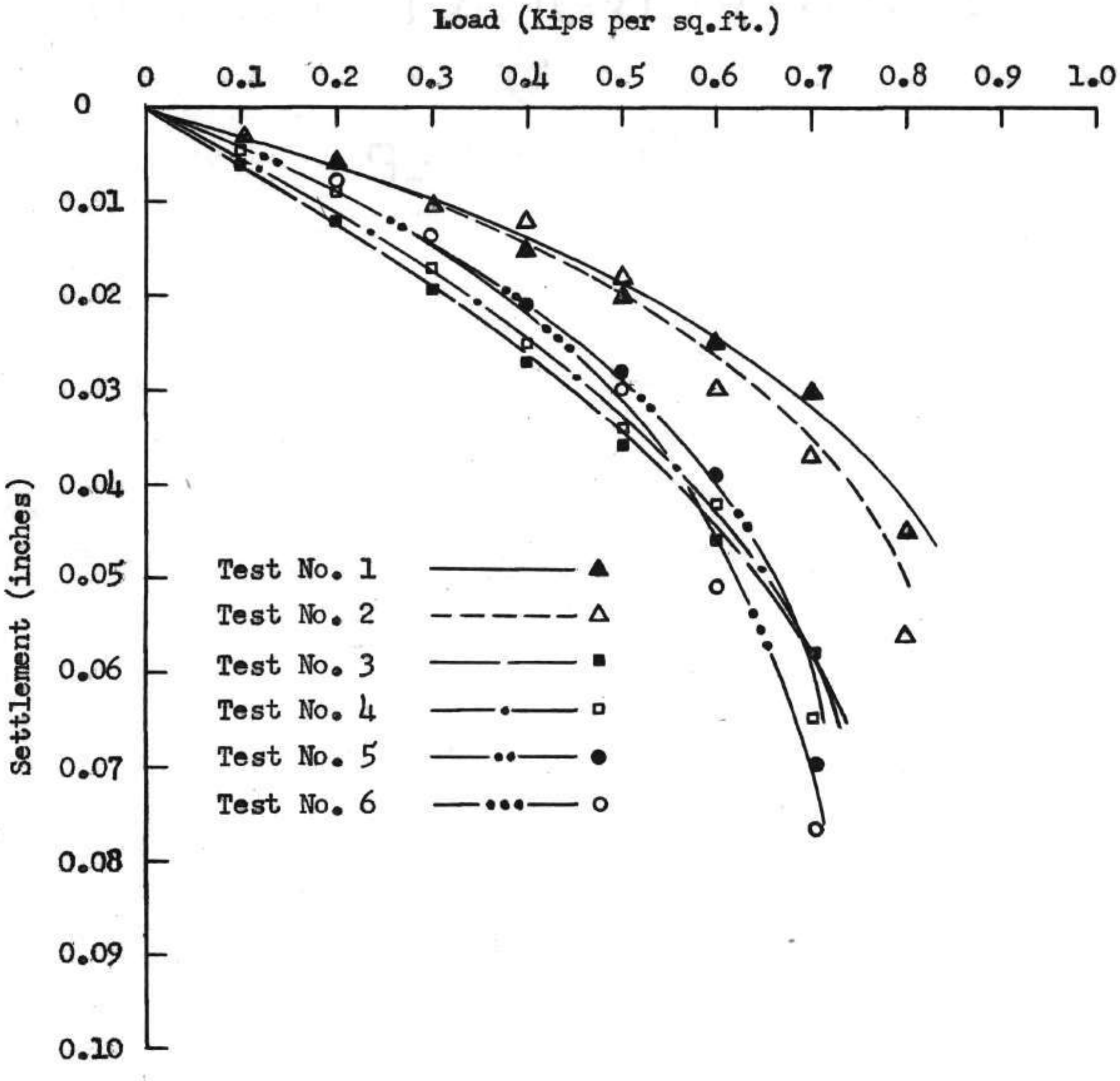


Fig. 36 Plate Load Test Results

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