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DEEP FOUNDATIONS

proceedings

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OHIO RIVER VALLEY SOILS SEMINAR

IX

OHIO RIVER VALLEY SOILS SEMINAR IX

SPEAKER SCHEDULE

8:30 am	Registration
9:00 am	Welcome Joseph Hagerty, Moderator
9:15 am	Interpretation and Analysis of Pile Load Tests Bengt H. Fellenius, BHF Consultants, Inc., Montreal, Canada
10:15 am	Axial Compression and Uplift Resistance of Steel H-Piles David R. Friels, Test, Inc., Memphis, Tennessee
10:45 am	Coffee Break
11:15 am	Load Transfer Measurements in Concrete Piles. William L. Durbin, Woodward-Clyde Consultants, Kansas City, Missouri
11:45 am	Friction Piles in Sand — A Review of Static Design Procedures William A. Cutter, ATEC Associates, Inc., Indianapolis, Indiana David L. Warder, ATEC Associates, Inc., Indianapolis, Indiana
12:15 pm	Lunch
1:30 pm	Reconvene for Afternoon Session Gerry Roberto, Moderator
1:45 pm	A Pile Design and Installation Specification Based on the Load Factor Concept G. G. Goble, University of Colorado at Boulder
2:45 pm	Tests to Obtain Behavior of Drilled Shafts under Axial Load Lymon C. Reese, University of Texas at Austin
3:45 pm	Break
4:15 pm	Contracting for Deep Foundations — Legal Aspects C. Robert Lennertz, The H. C. Nutting Company, Cincinnati, Ohio
5:00 pm	Questions and Answers
5:45 pm	Social Hour
6:30 pm	Dinner
7:30 pm	Evening Session — "The Salvaging of the Abu Simbel Temples" Lennart Berg, VBB, Stockholm, Sweden

Proceedings of the Ninth

OHIO RIVER VALLEY SOILS SEMINAR

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

October 27, 1978
Drawbridge Inn
Fort Mitchell, Kentucky
(across the Ohio River from Cincinnati)

Sponsored by

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OHIO RIVER VALLEY SOILS SEMINAR IX

TABLE OF CONTENTS

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INTERPRETATION AND ANALYSIS OF PILE LOAD TESTS

Bengt H. Fellenius, P. Eng., Dr. Tech.

BHF Consultants Inc.

Introduction

This paper deals with the analysis of results from axial testing of vertical single piles, that is, the most common test performed. Despite the numerous tests which have been carried out and the many papers which have reported on such tests and the analysis thereof, the understanding of pile test loading in current engineering practice leaves much to be desired. The reason for this is that the engineers have concerned themselves with mainly one question, only: "does the pile have a certain least capacity?", finding little of practical value in analysing the actual capacity and the pile-soil interaction. These lecture notes aim to show that engineering value can be gained from elaborating on a pile test, during the actual testing in the field as well as in the analysis of the results.

The first portion of the notes make use of an earlier paper by the Author (Fellenius, 1975). However, additional views and recent literature have been incorporated.

Testing methods

The most common test procedure is the slow maintained load method referred to as the "standard loading procedure" in the ASTM Designation D-1143 (ASTM 1974) in which the pile is loaded in eight equal increments up to a maximum load, usually twice the predetermined allowable load. Each increment is maintained until zero settlement is reached, defined as 0.01 in/h (= 0.002 in/10 min.). The final load, the 200% load, is maintained for 24 hours. The "standard method" is very time consuming requiring from 30 to 70 hours to complete. It should be realized that the words "zero settlement" are very misleading, as the settlement rate of 0.01 in/hr is equal to a settlement of 7 ft/yr.

The "standard method" can be speeded up by using the method of equilibrium proposed by Mohan et al. (1967), where the load (jack pressure) is allowed to drop rather than being maintained by pumping. The equilibrium load value is taken as the load applied on the pile.

Housel (1966) proposes that each of the eight increments be maintained exactly one hour regardless of having reached "zero" settlement or not. The Housel method of applying the load at equal time intervals allows an analysis of movement with time, which is not possible with the "standard method". By plotting the magnitude of movement obtained during the last 30 minutes of each one hour load duration versus the applied load, two approximately straight lines are obtained, that is, if the test has approached failure. The intersection of the two lines is termed yield value.

A maintained-load test according to Housel's method takes a full day to perform. The number of points on the curve are still very few, but Housel's method is a definite improvement of the "standard method" and it has been incorporated as an optional method in the ASTM Designation D-1143. However, it is the Author's opinion that a test consisting of 16 equal increments of say 30 tons applied every 30 minutes would provide a better test than 8 increments of 60 tons applied every 1.0 hour, because it would provide a better defined load-movement curve. A similar yield value, and one not much different, can be evaluated from the movement during the last 15 minutes, also, provided that readings are taken often enough and that they are accurate. But why stop at 16 increments, when 32 every 15 minutes determine the load deformation curve even better? The load is still applied at a constant rate in terms of tons per hour and no principal change is made.

Actually, the duration of each load is less important, be it 1.0 hour or 15 minutes. It is the fact that the duration of each load is the same, which is important. From this realization, we can progress to the one that even shorter time intervals, and an increase of the rate of loading in tons/hour, are possible without impairing the test. Actually, by using as short time intervals as practically possible, the time dependent influences are reduced and a more truly undrained test is obtained. In the cases, when a study of the time dependent, or drained conditions, creep etc., is desirable, the test duration should be measured in weeks, months or even years. A 48 or 72 hour test is then vastly inadequate, and results only in a mixing of apples and oranges.

Tests, which consist of load increments applied at constant time intervals of 5, 10 or 15 minutes, are called Quick Maintained-

Load Tests (ML-tests) and are from both technical, practical and economical point of view superior to the slow ML-tests. They are relatively recently introduced in North America, but are steadily gaining acceptance. The latest version of the ASTM Designation has one quick ML-method as an optional method. For instance, recently the Federal Highway Administration published an extensive users' manual for a quick ML-method (Butler and Hoy, 1977).

The quick ML-method should aim for 15 to 25 increments with the maximum load determined by the amount of reaction load available or the ultimate capacity of the pile. For routine cases, in order not to rock the boat too much, it may be diplomatically preferable to stay at a maximum load of 200% of the intended allowable load. For ordinary test arrangements, where only the load and the pile head movement are monitored, time intervals of 5 minutes are suitable and allow for the taking of 2 to 4 readings for each increment (for instance, when reaching the load, and at 2.5, 4.0 and 5.0 minutes after starting to load). When testing instrumented piles, where the instruments take a while to read (scan), the time interval may have to be increased. To go beyond 15 minutes, however, should not be necessary. Nor is it advisable, because of the potential risk for influence of time dependent movements, which may impair the test results. Usually, a quick ML-test is completed within two to three hours.

A quick test which has gained much use in Europe is the constant-rate-of-penetration test (C.R.P.-test), first proposed internationally for piles by Whitaker (1957 and 1963) and Whitaker and Cooke (1961). Manuals on the C.R.P.-test have been published by the Swedish Pile Commission (1970) and New York Department of Transportation (1974). In the C.R.P.-test, the pile head is forced to settle at a predetermined rate, normally 0.02 in/min (0.5 mm/min), and the load to achieve the movement is recorded. Readings are taken every two minutes and the test is carried out to a total penetration (i.e., movement of the pile head) of two to three inches (50 to 75 mm) or to the maximum capacity of the reaction arrangement, which means that the test is completed within two to three hours.

The C.R.P.-test has the advantage over the Quick ML-test that it enables an even better determination of the load-deformation curve. This is of particular value in testing friction piles, when sometimes the force needed to achieve the penetration gets smaller after a peak value has been reached. It also agrees with the testing in most other engineering fields, which regularly use C.R.P.-methods to determine strength and stress-strain relations. To perform a C.R.P.-test, access is required to a mechanical pump that can provide a constant and non-pulsing flow of oil. Ordinary pumps with a pressure holding device, manual or mechanical, are not suitable because of unavoidable loading variations. Also, the absolute requirement of simultaneous reading of all load and deformation gauges (changing continuously) could be difficult to achieve without a trained staff. For these reasons, the Quick ML-method is preferable for instrumented piles.

A fourth test method is cyclic testing. However, cyclic methods will not be described here. For details see Fellenius (1975), and references contained therein. In routine tests, cyclic loading or even single unloading and loading phases must be avoided. It is a common misconception that unloading a pile every now and then according to some more or less "logical" scheme will provide information on the tip movement. It will only result in a destruction of the chances to analyse the test results and the pile load-deformation behaviour. In non-routine tests and for a specific purpose, cyclic testing can be used, but then <u>after</u> completion of an initial test and when having the pile instrumented with at least a telltale to the pile tip.

There is absolutely no logic in believing that anything of value can be obtained from cyclic testing, or occasional unloadings, or one or a few resting periods at certain load levels, when considering that we are testing a unit that is subjected to the influence of several soil types, is already under stress of unknown magnitude, exhibits progressive failure, etc., when all we know is what we apply and measure at the pile head, while we really are interested in what happens at the pile end.

Interpretation of failure load

For a pile which is stronger than the soil, the ultimate failure load is reached when rapid settlements occur under sustained or slightly increased load - the pile plunges. However, this definition is inadequate, because plunging requires very large movements and it is often less a function of the capacity of the pile-soil system and more a function of the man-pump system.

In the past, a common definition of failure load has been the load for which the pile head movement exceeds a certain value, usually 10% of the diameter of the pile end. This definition does not consider the elastic deformations of the pile, which can be substantial for long piles and negligible for shor piles. In reality, a limit movement relates only to the allowable deformation limits of the superstructure to be supported by the pile, and not to the load test results.

Sometimes, the failure value is defined as load value at the intersection of two lines, approximating an initial pseudo-elastic portion of the load-movement curve and a final pseudo-plastic portion. This definition results in interpreted failure loads, which depend greatly on judgment and, above all, on the scales of the graph. Change the scales and the failure value changes also. A load test is influenced by many occurrences, but the draughting should not be one of these.

To be useful, a failure definition must be based on some mathematical rule and generate a repeatable value that is independent of scale relations and the opinions of the individual interpreter. In some way, it has to consider the shape of the load-movement curve or, if not, it must consider the length of the pile (which the shape of the curve indirectly does). Without a proper definition, every interpretation becomes meaningless.

The test results given as a load-movement curve in Fig. 1 will be used to present nine different definitions of failure. The example pile is a 12 inch concrete pile installed through 60 ft. of sensitive clay, 10 ft. of clayey silt and 6 ft. of silt. The pile was tested six weeks after driving. Method of testing was the C.R.P.-method. The pile started to plunge when test load reached 200 tons, but at the maximum load of 206 tons the load necessary to achieve the movement was still increasing.

In Fig. 2 is applied a method proposed by Davisson (1972) also referenced by Peck et al. (1974). Davisson's limit value is defined as the load corresponding to the movement which exceeds the elastic compression of the pile by a value of 0.15 inch (4 mm) plus a factor equal to the diameter of the pile divided by 120. For the 12 inch diameter example pile, the value is 0.25 inch (6 mm). The Davisson limit was developed in conjunction with wave equation analysis of driven piles and has gained a widespread use in phase with the increasing popularity of this method of analysis. It is primarily intended for test results from driven piles tested according to quick methods.

Fig. 3 gives the method proposed by Chin (1970 and 1971) for piles in applying the general work by Kondner (1963). The method assumes that the load-movement curve, when the load approaches the failure load is of hyperbolic shape. By the Chin method, each load value is divided with its corresponding movement value and the resulting value is plotted against the movement. As shown in Fig. 3, after some initial variation, the plotted values fall on straight line. The inverse slope of this line is the Chin failure load.

Generally speaking, two points will determine a line and third point on the same line confirms the line. However, beware of this statement when using Chin's method. It is very easy to arrive at a false Chin value if applied too early in the test. Normally, the correct straight line does not start to materialize until the test load has passed the Davisson limit. As a rule, the Chin failure load is about 20% to 40% greater than the Davisson limit. When this is not a case, it is advisable to take a closer look at all the test data.

The Chin method is applicable on both quick and slow tests, provided constant time increments are used. The ASTM "standard method" is therefore usually not applicable. Also, the number of monitored values are too few in the "standard test"; the interesting development could well appear between load increment number seven and eight and be lost.

Fig. 4 presents a method proposed by De Beer (1967) and De Beer and Wallays (1972), where the load movement values are plotted in a double logarithmic diagram. When the values fall on two approximately straight lines, the intersection of these defines the failure value. De Beer's method was originally proposed for slow tests.

Fig. 5 illustrates a method proposed by Brinch Hansen (1963), who defines failure as the load that gives twice the movement of the pile head as obtained for 90% of that load. This method, also called the 90%-criterion, gained widespread use in Scandinavia (Swedish Pile Commission, 1970). Brinch Hansen (1963) also proposes an 80%-criterion defining the ultimate load as the load that gives four times the movement of the pile head as obtained for 80% of that load. The 80%-criterion failure load can be estimated by extrapolation from the curve to be about 210 tons.

In Fig. 6 Brinch Hansen's 80%-criterion is shown in a plot, which is very similar to the Chin plot. However, the ultimate failure value is not as easily determined from the slope of the line, but must be calculated. In this case, the calculation results in a failure value of 211 tons, agreeing well with the value extrapolated from the load-movement curve directly.

Brinch Hansen's 80%-criterion postulates that the load movement curve is approximately parabolic. Chin postulates that it is approximately hyperbolic. The shape of the actual curve is obviously close enough to both mathematical curves to allow both approximations. Brinch Hansen's 80%-criterion results generally in a failure value about 10% lower than Chin's value. Note that both methods allow the latter part of the curve to be plotted according to a mathematical relation, and, which is often very tempting, they make an "exact" extrapolation of the curve possible. However, do not fool yourself to believe that the extrapolated part of the curve is as true as the measured.

In Fig. 7, the method by Mazurkiewicz (1972) is illustrated. A series of equal pile head movement lines are arbitrarily chosen and the corresponding load lines are constructed from the intersections of the movement lines with the load-movement curve. From the intersection of each load line with the load axis, a 45° line is drawn to intersect with the next load line. These intersections fall, approximately, on a straight line which intersection with the load axis defines the failure load. Also this method is based on the assumption that the load-movement curve is approximately parabolic. Consequently, the interpreted failure load of Mazurkiewicz's method is close to that of Brinch Hansen's 80%-criterion. However, when drawing the line through the intersections according to Mazurkiewicz, some freedom of choice is usually found.

In Fig. 8 a simple definition proposed by Fuller and Roy (1970) is shown. The failure load is equal to the test load where the load movement curve is sloping 0.05 inch per ton.

Fig. 8 also shows a development of the above definition proposed by Butler and Hoy (1977) defining the failure load as the load at the intersection of the tangent sloping 0.05 inch/ton and the tangent to the initial straight portion of the curve or to a line that is parallel to the rebound portion of the curve. As the latter portion is more or less parallel to the elastic line (see Fig. 2), the Author suggests that the intersection be that of a tangent parallel to the elastic line, instead.

The Fuller and Hoy method penalizes the long pile, because the larger elastic movements occurring for a long pile, as opposed to a short pile, causes the slope of 0.05 inch/ton to occur sooner. The Butler and Hoy development takes the elastic deformations into account, substantially offsetting the length effect.

Fig. 9 shows the construction of the failure load as proposed by Vander Veen (1953). A value of the failure load, P_{ult}, is chosen and values calculated from $\ln(1-P/P_{ult})$, are plotted against the movement. When the plot becomes a straight line, the correct P_{ult} has been chosen. The Vander Veen method was proposed long before programmable pocket calculators were available. Without those, however, the application is very time consuming.

In Fig. 10, the above determined nine values are plotted together. As shown, the Davisson limit of 181 tons is lower than all the others and the Chin value of 235 tons is the highest. The other seven values are grouped more or less together around an average of 200 tons.

It is difficult to make a rational choice of the best criterion to use, because the preferred criterion is heavily dependent on one's past experience. One of the main reasons for having a strict criterion is, after all to enable a set of compatible reference cases to be established. The Author prefers to use, not one, but two or three of the criteria given in Fig. 10. The preferred criteria are the Davisson limit load, the Chin failure load and the Butler and Hoy failure load. In case of an engineering report, the preference and experience of the receiver of the report may result in the use of one of the others, also.

The Davisson limit is chosen because it has the tremendous merit of allowing the engineer, when proof testing a pile for a certain allowable load, to determine in advance the maximum allowable movement for this load with consideration of the length and size of the pile. Thus, as proppsed by Fellenius (1975), contract specifications can be drawn up including an acceptance criterion for piles proof tested according to quick testing methods. The specifications can simply call for a test to at least twice the design load, as usual, and declare that at a test load equal to a factor, F, times the design load, the movement shall be less than the elastic column compression of the pile, plus 0.15 inch, plus a value equal to the diameter divided by 120. The factor F is a safety factor and should be chosen to a value of 1.5 to 1.8 depending on circumstances.

The Chin method is chosen, because it allows a continuous check on the test, if a plot is made as the test proceeds, and a prediction of the maximum load that will be applied during the test. Sudden kinks or slope changes in the Chin line indicate that something is amiss with either the pile or with the test arrangement. The Chin value has the additional advantage of being less sensitive to imprecisions of the load and movement values.

The Butler and Hoy method is chosen primarily because of its resemblance to the Davisson method. In some cases a Davisson limit load can be obtained without the interpreter being willing to accept intuitively that the pile has reached failure. (In such cases, the Chin value will be much higher than the Davisson limit). However, the Butler and Hoy slope of 0.05 inch/ton is not approached unless failure is imminent, and absence of a Butler and Hoy failure indicates - in addition to a high Chin value - that the Davisson value is imprecise. The reasons for the latter can be wrongly chosen values of pile elastic modulus or pile length, imprecise or erroneous load or movement values, or just that the pile load-movement curve is only slightly steeper than the elastic line. Also the Butler and Hoy method permits an acceptance criterion for proof tested piles to be formulated and included in the specifications. However, the Butler and Hoy method requires the pile head movement to be large enough to reach the Fuller and Hoy point, which restricts the use of the definition in this context.

Influence of errors

The test results shown in Fig. 1 and used in the preceeding discussions are from a test where an electrical strain gauge load cell was used to determine the load applied on the pile. In the test, the pressure in the jack was monitored by means of a manometer, which had been calibrated together with the jack. Yet, the load determined from the manometer readings was inaccurate. Fig. 11 shows the difference between the load determined from the jack pressure and the load determined by the load cell, as plotted against the load cell load.

The error (overestimation) in the jack pressure load is substantial and varies between 10 and 25%, being mostly 15 to 20%. In unloading the pile, the error was much smaller. This is not the worst, nor the best example the Author has met with, but an example typical for the equipment used in the industry of today.

Fig. 11 shows also similar results from another test, called Example 1 A, when in loading the error was less than 5%. On the other hand, the error in unloading is large. This seems to be a jacking equipment of a much better quality than the one used in Example 1. However, Example 1 A is from an identical pile located about 20 ft. away and tested two days later using the same equipment and method.

Based on the above and many similar measurements results, the Author concludes that if we want to ensure an imprecision smaller than about 20 to 25%, a load cell must be used. The jack and jack pressure are too erratic to be reliable. A calibration of the jack and manometer for one pile is not applicable on even neighboring test pile. The reasons for the unreliability is that we are requiring the system to do two things at the same time; both provide the load and measure it. Any measurement specialist can tell you that load cells with moving parts are considerably less reliable than those without moving parts. The same specialist does a beautiful job when he calibrates our equipment, ensuring that no excentric loadings, bending movements, or temperature variations influence the calibration. In the field, all of these factors are indeed at hand to influence the test results to an unknown extent, unless a load cell is used.

Naturally, many structures are safety supported on piles which have been tested with erroneous loads. Actually, as long as we are content to stay with the same old rules, loads and piling systems, we do not need to improve the precision. The error is included in the safety factor. That is why factors as large as 2.0

and 2.5 are applied and such numbers are really more ignorance factors than safety factors. However, if we want to economize and continue to increase the allowable loads as our geotechnical know-how increases, we cannot accept potential errors as large as 20 to 25%. In the Author's opinion, we cannot accept errors exceeding 10%.

However, the fact that a load cell is used is no guarantee for precise loads. Fig. 12 shows calibrations performed on a flat-jack load cell under varying conditions. The heavy center line is a regular calibration curve obtained when having 3 inches thick full-width steel plates on both sides of the load cell and applying the load through a spherical bearing (swivel plate). This curve is very repeatable. However, by moving the load only 2 inches off center, a different calibration was obtained. By letting the temperature drop, a third line was obtained. The greatest influence was by removing the steel plates and loading only a center area of the load cell.

Of course, the load cell of Fig. 12 is unsuitable for use in the field, where temperature variations and excentric loading cannot be avoided. In a load test, the geometric center does not necessarily coincide with the load center. Therefore, be sure to check the calibration of the cell and its sensitivity to excentric load application. Do not leave this important phase of the test to the specialist, but go to the laboratory yourself and "mess up the calibration" much like you will do the test in the field.

The above deals with the imprecision of the load value. Also the precision of the movement values can be critical. If the "failure" criterion is a maximum settlement of 1.75 inches, an error of 0.25 inch is of no consequence, when the maximum movement recorded is 1.5 inches or less, which on most proof testing occasions it is. However, errors of the mentioned magnitude greatly influence the shape of the curve and the various methods of interpretation of failure loads. In particular, Davisson's limit is sensitive to these errors.

It must be remembered that the minimum distances from the supports of measuring beam to the pile and the platform etc., as recommended in the ASTM Designation, are really minimum values, most often not giving errors of much concern for ordinary testing, but too close for research or investigative testing purposes.

One of the greatest villains spoiling a load test is the sun. The measuring beam must be shielded from sunshine at all times.

The analysis of results from pile tests using telltale data

Fig. 13 shows the results from a Quick ML-test on a 130 ft. long (40 m) 12-inch (300 mm) precast concrete pile. The pile has a total cross section of 124 in^2 (800 cm^2) and the area of steel reinforcement is 1.9 in^2 (12 cm^2). Pile circumference is 42 inches (107 cm). The pile was loaded in steps of 22.4 tons. The failure loads evaluated according to the mentioned nine methods are given in the graph. The scatter of values is similar to that shown in Fig. 10. Naturally, a load cell was used to determine the test load.

In the test, a center pipe had been cast in the pile allowing a telltale to be inserted to the pile tip to monitor the compression of the pile and the pile tip movement. As will be shown, this relatively simple and cheap additon to the test enhanced tremendously the value of the test.

The graph in Fig. 13 shows also the movement of the pile tip and the measured compression of the pile. After a load of 70 to 90 tons, the measured compression plots in a straight line indicating that the part of the added load used for overcoming shaft resistance is constant. It would be highly improbable that the constant value is other than zero. Therefore, the applied additional load goes unreduced by shaft friction straight down to the pile tip, and the slope of the compression line is equal to the slope of the elastic line. The combined elastic modulus of the pile determined from this slope is $5.1 \cdot 10^6$ psi (36000 MPa).

According to a method of analysis proposed by Trow (1967), the pile tip starts to move, when the elastic line becomes tangent to the load movement curve of the pile head, and the load applied thereafter goes straight to the pile tip. The analysis by Trow is valid for a linear, i.e. triangular or rectangular, distribution of shaft resistance. The test results presented in Fig. 13 show that at a load of about 70 to 90 tons, the elastic line, established from the measured compression, becomes parallel to the load-movement curve. As mentioned, the data show that the load applied after the 70 to 90 ton load goes straight to the pile end, and the pile tip starts to move. Consequently, according to Trow's method of analysis, the shaft friction must be approximately linearly distributed, and the shaft friction value cannot be greater than about 70 to 90 tons.

When assuming constant unit shaft friction, the distribution of load in the pile becomes linear and from knowledge of the compression of the pile, Fellenius (1969) has shown that simple relations can be established for the load at the pile end and the total shaft resistance, as shown in Fig. 14. At pile head loads of 224, 246 and 280 tons, the measured compressions were 0.96, 1.07 and 1.24 inches, respectively. The values result in calculated pile tip loads of 160, 182 and 216 tons, respectively. The corresponding calculated pile shaft resistance was 64 tons for all three pile loads.

The value of 64 tons is less than the previously established maximum possible of about 70 to 90 tons. For many reasons, it is probable that the unit shaft resistance is not constant. Recently, Leonards and Lovell (1978) proposed a method of analysis using measured pile compression, which allows a variety of distributions of shaft resistance.

Leonards and Lovell established the following relations:

$$a = \frac{C' - C}{1 - C}$$

where

- a = ratio between the pile tip load and the load applied to the pile head. ($P_{tip} = aP$)
- C' = ratio of measured compression to column compression, the latter being the compression of a free column subjected to the same load as the pile.
- C = ratio of elastic compression of the pile at a load P supported totally by shaft friction to the column compression for the same load.

The ratio C' is known from the measured data. The purpose of the analysis is to from either knowledge of a, i.e. the tip load, determine C, i.e. the relative distribution of shaft resistance, or, inversely, from knowledge of relative distribution of shaft resistance determine the tip load.

The factual shaft resistance is not necessary to know in order to establish the ratio C. Leonards and Lovell (1978) have determined C for two principal patterns of shaft friction, which are presented in the nomograms in Figs. 15 and 16. The case of constant unit shaft resistance is a special case of the nomogram of Fig. 15; the two friction values are equal and C is 0.5. The previously mentioned three loads and average loads give values of C' of 0.714, 0.740 and 0.772 respectively. Insertion in the Leonards and Lovell relation gives values of the tip loads, which are equal to the ones calculated previously, i.e., 160, 182 and 216 tons.

The nomogram of Fig. 16 is applicable, when assuming a triangular distribution of shaft resistance. In this case, the ratio C becomes 0.667 and the calculated values of are 0.571, 0.610 and 0.658 respectively, resulting in the corresponding pile tip loads of 128, 150 and 184 tons, and a shaft resistance of 96 tons for all three loads.

The assumption of constant unit shaft friction along the entire length of the pile resulting in a shaft load of 64 tons, is probably incorrect. However, the shaft load of 96 tons calculated on the assumption of triangular distribution of shaft resistance is greater than the maximum possible shaft load. To arrive at a shaft load in between 64 and 96 tons, the analysis could be repeated with a C-ratio between 0.500 and 0.667, chosen either from Fig. 15 with two rectangular shaft friction patterns or from Fig. 16 with an upper triangular and a lower rectangular pattern. For instance, C = 0.58 determines the shaft load to 76 tons. However, no justification is available for further refinement. Such justification whould have been, for instance, a definite change of soil profile at some depth. Also, the load increments of 22 tons are too large to justify the refinement. An increment of ten tons, instead, would have shown much more precisely, the load-movement development during the first 100 tons of applied load.

The ratio C' is plotted in Fig. 17, as a function of the load at the pile head. The ratio C' is also plotted against the inverse of the load. According to the derivation of Leonards and Lovell (1978), the plot of C' vs the inverse of P is a straight line, if the change of compression, $d\Delta$, for a change of load, dp, is a constant value.

The equation for the line is:

$$C' = n - K\frac{1}{P}$$

where

$$n = \frac{AE}{L} \cdot \frac{d\Delta}{dp}$$

The factor n is equal to 1 only if all shaft friction has been mobilized, as in this case, which, as pointed out by Leonards and Lovell, is not necessarily always the case.

In Fig. 18, the calculated tip and shaft loads have been plotted for the two distributions of shaft resistance.

Fig. 19 shows another method of presenting the results of the analysis. The graph represents the load distribution in the pile for the different loads at the pile head. The straight line is the load distribution for constant unit shaft resistance (rectangular distribution) and the curved line is that for a triangular distribution of shaft resistance. The interesting fact in this graph is that to fulfill the condition that both load distributions must give the same average load in the pile, the two areas A' and A" must be equal.

The condition indicated in Fig. 19 can be developed to determine non linear load distributions in test piles, as proposed by Fellenius (1969). For instance, in a two layer soils profile, backfill and sand, where minimum two telltales have been inserted in the pile. In Fig. 20, a principal sketch shows a telltale placed at the pile tip and a second telltale located some distance above the pile tip. The measurements of the telltales give three values of average load in the pile, as marked in the graph. The straight lines "1" and "2" from the pile head load value through the two upper average loads plotted at midpoints between the telltale locations and the pile head, will be mathematically possible load distributions considering one telltale at a time. The true load distribution line must fulfill the conditions that the areas, A' and A" and B' and B", between the true line and the mathematical lines must be equal. In the graph, the "true" load distribution in the sand consists of a straight line, which necessitates that the load distribution line must go through the bottom average load. It also means that constant unit shaft resistance has been assumed. A triangular distribution of unit shaft resistance results in the curved load distribution line shown to the right.

In the analysis of example 2, use was made of measurements of tip movement (pile compression). The increase to the total cost of the test was minimal. The purpose of the presentation has been to demonstrate what tremendous value measurements of tip movement can give to the analysis of a test. It is an addition that is strongly recommended also for routine tests. For closed-end steel tube piles, it is simple to arrange. For precast prestressed concrere piles it requires a litte bit of advance planning so that a center tube can be cast in the pile. H-piles could necessitate some field welding and a few hours of preparations. Cast-in-place piles are not excluded. In view of the simpleness and low relative cost coupled with the large amount of extra information gained, by no means exhausted in this lecture, there is not much excuse for not incorporating tip measurements even in routine tests.

When in addition, the routine tests are run according to quick testing methods consisting of many more readings of load and movement than eight, all taken at equal time intervals and a reliable load cell is used, the state-of-the-art of pile testing and understanding of pile-soil interaction will take a giant leap forward.

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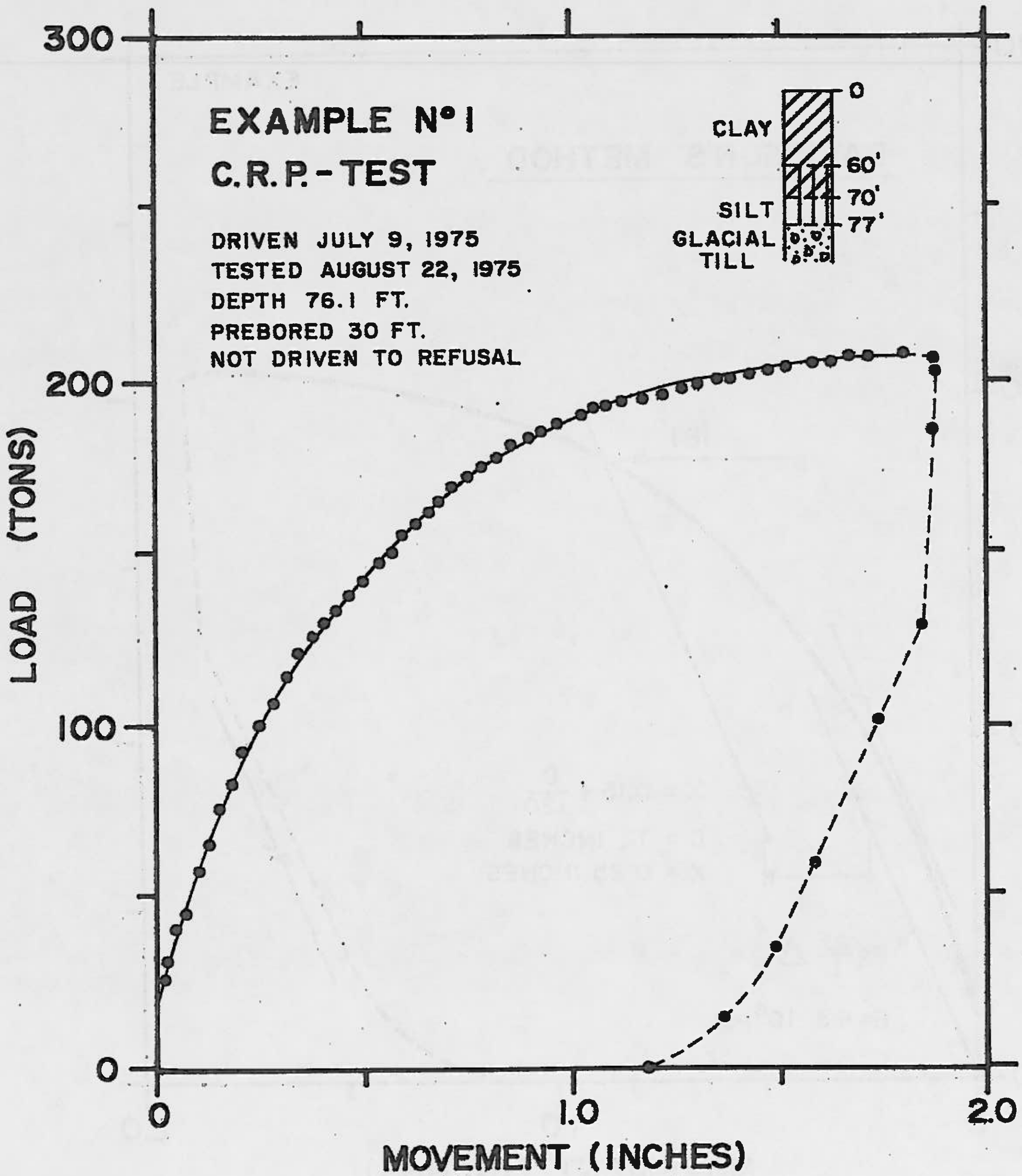


FIG. 1. LOAD-MOVEMENT DIAGRAM FROM C.R.P.-TEST.

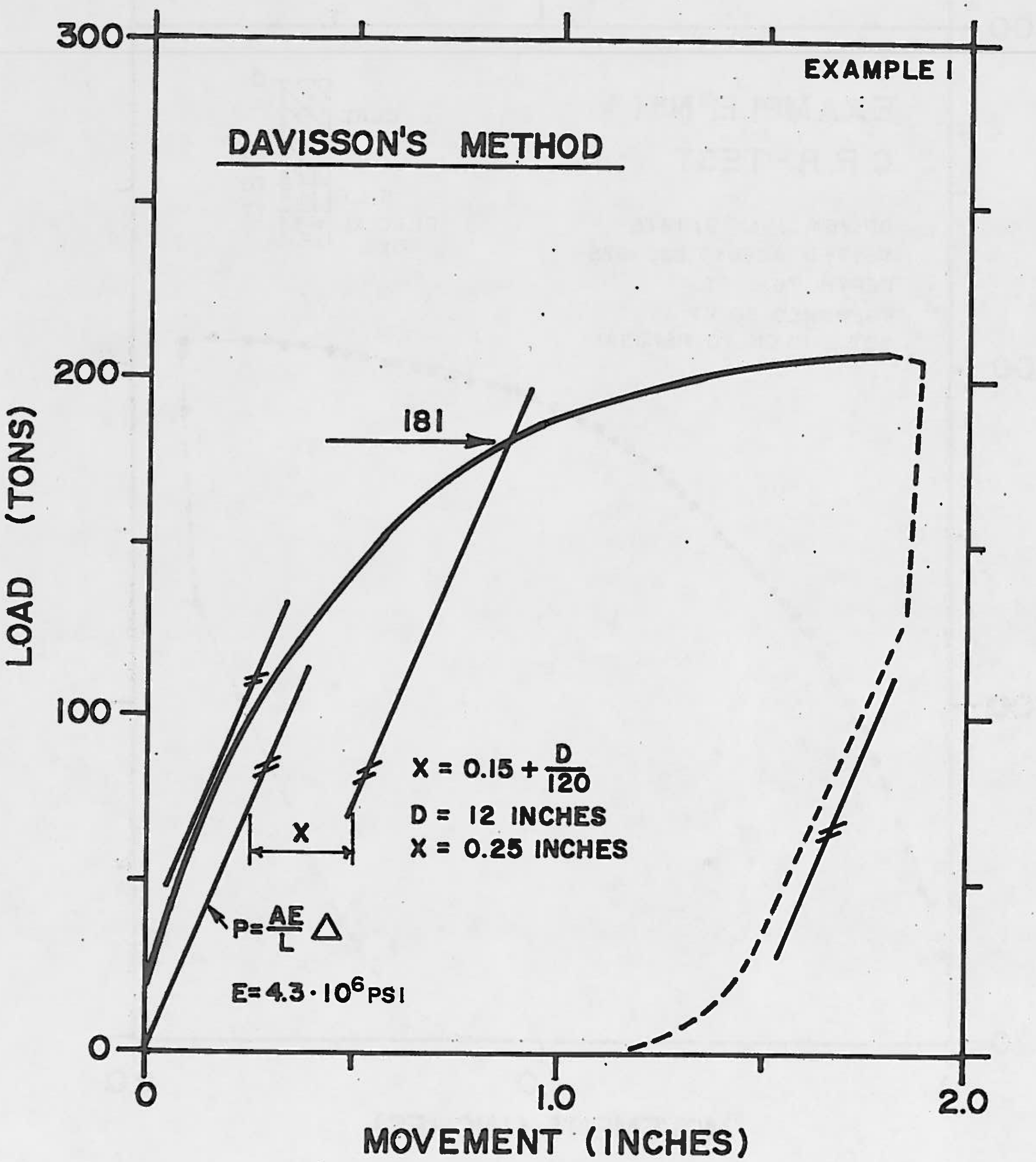


FIG. 2. CONSTRUCTION OF DAVISSON'S LIMIT.

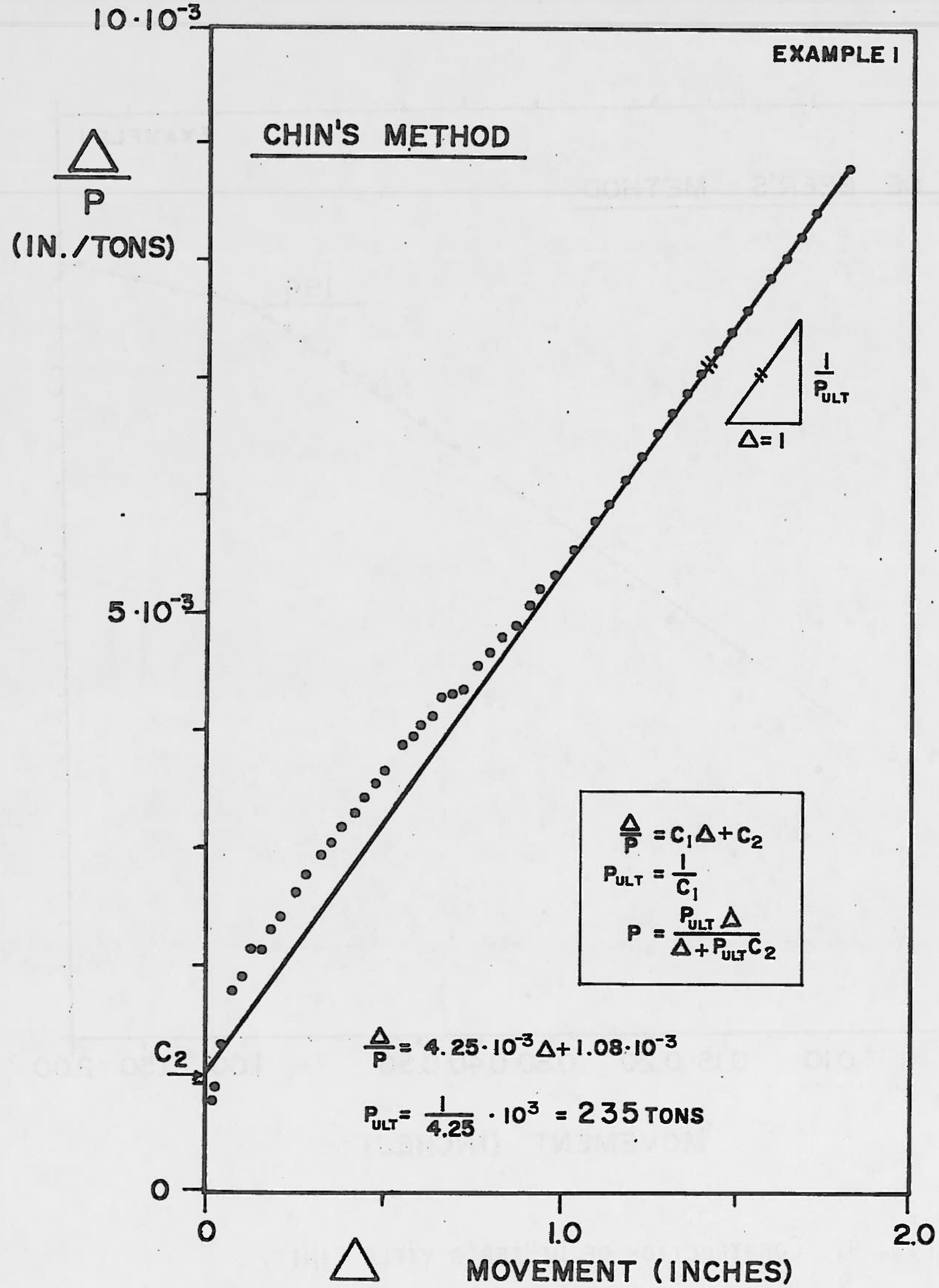


FIG. 3. ULTIMATE FAILURE ACCORDING TO CHIN.

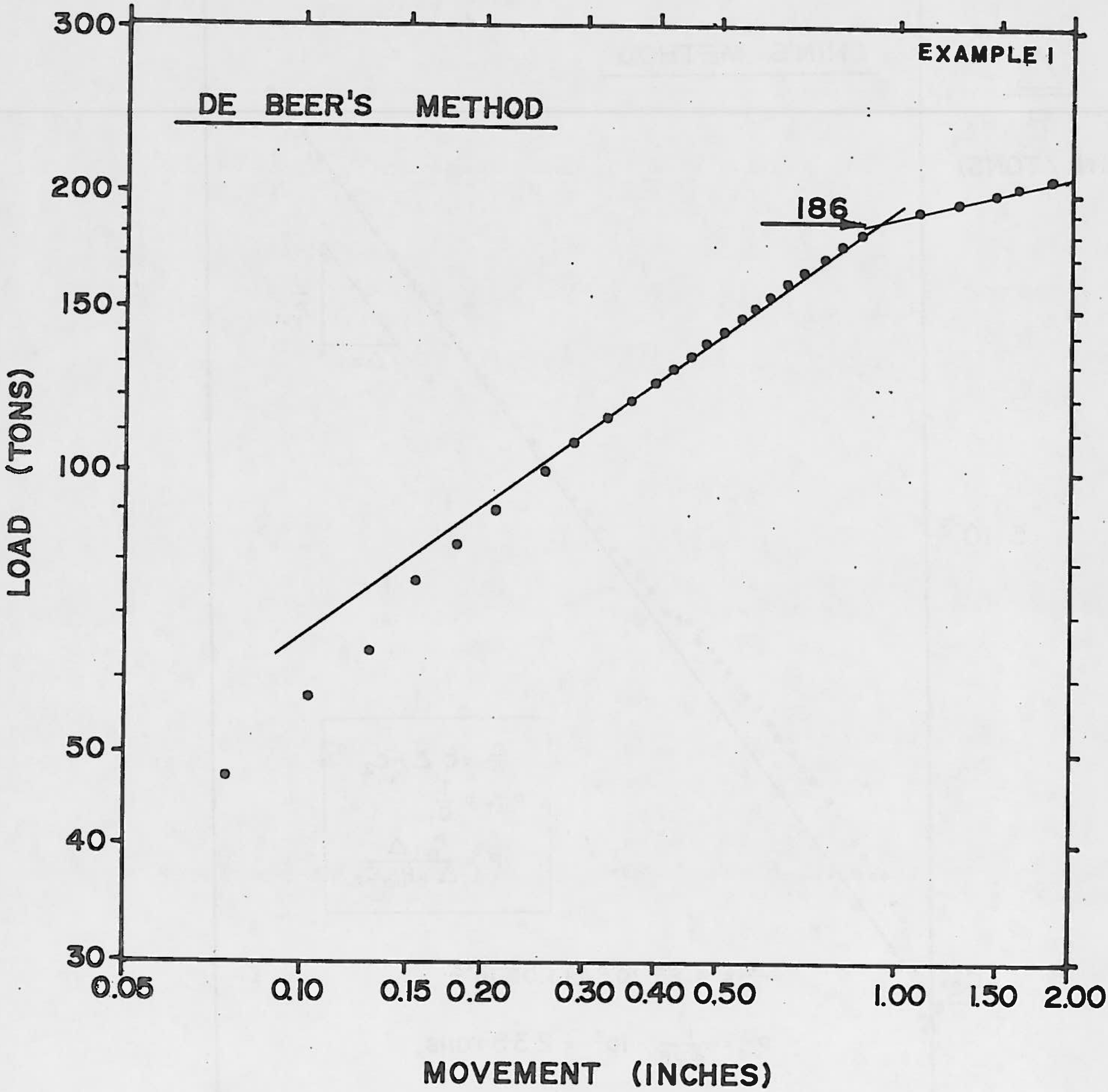


FIG. 4. CONSTRUCTION OF DEBEER'S YIELD LIMIT.

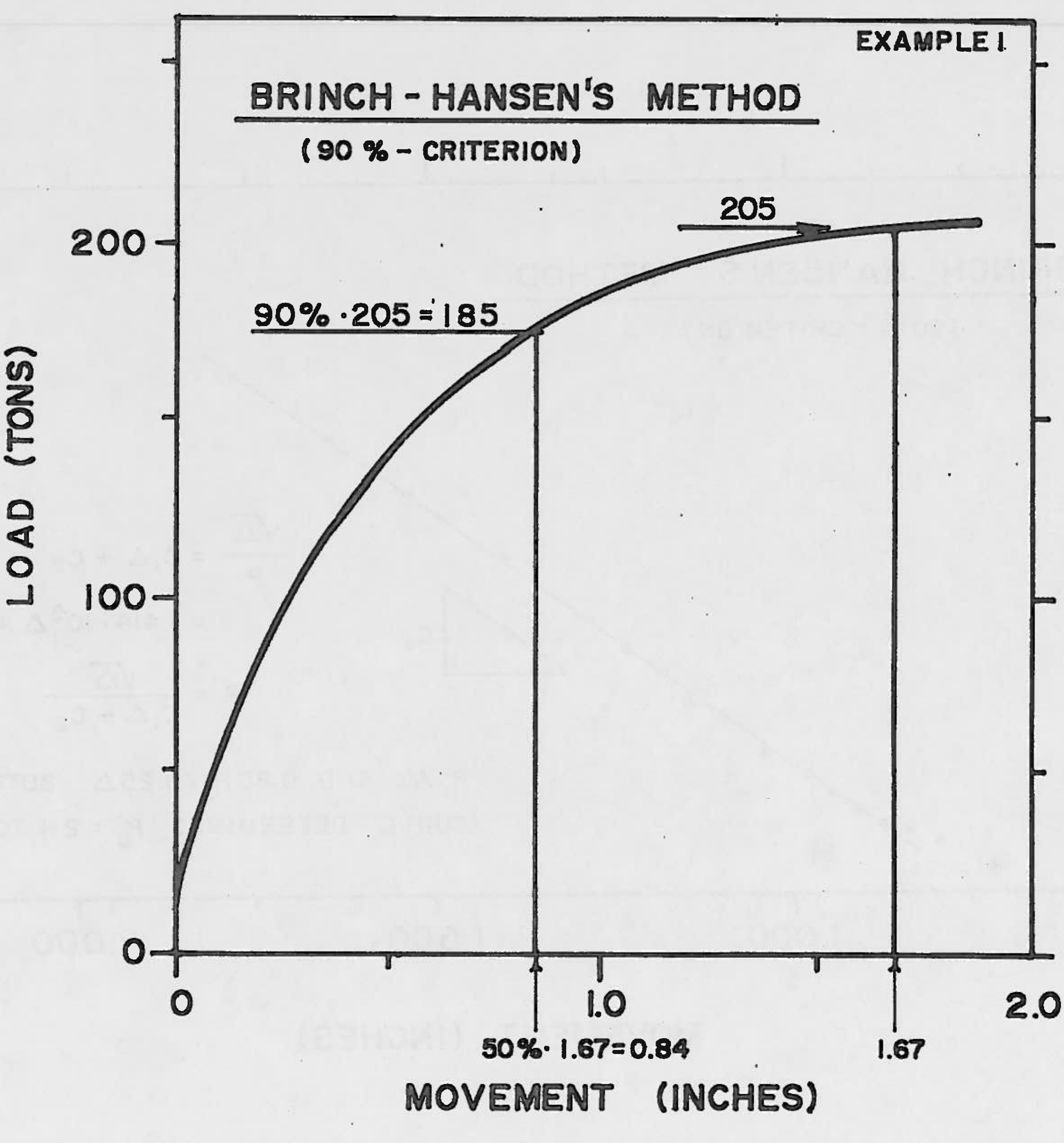
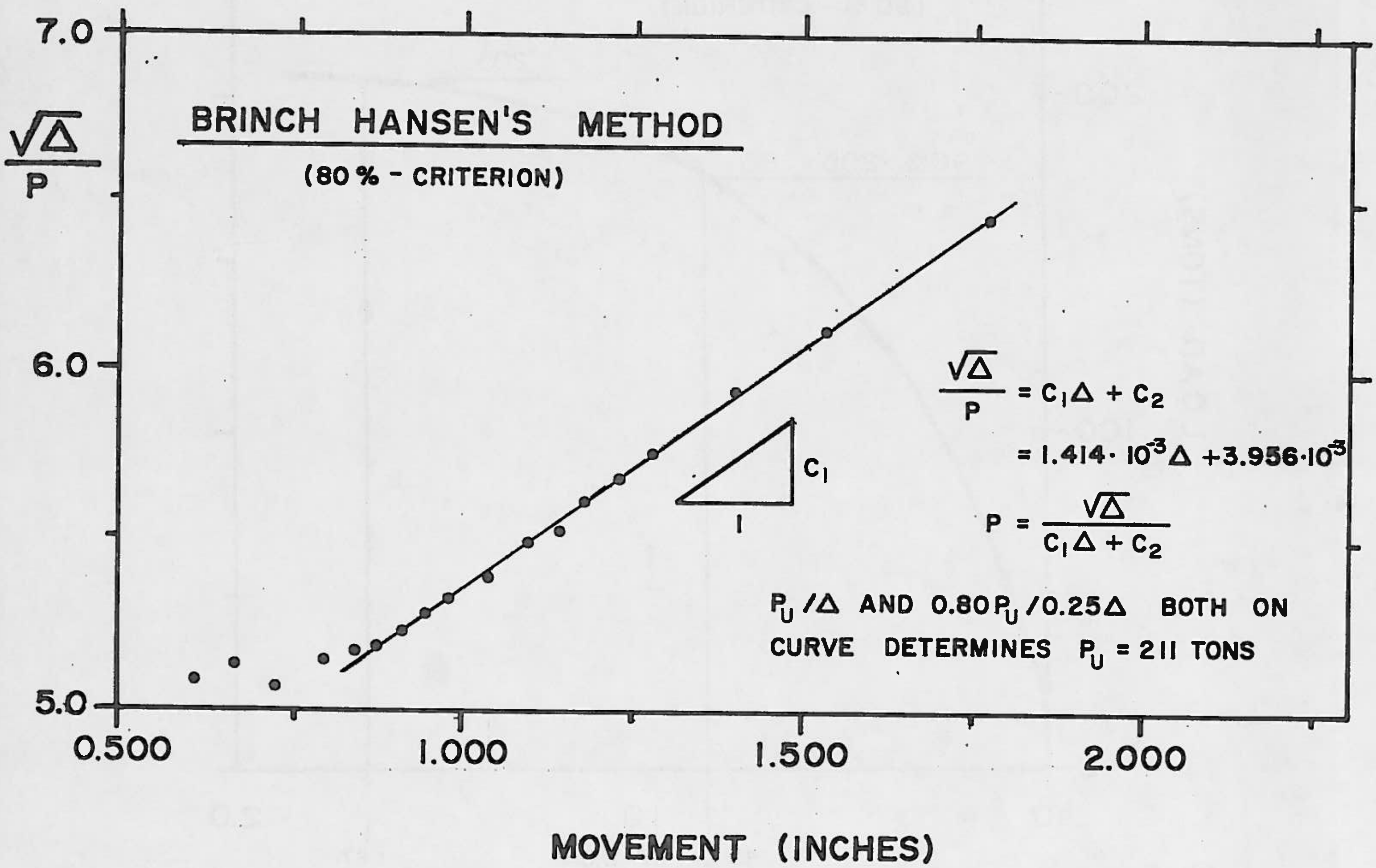


FIG. 5. ULTIMATE FAILURE ACCORDING TO THE 90%-CRITERION BY BRINCHHANSEN.



FIG, 6, ULTIMATE FAILURE ACCORDING TO THE 80%-CRITERION BY BRINCHHANSEN,

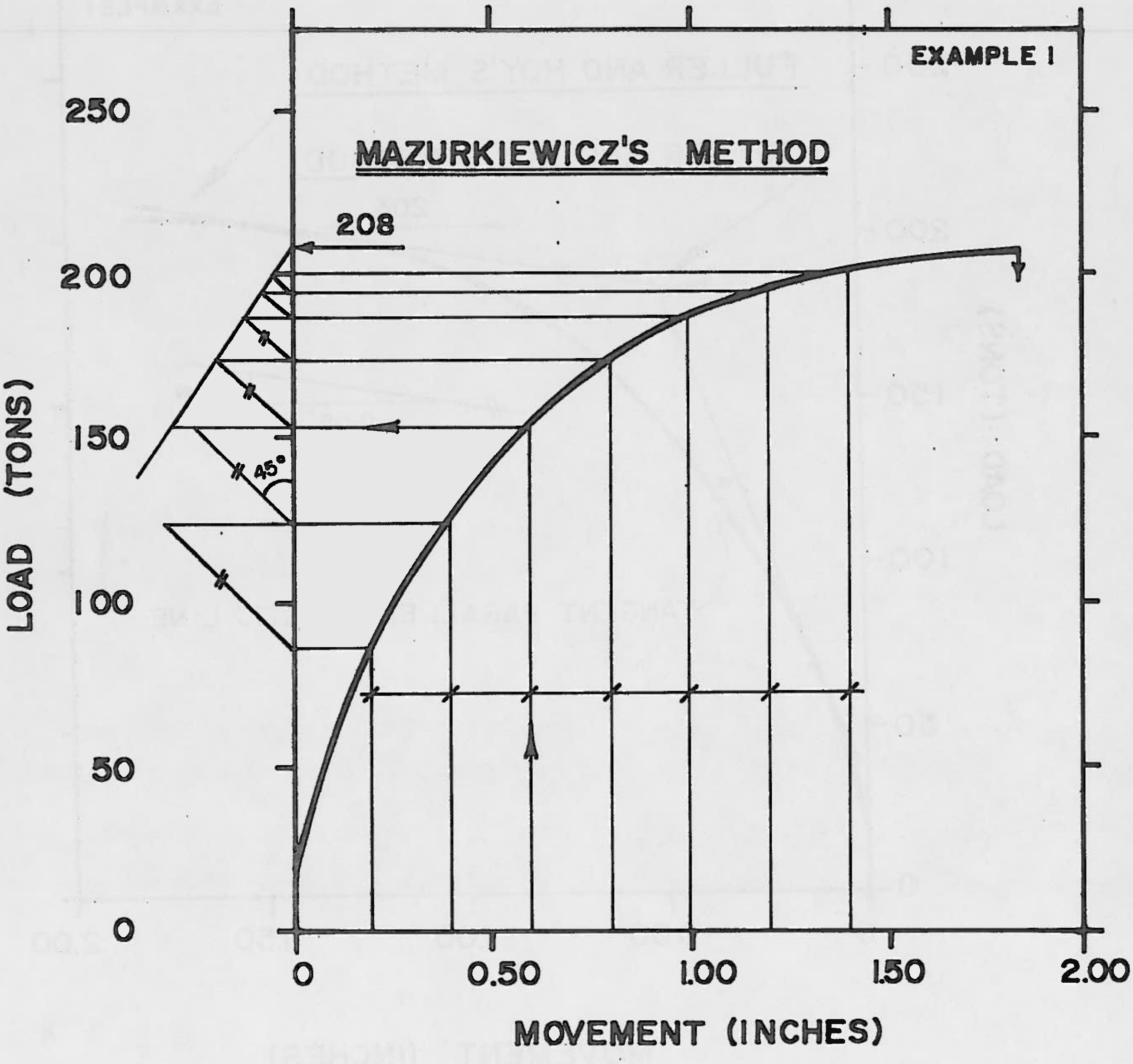


FIG. 7. ULTIMATE FAILURE LOAD ACCORDING TO MAZURKIEWICZ

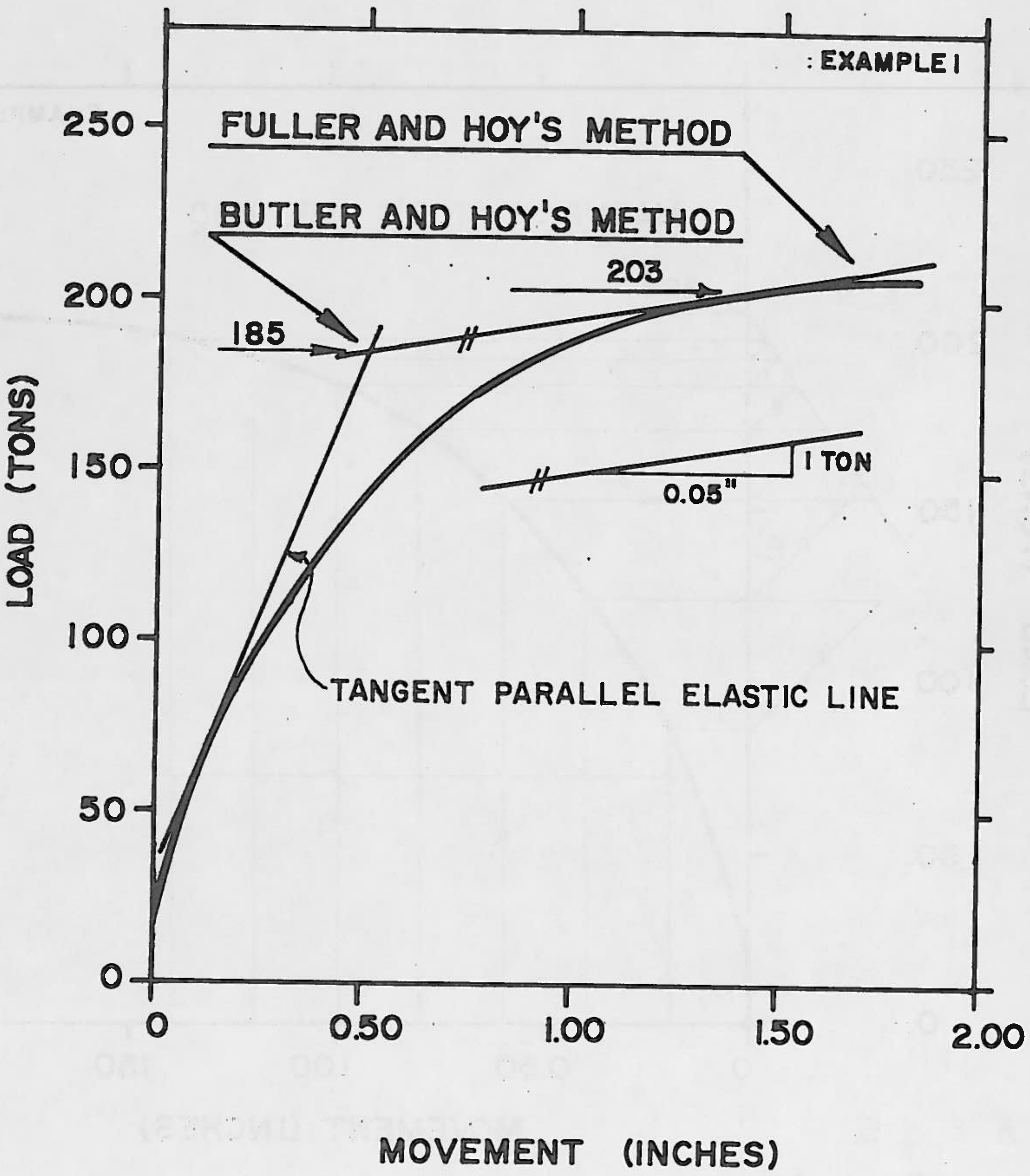


FIG. 8. ULTIMATE FAILURE ACCORDING TO FULLER-HOY AND BUTLER-HOY.

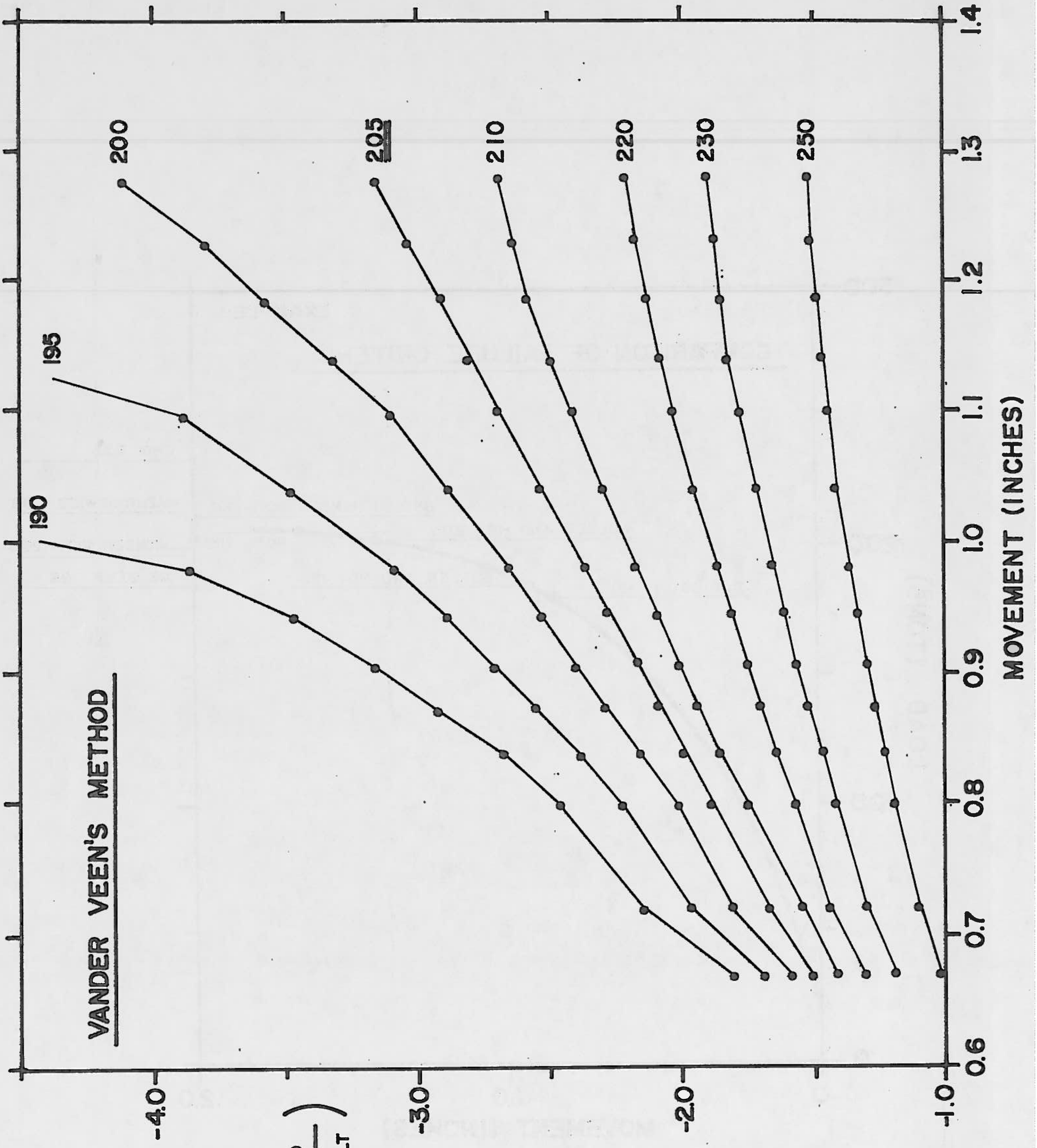


FIG. 9. ULTIMATE FAILURE ACCORDING TO VANDERVEEN.

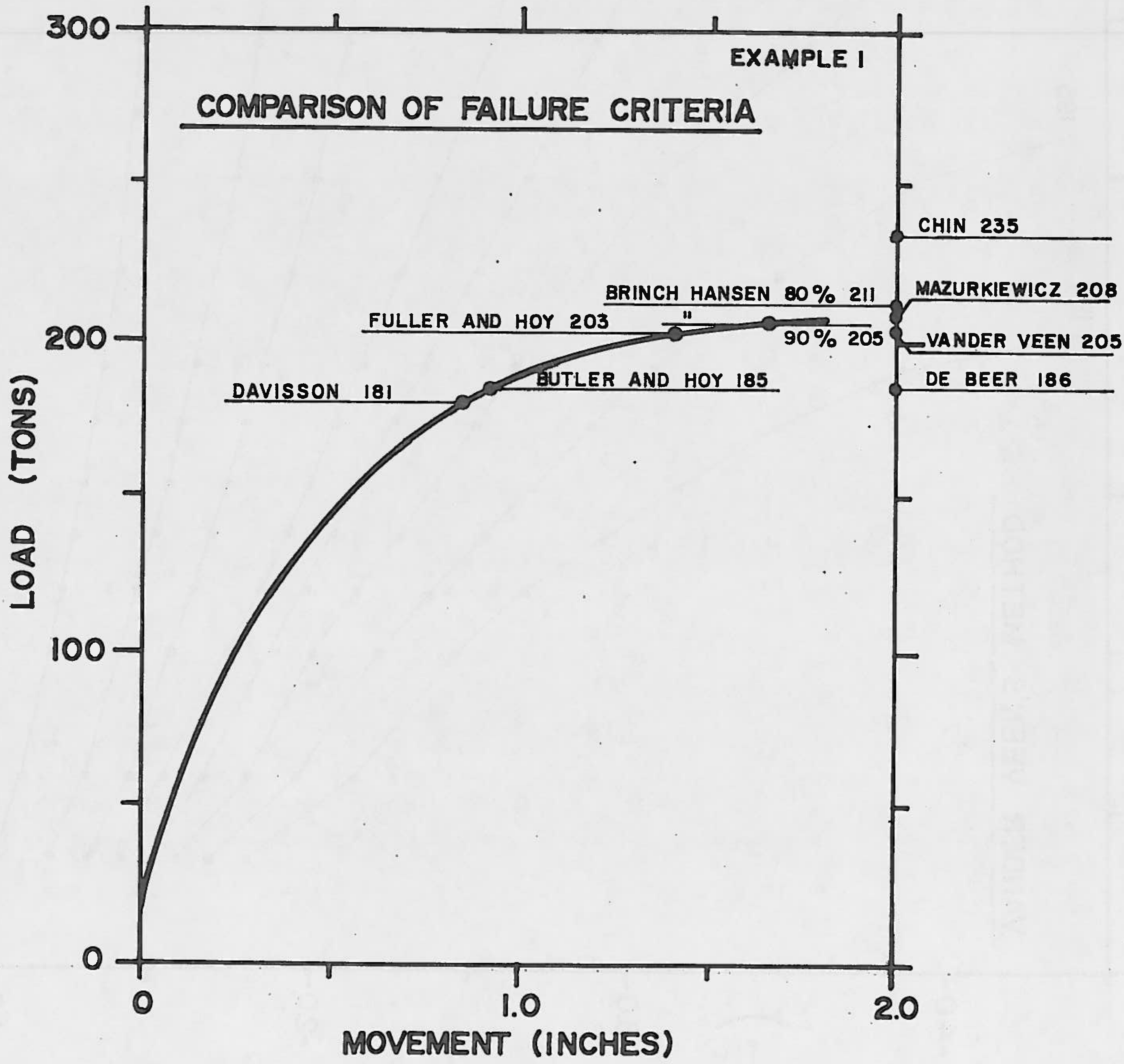


FIG. 10. COMPARISON OF NINE FAILURE CRITERIA.

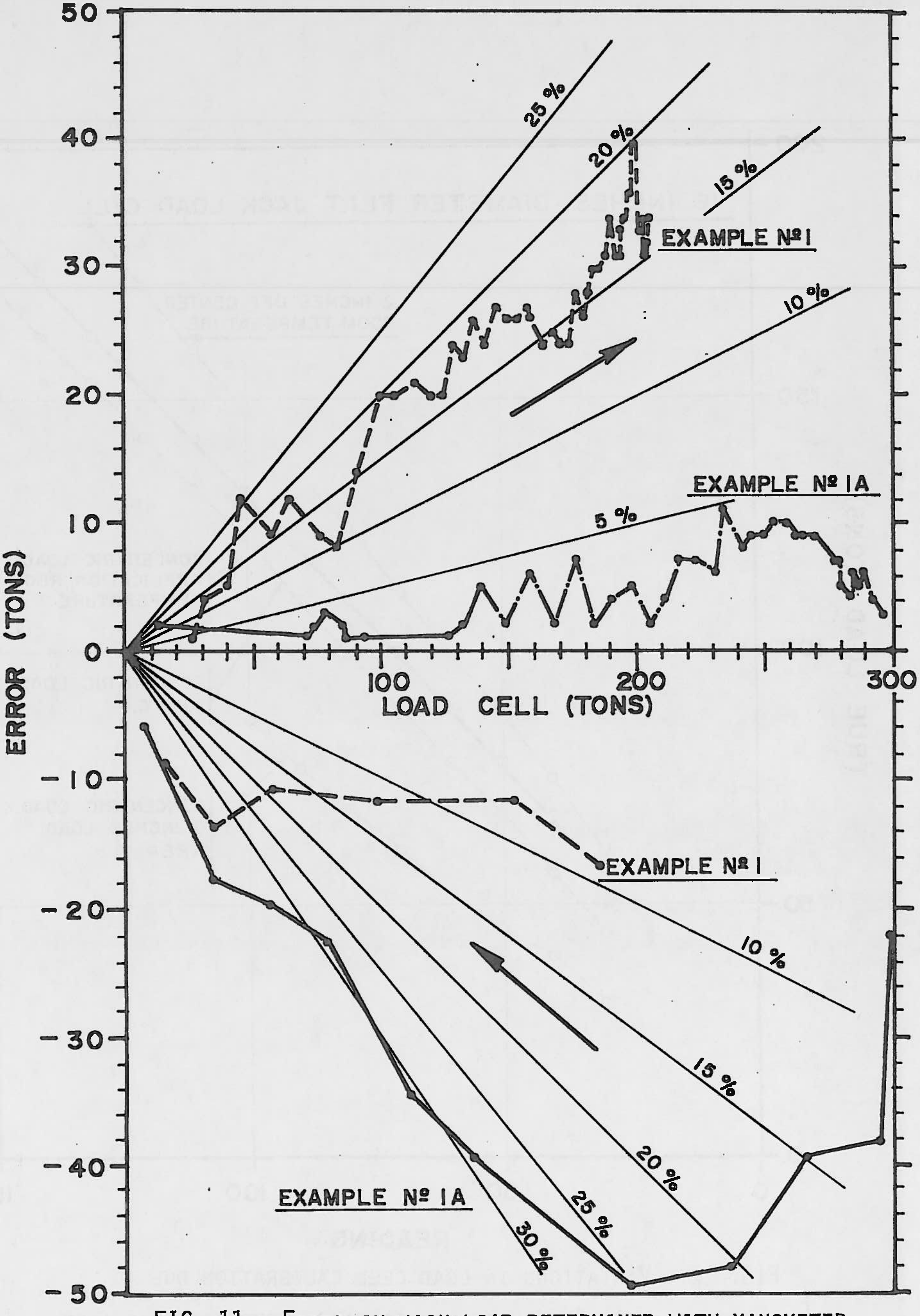


FIG. 11. ERROR IN JACK-LOAD DETERMINED WITH MANOMETER
VS CORRECT LOAD DETERMINED WITH LOAD CELL

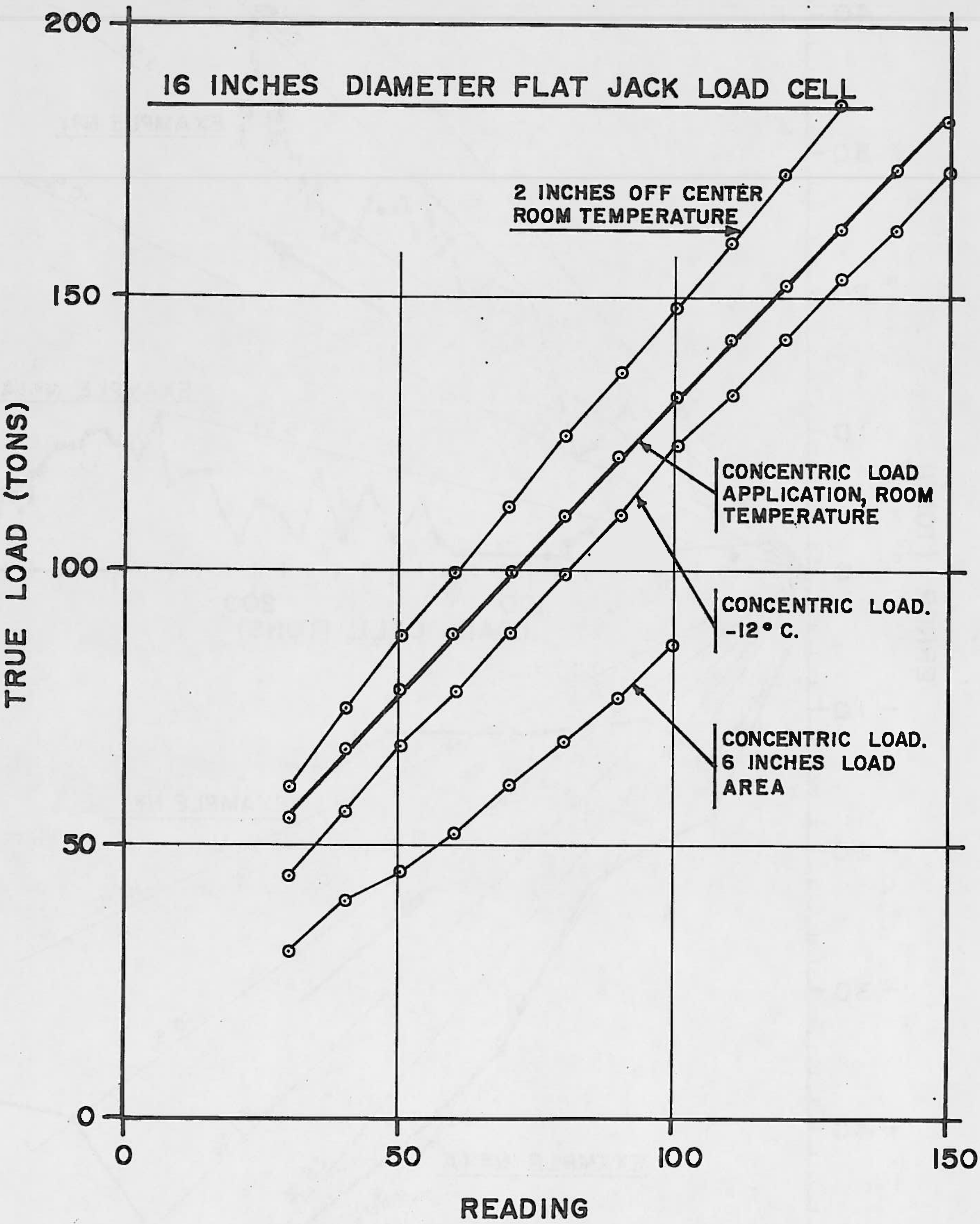


FIG. 12. VARIATIONS IN LOAD CELL CALIBRATION DUE TO EXCENTRIC LOAD APPLICATION, TEMPERATURE CHANGE, AND REDUCED LOADING AREA

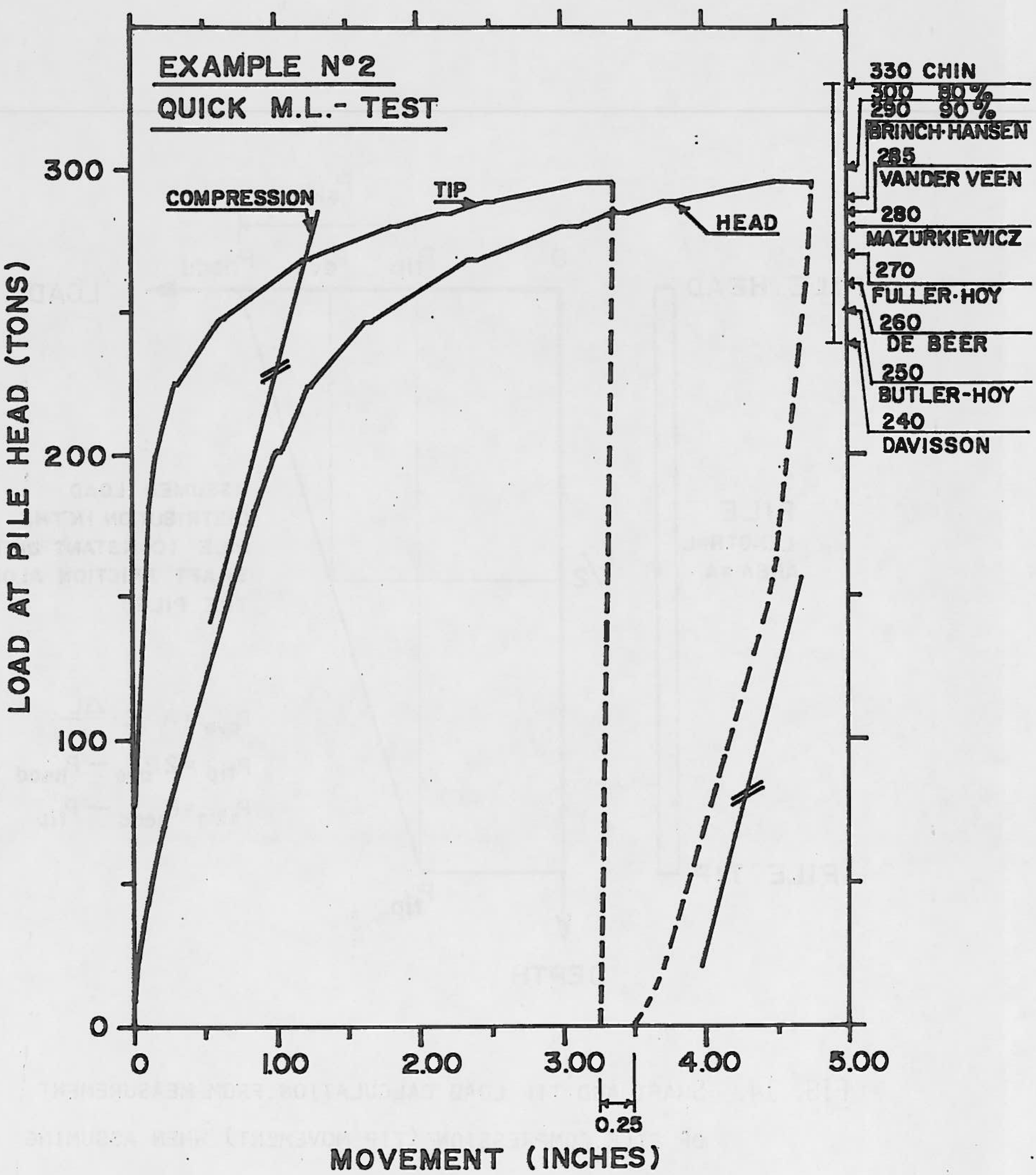


FIG. 13. LOAD-MOVEMENT DIAGRAM FROM QUICK M.L.-TEST WITH MEASUREMENT OF PILE TIP MOVEMENT

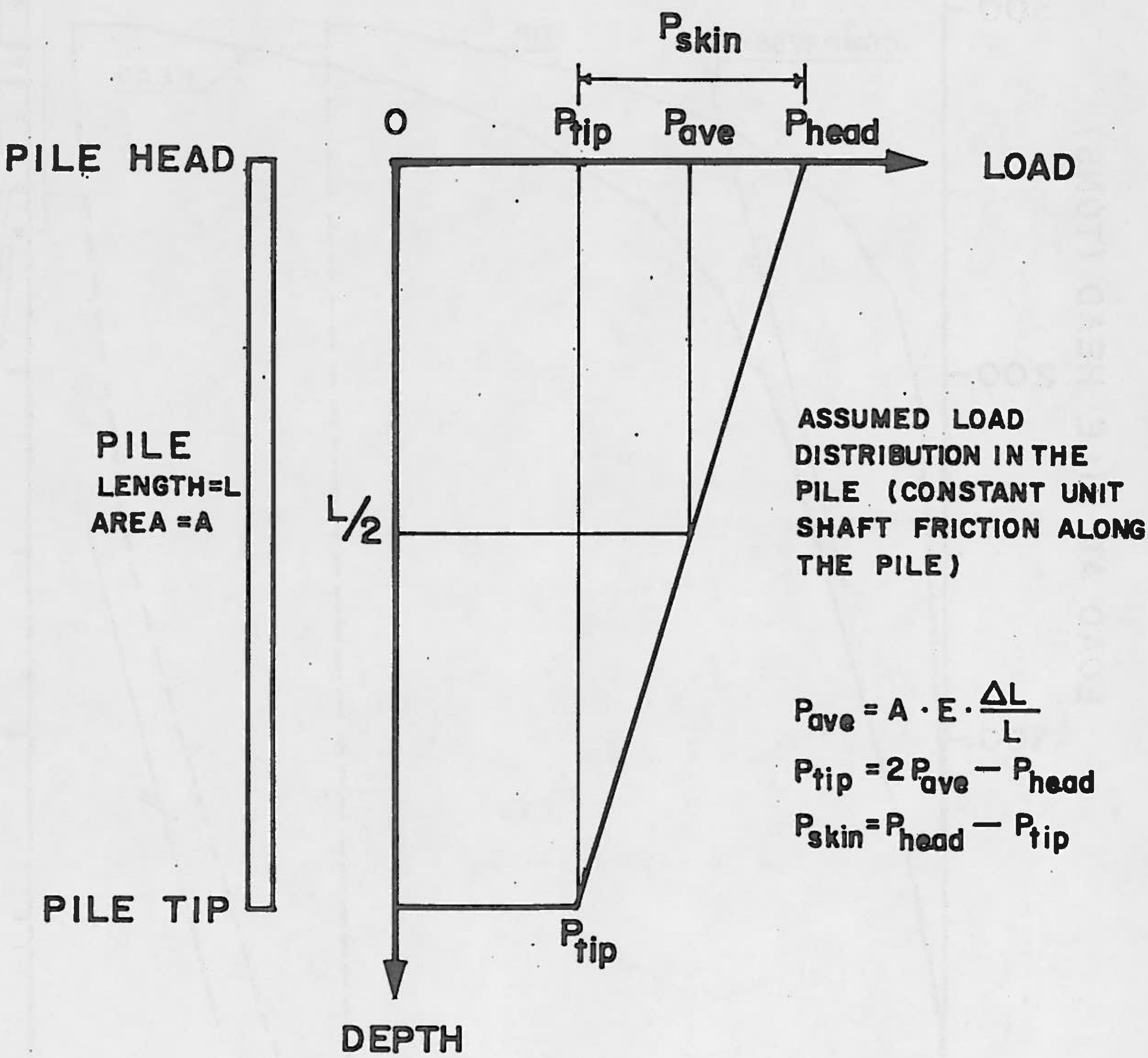


FIG. 14. SHAFT AND TIP LOAD CALCULATION FROM MEASUREMENT OF PILE COMPRESSION (TIP MOVEMENT) WHEN ASSUMING CONSTANT UNIT SHAFT FRICTION.

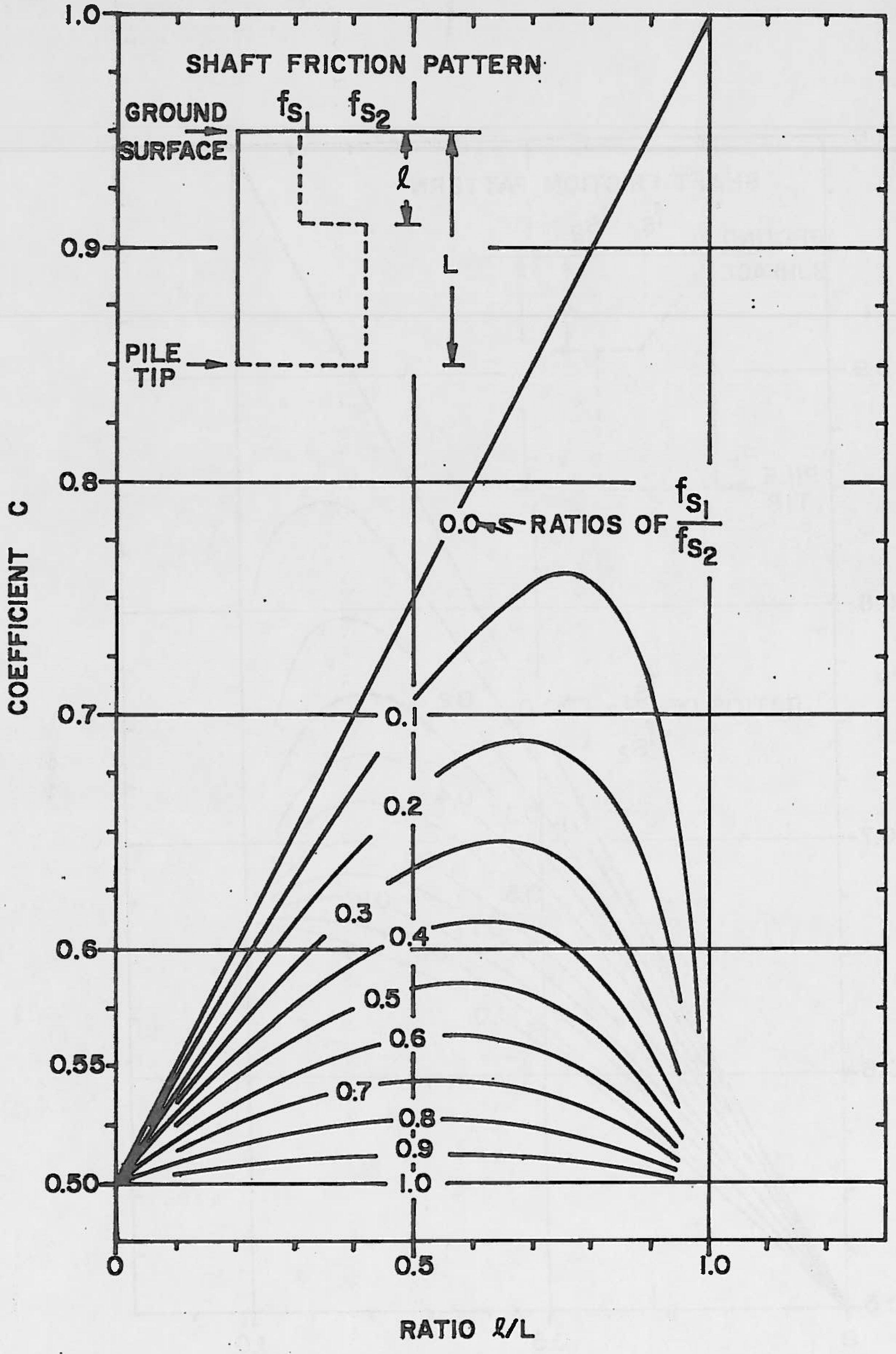


FIG. 15. COEFFICIENT C FOR VARIOUS DISTRIBUTIONS OF UNIT SHAFT FRICTION.

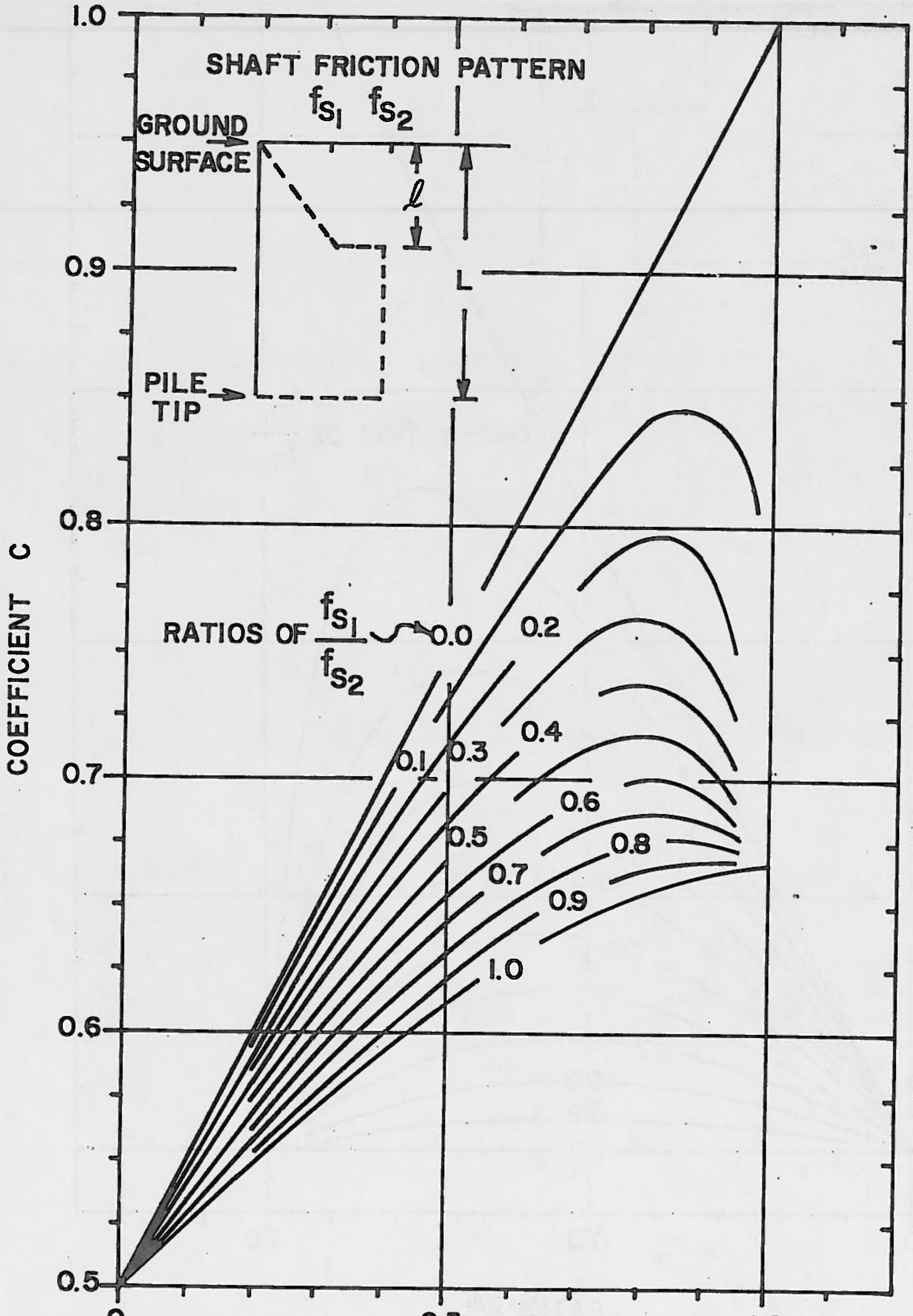


FIG. 16. COEFFICIENT C FOR VARIOUS DISTRIBUTIONS OF UNIT SHAFT FRICTION.

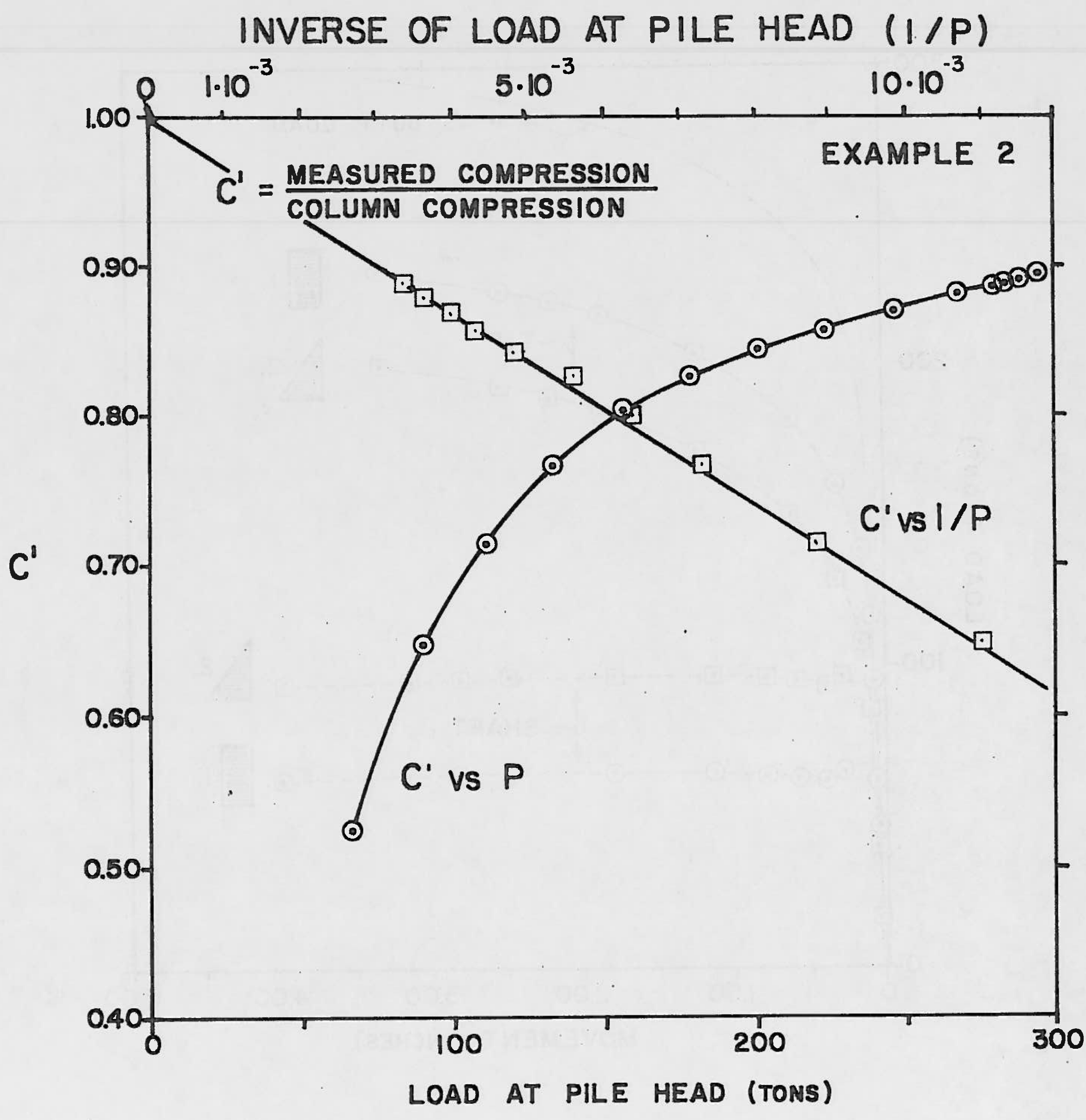


FIG. 17. COEFFICIENT C′ PLOTTED VS LOAD AT THE PILE HEAD AND VS THE INVERSE LOAD.

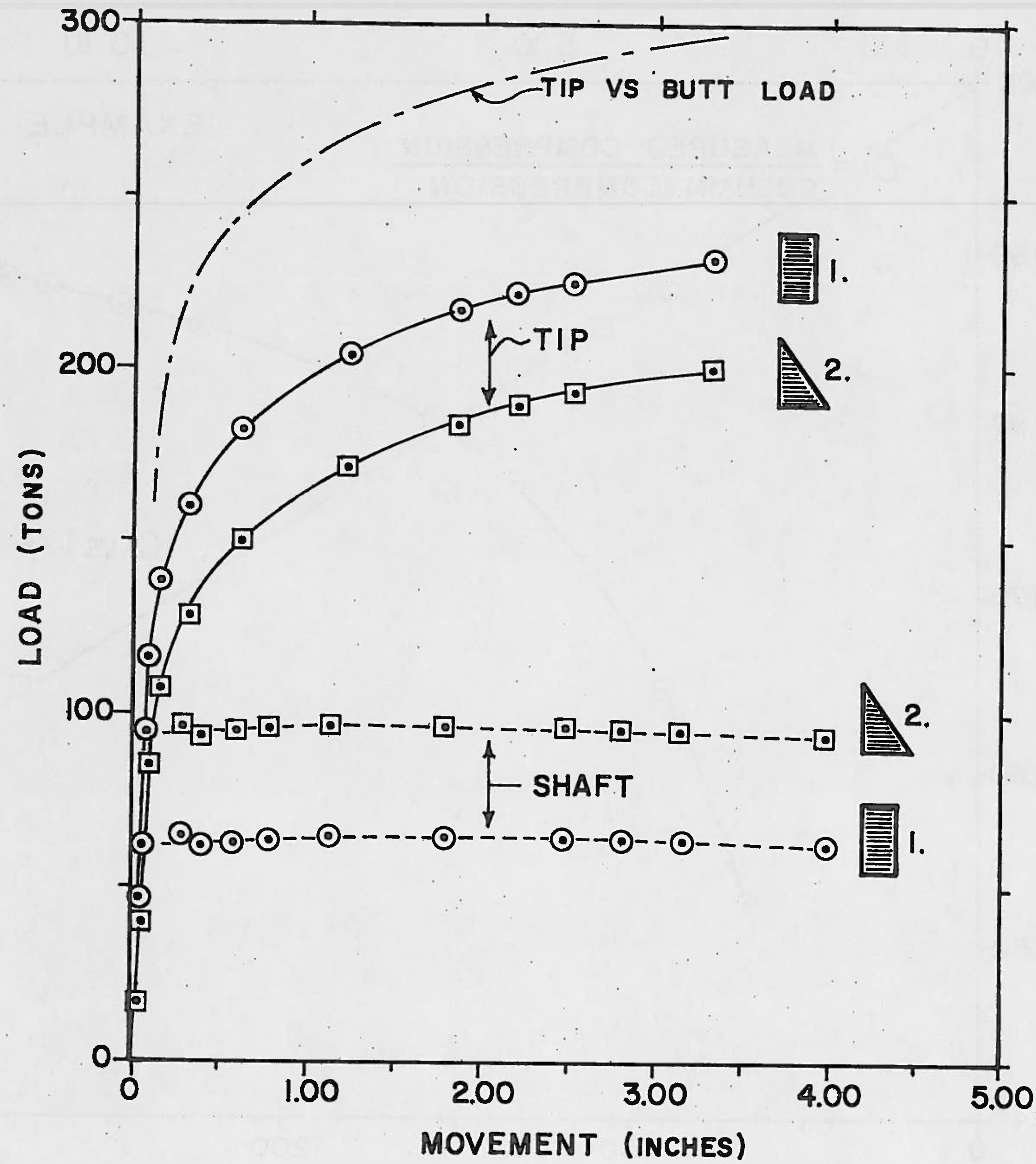


FIG. 18. LOAD-MOVEMENT DIAGRAM FOR THE SHAFT AND TIP LOADS.

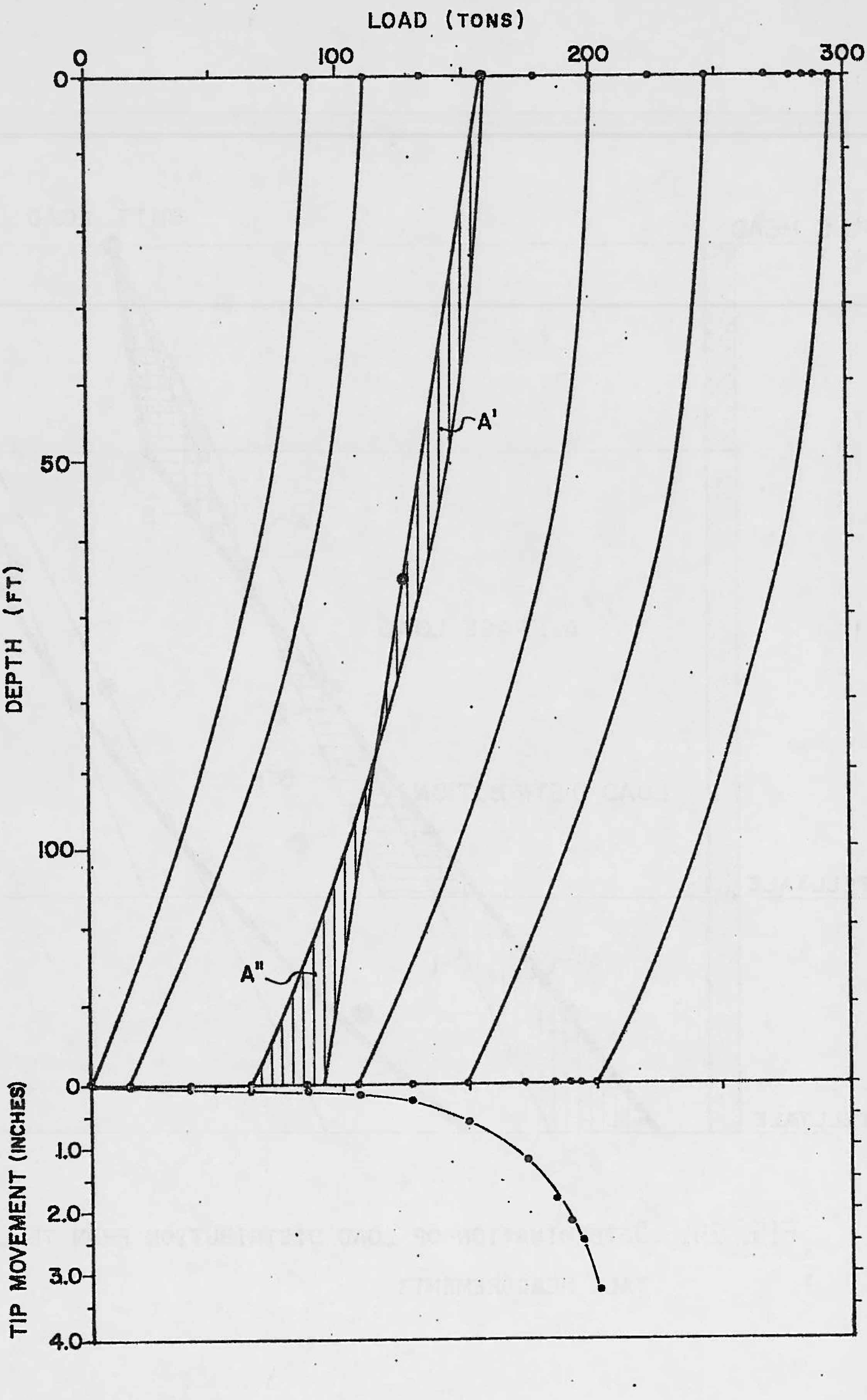


FIG. 19. LOAD DISTRIBUTIONS IN THE PILE DURING THE TEST.

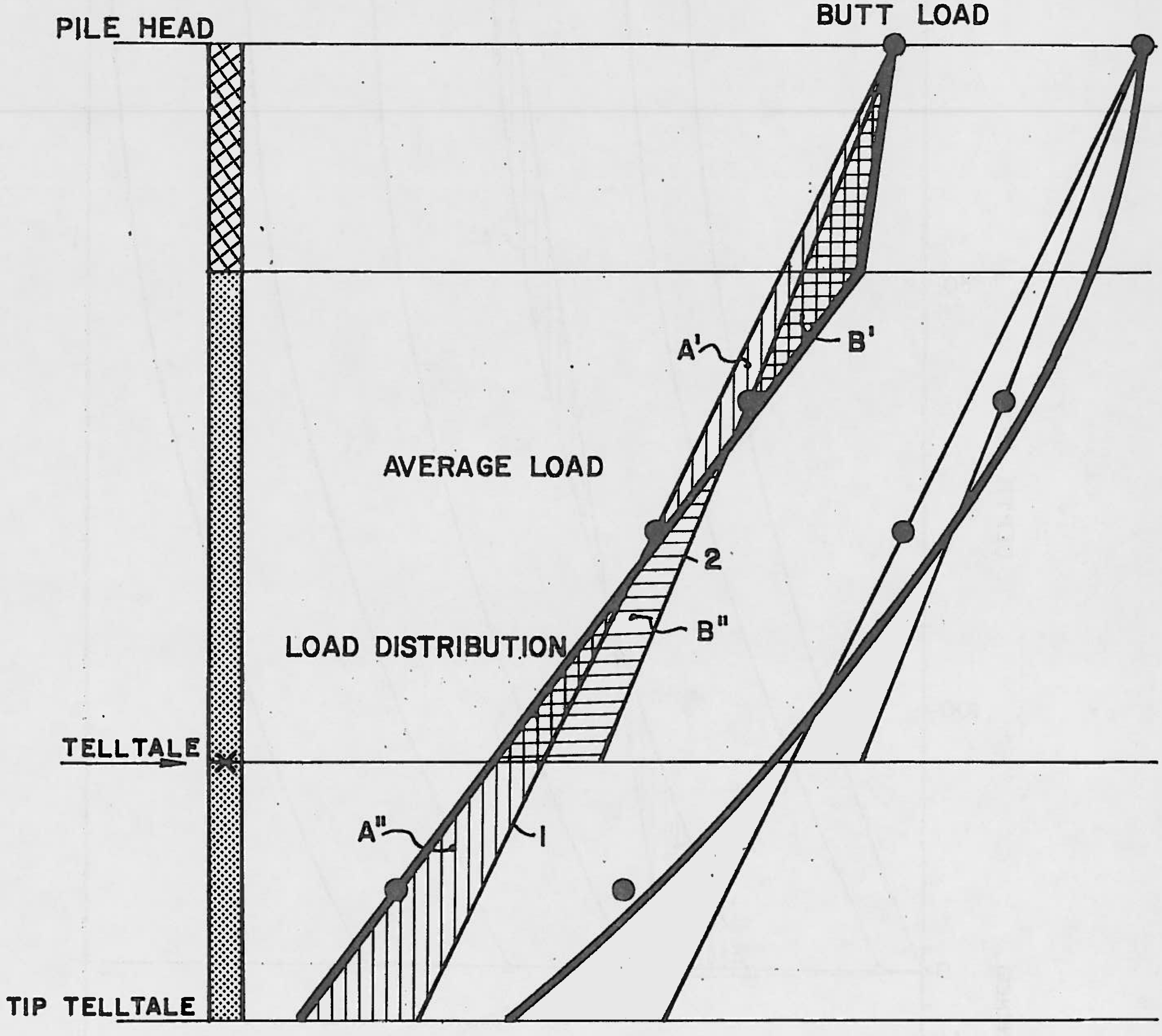


FIG. 20. DETERMINATION OF LOAD DISTRIBUTION FROM TELL-TALE MEASUREMENTS

Axial Compression and Uplift Resistance of Steel H-Piles

David R. Friels, P. E.[1]

INTRODUCTION

The Mississippi River alluvial terrace deposits in the Shelby County, Tennessee, area provide an excellent bearing stratum for a deep foundation. A number of in-situ pile load tests have been conducted to verify allowable design loads. Only limited data, however, is available on the total compression and uplift (tension) resistance of driven piles in this area.

Steel H-piles were selected to support the structure and machinery foundations for a recent industrial project in Memphis, Tennessee. Driven piles were selected because of the severe loading conditions of the equipment and the very small differential settlement permitted by the machinery manufacturer. Since it was not considered feasible to isolate the critical machinery foundations from the floor slabs and building frame, driven piles were selected for the entire project. Because of the load conditions and maximum spacing requirements, the development of high pile capacities was not considered of economic importance. HP 10 x 42 piles with cast steel driving shoes were specified at a tip depth of about 45 feet (14 m) below the existing surface. The piles were driven with a Vulcan I hammer which had a rated energy of 15,000 ft-lbs (20,300 J).

The project specifications required two load tests with a test load of at least twice the design load of 40 tons (356 kN). The piling contractor proposed a series of load tests on piles driven to different depths which would include loading the piles to failure and then pulling them to obtain uplift resistance data. The

[1]Director of Engineering and Testing, TEST, Inc., Memphis, Tennessee

pile cap for the largest piece of machinery was selected for the load test location.

TEST PROGRAM

The reaction piles were numbered 45 and 50 and were used to anchor the reaction beam which was 18 ft (5.5 m) long. The load test piles were numbered 46 through 49. The reaction and test pile arrangement is depicted in Figure 1. Pile 49 had a tip depth of 27 ft (8.2 m) which was near the contact of the silt and the underlying silty sand. Pile 46 was driven to a tip depth of 39.5 ft (12 m) which was bearing in an isolated sandy clayey silt layer. Piles 47 and 48 had tip depths of 55.2 ft (16.8 m) and 45.5 ft (13.9 m) respectively and were bearing in the silty sand deposit. A summary of the test data from boring B-1 which was located within about 10 feet of the load test and the penetration resistance for the two reaction piles are included on Figure 2. The driving resistance of the other piles was similar to that of the two reaction piles.

The testing procedures generally conformed to the optional quick load test described in ASTM D1143-74. The test load was applied with a hydraulic jack positioned between the pile and reaction beam. The uplift force was applied by jacking against the reaction beam, with the beam free at one anchor pile but connected to the test pile and the other anchor pile with a hinged connection (see force diagram on Figure 4). The vertical load on the test pile was determined by summing moments about the anchor pile.

TEST RESULTS

The compression load-settlement curves for piles 47 and 48 are presented on Figure 3, and the uplift resistance versus deflection curves are included on Figure 4. The curves for other piles were similar.

Pile failure was to be defined by a plunging load; however, because of yielding of the tie-down piles this was not achieved for all tests. Where actual plunging did not occur, an increase in gross settlement disproportionate to the load increase was used to select the failure load. By the above criteria piles 45 & 50 did not fail during the uplift loading.

The test results are summarized in the following table:

Pile No.	Tip Depth (Feet)	Driving Resistance Last Foot (Blows/Ft)	Maximum Axial Compression Load (Tons)	Maximum Uplift Resistance (Tons)	Uplift Capacity (Percent of Compression Capacity)
45	59.6	13	Reaction Pile	50	---
46	39.5	9	37.5	23	61.3
47	55.2	19	102.5	68	66.3
48	45.5	20	90.0	52	57.8
49	27.0	6	27.0	18	66.7
50	59.0	18	Reaction Pile	54	---

(1 ft = 0.305 m; 1 Blow/ft = 1 Blow/0.3 m; 1 Ton = 8.9 kN)

ANALYSIS OF DATA

The percentage of uplift capacity to total compression load ranged from 57.8 to 66.7 with an average of about 63 percent. Using the net perimeter of 3.33 ft (1 m), an uplift resistance of 18 tons (160 kN), and the tip depth of 27 ft (8.2 m) an average shaft friction of about 400 psf (19 kN/m^2) was developed in the upper fine-grained silt layer by Pile 48. Further, assuming 18 tons (160 kN) of the total uplift resistance was developed in the upper 27 ft (8.2 m) of the five longer piles, an average shaft friction value of 1000 psf (48 kN/m^2) was developed for tension loads in the silty sand layers. The average value indicated for piles 47 and 48 in the silty sand layers was 1100 psf (53 kN/m^2).

If an assumption regarding the relationship of shaft friction for piles in tension and piles in compression is made, approximations for the load carried by the tip can be obtained. Published information indicates that for sand deposits the shaft friction for tension loads could range from about 50 percent to 100 percent of the value developed in compression. If the shaft friction developed in compression is assumed equal to the friction value developed in tension, the load carried by the tip would range from 9 tons (80 kN) for pile 49 to 38 tons (338 kN) for pile 48. The percentage of load carried by the tip would range from 33.3 to 42.2 with an average of about 37 percent. The end bearing in the silt for pile 49 would be about 13 tsf (1240 kN/m^2). The tip load for pile 48 which was 45.5 ft. (13.9 m) deep was 38 tons (338 kN); whereas, a tip load of only 34.5 tons (307 kN) is indicated for pile 47 which was 55.2 ft. (16.8 m) deep. This would indicate an average end bearing in the silty sand of about 52.2 tsf (5000 kN/m^2) for piles 47 and 48. Since the bearing soil beneath the tip of both piles had essentially the same density, it appears that the practical limit for development of tip bearing had been reached (i.e., a depth such that the point resistance does not increase significantly with additional depth). A limiting tip depth of 15 to 20 diameters in a homogeneous soil and a penetration of 10 diameters into a homogeneous bearing stratum underlying a weaker stratum have frequently been suggested.

Assuming the shaft friction developed in the silty sands for tension loads to be 75 percent of the value for compression loads would give somewhat different values for the tip load. If the shaft friction developed in the silts for compression is again assumed equal to the tension value, the average shaft friction developed in the silty sands would be on the order of 1330 psf to 1470 psf (64 to 70 kN/m^2). The tip load would range from 9 tons (80 kN) for pile 49 to 27.1 tons (241 kN) for pile 48, and the percentage of load carried by the tip would range from 17.8 to 35.4 with an average value of 29.2 percent. The average end bearing values in the silty sands for piles 47 and 48 would be about 32.7 tsf (3130 kN/m^2).

CONCLUSIONS

1. Medium range design loads can be achieved with H-piles with relatively shallow penetrations (10 to 15 pile diameters) into the granular alluvial deposits.

2. Uplift resistances on the order of 63 percent of the maximum compression load can be developed.

3. Average Shaft friction values on the order of 400 psf (19 kN/m^2) in the fine-grained silt and 1000 psf (48 kN/m^2) in the medium dense to dense silty sand deposits can be developed for H-Piles loaded in tension.

4. The total load carried by the pile tip should not be expected to increase significantly with an increase in penetration into a homogeneous deposit after reaching a critical tip depth.

5. Average shaft friction values for compression loads in the silty sands could be as much as 1470 psf (70 kN/m^2).

6. End bearing values on the order of 32.7 tsf to 52.2 tsf (3130 to 5000 kN/m^2) can be developed for H-Piles bearing in dense to very dense silty sands. An end bearing value of about 13 tsf (1240 kN/m^2) can be developed in the silts.

ACKNOWLEDGEMENTS

The writer would like to acknowledge his appreciation to the client, Southern Tim Compress Company, and the client's Consulting Engineer, Pickering, Wooten, Smith & Weiss for their cooperation and permission to use the test data. The ingenuity and efforts of the piling contractor, L. V. Tvedt Construction Company, in developing the load test set-up are also greatly appreciated.

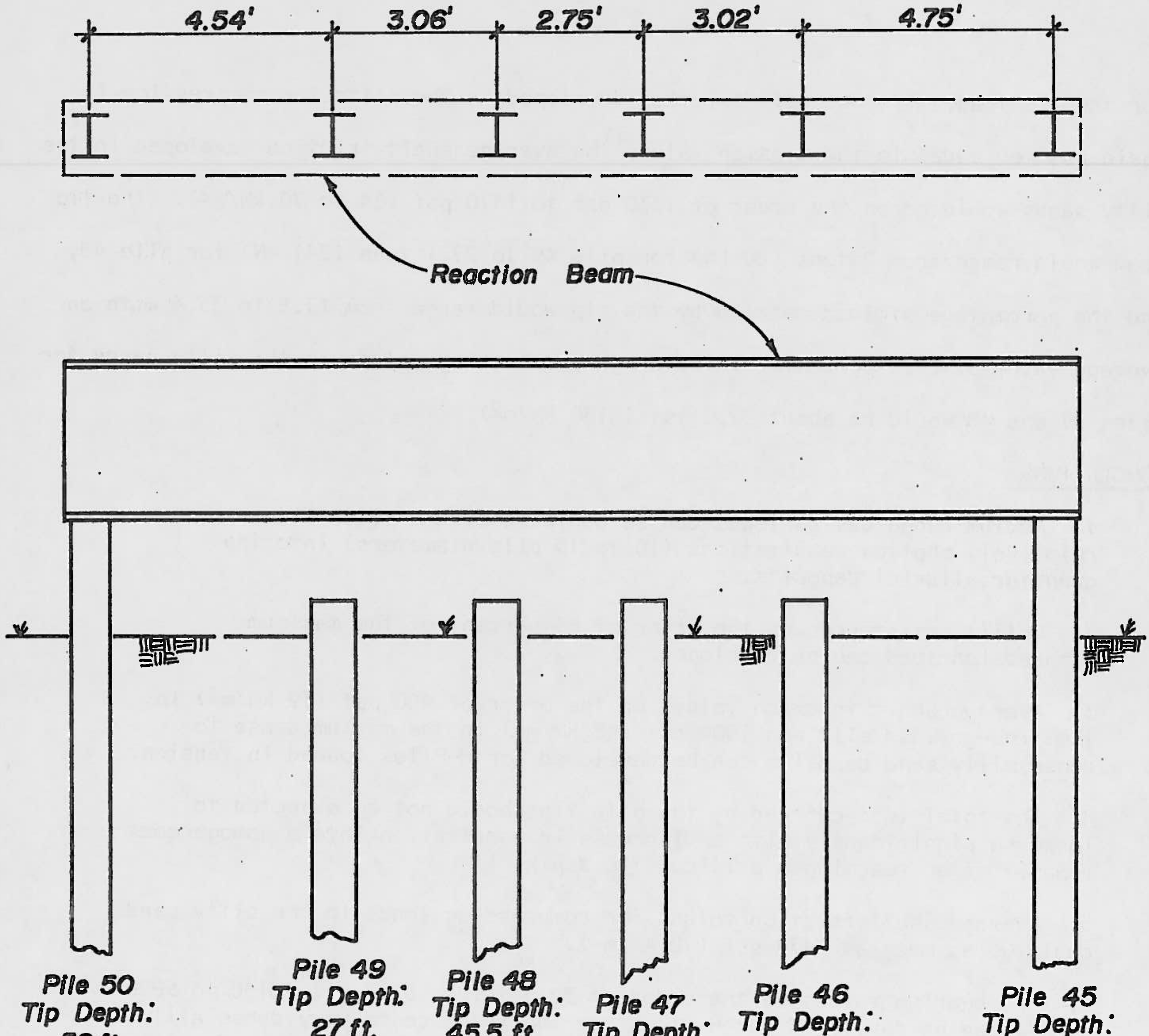


FIGURE I — DIAGRAM OF TEST PILE ARRANGEMENT

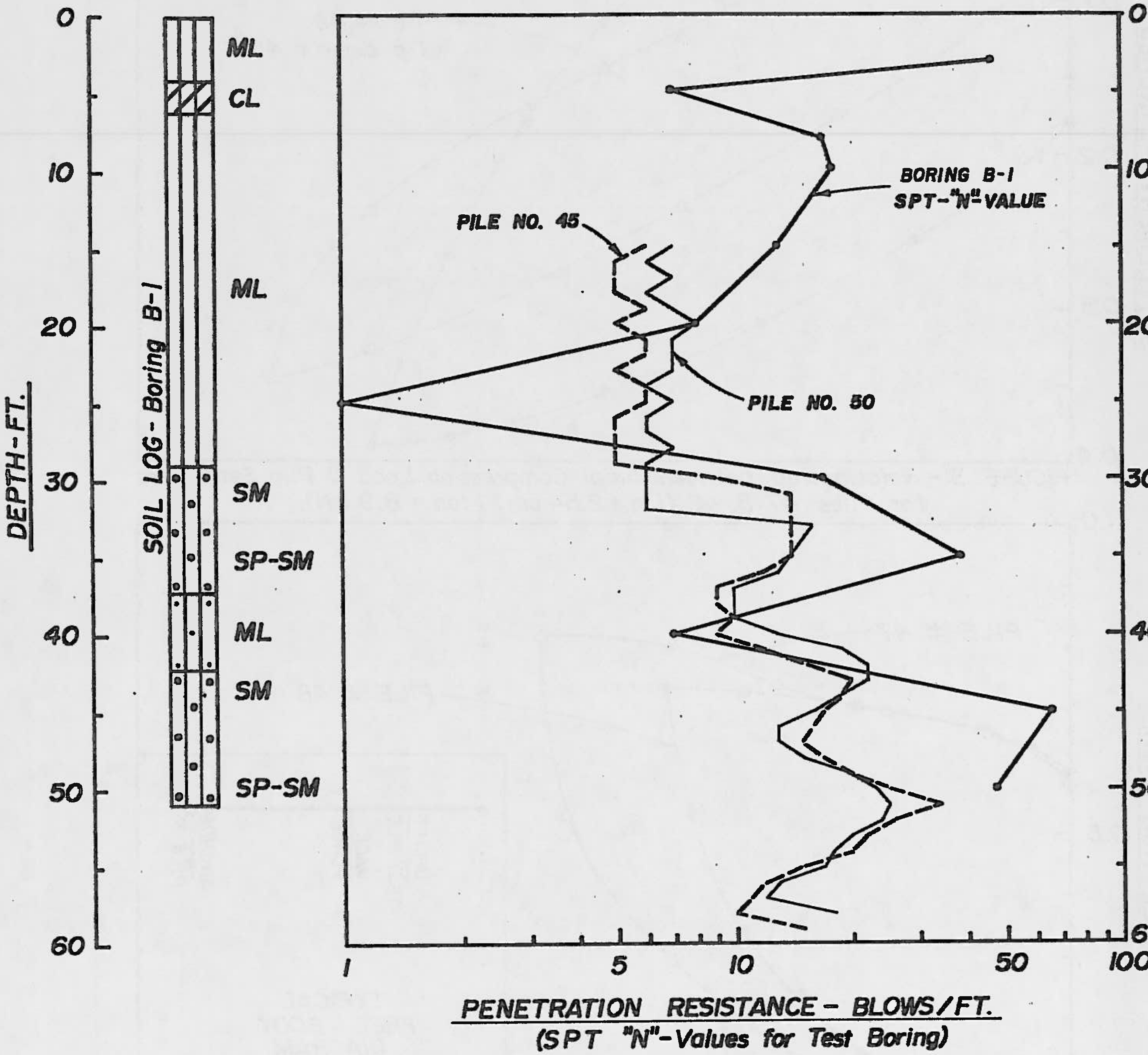


FIGURE 2 - Summary of Test Data; Soil Classification, Standard Penetration Test "N"-Values/12 in., Test Pile Driving Resistance - Blows/ft. Vs. Depth. (1 ft. = 0.305 m, 1 Blow/ft. = 1 Blow/0.3 m).

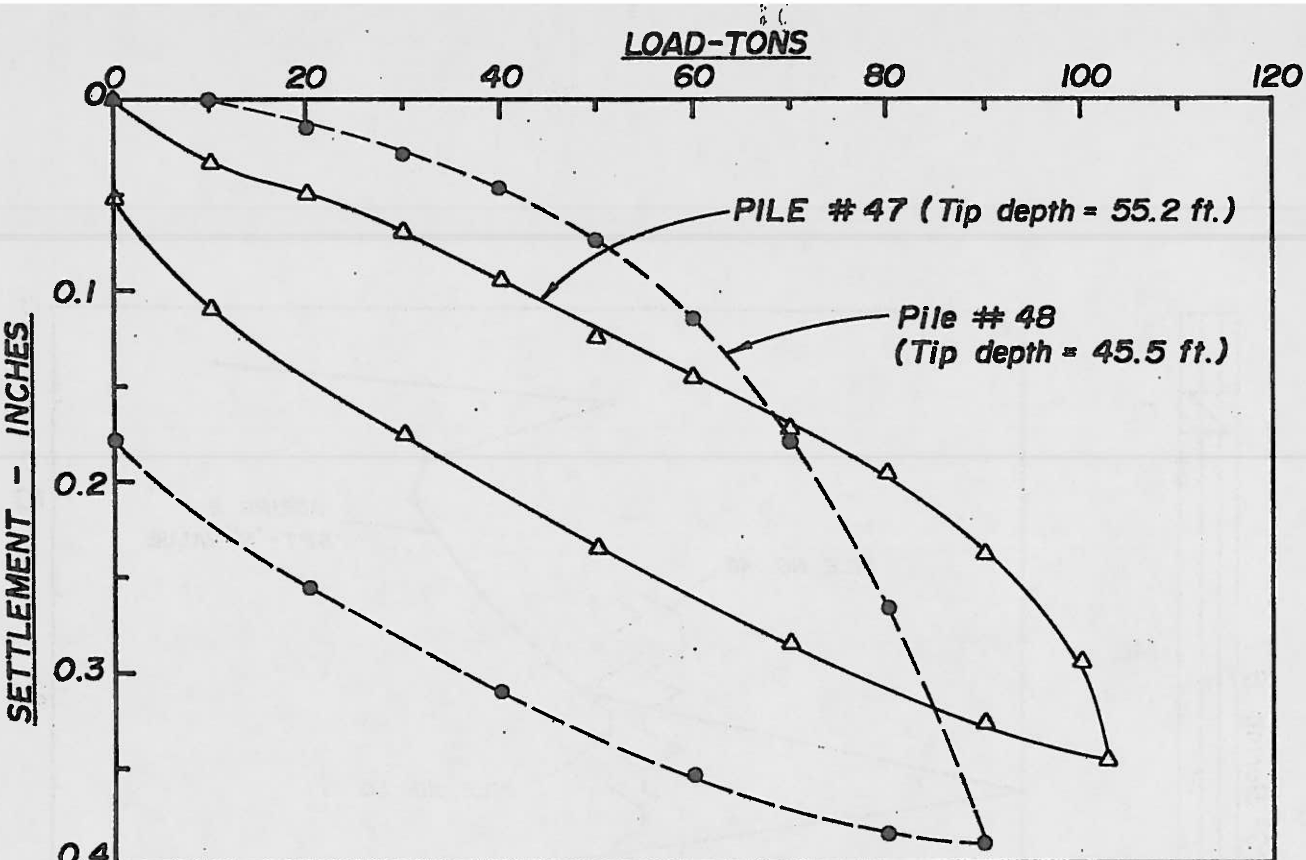


FIGURE 3 - Relationship between Axial Compression Load & Pile Settlement for Piles 47 & 48. (1 in. = 2.54 cm, 1 ton = 8.9 kN).

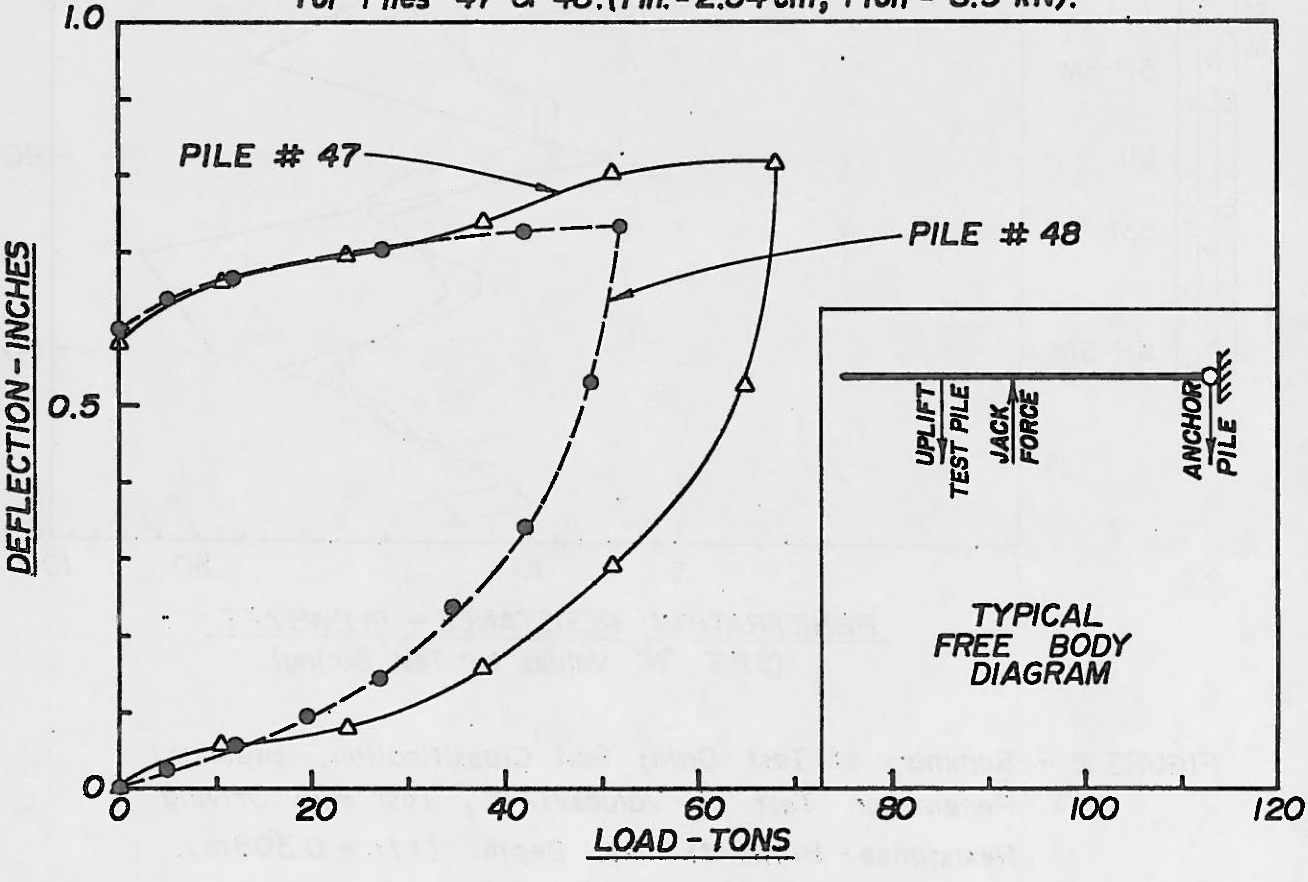


FIGURE 4 - Relationship between uplift force and Pile deflection for Piles 47 & 48. (1 in. = 2.54 cm, 1 ton = 8.9 kN).

LOAD TRANSFER MEASUREMENTS IN CONCRETE PILES

William L. Durbin(1), P. Denny Retter(2)
Pauletta France(3)

INTRODUCTION

In the design of pile foundations, it is common practice to perform load tests to evaluate capacity and develop driving criteria for the installation of production piles. The additional information on pile behavior obtained by determining the relative components of side friction and end bearing is often useful, and sometimes necessary, for best design. This is particularly true in cases where piles are driven through layered profiles and the elevation of the pile load test is different from the elevation at which the production piles will be driven. Kezdi (1975) describes methods for making load transfer analyses.

The Kansas City office of Woodward-Clyde Consultants first began using strain gauges to determine load transfer in concrete piles in 1977 as part of studies made for the L. G. Barcus & Sons Construction Company.

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(1) Associate, Woodward-Clyde Consultants, Kansas City

(2) Project Engineer, Woodward-Clyde Consultants, Kansas City

(3) Senior Staff Engineer, Woodward-Clyde Consultants, Kansas City

This contractor had just acquired franchise rights to install Interpile[1] foundations in the Central United States and was interested in having load tests made in different soil profiles to provide documentation as to the load-carrying capacity of this pile type. Other special tests to determine effects of pile driving on previously driven piles, uplift capacity, and lateral load capacity were also made but are not described in this paper.

Initially, a series of conventional load tests with telltales installed to the pile tip were carried out on the contractor's property, located in the flood plain of the Missouri River in Kansas City, Kansas. A more detailed test program was then undertaken at the site of a new power station in Fountain, Colorado. The soil profile consisted of a layered sand and clay profile overlying a very dense gravel and/or shale layer present at depths of 30 to 40 ft. To better evaluate the load transfer it was decided to try concrete embedment strain gauges along with telltales. If successful, it was felt this information would also be helpful in evaluating the uplift capacity of the pile.

Both test piles at Fountain failed prematurely because of inferior concrete. In this particular situation the load-transfer data were

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(1) The pile cavity is formed by driving an expendable over-sized drive-shoe using a perforated heavy-wall steel pipe mandrel. Grout consistency concrete is placed as the pipe is driven by driving through an open-bottom grout trough. Grout enters the mandrel through periodic perforations and fills the annular space formed between the drive-shoe and pipe mandrel. Hammer energy is reflected by soil resistance at the pile shoe.

particularly beneficial since they demonstrated that only a small percentage of applied load reached the pile tip prior to the structural failure. Analysis of observations supported the conclusion that with sound concrete these type piles would have carried several times the design load. Largely on the basis of this successful program, the designers allowed the Interpile as a bid alternate. (Barcus proved the successful contractor and has now finished placing production piles at this site.)

Subsequently, strain gauges have been used (1) to compare Interpiles and auger cast piles at Hawthorne Station in Kansas City, Missouri, (2) to evaluate Interpile tests at the Phillips refinery in Kansas City, Kansas, (3) to evaluate pre-cast concrete piles driven to end-bearing in Omaha, Nebraska, and (4) to evaluate friction Interpiles at Nearman Station in Kansas City, Kansas.

METHODS OF DETERMINING LOAD TRANSFER

The most common method of determining load transfer in concrete piles is by means of telltales installed to the pile tip or to intermediate depths in the piles. Telltales function as displacement gauges and by averaging the displacements between the telltale and the butt measurements (or between successive telltales) the average strain over the corresponding length of pile can be calculated. In reducing data obtained from telltale measurements, it is normally assumed that the load transfer is linear; however, other load-transfer functions could also be assumed. Telltales are subject to several sources of measurement errors. Our experience with telltales in cast-in-place piles was not entirely satisfactory; therefore, we decided to look for other ways of instrumenting the piles.

Load cells or stress meters could also be used to determine the average stress in a pile at a given depth. The writers have never tried this approach but our experience with stress gauges on other types of installations has been discouraging. Glotzel stress meters were used in concrete filled pipe piles for the Alaska pipeline and we understand the results were not satisfactory. Holtz and Baker (1972) have described a case history in which the side friction of a drilled shaft was determined by placing a compressible material below the tip so that all of the load had to be carried in side friction. This approach was used in conjunction with telltales to determine the load transfer as a function of depth.

Concrete embedment strain gauges can also be used to determine load transfer by assuming that the strain across the pile section at the location of the strain gauge is uniform and that the stress in the pile at that location is proportional to the strain. The constant of proportionality is, of course, the concrete modulus. The applicable equations are as follows:

$$P_z = \sigma_z A \text{ and}$$

$$\sigma_z = E\epsilon_z, \text{ therefore}$$

$$P_z = E\epsilon_z A$$

where: P_z = Load in pile at depth z

σ_z = Stress in pile at depth z

ϵ_z = Strain in pile at depth z

A = Pile cross sectional area

E = Modulus of elasticity of pile

In each case, the concrete modulus of a 6 inch by 12 inch cylinder cast from the batch of grout or concrete used in the test pile is obtained by determining the stress-strain curve in the laboratory. The lab tests are generally made at the same time as the pile load tests. The correlation between strength and modulus obtained to date for grout is given in Figure 1.

The use of strain gauges allows determination of the load carried by the pile at multiple depths and appears to be a cost effective approach to this problem. It is especially useful in layered clay and sand profiles. The strain meters have an advantage over telltales in that many gauges can be installed in a pile and the gauge length can be as small or large as desired. On the other hand, telltales do give a direct measurement of the tip settlement, which strain gauges do not provide. It is possible to calculate the tip movement by integrating the strains and subtracting the results from the butt displacements. This was done for the test piles at Fountain, Colorado. For the range of applied load (175 tons), the strain depth relationship was approximately linear even though the soil profile was quite layered. The tip displacement calculated from the strain gauges was 0.185 inch at 175 tons as compared to a displacement of 0.103 inch measured on the telltale. (The telltale measurements made no sense at all on this project.)

DESCRIPTION OF STRAIN GAUGE APPROACH

Instrumentation

The instrumentation used in these studies consists of concrete embedment strain meters manufactured by the Carlson Instrument Company in Campbell, California. The meter is shown in Figure 2. The meter is a resistance wire strain gauge that also functions as a temperature gauge. According to the manufacturer, the meter contains two coils of highly elastic steel wire, one of which increases in length and electrical resistance when strain occurs while the other decreases. The ratio of the two resistances is independent of temperature (except for thermal expansion) and, therefore, the change in resistance ratio is a measure of strain. The total resistance is independent of strain since one coil increases the same amount as the other decreases due to a change in length of the meter. Therefore, the total resistance is a measure of temperature.

The A series meters come in lengths of 8, 10 or 20 inches. The miniature strain meter comes in lengths of 4, 8 and 10 inches. All of our experience to date has been with the 8-inch A series gauge. Its specifications are as follows:

Range* (Micro-strain)	2600
Least reading (micro-strain), max.	3.6
Least reading, temperature (°F)	0.1
Gauge length (inches)	8
Weight (lbs.)	.8

*Normally set at factory for 2/3 of range in compression; within limits, other settings may be specified.

The output from the strain gauge is monitored by a Wheatstone Bridge testing set manufactured by the James G. Biddle Company in Plymouth Meeting, Pennsylvania. It is capable of reading resistances to 0.01 ohm and resistance ratios to 0.01 percent. Strain is measured by setting the mode switch to the 2-lead position. In each case the instrument is nulled and the corresponding output read off the dial indicators. The readout device is compact, lightweight, and ruggedly designed for field application. It is not environmentally sealed and difficulty was encountered in one case when heavy rains caused the electronics to get wet and the readout began to drift. This was corrected by drying the instrument. When more than one strain meter is installed in a test pile, it is convenient to have a junction box to facilitate switching from one gauge to another. The wiring of this junction box must be compatible with that of the gauge lead wires.

Ailtech embedment strain gauges have been installed in pre-cast piles by our San Francisco office and used to measure dynamic strains during driving. The results of that study have not yet been published. We understand this gauge is an improvement over the instrumentation used in our studies.

Installation

Concrete embedment strain gauges have been installed in piles by a variety of means. These include taping the gauges to either a PVC pipe or a reinforcing rod positioned in the center of the pile. Several attempts were made to install the gauges without the use of the PVC pipe or rebar; however, we could not be certain that the gauge remained vertical and at the desired depth. The results obtained by the other methods did not appear to warrant the extra effort and uncertainty associated with this procedure.

The distance from the bottom of the pipe or rebar to the gauges is recorded and then the pipe or rebar is installed to the bottom of the pile. Each gauge is marked and a metal tag with the same number embedded on it is attached to the tail end of the leads by the manufacturer so it is always possible to determine which gauge is being monitored.

Concrete embedment strain gauges have also been installed in precast piles in a project handled by the Omaha office of WCC by suspending the gauges from reinforcing tendons with light wires. One gauge was also installed by drilling a hole in the butt of the pile and epoxying the meter in place. This was done to determine the modulus of the concrete when it was learned that the testing lab had failed to measure the modulus of the concrete test cylinder.

Calibrations

A strain meter was embedded in a concrete test cylinder and cyclically loaded and unloaded in the laboratory to determine the linearity, hysteresis, and repeatability of the meter. The stress-strain relationship for the first two of these cycles is found in Figure 3. Note that after rebounding, the strain meter returned to zero (within the accuracy of the readout device). Recently, two of these strain meters were calibrated by the WCC Oakland laboratory using the MTS loading equipment with its sophisticated electronic measurement system. The results of these tests are shown in Figures 4 and 5. These tests show substantial hysteresis and residual strain upon unloading. The range of strain in these tests was much greater than in our applications and may have been beyond the intended range of the meter.

Data Reduction

The data are obtained during the pile load tests by reading the strain meters after each load increment. Temperature measurements are made at the beginning of the test and occasionally during the test period. Our experience to date suggests that temperature changes during the load test are small and do not have a significant affect on the calculated strains. A sample data reduction form provided by the manufacturer is shown in Figure 6. Most of the computations summarized on this table are only necessary in long-term measurements in which temperature changes are significant. In the case of short duration pile load tests where temperature changes are small, the data reduction simplifies to that shown in Figure 7.

Most of the tests have involved some form of cyclic loading. Typically, the piles have been loaded to twice the design load in accordance with ASTM D1143-74, paragraph 4.2, and rebounded to zero after a twenty-four hour hold. The piles were then loaded to failure using maintained loading procedures, the CRP method, or the quick test method. We have observed that in most cases there has been a residual strain in the gauge after unloading. This residual strain has been typically equivalent to a load of about 5 to 10 tons, but as large as 41 tons in one case. The residual strain has been greatest when there has been substantial creep deformation in the pile. The magnitude of the residual strain is compared to the creep deformation and load in Figure 8. At first this was thought to be the result of changes in load transfer with time. We now believe this is a characteristic of the gauge. After correcting the calculated load by subtracting the residual strain, we found that the percent of the applied load carried by a given gauge was essentially constant over a wide range of applied load, e.g., from twice design load to failure. A typical comparison of corrected and uncorrected data are given in Figure 9 and Table 1.

TEST RESULTS

Fountain, Colorado

The subsurface profile at the Fountain, Colorado site consisted of sand and clay layers underlain by dense gravel and/or shale. Standard penetration test values ranged from 6 to 50 blows per foot and are plotted in Figure 10B.

Two 14½ inch instrumented Interpiles were tested in compression at the Fountain, Colorado site. These piles were tested in accordance with ASTM D1143-74, cyclic option. Both piles failed structurally, because of low-strength grout, before the load reached 200 tons, twice the assumed design load.

Load transfer data is summarized for these piles in Figure 10E and F. Residual strains recorded in the strain meters after the structural failure were small.

Hawthorne Station - Auger Cast Piles

The subsurface conditions at Hawthorne Station consist of 17 ft of man-made fill, clay, and silt overlying alluvial sands deposited by the Missouri River. The fill was removed prior to the load test leaving 8 ft of clay and silt overlying the sands. The standard penetration test results are plotted in Figure 11B. The SPT values range from about 10 to 50 blows/ft.

Three 12 inch diameter auger cast piles were load tested. The lengths of the piles and the positions of the strain meters are shown in Figure 11A. The strain meters were installed without tying them to a PVC pipe or steel rod. The gauges were lowered to the desired depth in the pile by a device which was then extracted. This had the advantage that we were not changing the stiffness of the pile but we could not be sure the gauges were vertical or at the desired depth.

All of the piles were loaded to twice the design load in eight increments. The load was increased when the rate of settlement was less than .01 in/hour; a 24 hour hold was not used. After rebounding from twice the design load, the piles were loaded to failure using the constant rate of penetration (CRP) method. The load settlement curves are shown in Figure 11C. The loads at different depths calculated from the strain meters are shown in Figure 11D. A residual strain was recorded on each meter and the data have been corrected by subtracting this strain from the subsequent readings.

The load transfer curves are shown in Figure 11E, F and G. The lower gauge in Pile 4 appears to have malfunctioned. The reason for malfunctioning is not known.

Hawthorne Station - Interpiles

Two 14 inch diameter Interpiles were also instrumented and tested at Hawthorne Station. The pile lengths were 36 and 20 ft. The pile driving record is plotted in Figure 12B. These piles were first loaded in accordance with the ASTM D1143-74 maintained load procedure and after rebounding, loaded to failure in CRP tests. The south pile which was 36 ft long and extended 27 ft into the sands, failed structurally at a load of 220 tons. The 20 ft long Interpile failed at a load of about 120 tons which was comparable to the 33 ft long auger cast pile.

The load transfer data for the Interpiles are summarized in Figure 11E and F. Because of the residual strain recorded on the strain meters after rebounding from 70 tons, it was necessary to correct the data for the subsequent cycle to failure.

Phillips Refinery

The subsurface conditions at the Phillips Refinery site consist of about 8 ft of silt and/or clay overlying Missouri River sands. Most of the surficial fine-grained soils were removed prior to the load tests. The standard penetration test results are plotted in Figure 13B.

Two 14 inch diameter Interpiles were load tested at this site. Pile lengths were 29 and 35 ft. A strain meter was installed in the shorter pile by attaching it to a Dywidag 1¼ inch diameter re-bar. This pile was first loaded to failure in accordance with the CRP method. After rebounding, it was reloaded in accordance with the "Quick" Method (ASTM D1143-74, paragraph 4.7).

There was only a small residual strain on this gauge after rebounding.

Nearman Bottoms

Subsurface conditions at the Nearman Bottoms site consist of approximately 8½ ft of dredged sand overlying about 9½ ft of clayey silt and silty clay. Missouri River alluvial sands underlie the silt. Standard penetration test values in the alluvial sands ranged from 15 to 40 blows per foot and are given in Figure 14B. Although the test piles were installed through this profile, the production piles had design cutoffs near the bottom of the dredged sand layer; or, in the case of deep structures, in the natural sand.

Two 14 inch instrumented Interpiles were driven and subsequently tested at the Nearman Bottoms site. The piles were driven to depths of 29 and 33 ft. After installation of the piles, three strain meters were placed in each pile. The positions of the gauges were approximately at the dredged sand-silt interface, the silt-alluvial sand contact, and 2 ft from the pile tip. These piles were loaded in accordance with ASTM D1143-74, paragraph 4.2. After rebounding, the piles were loaded to failure using the quick test (ASTM D1143-74, paragraph 4.7).

Load transfer data for these piles are summarized in Figure 14E and F. Because of the residual strains recorded in the strain meters after rebounding from 140 tons, it was necessary to correct the data for the subsequent loading cycle to failure.

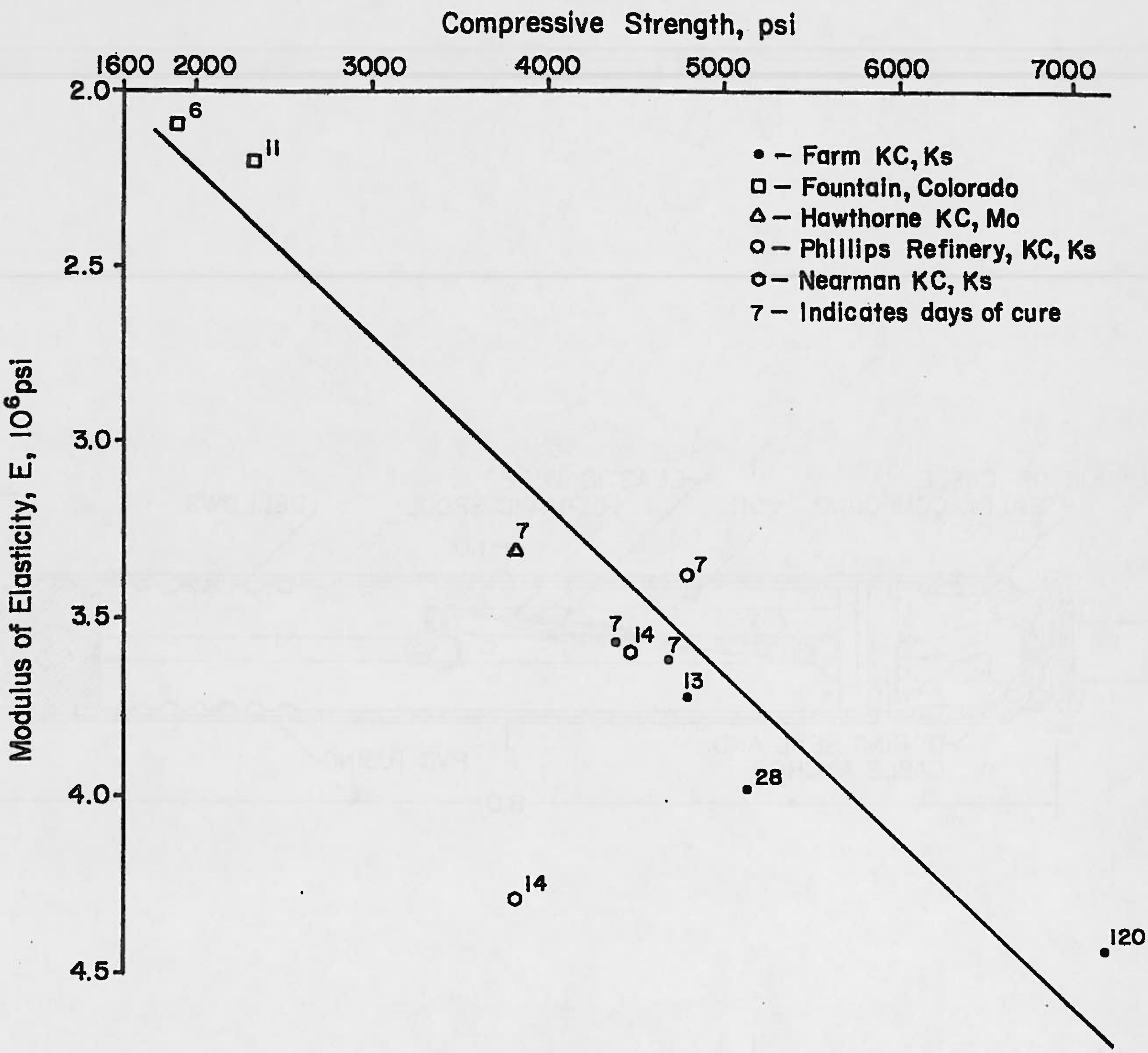
SUMMARY OF TEST RESULTS

There has not been enough data generated in these studies to make broad generalizations. Nonetheless, preliminary comparisons have been made with the empirical correlations presented by Meyerhof (1976). The unit friction is compared with the average standard penetration resistance SPT in Figure 15. Also, the unit end bearing capacity is compared with the final pile penetration resistance. The pile penetration resistance was selected rather than the SPT because it is less susceptible to anamolous conditions such as a gravel pocket or erratic variations in density. Unfortunately, the range in penetration resistance for most of these sites was rather narrow.

These comparisons indicate that the unit end bearing for the Interpile is less than that predicted by Meyerhof for driven piles but greater than for bored piles. On the other hand, the unit frictional capacity of the Interpile appears to be many times greater than the values predicted by Meyerhof. Hopefully, additional data can be obtained to extend these relationships.

CONCLUSION

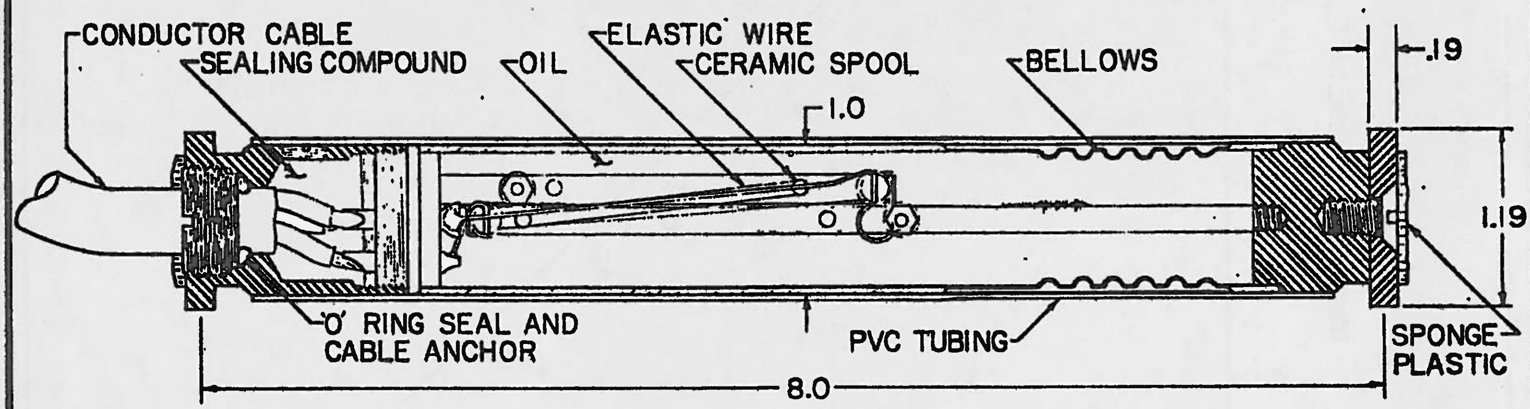
Concrete embedment strain gauges appear to offer a ready, cost effective means to determine load transfer in concrete piles. The increased understanding of pile behavior thus obtained can lead to more effective use of piles, as well as providing the designer with a better characterization of conditions at working load and at failure. From the several tests discussed, it appears construction methods, as well as the foundation soils, influence the load transfer characteristics of a concrete-sand interface.



COMPRESSIVE STRENGTH VS. MODULUS OF ELASTICITY
FOR GROUT CYLINDERS TAKEN
DURING PLACEMENT OF INSTRUMENTED PILES

WCC K78-17
11 JULY 78 KM

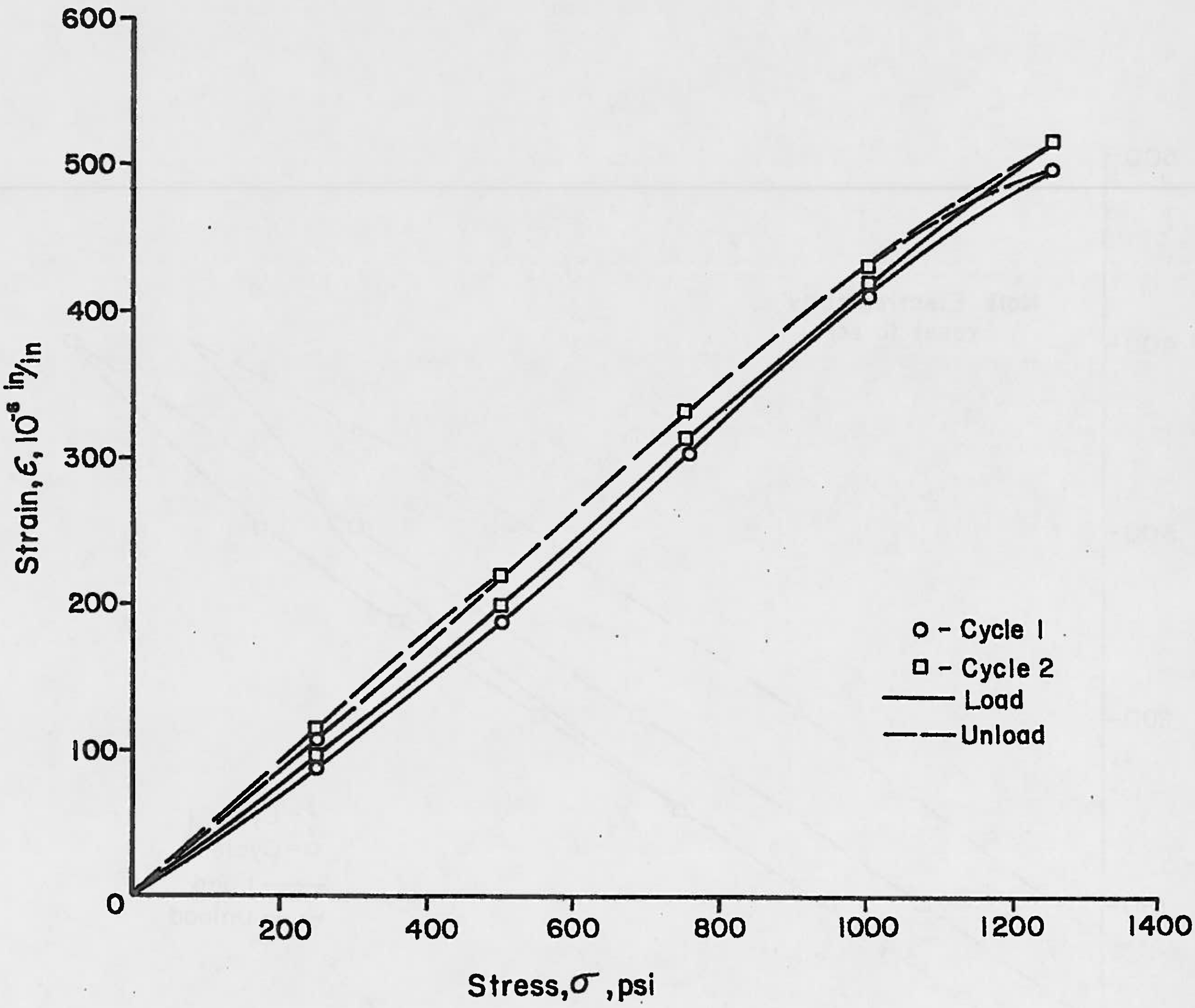
FIGURE 1



CROSS-SECTION OF CARLSON STRAIN GAUGE

WCC K 78-17
14 JULY 78 KM

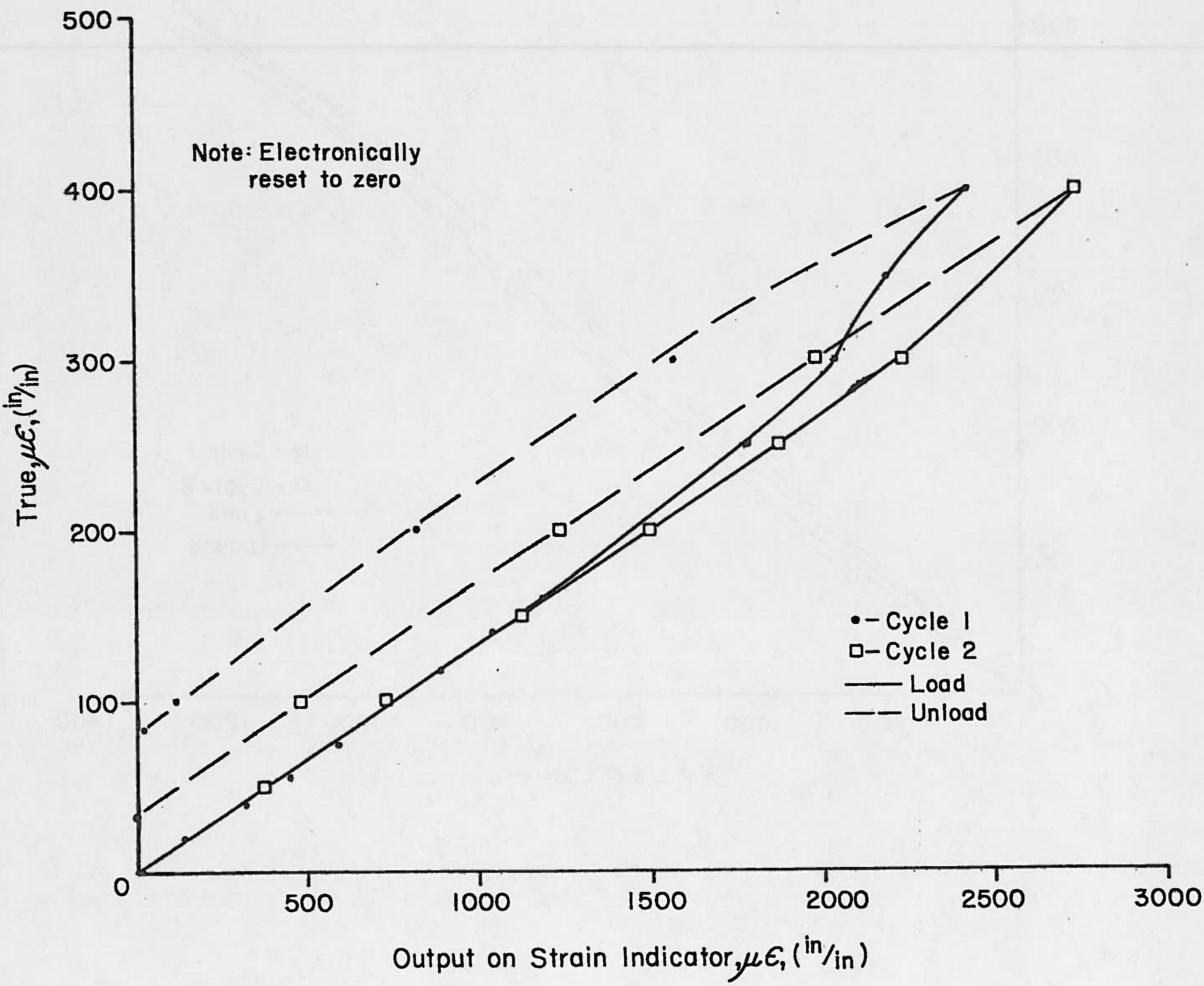
FIGURE 2



STRESS-STRAIN CURVE FOR STRAIN METER
EMBEDDED IN CONCRETE CYLINDER

WCC K78-17
8 JULY 78 KM

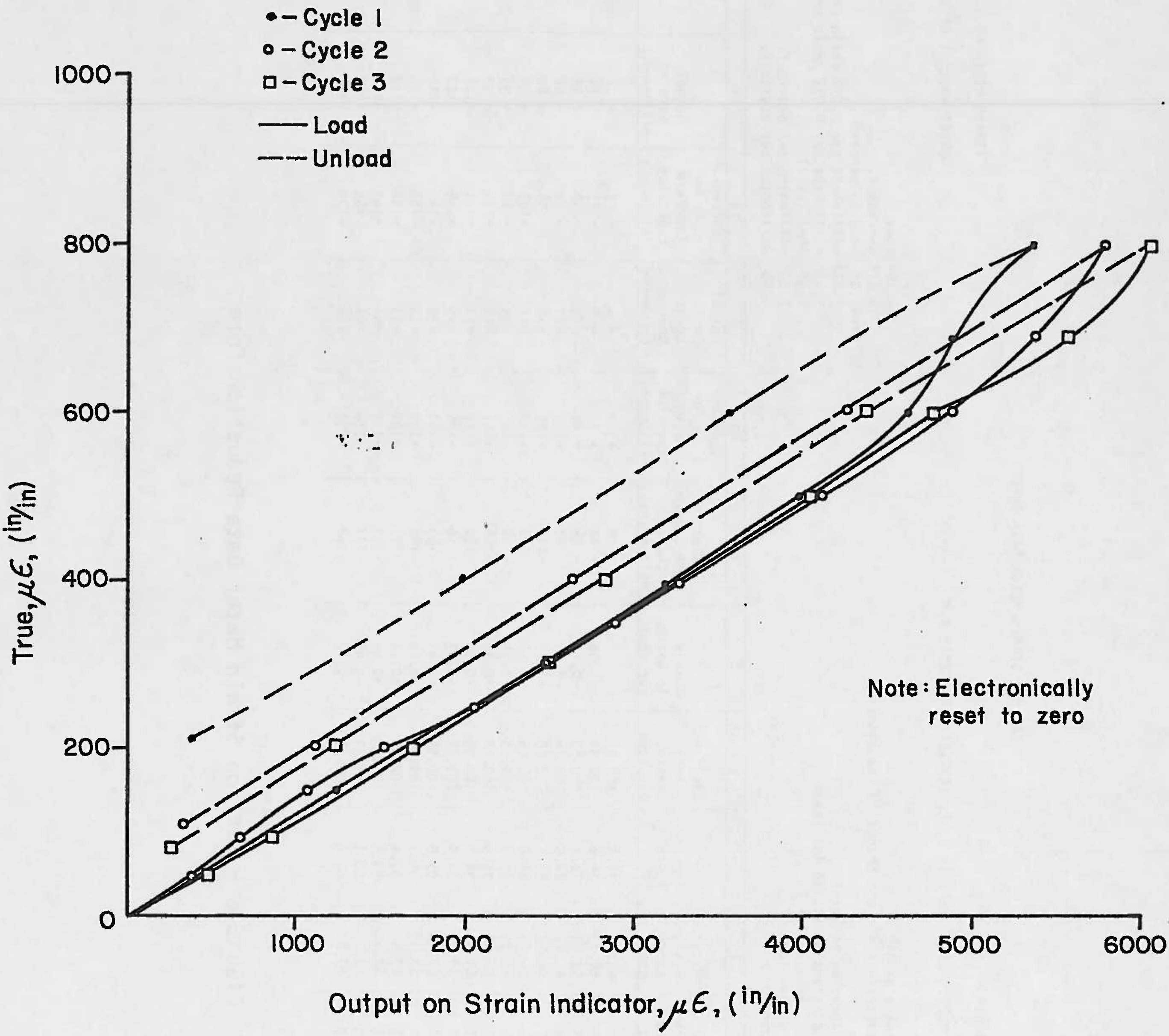
FIGURE 3



STRAIN–TRUE STRAIN DATA FOR
CARLSON STRAIN METER A2039

WCC K 78-17
12 JULY 78 KM

FIGURE 4



STRAIN–TRUE STRAIN DATA FOR CARLSON STRAIN METER A2040

WCC K 78-17
13 JULY 78 KM

FIGURE 5

CARLSON STRAIN METER DATA SHEET

PROJECT: CALIFORNIA ARCH DAM | STRAIN METER NO.: 22-V

LOCATION: BLOCK NO. 15, ELEV. 2,142, STA. 11 + 42, VERTICAL (PLACED 9 A.M., 2-4-54) | SHEET NO.: 1 OF 2

CALIBRATIONS:

Meter resistance at 0*° F.	56.10* ohms
Change in temperature per ohm change in resistance	8.61* degrees F.
Useful range	97.2-103.0* ratio in percent
Original calibration constant	3.82* millionths per 0.01% ratio change
Calibration constant corrected for leads	3.98 millionths per 0.01% ratio change
Resistance of leads at 70° F.	2.61 ohms (pair)
Temperature correction	7.50* millionths per degree F.
Concrete expansion	5.50 millionths per degree F.

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Date	Time	Total Resis-tance, ohms	Lead Resis-tance, ohms	Meter Resis-tance, ohms	Temper-ature °F.	Resis-tance Ratio, per cent	Change in Ratio, per cent	Indicated Unit Length Change, millionths.	Correc-tion for Meter Expansion, millionths	Actual Unit Length Change, millionths	Correc-tion for Concrete Expansion, millionths	Actual Strain, millionths	
2- 5-54	9 a.m.	66.91	2.61	64.30	70.5	100.97		0	+ 4	+ 4	- 3	+ 1	age 24 hr.
2- 6-54	10 a.m.	67.36	2.63	64.73	74.2	100.91	-0.06	- 24	+ 32	+ 8	- 23	- 15	
2- 7-54	9 a.m.	67.94	2.64	65.30	79.1	100.83	-0.14	- 56	+ 68	+12	- 50	- 38	
2- 9-54	9 a.m.	68.56	2.64	65.92	84.5	100.74	-0.23	- 92	+109	+17	- 79	- 62	
2-11-54	9 a.m.	68.98	2.65	66.33	88.1	100.68	-0.29	-116	+134	+18	- 98	- 80	
2-13-54	9.a.m.	69.21	2.66	66.55	89.8	100.65	-0.32	-128	+149	+21	-109	- 88	
2-15-54	9 a.m.	69.41	2.66	66.75	91.5	100.63	-0.34	-136	+161	+25	-118	- 93	
2-17-54	9 a.m.	69.55	2.67	66.88	92.6	100.61	-0.36	-144	+169	+25	-124	- 99	
2-19-54	9 a.m.	69.69	2.67	67.02	94.1	100.59	-0.38	-152	+179	+27	-131	-104	
2-21-54	9 a.m.	69.79	2.67	67.12	94.9	100.58	-0.39	-156	+185	+29	-136	-107	
2-23-54	9 a.m.	69.92	2.68	67.24	95.9	100.56	-0.41	-163	+193	+30	-141	-111	
2-25-54	9 a.m.	70.01	2.68	67.33	96.7	100.51	-0.46	-183	+199	+16	-146	-130	Doubtful
2-27-54	9 a.m.	70.09	2.68	67.41	97.4	100.54	-0.43	-171	+204	+33	-150	-117	
3- 6-54	9 a.m.	70.33	2.68	67.65	99.4	100.53	-0.44	-175	+219	+44	-161	-117	
3-13-54	9 a.m.	70.46	2.69	67.77	100.5	100.53	-0.44	-175	+226	+51	-166	-115	
3-20-54	9 a.m.	70.52	2.69	67.83	101.0	100.55	-0.42	-168	+232	+64	-170	-106	

FIGURE 6 - Carlson Strain Meter Data Reduction Form

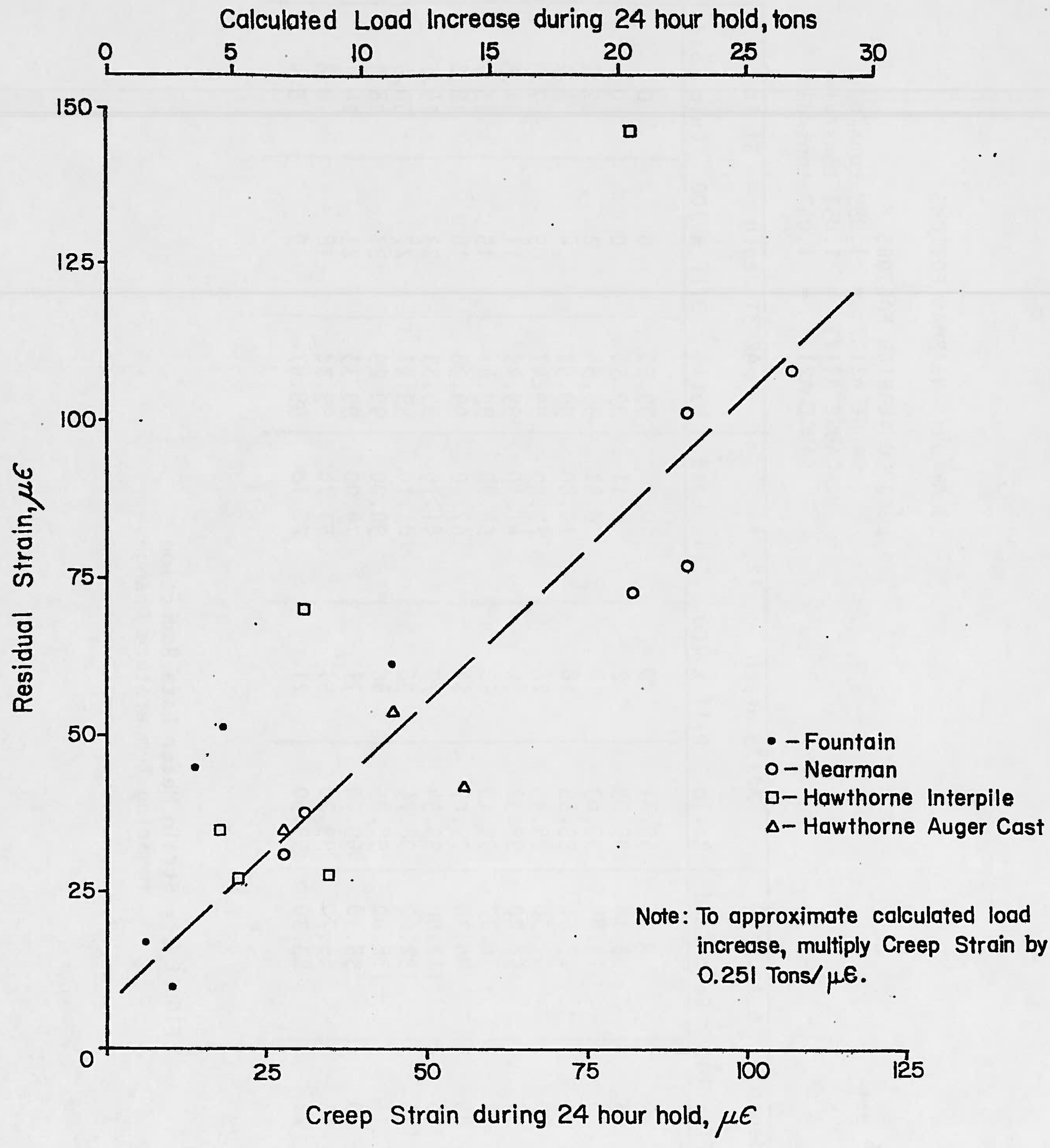
PILE NO. 7
PILE LENGTH 33 Ft.
PILE AREA 107 $Ft.^2$
CONCRETE MODULUS 3.94×10^6 psi

LOCATION: NEARMAN BOTTOMS

LOAD CONVERSION FACTORS
GAUGE A2127 = 1.059 tons/div
GAUGE A2129 = 1.053 tons/div
GAUGE A2131 = 1.047 tons/div

Applied	A2127 depth = 6 ft.			A2129 depth = 12 ft.			A2131 depth = 31 ft.		
Loads, tons	Ratio	Diff x 100	Calc Load	Ratio	Diff x 100	Calc Load	Ratio	Diff x 100	Calc Load
0	100.21	0	0	99.71	0	0	99.56	0	0
17.5	100.18	3	3.18	99.69	2	2.11	99.56	0	0
35.0	100.07	14	14.80	99.63	8	8.42	99.54	2	2.1
52.5	99.95	26	27.50	99.55	16	16.80	99.52	4	4.2
70.0	99.82	39	41.30	99.45	26	27.40	99.47	9	7.3
87.5	99.66	55	58.20	99.32	39	41.10	99.45	11	11.5
105.0	99.49	72	76.20	99.19	52	54.80	99.41	15	15.7
122.5	99.32	89	94.30	99.07	64	67.40	99.38	18	18.8
140.0 1 hr.	99.17	106	112.00	99.94	77	81.10	99.33	23	24.1
140.0 24 hr.	98.96	125	132.00	98.75	96	101.10	99.27	27	30.4
105.0	99.10	111	118.00	98.85	86	90.00	99.29	27	28.3
70.0	99.31	90	95.30	99.00	71	74.00	99.33	23	24.1
35.0	99.58	63	66.70	99.20	51	53.70	99.38	18	18.8
0	99.99	22	23.30	99.50	21	22.10	99.47	9	9.4

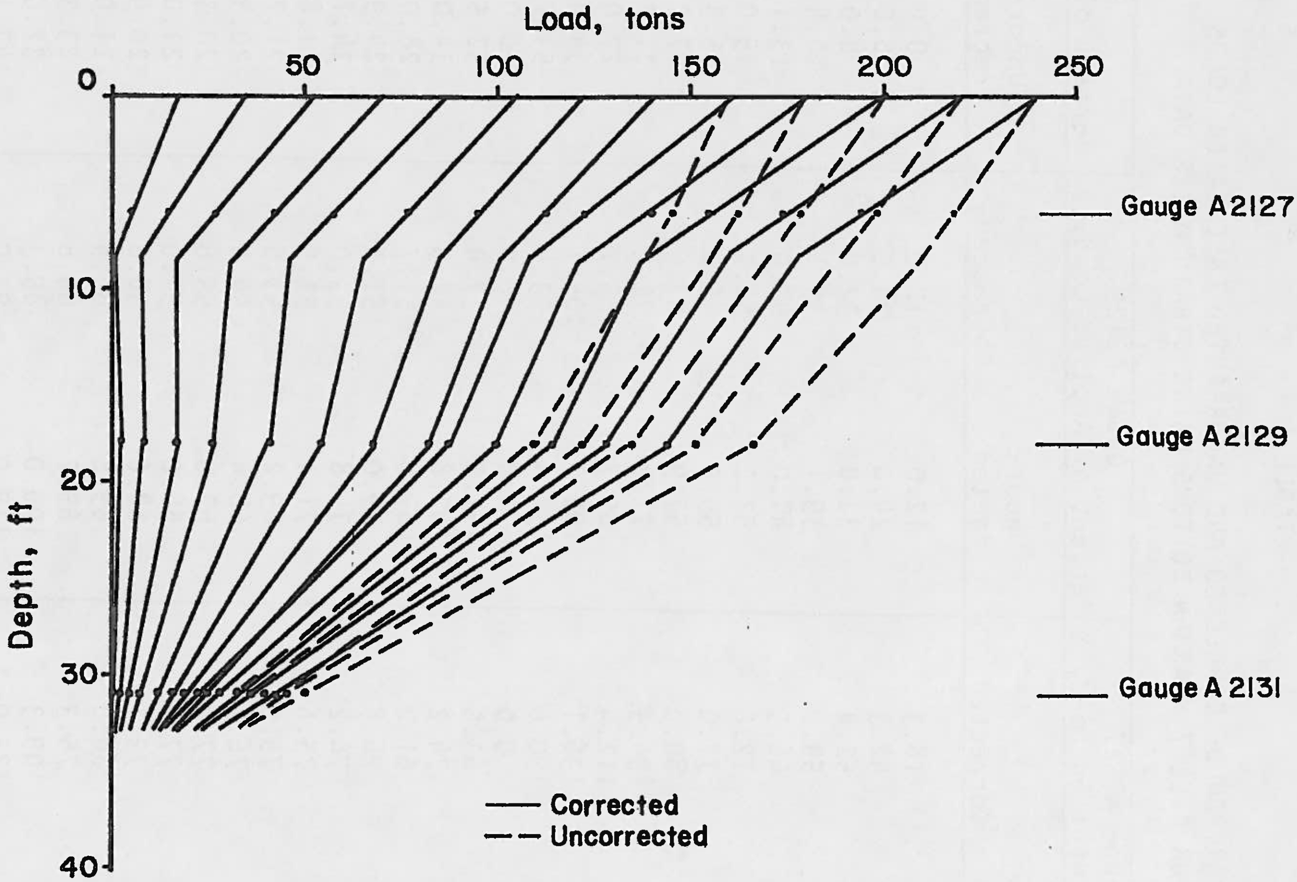
FIGURE 7 - Strain Meter Data Reduction
Ignoring Temperature Changes



CREEP STRAIN VS. RESIDUAL STRAIN DATA
OF INSTRUMENTED PILES

WCC K78-17
11 JULY 78 KM

FIGURE 8



CORRECTED AND UNCORRECTED LOAD TRANSFER DATA
OF PILE 7
NEARMAN POWER PLANT
KANSAS CITY, KANSAS

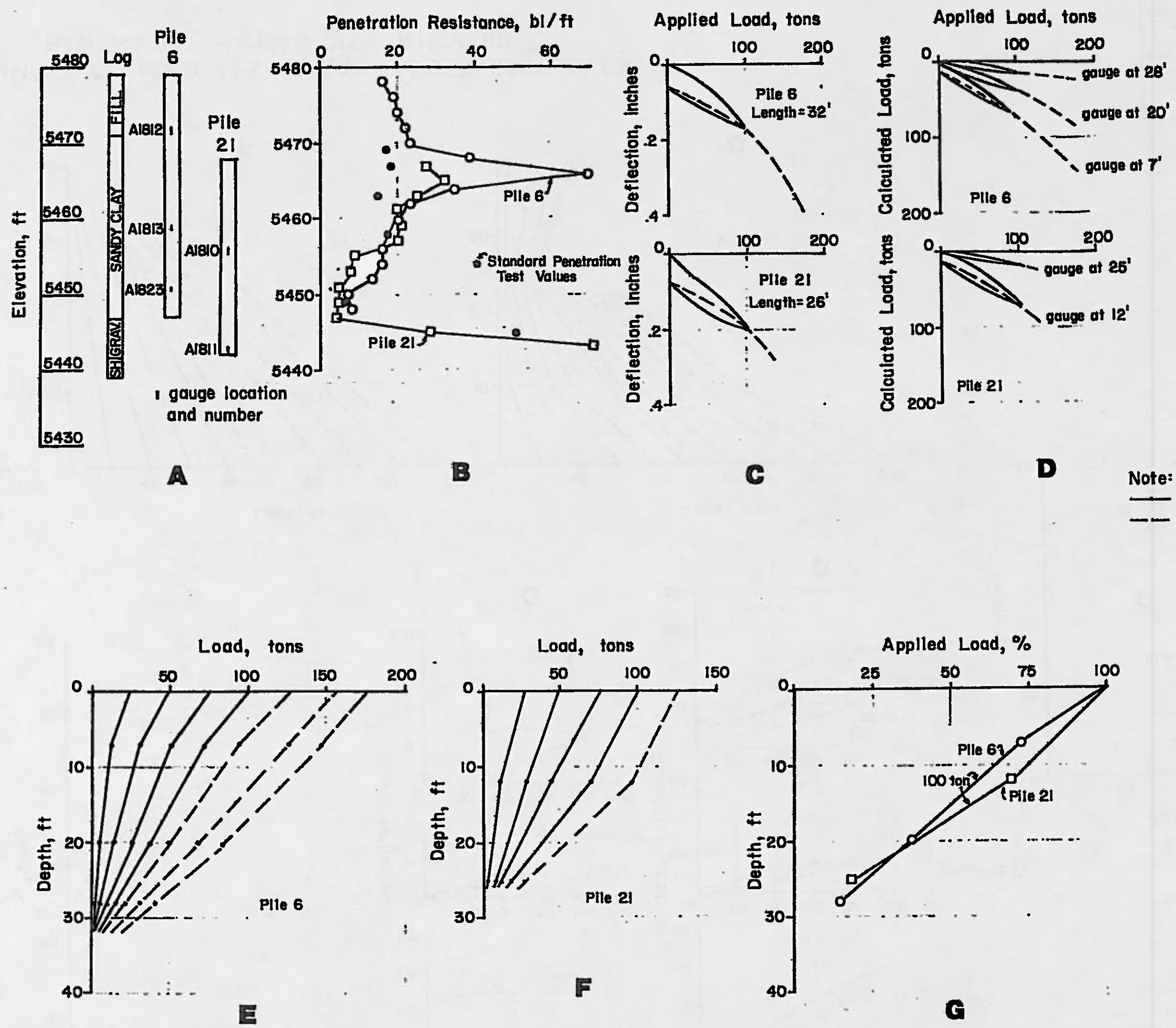
WCC K78-17
12 JULY 78 KM

FIGURE 9

TABLE I

COMPARISON OF CORRECTED AND UNCORRECTED CALCULATED LOADS
FOR PILE 7 NEARMAN BOTTOMS USING STRAIN METER DATA

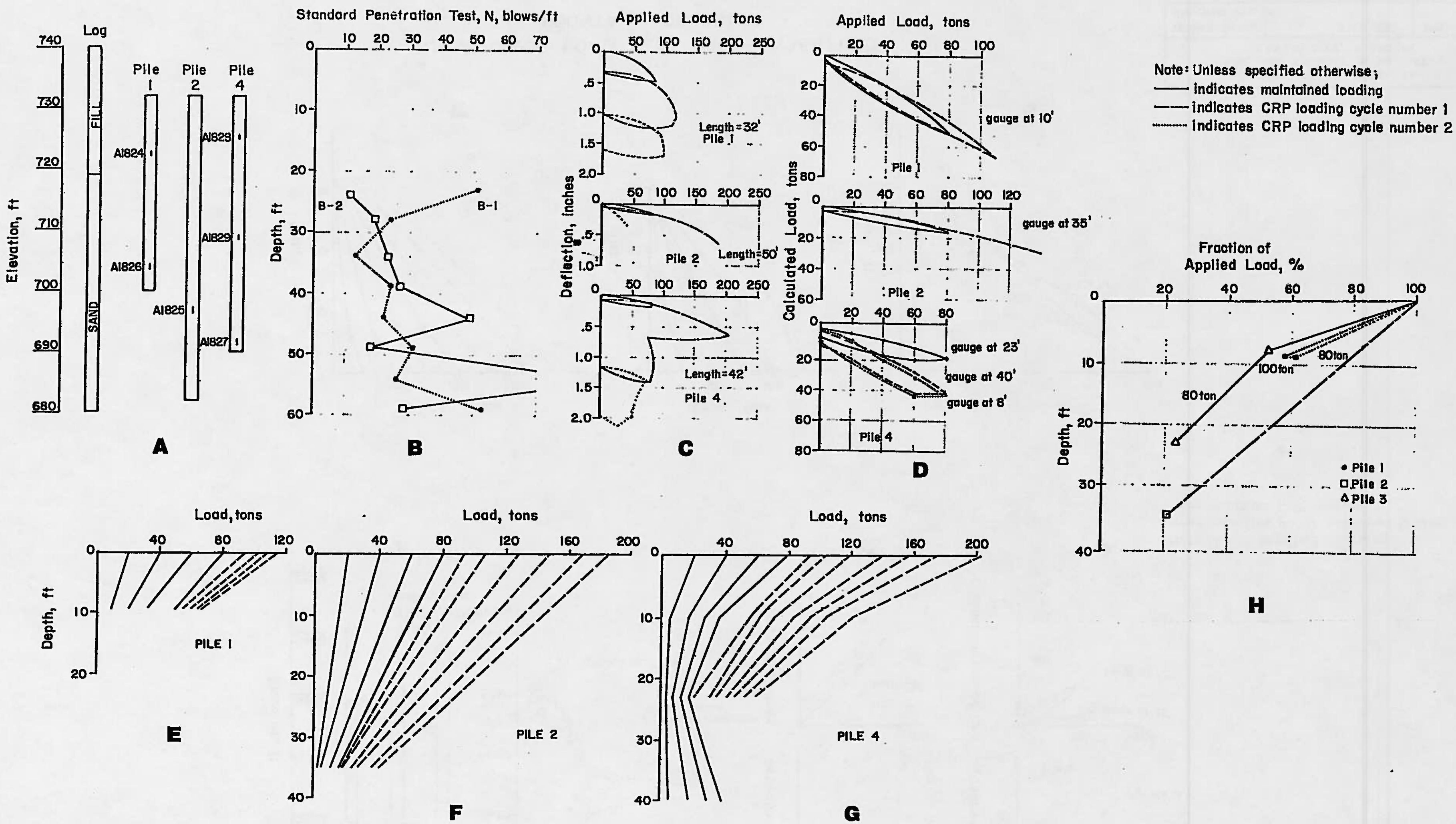
	Gauge A2127 Percent of Applied Load, %		Gauge A2129 Percent of Applied Load, %		Gauge A2131 Percent of Applied Load, %	
Applied Load, Tons	Uncor-rected	Corrected	Uncor-rected	Corrected	Uncor-rected	Corrected
17.5	18.3	18.3	12.0	12.0	0.0	0.0
35.0	42.3	42.3	24.0	24.0	6.0	6.0
52.5	52.4	52.4	32.0	32.0	8.0	8.0
70.0	59.0	59.0	39.1	39.1	10.4	10.4
87.5	66.5	66.5	47.0	47.0	13.1	13.1
105.0	72.6	72.6	52.2	52.2	15.0	15.0
122.5	77.0	77.0	55.0	55.0	15.3	15.3
140.0 1 hr.	80.0	80.0	57.9	57.9	17.2	17.2
140.0 24 hr.	94.3	94.3	72.2	72.2	21.7	21.7
105.0	112.4	112.4	85.7	85.7	27.0	27.0
70.0	136.1	136.1	105.7	105.7	34.4	34.4
35.0	190.6	190.6	153.4	153.4	53.7	53.7
0.0	0.0	0.0	0.0	0.0	0.0	0.0
20.0	180.0	63.5	147.5	37.0	52.5	5.5
40.0	132.5	74.2	102.8	47.5	34.0	10.5
60.0	114.7	61.2	87.8	51.7	28.0	12.3
80.0	105.9	76.8	79.0	51.2	24.9	13.1
100.0	99.5	76.2	74.8	53.0	24.1	15.0
120.0	95.0	75.6	71.1	52.5	21.8	14.2
140.0	92.1	75.5	69.2	53.6	21.7	15.0
160.0	90.6	76.2	68.4	54.4	20.3	14.4
180.0	90.0	77.2	67.8	55.6	20.3	15.0
200.0	89.0	77.5	68.0	57.0	21.0	16.5
220.0	89.5	79.1	68.6	58.6	20.4	16.4
240.0	90.4	80.8	69.6	60.4	21.0	17.1
260.0	88.8	75.3	69.2	60.8	23.3	19.6
150.0	125.3	109.3	100.0	85.3	37.0	30.7
100.0	147.0	123.0	119.0	97.0	47.1	38.0
50.0	202.0	156.0	164.0	120.0	71.3	52.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0



Note: Unless specified otherwise,
— indicates ASTM maintained loading
— — indicates CRP loading

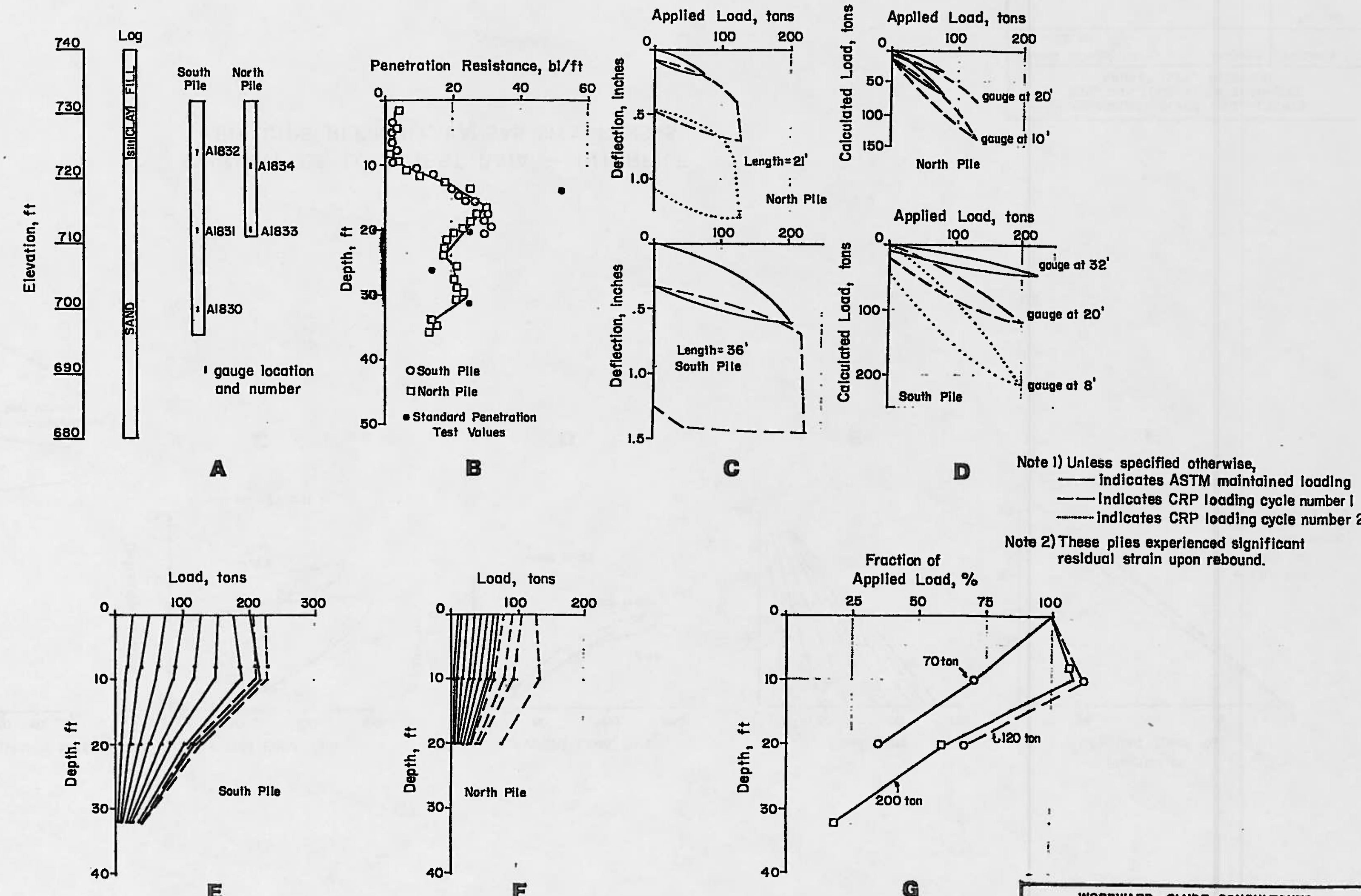
SUMMARY OF LOAD TEST DATA—INTERPILES
FOUNTAIN, COLORADO

WOODWARD-CLYDE CONSULTANTS SOIL AND FOUNDATION ENGINEERS KANSAS CITY, MISSOURI		
DRAWN BY: Kach CHECKED BY: BJR	7-13-78	PROJECT NO.
		FIG. NO. 10



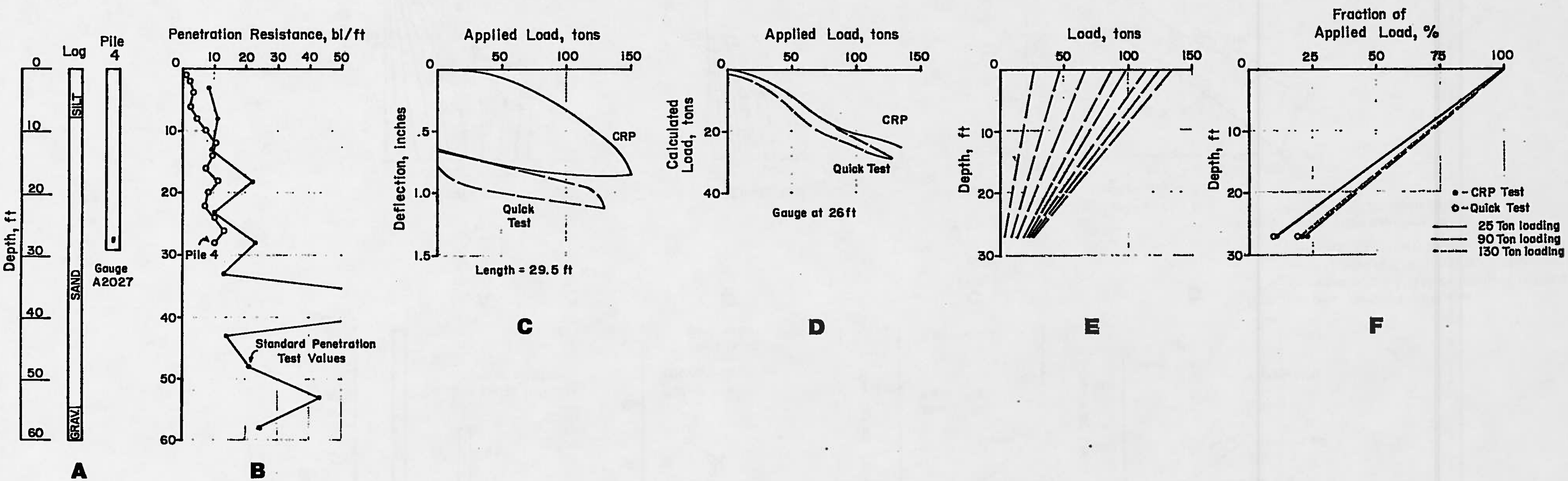
SUMMARY OF LOAD TEST DATA — AUGER CAST PILES
HAWTHORNE, KANSAS CITY, MISSOURI

WOODWARD — CLYDE CONSULTANTS SOIL AND FOUNDATION ENGINEERS KANSAS CITY, MISSOURI		
DRAWN BY: Kah CHECKED BY: Rgr	7-15-78	PROJECT NO.
		FIG. NO. 11



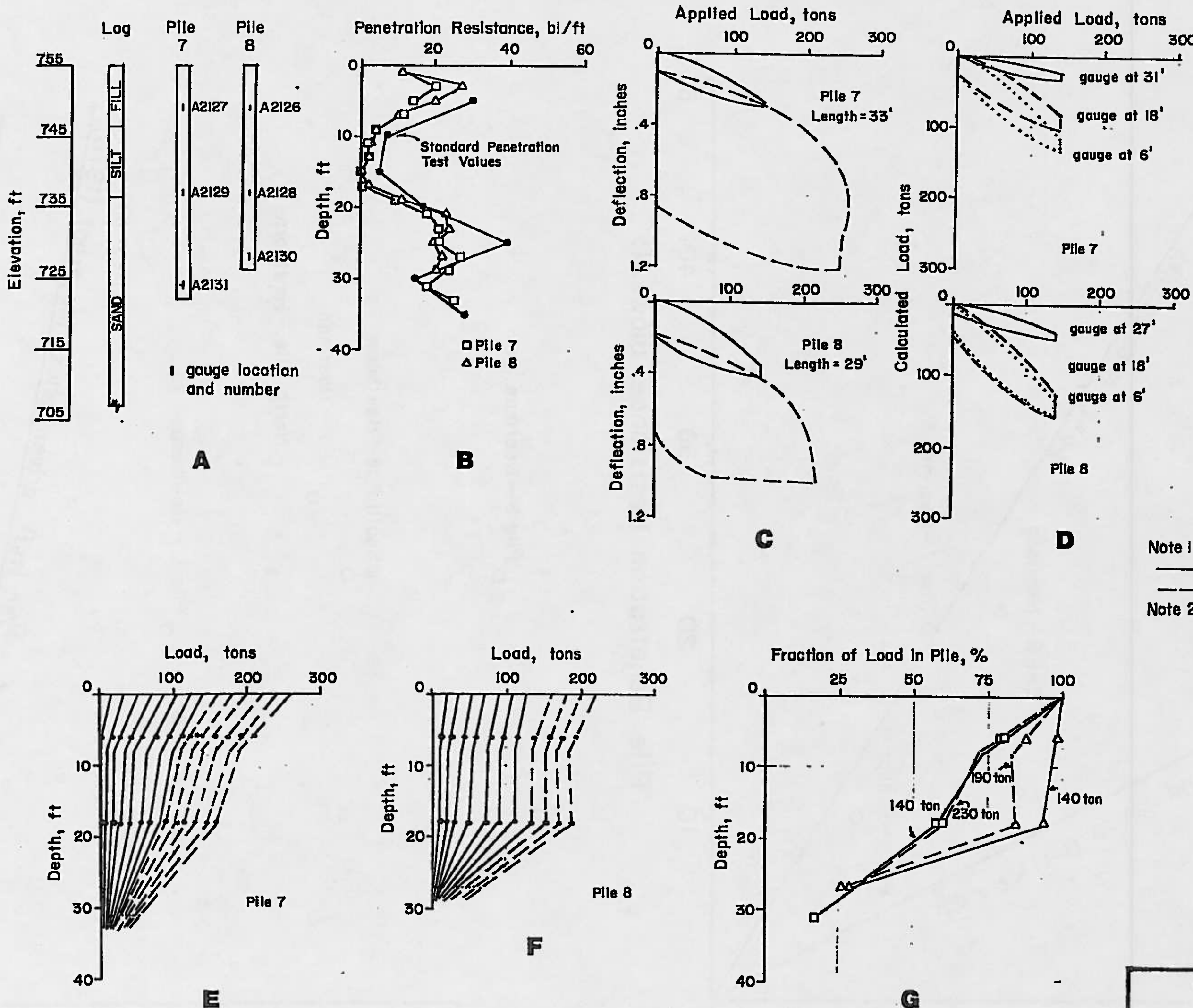
SUMMARY OF LOAD TEST DATA — INTERPILES
HAWTHORNE, KANSAS CITY, MISSOURI

WOODWARD-CLYDE CONSULTANTS SOIL AND FOUNDATION ENGINEERS KANSAS CITY, MISSOURI		
DRAWN BY: Kath CHECKED BY: Rgr	7-12-78	PROJECT NO.
		FIG. NO. 12



SUMMARY OF LOAD TEST DATA — INTERPILE
PHILLIPS REFINERY, KANSAS CITY, KANSAS

WOODWARD—CLYDE CONSULTANTS SOIL AND FOUNDATION ENGINEERS KANSAS CITY, MISSOURI		
DRAWN BY: Keth CHECKED BY: RSL	7-10-78	PROJECT NO.
		FIG. NO. 13

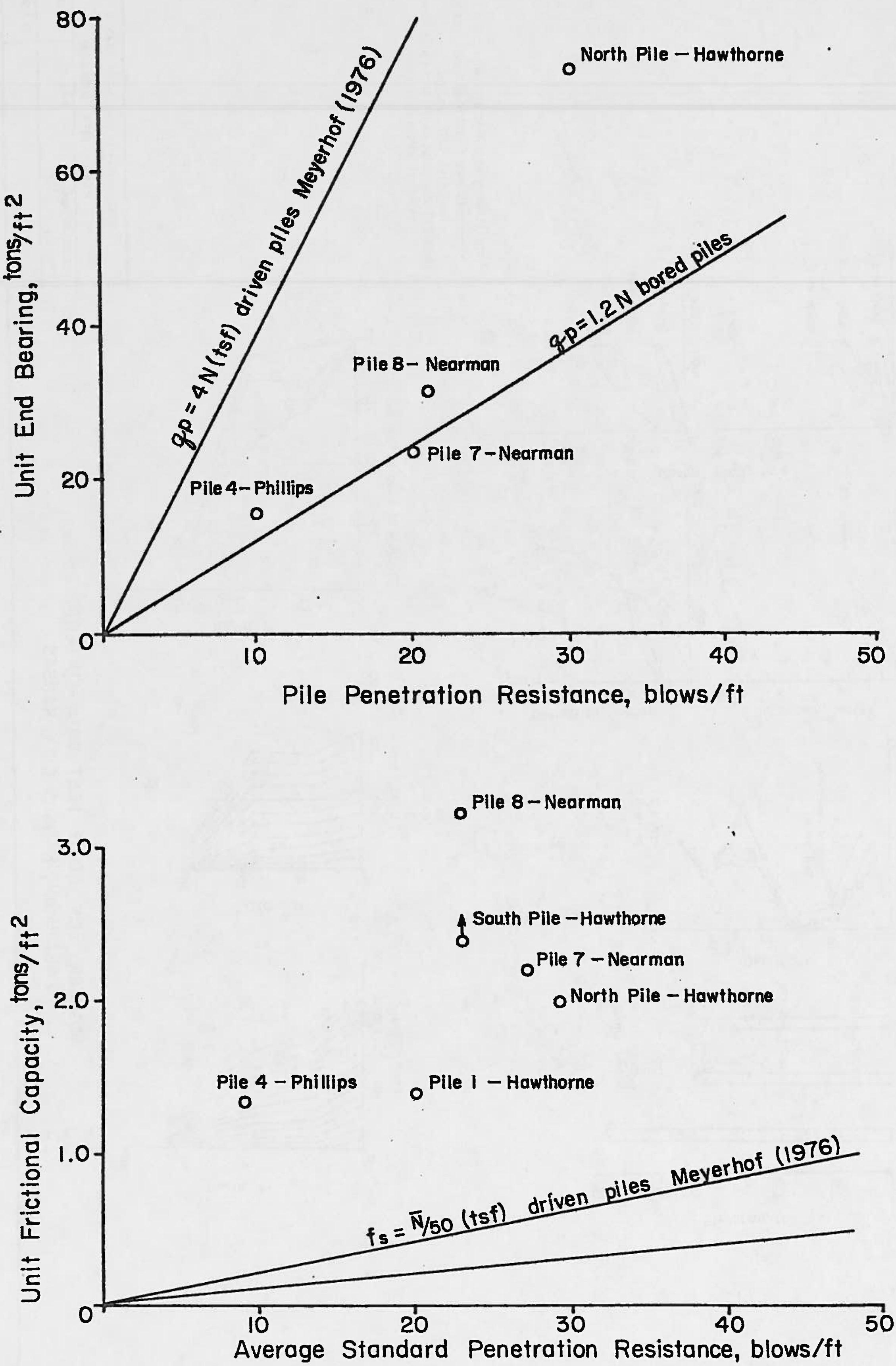


Note 1) Unless specified otherwise,
—— indicates ASTM maintained loading
— — indicates CRP loading
Note 2) These piles experienced significant residual strain upon rebound.

SUMMARY OF LOAD TEST DATA — INTERPILES
NEARMAN, KANSAS CITY, KANSAS

WOODWARD—CLYDE CONSULTANTS
SOIL AND FOUNDATION ENGINEERS
KANSAS CITY, MISSOURI

DRAWN BY: [handwritten initials] 7-11-78 PROJECT NO.
CHECKED BY: [handwritten initials]
FIG. NO. 14



CORRELATION OF DATA FROM INSTRUMENTED PILES
WITH MEYERHOF ANALYSIS

WCC K78-17
13 JULY 78 KM

FIGURE 15

BIBLIOGRAPHY

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5

FRICTION PILES IN SAND —
A REVIEW OF STATIC DESIGN PROCEDURES

William A. Cutter, Senior Project Engineer
ATEC Associates, Inc., Indianapolis, Indiana

David L. Warder, Senior Project Engineer
ATEC Associates, Inc., Indianapolis, Indiana

ABSTRACT

A review is presented of five static procedures for estimating shaft friction of piles driven into sand. Predictions based on these static relationships are compared with the results of pile load tests conducted at several sites. Since the test piles were not fully instrumented, it was necessary to establish the component of pile resistances due to shaft friction primarily by interpreting the load-gross settlement curves. While this procedure is imprecise, the scatter in computed values was generally much greater than the range of values interpreted from the load test results. The results tend to indicate that none of the static methods considered accurately accounts for all of the factors bearing on the complex load transfer mechanism governing the behavior of piles driven into sand. Conclusions are drawn relative to the reliability of the various static procedures in predicting friction for the pile types and lengths and the variety of subsurface conditions represented by the load tests.

INTRODUCTION

Estimating the load carrying capacity of piles provides the geotechnical engineer with one of his more difficult tasks. The ideal means by which to pursue such an endeavor is to select several reasonable pile types and sizes based on static relationships and experience and then to establish firm design and installation criteria based on the results of field pile load tests. Unfortunately, time and cost considerations seldom allow this procedure. More often, the engineer is forced to make pile capacity estimates which will only be proof-tested in the field. To further complicate the problem, the proof-test is often conducted after the production piles have been delivered to the site. Low test results at this stage result in either additional load tests, changes in the design load or alterations to the pile size or length, all of which cause embarrassment to the designer. This dilemma forces the practicing geotechnical engineer to rely heavily on estimates based on static relationships.

Static equations often produce unreliable results which are not necessarily on the conservative side. This stems primarily from the lack of a sound understanding of the load transfer mechanism and the absence in many relationships of factors that are known to influence shaft friction. The end result may be either an unnecessarily expensive foundation or, worse yet, a foundation with an inadequate safety factor.

The primary objective of this paper is to review several of the more common static methods for estimating the frictional load carrying component of piles driven into granular soils and to compare these predictions with the results of pile load tests conducted at several sites. A secondary objective is to report the results of several pile load tests (with accompanying subsurface data) which are currently unpublished.

DISCUSSION OF INVESTIGATION

The data base is composed of pile load test results taken from the files of ATEC Associates, Inc., from tests solicited from interested parties and, to a lesser extent, from available literature. From these data, 17 tests were selected for further consideration. The pile load tests considered herein were not instrumented beyond that required to measure the total load on the pile and the corresponding butt movement. Therefore, a significant limitation of this study is that the frictional component of load transfer was interpreted primarily from the load-gross settlement curve. While it is not possible to precisely determine the frictional portion of load transfer from only the load-gross settlement curves, reasonable estimates of the probable maximum and minimum friction values were interpreted for each test. Driving records were not available for all piles and, in general, were not considered in the interpretation of the data.

Five static methods of estimating shaft friction were considered in this study. The results of each were compared with the interpreted friction range (based on the pile load test results) to assess the accuracy and consistency of predictions for the piles considered.

INTERPRETATION OF PILE LOAD TESTS

From the initial collection of 55 pile load tests, 17 tests were selected for detailed consideration. The selection was based primarily on the completeness of the soil and pile load testing data as well as the reliability with which the component of shaft friction could be interpreted from the load-gross settlement curves. Of the 17 tests selected, four pile types are represented ranging in length from 42 to 85 ft. The pile type, length, test site location and the test number designations utilized in this paper are shown in Table 1. The 17 tests include 13 compression tests and 4 tension (or uplift) tests. The tension tests were conducted on piles

TABLE 1 - IDENTIFICATION OF PILE TESTS

Test Number	Pile Type	Embedment, ft	Location
20	Raymond Step-Taper	70.4	Council Bluffs, IA
25	Steel Pipe	58	Muskegon, MI
26	Monotube	58	Muskegon, MI
27	Steel Pipe	58	Muskegon, MI
28	Raymond Step-Taper	58	Muskegon, MI
37	Steel Pipe	84.8	Louisville, KY
38	Precast Tapered Concrete	53	Louisville, KY
39	Precast Tapered Concrete	53	Louisville, KY
40	Raymond Step-Taper	70	Louisville, KY
41	Steel Pipe	70	Louisville, KY
42	Raymond Step-Taper	55	Louisville, KY
44	Raymond Step-Taper	42	Palatka, FL
45	Steel Pipe	43.3	Palatka, FL
21	Tension test corresponding to compression Test No. 20		
34	Tension test corresponding to compression Test No. 38		
33	Tension test corresponding to compression Test No. 40		
35	Tension test corresponding to compression Test No. 41		

which had previously been tested in compression and which are also included for consideration in this study.

The test piles from Louisville are part of an extensive program conducted for Louisville Gas and Electric Company's Mill Creek Generating Station. The Muskegon piles are part of the Michigan pile study (1965) and were considered primarily because of the loose sand present at this test site. The remaining 3 piles were tested at two independent locations.

As noted above, none of the piles considered herein were instrumented. In addition, complete net settlement curves were available for only two tests. Consequently, estimates of the most probable maximum and minimum value of shaft friction were generally interpreted from the gross settlement curves. In arriving at these limiting values, primary consideration was given to the stiffness of the pile and the subgrade soils, the shape of the gross settlement curve and the limiting values of butt movement which are generally associated with full (or near full) development of shaft friction. For those piles which were tested in both tension and compression, the results of the tension tests were considered in establishing limiting minimum friction values for the compression tests. Admittedly, considerable judgement was exercised in estimating these limiting values. However, for the piles selected, it is felt that the data support the ranges of frictional resistance selected. The limiting values of maximum and minimum shaft friction estimated from the 13 compression tests are shown in Table 2. Tension test interpretations are tabulated in Table 3.

TABLE 2 - SUMMARY OF COMPRESSION TEST RESULTS

Test Number	Interpreted Frictional Capacity, tons		
	Minimum	Maximum	Mean
20	120	130	125
25	40	45	42
26	40	50	45
27	25	35	30
28	65	80	72
37	120	160	140
38	105	140	122
39	110	135	122
40	120	150	135
41	60	85	72
42	90	120	105
44	110	150	130
45	90	120	105

TABLE 3 - SUMMARY OF TENSION TEST RESULTS

Test Number	Load (tons) at 0.25 in. Movement	Load (tons) at 1 in. Movement
21	115	*
33	60	115
34	60	95*
35	39	50

* Test taken to maximum butt movement of only 0.25 in.
**Estimated value; test taken to only 0.61 in. of butt movement.

Where a sharp change in curvature was noted in the gross settlement curve, the range between the maximum and the minimum estimated friction values is relatively small. Conversely, difficulties in predicting these minimum and maximum values are implied for some tests by the relatively large range in limiting values.

The load settlement curves, along with descriptions of the foundation soils and the results of Standard Penetration Tests, are shown on Figures A1 through A13 in the Appendix.

STATIC RELATIONSHIPS

Five static relationships for predicting the frictional capacity for piles in sand were considered in this study. A brief description of each method is provided below:

Method 1 - Meyerhof Equation

Meyerhof (1956) proposed the following relationship between ultimate unit shaft friction and the results of the Standard Penetration Test (N-values):

$$f = N/50$$

where f = average unit shaft friction over the embedded length of the pile in tsf

N = the average N-value in blows/ft from the Standard Penetration Test

For this consideration, no correction is made to the N-values to account for overburden pressure.

For most piles considered, the N-values were used directly. In the case of the Louisville piles, the borings were made from the original ground surface while the pile load tests were conducted from an excavated lower level. For this case, the N-values were adjusted to account for this change in overburden pressure using the relationship proposed by Gibbs and Holtz (1957).

Method 2 - Conventional Equation

For piles embedded in sand, unit shaft friction can be computed using the familiar expression,

$$f = K\, P_v \tan\delta$$

in which f = average unit shaft friction for the embedded length,

K = coefficient of lateral earth pressure,

P_v = average effective vertical pressure for the embedded length and,

δ = the angle of shearing resistance between soil and the pile material

This equation implies that the increase in unit shaft friction with depth is not bounded. However, studies conducted by Vesic (1966) indicate that unit friction increases only to a certain depth, which depends primarily on the size of the pile and the density of the subgrade material. Below this level, unit friction has been observed to remain constant (or decrease slightly) with penetration. The results of Vesic's study indicate that a limiting value of shaft friction develops at about 20 pile diameters in very dense sand and at about 10 pile diameters for very loose sand.

For this study, it was assumed that the value of shaft friction reached a limit at 15 pile diameters. Where layers with properties different from those encountered in the upper 15 pile diameters were penetrated at a deeper level, it was assumed that the deeper material extended to the ground surface and the limiting value of friction (to be used in the deeper stratum) computed as outlined below.

Values of the lateral earth pressure coefficient K and the angle of friction between the soil and the pile material, δ, are available in the literature. However, the ranges of these values, especially K, are extremely broad and require considerable judgement on the part of the designer.

The values of K and δ were established as those generally used by the authors when considering deep granular deposits along the Ohio River. The values of the friction angle between the soil and pile material, δ, used for Method 2 are shown below:

Pile Material	Clean Sands and Gravels	Silty Sands and Gravels
Steel	25 degrees	20 degrees
Concrete	30 degrees	25 degrees

The values for clean sand are consistent with the data developed by Mansur and Hunter (1970).

The value of the lateral earth pressure coefficient (K) is regarded as a function of in-situ soil density and the relative displacement of the pile. The K-values used in this study for Method 2 are as follows:

Degree of Displacement	Loose N<11	Medium Dense 11<N<30	Dense N>31
Low Displacement (H-piles)	0.5	0.7	0.9
High Displacement (Straight)	0.7	1.0	1.15
High Displacement (Tapered)	0.8	1.25	1.5

Method 3 - Nordlund Equation

The component of Nordlund's equation (1963) which relates to shaft friction is as follows:

$$f = K_\delta \; P_d \; C_d \; \Delta d \; \sin\delta$$

where f = the average ultimate unit shaft friction

K_δ = the coefficient of lateral earth pressure

δ = the angle of shearing resistance between the pile and the soil

C_d = circumference of the pile

Δ_d = length of the pile segment being considered

P_d = effective vertical stress at the midpoint of the segment being considered

Charts for evaluating the coefficient of lateral soil pressure (K_δ) appear in the original Nordlund paper (1963) and are not reproduced herein. The value of K_δ is a function of the volume displacement of the pile, the internal angle of shearing resistance (ϕ) of the soil, the angle of friction between the pile and soil (δ) and the angle of pile taper. Nordlund proposed that ϕ be estimated from results of the Standard Penetration Test (N-values) using the relationship between ϕ and N-values proposed by Peck, Hanson and Thornburn (1953). The N-values were not corrected for overburden pressure.

As evidenced by the form of the equation, the increase in unit shaft friction with depth is not limited. For the sake of comparison, the authors have utilized the Nordlund approach as proposed in 1963. Modifications to the Nordlund approach which limit the value of unit shaft friction have been proposed by others. Method 4, described below, presents one such modification.

Method 4 - Michigan Formula

A modification to the Nordlund method was proposed by Williams (1965). Utilizing the same basic equation as Nordlund, Williams proposed modifications in the procedures for estimating the lateral earth pressure (K_δ). Graphs used for this evaluation were provided in the 1965 paper and are not presented here. The following values of δ (which also differ from Nordlund's) were proposed by Williams:

Steel Pipe or H-Pile	$\delta = 18.5^{\circ}$ but not to exceed ϕ
Timber Pile	$\delta = 21.0^{\circ}$ but not to exceed ϕ
Precast Concrete Pile	$\delta = 23.0^{\circ}$ but not to exceed ϕ
Fluted Taper Pile	$\delta = 25.0^{\circ}$ but not to exceed ϕ
Raymond Step-Taper Pile	$\delta = 27.0^{\circ}$ but not to exceed ϕ
Raymond Standard Pile	$\delta = 30.0^{\circ}$ but not to exceed ϕ

Williams further proposed a modification to the method of predicting ϕ from the Standard Penetration Test results. The effect of overburden pressure is included for low confining pressures and special consideration is given to very silty materials in the low density range.

It has also been proposed (subsequent to 1965; unpublished) that the component of the equation, $K_\delta \sin\delta$, be limited to unity. This modification, although not included by Williams in the original paper, was used by the authors in this study.

<u>Method 5 - Vesic Procedure</u>

Vesic (1970) suggested that for driven piles in medium-to-dense sand, a limiting value of unit shaft friction will develop below a certain level. The relationship for evaluating shaft friction, which is a function only of the relative density of the soil, is provided below:

$$f_s = 0.08(10)^{1.5D_r^4}$$

where f_s = the limiting value of unit friction in tsf

D_r = relative density of the sand

According to Vesic, subsequent investigations have indicated that limiting unit shaft friction values on the order of 1.5 times greater than those produced by this relationship may be more appropriate. However, the equation presented above was used directly in this study.

In this paper, the values of D_r were established by using the results of Standard Penetration Tests and the relationships proposed by Gibbs and Holtz (1957). The value of shaft friction was assumed to increase linearly to a depth of 15 pile diameters below which it remains constant. Where layers with properties different from those encountered at 15 pile diameters were penetrated at a deeper level, it was assumed that the deeper layer extended to the ground surface and the limiting value of friction computed as outlined above.

Summary of Predictions

Computations of ultimate shaft friction using the methods described above were made for the 13 compression load tests considered in this study. The predictions are summarized in Table 4. For comparison, the mean values of shaft friction estimated from the load tests have also been included in this table.

DISCUSSION OF RESULTS

For graphical comparison, the interpreted upper and lower bound limits of shaft friction taken from the load test results are shown in Figure 1. For this representation, the load interval between the maximum and minimum interpreted values is shown by the shaded areas on the base line for each test. The predicted values of ultimate friction for each of the five static methods considered are also shown.

Compression Tests

The results of the compression tests are plotted in Figure 2 as average ultimate unit shaft friction versus average N-value over the length of the pile. In this plot, the mean value of the maximum and minimum interpreted values of total friction (taken from the load tests) was averaged over the embedded surface area of the pile. In three cases (Test Nos. 9, 44 and 45), where cohesive layers were included in the

TABLE 4 - SUMMARY OF PREDICTIONS

Test Number	Interpreted Friction*, tons	Computed Friction, tons Method 1	2	3	4	5
20	125	85	86	301	269	38
25	42	29	23	29	42	18
26	45	26	22	62	73	16
27	30	29	23	29	42	18
28	72	29	29	97	101	16
37	140	119	123	250	276	42
38	122	85	148	891	292	29
39	122	84	154	813	285	29
40	135	103	127	1139	370	34
41	72	102	96	190	180	36
42	105	69	85	561	235	23
44	115	115	44	486	98	56
45	105	110	39	61	67	53

* Mean of maximum and minimum friction values interpreted from load test.

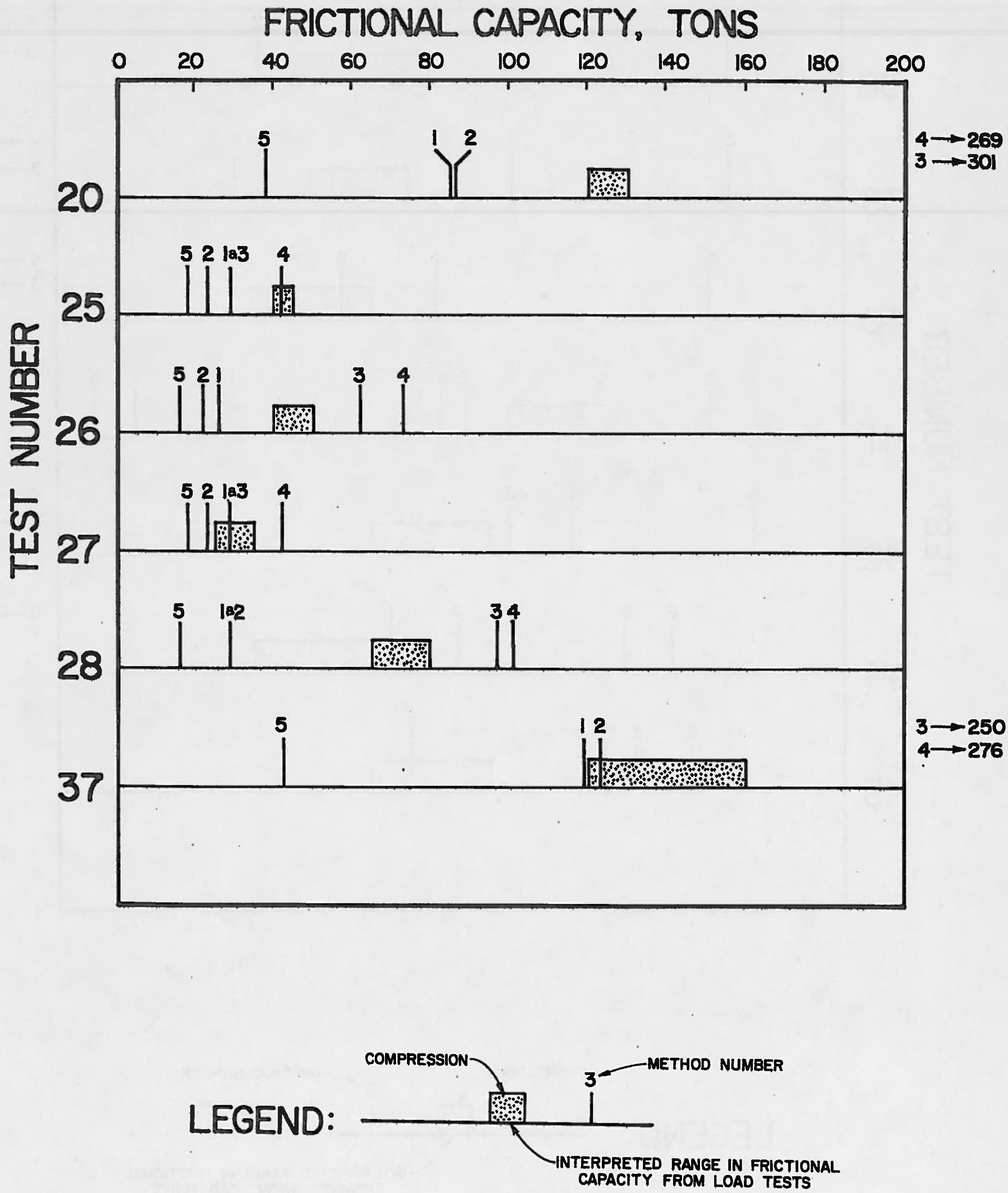


FIGURE 1a COMPARISON OF OBSERVED AND PREDICTED FRICTION CAPACITIES

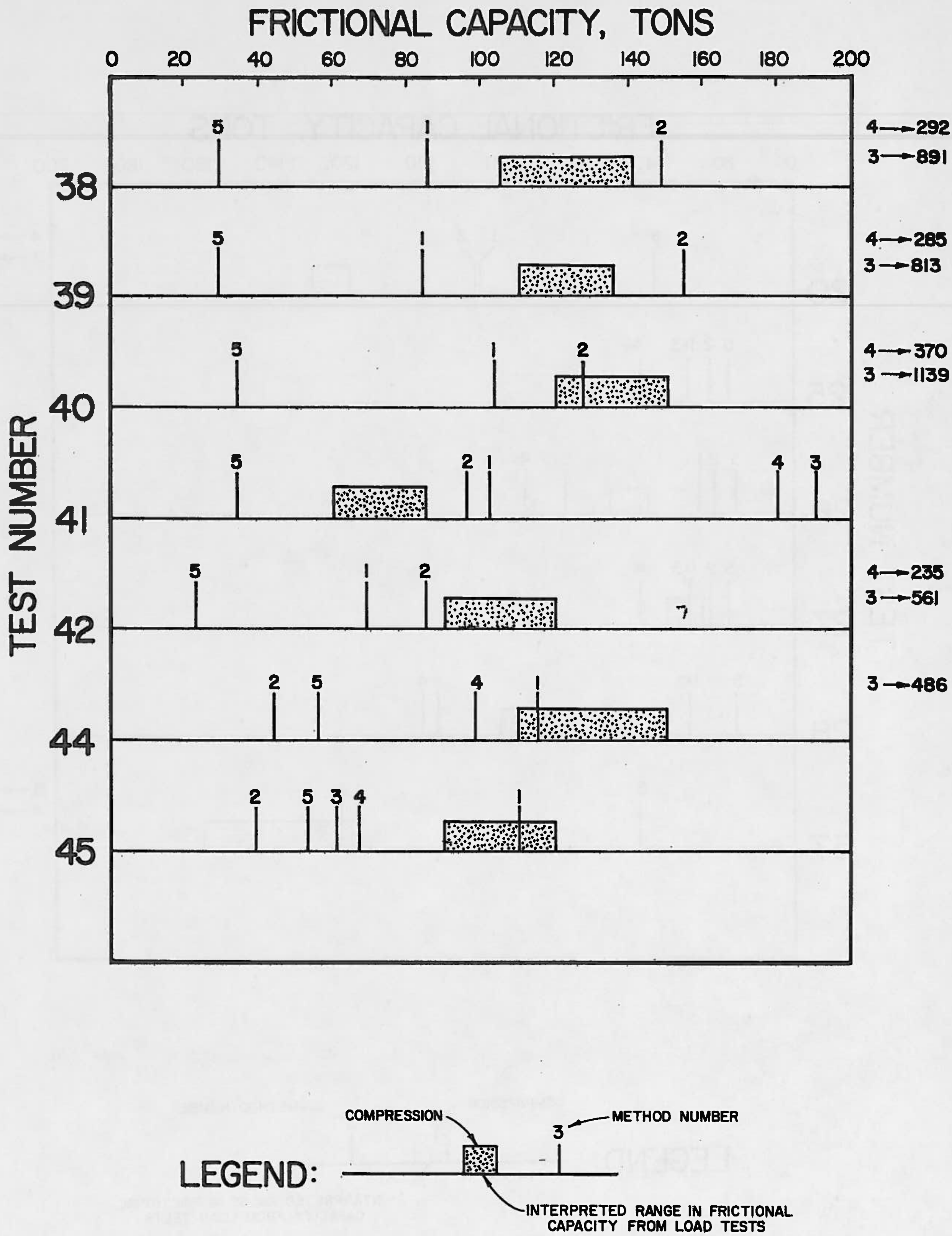


FIGURE Ib COMPARISON OF OBSERVED AND PREDICTED FRICTION CAPACITIES

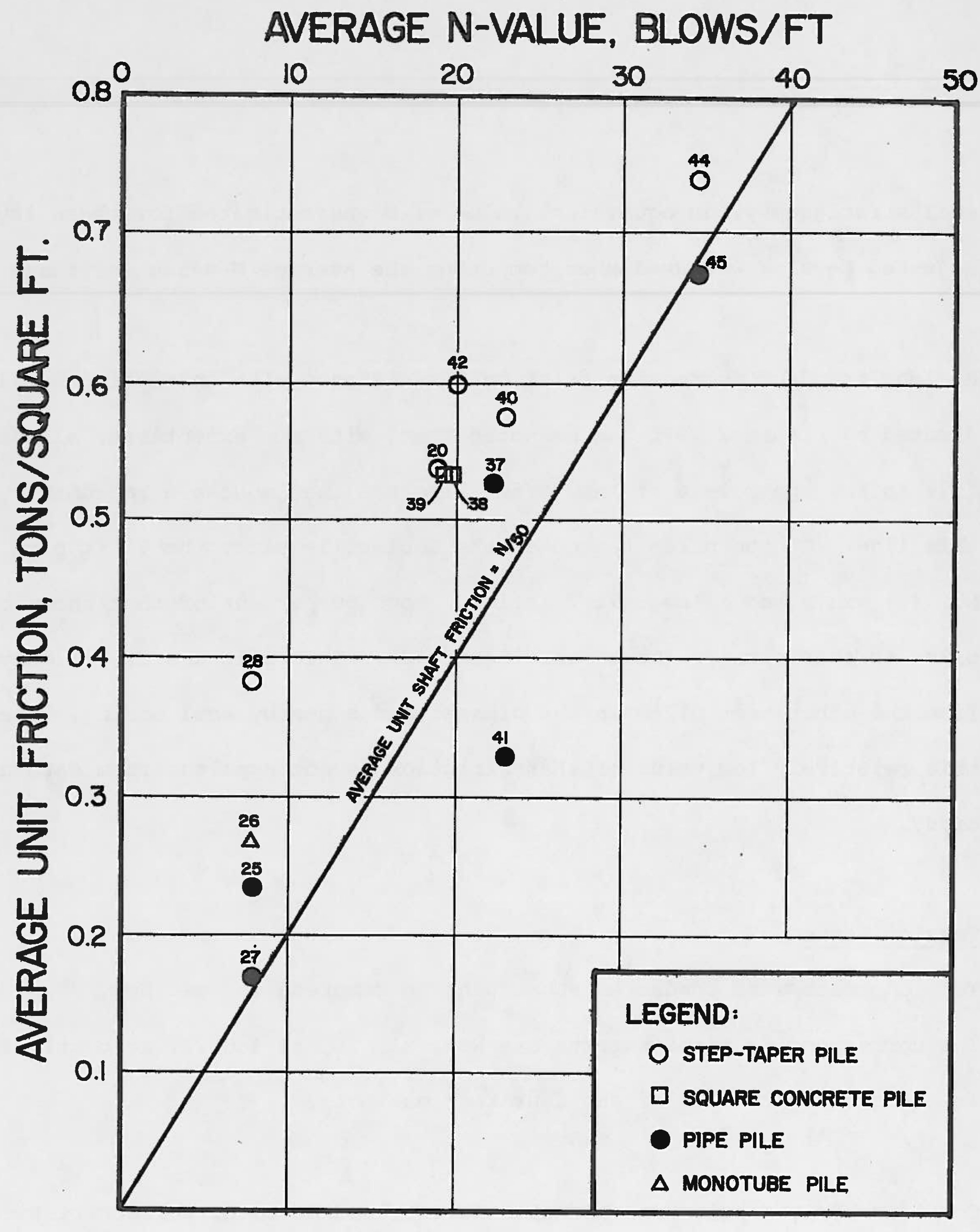


FIGURE 2 AVERAGE N-VALUE vs. AVERAGE UNIT SHAFT FRICTION

soil stratigraphy, an equivalent value of N was estimated for these layers. This adjusted N-value was used when computing the average N-value for these piles.

A clear trend of increasing frictional resistance with increasing N-values is indicated by Figure 2. It can be noted that, with two exceptions, all data points fall to the upper left of the "N/50" line and that most are reasonably close to this line. Of the piles tested at the Louisville site, the 70 ft pipe pile (Test No. 41) exhibited a low unit friction, about 60 percent of that shown by other piles in this cluster (Test Nos. 38 and 40). This pile was driven only about 10 ft from the other test piles in the cluster and a nearby soil boring. The reason for this relatively low value of shaft friction is not apparent from data used in this study.

Tension Tests

Tension tests were conducted subsequent to compression Test Nos. 20, 38, 40 and 41. The corresponding tension tests are Nos. 21, 34, 33 and 35, respectively. Table 3 summarizes the results of the four tension tests.

Considerable variation can be noted in the load-movement characteristics of the four piles when tested in tension. The tests show that a significant portion of the uplift capacity is developed for some piles at movements well in excess of 0.25 in., a deflection which is normally associated with near full development of shaft friction. If, for example, the pullout failure is defined as the load at 1 in. total butt deflection, the 70 ft pipe pile (Louisville, Test No. 35) developed about 80 percent of its ultimate load at a butt movement of 0.25 in., while the 70 ft step taper pile (Test No. 33) at the same site developed only about one-half of its ultimate capacity at this deflection. While neither Test No. 34 nor Test No. 21 were taken to failure, it appears that both piles were capable of developing additional friction beyond a butt movement of 0.25 in. This indicates that rather large movements may be required in order to develop full shaft friction for piles in tension.

The results of tension Tests No. 33, 34 and 35 appear to be reasonably consistent with the corresponding compression tests (Test Nos. 40, 38 and 41, respectively), assuming a 1 in. butt movement is the definition of failure. In each of these cases, the total frictional resistance in tension is between 74 and 85 percent of the mean interpreted friction resistance in compression, which is in general agreement with the findings of Mansur and Hunter (1970). However, when the same comparison is made for these piles considering the load developed at a butt movement of 0.25 in., the uplift load at this deflection is reduced to between 44 and 59 percent of the interpreted compression values. No conclusion in this regard can be drawn for the remaining tension test (No. 21) since the shapes of the load versus butt movement curves for Test Nos. 20 (compression) and 21 (tension) are almost identical.

Static Methods

Method 2 underestimates the interpreted values in most cases. A comparison of ratios of the estimated to observed unit friction ratios for this relationship is shown as a function of depth in Figure 3. A similar plot is shown in Figure 4

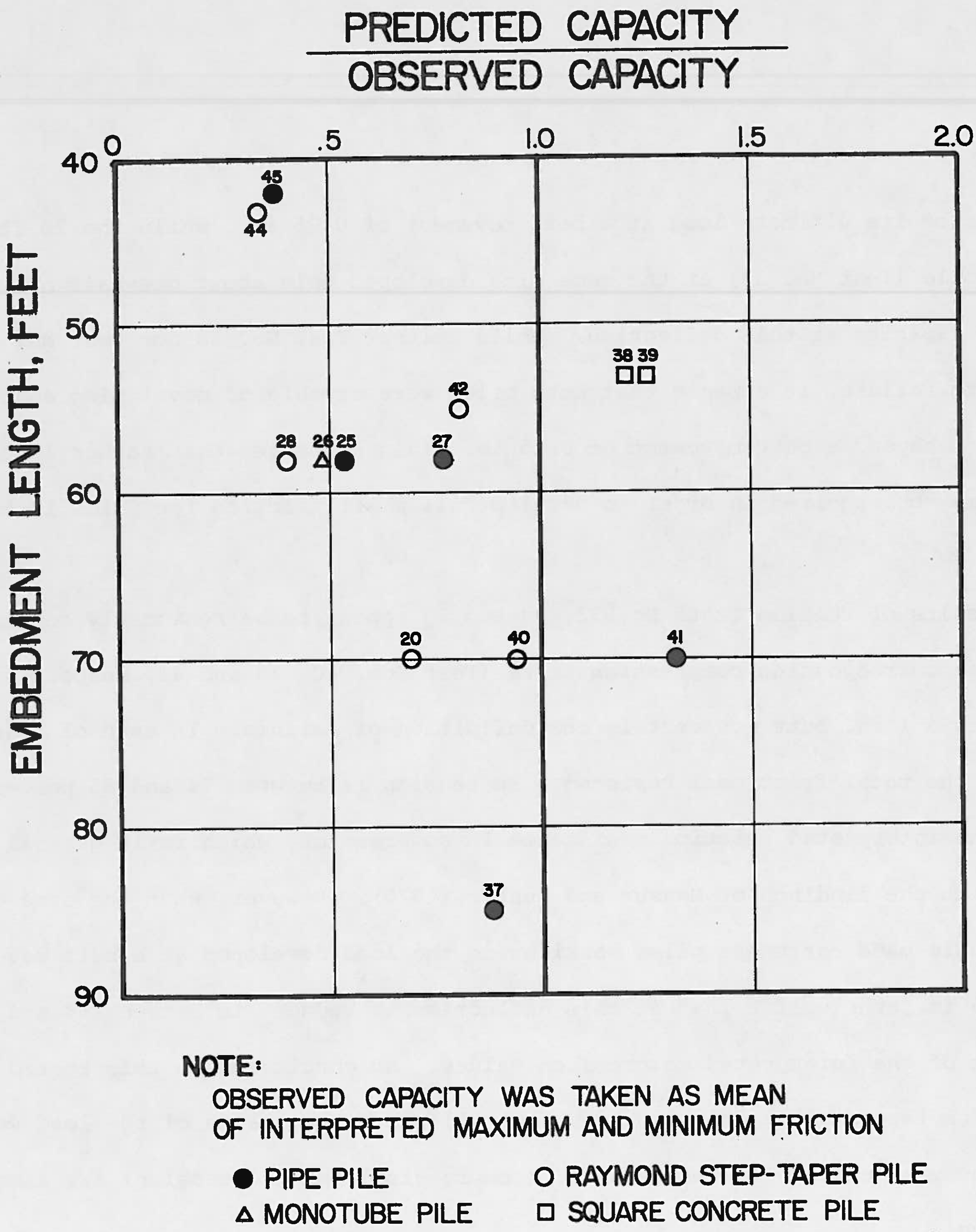


FIGURE 3 COMPARISON OF OBSERVED AND PREDICTED FRICTIONAL CAPACITY FOR METHOD 2

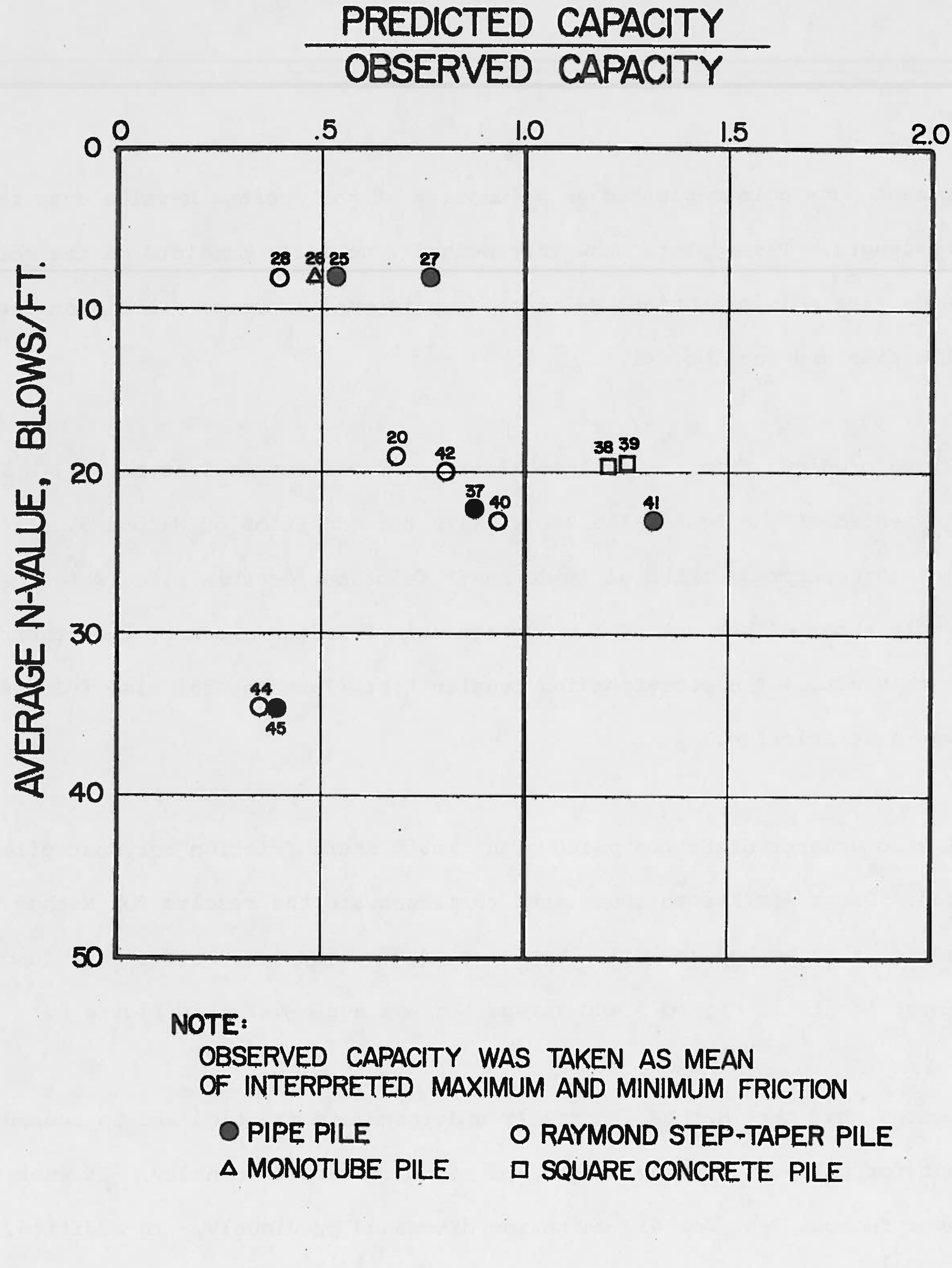


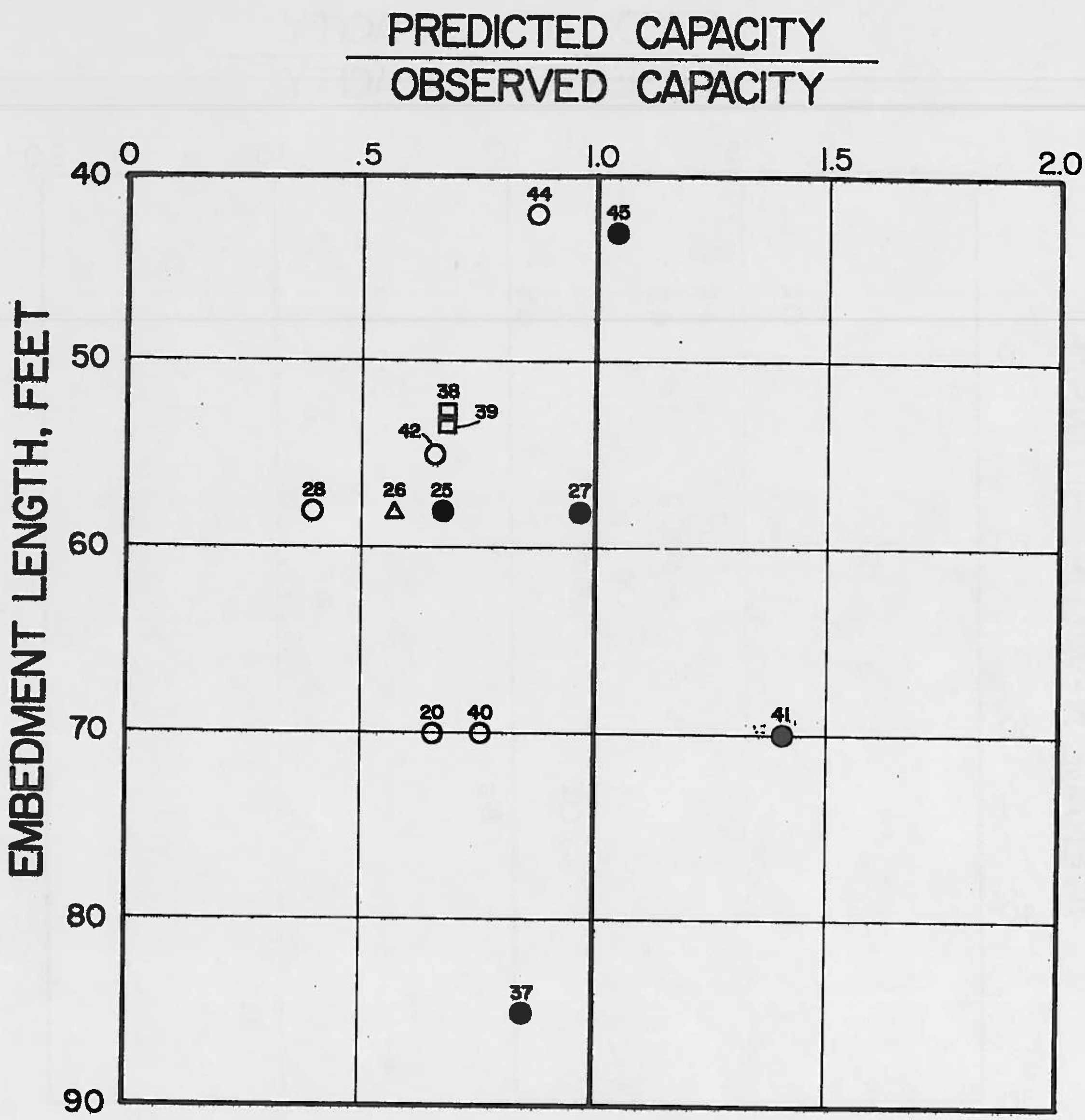
FIGURE 4 COMPARISON OF OBSERVED AND PREDICTED FRICTIONAL CAPACITY FOR METHOD 2

with the same data points plotted as a function of the average N-value over the embedment length. These plots show that Method 2 normally predicts on the conservative side, the only exceptions being the two tapered concrete piles from the Louisville site and Test No. 41.

Method 2 overpredicts friction by approximately 40 percent on Test No. 41, a 70 ft pipe pile tested at the Louisville site. With the exception of Method 5, all the static methods overpredict the ultimate shaft friction for this pile, which developed only about 60 percent of the average unit friction shown by the other test piles at this site. The corresponding tension test (Test No. 35) also indicates a much lower unit friction.

Method 1 also underpredicts the value of ultimate shaft friction for most piles considered. Plots similar to those used to illustrate the results for Method 2 were developed for this method as well. Method 1 predictions are plotted as a function of embedment length in Figure 5 and versus the average N-value in Figure 6.

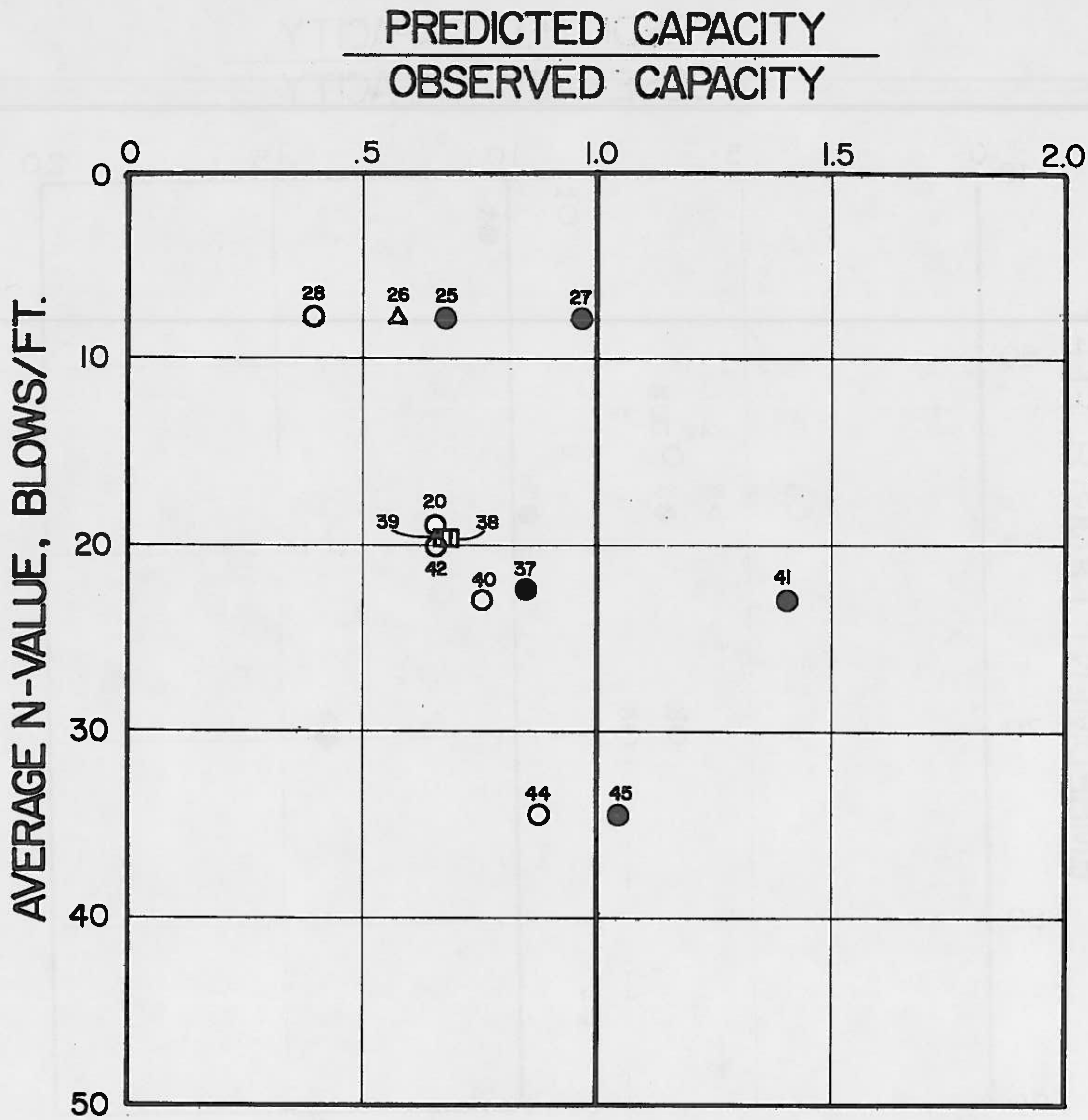
These figures show that Method 1 normally underpredicts friction and is reasonably consistent for piles of varying length and sites of varying density. It substantially overpredicts Test No. 41, which was discussed previously. In addition, this method significantly underpredicts the two tapered piles at Muskegon, a loose sand site. Method 1 gives better correlation at this site for the two pipe piles than for the tapered piles. This is expected since the influence of taper is not included in this method. The influence of taper is not reflected to this extent



NOTE:
OBSERVED CAPACITY WAS TAKEN AS MEAN OF INTERPRETED MAXIMUM AND MINIMUM FRICTION.

● PIPE PILE
△ MONOTUBE PILE
○ RAYMOND STEP-TAPER PILE
□ SQUARE CONCRETE PILE

FIGURE 5 COMPARISON OF OBSERVED AND PREDICTED FRICTIONAL CAPACITY FOR METHOD I



NOTE:
OBSERVED CAPACITY WAS TAKEN AS MEAN OF INTERPRETED MAXIMUM AND MINIMUM FRICTION

● PIPE PILE
○ RAYMOND STEP-TAPER PILE
△ MONOTUBE PILE
□ SQUARE CONCRETE PILE

FIGURE 6 COMPARISON OF OBSERVED AND PREDICTED FRICTIONAL CAPACITY FOR METHOD I for the Palatka or the Louisville sites, where the average N-values were significantly greater.

It is interesting to note (in Figure 5) that Method 1 gives reasonable predictions for long piles (Test Nos. 20, 37 and 40). Since this relationship depends only on the Standard Penetration Test results (no limiting friction value with depth is enforced), the results imply that if friction is indeed more or less constant below a certain pile penetration, then this factor may be directly reflected by the Standard Penetration Test result.

The predicted values of ultimate shaft friction using Method 3, in general, exceed those produced by the 13 compression tests considered. The discrepancy is most pronounced for long and/or tapered piles. For example, in the case of the Louisville piles, which are all either tapered or over 60 ft in length, or both, Method 3 typically overpredicts by several hundred percent. This appears to be primarily due to the absence of a limiting value of unit friction for this method. In addition, where ϕ is large and the pile tapered, the values of the coefficient of lateral pressure (K_δ) become very large.

Method 4 produces better correlation with the load test data considered; however, this method also generally overpredicts. By limiting the expression, $K_\delta \sin\delta$, to unity, the large values of unit shaft friction are reduced considerably. This is particularly significant for dense sites and for tapered piles. Method 4, however, allows the value of unit friction to increase indefinitely with depth. This is the primary reason that this method overpredicts by the greatest margins for long piles (i.e., Test Nos. 20, 37, 38, 39 and 40).

Method 5, which expresses friction as a function of relative density, typically predicts frictional capacities that are substantially less than those interpreted from the 13 compression tests. In fact, the estimates were generally less than half of the observed values. It was later suggested by Vesic that the computed values of unit shaft friction should be increased by 50 percent. The test results reported herein imply that an even larger increase may be in order.

Conclusions

1. The data presented indicates that none of the methods considered for predicting shaft friction for piles driven into sand accurately accounts for all the factors bearing on this problem. The summary presented in Figure 1 illustrates that, depending on pile type, length, sand density, gradation, etc., any of the five methods studied can either underpredict or overpredict the frictional load carrying capacity of a pile.

2. Of the static design procedures investigated, predictions of shaft friction based on Methods 1 ($f = N/50$) and 2 ($f = K\ P_v \tan\delta$, with values of K and δ as presented herein) provide the best correlation with the interpreted test data for the pile load tests used in this study. These methods predict shaft friction values which are generally conservative.

3. Methods 3 (Nordlund) and 4 (Michigan formula) were found to overpredict the frictional capacities for most piles studied, particularly those which were long

and/or tapered. This appears to be due primarily to the unlimited increase in unit friction with depth employed by these methods. It is also possible that these procedures overestimate the beneficial effect of pile taper.

4. Predictions based on Method 5 (Vesic) were, in most cases, well below the shaft friction values interpreted from the load tests. A constant multiplier applied to this equation would produce better correlation with test data.

5. While there is some scatter in the data and the data base somewhat limited, there are indications from the piles tested in tension that shaft friction continues to increase for butt movements of up to 1 in.

6. While this paper provides useful comparisons of several static methods of prediction with load test data of the types typically available to a geotechnical engineer, it sheds little light on the vital question of load transfer. Much additional research is needed on instrumented piles if the load transfer mechanism for friction piles in sand is to be understood.

ACKNOWLEDGEMENTS

The authors are grateful to the Louisville Gas and Electric Company, Rust Engineering Company and Black and Veatch, Consulting Engineers, for the use of their data and to the numerous other concerns who donated data that were not included in the paper. Thanks are also due Mr. Mark Erwin for his assistance in carrying out the many calculations required for this study. The paper was reviewed by Messrs. Peter

Bergman, Don Bryenton and Richard Laughlin. The manuscript was typed by Ms. Terra Nicholas and the drawings prepared by Mr. Mike Probst.

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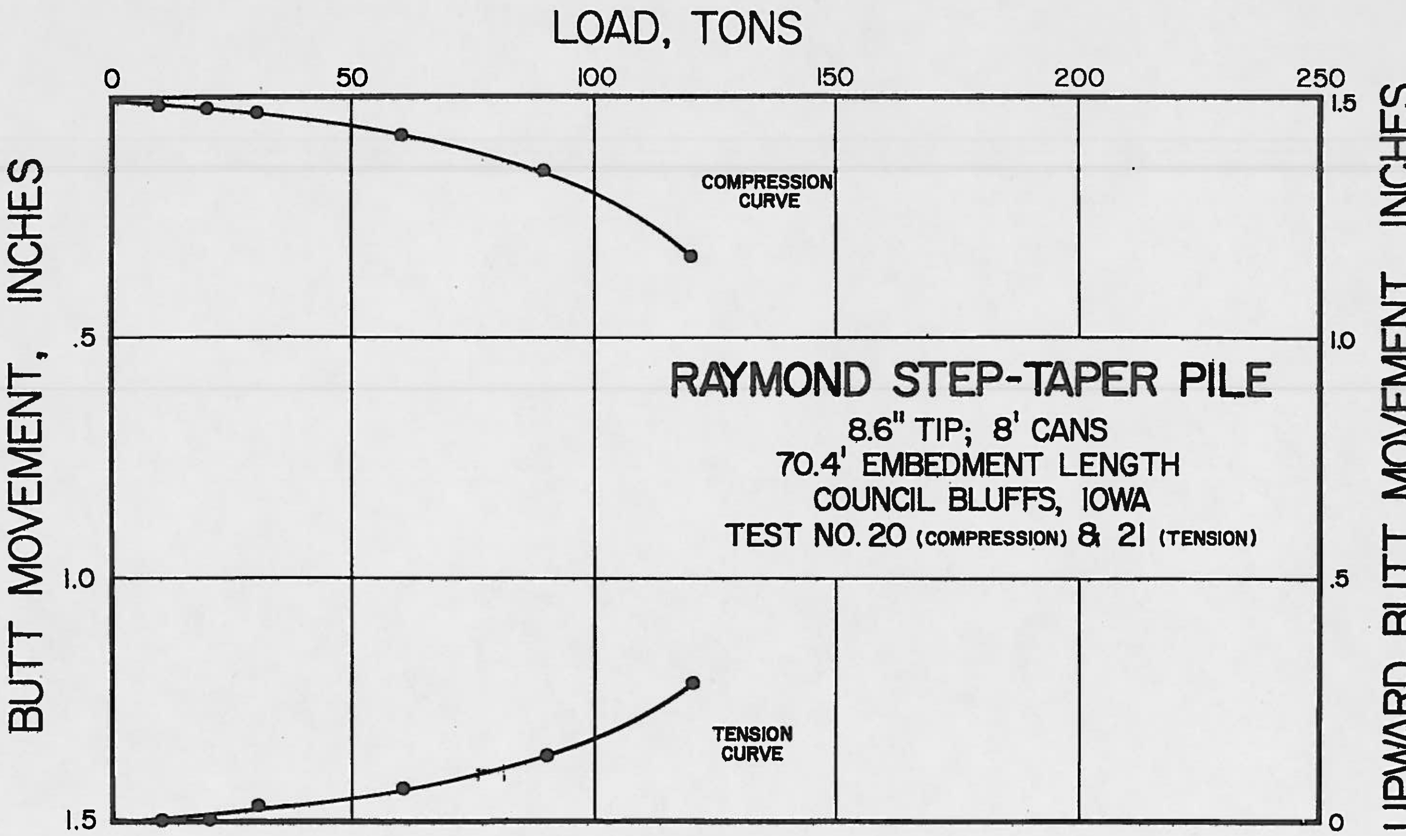
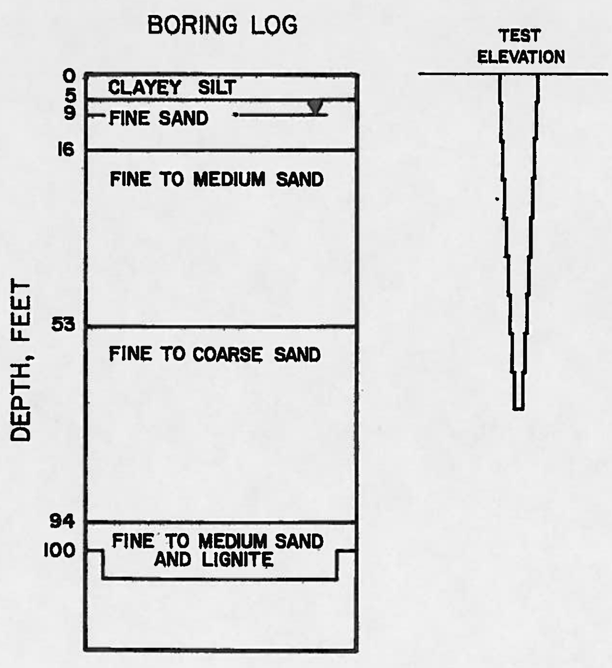
APPENDIX





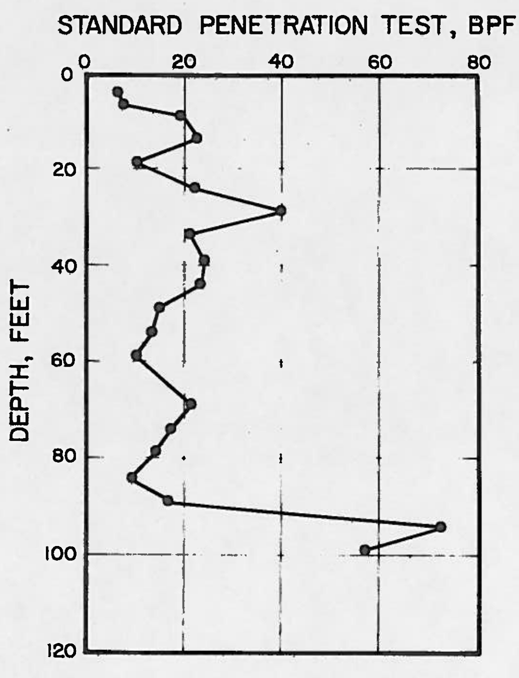
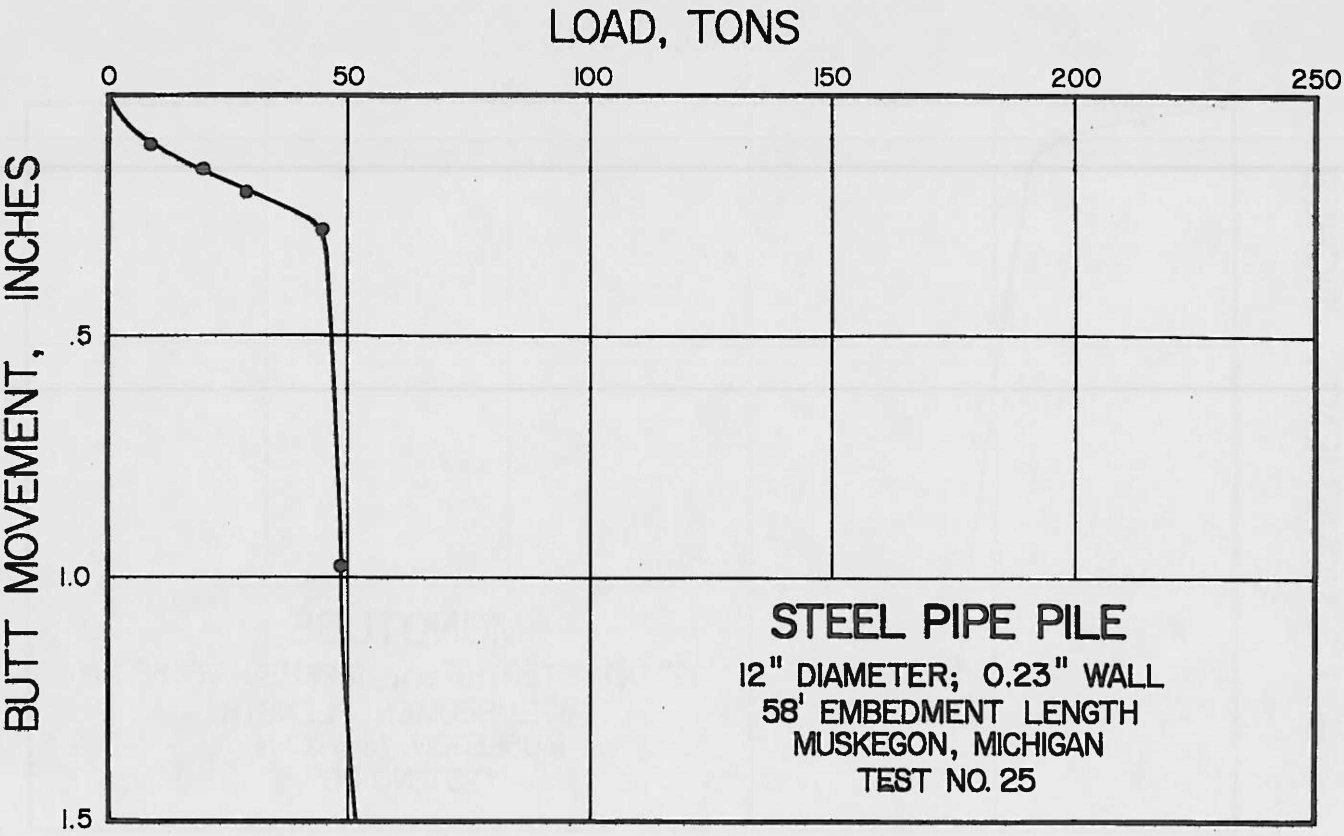
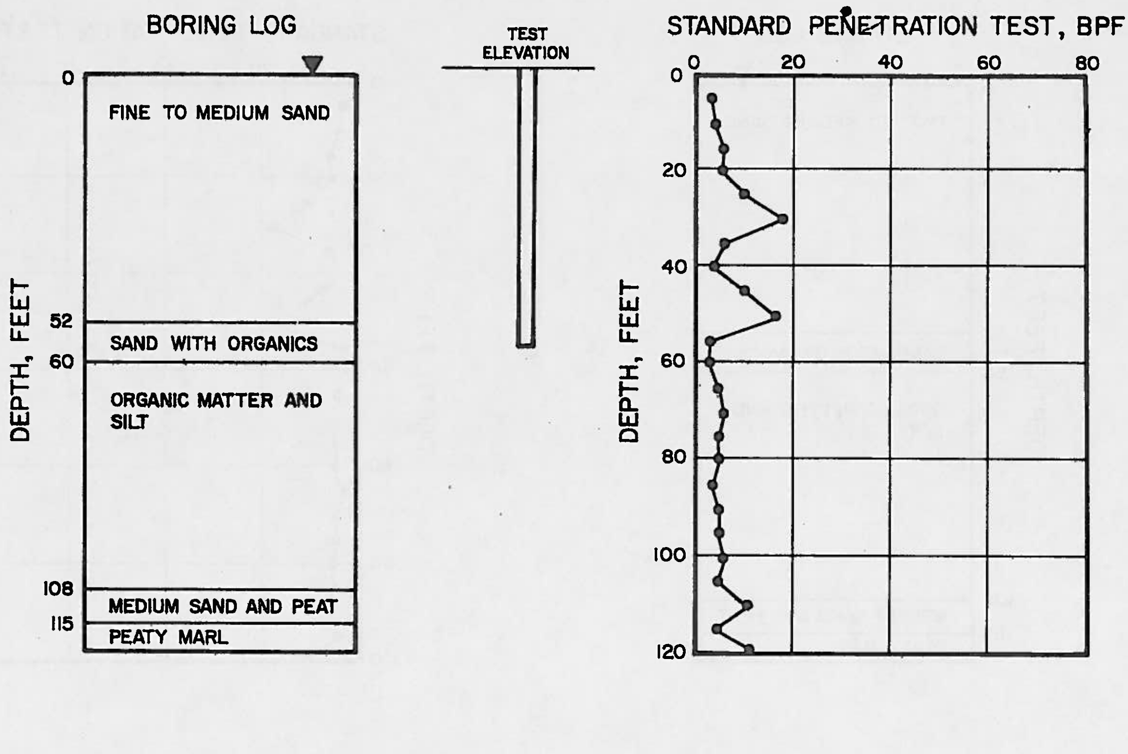


FIGURE A1 PILE LOAD TEST RESULTS





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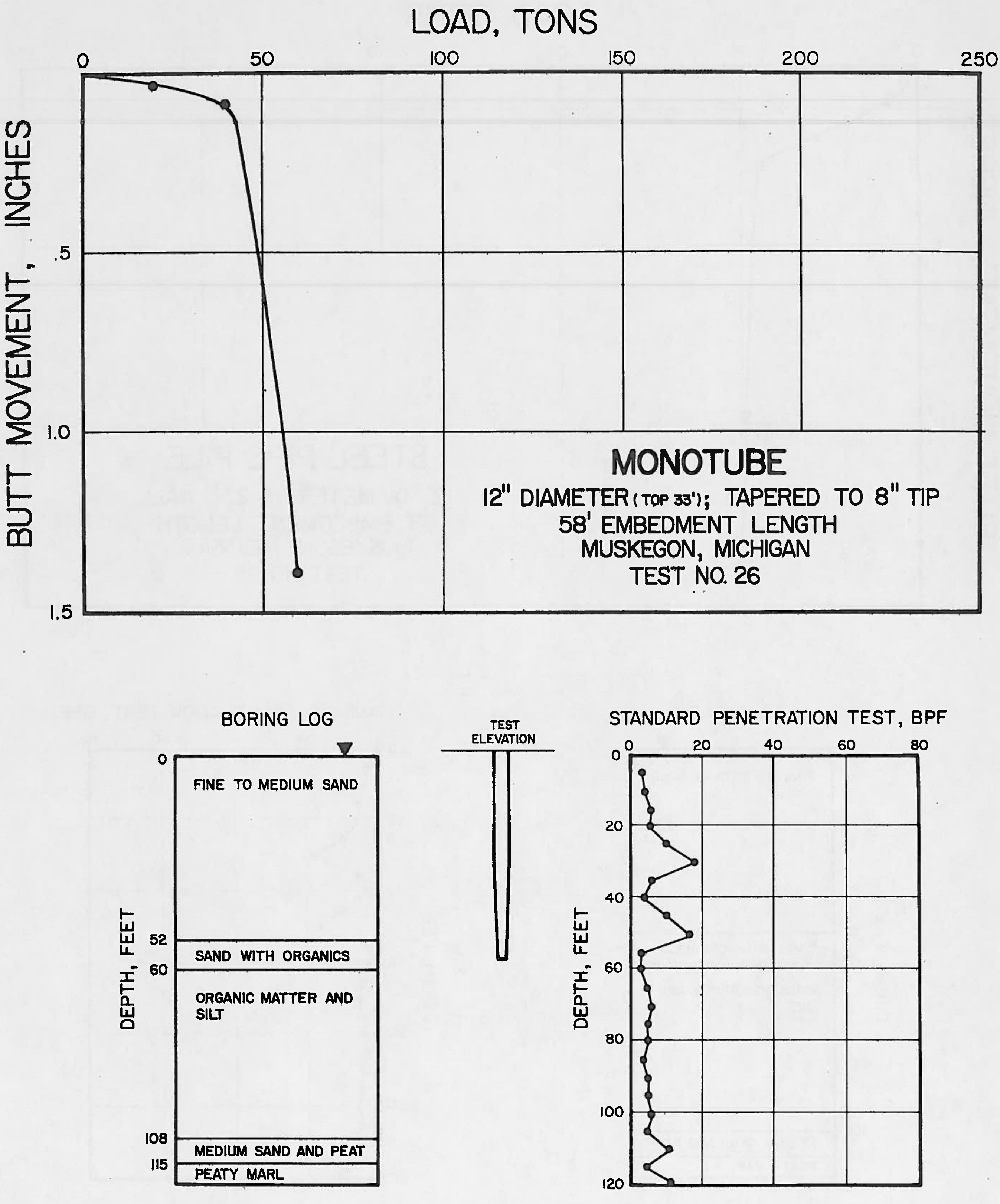

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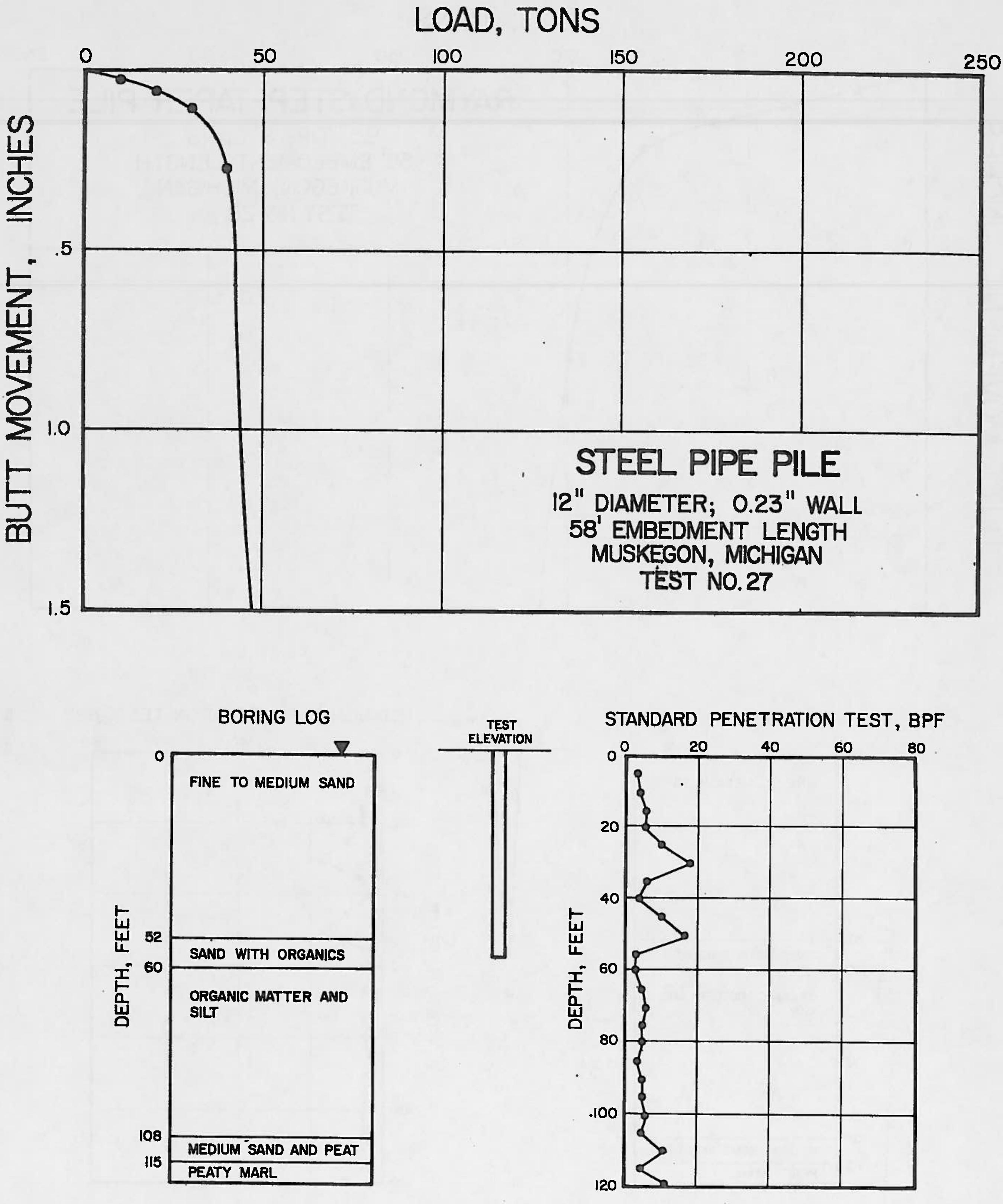

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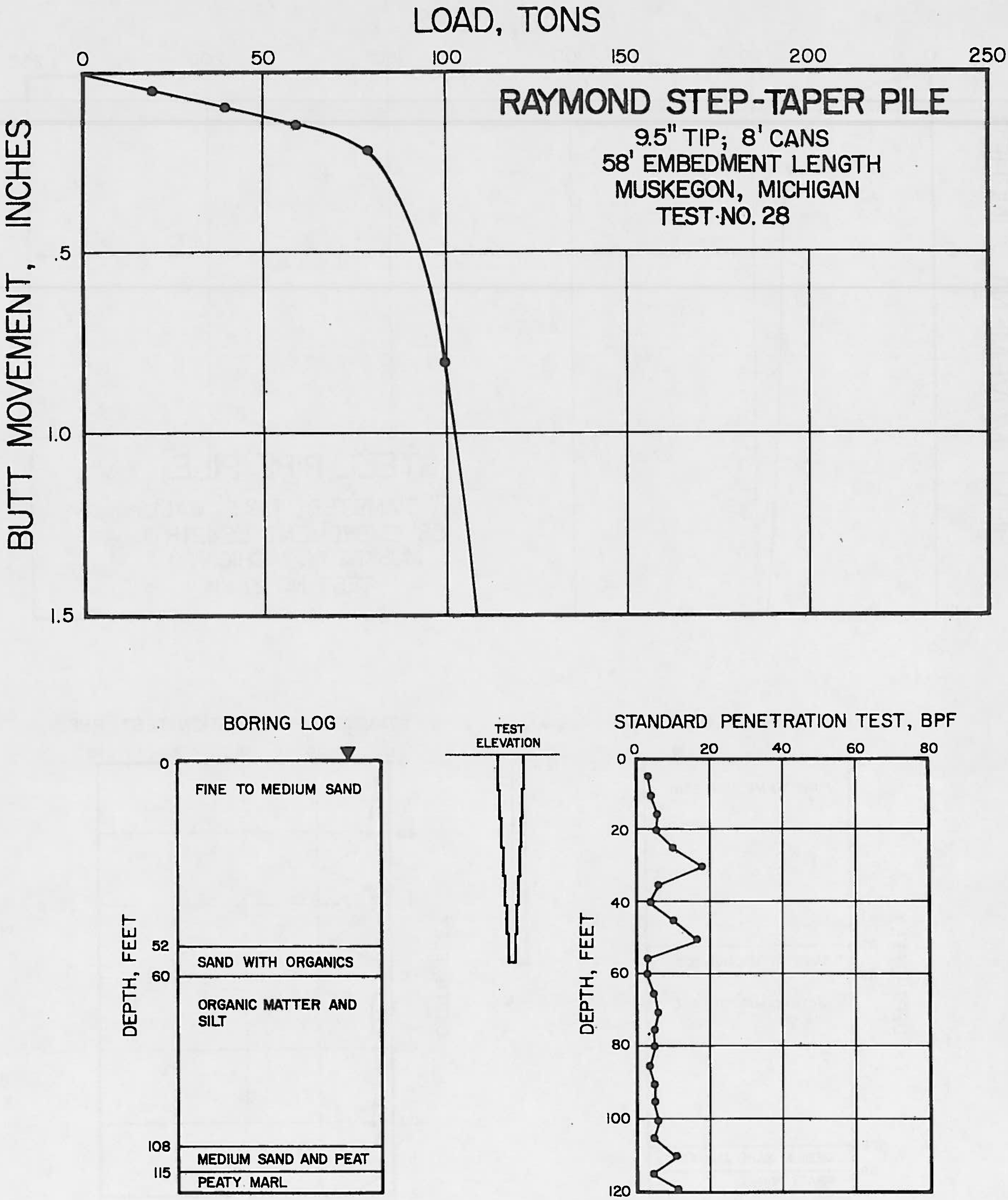

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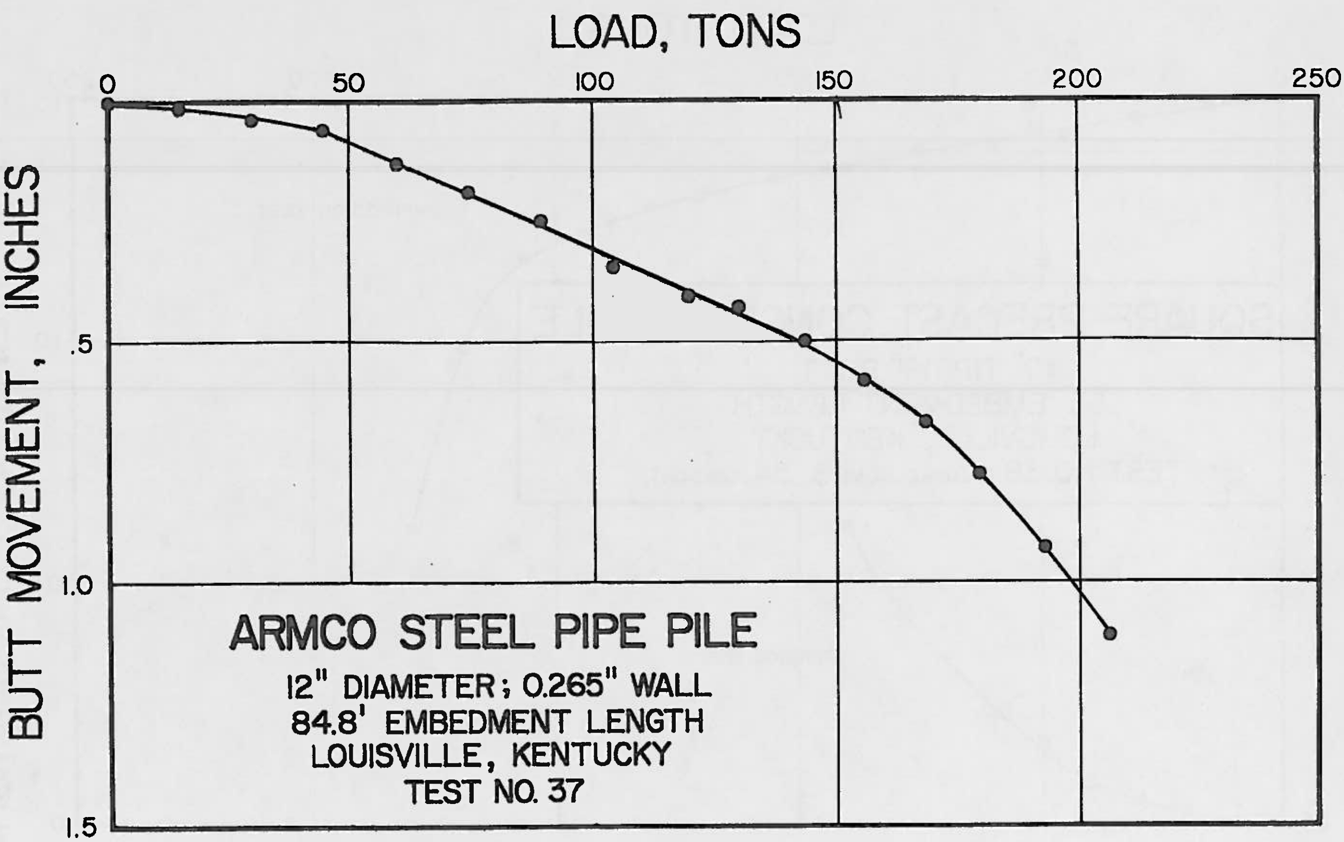

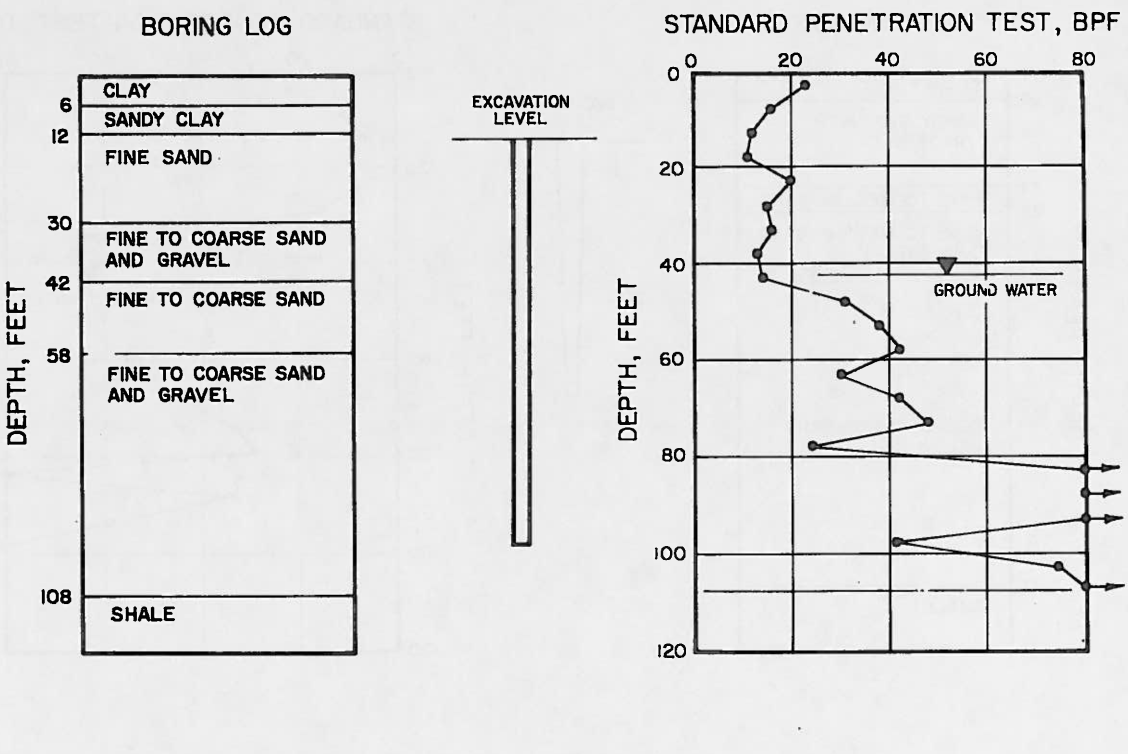

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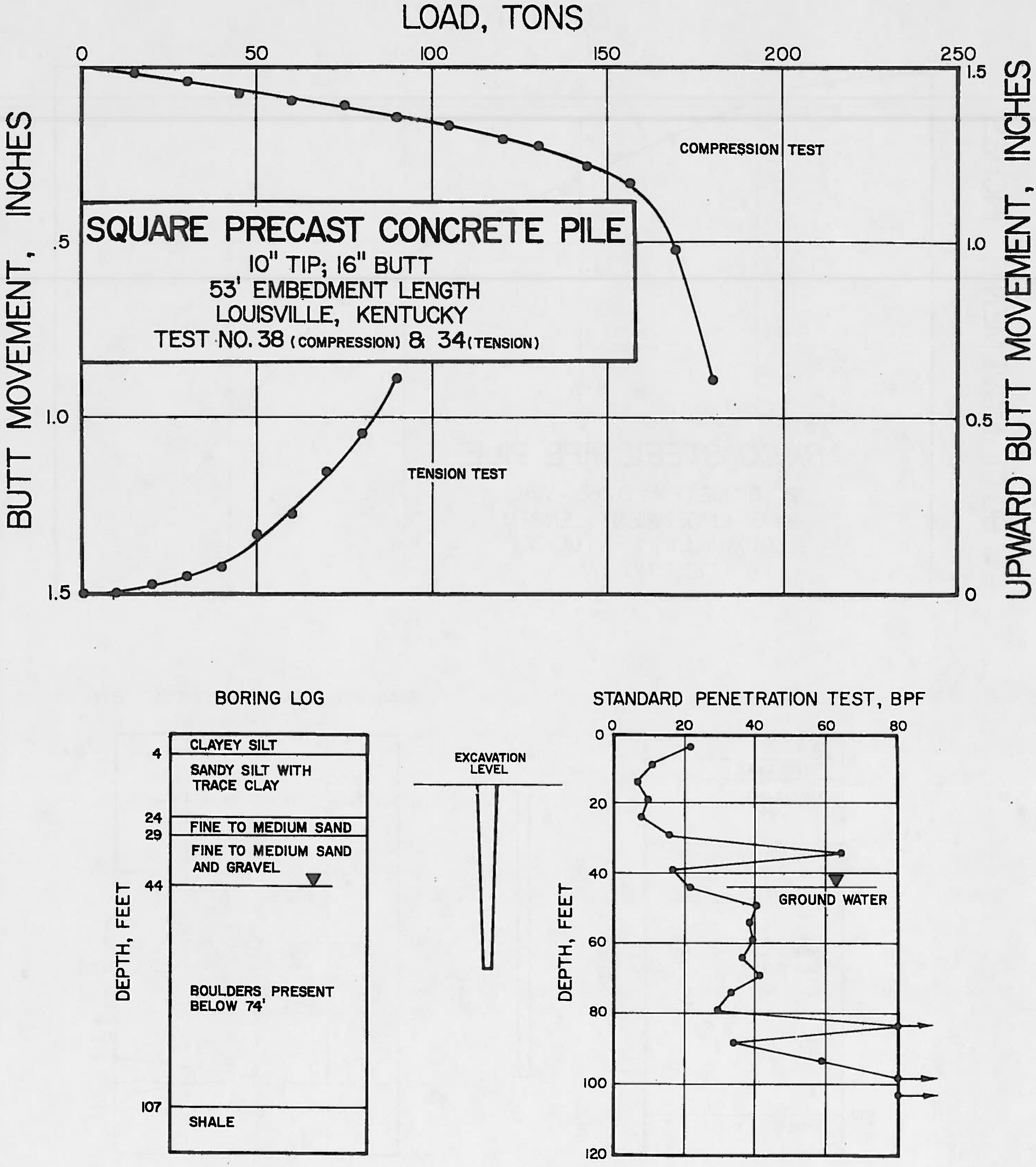

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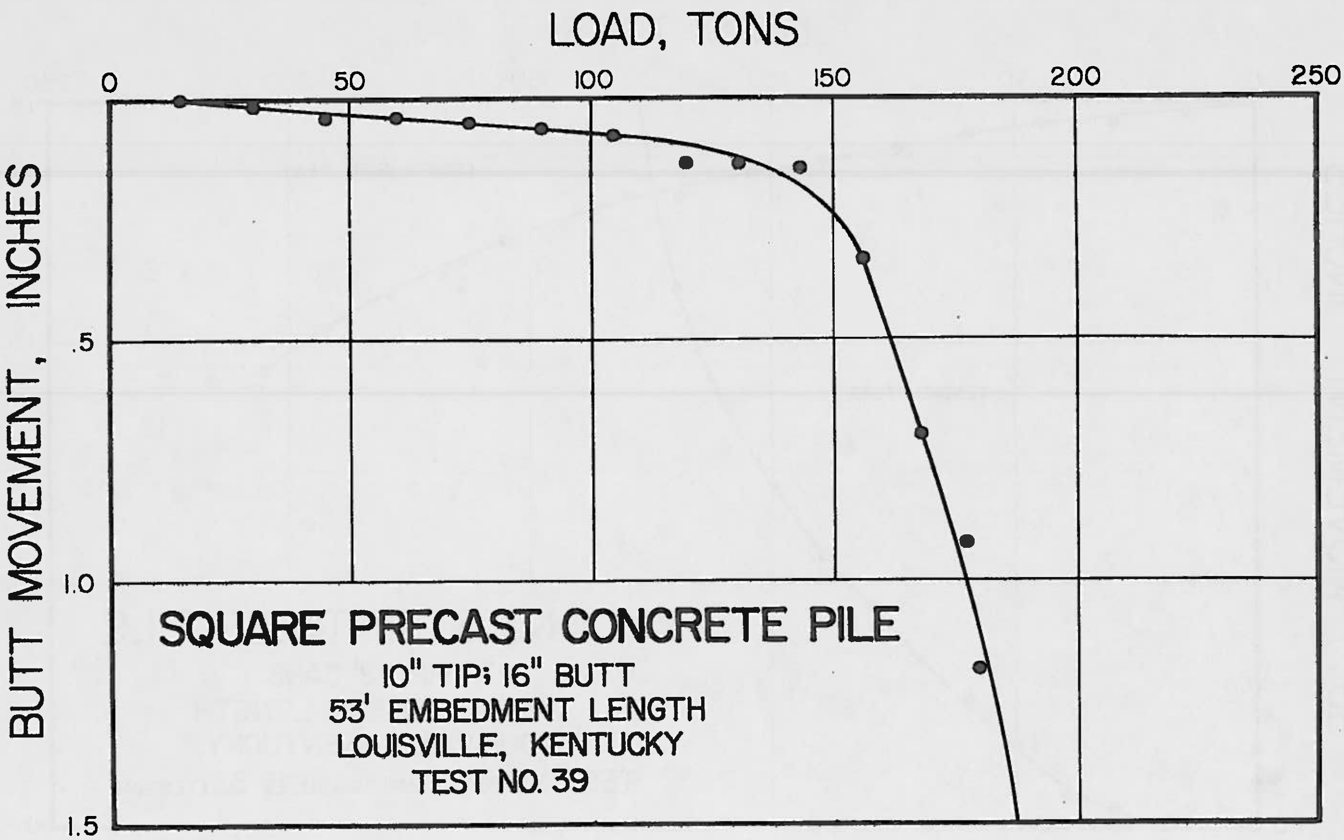

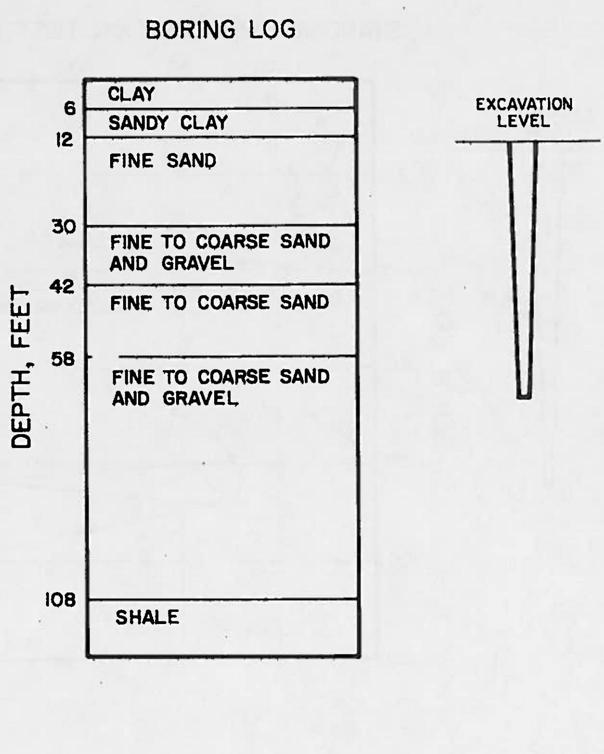

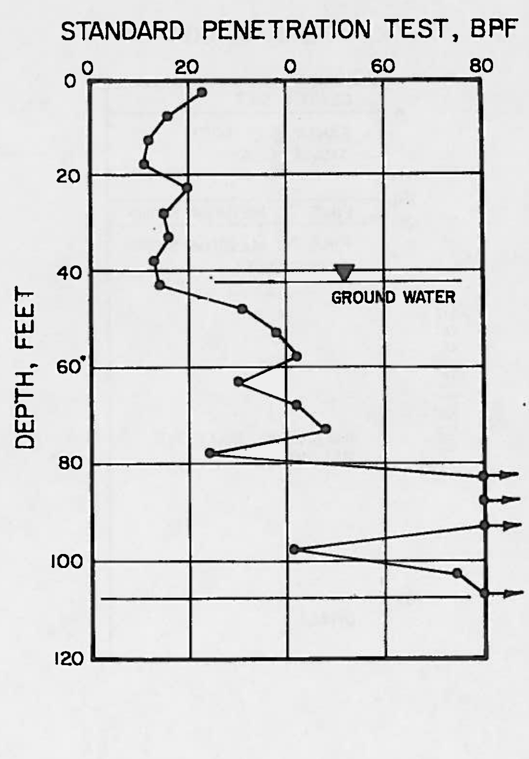

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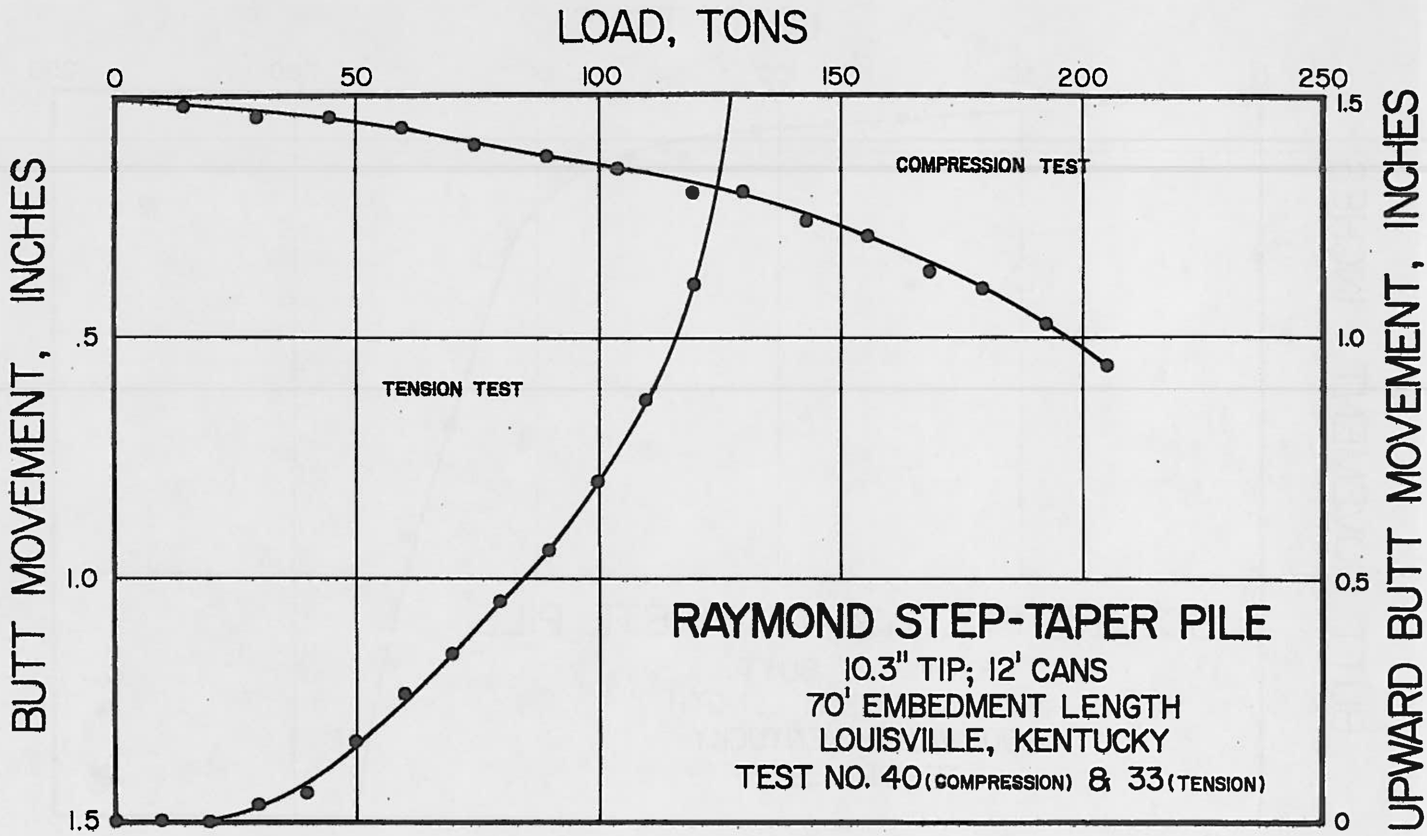

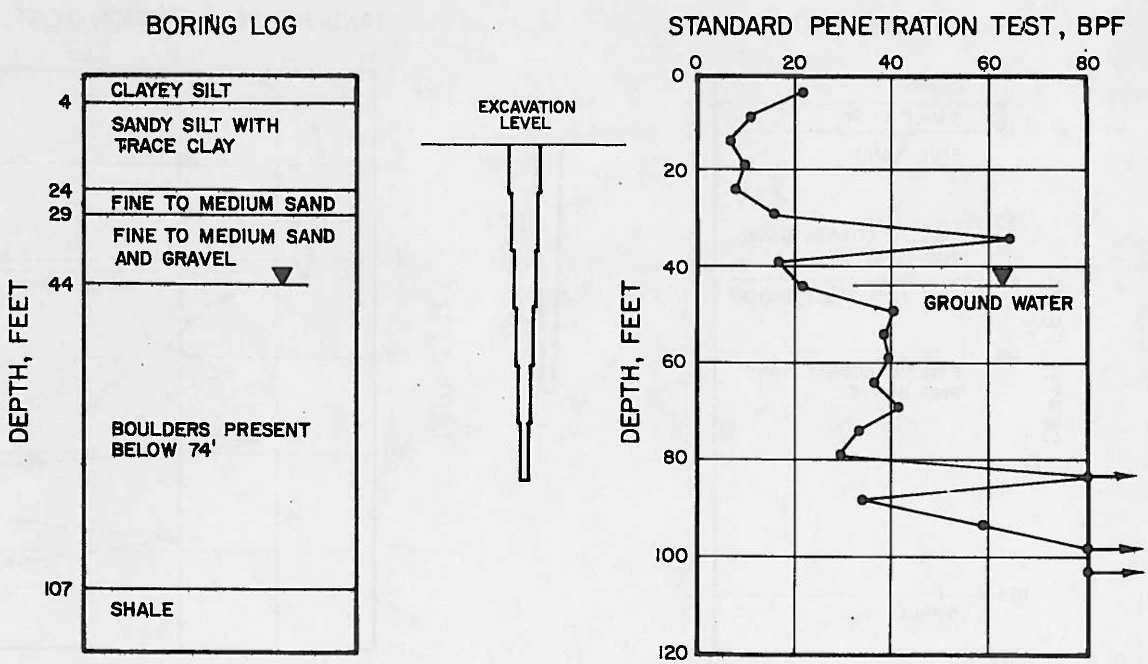

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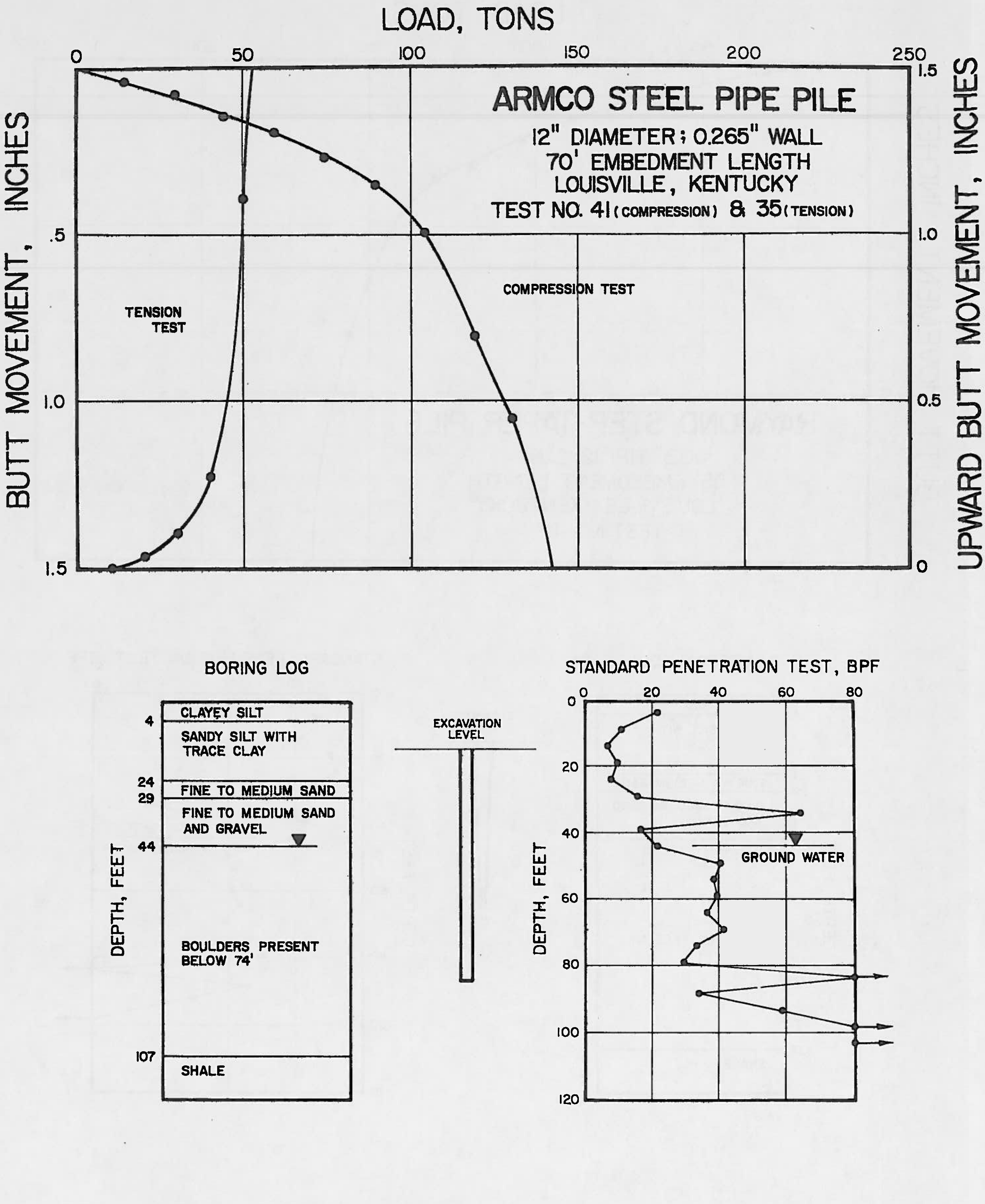

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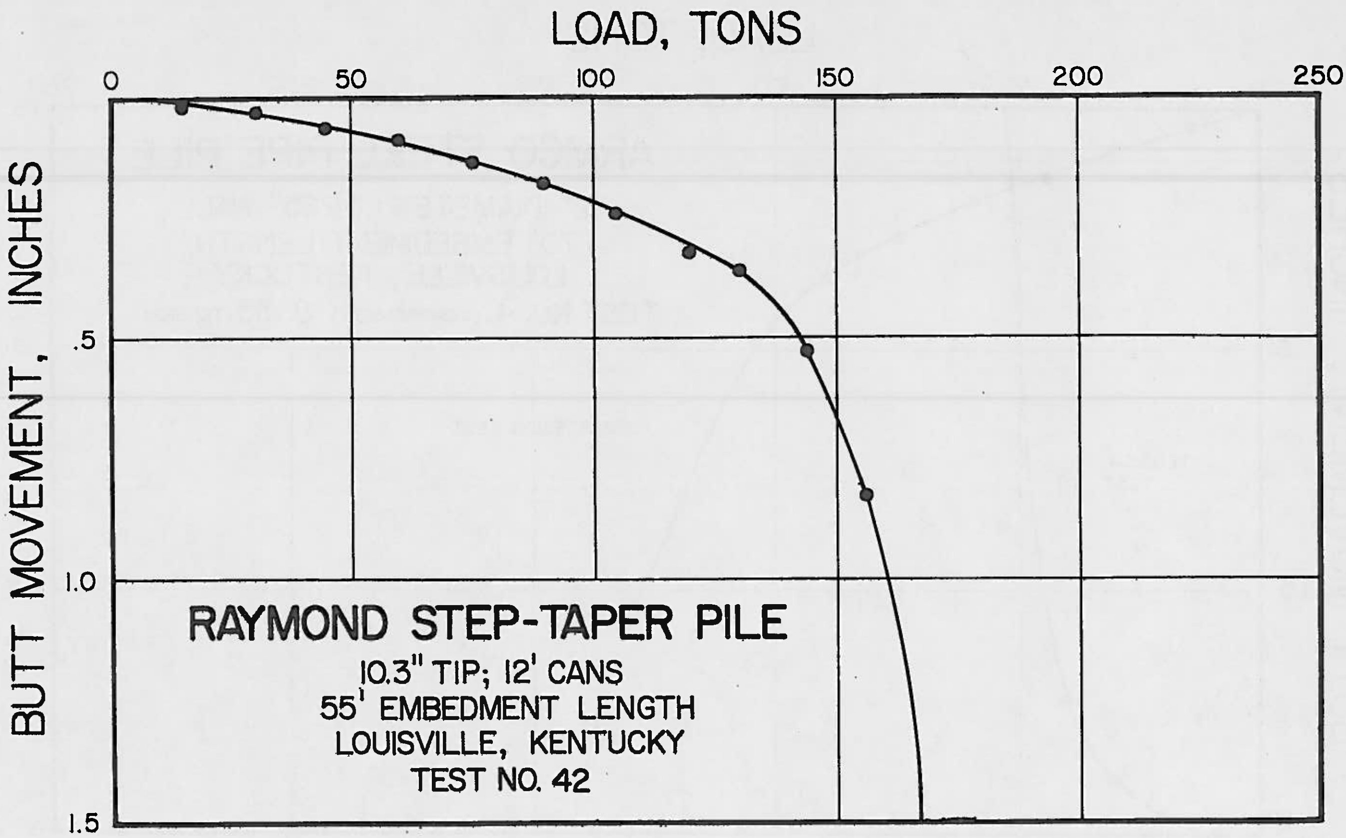

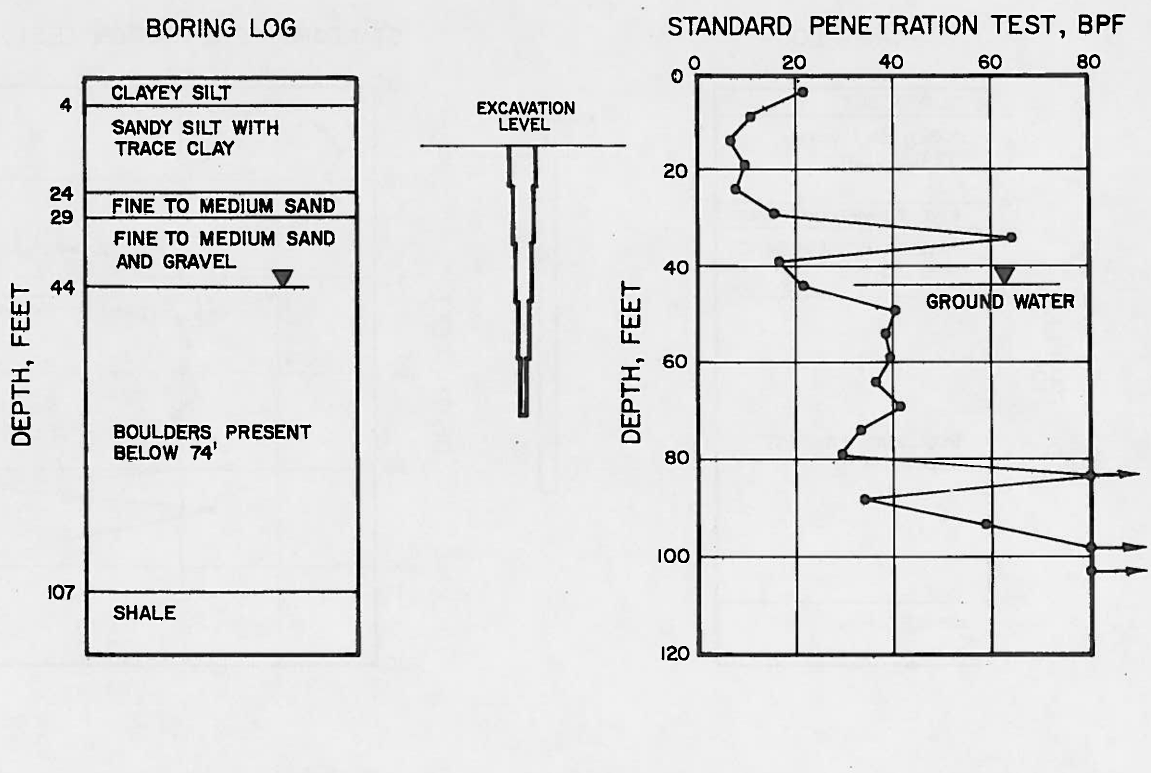

FIGURE A11 PILE LOAD TEST RESULTS

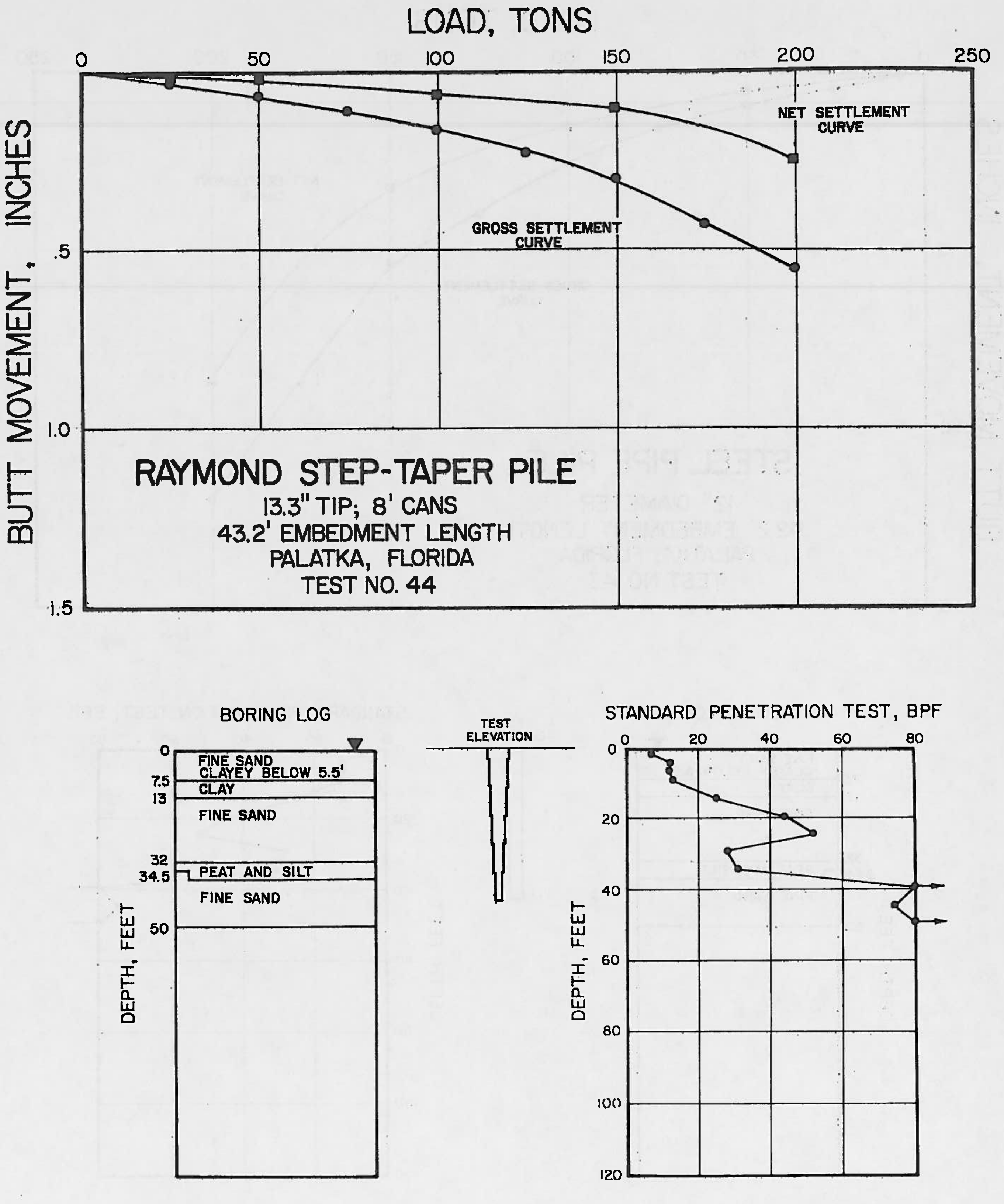

FIGURE A12 PILE LOAD TEST RESULTS

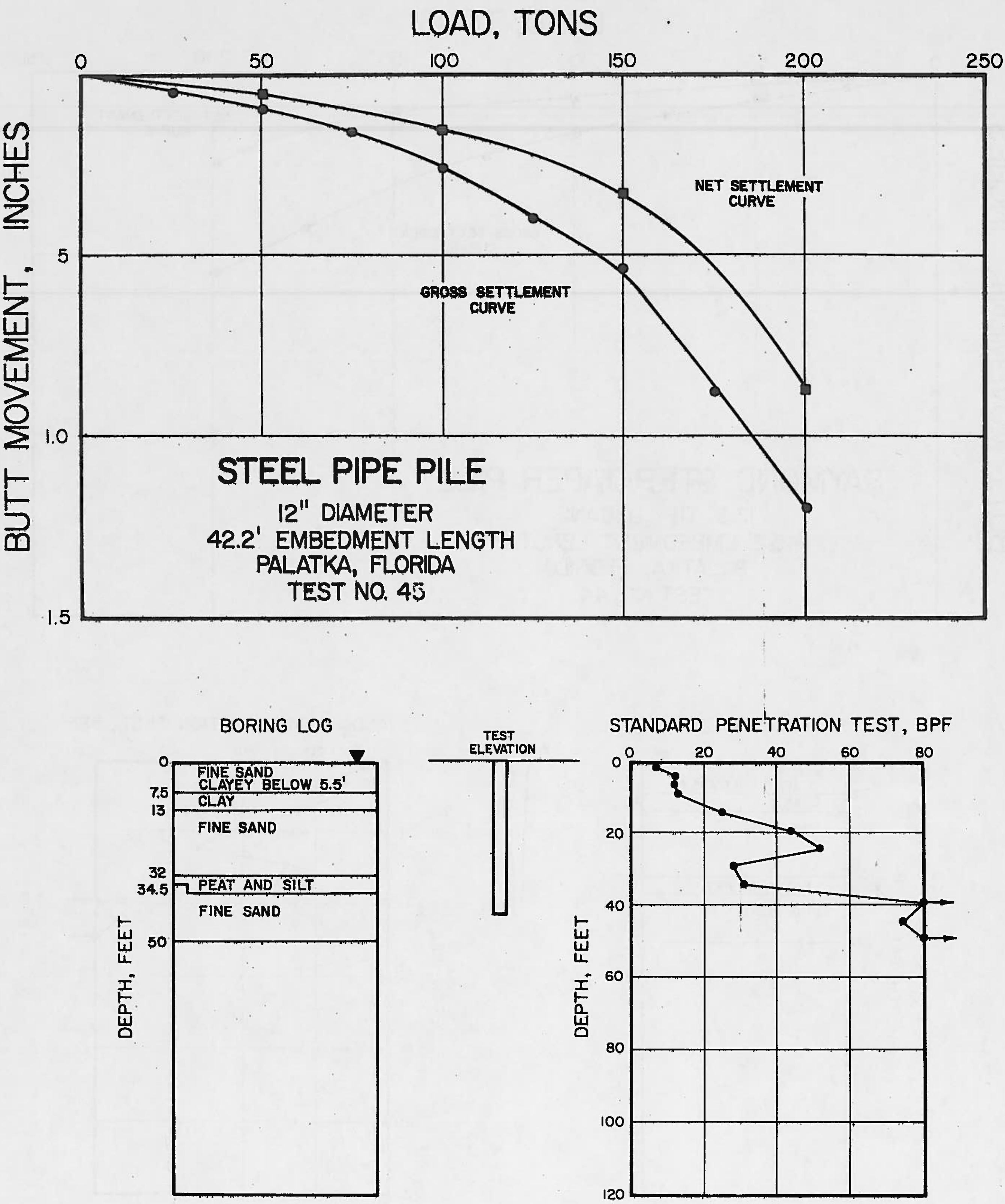

FIGURE A13 PILE LOAD TEST RESULTS

A PILE DESIGN AND INSTALLATION SPECIFICATION BASED ON THE LOAD FACTOR CONCEPT

by

G. G. Goble*

Introduction

Beginning about two decades ago, dramatic changes began to occur in structural design philosophy. Prior to that time in most of the twentieth century, the structural design activity sought to develop a structural system that would resist the effects of an expected load application with no structural distress. This was achieved by requiring that the stresses calculated from an elastic analysis of the structure when subjected to the expected design or working loads not exceed some accepted, allowable stress. These allowable stresses were usually defined either explicitly or implicitly as a fraction of the yield or ultimate strength of the material involved.

There are clear advantages to the above approach. Since the structure is subjected to an elastic analysis and the limit on allowable stresses is placed well below the elastic region, it can be expected that even though the structural engineer is primarily concerned with the design of a structure having sufficient "strength", many serviceability questions will be satisfied indirectly. For instance, one can expect in such an approach that deflections will be tolerable and acceptable. The structure is subjected to elastic analysis and, therefore, indirectly deflections are controlled.

Another important but less understood advantage of an elastic analysis and a working stress approach is that there is a clear and simple redesign process available to the structural engineer. Those portions of the structure which are found to be overstressed in the analysis can be increased in size while other parts of the structure where stresses are less than the allowable can be decreased in size. This approach provides a simple redesign algorithm.

There are also important disadvantages in working stress design. For instance, a statically indeterminate structure having a high degree of redundancy will have a different factor of safety to collapse than will a statically determinate structure. When such structures are designed by working stress procedures, the actual factor of safety for particular structures can be quite variable. Since the loads that must be carried by the design can come from a variety of sources (wind, live, dead, etc.) the accuracy and reliability of our

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*Chairman, Department of Civil and Architectural Engineering, University of Colorado, Boulder, Colorado.

determination of their magnitude can differ widely. There are other considerations which motivate the change in practice. For instance, the behavior of reinforced concrete members does not satisfy working stress analysis due to time dependent and inelastic deformations that usually occur.

On the other hand, if working stress analysis is completely abandoned for an exclusively strength-design based procedure, then difficulties can arise with other performance aspects of the structure. With the usual strength evaluation procedure one completely neglects questions of deflection.

As a solution to these kinds of problems a procedure known as load factor design has evolved which is becoming increasingly accepted in the design of various kinds of structural elements. This procedure deals very directly with the questions involved in structural design. The structure is designed to satisfy the requirements of strength and serviceability, directly and separately. By serviceability in structural design we are referring to such considerations as deflections, long term deformations, vibrations, corrosion control, and a variety of other such influences.

Strength considerations are solved directly by insuring a specific factor of safety against a collapse. This factor of safety, however, can be quite variable since the method recognizes that under different conditions different factors of safety are appropriate. For instance, if the magnitude of the load applied to a structure is very well known, then it seems reasonable that a smaller factor of safety can be used than when the load magnitude might be quite variable.

Other factors which come into such a design procedure include considerations of the reliability of member performance. As an example, the flexural behavior of an under-reinforced concrete beam can be accurately predicted and furthermore, the member will show a substantial deflection prior to losing the capability to carry a small amount of increasing load. It gives a strong warning of impending failure. On the other hand, the same material in a reinforced concrete column will exhibit less ductility and give far less warning of failure. It is appropriate in the first case that the factor of safety be smaller than in the second case.

This kind of an approach to design is particularly well suited to the design of pile foundations. In fact, it may be very well suited to all kinds of foundations. Only pile foundations will be discussed here.

Let us consider one further problem currently faced by the structural designer when he approaches the design of either a pile supported foundation or a spread footing. As elements are proportioned, usually from the top of the structure downward, the loads are collected and carried along. At the base of the structure the foundation loads have been collected. However, these loads, derived from the structural design, will be in the form of a factored load to be applied to the ultimate strength of the foundation. But, current practice requires that soil limitations be handled in terms of working loads, so working loads appropriate to the design of this particular element must be assembled. After the allowable soil loads imposed by the foundation engineer are satisfied, the design of the footing element itself must be accomplished using a load factor procedure. This approach is not only inconvenient, but it lacks a great deal in philosophical clarity.

The problem is further complicated by the fact that during the evaluation of the strength of a pile foundation, the foundation engineer will probably determine the ultimate capacity of an individual pile. He or the structural engineer will then assign a rather arbitrary factor of safety. Traditionally, for well controlled designs, it has been the intent that this number be approximately two.

This paper will propose the framework for a design specification for pile foundations which will avoid the inconvenience of dealing with both factored and working load and at the same time provide a more rational approach for dealing with pile design. The particular specification which has been used as the framework is the American Association of State Highway and Transportation Officials' Bridge Design Specification.

This specification, as is the case with most load factor design procedures, divides the factor of safety into two parts. The first part is the factor which is applied to the design load. It is usually expressed as a constant appropriate to the particular load type times the load in question. A much larger factor of safety is used for live loads since their magnitudes are not as accurately predicted as is the dead load. On the other hand, factors of safety for dynamic loads such as those induced by wind or earthquake tend to be considerably smaller because the duration of the load is short and therefore the structure, for that short time, might be expected to perform in a more desirable fashion. The other portion of the factor of safety is used to reduce the predicted strength of a structural element based on an evaluation of the accuracy with which this element capacity can be predicted, the variability of the element capacity, the warning of failure that it will give, and the consequences of failure.

As indicated above serviceability conditions are handled directly in load factor design procedures. This specification divides the problem of determining an acceptable pile design into three separate considerations: strength, serviceability, and installability. In the context of pile foundations serviceability refers to such factors as long term settlements, corrosion and other such considerations. These factors, while frequently difficult to analyze, are extremely important in pile design.

One reason for the low allowable stresses that are enforced on some piles is the consideration that sometimes they cannot be installed to higher working loads due to driving difficulties. It seems unrealistic to limit allowable stresses in all piles because some of them cannot be installed for those stresses. Installability should be evaluated as a separate consideration.

Design for Strength

The selection of a pile design for strength considerations involves assuring that the applied load is less than the pile strength. Recognizing that there is a statistical variation in the load and likewise a variation in the strength, the purpose of the factor of safety is to assure that the probability of the strength being less than the load is sufficiently small. This requirement is illustrated in Figure 1. In Figure 1(a) and (b) hypothetical distributions of load and strength are shown with a normal distribution assumed. When they are superimposed, the cross-hatched area indicates that portion of the cases where failure occurs (the load is less than the strength). In Figure 1(d) the effect of increased strength variability is shown. Even though the average strength is the same in both cases the case with the greater variability will have a greater probability of failure.

In the case of the AASHTO Bridge Code the load expression is currently defined in load factor form as

$$U = 1.3 \left[D + \frac{5}{3} (L+I)\right] \tag{1}$$

where U is the factored load, D is the actual dead load, L is the working live load and I is the impact load. The AASHTO Bridge Design Specification contains additional ultimate load equations that must also be satisfied but they will not be discussed here.

In foundation design for bridges the contribution of the dead load is usually the dominant influence. Therefore, the foundation loads can be approximated by

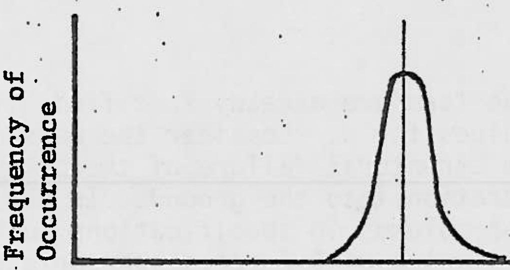
$$U = 1.3\,D \tag{2}$$

In order to assure adequate safety against failure the ultimate strength of the pile, R', must be reduced by a factor, ϕ. Thus,

$$R = \phi R' \tag{3}$$

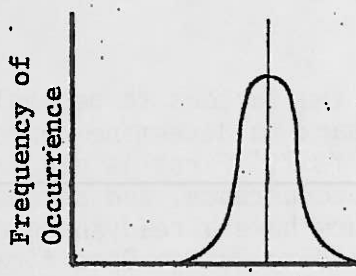
where ϕ is the capacity modification factor and R is the allowable ultimate strength. If a design has sufficient strength

$$R > U \tag{4}$$



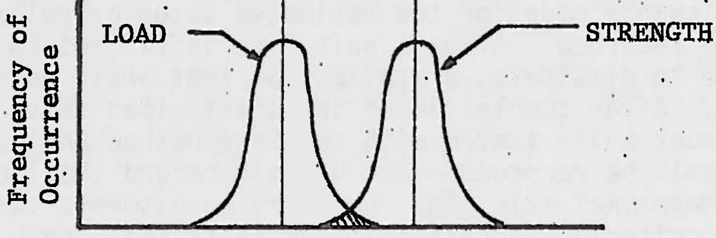
(a)

Strength Distribution



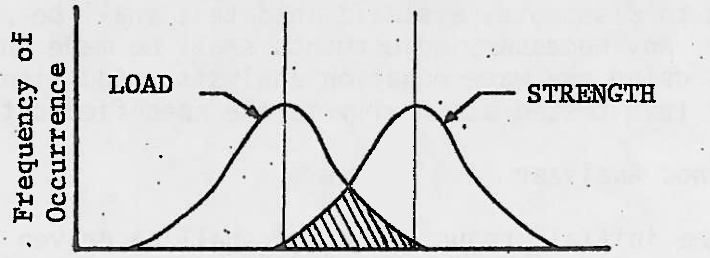
(b)

Load Distribution



(c)

Load and Strength Distributions
with Frequency of Failure



(d)

Effect of Variability on Frequency of Failure

Figure 1

Frequency Distributions for Strength and Load for a Structure

Since the factors to be applied to the load are already specified, it is only necessary to determine appropriate values for ϕ. Consider the ways in which a pile can fail. First it can fail due to structural failure of the pile (an infrequent occurrence) and second by penetration into the ground. In the first case ϕ values have already been defined for columns in specifications such as the ACI Building Design Specification and a value of 0.7 seems appropriate when applied to piles.

The establishment of ϕ for the second and most likely failure mode is more difficult. In order that ϕ be related to the variability of the pile strength it should be dependent on the means used to establish pile capacity, the variability of the soil and the construction control procedures used. Six different procedures now in use can be defined.

(1) Case Method Analyzer with Static Load Test

One of the initial production piles shall be driven to the required ultimate capacity as determined by the Case Method Analyzer (Ref. 1) with allowance made for the estimated setup or relaxation. Blow counts shall be recorded. After a wait time sufficient to allow pore water pressure to dissipate, a static load test shall be performed to failure. After completion of the static load test, the pile shall be restruck while tested with the Case Method Analyzer and the blow count shall be recorded. The dynamic record shall be examined for pile damage (Ref. 2). Any necessary adjustments shall be made in the driving criteria. Additional pile tests shall be by the Case Method Analyzer.

(2) Static Load Test

One of the initial production piles shall be driven to the required ultimate capacity as determined by wave equation analysis. Allowance shall be made for estimated setup or relaxation. Blow count shall be recorded. After a wait time sufficient to permit excess pore water pressure to dissipate, a static load test shall be performed to failure. Any necessary adjustments shall be made in the driving criteria using the wave equation analysis. Additional piles shall be proof load tested statically to the specified ultimate capacity.

(3) Case Method Analyzer

One of the initial production piles shall be driven to the required ultimate capacity as determined by the Case Method Analyzer. Allowance shall be made for the estimated setup or relaxation and blow count shall be recorded. After a wait time sufficient to permit excess pore water pressure to dissipate, the pile shall be restruck while tested with the Case Method Analyzer and the blow count recorded. The dynamic record shall be examined for pile damage. Any necessary adjustments shall be made in the driving criteria. Additional piles shall be tested by the Case Method Analyzer.

(4) Wave Equation Analysis

The driving criteria shall be set by Wave Equation analysis with allowance made for setup or relaxation. Blow count shall be recorded. After a wait time sufficient to permit excess pore water pressure to dissipate, selected piles shall be restruck and the blow count carefully measured at the beginning of the restrike.

(5) Analysis Based on Soil Data (Static Analysis)

The required depth of penetration shall be set by an appropriate static analysis based on soil boring data. The piles shall be driven to that penetration independent of blow count.

(6) Dynamic Formula

The driving criteria shall be set by use of the dynamic formula with allowance for setup or relaxation. Blow count shall be recorded. After a wait time sufficient to permit excess pore water pressure to dissipate selected piles shall be restruck and the blow count carefully measured at the beginning of restrike.

It is difficult to arrive at rational values for ϕ since sufficient data is not available for a thorough systematic analysis. Recommendations are contained in Table I together with the factor of safety that exists when used with the AASHTO load factors, assuming dead load is dominant. As live loads become a larger part of the total load the factor of safety increases.

TABLE I

ϕ-FACTORS FOR PILE-SOIL FAILURE

Inspection Class	Soil: Uniform	Soil: Variable
1	.70 (1.86)*	.70 (1.86)
2	.65 (2.00)	.60 (2.17)
3	.55 (2.36)	.55 (2.36)
4	.45 (2.89)	.45 (2.89)
5	.45 (2.89)	.35 (3.71)
6	.22 (5.91)	.22 (5.91)

*Quantities in parentheses give the total factor of safety under the assumption that the applied load is exclusively dead load.

It should be emphasized that the numbers in Table I are selected intuitively so that they follow traditional values. If load test data is assembled, it is possible to arrive at ϕ values rationally but this quantity of information is not now available.

Design for Serviceability

Serviceability considerations are very important in pile foundation design. Of primary interest are long term deformations (settlements). Settlement computations for pile foundations are very difficult to make with any reliability and accuracy. They must be made using working loads and they should be calculated independently of strength evaluations. Other serviceability limitations (for example, durability) tend to involve subjective judgements and are not directly related to structural considerations. Further discussion of serviceability considerations is beyond the scope of this paper.

Design for Driveability

In the past attempts have been made to place simple limitations on some pile and driving system parameters to make sure that critical driving stresses are not exceeded. Of particular concern is the question of tension stresses induced in concrete piles during easy driving. The most common approach has been the arbitrary limitation of pile-ram weight ratios. These limitations have been shown to be inadequate and even incorrect (Ref. 3). The problem may be solvable with closed form solutions of the one-dimensional wave equation, but this has not been done as yet. The most reliable approach is the use of a "wave equation" computer program. However, the program must properly model the driving system and proper input data must be used.

If a wave equation analysis is used, the next question that arises is the determination of acceptable values for dynamic driving stresses. Since this is a short term load that can be controlled, it is reasonable to approach closely to the failure stress. Furthermore, the consequence of failure during installation is only that a pile must be replaced (providing that proper inspection methods are being used).

Suggested values for allowable driving stresses for steel and concrete piles are given in Table II.

TABLE II

ALLOWABLE DRIVING STRESSES

Material	Allowable Stress
Steel	1.1 Fy
Concrete*	
Compression	$.85\ f'_c$
Tension	$3\sqrt{f'_c}$

*The allowable dynamic stresses for prestressed concrete piles refers to the total pile stress including prestress.

Comments and Discussion

The load factor design procedure is now the dominant method for accomplishing structural design. Its use is increasing and expanding. However, it has not been used for foundation design even though it fits well philosophically with the methods of foundation design, and particularly for deep foundations. The AASHTO Bridge Design Specification load factor expressions were used in organizing this specification. Of course, other codes could have been used equally well since they all have the same general form.

Other construction control procedures can be inserted in this framework and improvements in the state-of-the-art can be readily incorporated. A proper and reasonable ϕ factor must be used. Hopefully, the use of such a procedure will encourage the assembly of additional pile load test data (to failure) so that improved ϕ factors can be determined.

One of the important attractions of the procedure described here is that the cost trade-off of improved field testing and construction control can be directly evaluated. Thus, the engineer can show the owner the advantages of improved engineering on large jobs.

The field testing and construction control procedures are not described in detail since those aspects are beyond the scope of this paper. It should be noted that emphasis is placed on restrike testing. This procedure is one of the most important tools for improving pile capacity analysis. It is usually quite inexpensive to perform and will probably justify increased capacities. On the other hand, one of the most dangerous problems is the relaxation of pile capacity. Relaxation will be detected by restrike.

One of the principal advantages of the load factor philosophy is the separation of strength and driveability considerations. Currently used allowable stresses in steel and timber piles are being held at a low level because sometimes they cannot be driven to higher capacities due to excessive driving stresses. The two problems must be separated and dealt with independently since they are quite unrelated. The above procedure accomplishes this separation.

Pile foundation design specifications have remained essentially unchanged for several decades. During this same time, structural design codes and procedures have undergone a gradual change to greater rationality and realism. The procedures suggested here will accomplish the same thing for pile foundation design.

Acknowledgements

The work discussed here was sponsored by the Federal Highway Administration. However, they have not adopted these procedures and they do not necessarily support the opinions expressed in this paper.

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TESTS TO OBTAIN
BEHAVIOR OF DRILLED SHAFTS UNDER AXIAL LOAD

by Lymon C. Reese
T. U. Taylor Professor of Civil Engineering
and Associate Dean
College of Engineering
University of Texas at Austin

Introduction

Drilled shafts or other types of deep foundations may be subjected to field load tests to prove that a particular design is capable of sustaining the proposed working load with a reasonable factor of safety, or to develop correlations between the load-transfer characteristics of the foundation and the properties of the supporting soil. The former type of test is uninstructive in many respects and the latter type of experiment should be selected when possible (Reese, 1978). If one wishes to obtain data that indicate the manner in which the load is transferred to the soil, it is necessary to employ instrumentation along the length of the foundation. This paper describes some types of instrumentation that may be employed and indicates methods that can be used to interpret the data from the instrumentation.

Prior to discussing the instrumentation for drilled shafts, it is desirable to present a brief discussion of the mechanics of axially loaded deep foundations.

Mechanics of Deep Foundations Under Axial Load

A free body of a drilled shaft under an axial load is shown in Fig. 1(a). An equilibrium equation can be written as follows (Coyle and Reese, 1966).

$$Q_T = \int_o^L \int_o^{2\pi} s_x r d\theta dx + \int_o^r 2\pi q \rho d\rho \qquad (1)$$

where

Q_T = load at top of shaft

s_x = unit load transfer in skin friction

q = unit load transfer in end bearing

r = radius of drilled shaft.

If the load remaining in the shaft at any depth x, is desired, Eq. 2 may be used.

$$Q_x = Q_T - \int_o^{x_1} s_x r d\theta dx. \qquad (2)$$

If it is assumed that internal instrumentation has been installed and that the distribution of axial load has been measured as a function of depth, as shown in Fig. 1(b), not only can the load carried in skin friction Q_s and the load carried in end bearing Q_b be obtained, but the unit load transfer values can be obtained, as follows:

$$s_{x_1} = \left(\frac{dQ}{dx}\right)_{x=x_1} /2\pi r, \qquad (3)$$

and

$$q = \frac{Q_b}{\pi r^2}. \qquad (4)$$

Further analysis of the data from the instrumented load test can be made if the axial stiffness AE of the pile is known. The shortening of the drilled shaft δ_{x_1} can be computed as follows:

$$\delta_{x_1} = \frac{1}{AE} \int_o^{x_1} Q_x \, dx. \qquad (5)$$

Thus, the downward movement z of the pile at the depth x, can be computed.

$$z_{x_1} = \delta_T - \delta_{x_1} \qquad (6)$$

where

δ_T = settlement measured at the top of the drilled shaft for the particular load that was applied.

Similarly, the settlement at the base of the pile can be computed.

$$\delta_L = \frac{1}{AE} \int_o^{L} Q_x \, dx, \qquad (7)$$

and

$$z_L = \delta_T - \delta_L. \qquad (8)$$

Figures 1(c) and 1(d) show the unit load transfer values, s and q, respectively, plotted as a function of the downward movement of the drilled shaft at the respective depths.

If a family of curves is obtained for a load test for the case where the axial load is sufficient to cause the drilled shaft to plunge, load transfer versus movement curves can be developed such as shown by the dashed lines in Figs. 1(c) and 1(d). A number of such curves could be developed, of course, for load transfer in skin friction.

In all instances where internal instrumentation is used in the performance of a load test, it is necessary to do a subsurface soil investigation and to obtain the shear strength of the soil using in situ or laboratory techniques. For clays the undrained shear strength c_u is usually obtained, and for sands the shear strength is frequently assumed to be measured by the overburden stress times the

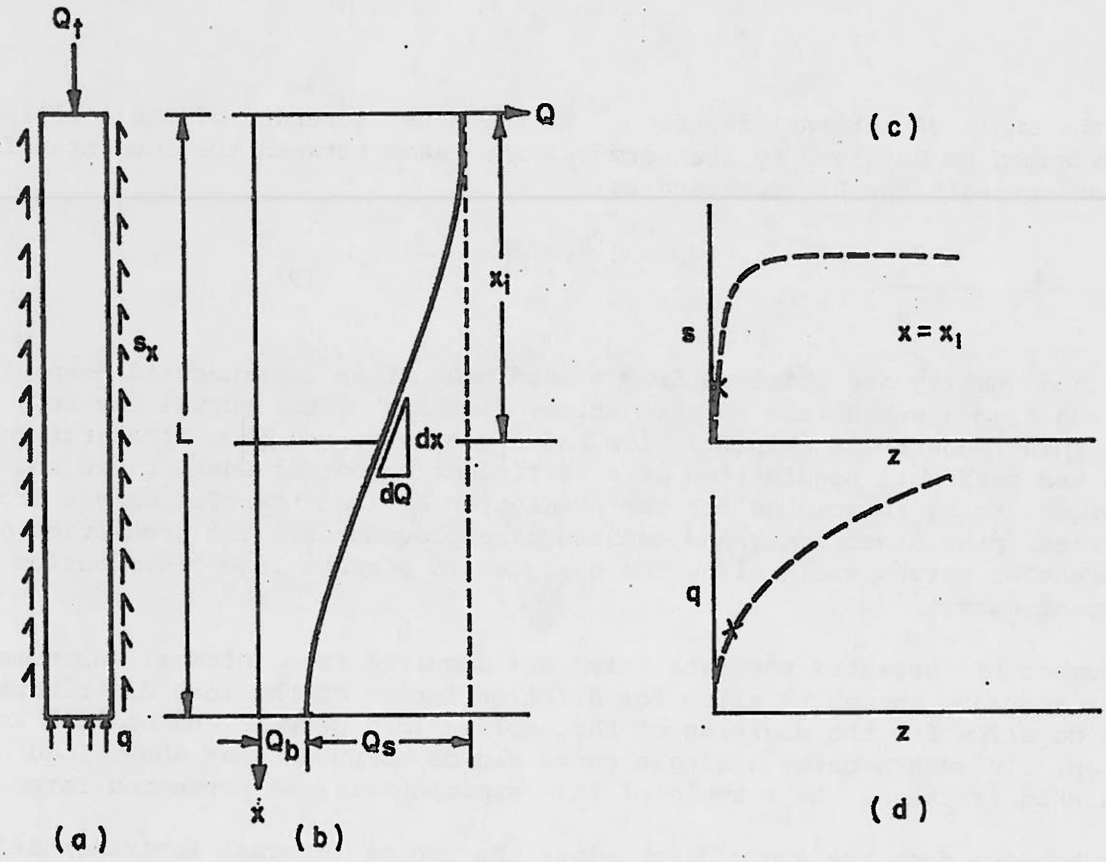

Fig. 1. Concepts of Load Transfer from Deep Foundations to Soil

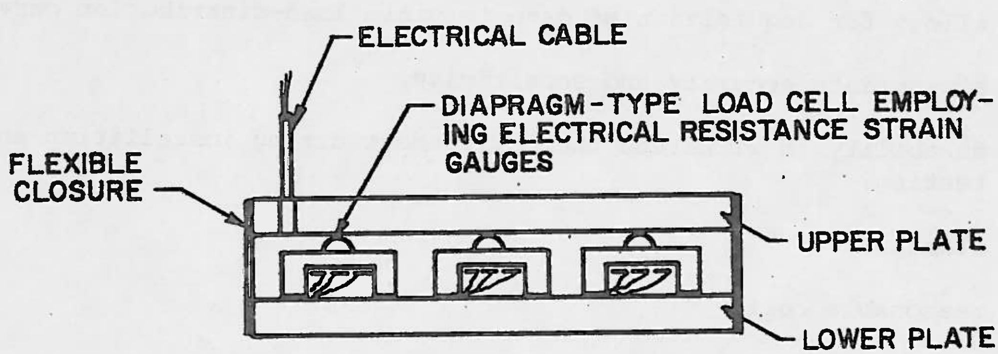

Fig. 2. Sketch of Bottom-hole Load Cell

tangent of the angle of internal friction. If the shear strength of the soil in general is assumed to be given by the term τ_x, the ratio between the load transfer and the shear strength can be expressed as:

$$t_x = \frac{S_x}{\tau_x} . \qquad (9)$$

If data of good quality are obtained from a load test of an instrumented deep foundation and from a subsurface investigation, a family of t-z curves can be plotted for load transfer in skin friction and a q-z curve can be plotted for load transfer in end bearing. Acquisition of a sufficient number of such curves should allow techniques to be formulated for the prediction of load transfer curves from soil properties, pile dimensions, and construction procedures. The prediction of such load transfer curves would allow the designer to compute load distribution and load settlement curves.

In a number of instances the data which are acquired from internal instrumentation are not accurate enough to allow for differentiation of the load distribution curves, but do allow for the division of the applied load between end bearing and skin friction. In such a case, a single curve can be computed that shows load transfer in skin friction. An example of this approach will be presented later.

As may be seen from the above discussion, the use of internal instrumentation in the testing of a deep foundation under axial load can lead to the acquisition of valuable information on load transfer. Desirable characteristics of internal instrumentation will be discussed in the next section.

Instrumentation for Measuring Axial Load

A sizeable number of devices and techniques have been employed for the measurement of axial load in deep foundations. A few of the methods will be discussed in this paper. There are several desirable features that all such methods should have (Barker, 1970, pg. 7). These are:

(1) allows for acquisition of data to yield load-distribution curves,

(2) appropriate accuracy and sensitivity,

(3) durability to withstand the environment during installation and testing,

(4) stability,

(5) reasonable cost,

(6) allows easy acquisition of readings, and

(7) easy method of installation.

Several methods of instrumentation have been employed at The University of Texas at Austin over the past decade for instrumenting drilled shafts. Three of the methods that have been employed with good results are described.

Bottom-Hole Load Cell

The device that can be used directly to separate the load transferred by skin friction and by end bearing is the bottom-hole load cell (Whitaker, T., R.W. Cooke, and G.W. Clarke, 1962; Barker, 1970). The cell violates the first item in the list of desirable features in that it does not allow the development of an entire load-distribution curve and violates another desirable feature in that it is expensive; however, the cell has good accuracy and sensitivity, is durable, is stable, presents no problems in acquiring data, and is easy to install.

A sketch of the bottom-hole load cell is shown in Fig. 2. The cell consists of heavy, circular top and bottom plates that are separated by diaphragm-type load cells. Electrical resistance strain gages are affixed to the diaphragm and measure the bending strain. The bending strain is converted into load by employing a calibration curve. The diaphragm-type cells are placed in a symmetrical pattern so that they share equally in the load and in sufficient numbers as not to be over-stressed. A flexible closure seals the space between the plates to prevent moisture intrusion and the lead wires are brought to the ground surface in a copper tube. Waterproofing of the electrical system is further provided by packing desiccant between the plates or by pressurizing the space with dry nitrogen.

The cell is placed in the bottom of an excavation prior to placing the reinforcing steel and the concrete for the drilled shaft. Thus, the load that reaches the lower end of the drilled shaft is taken by the bottom-hole cell.

The bottom-hole cell was used successfully by O'Neill (1970) but its use was discontinued because of its expense and because the methods of obtaining the distribution of load with depth can be used to obtain the load coming to the base of the foundation.

Telltales

A simple mechanical system can be used in some instances to obtain the distribution of load along drilled shafts. Telltales are unstrained rods that extend to full depth of the shaft and to several fractions of the full depth. A mechanical gage (dial indicator) is seated on top of the drilled shaft with its stem on top of the telltale and will record the shortening of the shaft from its top to the point where the base of the telltale is located. With telltales to different depths in the deep foundation, a family of curves can be plotted for a load test showing deformation as a function of depth. Differentiation of the curves yield strain. If the axial stiffness of the foundation is known, the load can be computed as a function of depth.

Because of the simplicity of the system, unstrained rods were probably used many years ago during pile load tests. An experiment using the scheme was reported in literature in 1942 (Hansen and Kneas). Raymond International has reported in their company literature on the use of unstrained rods and probably coined the term, "telltale" (Snow, 1965).

There are no standard dimensions and details for telltales. One design is shown in Fig. 3 (Reese and Hudson, 1968). The telltale concept is simple and does not involve electronic instrumentation, but the application of the system is difficult and the results are not always successful. As is readily seen, errors will be introduced if there is friction between the rod and the case or if the temperature of the rod changes during the test. Further errors can be introduced in the measurement of deformation. A special loading head must be fabricated, making use of struts to transfer the jack load from a top plate to a bottom plate, in order to provide space for the tops of the telltales and the dial gages. Relative movement between the stem on the dial gage and the top of the unstrained bar during the loading procedure can introduce an error.

The load in the drilled shaft at a particular depth can be computed by us of the following equation:

$$Q_x = AE \left. \frac{d\delta}{dx} \right|_x \qquad (10)$$

where

$\left. \frac{d\delta}{dx} \right|_x$ = slope of the curve giving deformation in the shaft at point x.

Examination of Eq. 10 shows that it is necessary to be able to plot accurately the deformation in the drilled shaft as a function of depth; thus, it is necessary to use telltales of a number of different lengths. The telltales should be paired in order to eliminate bending so there could be trouble in getting enough telltales in the drilled shaft.

There is no easy way to calibrate the system so the values of A and of E in Eq. 10 must be computed, introducing additional errors.

In spite of their limitations, telltales have been used effectively in some tests of drilled shafts, particularly in the cases where there were loads that produced relatively high strains in the concrete. A definite advantage of the telltale is that it can give a direct indication of the downward movement of the deep foundation. If the physical arrangement will allow, the use of a pair of telltales to a point near the base of the drilled shaft is advantageous even if electronic instrumentation is employed.

<u>Mustran Cells</u>

The device that is currently being used extensively for the internal instru-

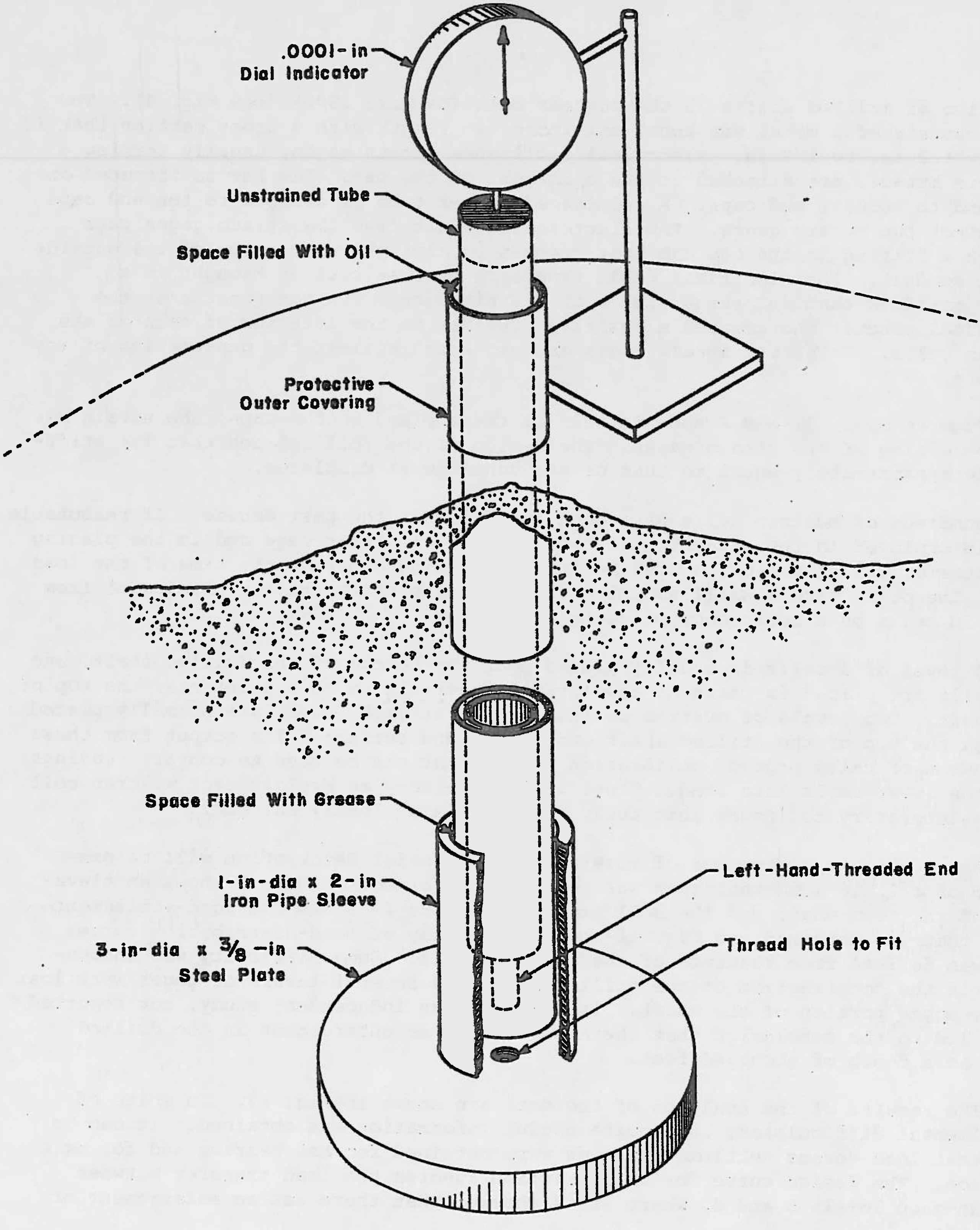

Fig. 3. Sketch of a Telltale

mentation of drilled shafts is the mustran cell (Barker, 1970) (see Fig. 4). The cell consists of a metal bar about six inches in length with a cross section that is usually 1/2 in. by 1/2 in. Electrical resistance strain gages, usually forming a complete bridge, are attached to the midheight of the bar. The bar is threaded on each end to receive end caps. A reinforced rubber tube is clamped to the end caps to protect the strain gages. The electrical conduit from the strain gages pass through a fitting in the top cap that creates an airtight connection at the outside of the conduit. The electrical cable from each mustran cell is brought to a common manifold that is pressurized with dry nitrogen. The gas penetrates the electrical conduit and creates a positive pressure on the interior of each of the mustran cells. Thus, the strain gages are protected against the penetration of any moisture.

Figures 5, 6, 7, and 8 show details of the top and bottom caps, the strain bar and the wiring of the strain gages. The design of the cell is such that its stiffness is approximately equal to that of the concrete it displaces.

Hundreds of mustran cells have been installed in the past decade. If reasonable care is employed in the attaching of the cells to the rebar cage and in the placing of concrete, all of the cells will be in good working order at the time of the load test. The cells take a small amount of space in the shaft and the placing of from 20 to 50 cells on a rebar cage presents no problem.

A level of 4 cells is usually placed near the bottom of the drilled shaft, and the cells are placed in pairs at appropriate intervals up to a point near the top of the shaft. Two levels of mustran cells, 4 cells at each level, are normally placed between the top of the drilled shaft and the ground surface. The output from these two levels of cells provide calibration curves that can be used to convert readings from the lower cells into load. Thus, it is necessary to preload each mustran cell in the laboratory to insure that their responses are closely matched.

As an example of the use of mustran cells, a brief description will be presented of a field load test that was recently performed. Figure 9 shows an elevation of the test shaft and the soil profile. Figure 10 shows the load-settlement curve that was obtained and Fig. 11 shows the family of load-distribution curves that was derived from readings of the mustran cells. Some difficulty was encountered in the construction of the drilled shaft and several levels of gages were lost in the upper portion of the shaft. In addition, an independent study, not reported here, led to the conclusion that there was likely an enlargement in the drilled shaft at a depth of about 40 feet.

The results of the analyses of the data are shown in Fig. 12. In spite of experimental difficulties, some quite useful information was obtained. It can be seen that load versus settlement curves were obtained for end bearing and for skin friction. The design curve for skin friction ignores the load transfer between mustran-gage levels 5 and 6, where it is thought that there was an enlargement of the shaft.

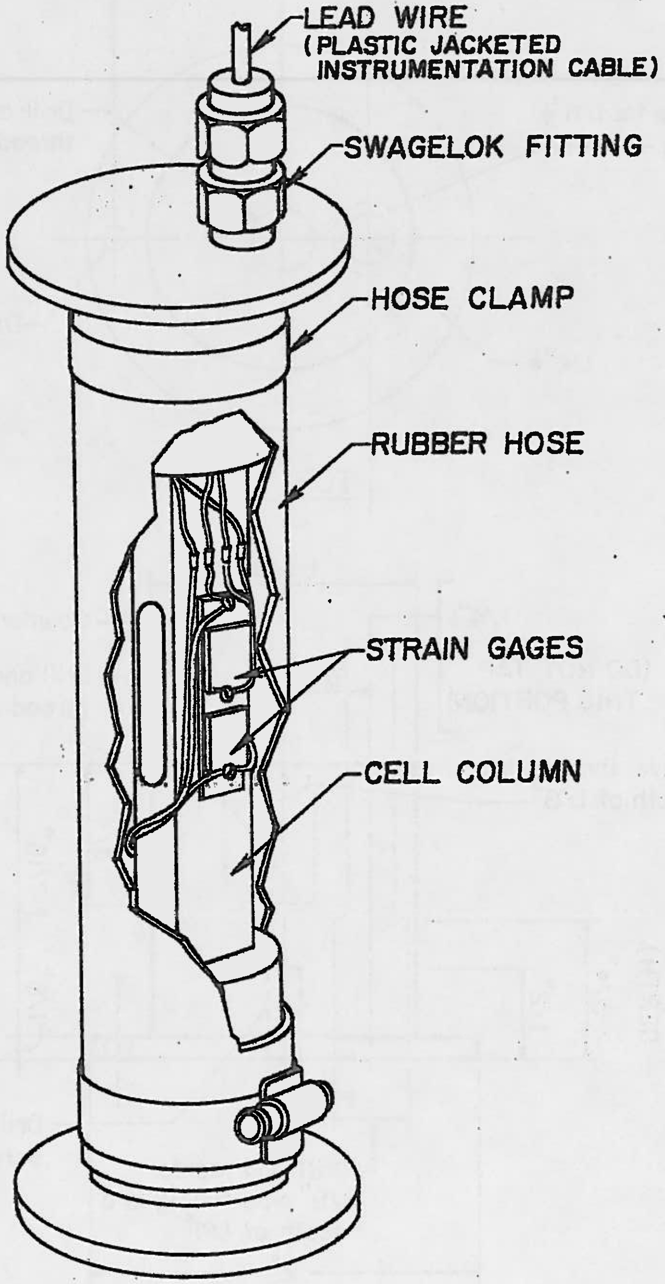

Fig. 4. Mustran Cell

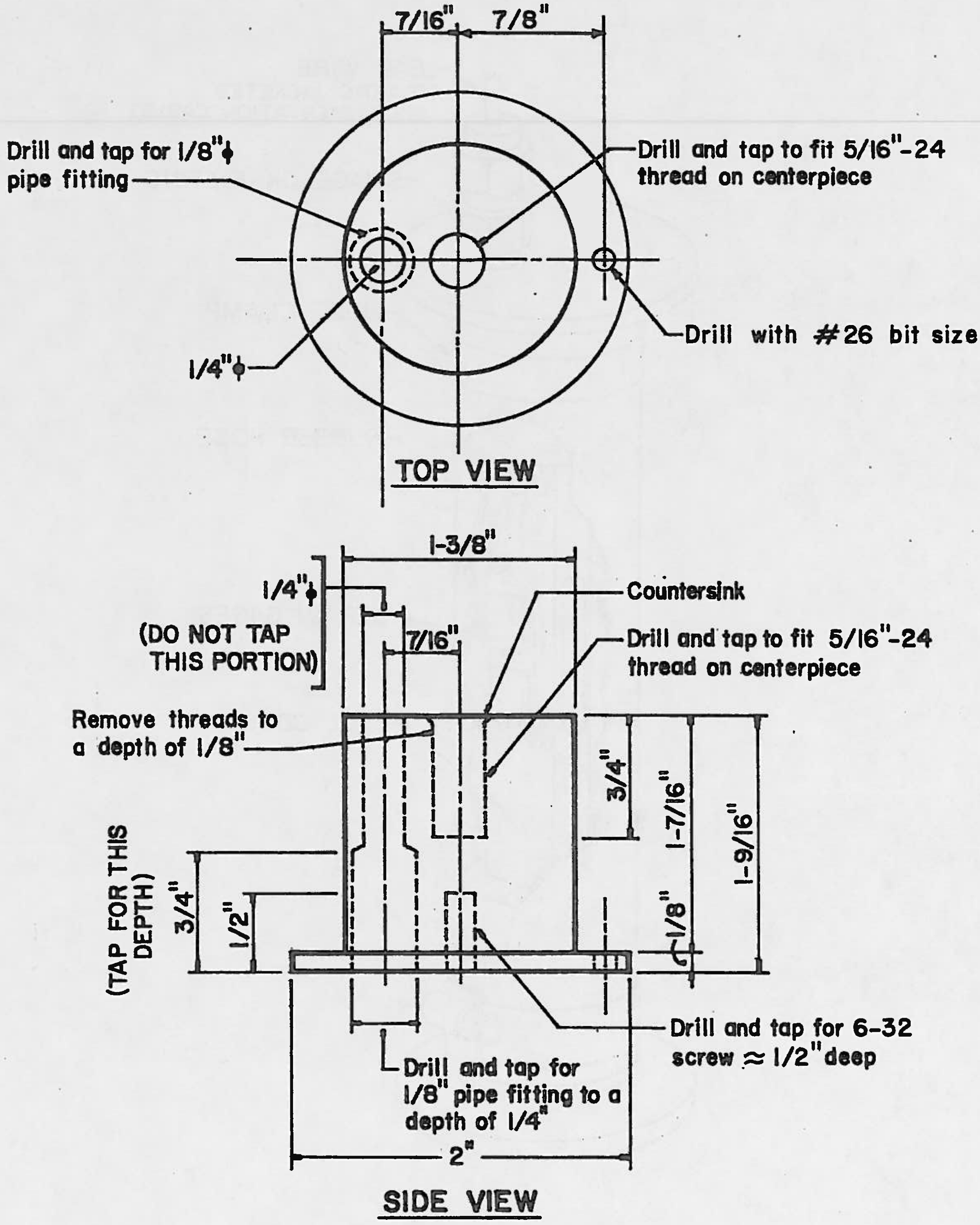

Fig. 5. Top Cap for Mustran Cell

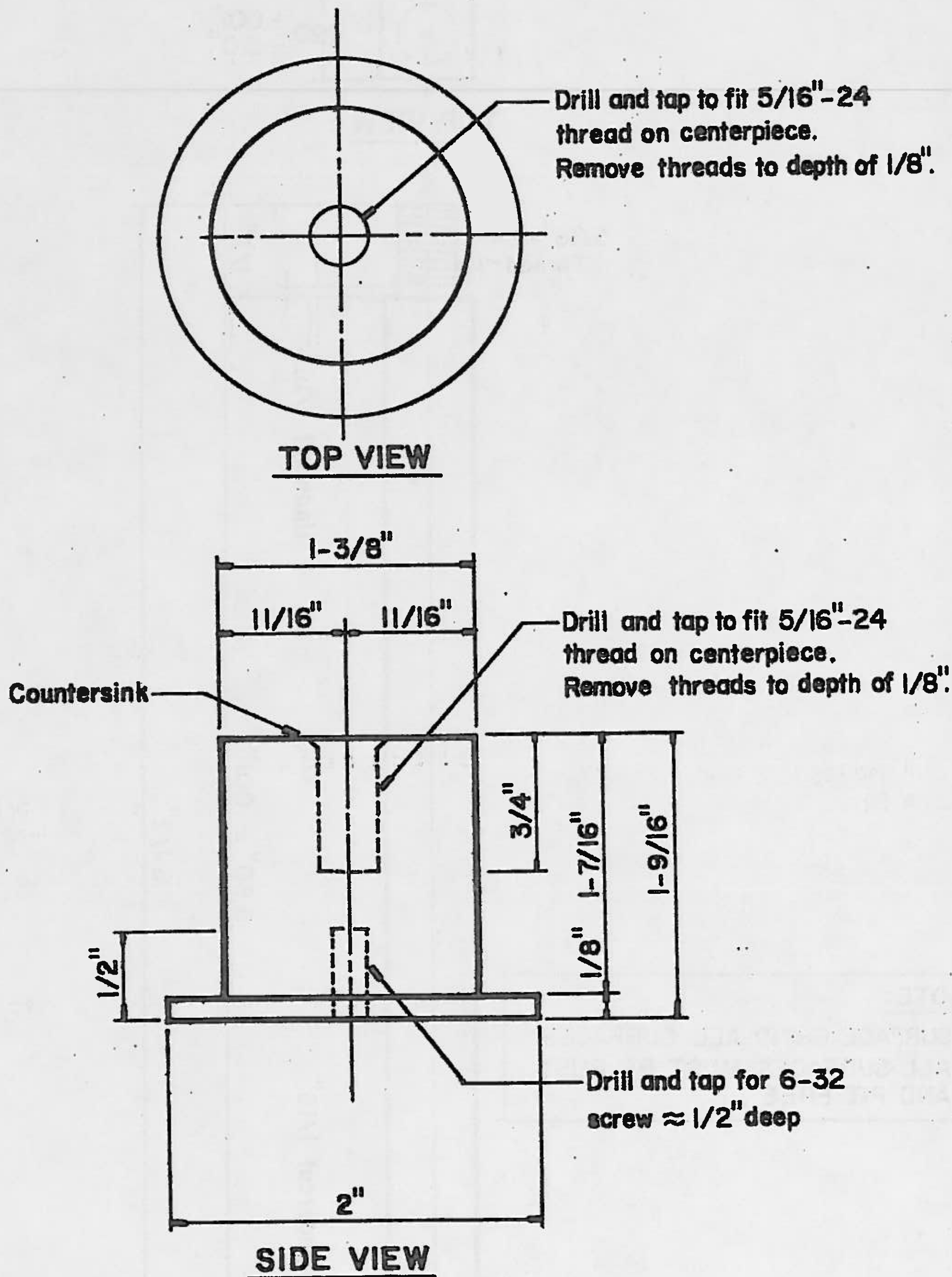

Fig. 6. Bottom Cap for Mustran Cell

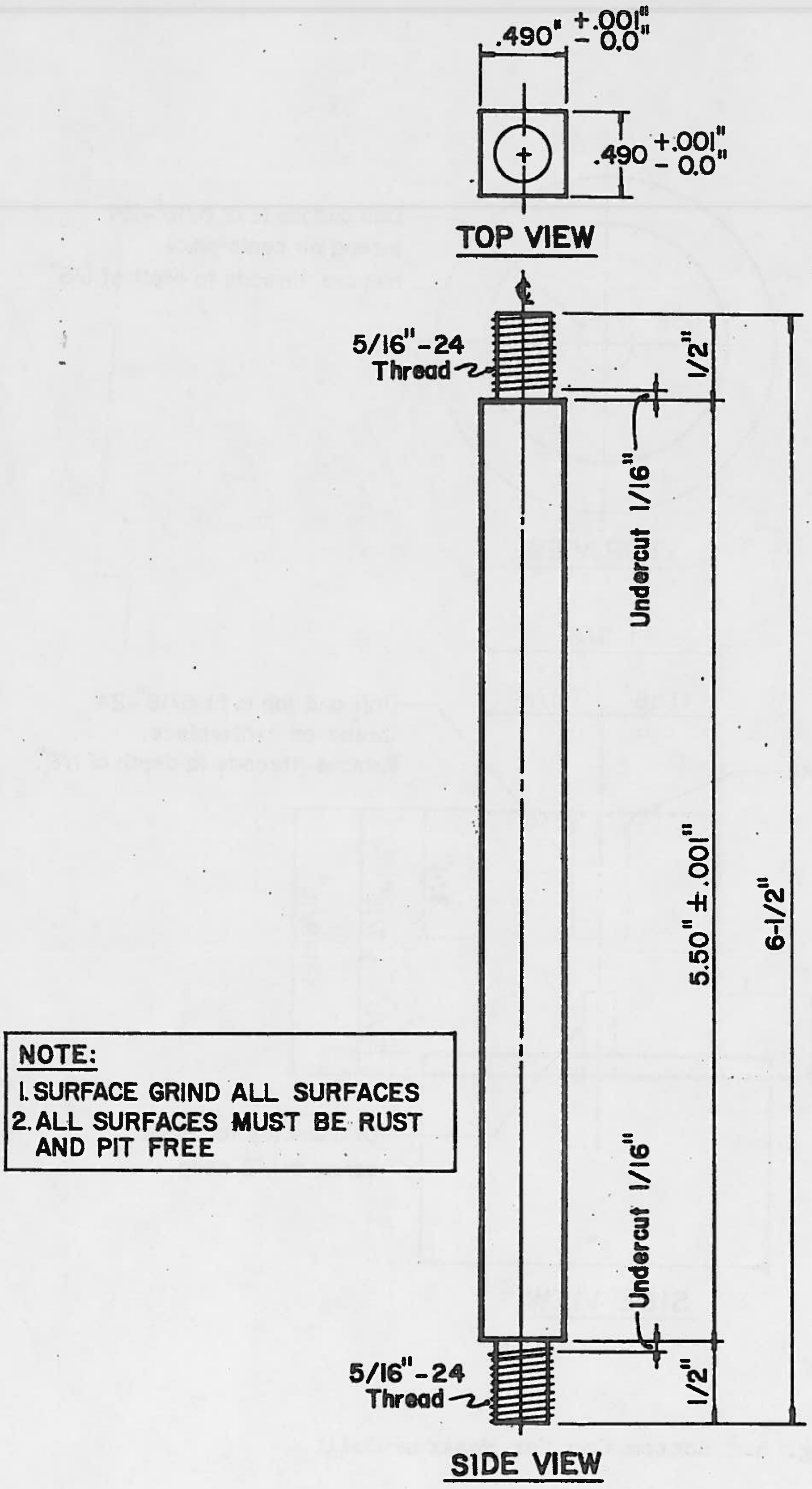

Fig. 7. Strain Bar for Mustran Cell

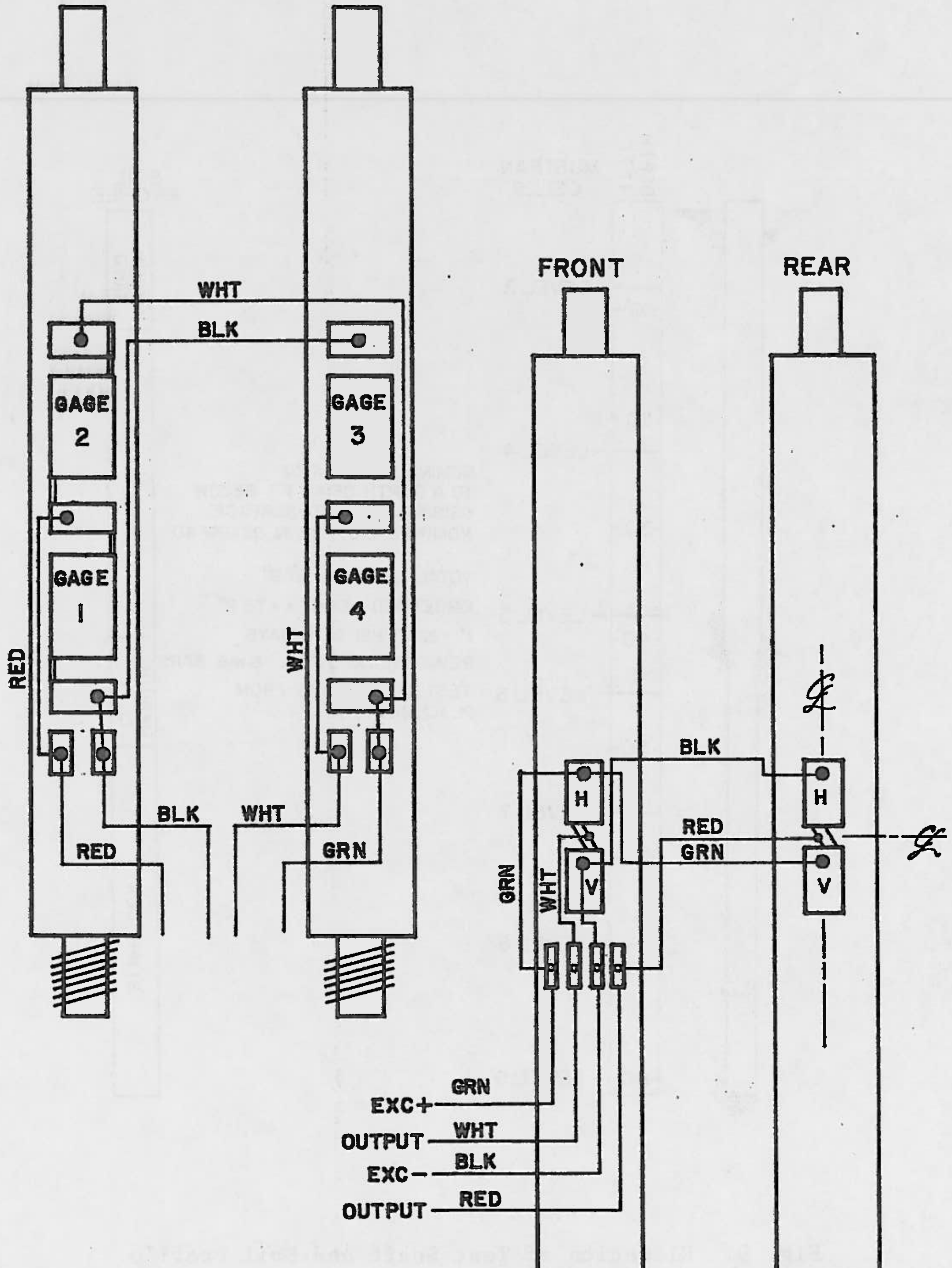

Fig. 8. Sketch Showing Wiring Connections

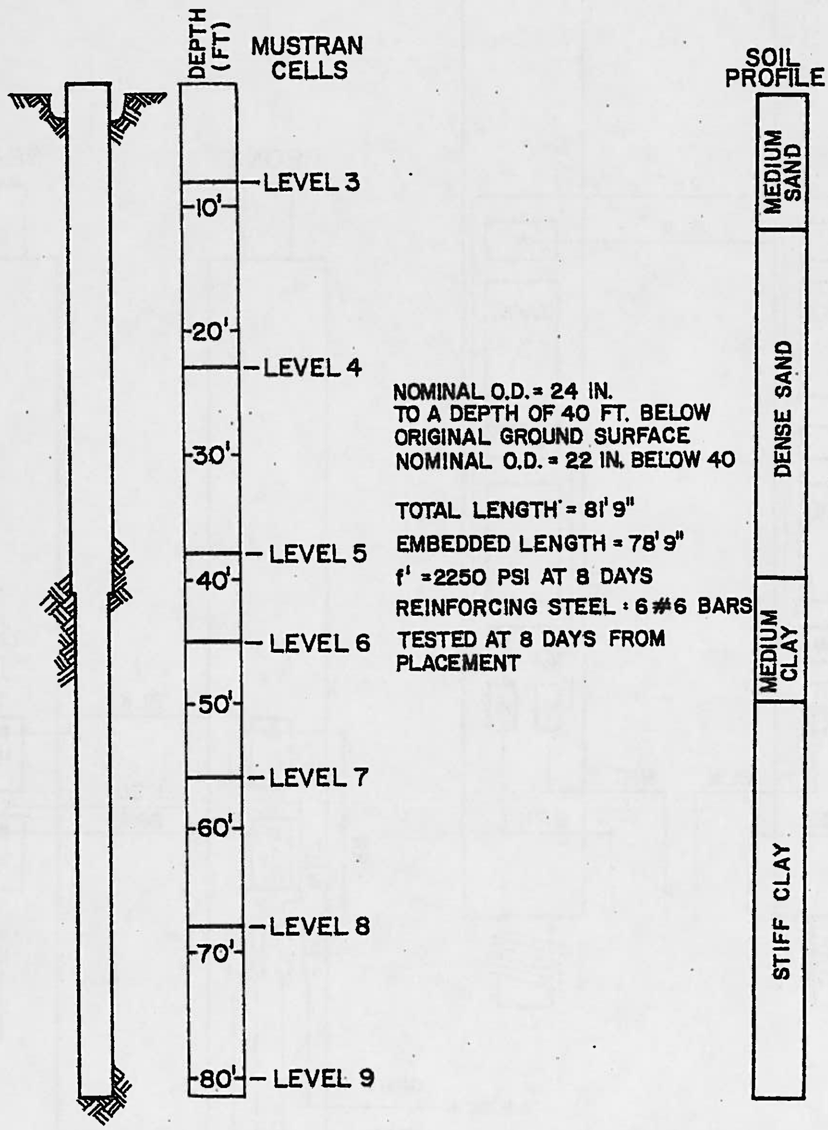

Fig. 9. Elevation of Test Shaft and Soil Profile

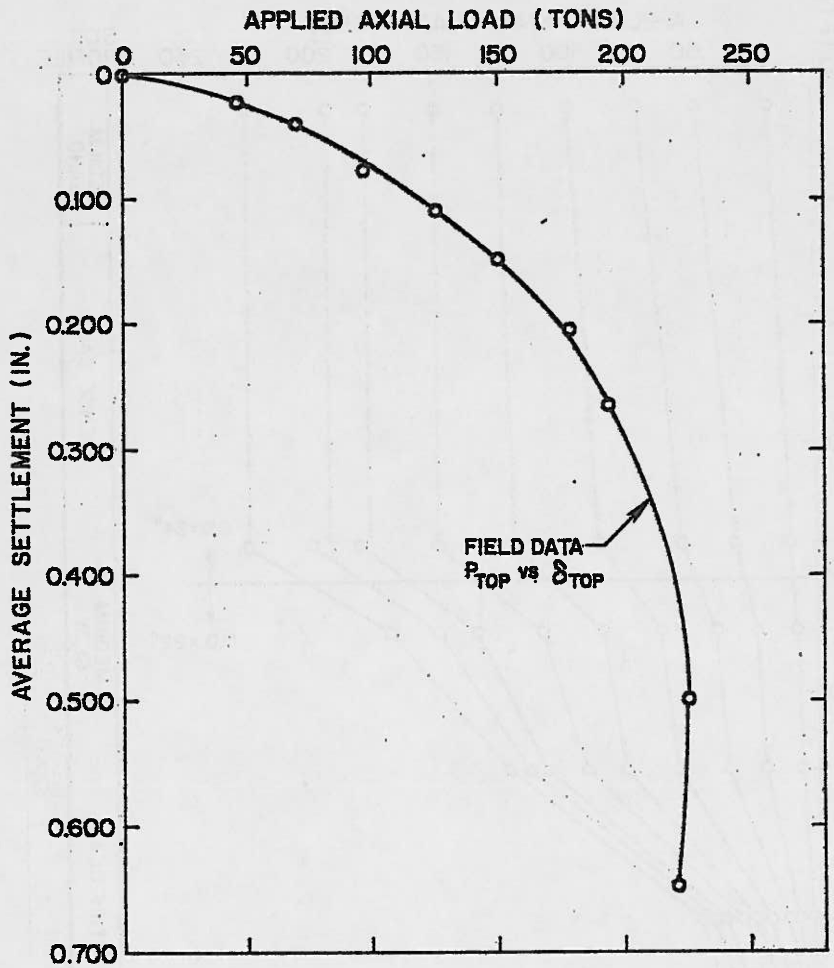

Fig. 10. Settlement of Test Shaft as a Function of Axial Load

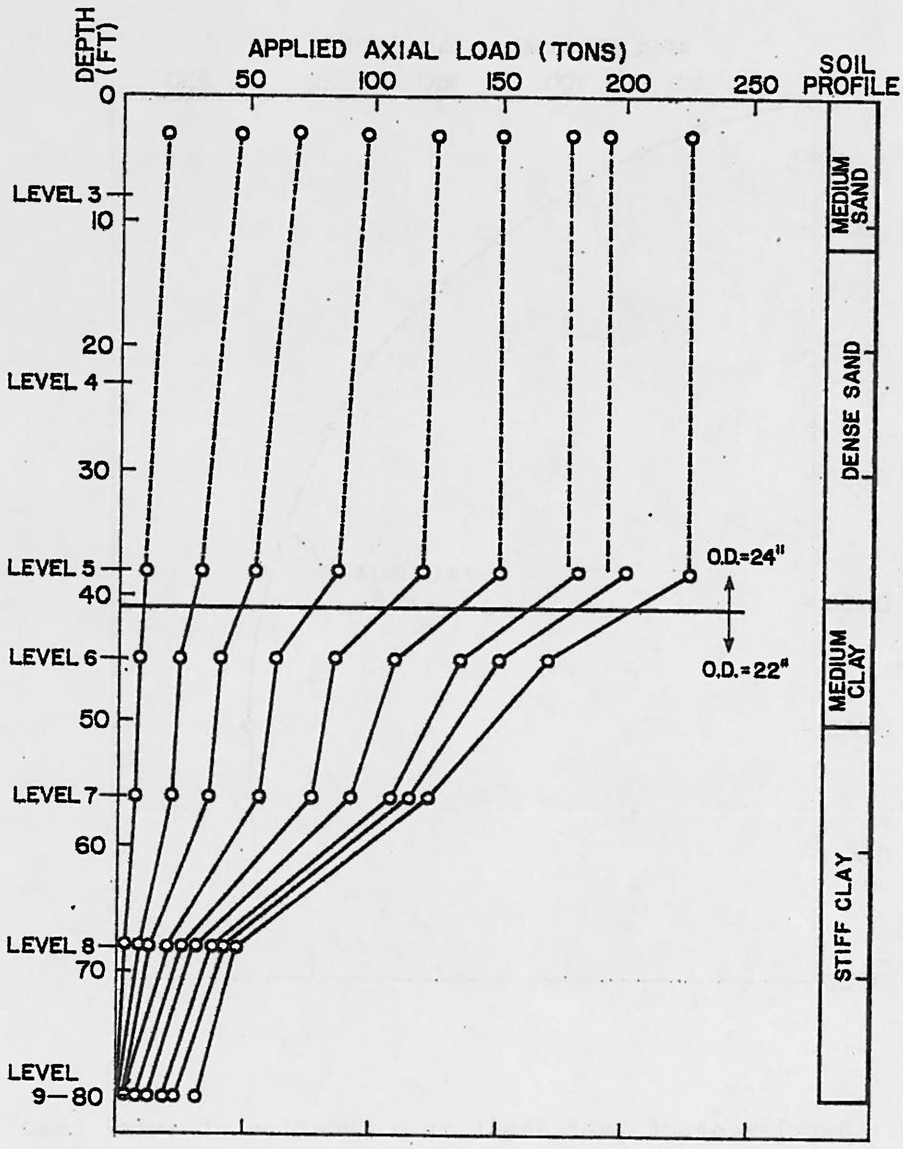

Fig. 11. Axial Load in Test Shaft as a Function of Depth

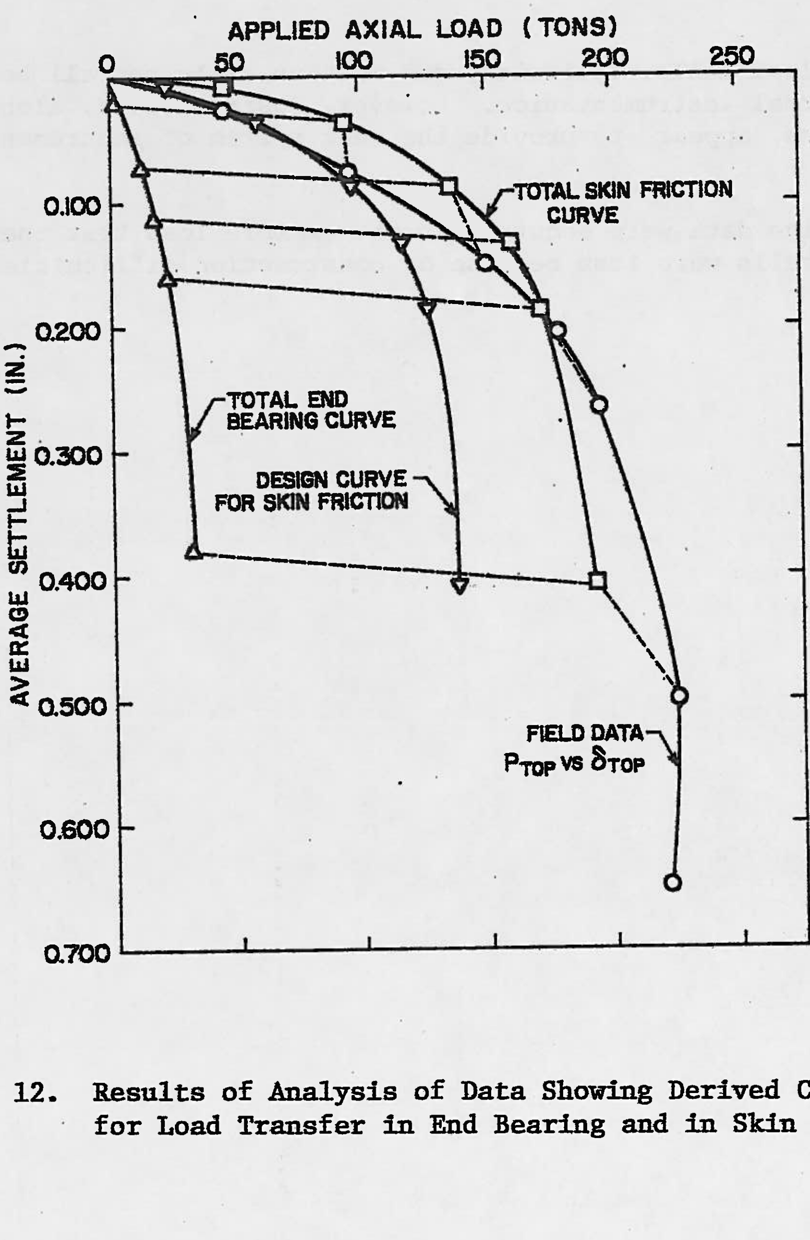

Fig. 12. Results of Analysis of Data Showing Derived Curves for Load Transfer in End Bearing and in Skin Friction

Conclusions

The use of internal instrumentation in drilled shafts that allows the development of curves showing the distribution of load as a function of depth is advantageous. Not only can useful information be gained about the design of foundations at a particular site but a contribution to the technical literature can possibly be made.

Bottom-hole load cells, telltales, and mustran cells can all be used to good advantage as internal instrumentation. However, mustran cells, along with a small number of telltales, appear to provide the best system of instrumentation at present.

Valuable design data were acquired in the example load test that was described even though some cells were lost because of construction difficulties.

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CONTRACTING FOR DEEP FOUNDATIONS - LEGAL ASPECTS

C. Robert Lennertz, P.E., Attorney-at-Law
Vice President and Chief Engineer
The H. C. Nutting Company

Deep foundations are usually installed as a part of the general contract for the structure to be supported thereon. For the typical situation there are five primary parties involved in the design and construction process: the owner, the architect-engineer, the geotechnical engineer who frequently has a direct contract with the owner with the A-E acting as the owner's agent relative to the geotechnical engineer's services, the general contractor and the specialty foundation subcontractor.

Usually the details of the system, the method of installation and the compensation are set forth in a written contract between the owner and the general contractor, most often as part of some larger work such as the entire building or a substructure contract for a large bridge. The duties and ultimate responsibilities of both the primary parties to the contract; i.e., the owner and general contractor, and also all others directly involved in the design and construction process are determined by both the explicit terms of the written contract and also other terms implied by law.

SUBSURFACE INFORMATION

Today it is a rare project on which some type of subsurface investigation is not made during the design stage. A case of negligence could very possibly be shown against an engineer who would prepare a design for a deep foundation without having rather specific information concerning the subsurface conditions which exist in the area where this foundation is to be installed.(1) Most commonly, the subsurface information is obtained by means of test borings. Occasionally, in areas where geologic conditions are known to be quite uniform, the design will be prepared on the basis of experience with nearby structures, however, this is certainly the exception rather than the rule.

The owner and his agents have a positive duty to disclose all relevant information at their disposal to all parties of a construction contract.(2) The owner cannot pick and choose what is to be disclosed.(3) He cannot plan an investigation so as to give a distorted view of the true subsurface conditions.(4) Also, there has been at least one case litigated where the owner has been found to be liable for the insufficiency of the subsurface information.(5)

The law is clear that the factual information, the logs of test borings and the data developed from laboratory tests, must be made available to bidders and to the successful contractor in a construction contract.[6] Today there is much variety in the manner in which this factual information is made available and there is no concensus as to whether the opinions developed by a soils engineer, usually in the form of a geotechnical report, must be made available.[7]

It has long been the practice to copy the logs submitted by the drilling company or geotechnical firm onto the drawings and not include as part of the contract documents or make available to the bidders the original test boring logs or any additional information in the form of opinions or analysis that were developed by the geotechnical engineer. There have been several cases in recent years that held that the owner is liable for any damages that can be shown where the information included in the contract documents was materially different from the original or total information available either in that the information included in the documents was not accurately copied from the original data or that some relevant data was not made available to the bidders.[8] In a recent case relative to a deep sewer tunnel construction, the boring logs that were included on the drawings showed water levels as actually encountered in the borings but failed to include notes that were on the original boring logs as to rise in water levels which indicated artesian conditions.[9] Another case, an administrative law decision by the Federal Engineering Board of Contract Appeals, upheld damages for changed conditions where a contractor encountered boulders in a foundation excavation where the borings included in the contract documents gave no indication of boulders but where the soils report, which was referenced in the documents and which was available for inspection by the bidders, did discuss the probability of boulders being encountered in the excavation. The decision held that the contractor was not bound by the soil report and "that we do not construe that clause as requiring the contractor to place reliance on the referenced data instead of the actual representations made by the specifications and drawings".[10]

On major projects and particularly those dealing with difficult subsurface conditions, it is not unusual that the design team will, in addition to using specific investigations made at their particular site, use data developed on other projects having similar subsurface conditions. In a recent Kentucky case the contractor alleged that the State was liable for withholding relevant information when they did not make reference in the documents to a nearby project on the same geologic formation where some employees of the State had knowledge of difficulties encountered during construction due to solution limestone. The court held that the Stage agency had no duty to make specific reference in the documents to any and all projects where subsurface conditions were similar and where experiences may have been considered in making the design decision on the subject project.[11]

Whenever subsurface information is included in the contract documents there is almost always accompanying this information a statement as to the owner's position relative to the presentation of this information. These clauses are generally in the form of disclaimers or exculpatory clauses. It has usually been the practice to make these clauses in a manner designed to disclaim any responsibility relative to the presentation of the subsurface data, not only as to the sufficiency of the data but also as to the accuracy of what is included. The following is a typical example of a general disclaimer clause.

A. The soil investigation data is available only for information and the convenience of bidders. The owner and architect/engineer disclaim any responsibility for the accuracy, true location and extent of the soils investigation that has been prepared. They further disclaim responsibility for interpretation of that data by bidders, as in projecting soil-bearing values, rock profiles, soil stability and the presence, level and extent of underground water.

B. Bidders are urged to examine soil conditions and to make their own investigation of the site before bidding.

C. Soil investigation data is not part of the contract documents.

There is a definite trend today to write the clauses so as to accept responsibility for any inaccuracy of the data that is presented; i.e., any error, for whatever reason, in the basic facts made available, but giving notice that conditions, other than those indicated by the borings or other data included in the documents, may occur at the site and that the owner does not warrant conditions indicated by the borings that have been made to represent those within the entire site. The following clause from the proposed ACI Standard Specifications for End Bearing Drilled Piers is representative of this type of clause.

> Subsurface Data - A subsurface investigation has been made by ___________. Logs of borings and test data are available for contractor's information and for his interpretation as to soil and water conditions that may be encountered at the site. Logs and test data are not represented as complete description of the site soil and water information but only display what was found in borings at the indicated locations. Contractor shall examine logs of borings and obtain additional information, if necessary in his judgment.

The basic rule is that these exculpatory provisions will effectively protect the owner unless he has withheld material information or has failed to properly represent information which he had or as to which he may be charged with constructive knowledge.[12] In the absence of ambiguity, the contract will be enforced according to its terms.[13] The parties to a contract can agree to limit their liability when there is no inequality in the bargaining position of either of the parties. The courts have held that such inequality in bargaining positions which would make such a term unconscionable does not, in general, exist in the construction industry.[14]

A study of relevant cases shows that in general the court will uphold the disclaimer clauses when (1) there has been no positive misrepresentation of fact by the owner on which the contractor could reasonably rely (the contractor's general experience and familiarity with the location will be considered in determining whether he has reasonably relied on this misrepresentation), (2) there is a clearly worded provision in the contract disclaiming the owner's responsibility for extra expenses resulting from different conditions encountered at the site, (3) the contractor had an opportunity to inspect the site plans and specifications before bidding.[15]

ALTERNATE DESIGNS

There is a presumption by law that the plans and specifications are sufficient for the subsurface conditions; i.e., if the foundations are installed in compliance therewith a satisfactory result will be obtained.[16]

On a typical building job very often a wide variety of deep foundations will provide adequate support. It is not at all unusual in glacial deposits for drilled shafts to be as feasible as piles and for a variety of pile materials and methods of installation to be suitable. It is often very difficult for the design engineer to determine the most economical system for a particular job. For a given subsurface condition some systems provide a more efficient use of materials than do others, however, determining this with any certainty is one of the most difficult technical matters in evaluating deep foundations; for example, predicting the total footage that will be required for a tapered shell pile as compared to a straight or augered pile. Also, the bid prices for different materials and, in particular, different methods of construction can vary dramatically in relatively short periods of time depending on economic factors.

Accordingly, the design engineer normally tries to find some compromise between adequate plans and specifications and sufficient flexibility to provide competition between all suitable materials and methods. If there is a geotechnical engineer employed on the project, then one of his most important duties will be to advise the structural engineer as to the most efficient methods and relative costs, particularly those costs influenced by subsurface conditions, such as boulders, groundwater, etc.

Typically, a decision is made to design for either piles or drilled shafts. Occasionally on a large project, both a pile and a drilled shaft design is prepared. This requires two complete sets of drawings and specifications. The bidders can bid on either one with the owner then taking that method selected by the low bidder for the entire project.

For a drilled shaft design it is not too difficult, if conditions warrant, to include on the same drawings both a straight shaft and a belled pier design.

Very frequently the designs for pile foundations can be and are set up to permit competition between both different materials and different methods of installation. Usually the geotechnical and structural engineers together choose the pile capacity that seems reasonable in consideration of both the subsurface conditions and the weight and other characteristics of the structure to be supported. Occasionally, again when the size of the job will warrant, two different pile groupings are shown on the drawings, one for perhaps a medium capacity pile and the other for a high capacity pile. Most typically, however, the documents will identify the required capacity of the pile and then state one or more materials that can be used, minimum dimensions for the various materials, approximate lengths that will be required, criteria for controlling the installed length and verifying the capacity of the pile.

Again, we note that the designer has a duty to prepare documents that will, if followed, result in a satisfactory foundation.[17] In the typical bidding process the bidders must conform to the bid documents; i.e., a bidder cannot unilaterally submit a proposal for an alternate design that does not conform to the bid documents.[18] In bidding on any kind of public construction, the owner cannot accept non-conforming bids.[19] When contracts are negotiated the methods of arriving at the most economical foundation and final price can be much more informal.

Most deep foundations are installed by specialty contractors. More often than not a given contractor will drill shafts or install piles but not both. Among pile contractors, one might specialize in mandrel driven shell, another in top driven pipe or H-piles and a third in some patented method, such as intrusion-type, pressure injected or the Interpile. It is common for the specialty subcontractor to take the initiative in selling their particular method to a structural engineer. Occasionally the engineer will be convinced to open up his specifications after they have been out for bidding to allow some system that was precluded by the specifications as they were written. It is not unusual for the specialty subcontractor who installs a patented method to provide a guide specification for his particular type of foundation. This action presents an interesting legal question as to the relative responsibilities of the engineer and the contractor. The engineer is still primarily responsible for the adequacy of the plans and specifications, therefore, if he accepts a proposal that was completely inappropriate for the particular conditions, then negligence could possibly be shown. However, common sense tells us that if a specialty subcontractor is successful in promoting his patented method using his guide specifications, then he has assumed greater responsibility than the contractor bidding on an open specification. Certainly he would be put in a hard position to complain, both from a business and legal viewpoint, if he encountered more difficulty in installing his foundation than he had expected. Also, there is some law to suggest that, in his action, he has warranted the suitability of this system from the design and construction viewpoint.[20]

LUMP SUM VS. UNIT PRICES

Certain types of projects are traditionally paid as a lump sum while on others payment is almost always on a unit price basis. For example, foundations for highway structures are almost always paid for on the basis of unit prices, while most buildings are bid for a lump sum of money.

The owners of private buildings have possibly a greater need to establish their cost in advance of commitment than does a government agency. When the building contract includes a deep foundation considerable thought should be given to the pro's and con's of lump sum vs. unit price payments. Common practice is to require a lump sum price for a specified quantity of work such as a certain lineal feet of piles or piers to a specified elevation and then have add and deduct items if the actual required quantities are more or less than the specified amount. In the following discussion this procedure is considered as a unit price basis of compensation as final payment is determined by the quantity of work actually installed.

The subsurface conditions play a primary role in the decision as to method of payment. There are some sites in the Ohio River Valley where the length of piles and bottom of piers can be determined very precisely. The bearing level will vary only a matter of 1 to 2 ft. over the entire building area. Where these conditions exist there is little risk to the contractor in figuring a lump sum bid and no obvious reason for a dispute to arise. For other geologic conditions the engineer will consider himself fortunate if he comes within plus or minus 20% in predicting the total footage of piles or length of piers actually required.

It is the owner's responsibility to establish the bottom of each pile or pier and the contractor should be paid for whatever length is, in fact, required; not for how long he takes to do it, but for all of the materials he is required to use. This can only be accomplished with a unit price contract.

In a lump sum contract the contractor bears the risk of unforeseen conditions which result in variations from the assumed bearing elevation. There is usually an exculpatory clause in the contract that disclaims any liability for any conditions that may cause any increase in quantities. In the lump sum contract the engineer does not usually estimate the quantity of work required, but rather states that the piles should have a specified approximate length driven to a minimum bearing capacity as determined by some method, such as a dynamic pile formula, or that piers shall be carried to a certain material at approximate elevations as indicated on the boring logs. A lump sum contract for deep foundations not only greatly increases the probability of claims but also the chances of recovery if the subsurface conditions vary substantially from those indicated. It also places an extra burden on the engineer in insuring that the piles or piers have extended to a sufficient depth to safely carry the load for which they are designed. The attitude of the contractor is certainly going to be different if he is paid a reasonable sum for whatever he is required to do than if he is going to get the same amount of money regardless of how long it takes or how much concrete he has to buy.

The legal basis for reformation of a fixed price contract is the doctrine of mutual mistake.[21] A substantial change in the quantity of work performed could result in the legal imposition of a new price based on the fair and reasonable value of the performance of the work; i.e., quantum meruit.[22]

In a unit price contract there is usually no basis for a claim unless the time required to do the work is substantially greater than what the contractor might have reasonably expected or some basic change in equipment is required, for instance, having to obtain different leads or mandrels because the piles are substantially longer than anticipated or having to bring in an extra crane to pull casing out of pier holes because depths were substantially greater than estimated or the necessity to use dewatering methods significantly different from what he might have reasonably expected. There has, however, been litigation where the quantities actually required varied from the estimated quantities.[23] Disputes most commonly arise when the actual quantities are substantially less than the bid quantities and the total monies received are insufficient to adequately cover the contractor's overhead.

Unit price contracts commonly state that the estimated quantities are set forth only to form a basis for comparison of bids. It is a general rule that no recovery may be had by reason of variance in the estimate regardless of how great the variance may be.[24] Some contracts, however, state that a certain variance, typically 25%, on a major item will serve as a basis for renegotiation.[25]

It is a common procedure on a private contract for drilled shaft foundation to require a lump sum price for the bottom elevation as shown on the plan and an add and deduct price for each diameter of shaft. In order to overcome tendencies for very large variations between the add price and the deduct price it is not uncommon to set up the bid sheet so that the add price must be the same as the deduct price. If the total quantities are substantially less than the estimated quantities over which the contractor distributed his fixed costs, then such a change could be a basis for reformation of the contract. A possibly more equitable procedure is to provide for add prices that are somewhat higher than the deduct prices with the owner exercising his right to refuse the bid if the prices are unreasonable. One method of overcoming the problem of evaluating bids with add and deduct prices that may vary considerably is for the owner to stipulate what the add and deduct payment will be (the amounts must, of course, be reasonable) and the competition is limited to the lump sum bid.

QUALITY CONTROL

In the usual building contract where AIA forms are used, the architect contracts with the owner to provide general supervision. Such an agreement requires the architect to periodically visit the job site to become familiar with the progress and quality of the work and to determine, in general, if the work is proceeding in accordance with the contract documents.[26] The architect is not required under this agreement to provide exhaustive or continuous on-site inspection or to provide testing services. Frequently, certain quality control tests are made a responsibility of the contractor, however, it is becoming quite common for the architect to act as an agent for the owner in arranging for independent geotechnical testing and inspection services. These services should be and normally are provided by the geotechnical engineer who prepared the original site investigation report.

Full-time inspection by the geotechnical engineer or qualified technician is essential in the installation of most deep foundations. A determination of the length of each pile or the depth of each pier is clearly the responsibility of the owner unless the contractor explicitly agrees to accept that responsibility. These determinations, which must be made in the field at the time the foundation unit is being installed, are highly technical and require specially qualified and trained personnel. Foundations differ from any other portion of the building. Once they are installed the cost and difficulties of verification are very high. Quality control must be exercised as the foundations go in the ground or not at all.

The presence of the geotechnical inspector clearly affects the responsibility and ultimate liability of the contractor. Although the contractor has a contract with the owner to meet the plans and specifications, independent of any action of the geotechnical inspector, courts have held that if the inspector has approved the work the contractor has fulfilled his obligation, even though the result may not be satisfactory. The legal point is the item of control.[27] Clearly the inspector has certain control over the project or there would be no purpose in his being there.

The matter of liability of the inspector to the owner and also to the contractor is beyond the scope of this paper. His presence does not warrant the end result; i.e., that no settlement beyond the specified amount will result, however, he must exercise due care in his work, the standard being the average for his particular service in that particular locale. He may also have certain duties relative to the safety of the contractor's personnel, particularly on matters of a technical nature.

CHANGED CONDITIONS

Little attention is paid to details for construction contracts until some problem arises. It is not at all unusual in the construction of deep foundations for a condition to be encountered that requires the use of some equipment or procedure different from what was thought would be necessary at the time of bidding the job or encountering conditions that result in the work going substantially slower than it was estimated that it would. The term that is usually used to describe this situation is "changed conditions". The issue is who is to bear the additional cost that results from such "changed conditions", the owner or the contractor.

If the contractor has discovered that conditions differ from those anticipated he is bound nevertheless to complete the project even if doing so causes him increased expense and greater difficulty than he expected.[28] In 1918 the Supreme Court of the United States in United States v Spearin,[29] stated, as a basic rule of contract law, that "when one agrees to do a thing possible of performance he will not be excused or become entitled to additional compensation because unforeseen difficulties are encountered." There have been substantial developments in the law in the 60 years since 1918 such that very often the contractor is due additional compensation when he encounters difficulties resulting from unanticipated subsurface conditions.

To consider the matter in detail we need a definition of "changed conditions". The following is given in the U.S. contract form for construction contracts. It is there defined as "(a) subsurface or latent physical conditions at the site differing materially from those indicated in this contract or (b) unknown physical conditions at the site of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in work of the character provided in the contract."[30]

The first portion of this definition relates to misrepresentations in the contract documents as to an existing condition, innocent or otherwise. The second portion does not rest upon misrepresentation but rests upon surprise; a physical condition one would not normally expect to encounter.[31]

If a subsurface condition is encountered that may be different from what could have been reasonably anticipated from the contract documents, then the next important matter is the wording of the contract relative to such changed or differing conditions; i.e., does the contract include a clause which provides for renegotiation for a valid changed condition or is the contract silent on this matter. If it is silent then it is the intent of the contract that the contractor shall bear all risks of unforeseen conditions.[32]

Just because it is the intent of the owner that the contractor shall bear these risks, this does not in itself mean that the contractor cannot in some cases recover additional costs. It does mean, however, that normally the probabilities of such recovery are less and the difficulties much greater than if there is an agreement in advance concerning unforeseen conditions.

The reason that the contractor may recover additional costs, even though it was the owner's intent to place all risks on him, is that the owner warrants the adequacy of the plans and specifications. The plans and specifications cannot be deemed adequate without reference to the conditions at the site at which the construction is to be performed. The contractor must base his claim on a theory of misrepresentation or possibly a mutual mistake. In a relatively recent case Peter Salvucci & Sons, Inc. v. State,[33] the court made note of the general rule laid down in United States v. Spearin, that one who has contracted to do a thing for a stated price will not be entitled to extra compensation because he has encountered difficulties that have not been provided for in the contract, but stated that, however, "the contractor who, acting reasonably, is misled by incorrect plans and specifications issued, as a basis for bids and who, as a result, submits a bid which is lower than he would have otherwise made, may recover in a contract action for extra work or expenses necessitated by the conditions being other than as represented."

The United States Supreme Court in a 1915 case, Christy v. U.S.,[34] listed the elements necessary for the contractor to establish in order to recover in an action for misrepresentation. The court stated that the first and paramount element is a positive representation in the body proper of the formal contract or information incorporated by reference into the contract must contain some reference to conditions presently existing at the site of construction. This representation must be material to the bargain and must be basic to the work done under the contract, otherwise the representation would be trivial and its accuracy inconsequential. Next, in making his bid for the job, the contractor must rely on the accuracy and veracity of the representation and must be justified in his reliance. Then the actual conditions encountered on the job site must differ from their description in the contract. Finally, the discrepancy between the actual conditions encountered and their representation in the contract must result in damages suffered by the contractor in the nature of additional expense, extra work, increased materials or augmented labor necessary to complete the project as specified in the contract.

In reviewing the requirements more specifically then the following is necessary to establish recovery on the theory of misrepresentation.[35]

1. There must be a positive representation;

2. The representation must be material to the bargain;

3. The representation must be basic to the work to be done under the contract;

4. The contractor must rely on the accuracy and veracity of the representation;

5. The contractor must be justified in his reliance;

6. The actual conditions encountered must differ from their description in the contract;

7. The discrepancy must result in damages suffered by the contractor.

An important aspect is whether the representation made in the contracts constituted a positive assertion or a mere suggestion. If the owner makes a positive and material representation as to a condition presumably within his knowledge then the contractor has a right to rely thereon.[36] The owner is considered to have warranted such a condition despite the ordinary requirement of an on-site inspection by the contractor and ordinary disclaimer as to subsurface data. If, however, statements made in good faith could be considered as suggestions only, the expenses caused by unforeseen conditions will remain on the contractor, especially if the contract so stipulates.[37]

In the case of deep foundations, the matter of whether the information provided in the documents is a positive assertion or merely a suggestion depends upon the facts of each individual situation. In the usual situation where the test borings have been conducted and results accurately reported and the contractor thereafter discovers that the test borings, though accurate, are not indicative of the total area to be excavated then the test borings are legally considered as only suggestive of the total conditions and the owner is not liable for any conclusions that the contractor may draw from such information.[38] Even if the design engineer forms a conclusion concerning the subsurface conditions which may have been incorrect, but he makes all of its borings and test results available to the contractor, the contractor is charged with the knowledge that the test borings are determinative only at the precise location of the hole.[39] The contractor is charged with the knowledge that the borings are not conclusive but are merely indications from which deductions might be drawn as to the total conditions.[40] In general, the only warranty made by the owner is that the borings and other tests were made and conducted by approved methods and were accurately reported.[41]

Typically, the contract which includes an exculpatory clause disclaiming liability or responsibility for the accuracy of borings or other subsurface data or for subsurface conditions generally also requires that the bidders inspect the site and further that they are to make their own tests in order to satisfy themselves that they have adequate knowledge of the subsurface conditions. If the contractor makes a claim on the basis of misrepresentation in a contract that included such a requirement, then the owner will cite as a defense the contractor's failure to make such tests. If the contractor can show that the data presented was clearly inaccurate, then the contractor's failure to make his own tests which may have shown this will not defeat his right to recover.[42] When the condition is indicated by the contract documents as existing the contractor has the right to rely on the representation. He need not check to verify it.[43]

Overall the courts have not been too sympathetic of the owner attempts to pass on to the contractor the responsibility for making subsurface investigations.[44] Considerations are both one of time; i.e., the amount of time allotted after the bid notices are out for the contractor to make such investigations, and also economic considerations. The cost of bidding is already a major item in the construction industry. The cost of making an adequate subsurface investigation can conceivably double the bidding cost. For typical building jobs, rarely does the contractor make an independent investigation, although occasionally he will move a caisson rig on the site for a day and make some test drilling to help him judge how difficult the drilling is going to be and to evaluate dewatering and caving problems. On large earthmoving projects such as highway or earth dam, supplementary investigations by the bidder to aid him in evaluating his excavation costs and to explore for borrow are much more common and the successful contractor is charged with the knowledge that he gains from any such explorations that he makes.[45]

It is the general concensus that the best procedure is for the owner to make a fully adequate investigation and to make all data developed therein, including interpretative reports, available to all bidders.[46] There is certainly the trend today to do this along with including changed condition clauses in the contract. Today all Federal construction contracts, many State contracts and all contracts using the AIA standard forms have some kind of changed condition clause. The basic reason is to attempt to get more competitive bids, ones that do not have large contingency factors, and also to avoid the considerable cost of disputes and litigation that are so frequent when there is not such a clause. When the wording of the contract clearly states that all unforeseen conditions are the contractor's risk, then it is very difficult for architects and engineers to advise the owner that he should make a settlement with the contractor, except in rare cases when the information included in the contract documents was clearly in error. Out and out errors are rare. Problems and disputes usually arise in cases where geologic conditions are difficult and erratic and the subsurface investigation, even though it may have been totally sufficient for the designer to make the decisions that he needed to make, just did not adequately show the bidders everything they would have to contend with.

In Federal construction contracts (Standard Form 23A) the changed condition clause is now called "differing site conditions". This more reasonably states the matter that we are dealing with; i.e., not a change from that which ever existed, but rather something different from what was thought to exist. Both the AIA and NSPE standard contract forms now include changed condition clauses.

Both the AIA clause and the Federal Government clause describe two distinct types of compensable unexpected conditions:

1. Conditions at variance or differing materially from those indicated by the contract documents, and

2. Unknown physical conditions of an unusual nature differing materially from those ordinarily encountered and generally recognized as inherent in work of the character provided in this contract.

However, the NSPE clause provides relief for only the first condition. Here, the contractor assumes more of the risk of a differing site condition.[47]

Most adjudicated disputes have arisen in connection with Federal construction contracts, however, judicial decisions interpreting NSPE and AIA type clauses have recognized the relevance of Federal precedent and rationale used by the Court of Claims and Board of Contract Appeals and have applied these principles to private construction cases.[48]

To recover for a Type 1 changed condition the contractor must generally show four things: (1) the actual conditions, (2) the conditions indicated by the plans, specs. and other contract documents, (3) the variance and (4) that he gave proper notice of that variance.[49] The most critical element is ultimately the conditions indicated in the contract. If the contract is silent

5

regarding subsurface conditions, there can be no Type 1 recovery because there was no representation or indication made by the contract documents. It is not required, however, that the contract indications be "explicit or specific, but only enough to impress or lull a reasonable bidder not to expect the adverse conditions actually encountered."[50]

How far the courts have gone on allowing recovery is indicated in Foster Construction C.A. v. United States[51] Here, the specifications directed that all concrete be poured in the dry without a seal or seal class concrete. This was deemed a sufficient indication that the subsurface conditions would be dry. When the contractor encountered subsurface conditions which made excavation in the dry "impracticable" he was allowed recovery for differing site conditions. Thus the performance standards, the design or the material specified in the contract may provide a sufficient indication of the site conditions to warrant recovery when actual conditions vary from such indications.[52]

The more usual situation relatively to a Type 1 recovery is illustrated by a contract involving deep sewer construction. This is the case previously referred to where, as is typical in sewer work, the boring logs were presented on the plan profile sheets. This data indicated water at certain levels, but gave no indication that the water rose in the test hole. Notes on the original boring logs did show the data concerning final water levels. Although there was reference in the bid documents to the original logs and an invitation to all bidders to inspect the test boring information on file with the owner, the court found that the inclusion of the summary of boring data on the contract documents was a sufficient indication that static and not artesian water could be expected.[53] When artesian water was encountered the cost of the project substantially increased and the contractor was found to be entitled to recovery for a Type 1 changed condition.

Type 2 changed conditions; i.e., unknown physical condition below the surface of the ground of an unusual nature differing materially from those ordinarily encountered and generally recognized as inherent in the work of the character provided for in the contract, are conditions for which a contractor can recover even though the contract is silent with regard to such conditions. To obtain a Type 2 recovery, instead of showing a mere variation from conditions indicated by the contract, the contractor must show that actual conditions encountered were unusual and differed materially from those which were reasonably anticipated. The owner need not have made any specific representation as to subsurface conditions. The crucial issue is whether the contractor reasonably expected what he found.[54]

The unknown requirement does not refer only to the actual knowledge of the contractor. If he reasonably should have expected the condition (based on what is ordinarily encountered and generally recognized as inherent in work of the character provided for in the contract), this constructive knowledge is sufficient to bar recovery.(55)

Buried timbers, undisclosed utility lines, boulders, old foundations and water tables at higher than anticipated levels are examples of subsurface conditions which have been found to warrant Type 2 recovery.(56)

It is not unusual to find contracts which include the standard AIA forms and a complete disclaimer concerning all soil investigation data. The disclaimer clause cited earlier was obtained from a contract which included the current AIA documents. The following is a full section on "Concealed Conditions" in the 1976 edition of the AIA documents.

> 12.2.1 Should concealed conditions encountered in the performance of the work below the surface of the ground or should concealed or unknown conditions in an existing structure be at variance with the conditions indicated by the Contract Documents, or should unknown physical conditions below the surface of the ground or should concealed or unknown conditions in an existing structure of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in work of the character provided for in this Contract, be encountered, the contract sum shall be equitably adjusted by Change Order upon claim by either party made within twenty days after the first observance of the conditions.

The architect here was trying to cover all bases. More specifically, he wanted to use the AIA standard contract, which has a changed conditions clause, however, he attempted to specifically exclude the soil investigation data from the contract documents, thereby eliminating this data as a basis for reference in trying to measure conditions actually encountered against those indicated by the contract documents.

It can be concluded with reasonable confidence that such an attempt to exclude the subsurface information data from the contract documents would be unsuccessful. It has been held in a number of cases that such exculpatory clauses will not restrict the application of the changed conditions clause. The court in <u>Metropolitan Sewage Commission v. R. W. Company</u> held that "to allow such provisions to cancel out the changed conditions clause

7

would destroy the whole equitable adjustment procedure which the contract provides for when materially different conditions are encountered."[57]

All changed condition clauses require the contractor to give prompt notice to the owner of the differing conditions so that the owner can examine and evaluate the conditions encountered and consider the changes in design or performance required and requirements to avoid or minimize the extra cost caused by the differing conditions and oversee the keeping of cost records on the differing conditions. It was held in Blankenship Construction Company v. North Carolina State Highway Commission that the owner should not be obligated to pay a claim unless it is given a reasonable opportunity to insure that the claim is based on actual determinations of work and cost.[58] In this case it was determined that the contractor, in making an inquiry of the engineer with regard to the presence of unexpected rock formations, rather than a forceful indication of changed conditions and demand for equitable judgment. had not satisfied contractual notice requirements.

The courts do not always require strict compliance with written notice requirements. Where the underlying purpose of the requirement is served by substantial compliance or where the owner suffers no prejudice from the contractor's failure to comply fully with the written notice requirement, the requirement will not bar recovery.[59] Although giving notice in writing creates a record of the fact that notice was given and is therefore preferable to oral notice which may be difficult to prove by the time a claim reaches trial, numerous courts have found that oral notice to an authorized representative of the owner satisfies the notice requirement.[60] Occasionally the requirement for written notice is not enforced if it can be shown that the owner had actual knowledge of the differing site conditions.[61]

The importance of record keeping is described in Construction Contracts 1977[62]. "A crucial fact that no contractor should forget is that proving the existence of differing condition is only the first step towards securing just compensation for the extra cost incurred. The contractor cannot recover a penny until he can prove with some degree of certainty what the changed condition cost him. The extent of the detail which a court may require is illustrated by the case of Ray D. Louder, Inc. v. North Carolina State Highway Commission.[63] In the Louder case the contractor's first job superintendent had kept daily records showing the progress of the work, the number of men and the equipment on the job. The second superintendent did not even show this information on his daily reports. When the contractor was preparing his differing site conditions claim he rehired the first superintendent to go over the daily reports and prepare a cost summary. The court held that the cost summary was inadmissible and would not allow the contractor to use it to prove how much he was damaged. Furthermore, the court said that even the daily reports themselves were incomplete and unreliable because even those prepared by the first superintendent failed to show how many hours each machine was in operation, whether any machines were broken down for part of the day or even used that day at all. This case emphatically demonstrates the value of (1) complete, (2) detailed and (3) accurate cost records. It is almost certain that the contractor, Louder, spent a great deal more money as a result of differing site conditions than he could prove with a reasonable certainty. A complete and detailed record keeping system would have provided the proof that the court found lacking. The law does not grant recovery on probable losses. It requires reasonable certainty as to amount. Maintaining sketchy or intermittent records is like leaving money on the table."

CONCLUSIONS

In 1973 and 1974 a study was conducted by Standing Subcommittee No. 4 of the U.S. National Committee on Tunneling Technology of contracting practices in underground construction. The results of this study are presented in a report "Better Contracting for Underground Construction, November 1974".[64] Although the study and report concern tunneling construction, much of the subject matter is common to all types of subsurface and foundation work. The report is recommended reading for all engineers and contractors who do subsurface work. One of the most important findings of the report is the importance of cooperation between all parties to a contract for subsurface construction.

Perhaps the first important step in a project involving deep foundations is the subsurface investigation. Too much emphasis cannot be placed on the importance of an adequate investigation. Most often the fact that a deep foundation is required is determined from the initial subsurface investigation. It is not unusual that the initial or preliminary subsurface investigation does not provide all the information needed for design and cost estimating, particularly where difficult or variable conditions exist. If there is a geotechnical engineer involved in the project, then he should never be embarrassed to recommend that additional work be done and the structural engineer should back his recommendation as long as he judges it to be reasonable. There is without question a relationship between the adequacy of the subsurface investigation and the probability of problems and disputes arising during construction.

One of the primary recommendations of the National Committee on Tunneling Technology was that "All factual subsurface data, professional interpretations thereof, and design considerations thereby raised should be made available to bidders, but with a careful distinction drawn between factual data and interpretation or opinion." The law is clear that withholding of factual data by the owner will provide a basis for a claim by the contractor on the theory of misrepresentation if the contractor can show that he suffered damages because he did not know of this factual data. There is no one best way to provide this data to the bidders. On private work where a geotechnical report was prepared, then it is becoming rather common to include the entire report of the geotechnical engineer into the project manual. This is not always feasible or desirable. On some projects, particularly large ones, such a report or reports could contain considerable data that is not relevant to the final design. As a minimum, the geotechnical engineer's or geologist's interpretations of the factual data should be made available to the bidders.

A "changed conditions" clause providing, in essence, for assumption by the owner of the risk concerning unknowns in subsurface physical conditions, should be included in all contracts for deep foundations. AIA General Conditions, Contract for Construction, Document A-201 is used in private work today. Section 12.2.1 of that document, as previously quoted, is an adequate "changed condition" clause for most work.

There should be no disclaimer clause in the contract documents. There should be a statement concerning the subsurface data. The statement previously quoted from the proposed ACI Standard Specifications for End Bearing Drilled Piers is considered as effectively presenting the proper position concerning the subsurface data. If interpretations and opinions of the geotechnical engineer are included, then the contract clause must make appropriate reference to such interpretations and opinions.

The contract for deep foundations should provide for compensation on some type of unit price basis. The contractor should be adequately compensated for all materials that are actually installed. Where a lump sum payment is agreed upon, on the basis of an estimated total footage, adding or deducting using unit prices to adjust for the footage actually installed, then the agreement should reasonably consider the fixed cost where the actual footage is substantially less than the estimated amount. One possible method of providing for these fixed costs is to have a pay item for mobilization. The contractor can include his moving and set up costs in this mobilization item.

There should be a representative of the owner on the job at all times that the foundations are being installed. Preferably, this representative should be an employee of the firm that did the subsurface investigation. There should be no contractual relationship between the inspector and the contractor Whenever possible, the inspector should be employed directly by the owner. The inspector should verify all aspects of foundation construction, including layout, geotechnical aspects and quality control on all materials. He should keep a daily record of all work done, lengths installed and any work that is to be paid for on a unit price basis such as rock excavation. He should immediately notify the engineers responsible for design of any problems or unusual conditions encountered.

If problems or unexpected conditions are encountered, then all parties involved in the design should be promptly notified regardless of whether or not they are represented in the field. It is not uncommon and extremely frustrating for a geotechnical engineer to be notified after foundation work has been completed that they ran into some problems, the contractor is asking for some additional money and we need your opinion as to why this happened. The geotechnical engineer and the structural engineer should report to the job site as quickly as possible. If the problem appears serious, then the work should be suspended until they arrive. It is possible that some additional investigation may be needed. If so, it should be promptly authorized and undertaken.

All parties should approach the problem in an objective and not a defensive manner. If you do not accept that problems and surprises in subsurface construction are to be anticipated, then you should not be in the business of designing and constructing such work.

The owner's representatives should make a determination as promptly as possible as to whether conditions encountered or exposed are different from what they anticipated and, if so, whether they require any change in the design or will cause, in their judgment, increased cost to the contractor. They should notify the contractor of their decision. If the contractor does not agree with their decision, he should promptly give notice in writing that he has encountered a differing site condition.

Regardless of whether an agreement is initially reached as to whether there is, in fact, a changed condition, the engineer and the contractor should cooperate in keeping records of any additional work that may be necessary. The theme of all of the above is cooperation in dealing with the problem when it is encountered. The most effective time that the parties to a contract for deep foundations can spend is when the work is being done, not two years later with each party having a lawyer at his side.

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FOOTNOTES

1. Peter Kiewit Sons Co. v. Metropolitan Sewer District of Illinois, USDC, Northern D. of Ill. (1973)

2. Helene Curtis Industries, Inc. v. United States, 312 F. 2d 774 (1963)

3. Hallerbach v. United States, 233 U.S. 165 (1914)

4. City of Salinas v. Souya & McCue Construction Co., 424 P2d 921 (1967)

5. Depot Construction Corp. v. The State of New York, 246 N.Y.S. 2d 527 (1964)

6. J. Sweet, Legal Aspects of Architecture, Engineering and the Construction Process, 351 (1970)

7. "Better Contracting for Underground Construction", National Committee on Tunneling Technology. NSF (1974)

8. Supra, note 6, at 432

9. Metropolitan Sewage Commission v. R. W. Construction, 241 N.W. 2d 391 (1976)

10. Appeal of Jackson-Swindell-Dressler, Eng. BCA No. 3614, 76-2 BCA No. 222 (1976)

11. Dravo Corporation v. Commonwealth of Kentucky, Dept. of Highways, Ky. App., 564 S.W. 2d 16 (1977)

12. Arthur A. Johnson Corp. v. City of New York, 295 N.Y.S. 547 (1936)

13. Mounts v. Roberts, Ky., 388 S.W. 2d 117 (1965)

14. Codell Construction Co. v. Commonwealth of Kentucky, 566 S.W. 2d 161 (1977)

15. "Construction Contracts 1977", Practicing Law Institute at 117

16. Jefferson Construction Co. v. United States 392 F. 2d 1006 (1968)

17. Supra, note 16

18. Miller v. Milford, 276 NW 826

19. Boder v. Sharp, 125 A 2d 499

20. Barraque v. Neff et al 11 So. 2d 697 (1942)

21. Supra, note 6 at 431

22. Foundation Co. v. State, 233 N.Y. 177, 135 N.E. 236 (1922)

23. Greenberg, "Problems Relating to Changes and Changed Conditions on Public Contracts", 3 Public Contract Law Journal, 135 (1970)

24. Peter Kiewit Sons Co., BCA 739, 3 CCF 733 (1945)

25. Kentucky Department of Transportation, Standard Specification for Road and Bridge Construction (1976)

26. "Supervisory Duties of an Architect", 3 Memphis St. L. Rev. 169 (1972)

27. Reber v. Chandler High School District #202, 13 Ariz. App. 133, 474 P. 2d 852 (1970)

28. United States v. Spearin, 284 U.S. 132 (1918)

29. Id.

30. 41 C.F.R. § 1-7-601-3 (1974)

31. Supra, note 23

32. 6 A. Corbin on Contracts 1333 at 566 (1962)

33. 110 N.H. 136, 268 A. 2d 899 (1970)

34. 237 US 234 (1915)

35. Brief for Appellee at 136, Dravo Corp., Commonwealth of Kentucky, Dept. of Highways, Ky. App. 564 S.W. 2d 16 (1977)

36. Supra note 3

37. Wunderlich v. State, 65 Cal. 2d 777, 423 P. 2d 545 (1967)

38. Chris Nelsen & Sons v. City of Monroe, 337 Mich. 438, 60 N.W. 2d 182 (1953)

39. J. A. Thompson & Sons v. State, 51 Hawaii 529, 465 P. 2d 148 (1970)

40. Elkin v. Sebastian Bridge Dist. 291 F. 532 (1923)

41. Supra, note 37

42. Supra, note 22

43. Supra, note 3

44. Supra, note 9

45. State Highway Dept. v. Wright Contracting Co., 107 Ga. App. 758, 131 S.E. 2d 808 (1963)

46. Supra, note 7

47. Supra, note 15 at 170

48. Supra, note 15 at 171 and note 9

49. Supra, note 15 at 171

50. Supra, note 9

51. 193 Ct. Cl. 587 (1970)

52. Supra, note 15 at 172

53. Supra, note 9

54. Supra, note 15 at 173

55. Charles J. Parker Construction Co. v. United States, 193 Ct. Cl. 320 (1970)

56. Supra, note 23, at 158

57. Supra, note 9

58. 222 S.E. 2d 452, N.C. App. (1976)

59. Peter Kiewit Sons Co., 60-1 BCA 2580

60. General Casualty Company v. United States, 127 F. Supp. Ct. Cl. (1955)

61. Supra, note 15, at 176

62. Supra, note 15, at 176

63. 217 S.E. 2d 682 (N.C. 1975)

64. PB 236 973, National Technical Information Service

OHIO RIVER VALLEY SOILS SEMINAR

HISTORY

Since 1970, the American Society of Civil Engineers (ASCE) Geotechnical Groups of Cincinnati and Kentucky have collaborated to present annual speciality seminars to serve engineering practitioners and students in the Ohio River Valley Area. Organized in cooperation with the Universities of Cincinnati, Kentucky and Louisville, these one-day programs are presented, on a rotating basis, in close proximity to the host university city. These seminars are CEU accredited and are designed as continuing education programs for the exchange of information and experience on a geotechnical topic selected by the host committee. The success of these programs can be attributed to the contributions made by a wide range of practitioners in design, consturction and education from throughout the world of geotechnical engineering practice.

Previous seminar topics include:

1970	Building Foundation Design and Construction, Lexington, Kentucky
1971	Earthwork Engineering, Start to Finish Louisville, Kentucky
1972	Lateral Earth Pressures Fort Mitchell, Kentucky
1973	Geotechnics in Transportation Lexington, Kentucky
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1975	Slope Stability and Landslides Fort Mitchell, Kentucky
1976	Shales and Mine Wastes; Geotechnical Properties, Design and Construction Lexington, Kentucky
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