WAVE EQUATION PREDICTION OF PILE BEARING CAPACITY COMPARED WITH FIELD TEST RESULTS

By

Richard E. Bartoskewitz
Engineering Research Associate

and

Harry M. Coyle
Associate Research Engineer

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ABSTRACT

The bearing capacities of full-scale instrumented friction piles in clay are predicted by using a numerical method for solving the one dimensional wave-equation. The predicted capacities are compared with field data from static load tests. The results obtained by using currently accepted soil parameters, which characterize the dynamic response of a soil to impact loading, are compared to those attained by using soil parameters which were recently developed from model pile tests.

A study is made to determine the qualitative affects that the soil parameters have on the predicted capacity. Results of the parameter study indicate that the ratio of point load to total load has a significant influence on the accuracy of the predicted pile capacity. Conversely, the soil quake, the tip damping parameter, and the distribution of frictional soil resistance do not have a significant influence on the predicted capacity.

Wave equation analyses of data from full-scale pile tests at three different locations show that a single value for the friction damping parameter will not yield an accurate predicted capacity for all clay soils. Data are presented which show an apparent relationship between the friction damping parameter and the plasticity index of a clay soil.

SUMMARY

This test program was conducted during the third year of a five-year study on "Bearing Capacity for Axially Loaded Piles." A numerical method for solving the one-dimensional wave equation was used to predict the bearing capacities of full-scale friction piles in clay. The predicted capacities were compared to the capacities measured by static load tests.

A study was made to determine the effects that various soil parameters have on the prediction of bearing capacity. Data are presented which show that the friction damping parameter J' can be estimated on the basis of the plasticity index of a particular clay soil.

A method is proposed which eliminates the necessity of conducting static load tests to determine soil set-up. The proposed method utilizes data obtained by redriving a pile after a time interval has elapsed during which soil set-up has occurred.

IMPLEMENTATION STATEMENT

This is a technical progress report which presents the results of a test program conducted to develop soil parameters for a predominately clay soil. The soil parameters are intended for use with the computer program for solving the one-dimensional wave equation for the purpose of predicting the bearing capacity of pile foundations.

Implementation of the results of this study should be limited to applications with metal shell friction piles in clay. A value of 0.10 is recommended for the quake and a value of 0.15 is recommended for the point damping parameter. Implementation of the relationship between the friction damping parameters and the plasticity indices of various soils should be deferred until further verification has been obtained from additional field tests. Future field tests should include the measurement of point load through instrumentation, and the 10-day static load test should be performed concurrently with redriving of the test pile so that measured static bearing capacity can be correlated with wave equation predictions, thereby yielding an estimated soil set-up. The implemented results of this study should be utilized with existing design procedures pending further verification by additional field tests on full-scale piles.

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

Figure | Page

INTRODUCTION

Nature of the Problem

One of the important problems encountered by Civil Engineers involved in the design of pile foundations is the determination of the maximum static load that can be safely supported by a pile. For many years engineers have relied upon "static" and/or "dynamic" bearing capacity formulas to compute the load-carrying capability of piling. Many simplifying assumptions are used in the development of these formulas which decrease their accuracy and restrict their application. For example, the Engineering News formula currently being used by the Texas Highway Department was derived by neglecting the loss of energy which occurs during impact and assuming 100% mechanical efficiency (2).*

On the other hand, the wave equation method of analysis is a mathematically correct method which accounts for all important parameters and can be applied to a wide variety of pile types and soil conditions. Of particular importance to this investigation is the fact that the nonlinear static and dynamic stress-strain relationship of soil can be accounted for in a wave equation analysis.

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*Numbers in parentheses refer to the references listed in Appendix I. (The citations on the following pages follow the style of the Journal of the Soil Mechanics and Foundations Division, ASCE.)

Present Status of the Question

Isaacs is believed to be the first person to demonstrate the fact that the principles of longitudinal wave transmission in slender rods could be applied to the problem of pile driving analysis (8). This work did not receive immediate application because of the number and complexity of the equations involved in the solution. In 1960 E. A. L. Smith (13) presented a numerical solution of the wave equation applicable to the problem of pile driving. Based upon personal experience with the problem of pile driving, Smith recommended a model to describe the dynamic characteristics of the soil. The soil parameters used with his soil model were considered adequate for practical use until more accurate parameters could be established. In 1967 Lowery, Hirsch, and Samson (8) published a computer program for solving the wave equation using Smith's numerical method. However, until that time no work had been done to determine more accurate values for Smith's soil parameters.

Within the past three years there has been a considerable amount of research performed to determine representative damping coefficients for various types of soil (4, 6, 11, 12). The research primarily involved laboratory studies on specially prepared soil samples and model pile tests both in the laboratory and in the field. At present there have been no studies made to determine if the values thus obtained can be used to reliably predict the soil response when driving a full-scale pile under field conditions.

Objectives

The objectives of this investigation are:

a. To obtain static and dynamic field test data on full-scale instrumented piles.

b. To predict the ultimate static bearing capacity for each pile using the one-dimensional wave equation analysis.

c. To make comparisons between the predicted pile capacities and the actual capacities observed in the field.

d. To evaluate the accuracy of the parameters used with the soil model to describe the dynamic characteristics of the soil.

WAVE EQUATION IDEALIZATION OF SOIL BEHAVIOR

Smith's Soil Model

The model used by Smith to simulate the pile-soil system is shown in Fig. 1. The real pile is represented by a series of concentrated masses connected by weightless springs. The soil surrounding the pile is idealized by a series combination of a spring and sliding friction block connected in parallel with a dashpot. The load-deformation characteristics of the soil as shown in Fig. 2 are described by the parameters R_u, Q, J, J', and V, where

R_u = dynamic or static soil resistance in pounds;

Q = maximum elastic soil deformation, or quake, in inches;

J = a damping constant for the soil at the point of the pile, in seconds per foot;

J' = a damping constant for the soil along the side of the pile, in seconds per foot; and

V = the instantaneous velocity of a pile segment during a given time interval, in feet per second.

The total soil resistance mobilized during dynamic loading was given by Smith as:

$$R_{u_{dynamic}} = R_{u_{static}} (1 + JV) \tag{1}$$

Smith's recommended values for Q, J, and J' are shown in Fig. 2.

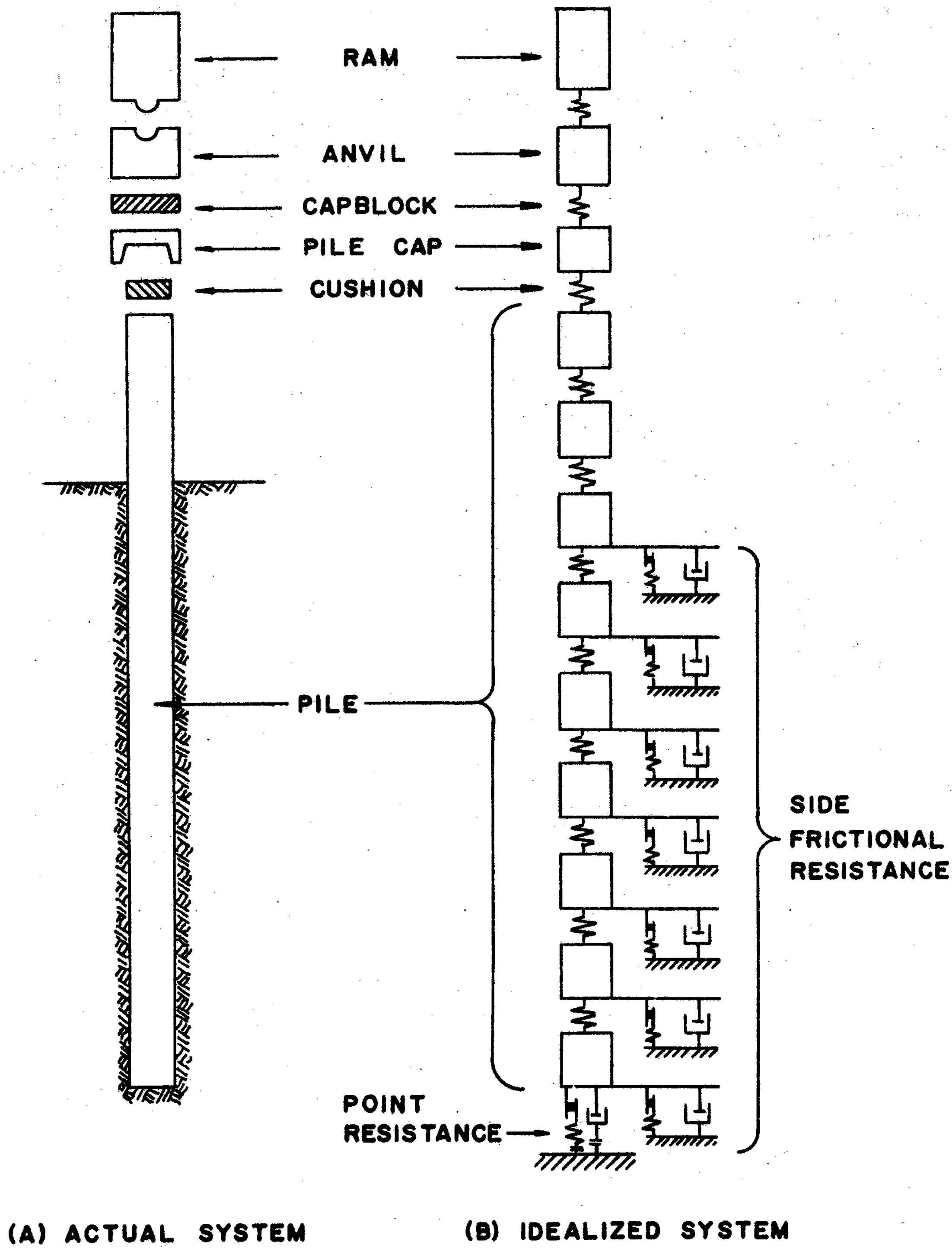
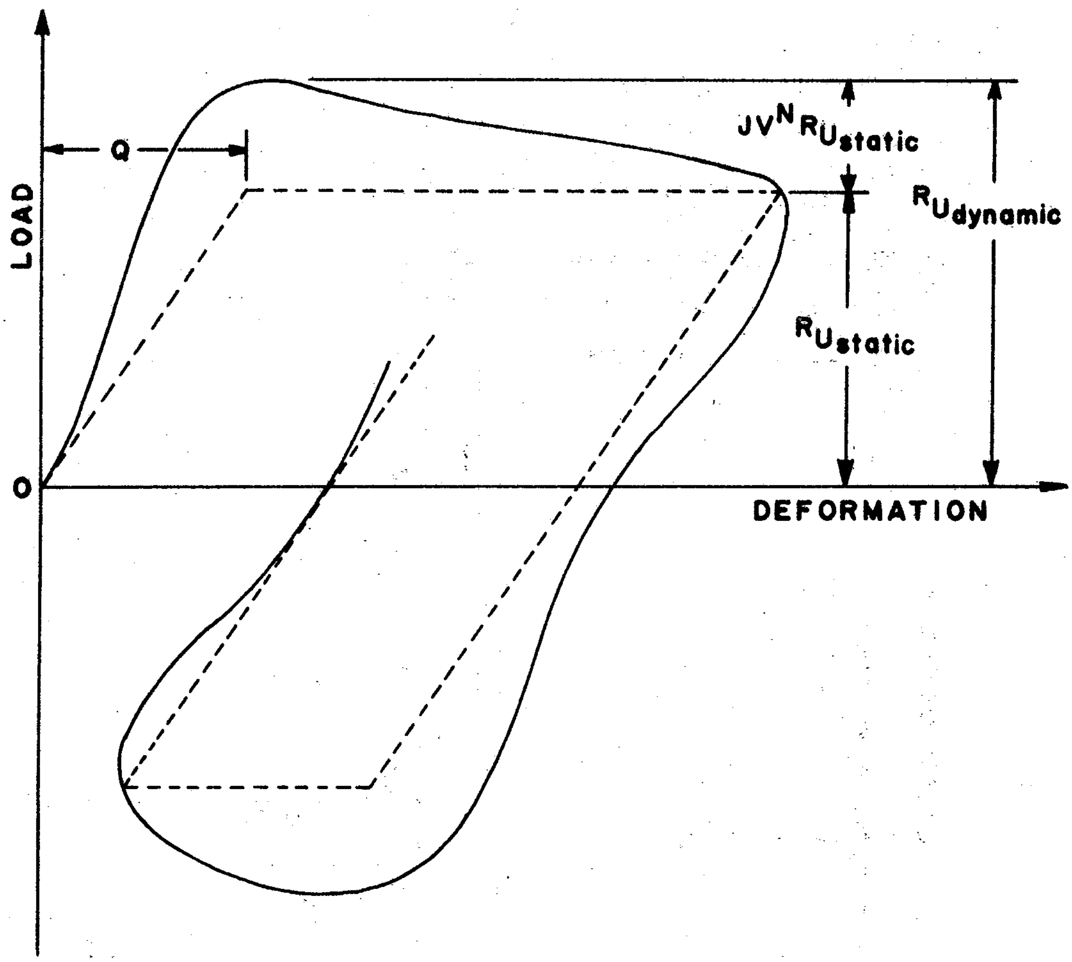


FIGURE 1 - COMPARISON OF A TYPICAL HAMMER-PILE-SOIL SYSTEM TO SMITH'S IDEALIZATION



SMITH'S SOIL PARAMETERS:

Q = 0.10 INCH
J' = 0.05 SECONDS PER FOOT
J = 0.15 SECONDS PER FOOT

MODIFIED SMITH EQUATION:

$$R_{U_{dynamic}} = R_{U_{static}}(1 + JV^N) \qquad 0 < N \leq 1.0$$

FIGURE 2 – ASSUMED STATIC AND DYNAMIC LOAD – DEFORMATION CHARACTERISTICS OF SOIL

Modification of Smith's Soil Parameters

In order to obtain quantitative values for Q, J, and J' for various types of soil a research program was initiated by the Texas Transportation Institute. Reeves (12) developed a dynamic loading apparatus and established procedures for testing saturated sands. By modifying Smith's equation for dynamic soil resistance [Eq. (1)], Reeves was able to obtain a constant tip damping parameter J for a specific range of deformation velocities.

Gibson (4) extended the work of Reeves to include clay soils. The affect of confining pressure on the dynamic properties of organic material tested in triaxial compression was also studied. A correlation was made between the tip damping coefficients and common index properties of soils such as the angle of internal friction and void ratio in the case of sands and liquidity index and moisture content for clays. Gibson found that by raising the velocity of deformation V in Eq. (1) to some exponential power N less than one the damping coefficient J is constant for a specified range of velocity. Thus, Eq. (1) was rewritten as:

$$R_{u_{dynamic}} = R_{u_{static}} (1 + JV^N), \; 0<N\leq 1.0 \qquad (2)$$

Raba (11) and Korb (6) performed tests on model piles driven in clays to determine the side damping coefficient J'. Korb obtained data to determine Q for sands and clays.

The values for the soil parameters obtained as a result of these research programs are listed in Table 1. These values are based primarily on the work of Korb because they were obtained from model pile tests on a wide variety of soils in the field. Hereafter they will be referred to as the modified soil parameters. The use of values which differ from those in Table 1 will be indicated in the text.

TABLE 1.--SUMMARY OF MODIFIED SOIL PARAMETERS FOR CLAY

Q side, inches (1)	Q point, inches (2)	J' seconds per foot (3)	N side (4)	J seconds per foot (5)	N point (6)
0.03	0.10	1.25	0.35	0.15	1.0

PREDICTION OF BEARING CAPACITY BY WAVE EQUATION ANALYSIS

General

The computer program developed by Lowery, Samson, and Hirsch (8) for solving the one-dimensional wave equation using Smith's numerical method was used consisently throughout this investigation. Any future reference to a wave equation analysis or solution should be construed to mean the analysis or solution obtained by use of the computer program. The program was run on the IBM 360/65, FORTRAN IV G, Release 18, OS360 facilities of the Data Processing Center, Texas A&M University.

Static Soil Resistance Versus Dynamic Driving Resistance

The prediction of the static bearing capacity of a pile by the wave equation analysis is predicated on the fact that a relationship can be established between the static soil resistance and the dynamic pile penetration resistance at the time of driving. Static soil resistance is usually expressed in convenient units of force such as kips or tons. Dynamic pile penetration resistance is expressed as the number of blows required by the pile driving hammer to produce a unit penetration of the pile into the soil. The more common nomenclature is driving resistance or blow count, and these terms will be used interchangeably throughout this work. The units most often used are blows per inch or blows per foot. In general, the relationship between static soil resistance and dynamic driving resistance is nonlinear. Of the many factors which govern this relationship, those which are considered to have the most prominent effect are: (1) the type of soil into which the pile is to be driven; (2) the size, geometry, and material of the pile; (3) the type, energy rating, and efficiency of the pile driving hammer; and (4) the accessories incidental to the driving assembly, e.g., cushions, adapters, and load cells. The purpose of this study is to investigate only the first of these factors, i.e., the parameters which are used with Smith's model to describe the dynamic response of the soil. The Michigan State Highway Commission (9) published a voluminous report on the driving energy output of various hammers and pile configurations. Detailed studies of hammer energies, dynamic properties of cushioning materials, etc., were published by Lowery, et al. (7, 8), and by Hirsch and Edwards (5). Although these factors are an integral part of a wave equation analysis, their investigation is not within the scope of this study and will not be discussed.

For a specific hammer-pile-soil system and a predetermined embedded depth of the pile, an arbitrary level of static soil resistance, RUT, is selected. The wave equation can then be used to compute the permanent set of the pile which would be caused by one blow of the hammer. The reciprocal of the permanent set gives the driving resistance in blows per unit of net pile movement. In this study RUT represents the ultimate static bearing capacity of the pile which would be measured if the pile could be load tested immediately upon completion of driving. If several values of RUT are selected and the corresponding dynamic driving resistances are computed by the wave equation, a curve similar to the one shown in Fig. 3 is obtained by plotting static soil resistance RUT versus dynamic driving resistance. This curve can then be used to predict the static bearing capacity of the pile if the actual driving resistance in the field is known for the last few blows of the hammer. For example, if Fig. 3 represents the actual curve obtained from a wave equation analysis of a particular hammer-pile-soil system, and the blow count recorded in the field during the last several feet of driving was 20 blows per foot, the indicated static bearing capacity of the pile as shown in Fig. 3 would be 30 tons at the time of driving.

At this point it must be emphasized that the predicted bearing capacity of a pile as obtained from a wave equation analysis does not reflect the increase in capacity which can be expected to occur after the pile is driven if the soil profile contains a significant amount of clay. Although a thorough study of "soil set-up" is beyond the scope of this investigation, some discussion on the subject is warranted. It is a widely known fact that as the pile is driven into the ground the soil beneath the point and

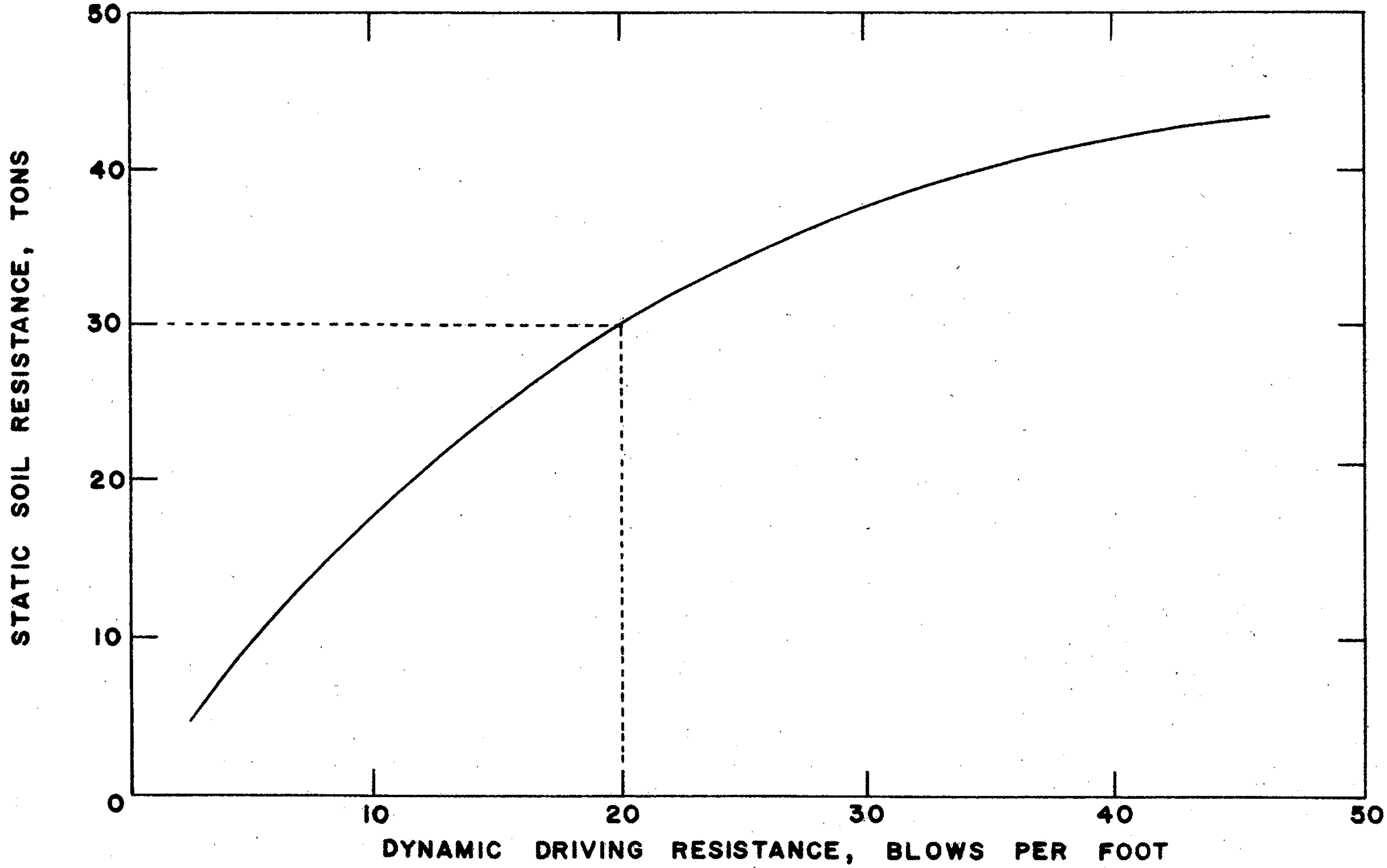


FIGURE 3 – TYPICAL RELATIONSHIP BETWEEN STATIC SOIL RESISTANCE AND DYNAMIC DRIVING RESISTANCE

along the side of the pile is remolded and compacted. In clay soils this results in an increase of pressure in the pore fluid within the voids of the soil skeleton. With the passage of time this excess pore pressure gradually dissipates and the soil consolidates around the pile with an attendant increase in the shear strength of the soil. The phenomenon accounts for the increase in bearing capacity and is frequently referred to as soil set-up. If the magnitude of the soil set-up which can be expected to occur at a particular site is not known, an approximate set-up factor of two can be used. That is, an approximate estimate of the ultimate static bearing capacity of the pile some time in the future can be obtained by multiplying the bearing capacity at the time of driving by a factor of two. In some instances the actual set-up will be in excess of two. For example, data recorded during the course of the Port Arthur, Texas, pile tests (which are described in a subsequent section of this investigation) revealed that set-ups of 2.16 and 2.43 had occurred. On the other hand, data reported in the Michigan Study (9) show that the set-up was 1.91 at the Belleville test site and only 1.45 at the Detroit site. Tomlinson (14) has presented data in the form of bearing capacity versus time curves from which a set-up factor of approximately two has been calculated. Thus, it is suggested that a set-up factor of two can be assumed in the absence of conclusive static load test data, but this assumption should be tempered by sound engineering judgement.

Parameter Study of Beaumont Field Test Data

In order to determine the qualitative affects which the soil parameters Q and J have on the relationship between static soil resistance and dynamic driving resistance, a parameter study was made utilizing field data from a full scale pile test. The prerequisite data for the wave equation analyses included information relating to the size and type of pile and hammer, the results of static load tests, and soils investigation information. The data were obtained from a report by Airhart, Hirsch, and Coyle (1) on a pile load test conducted in Beaumont, Texas. The pile tested was a 16-in. OD, $\frac{3}{8}$-in. wall, 53-ft long steel pipe pile driven into predominantly clay soils by a Delamg D-12 hammer. Smith's parameters and the modified parameters were used to develop the curves relating static soil resistance to dynamic driving resistance. This was done so that a comparison could be made between similar curves in order to determine the differences which are caused by using the different soil parameters.

In order to apply the wave equation analysis to a particular pile driving problem a certain percentage of RUT must be designated as point bearing resistance at the tip of the pile. Point bearing resistance will be referred to hereafter as RUP. The remainder of RUT, i.e., RUT minus RUP, acts as skin friction along the side of the pile. The ratio RUP/RUT can be chosen anywhere within the range of 0.0 (skin friction piles) to 1.0 (point bearing piles). Furthermore, skin friction can be distributed along the side of the pile either in a triangular or a uniform fashion.

To determine the affect of skin friction distribution, four curves of static soil resistance versus dyanmic driving resistance were developed for various ratios of RUP/RUT, assuming a uniform distribution of skin friction in one case and a triangular distribution in the other. The resulting curves are shown in Fig. 4. From these curves it is apparent that the distribution of skin friction has a minor effect on the overall solution.

Using Smith's parameters and assuming a uniform soil resistance distribution, the solid curves shown in Fig. 5 were obtained by assuming various RUP/RUT ratios. The dashed curves were obtained by using the modified soil parameters and varying RUP/RUT while all other parameters were held constant. These curves illustrate the remarkable influence which the ratio of RUP to RUT has on the solution, regardless of which soil parameters are used. The error in the predicted pile capacity obtained by using the modified soil parameters is not as sensitive to the RUP/RUT ratio for friction piles in clay. This can be observed from Fig. 5 by noting that for a blow count of 200 and an increase in RUP/RUT from 5% to 15% the static soil resistance increases from 130 to 138 tons. However, as the RUP/RUT ratio increases, the corresponding change in

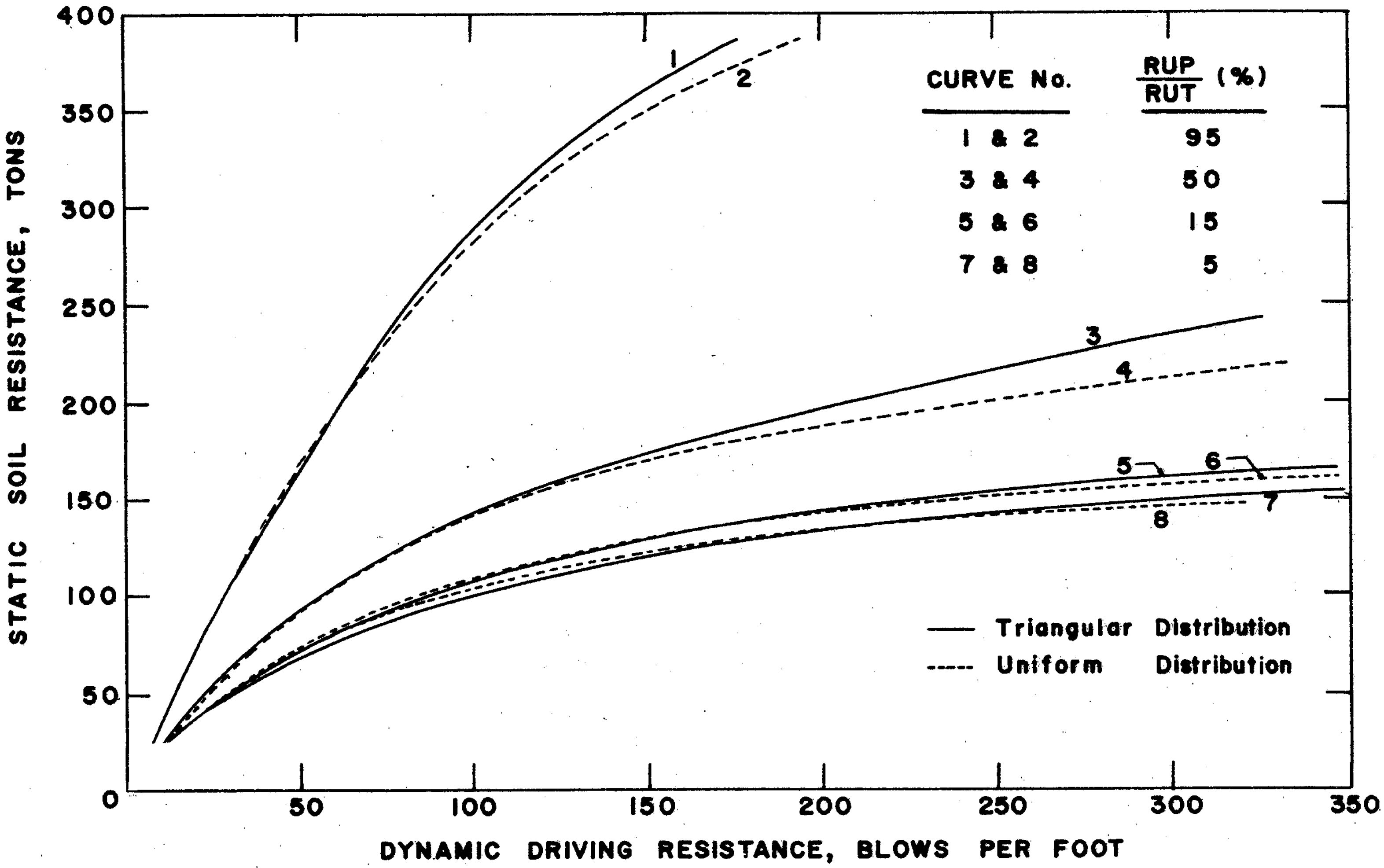


FIGURE 4 - EFFECT OF ASSUMING UNIFORM AND TRIANGULAR DISTRIBUTIONS OF STATIC SOIL RESISTANCE

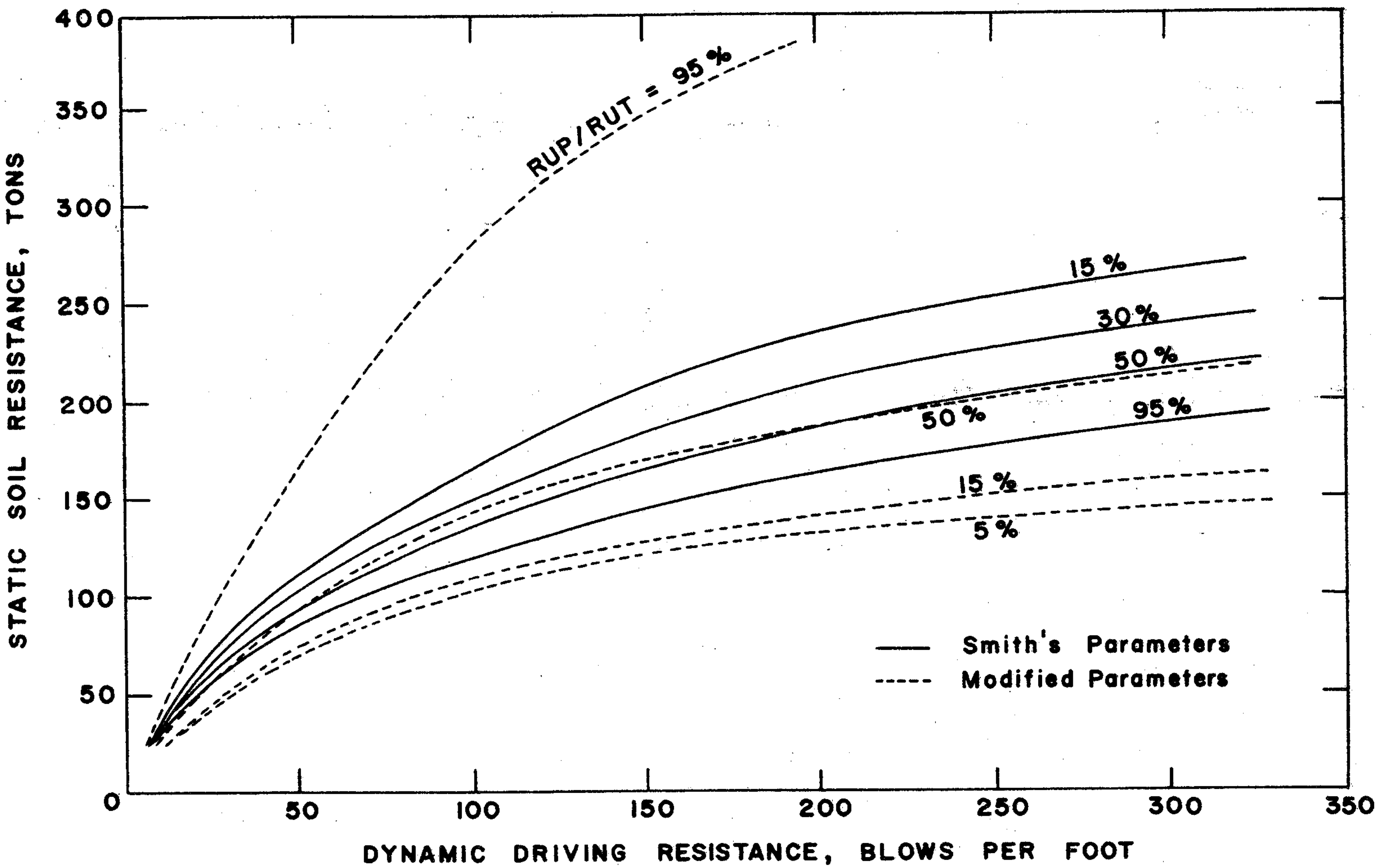


FIGURE 5 - EFFECT OF VARYING RUP/RUT USING SMITH'S SOIL PARAMETERS AND THE MODIFIED SOIL PARAMETERS

RUT becomes much more prominent. For a blow count of 200, and an increase of RUP/RUT from 50% to 95%, RUT increases from 186 to 386 tons. This illustrates the necessity of making a reasonably accurate estimate of the RUP/RUT ratio for the purpose of predicting bearing capacity by a wave equation analysis.

An important difference caused by using Smith's parameters in one case and the modified parameters in the other is readily apparent in Fig. 5. Using Smith's parameters the curves move down and to the right as RUP/RUT increases; the pattern is reversed when the modified parameters are used. A possible explanation for this apparent contradiction can be obtained by considering the two different equations used to compute the dynamic soil resistance which is mobilized during driving. Fig. 6 shows the ratio of dynamic soil resistance to static soil resistance plotted as a function of velocity. The lower curve was obtained by using Smith's equation for dynamic soil resistance [Eq. (1)]. The modified equation [Eq. (2)] was used to obtain the upper curve. To illustrate, consider a horizontal line of constant RUT in Fig. 5. For a small RUP/RUT ratio, the skin friction resistance is dominant and Eq. (2) with $N = 0.35$ yields a much greater dynamic resistance relative to Eq. (1). Hence, the blow count is much higher when the modified parameters are used. As RUP/RUT increases, the skin friction becomes less dominant and the difference between the two dynamic resistances computed by Eq. (1) and Eq. (2) decreases. Thus, the curves tend to move in opposition

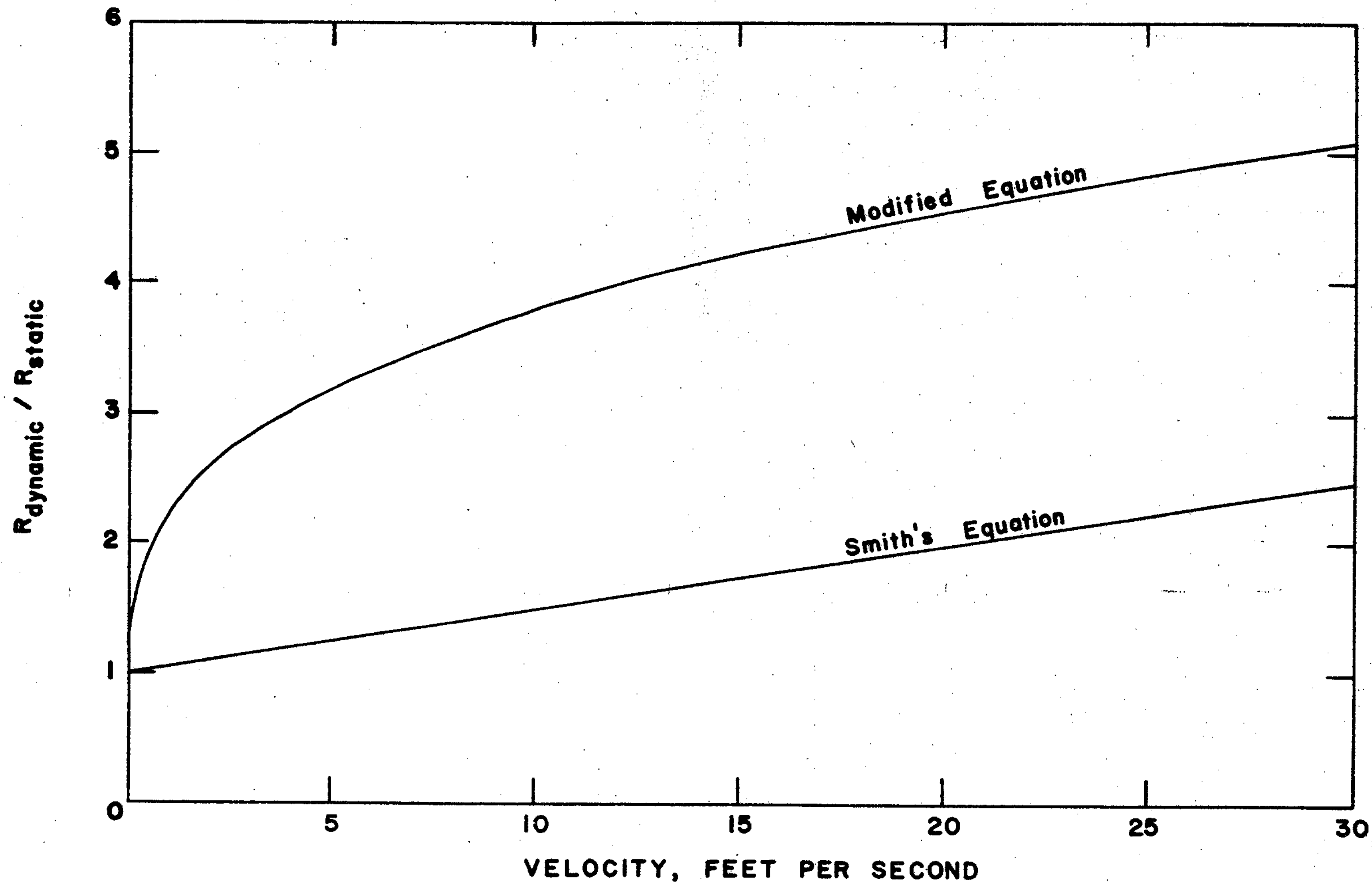


FIGURE 6 – RATIO OF DYNAMIC SOIL RESISTANCE TO STATIC SOIL RESISTANCE VERSUS VELOCITY

to each other. The full ramification of this trend will not be clearly understood until more field data from instrumented piles are available to verify the wave equation analyses.

To determine the affect of Q on the static soil resistance versus dynamic driving resistance curves, the modified soil parameters were used and Q was varied from 0.01 in. to 0.10 in. The curves which were obtained are shown in Fig. 7. These curves indicate that the affect of Q is not as great as the affect of RUP/RUT in determining the required relationship. As noted previously, the data obtained by Korb from model pile tests in clay yielded a value of 0.03 in. for Q. Coyle (3) has developed curves which relate load transfer and skin friction to pile movement for friction piles in clay. These curves indicate that skin friction reaches a limiting value for pile movements on the order of 0.08 in. Because of the fact that Coyle's work involved full scale piles, a Q of 0.08 in. would appear to be more appropriate for practical applications. However, referring to the curves in Fig. 7, the error involved by assuming Q = 0.08 in. and Q = 0.10 in. is less than 5%. This error is considered to be negligible and therefore the use of Q = 0.10 in. as suggested by Smith is recommended for routine wave equation analyses.

The next step of the parameter study was to determine what affect the point damping parameter J has on the relationship between static soil resistance and dynamic driving resistance. Three curves were developed using the modified parameters with

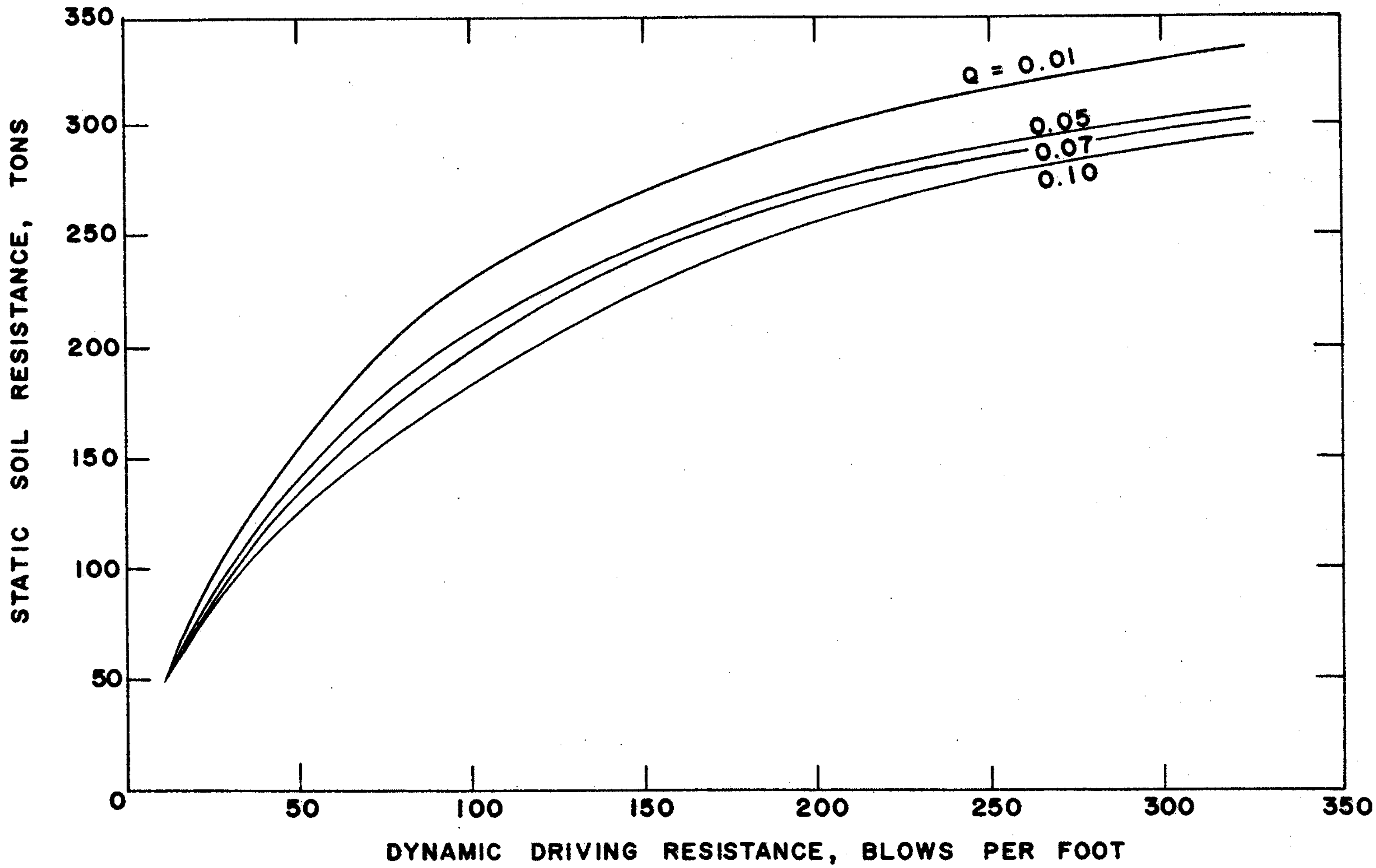


FIGURE 7 - EFFECT OF VARYING SOIL QUAKE USING THE MODIFIED SOIL PARAMETERS

RUP/RUT = 15%, and J was varied from 0.05 to 0.25 seconds per foot. The resultant curves are shown in Fig. 8. From these curves it is apparent that the point damping parameter J does not have as much influence on the driving characteristics of the pile as the RUP/RUT ratio. One of Korb's conclusions (6) was that "The tip damping constant (J) as determined from field test data was relatively constant in the fine grained soils tested. The average value of J was 0.18 seconds per foot." Based on this evidence, the use of J = 0.15 seconds per foot as suggested by Smith is recommended.

The final phase of the parameter study was an investigation of the friction damping parameter J'. This was accomplished by developing curves using the modified soil parameters and varying J' from 0.4 to 1.6 seconds per foot in increments of 0.4. It is evident from the curves of Fig. 9 that J' has a significant effect. As an example, consider the effect on RUT when J' changes from 0.8 to 1.2 seconds per foot if the blow count is 25 blows per foot. From Fig. 9, the corresponding static soil resistance decreases from 52 to 43 tons, a reduction of 17%. For higher blow counts, the change in static soil resistance is even more pronounced. Therefore, this final phase of the parameter study of the Beaumont pile illustrates the importance of determining accurate values of J' for predicting bearing capacities by the wave equation analysis.

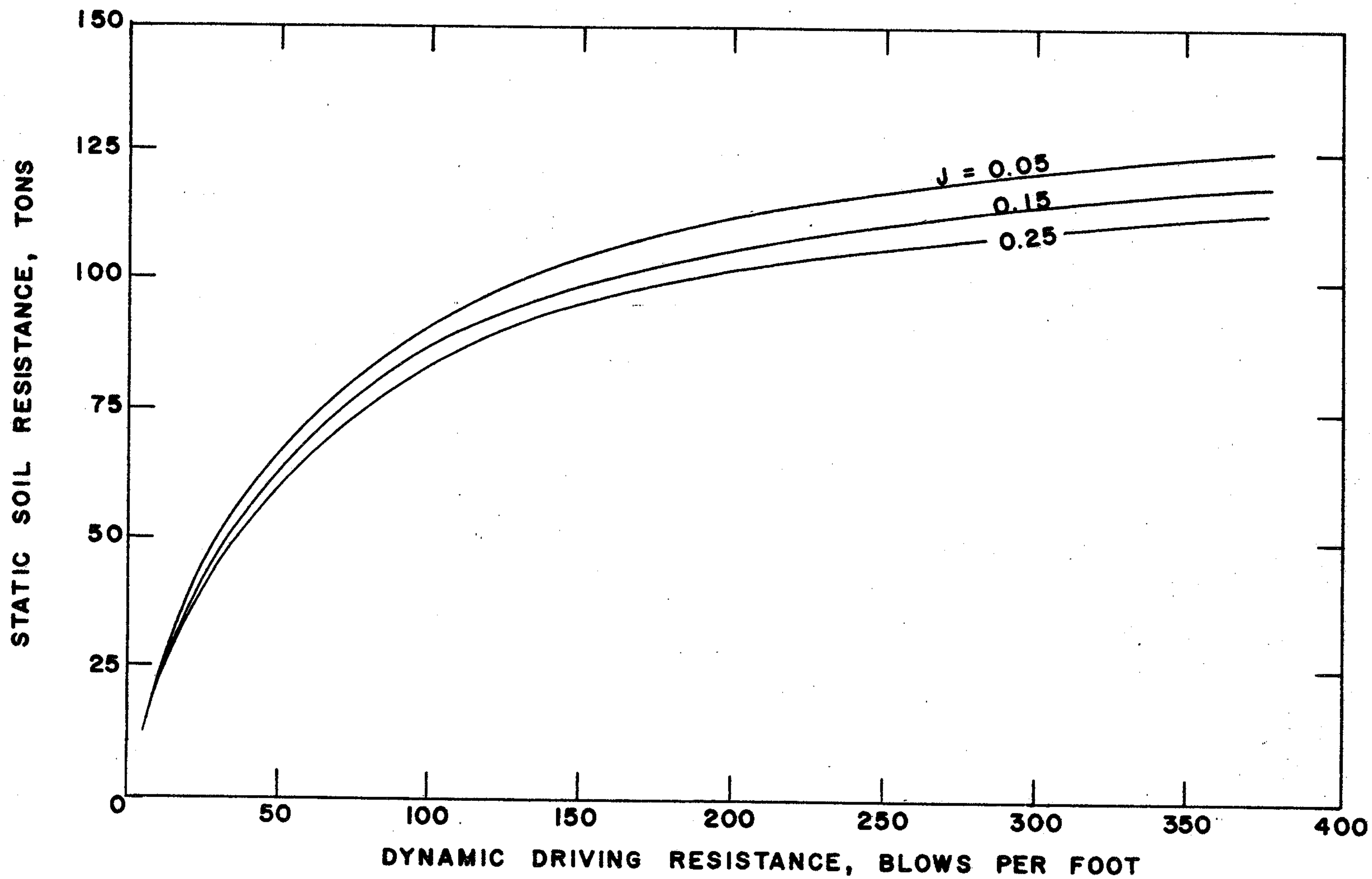


FIGURE 8 - EFFECT OF VARYING J USING THE MODIFIED SOIL PARAMETERS

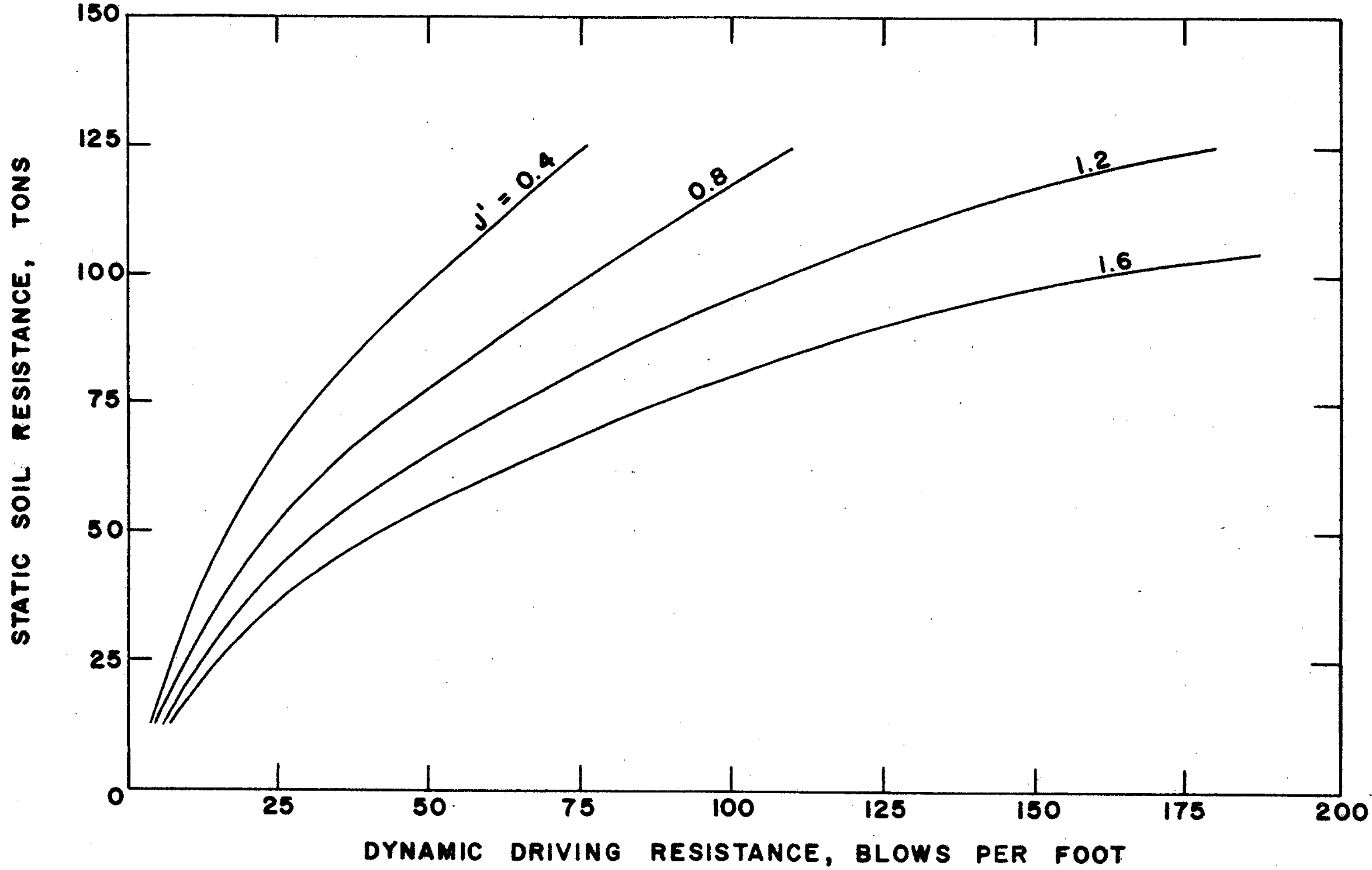


FIGURE 9 - EFFECT OF VARYING J' USING THE MODIFIED SOIL PARAMETERS

WAVE EQUATION ANALYSIS OF PORT ARTHUR FIELD TEST DATA

General

The parameter study which was made using field test data from the Beaumont pile test program has shown that the ratio of point load to total load, RUP/RUT, is a prominent factor in developing the relationship between static soil resistance and dynamic driving resistance. It was noted, however, that the ratio is not as critical for friction piles in clay as it is for point bearing piles. Furthermore, it was shown that a small variation of the soil parameter Q does not cause a large variation in the predicted value of static soil resistance. This was also found to be true for small variations of the point damping parameter J. In contrast, it was shown that the friction damping parameter J' is an extremely critical factor. A small variation of J' will cause a significant change in the relationship between static soil resistance and dynamic driving resistance.

The ratio of point load to total load can be obtained directly, provided that a test pile has been properly instrumented with electric resistance strain gages. At the present time, it is not possible to make direct measurements for the evaluation of J'. Using data obtained from pile tests conducted at Port Arthur, Texas, it will be shown that the wave equation can be used to evaluate J' by an indirect method.

Ratio of Point Load to Total Load

To accomplish the first objective of this investigation, two piles were driven and load tested in the vicinity of Port Arthur, Texas, during November, 1969. Both piles were 16-in. OD, $\frac{3}{8}$-in. wall thickness, steel pipe piles. The length of test pile No. 1 was 67 ft and it was driven to an embedded depth of 64 ft. Test pile No. 2 had a length of 78 ft and it was driven to an embedded depth of 74 ft. Both piles were driven by a Link-Belt 520 diesel hammer. A complete description of the soil properties, pile instrumentation, data recording equipment, and detailed static load test data can be found in the report by Perdue (10). Both piles were load tested approximately $1\frac{1}{2}$ hours after driving and again just prior to redriving 11 days later. Strain gages at the head and tip of the piles made it possible to evaluate RUP/RUT for each pile. The ultimate loads and corresponding RUP/RUT ratios, obtained by static load testing the piles, have been tabulated in Table 2. To illustrate the calculations, consider the data obtained for test pile No. 1. The static loads required at the head of the pile to cause a plunging failure were 46.2 tons $1\frac{1}{2}$ hours after driving and 100 tons eleven days later. The corresponding point bearing loads at the tip of the pile were 9 tons and 5 tons, respectively. Consequently, the RUP/RUT ratios were $9/46.2 = 0.195$ at the time of driving and $5/100 = 0.05$ eleven

TABLE 2.--SUMMARY OF STATIC LOAD TEST DATA OBTAINED FROM PORT ARTHUR PILE TESTS

Test pile No.	Capacity by static load test, RUT, tons		Point-bearing resistance, RUP, tons		Soil set-up	$\frac{RUP}{RUT}$	
	Immediate Test	Eleven-day Test	Immediate Test	Eleven-day Test		Immediate Test	Eleven-day Test
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	46.2	100.0	9	5	2.16	0.195	0.050
2	50.1	122.0	8	10	2.43	0.160	0.082

days after driving.

Soil Set-up Factors

The static load test results also make it possible to evaluate the amount of soil set-up which has occured. The soil set-up factors for each pile are shown in Table 2. Again taking test pile No. 1 as an example, the static bearing capacity of the pile increased from 46.2 tons $1\frac{1}{2}$ hours after driving to 100 tons eleven days later. Thus, the soil set-up factor for that particular time interval was 100/46.2 = 2.16. If the pile had been load tested again at a later date, e. g., 30 days after driving, the bearing capacity of the pile probably would have exceeded 100 tons. Hence, the soil set-up factor after 30 days may have exceeded the eleven-day factor of 2.16.

Prediction of Pile Capacities

Using Smith's soil parameters and a uniform distribution of soil resistance, the curves shown in Fig. 10 were obtained. Driving records taken during the last few feet of driving on the date of installation show that the dynamic driving resistance (blow count) for test pile No. 1 was $14\frac{1}{2}$ blows per foot, and 16 blows per foot for test pile No. 2. Using the procedure outlined previously, the predicted bearing capacity at the time of driving was 60 tons and 72.5 tons (from Fig. 10) for test piles 1 and 2 respectively. Applying the set-up factors shown in Table 2, the

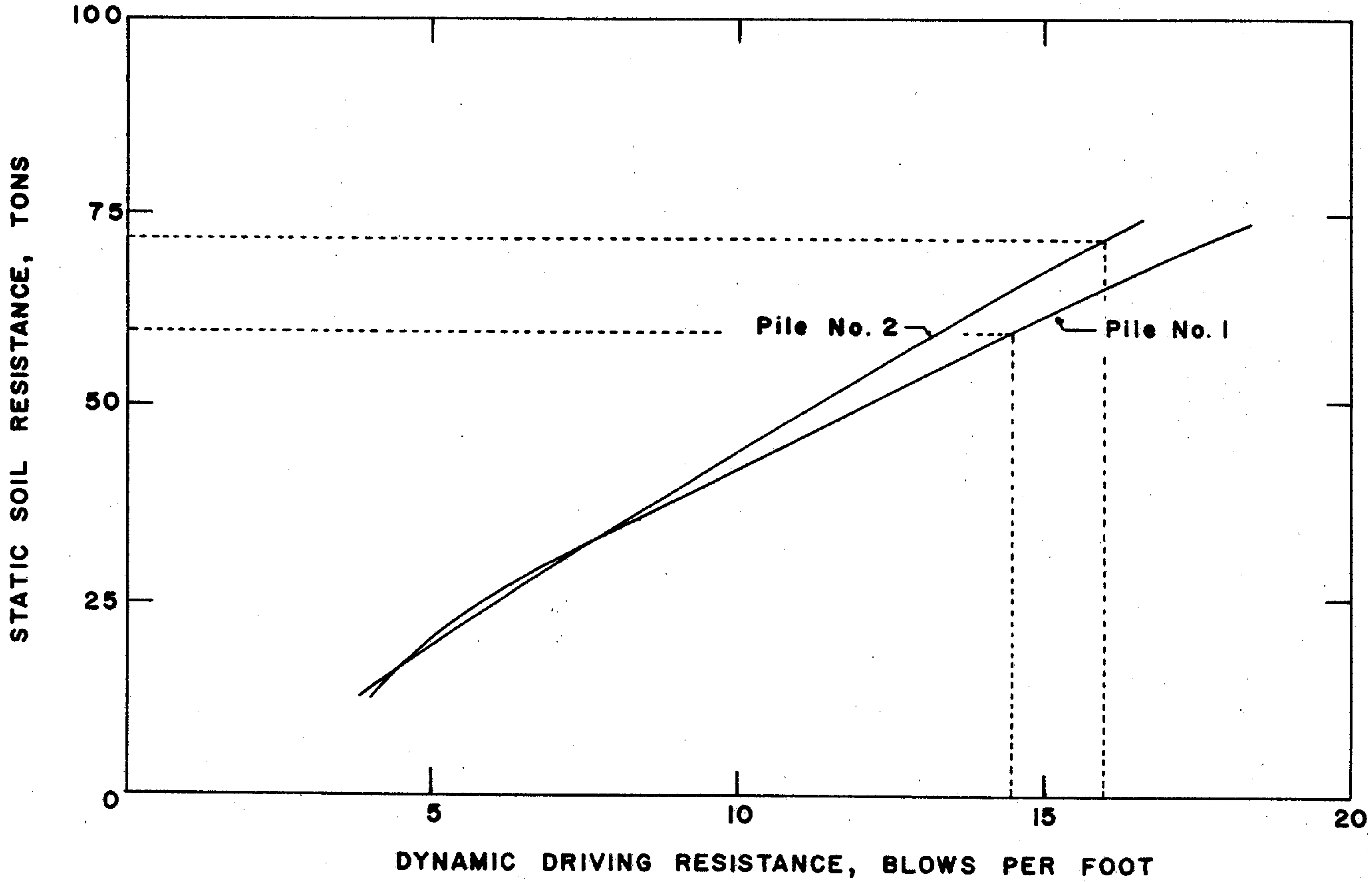


FIGURE 10 – SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR PORT ARTHUR PILES USING SMITH'S PARAMETERS

ultimate predicted pile capacities are 2.16 x 60 = 130 tons for test pile No. 1 and 2.43 x 72.5 = 176 tons for test pile No. 2. A comparison of these values with the actual capacities determined by the load tests after eleven days shows that Smith's soil parameters result in predicted pile capacities which are too large. The predicted capacity exceeds the measured capacity by 30% for test pile No. 1, and 44% for test pile No. 2.

The static soil resistance versus dynamic driving resistance curves obtained by using the modified soil parameters given in Table 1 are shown in Fig. 11. The pile capacities obtained after allowing for set-up are 2.16 x 31.5 = 68 tons for test pile No. 1, and 2.43 x 36 = 87.5 tons for test pile No. 2. A comparison of these values with the eleven-day static load test capacities shows that, for these particular piles and soil conditions, the modified soil parameters yield conservative results. For test pile No. 1 the error in the predicted value is - 32%. For test pile No. 2 the error is - 28%.

In addition to recording the blow count for each foot of pile penetration, the time-dependent dynamic forces applied on the top of each pile were recorded for the last several blows of the hammer. This was accomplished by inserting a 16 in. diameter, 3-ft high load cell between the pile and the hammer. The dynamic strains during driving were recorded by a Honeywell 1508 Visicorder. The strains were translated to forces which can be used as input data for the wave equation computer program. The use of

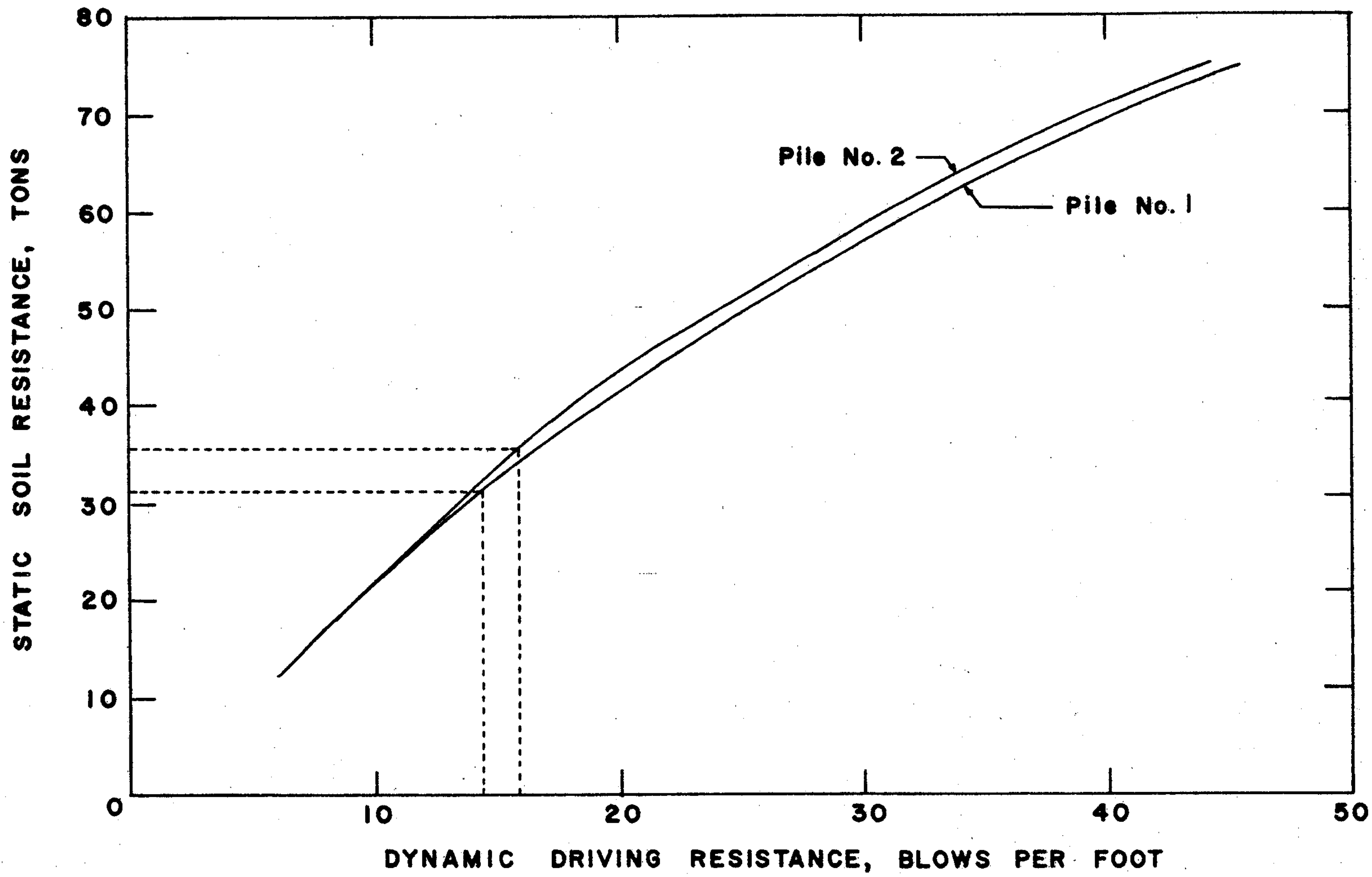


FIGURE 11 – SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR PORT ARTHUR PILES USING MODIFIED PARAMETERS

a force versus time input eliminates the necessity of determining certain variables which are used to simulate the pile driving hammer and accessories. Specifically, these variables include the ram velocity at impact (or alternatively, the height of fall of the ram and the operating efficiency of the hammer), and the coefficients of restitution and dynamic stiffnesses of the cushions. A tentative study was made using the time-dependent forces recorded during the course of the Port Arthur tests. However, the results were not conclusive and additional work is necessary before any specific conclusions can be made.

Determination of Friction Damping

The large discrepancies between the measured and predicted pile capacities suggested that a value be determined for J' which would reduce the error to zero. This was accomplished by setting RUT equal to the static capacity of the pile at the time of driving. J' was then varied from 0.1 to 1.25 seconds per foot to obtain a curve of J' versus dynamic driving resistance. The curves obtained for the two piles are shown in Fig. 12. Thus, knowing the actual blow count at the time of driving, a value of J' can be obtained which will cause agreement between the measured and predicted capacities. The values obtained from Fig. 12 were J' = 0.535 seconds per foot for test pile No. 1 and J' = 0.67 seconds per foot for test pile No. 2. These values were then used to develop the curves which are shown in Fig. 13. By

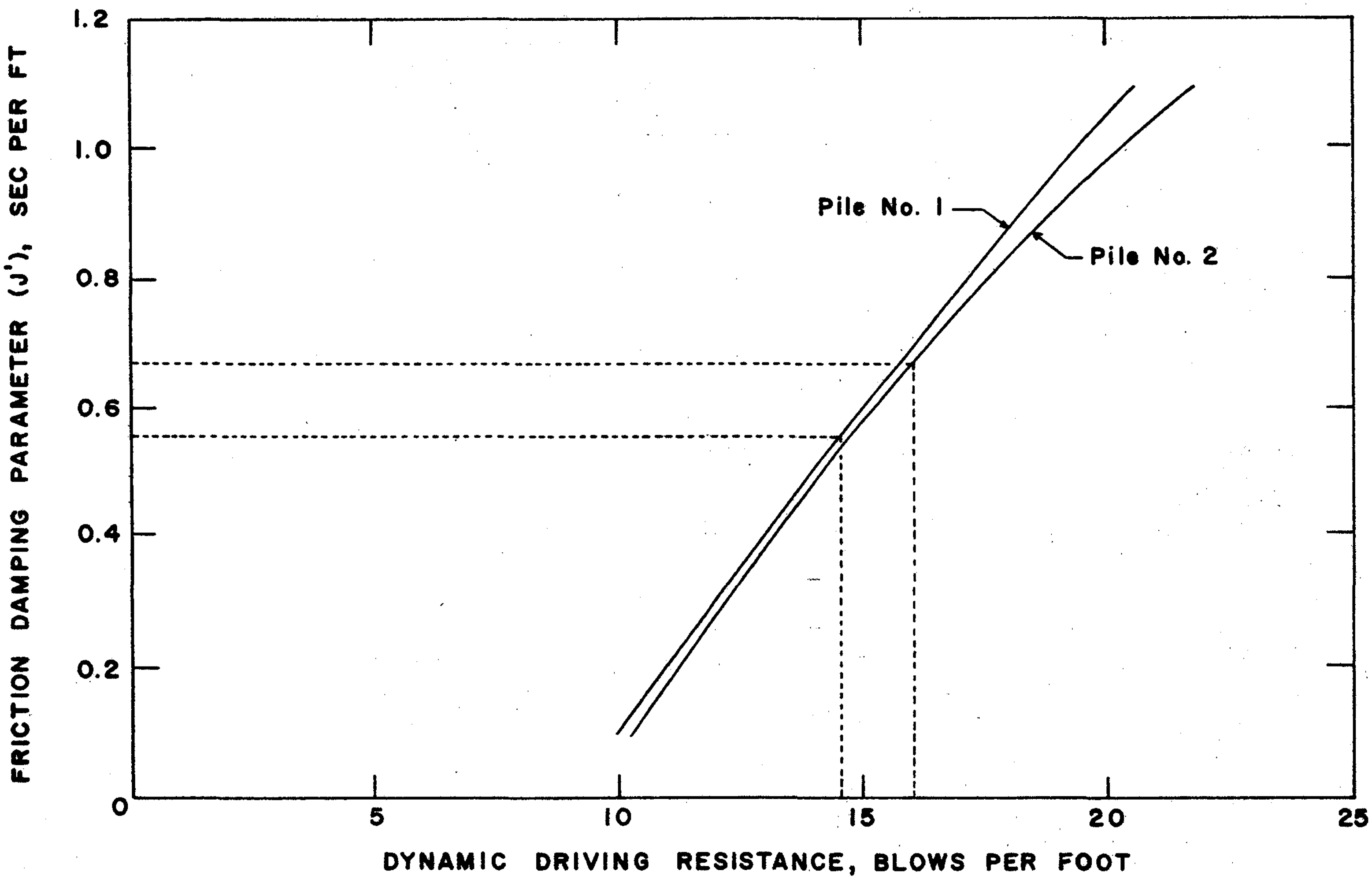


FIGURE 12 - FRICTION DAMPING PARAMETER VERSUS DRIVING RESISTANCE FOR PORT ARTHUR PILES

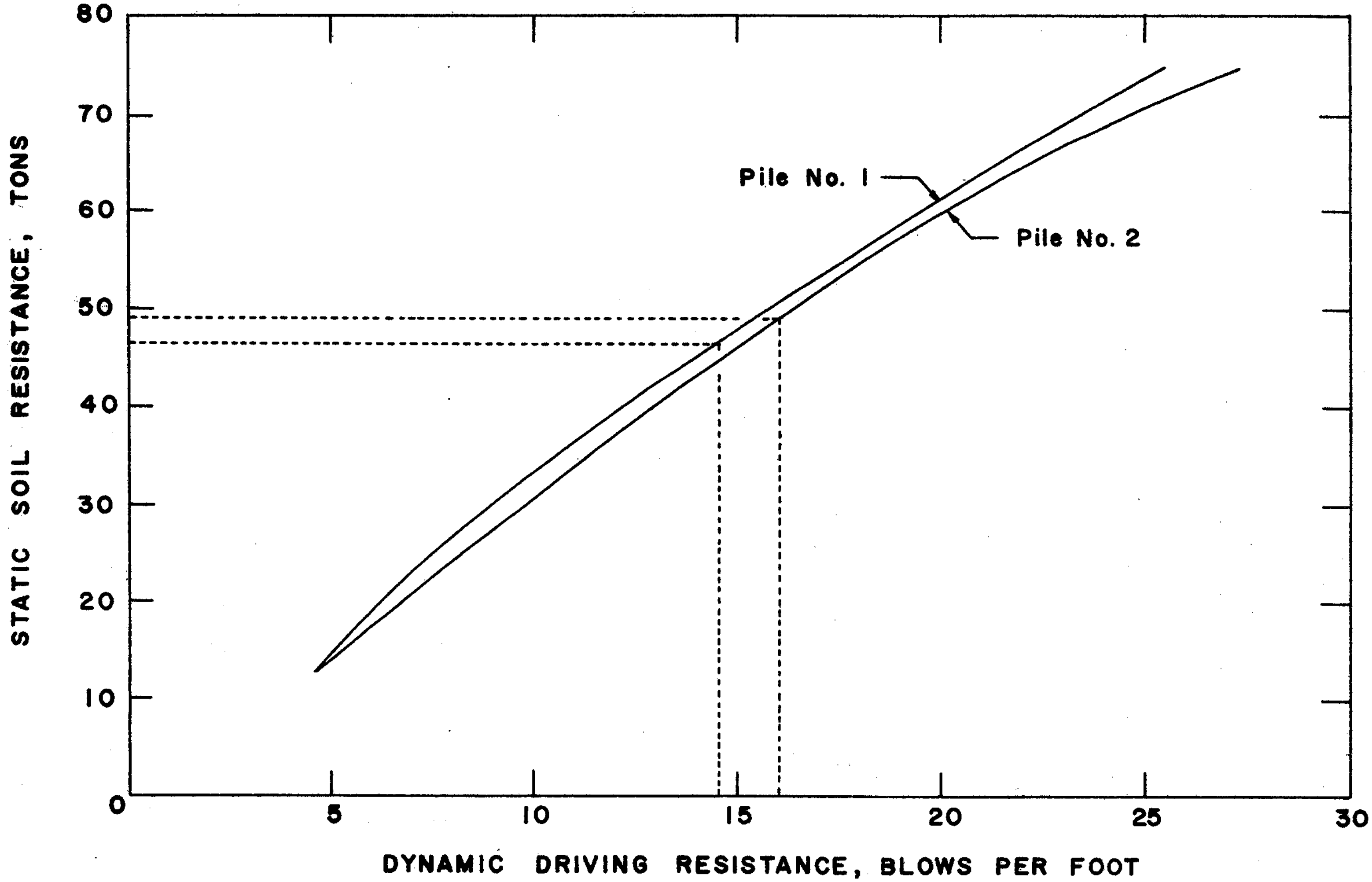


FIGURE 13 - SOIL RESISTANCE VERSUS DRIVING RESISTANCE USING J' REQUIRED FOR EXACT AGREEMENT

entering these curves with the appropriate blow count it is shown that the static soil resistance thus obtained is indeed the same as that which was determined by the static load tests. To illustrate, entering the curve for test pile No. 1 with a blow count of 14.5 blows per foot yields the correct value of 46 tons for the static soil resistance.

Estimating Soil Set-up

Any of the so-called "dynamic pile driving formulas" can, at best, only be used to predict the static bearing capacity at the time of driving. To this end the wave equation is no exception. The magnitude of soil set-up which has occurred after a specified period of time can be determined by a static load test. From a practical point of view this method is often undesirable because of the time and expense required to conduct this test. If, however, the pile can be redriven for approximately three or four feet after sufficient time has been allowed for soil set-up to occur, it is plausible that the driving record thus obtained can be used to predict the ultimate bearing capacity of the pile directly from the wave equation analysis. Two advantages of this method are that it can reduce the amount of time required to test the pile and it eliminates the necessity of multiplying the static bearing capacity at the time of driving by a soil set-up factor to obtain the ultimate static bearing capacity.

With this concept in mind, the Port Arthur test piles were redriven upon completion of static load testing eleven days after the piles were initially installed. Curves relating static soil resistance to dynamic driving resistance were developed for the eleven day test. These curves are shown in Figs. 14 and 15. They differ from the corresponding curves shown in Figs. 10, 11 and 13 because RUP/RUT was different at the time the piles were redriven. If it is assumed that the soil set-up had completely developed by the time the piles were redriven, and no further set-up will occur in the future, the predicted static capacity of the piles obtained from Fig. 14 and 15 would represent the ultimate static capacity of the pile. In reality, however, such an assumption would generally be invalid. Depending on the exact nature of the soil involved, set-up may continue for months or years. Consequently, any predicted capacity obtained prior to the cessation of set-up will be conservative.

The predicted eleven-day capacities of the Port Arthur piles are shown in Column (5) of Table 3. Column (6) of the table shows the soil set-up factors calculated using the predicted capacities at the time of driving and the predicted eleven-day capacities. The results show a significant error in every case. However, it is believed that after soil parameters have been developed which will yield capacities within ± 10% accuracy the concept of determining soil set-up and ultimate capacity by redriving a pile will become a valuable tool for the design of pile foundations.

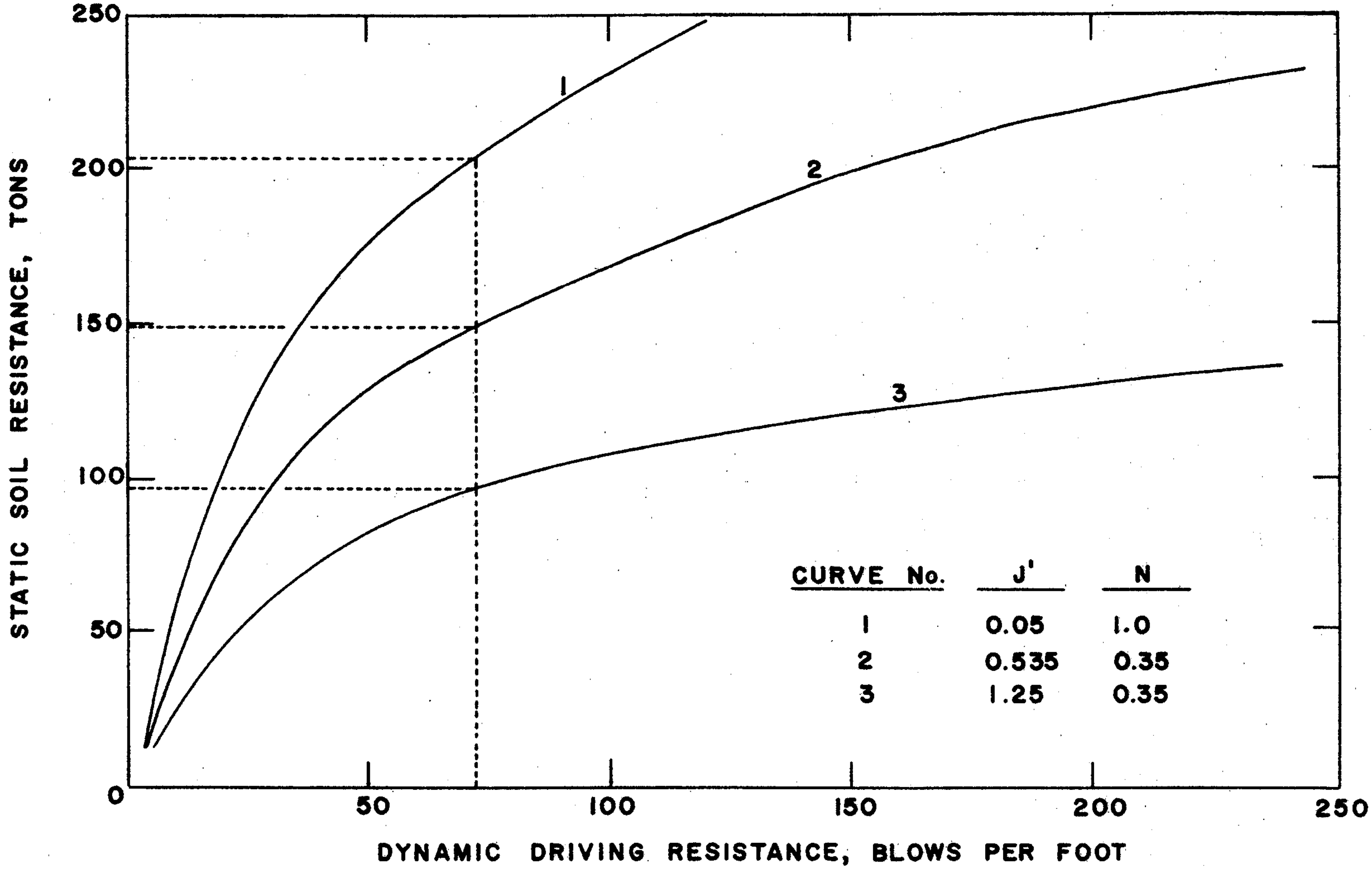


FIGURE 14 – SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR PORT ARTHUR PILE NO. I ELEVEN DAYS AFTER DRIVING

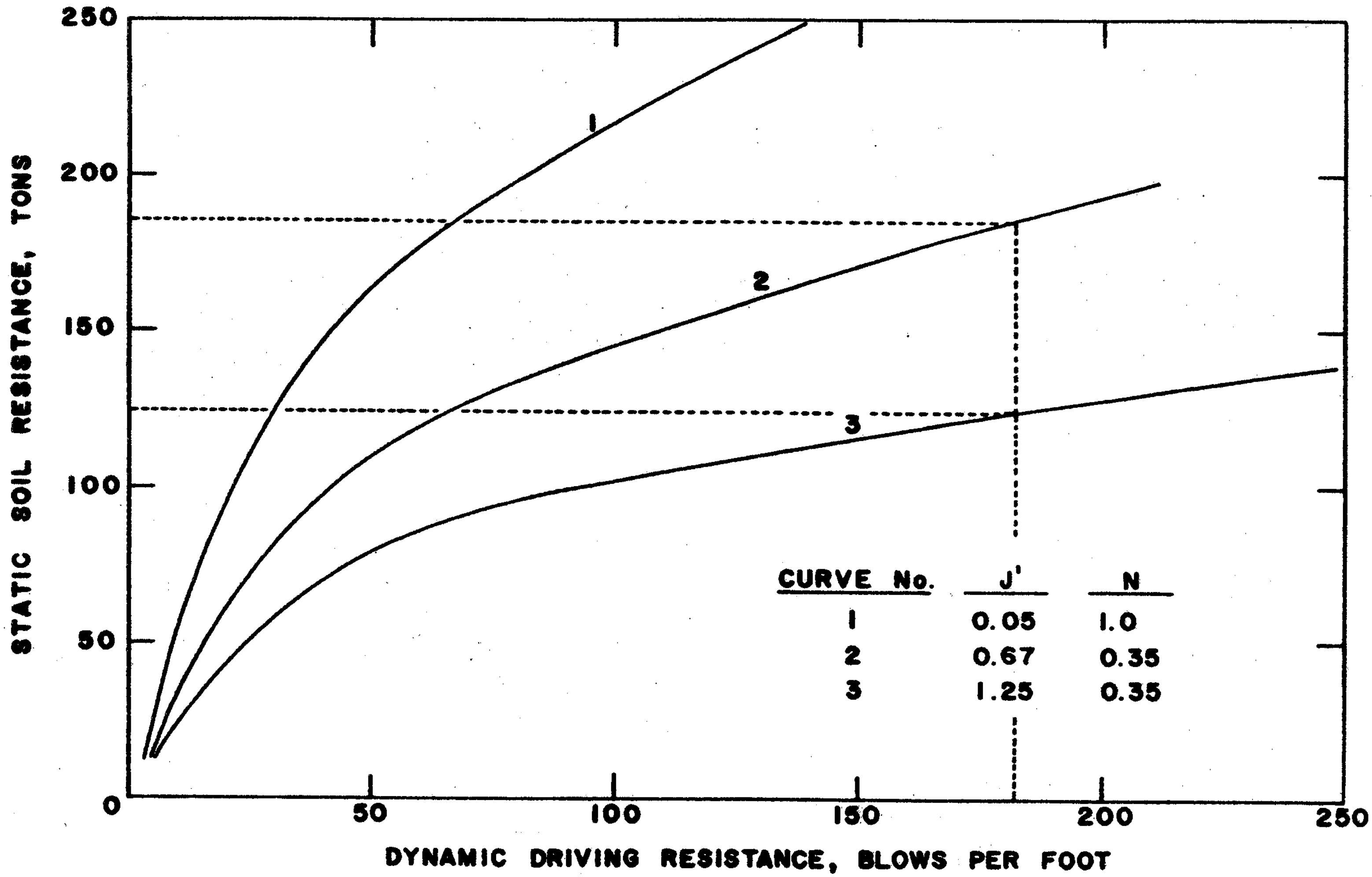


FIGURE 15 – SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR PORT ARTHUR PILE NO. 2 ELEVEN DAYS AFTER DRIVING

TABLE 3.--ESTIMATED SOIL SET-UP FACTORS

Pile No.	Actual set-up by load test	Predicted capacity from wave equation analysis			Estimated soil set-up factor, $\frac{\text{Col. (5)}}{\text{Col. (4)}}$
		J' seconds per foot	Immediate test, tons	Eleven-day test, tons	
(1)	(2)	(3)	(4)	(5)	(6)
1	2.16	0.05	60	202.5	3.38
1	2.16	1.25	31.5	96	3.04
1	2.16	0.535	46.2	149	3.22
2	2.43	0.05	72.5	285	3.93
2	2.43	1.25	36	124	3.44
2	2.43	0.670	49.5	184	3.72

ADDITIONAL CASE STUDIES

General

The wave equation analyses of the Port Arthur test data have shown that neither Smith's parameters nor the modified soil parameters were satisfactory for predicting the ultimate bearing capacity of the piles. A study of the Port Arthur data was made to determine what values of the friction damping parameter J' would yield an exact agreement between the predicted and measured pile capacities. No completely general conclusions could be made, regardless of the results, on the basis of this limited amount of data. Consequently, two other piles were analyzed with the intention of finding any trends, with respect to the friction damping parameter, which might prove noteworthy.

Beaumont Test Pile

The Beaumont pile test data which were used for the parameter study were again utilized to further investigate the friction damping parameter. The J' value required to yield an exact agreement between the predicted and the measured static load capacity of the pile was determined in a manner identical to that used for the Port Arthur data. The curve relating the friction damping parameter to dynamic driving resistance for the Beaumont pile is shown in Fig. 16. The blow count just prior to the termination

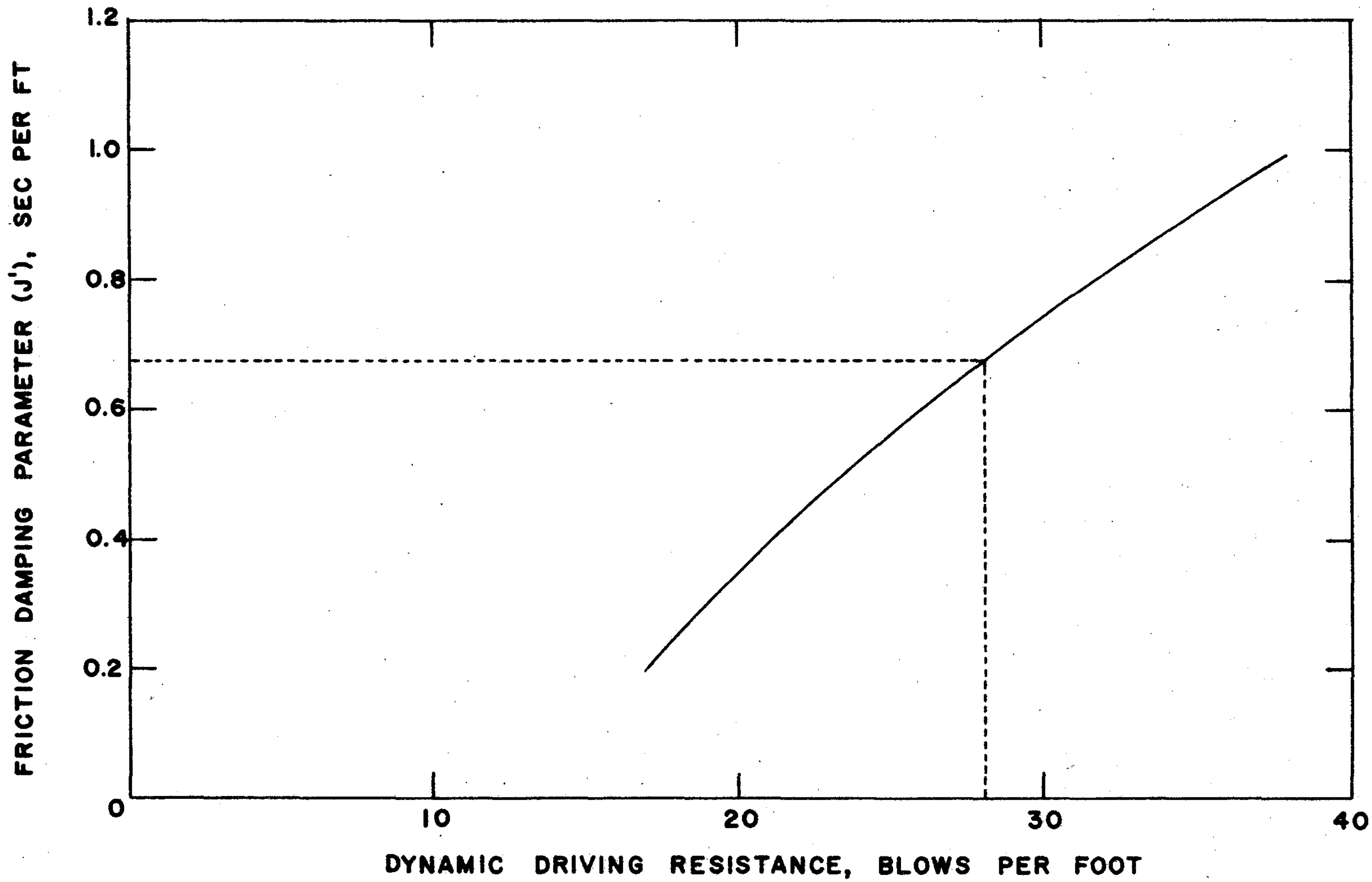


FIGURE 16 – FRICTION DAMPING PARAMETER VERSUS DRIVING RESISTANCE FOR BEAUMONT TEST PILE

of driving was 28 blows per foot. Fig. 16 shows that the required value for J' is 0.7 seconds per foot. The corresponding relationship between static soil resistance and dynamic driving resistance, obtained for J' = 0.7, is shown in Fig. 17.

Several assumptions regarding the Beaumont data were necessary in order to develop the curves shown in Figs. 16 and 17. These assumptions were: (1) the static bearing capacity of the pile on the day it was driven was 61 tons; (2) the soil set-up factor after 13 days was 2.0; and (3) RUP/RUT on the day of driving was 15%. The Beaumont pile was not load tested on the day it was driven, hence, RUP/RUT and the soil set-up factor could not be determined. The details which support these assumptions are given in the report by Airhart (1).

Belleville Test Pile No. 1

The Michigan Report (9) contains load test data which can be used to calculate the set-up factor for one of the piles driven at the Belleville test site. Belleville load test pile No. 1 was a 61.1-ft long, 12-in. OD, 0.25-in. wall thickness, steel pipe pile, driven to an embedded depth of 66.7 ft by a Delmag D-12 hammer. Maximum static test loads applied on the pile were 55 tons 4 hours after driving, and 105 tons approximately 51 days after driving. Thus, the set-up factor was 105/55 = 1.91. Data required to evaluate RUP/RUT were not reported, consequently a ratio of 6% was determined from the soil shear strength data presented in the

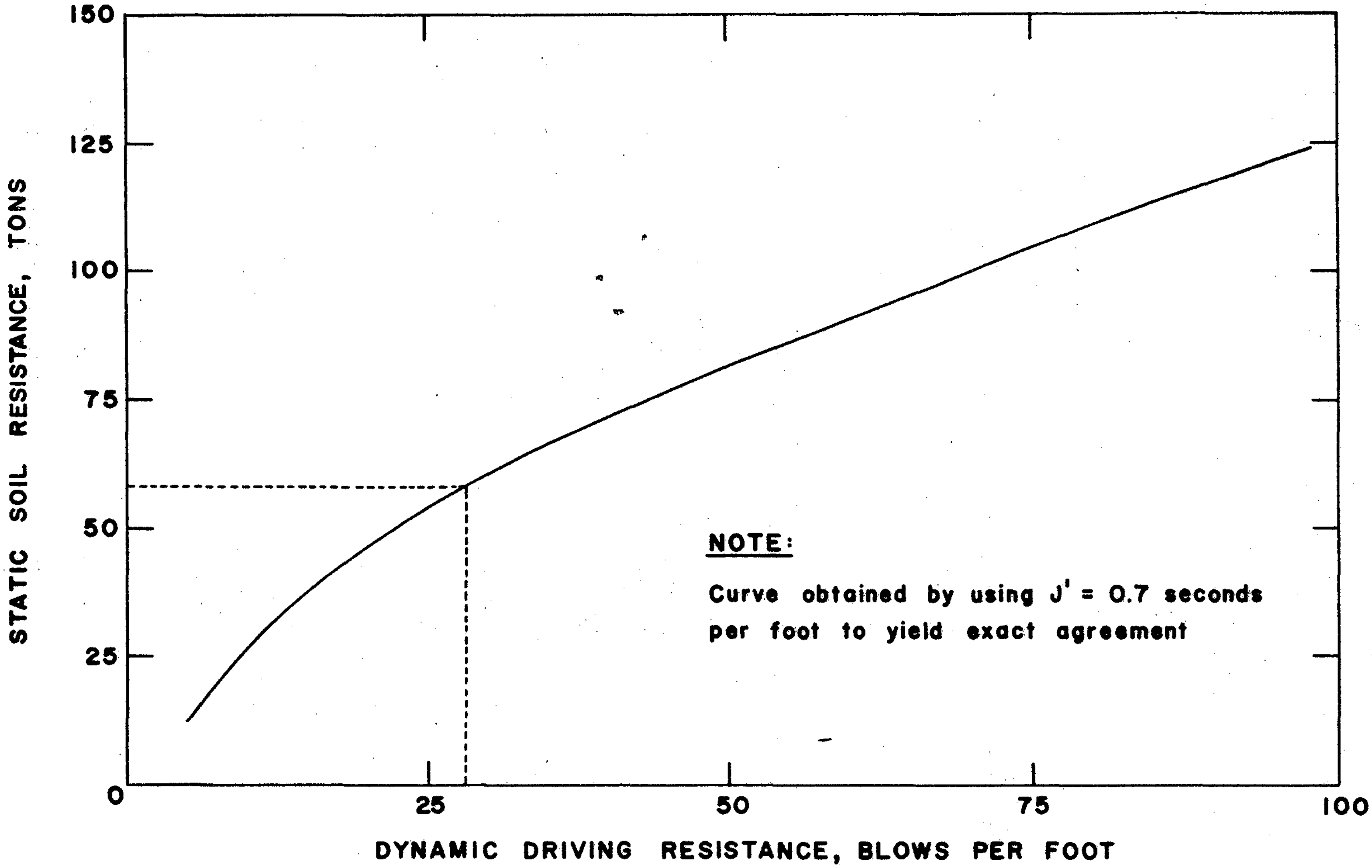


FIGURE 17 – SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR BEAUMONT TEST PILE

report. The blow count during the last foot of driving was 132 blows per foot. The curve relating the friction damping parameter J' to the dynamic driving resistance is shown in Fig. 18. For a blow count of 132 blows per foot, the J' value required to obtain an exact agreement between the predicted and measured static bearing capacity on the day of driving is 1.6 seconds per foot. The corresponding relationship between static soil resistance and dynamic driving resistance, obtained for J' = 1.6, is shown in Fig. 19.

Friction Damping Related to Plasticity Index

It has been shown that the friction damping parameter J' is indeed not a constant for all clay soils. For each pile which was analyzed by the wave equation, a different value for J' was required to achieve agreement between measured and computed pile capacities. As a consequence of this fact, it would obviously be advantageous to isolate some soil property which might be used as a guide in selecting a J' value for a given clay soil. One hypothesis was that the friction damping characteristics of the soil can be estimated on the basis of the plasticity index of the soil. The hypothesis was tested by calculating the average plasticity index of the soil for each pile test location described in this investigation. The indices were then compared to the friction damping parameters required to obtain an exact agreement between measured and predicted pile capacities.

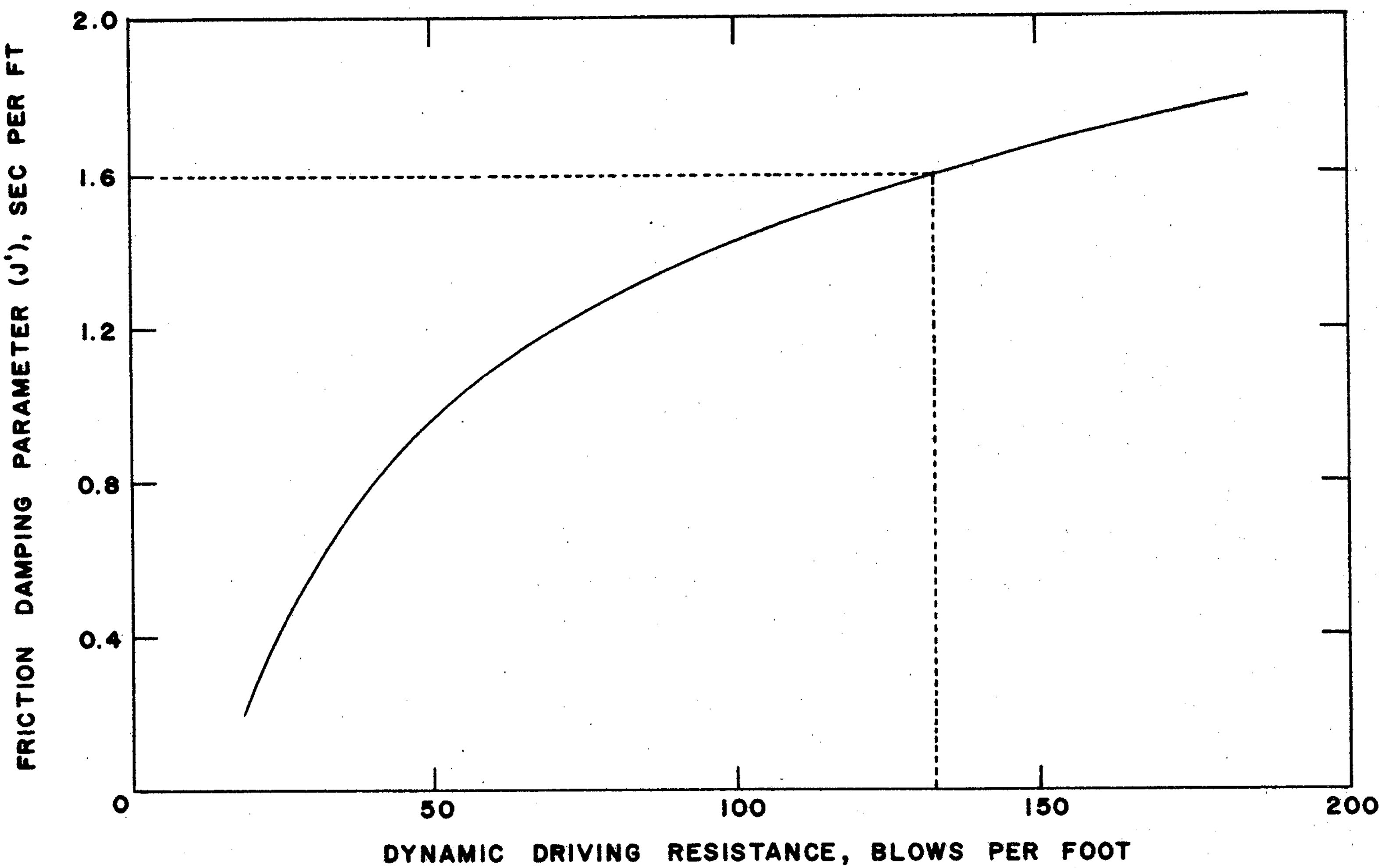


FIGURE 18 – FRICTION DAMPING PARAMETER VERSUS DRIVING RESISTANCE FOR BELLEVILLE LOAD TEST PILE NO. 1

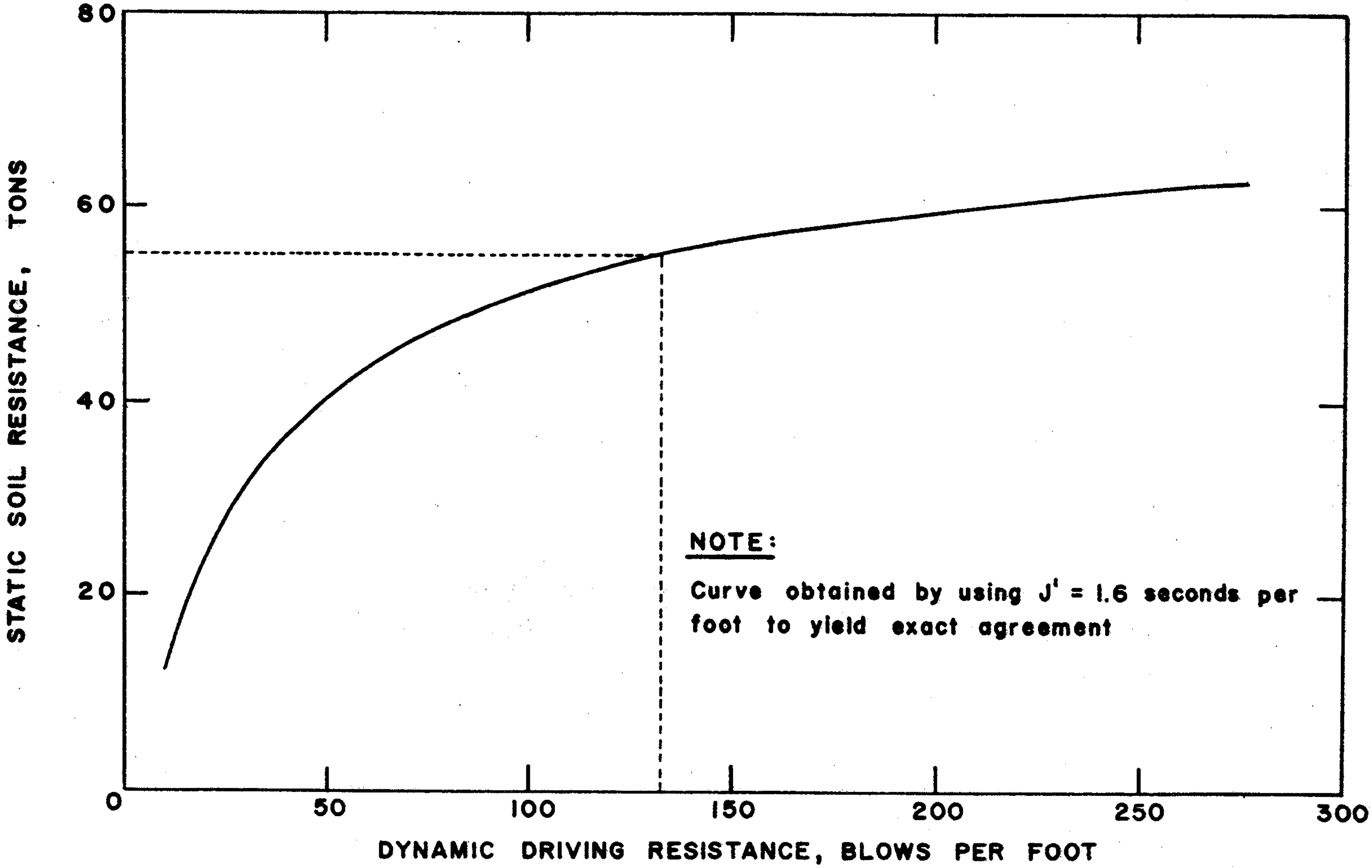


FIGURE 19 - SOIL RESISTANCE VERSUS DRIVING RESISTANCE FOR BELLEVILLE LOAD TEST PILE NO. 1

Table 4 presents a summary of this information. It should be noted that the J' values shown in Table 4 were obtained by using a value of N = 0.35 in Eq. (2) in all cases. An examination of the data given in Table 4 shows that a correlation does in fact exist. For soils which have a high plasticity index, the friction damping capacity of the soil appears to be relatively small. As the plasticity index decreases, the friction damping parameter J' apparently increases.

TABLE 4.--SUMMARY OF PLASTICITY INDEX AND FRICTION DAMPING PARAMETER J' FOR EACH TEST SITE

Pile test location (1)	Average plasticity index (2)	Friction damping parameter, J', seconds per foot (3)
Port Arthur site 1	49	0.535
Port Arthur site 2	39	0.67
Beaumont	33	0.7
Belleville	18	1.6

A possible explanation for this apparent correlation can be obtained from a consideration of the permeability of the soil. A study of typical values for the permeabilities of various soils used in association with the Unified Soil Classification System indicates that the permeability of a cohesive soil is inversely related to the plasticity index of the soil (15). Thus, for a soil which has a high plasticity index, the permeability will be small. While a pile is being driven into the ground, it is possible that a thin film of water accumulates along the pile-soil interface. Due to the low permeability of the soil, the water cannot rapidly escape and therefore it effectively serves as a lubricating agent and reduces the damping capacity of the soil. On the other hand, a soil having a low plasticity index will have a relatively high permeability. In this case, the film of water will not develop and an adhesive bond can be formed at the interface. The bond thus formed could be the cause for a higher damping effect being associated with the soil.

The comparison of plasticity index to the friction damping parameter J' and the acknowledgement of an apparent correlation should not be construed to mean that a definite correlation has been established. However, the trend does offer fertile ground for future research and it was with this idea in mind that the comparison was made.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The broad objective of this study was to determine if a given set of soil parameters, which are used to describe the dynamic characteristics of a soil, would yield reliable and accurate predictions of the static bearing capacity of full scale piles. General conclusions cannot be made on the basis of results realized during the course of this investigation because of the limited amount of field data available from instrumented full scale piles. The results are limited in scope because they were obtained from analyses of piles which are primarily friction piles driven in cohesive soils. However, specific conclusions can be made which are appropriate for the type of piles and soils which were considered. They should not be generalized to be all-inclusive and applicable to any type of pile or soil.

From the results of the parameter study that was made with the Beaumont test pile data, the following specific conclusions can be made:

1. Predicted pile capacities obtained using a uniform distribution of static soil resistance do not differ substantially from those obtained by assuming a triangular distribution. Either method will yield satisfactory results.

2. The ratio of point load to total load is a critical factor in predicting the bearing capacity of a pile. The error caused by using an inaccurate value is greater for point-bearing piles than it is for friction piles.

3. The value of the soil quake Q does not have a significant effect on the predicted bearing capacity. Smith's recommended value of Q = 0.10 in. should be used.

4. The accuracy of predicted pile bearing capacities are not significantly influenced by the point damping parameter J. (This conclusion may not be valid for point-bearing piles.) Smith's recommended value of 0.15 seconds per foot will yield acceptable accuracy for piles which are driven into clay soils.

Based on the results from the wave equation analyses of the Port Arthur, Beaumont, and Belleville data, the following conclusions can be made:

1. There is no single value for the friction damping parameter J' which can be used for all types of clay. The complex nature of clay soil, and variations in the engineering properties of any specific type of clay which are caused by environmental conditions, preclude the existence of a unique value which adequately describes the dynamic response of all clays.

2. There is an apparent relationship between the plasticity index and the friction damping characteristics of clay soils.

Recommendations

The various pile analyses reported herein are based on the driving records and load test data obtained from tests on piles driven into cohesive materials. At present there is an acute need for field test data obtained from fully instrumented piles driven into cohesionless material, both sand and silt. These field tests must be conducted in strictly cohesionless material to eliminate the effect of cohesive soils on the dynamic response, thereby allowing an independent assessment of the tip damping parameter J and the friction damping parameter J' for sands and silts.

In this investigation, only steel pipe piles have been taken into consideration. Future studies should investigate the effects of different pile materials, such as concrete and wood, on the predicted pile capacities obtained from a wave equation analysis. In addition, the effect of pile geometry should be analyzed. Particular attention should be given to the influence of these variables on the friction damping characteristics of the particular pile-soil system involved.

It has been shown that the load being carried by point bearing at the tip of the pile is extremely important to a meaningful wave equation analysis. It is therefore recommended that future pile tests include a measurement of the point-bearing load whenever possible.

Soil set-up is an important aspect of the total problem in predicting pile capacity by dynamic measurements. Future pile tests should include a static load test at a minimum of 10 days after initial driving, with two weeks or more being the preferred time interval. This should be done concurrently with a re-driving of the pile so that measured capacities after soil set-up has occurred can be correlated with the wave equation predictions. Measurements should be made for a wide variety of pile and soil types.

The concept of using a force versus time relationship, measured at the head of the pile during driving, should be pursued further. By eliminating the variables associated with pile driving hammers and accessories, a significant obstacle will be overcome which is, at the present time, detrimental to the accuracy of predicted pile capacities and the development of soil parameters for wave equation analyses.

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