PILE DRIVING ANALYSIS USING THE WAVE EQUATION

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by

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Summary

The possibility of predicting the ultimate static bearing capacity of a pile from its dynamic behavior during driving is investigated by the wave equation method of pile driving analysis. The method developed by E. A. L. Smith over a period of years is followed, but its validity is first shown by proving that the mathematical model used is equivalent to the wave equation. Computer programs developed for purposes of this investigation are presented in their entirety together with detailed instructions on their use so that they may be utilized by future investigators who may not be familiar with computer work.

The soil engineering aspects of the problem are explored. The information available on dynamic soil properties from published test data is reviewed to obtain information applicable to pile driving. Using values thus deduced and the computer programs, computations of ultimate resistance are made from 24 published pile driving records and correlated with their load test results.

Although not conclusive because of the relatively small number of correlations attempted, the results are very encouraging. It appears that the wave equation method of pile driving analysis may become an accurate tool in predicting a pile's static bearing capacity from its driving record. This appears to be the case regardless of the type or size of pile and driving equipment, and indications are that predictions may be possible for piles driven in cohesive as well as granular soils. General limitations of the method are summarized.

Values deduced for point and side damping and ground quake in various soils are presented. Corresponding values of friction acting on the pile sides as a percentage of the ultimate resistance are given for the piles investigated. More research into this problem is indicated, and recommendations are made for such work.

It is concluded that even with inexact knowledge of dynamic soil properties the wave equation method of pile driving analysis appears to give good results.

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LIST OF SYMBOLS

A cross sectional area in square inches

C compression of spring in inches

D longitudinal displacement of part of rod or pile measured from its initial position in inches during time interval n

d longitudinal displacement of part of rod or pile measured from its initial position in inches during time interval n-1

d^* longitudinal displacement of part of rod or pile measured from its initial position in inches during time interval n-2

E Young's modulus of elasticity

e coefficient of restitution

J a damping constant applicable at the pile tip

J' a damping constant applicable at the pile sides

F force at any cross section in pounds, time interval n

f force at any cross section in pounds, time interval n-1

f^* force at any cross section in pounds, time interval n-2

g acceleration of gravity

K spring constant in pounds per inch for pile

K' spring constant in pounds per inch for soil

ℓ initial length of spring

ℓ' length of spring at end of time interval

Q ground quake

R resistance to driving in pounds

s set of pile in inches

T critical time interval in seconds

T_{min} minimum value of T in seconds

t	time in seconds
Δt	time interval in seconds
V	velocity of weight in feet per second in time interval n
v	velocity of weight in feet per second in time interval n-1
v^*	velocity of weight in feet per second in time interval n-2
W	weight
W_{ram}	weight of hammer or ram
W_{cap}	weight of pile cap
v_{ram}	velocity of ram before impact
v_{cap}	velocity of cap before impact
v'_{ram}	velocity of ram after impact
v'_{cap}	velocity of cap after impact
x	distance along longitudinal axis of rod
Z	net force in pounds acting on weight in time interval n
z	net force in pounds acting on weight in time interval n-1
z^*	net force in pounds acting on weight in time interval n-2
ν	Poisson's ratio
ϕ	overall "factor of safety" against instability of calculations
ϕ_{min}	individual "factor of safety" against instability of calculations
	density

Chapter I

INTRODUCTION

The possibility of relating the behavior of a pile during driving to its static load carrying capacity by the use of the wave equation method of pile driving analysis is encouraging. Many engineers today realize that the numerous dynamic pile driving formulas which have been used, and are still being used, have serious limitations and cannot be depended upon to give reliable results, but they are still used for lack of an adequate substitute. Some feel that the dynamic formula approach is not valid, and have turned instead to trying to determine the bearing capacity of a pile from purely static soil mechanics and structural considerations for the cases where the pile-soil system is required to carry only static loads. Such an approach is certainly valid, and it requires a thorough investigation of the soil conditions, together with a careful evaluation of their engineering properties, and the use of a great deal of personal judgment in its application.

It is believed that the wave equation method of analysis of the dynamic pile driving problem is an equally valid approach and may offer advantages over the static approach if it can be successfully applied. This is not to say that one approach should be used to the exclusion of another, but that they are working toward a common goal and should supplement each other. The wave equation method is not just another dynamic pile driving formula, but, rather, it is a method of analysis which is well founded mathematically. It requires

a knowledge of the soil and its properties, both static and dynamic, for its successful application. Current knowledge of soil dynamics is incomplete, but an increasing amount of work is being done in this field.

Approaching the problem from the dynamic side, the over-all consideration is to relate the dynamic behavior of the driving equipment-pile-soil system to the static behavior of the structure-pile-soil system with due consideration to changes in properties of the soil during and after driving of the piles. A part of this broader problem is to relate the dynamic resistance to driving of a single pile under the last hammer blow to its static bearing capacity, with time effects between driving and static loading minimized. It is for this more limited problem that the wave equation method of analysis is useful.

The wave equation alone will not provide a solution to the more general consideration which includes the group effect of piles, development of negative friction, consolidation of a clay layer beneath the pile tip, long-term changes in water table, deterioration of the pile due to insect attack or deleterious chemicals, set up or relaxation of the soil with time, or development of hydrostatic uplift. That it does not provide a solution to such problems may be obvious, but the statement is made here in an effort to clear up the type of misunderstandings and objections already voiced in opposition to the application of the wave equation to pile driving analysis. In short, it is not a magical formula, but simply a tool which is well founded

mathematically and is available to assist in visualizing and evaluating the dynamics of the problem.

The wave equation can be utilized to advantage for investigation of other facets of pile driving such as stresses in the pile and selection of appropriate pile driving equipment for a particular set of field conditions; but the main points of this investigation will be to consolidate the wave equation theory as applicable to pile driving, make readily available computer programs to facilitate the work of future investigators, explore the unsolved problems of interaction between soil and pile, attempt correlation between wave equation solutions and a few pile driving records and load tests, and to suggest possibilities for further research work in this general area.

Chapter II

BACKGROUND

Although use has been made of piles for at least 2,000 years, attempts to determine the bearing capacity of a pile from its driving record have been generally unsuccessful in spite of the amount of effort spent in this direction. An extensive treatment of the development and interrelatedness of dynamic formulas by Chellis (ref. 5) is included, and he shows that they may be grouped as follows:

The first dynamic formula was proposed by Major Saunders in 1851 and is perhaps the simplest and most direct. He equated the weight of the ram multiplied by the stroke, with the driving resistance multiplied by the set, and applied a factor of safety of 8.

Another group of proposed formulas contained a fixed coefficient which was supposed to compensate in some degree for factors affecting the results which were not included as terms in the formulas. The Engineering News, Wellington, Vulcan, and Bureau of Yards and Docks (not now used by Navy) formulas are of this type.

Another group of formulas attempts to account for the variables by using expressions for efficiency of applied energy by including relative weights of the pile and the hammer. The Dutch, Ritter, and Benabencq formulas are of this type.

Other similar formulas try to include the effect of variables by using both fixed coefficients and expressions for the relative weights of the pile and ram. The Eytelwein and Navy-McKay (not now used by the Navy) are of this type.

[illegible]

[illegible]

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The next group contains either all or part of a series of terms designed to represent impact losses in the driving cap, soil, and pile during driving. The Redtenbacher and Hiley formulas are among those which fall into this category.

Only a few of the formulas proposed have been mentioned. Chellis (ref. 5) lists 38, while it is understood that the Editors of Engineering News Record have on file 450 such formulas. Some of these formulas are extremely simple, while others are quite complicated. Some try to approximate the dynamic situation, while others are contrived from a purely statistical approach without reference to the numerous variables actually involved. For example, Marvin Gates has proposed (ref. 14) that for a pile driven with a Vulcan Number 1 Hammer the ultimate bearing capacity may be determined by multiplying 48 by the log of (10/set in inches). He claims, ". . .this relationship gives more consistently accurate results than the most complex dynamic formula yet advanced," in spite of the fact that it contains but two simple parameters. That this can be true for a formula which seems to ignore the basic aspects of the problem is not surprising when one considers the shortcomings and omissions of the other dynamic formulas proposed.

Some of the dynamic formulas were based on Newtonian theory of impact which has been shown by A. E. Commings (ref. 10) as inapplicable to the pile driving problem. Other formulas subtracted the same energy losses twice, as also shown by Cummings (ref. 10). The Committee on the Bearing Value of Pile Foundations of the American

Society of Civil Engineers recognized the many limitations of dynamic pile driving formulas (ref. 8). After this point of agreement, engineers seem to be divided into two general schools of thought. One group is optimistic and feels that rational use can be made of such formulas. The other group stresses the limitations of the formulas, and basically takes the position that, since none of them are reliable, you may as well save yourself work and choose a simple formula. The latter school of thought may seem rash at first reading; but it does have some merit, for it is simply recognizing that all of the dynamic formulas are over-simplifications of a very complex problem.

It is certain that each of the 450 formulas available to choose from will give reliable results under particular combinations of driving equipment and soil conditions for a particular pile, but there would seem to be an almost infinite number of combinations of the variables involved making the choice of a formula a difficult one. That so many formulas have been developed is indicative of the interest in the problem and the ingenuity of engineers in attempting to establish a relationship between static and dynamic resistance of piles. Empirical formulas certainly have their place when their range of applicability is clearly established and when their limitations are kept clearly in mind; however, this has not always been the case in pile driving.

In spite of the interest exhibited, there has not been a great deal of work done theoretically or experimentally in defining or attacking the fundamental problems involved. The complexity of the

problem, the difficulty in isolating the variables, and the limited techniques and equipment available for detailed analysis have generally limited advances in this field. Full utilization, however, of the facilities available has not been realized.

The basic problem is to relate the dynamic resistance to driving of a single pile to its static bearing capacity with time effects between driving and static loading minimized. Available to assist in the analysis of this problem is the wave equation which was developed over a hundred years ago by De Saint Venant and Boussinesq for end impact on rods. It is well founded mathematically. D. V. Issaccs, in 1931, was the first to point out that wave action occurred during the driving of piles. In 1938, E. N. Fox published a solution to the wave equation applied to pile driving, but, as no electronic computers were available at that time, he made a number of simplifying assumptions. Again in 1940 and 1941, A. E. Cummings reported on the work of the foregoing investigators. Ten years elapsed before the wave equation was again proposed as being applicable to pile driving analysis, this time by E. A. L. Smith (ref. 27). Then in 1961 Smith published a more comprehensive article on application of the wave equation to pile driving and described a numerical approach together with a description of his efforts in developing a computer program for the solution (ref. 28). His efforts have been untiring in developing this approach and have opened the door for much needed work in this field.

The wave equation, as will be shown, mimics the behavior of a pile and provides a good indication of what will happen to the pile

under conditions of impact for specific boundary conditions. For this reason, a distinction is made between the wave equation and dynamic formulas which appear to be mathematical but are in reality empirical. The present state of knowledge up through Smith's work can provide a prediction of effects caused by a ram hitting a pile. Still to be investigated is the nature of the effects caused by interaction between the pile and the soil. This, too, is a dynamic problem, and it lies in the new field of soil dynamics. This matter will be explored, but, first, the wave equation theory and its numerical solution will be developed.

Chapter III

DEVELOPMENT OF BASIC EQUATIONS AND THEORY

A. Theory

A. E. L. Smith, over a period of several years, derived equations suitable for numerical calculations and computer adaptation for pile driving analysis by the wave equation. His derivations are based on a mathematical model using weights and springs. This work has generally been accepted as correct and as equivalent to the wave equation, but the full derivation establishing this equivalence has not been published. In order to establish this equivalence and the validity of the numerical method, a step-by-step derivation from first principles is presented. For completeness, Smith's derivations from the mathematical model are included.

The general plan for establishing this link is as follows:

a. A partial differential equation is developed and is shown to be a form of the wave equation.

b. This equation is converted into a difference equation for ease of numerical solution.

c. To simplify further numerical solution, a mathematical model consisting of weights and springs is assumed as being equivalent to the elastic rod used as a basis for steps a and b above. Five equations are developed from this mathematical model which is based on the assumption that all springs are perfectly elastic, and the pile is represented typically as shown by figure (3).

d. It is then shown that the equations of step c. can be combined to form the same equation developed in step b. above, and it is concluded that the mathematical model and the five equations developed from it are equivalent to the elastic rod, and the wave equation is applicable to it.

Beginning with development of the partial differential equation, as shown by Timoshenko and Goodier (ref. 32), consider the longitudinal waves in a prismatic bar of constant cross section such as shown in figure (1). The axis of the bar is taken as the x-axis. It is assumed that:

a. Cross sections of the bar remain plane during deformation.

b. The unit elongation at any cross section mn, due to a longitudinal displacement D, is equal to $\frac{\partial D}{\partial X}$, and the corresponding tensile force in the bar is AE $\frac{\partial D}{\partial X}$, where A is the cross sectional area and E is Young's modulus.

c. A simple tension in the x direction and the unit elongation $\frac{dD}{dX}$ is accompanied by lateral contraction of the amount $\nu \frac{dD}{dX}$, where ν is poisson's ratio. Inertia forces corresponding to lateral particle motion are neglected since the length of the waves is large in comparison with the cross sectional dimensions of the bar.

The foregoing leads to the equation for force at any section in the bar of uniform cross section:

$$F = EA \frac{\partial D}{\partial X} .$$

The net force on a section of bar dX, figure (1), is:

$$\Delta F = F_2 - F_1 = A_1\left(E_1 \frac{\partial D}{\partial X} + \frac{\partial}{\partial X}\left(E_1 \frac{\partial D}{\partial X}\right) dX\right) - E_1 A_1 \frac{\partial D}{\partial X} = A_1 \frac{\partial}{\partial X}\left(E_1 \frac{\partial D}{\partial X}\right) dX,$$

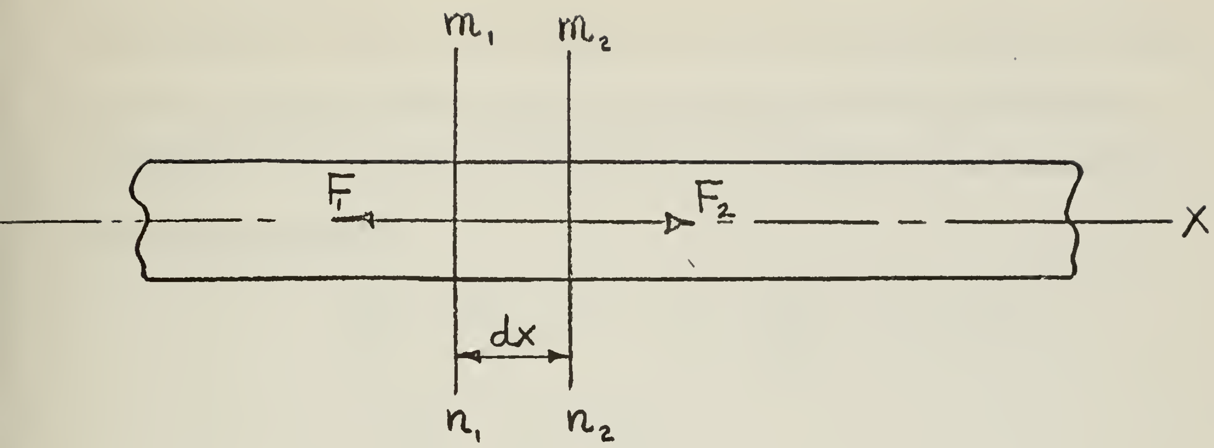


FIGURE (1)
(after ref 32)

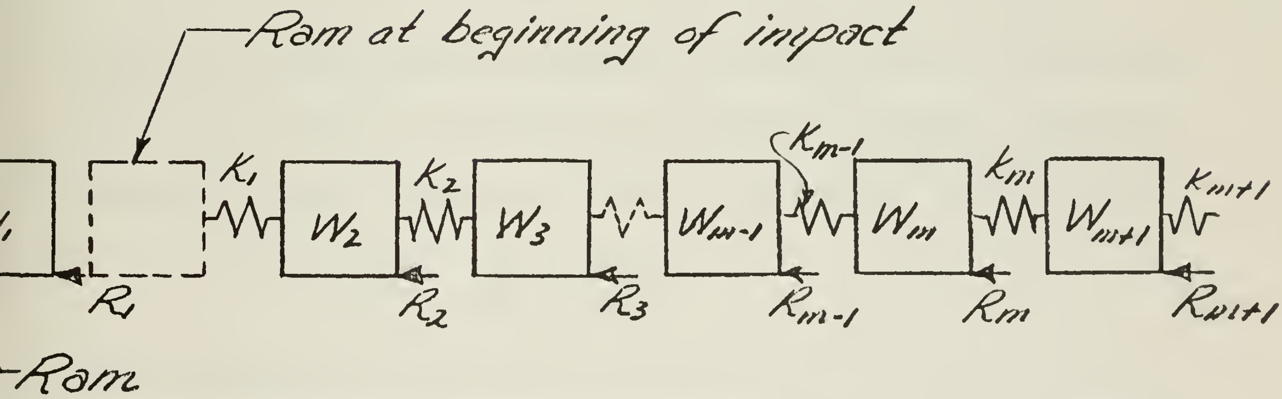


FIGURE (2)
(after reference 27)

assuming that E may not be constant for purposes to be shown below. Considering the motion of the particle dX, and applying Newton's second law that a particle acted upon by an unbalanced force system has an acceleration in line with and directly proportional to the resultant of the force system,

$$A \frac{\partial}{\partial X} (E \frac{\partial D}{\partial X})\, dX = A\, dX\, \rho \frac{\partial^2 D}{\partial t^2}, \quad \text{simplifying}$$

$$\rho \frac{\partial^2 D}{\partial t^2} = \frac{\partial}{\partial X} (E \frac{\partial D}{\partial X}), \tag{1}$$

and if E is constant,

$$\frac{\partial^2 D}{\partial t^2} = \frac{E}{\rho} \frac{\partial^2 D}{\partial X^2} .$$

Introducing a constant c of a value such that $c^2 = \frac{E}{\rho}$, this reduces to the familiar form of the wave equation, ref. (14):

$$\frac{\partial^2 D}{\partial t^2} = c^2 \frac{\partial^2 D}{\partial X^2} .$$

Thus, it is established that equation (1) is a form of the wave equation. It is desired to modify it to account for boundary conditions expected in a pile. This can be done in the general case as follows:

$$\rho \frac{\partial^2 D}{\partial^2 t} = \frac{\partial}{\partial X} (E \frac{\partial D}{\partial X}) \pm R , \tag{2}$$

with R representing resistance to driving. Equation (2) can be converted into a difference equation for numerical solution by standard methods (ref. 24):

$$\frac{\rho}{\Delta t^2}\left[D_{(x,t+\Delta t)} - 2D_{(x,t)} + D_{(x,t-\Delta t)}\right]$$

$$= \frac{L}{A\Delta X^2}\left\{\left[K_{(x+\frac{\Delta x}{2})}\right]\left[D_{(x+\Delta x,t)} - D_{(x,t)}\right] - \left[K_{(x-\frac{\Delta x}{2})}\right]\left[D_{x,t} - D_{(x-\Delta x,t)}\right]\right\}$$

where $E = \frac{KL}{A}$ and $D(x,t)$ is the displacement of point x during time interval t.

It is convenient at this point to introduce a simplified terminology which will facilitate comparison with model equations to be developed later, and to correlate with existing literature on this subject (ref. 28).

Let

$$D_{(x,t+\Delta t)} = D_m$$

$$D_{(x,t)} = d_m$$

$$D_{(x,t-\Delta t)} = d^*_m$$

$$K_{(x+\frac{\Delta x}{2})} = K_m$$

$$K_{(x-\frac{\Delta x}{2})} = K_{m-1}$$

$$D_{(x+\Delta x,t)} = d_{m+1}$$

$$D_{(x-\Delta x,t)} = d_{m-1}$$

$$R = R_m$$

Then

$$\frac{\rho}{\Delta t^2}\left[D_m - 2d_m + d^*_m\right] = \frac{L}{A\Delta X^2}\left\{K_m(d_{m+1} - d_m) - K_{m-1}(d_m - d_{m-1}) - R_m\right\}$$

Simplifying,

$$D_m - 2d_m + d^*_m = \frac{L}{A}\,\frac{\Delta t^2}{\rho\Delta X^2}\left\{K_m(d_{m+1} - d_m) - K_{m-1}(d_m - d_{m-1}) - R_m\right\}$$

$$D_m = 2d_m - d^*_m + \frac{g\Delta t^2}{W_m}\left\{(d_{m-1} - d_m)K_{m-1} - (d_m - d_{m+1})K_m - R_m\right\}$$

If the acceleration of gravity, g, is expressed in feet per second, then

$$D_m = 2d_m - d_m^* + \frac{12g\Delta t^2}{W_m}\left\{(d_{m-1} - d_m)K_{m-1} - (d_m - d_{m+1})K_m - R_m\right\} \quad (3)$$

For purposes of numerical calculation, it is more convenient to use a mathematical model consisting of weights and springs such as is shown in figure (2). The following notation will be used and is consistent with that used in equation (3) above and in reference (28).

1, 2, 3, . . . m-1, m, . . . are subscripts designating particular weights and associated springs,

1, 2, 3, . . . n-1, n, . . . are subscripts designating specific time intervals. Zero designates the initial instant.

The following apply to any time interval n:

D_m = displacement of weight m measured from its initial position in inches

C_m = compression of spring m in inches

F_m = force exerted by spring m, in pounds

Z_m = net force acting on weight m, in pounds

V_m = velocity of weight m, in feet per second

The following apply to the preceding time interval n-1:

d_m = displacement of weight m measured from its initial position in inches

c_m = compression of spring m, in inches

f_m = force exerted by spring m, in pounds

v_m = velocity of weight m, in feet per second

z_m = net force acting on weight m, in pounds

The following are usually constants:

W_m = magnitude of weight m, in pounds

K_m = spring constant for spring m, in pounds per inch

R_m = external force or resistance acting on weight m, in pounds

g = acceleration due to gravity (32.2 feet/second)

E = Young's modulus of elasticity in pounds/square inch

A = cross sectional area of pile in square inches

ℓ = unit length in inches

Δt = time interval used for numerical calculations, in seconds

T_m = critical time interval for spring m, in seconds

T_{min} = minimum value of T_m, in seconds

ϕ = over-all "factor of safety" = $T_{min}/\Delta t$

ϕ_{min} = individual factor of safety for spring m = $T_m/\Delta t$

The numerical calculations will be a step-by-step process in which the five variables, D_m, C_m, F_m, Z_m, and V_m, will be calculated for each weight or spring in each successive time interval. Formulas for these five variables are developed by Smith (ref. 28) as follows:

$$D_m = d_m + v_m(12\Delta t) \quad (4)$$

The coefficient 12 is included since D_m and d_m are expressed in inches and v_m in feet per second.

Considering C_m next, and referring to figure (3), let the dashed squares represent the initial positions of weights m and m + 1, and let the solid squares represent their positions at the end of time interval n. The initial length of spring m is ℓ, and at the end of time interval

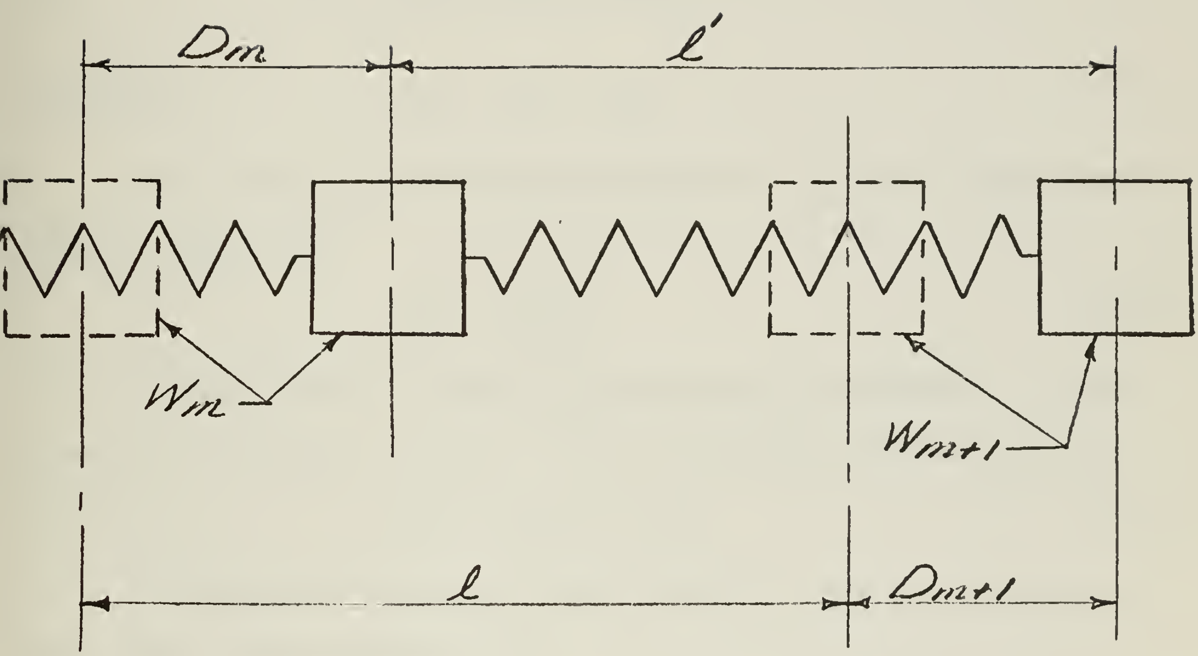


FIGURE (3)
(after ref. 27)

it is ℓ'. D_m and D_{m+1} are the displacements of weights m and m + 1 respectively, during time interval n. Then,

$$C_m = \ell - \ell'$$

but $$\ell + D_{m+1} = \ell' + D_m$$

or $$D_m - D_{m+1} = \ell - \ell'$$

therefore, $$C_m = D_m - D_{m+1} \tag{5}$$

The force F_m is directly proportional to C_m and to the spring constant K_m, so that

$$F_m = C_m K_m \tag{6}$$

From figure (2), weight m is acted upon by springs m - 1 and m and by the external force R_m . The accelerating force is given by

$$Z_m = F_{m-1} - F_m - R_m \tag{7}$$

From Newton's second law that change in velocity is equal to force times time divided by mass,

$$V_m = v_m + Z_m\left(\frac{\Delta tg}{W_m}\right) \tag{8}$$

It remains to be shown that the model of figure (2) and equations (4) through (8) are equivalent to the wave equation. This is done by using equations (1) through (8) in the following manner:

Rewriting equation (8) for the previous time interval gives

$$v_m = v_m^* + z_m\left(\frac{\Delta tg}{W_m}\right) \tag{8a}$$

Similarly, equation (7) for the previous time interval is

$$z_m = f_{m-1} - f_m - R_m \tag{7a}$$

Substituting equation (7a) into equation (8a) gives

$$v_m = v_m^* + (f_{m-1} - f_m - R_m)\frac{\Delta tg}{W_m} \qquad (9)$$

Substituting equation (9) into equation (4) gives

$$D_m = d_m + \left[v_m^* + (f_{m-1} - f_m - R_m)\frac{\Delta tg}{W_m}\right] 12\,\Delta t$$

or

$$D_m = d_m + 12\,\Delta t v_m^* + \frac{12\,\Delta t^2 g}{W_m}\left[f_{m-1} - f_m - R_m\right] \qquad (4a)$$

But equation (4) for the previous time interval is

$$d_m = d_m^* + v_m^*\,(12\,\Delta t)$$

or

$$v_m^* = \frac{d_m - d_m^*}{12\,\Delta t} \qquad (4b)$$

Substituting (4b) into (4a) yields

$$D_m = d_m + 12\,\Delta t\left[\frac{d_m - d_m^*}{12\,\Delta t}\right] + \frac{12\,\Delta t^2 g}{W_m}\left[f_{m-1} - f_m - R_m\right]$$

Substituting equations (5) and (6) for the previous time interval into the above equation and simplifying gives

$$D_m = 2d_m - d_m^* + \frac{12\,\Delta t^2 g}{W_m}\left[(d_{m-1} - d_m)K_{m-1} - (d_m - d_{m+1})K_m - R_m\right]$$

which is equation (3). Thus equations (4) through (8) are equivalent to the wave equation, and the mathematical model of figure (2) has been shown to be a valid one.

Of course, solving the partial differential equation by a numerical integration process is an approximate one, but the results should be accurate within five per cent (ref. 28) provided that the time interval chosen is small enough and is coordinated with the length of pile selected to be represented as a block and spring in the mathematical model so that a stable set of calculations is produced. This

matter will be discussed in more detail later, but experience indicates that 5- to 10-foot sections of a pile may be so represented with a time interval of from 0.00025 to 0.00033 seconds, depending upon the type of pile material. As to the numerical method, it may be considered sufficiently accurate in light of present knowledge upon which the many factors influencing any pile driving operation are evaluated.

B. Mathematical Model Representation

That a pile may be treated mathematically as a series of weights and springs not only facilitates the computations but also assists in visualizing the problem. For that reason, the remainder of this paper will deal principally in terms of the mathematical model of a pile. It is, therefore, important that this method of pile representation be thoroughly understood at the outset. The following sections will adhere closely to reference (28).

The Hammer-Ram is usually a short, heavy, rigid object which may be represented by an individual weight without elasticity. In figure (4) the first weight W_1 represents the ram.

The velocity of the ram at the instant of impact is needed to start the numerical calculations. Ordinarily the rated foot pounds of energy of the hammer is given by the manufacturer. The efficiency is sometimes given, and sometimes must be assumed. From these data the velocity may be calculated at impact by the following formula:

$$\underset{\text{(ft/sec)}}{\text{velocity at impact}} = \sqrt{\frac{\text{Rated energy (ft-lb)} \times \text{efficiency} \times 64.4}{\text{weight (lb)}}} \qquad (10)$$

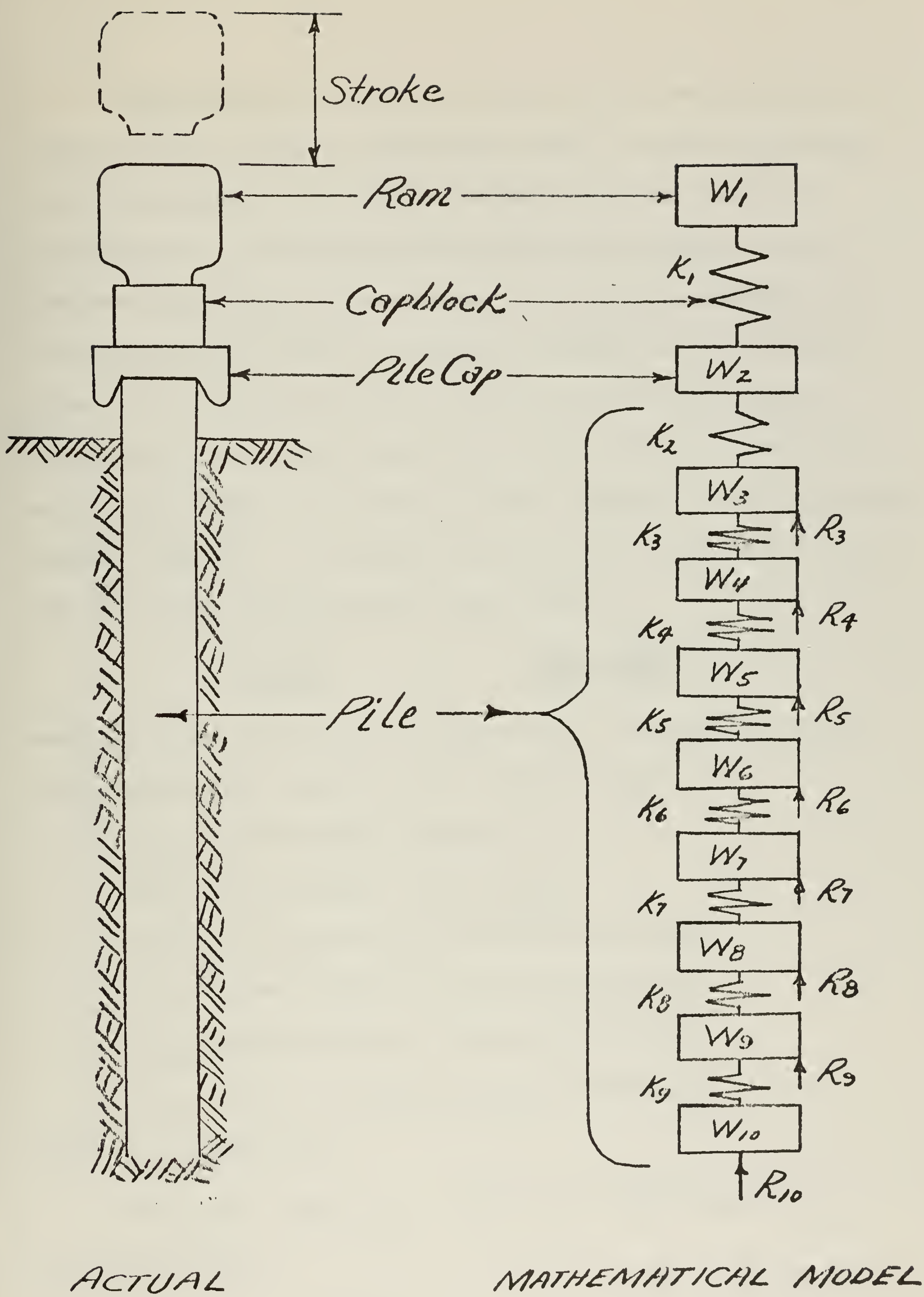


FIGURE (4)
(after ref. 28)

The Capblock is a short, springy object of wood, plastic, or similar material which is comparatively light, and which may therefore be represented by a spring. In figure (4) this spring is identified as K_1. The form of the stress-strain diagram (or the hysteresis loop) that is produced as the capblock is suddenly compressed and then allowed to re-expand is assumed to be as shown by figure (5). Compression occurs along the line AB whose slope is determined by the elastic constant K_1 of the capblock Restitution occurs first along the line BD, and then, because the capblock cannot transmit tension, it is completed along line DA. ABDA is the hysteresis loop. The computations are made so that

$$\frac{\text{Area BCD}}{\text{Area ABC}} = (e_1)^2 = \frac{\text{Energy output}}{\text{Energy input}}$$

where e_1 is the coefficient or restitution of the capblock. This relationship may be derived in the following manner:

Let W_{ram} be the weight of the ram

W_{cap} be the weight of the capblock

v_{ram} be the velocity of the ram before impact

v_{cap} be the velocity of the cap before impact

v'_{ram} be the velocity of the ram after impact

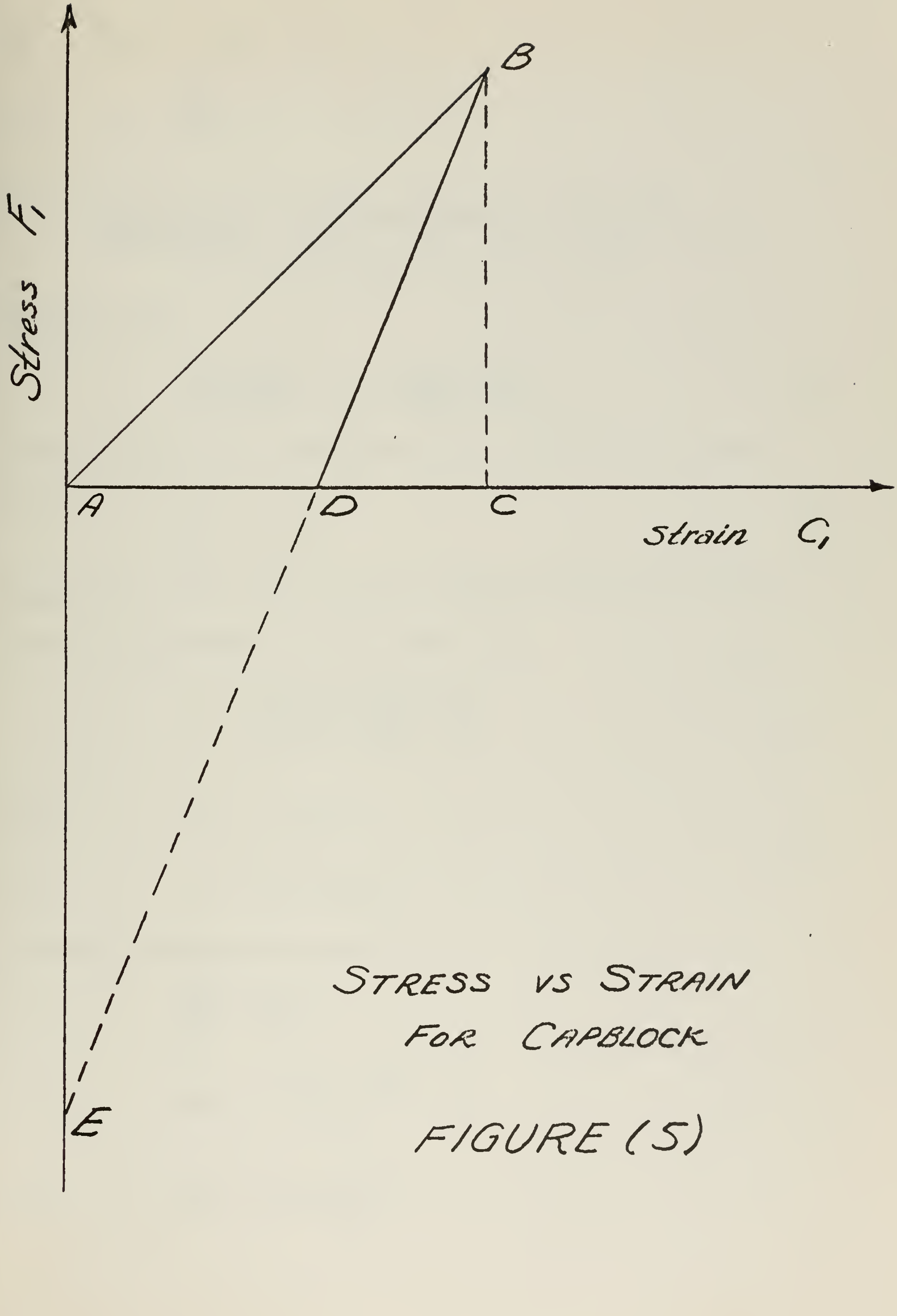
v'_{cap} be the velocity of the cap after impact

Then, from Newton's laws,

$$W_{ram}\, v_{ram} + W_{cap}\, v_{cap} = W_{ram}\, v'_{ram} + W_{cap}\, v'_{cap}$$

and by definition,

$$e_1 = \frac{v'_{cap} - v'_{ram}}{v_{ram} - v_{cap}}$$



STRESS VS STRAIN
FOR CAPBLOCK

FIGURE (5)

If $v_{cap} = v'_{cap} = 0$,

then $e_1 = \frac{v'_{ram}}{v_{ram}}$

$$\frac{\text{energy output}}{\text{energy input}} = \frac{(1/2)(W_{ram}/g)(v'_{ram})^2}{(1/2)(W_{ram}/g)(v_{ram})^2} = \frac{(e_1 \cdot v_{ram})^2}{(v_{ram})^2} = e_1^2$$

From figure (5),

$$\frac{\text{area BCD}}{\text{area ABC}} = e_1^2 = \frac{\text{energy output}}{\text{energy input}} \qquad (11)$$

During compression the spring force may be computed from equation (6),

$$F_1 = C_1 K_1 \qquad (12)$$

Inelasticity of the capblock must be considered during restitution, however. From equation (11) and figure (5),

$$e_1^2 = \frac{(1/2)\,\overline{BC} \times \overline{CD}}{(1/2)\,\overline{BC} \times \overline{AC}} = \frac{\overline{CD}}{\overline{AC}}$$

but, $\overline{AC} = C_{1\ max}$

and $CD = e_1^2\, C_{1\ max}$

by similar triangles ADE and BCD,

$$\frac{\overline{AE}}{\overline{BC}} = \frac{\overline{AD}}{\overline{CD}}$$

$$\overline{AE} = \frac{C_{1\ max} - \overline{CD}}{\overline{CD}} \cdot \overline{BC}$$

and $\overline{AE} = -\overline{BC}\left(\frac{1-e_1^2}{e_1^2}\right)$

Writing the equation for line DB gives

$$F_1 = \frac{\overline{BC}}{\overline{CD}} C_1 - \overline{BC} \left(\frac{1-e_1^2}{e_1^2}\right)$$

and substituting for $\overline{CD}$,

$$F_1 = \frac{\overline{BC}}{e_1^2 C_{1\,max}} \cdot C_1 - \overline{BC} \left(\frac{1-e_1^2}{e_1^2}\right)$$

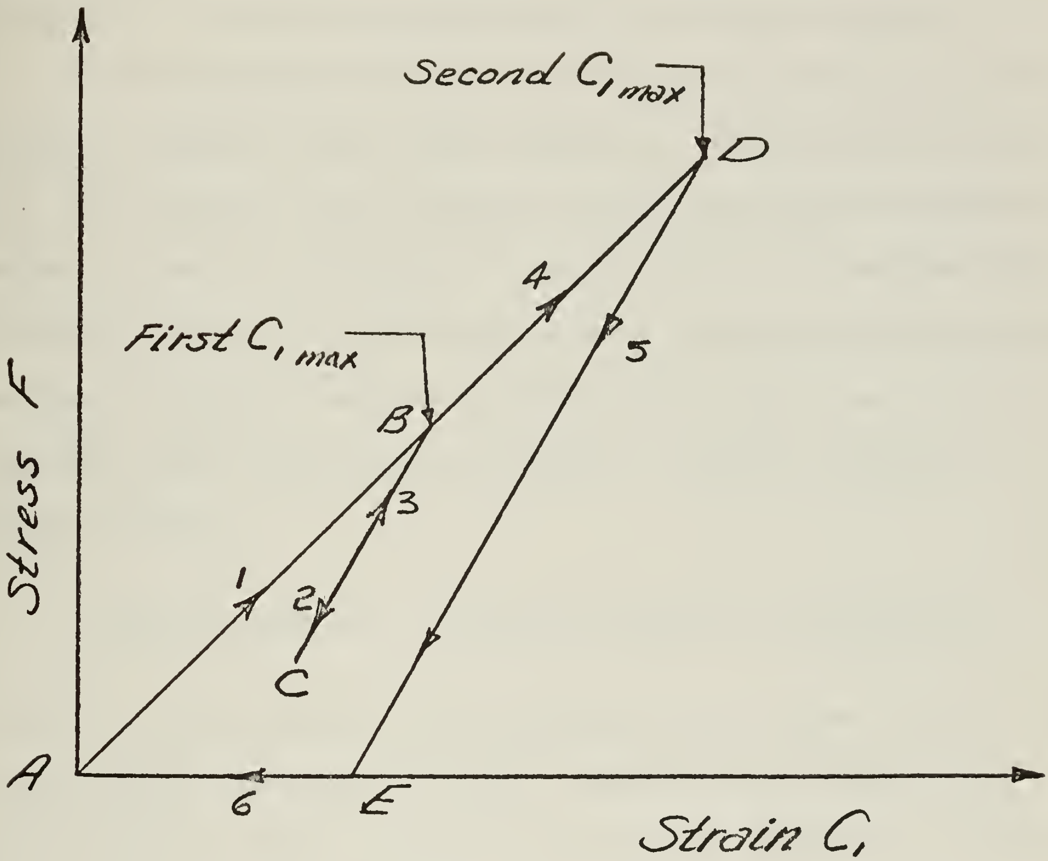
Since $\overline{BC} = C_{1\,max} K_1$,

$$F_1 = \frac{C_{1\,max} K_1 C_1}{e_1^2 C_{1\,max}} - C_{1\,max} K_1 \left(\frac{1-e_1^2}{e_1^2}\right)$$

which reduces to

$$F_1 = \frac{K_1 C_1}{e_1^2} - C_{1\,max} K_1 \left(\frac{1}{e_1^2} - 1\right) \qquad (13)$$

Calculations involving the capblock will, therefore, utilize alternately equation (12) during compression and equation (13) during restitution. $C_{1\,max}$ should not be thought of as a constant, but as the maximum compression occurring in spring K_1 before restitution begins. It often happens that recompression occurs and a new value of $C_{1\,max}$ is reached before restitution begins again. This is illustrated by figure (5A) where the stress-strain diagram begins at A, goes to B during compression following equation (12), goes to C during restitution as governed by equation (13), at C recompression begins, goes to B following equation (13), continues to D by equation (12), and thence to E by equation (13). As previously described the return to A is along EA because no tension can be transmitted.



STRESS VS STRAIN FOR CAPBLOCK AND SHOWING RECOMPRESSION

FIGURE (5A)

That recompression can occur may not be apparent readily, but a feeling for it may be acquired in solving a problem or two manually.

As will be noted later, equations (12) and (13) are used in the case of a cushion block, but with the subscripts changed to 2.

Few tests have been made to determine the elastic characteristics of capblocks under impact conditions. Raymond International, Inc., in conducting a small number of tests found that the characteristics of a wooden capblock vary during driving, and concluded that in order to be on the conservative side in computing pile penetration per blow, a hardwood capblock with the grain vertical and of an original six-inch height, may be assumed to have the following characteristics:

$$\text{Spring constant } K = 20{,}000\ A \text{ (lb/inch of compression)}$$

where A is the horizontal cross sectional area in square inches. The coefficient of restitution was 0.5. The tests also showed that a Micarta capblock (Nema Grade "C") of 12 inches' height had a spring constant given by the following relation:

$$K = 45{,}000\ A$$

and the coefficient of restitution was 0.8.

Pile cap, or Follower, or Helmet--like the ram of the hammer, the pile cap is ordinarily a short, heavy, rigid object that can be represented by a single weight without elasticity, such as W_2 in figure (4). If the pile cap is long and slender, as is the case when

used as a follower to drive piles below water or the ground, it is better to represent it by a number of weights and springs as shown in figure (6e). In this case elastic constants must be computed by the formula

$$K = \frac{AE}{\ell} \tag{14}$$

where A is the cross sectional area in square inches, E is the modulus of elasticity in pounds per square inch, and ℓ is the unit length in inches represented by one spring.

Cushion Blocks--(called "head packing in the U.K.)--in figure (4) springs K_2 to K_9 inclusive represent the elasticity of the pile itself. However, if a precast concrete pile is being driven, a cushion block must be used under the pile cap W_2 to protect the concrete from shattering. In this case, spring K_2 would represent the cushion in combination with the first spring of the pile. Figure (6c) applies to the cushion block as well as the capblock, and they are treated mathematically in the same manner. Dynamic tests similar to those described for capblocks show that 4-inch-thick pine boards with the grain horizontal as used on top of a precast pile to distribute the blow evenly may be assumed to have a spring constant equal to 3,480 A and a coefficient of restitution of 0.5. Of course, the characteristics of the wood vary during driving, but use of the above relationship is on the conservative side when used for calculating penetration of the pile hammer blow.

The Pile will be either of wood, concrete, or steel. All of these materials have traditionally been treated structurally as behaving

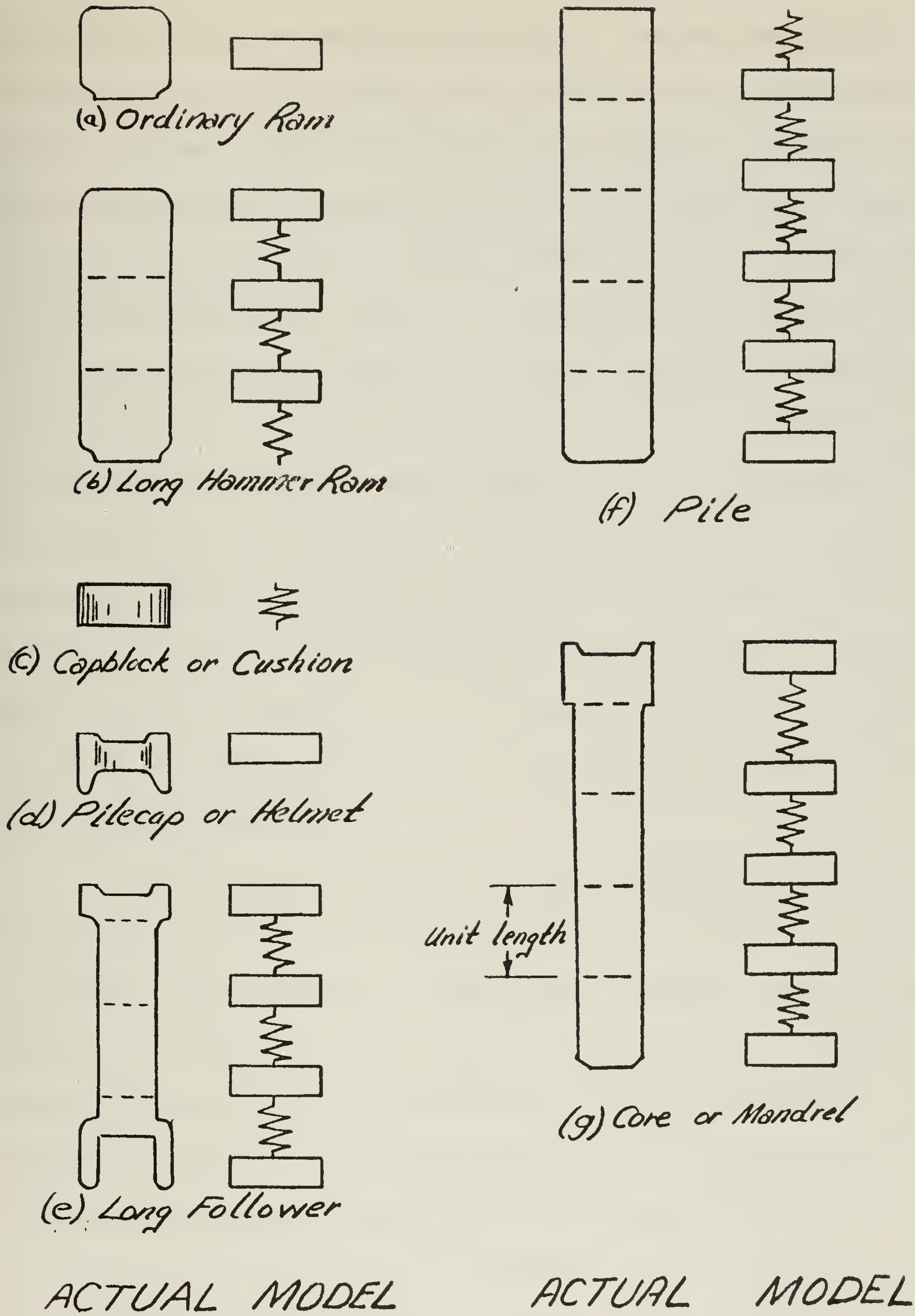
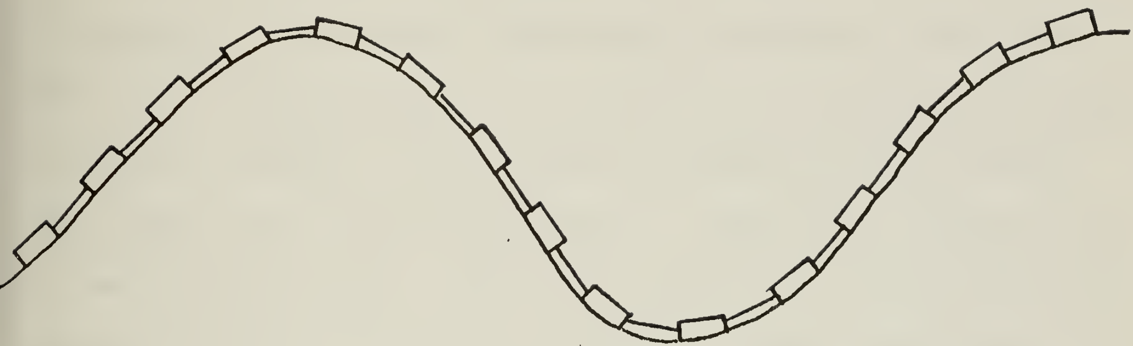


FIGURE (6)
MODEL REPRESENTATION OF
PILE DRIVING ELEMENTS
(after ref. 28)

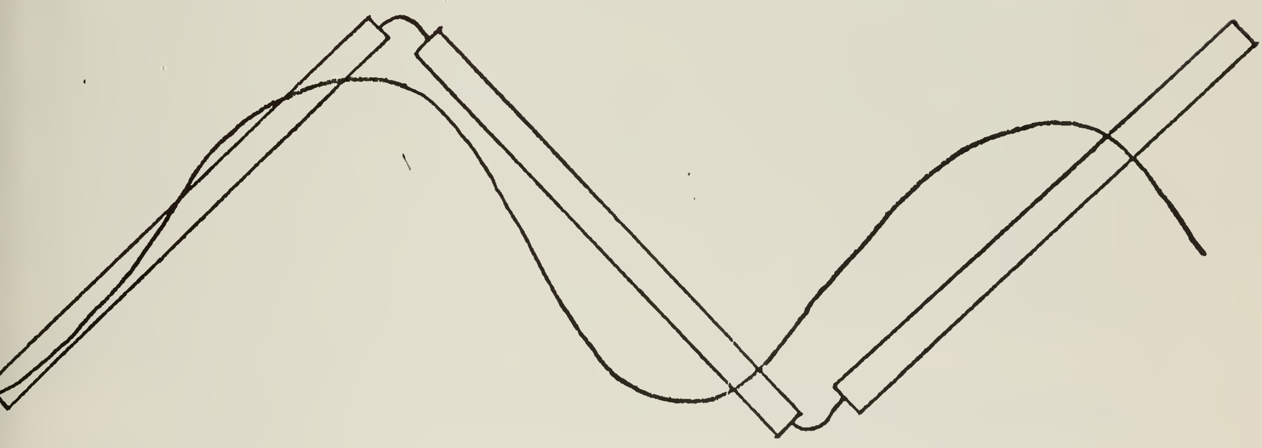
elastically within the working load ranges. Because the pile is compressible due to its length, it is subject to wave action under the blow of a hammer. This wave action may be analyzed by dividing the pile into short unit lengths of 5 to 10 feet. The weight of each length is noted as W_3 through W_{10}, and the elasticity of each length by an individual spring as shown by K_2 through K_9 in figure (4). If the pile is of uniform section, the weights and springs representing it are identical. If the pile is tapered or is a composite pile, or is in any way of irregular cross section, then the weights and springs are modified to represent the pile under investigation. That comparatively large unit lengths, such as 10 feet, give desired accuracy in computing the action of impact waves may not be clear. Although water waves are transverse waves and the waves in a pile are longitudinal waves, water waves nevertheless can be used as an analogy to illustrate the principles involved.

If a long strip of flexible material were allowed to float on the surface of a body of water, it would follow the wave action. If, for purposes of mathematical analysis, this flexible strip were represented by a number of short floats of rigid construction but connected by flexible links, the mathematical model would appear as in figure (7). This representation would be an approximation, but it would involve negligible error because the small floats would ride the waves almost exactly like the flexible strip. If, however, the rigid floats are made comparatively long and they approach or exceed the length of the water wave, then the model would be as shown by figure (8). Obviously these long floats cannot follow the wave form closely.



SHORT FLOATS

FIGURE (7)
(after ref. 28)



LONG FLOATS

FIGURE (8)

(after ref 28)

Applying this analogy to a pile being hit by a ram, the importance of dividing the pile into unit lengths considerably shorter than the stress wave produced by a hammer blow can be appreciated. Fortunately, a pile driving impact usually produces a fairly long wave form, and unit lengths of the order of 5 to 10 feet will produce acceptable accuracy. In the special case where the exact form of the impact wave is being investigated, a smaller unit length of 1 or 2 feet may be advisable. Under these conditions, it may also be advisable to divide the ram into a series of weights and springs as is done with the pile. For normal pile driving analysis, such refinements are not considered justified.

There is a very important relationship between the unit length of pile selected and the time interval chosen for the calculations. the time interval to be used in the calculations is a function of the weight of the unit length selected and of the elasticity and density of the pile material. This relationship is derived and presented later, and governs the upper bound of the time interval to be used. On the other hand, choice of too small a time interval results in excessive computation for the accuracy required.

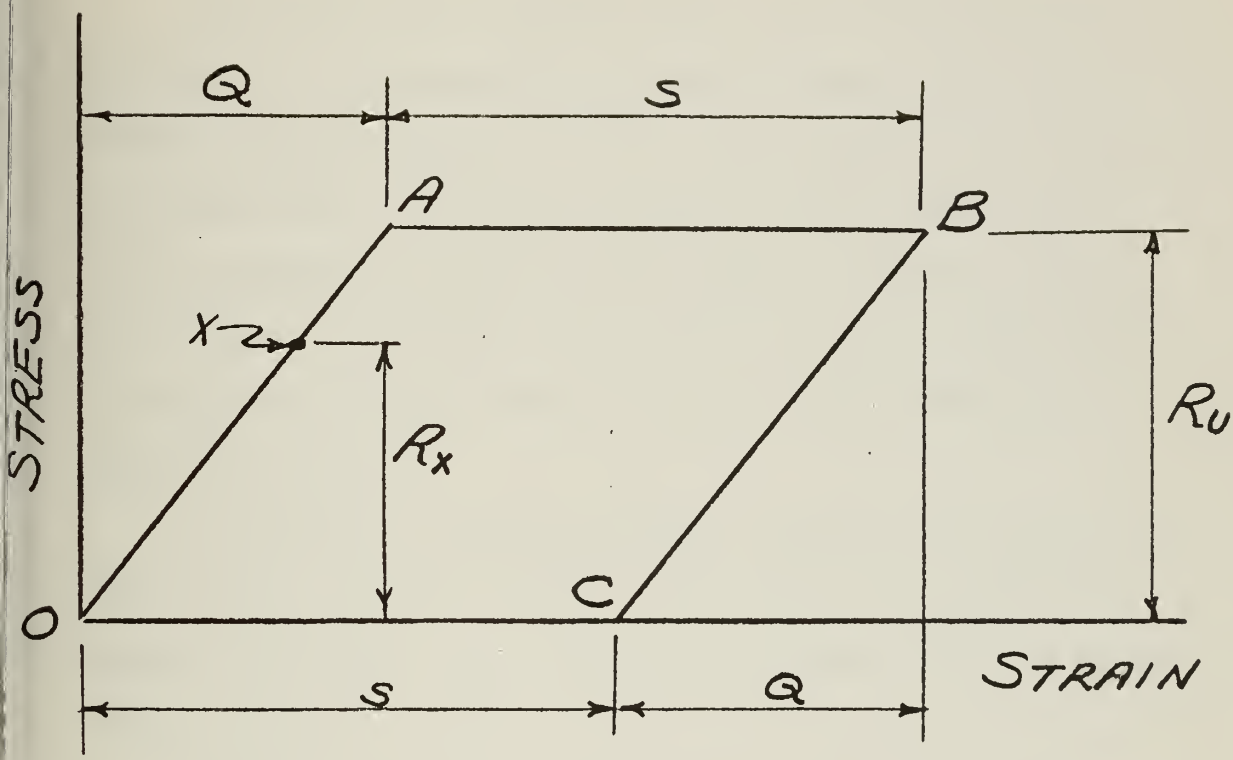
The spring constants are determined by the formula given for K under "Capblock" above.

<u>The Ground</u>--as indicated by figure (4) provision is made for ground resistance to be applied on each section of the pile. In order to make a calculation using the wave equation, an assumption must be made as to the amount of resistance which will be offered by the ground at each of the pile sections. Furthermore, a stress-strain

relationship of the soil and viscous damping must be considered. For the purpose of this section, it will be assumed that these factors can be evaluated so that the method of their inclusion in the analysis may be developed.

Resistance at the pile point--according to Chellis (ref. 5) the ground compresses elastically for a certain distance and then fails plastically with constant ultimate resistance R_u. This concept is illustrated by figure (9). The elastic deformation will be called "quake," and is denoted by the letter 'Q.' The plastic deformation will be referred to as "permanent set," and is identified as 's'; in figure (9) it is the distance AB or OC. There is a factor of importance which has not yet been included, and this is the rate of stress application. It is clear that the ground will offer more instantaneous resistance to a rapid advancement of the pile tip than to a slow advancement. Viscous damping, therefore, will be included in the analysis to provide for the effect of rate of penetration.

As has been shown, the wave equation calculation will give the instantaneous velocity of the point of the pile in any time interval. If J is designated as a damping constant and v_p the velocity of the pile point, then their product, Jv_p, can be used to increase or decrease ground resistance at the point of the pile so as to produce damping. The instantaneous damping resistance at point x, for example, in figure (9) is Jv_pR_x. This value would be used for R_{10} for a particular time interval applied as shown by figure (4). This damping resistance is a resistance to driving the pile and does not, of course,



STRESS VS STRAIN AT PILE POINT

FIGURE (9)

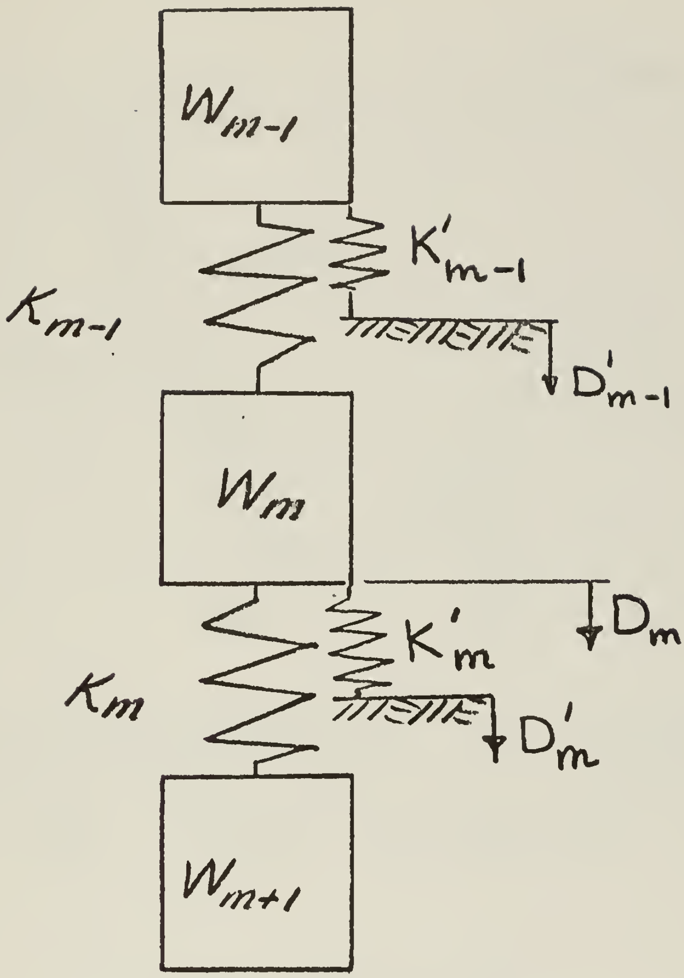
(after ref. 28)

contribute to the bearing capacity of the pile in supporting static loads.

Resistance Along the Sides of the Pile--side resistance is calculated in the same manner as at the pile point, except that a different damping constant J' is used, and it applies to R_3 through R_9 in figure (4). As the pile is driven, the soil under the pile point is displaced and caused to flow aside very rapidly. On the other hand, the soil at the sides of the pile, as it is being dirven, is not displaced nearly as much. It may be assumed, then, that J' should be less than J.

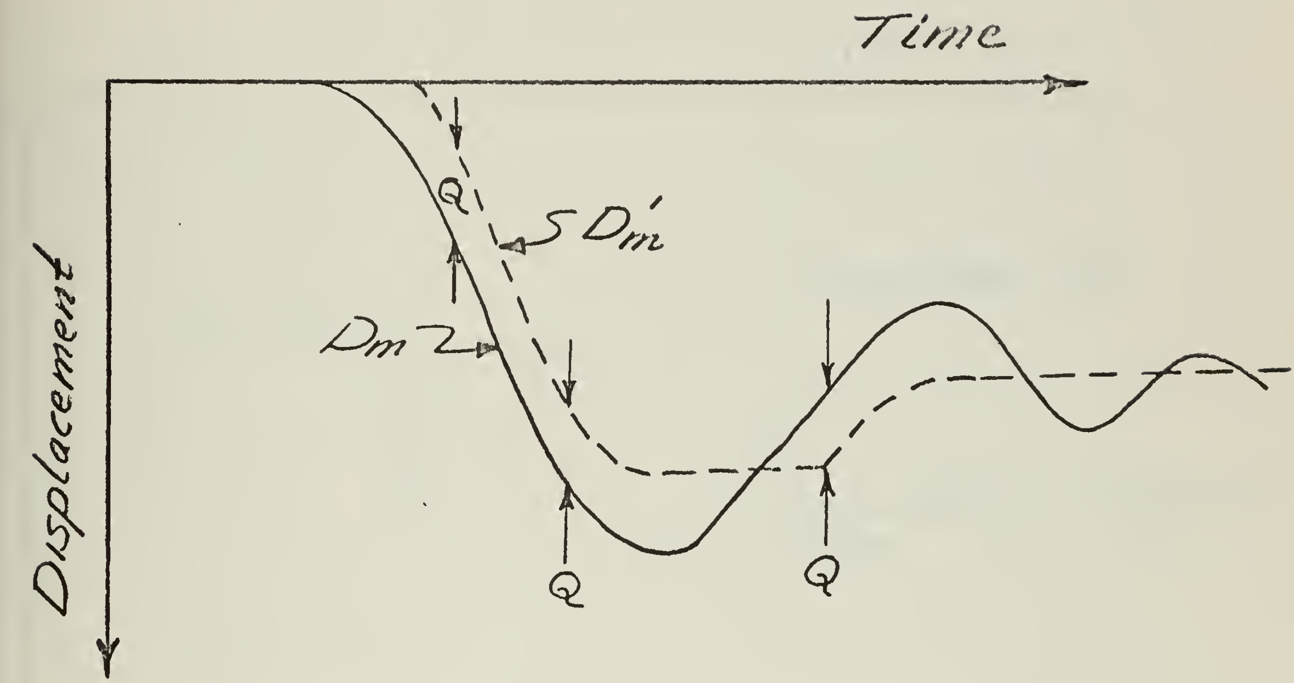
Distribution of Ground Resistance Between the Pile Sides and Point--the wave equation computation will utilize any distribution chosen. The problem of what distribution to choose will be discussed at length later.

Figure (10) shows the model representation of the ground resistance along the sides of the pile. It must be remembered that ground quake is an elastic, or recoverable, deformation and that the set associated with it is of a temporary nature. The deflection of the spring K'_m must, therefore, be limited to a maximum value less than or equal to Q, the ground quake. As the maximum allowed deflection of spring K'_m is reached, provision is made for plastic deformation of the ground, D'_m, in the normal downward direction, or in an upward direction in the case of pile rebound. Figure (11) illustrates the way in which D'_m is handled in the calculations. It may be noted that D'_m does not take on a value until D_m exceeds Q. From that point on, D'_m cannot be less than $D_m - Q$ or more than $D_m + Q$.



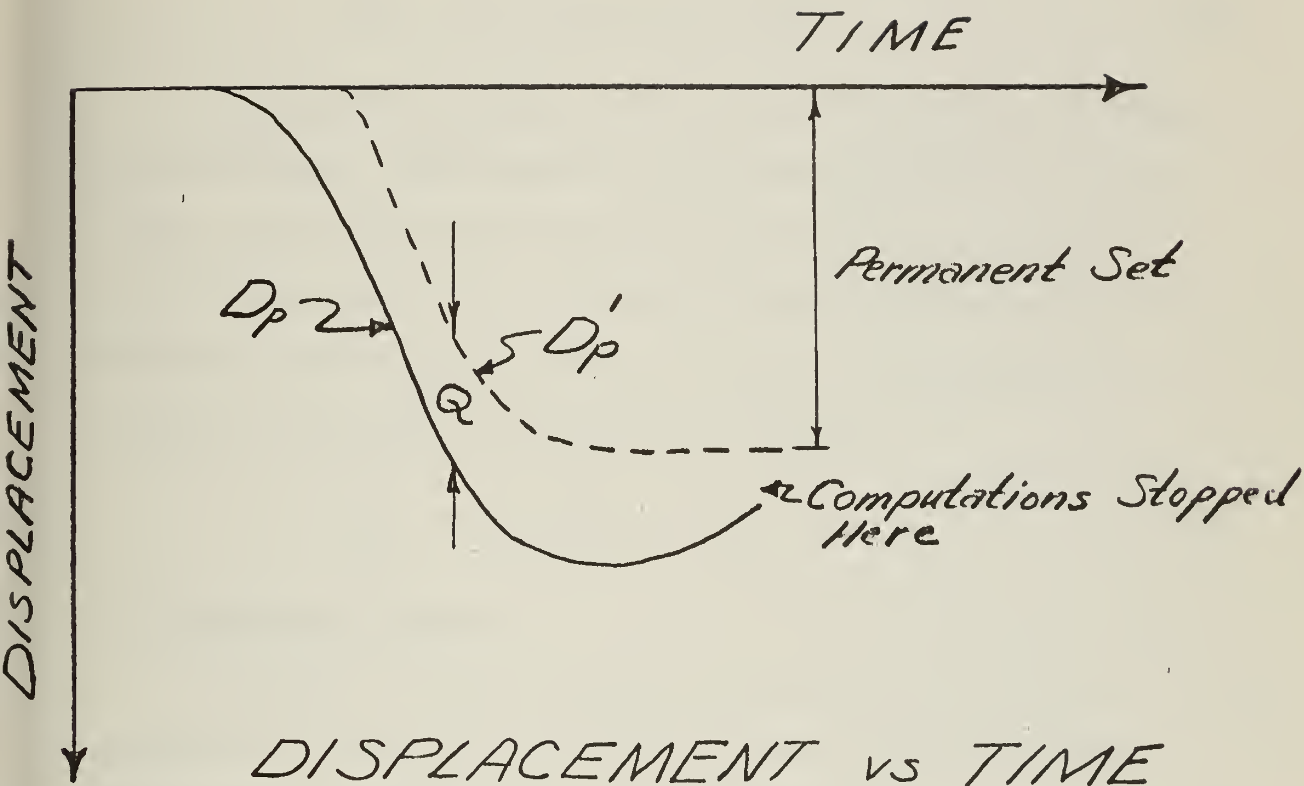
MODEL REPRESENTATION OF GROUND RESISTANCE AT PILE SIDES

FIGURE (10)
(after ref. 28)



DISPLACEMENT VS TIME
AT PILE SIDES

FIGURE (11)
(after ref. 28)



DISPLACEMENT vs TIME
AT PILE POINT

FIGURE (12)
(after ref. 28)

The value of the ground resistance on a particular pile section is computed by the equation

$$R_m = (D_m - D'_m) K'_m (1 + J'v_m) \qquad (15)$$

The point of the pile is dealt with in a similar, but slightly different, manner. Only downward plastic ground movement D'_p is considered. The maximum value of D'_p is the permanent set of the pile which is designated as s . Figure (12) illustrates the way in which the calculations are handled. The value of the ground resistance at the point is computed by the equation

$$R_p = (D_p - D'_p) K'_p (1 + Jv_p) \qquad (16)$$

C. Illustrative Problem

In order to bring the theory more sharply into focus, a hypothetical problem will be set up and solved manually using the general format described by E. A. L. Smith (ref. 27).

Let figure (13) represent a steel pile 30 feet long. The weight of the ram is 5,000 pounds, and its velocity at impact as determined by equation (10) is found to be 10 feet per second. The capblock is hardwood, has a coefficient of restitution of 0.5, and its spring constant as determined by equation (5) is 2,000,000 pounds per inch. The pile cap weighs 500 pounds, and is not able to transmit tension. No cushion block or head packing will be used, so K_2 will be the elasticity of the first pile section whose coefficient of restitution will be taken as 1.00. The pile will be divided into 10-foot unit lengths, each of which weighs 500 pounds. The spring

EXAMPLE PROBLEM

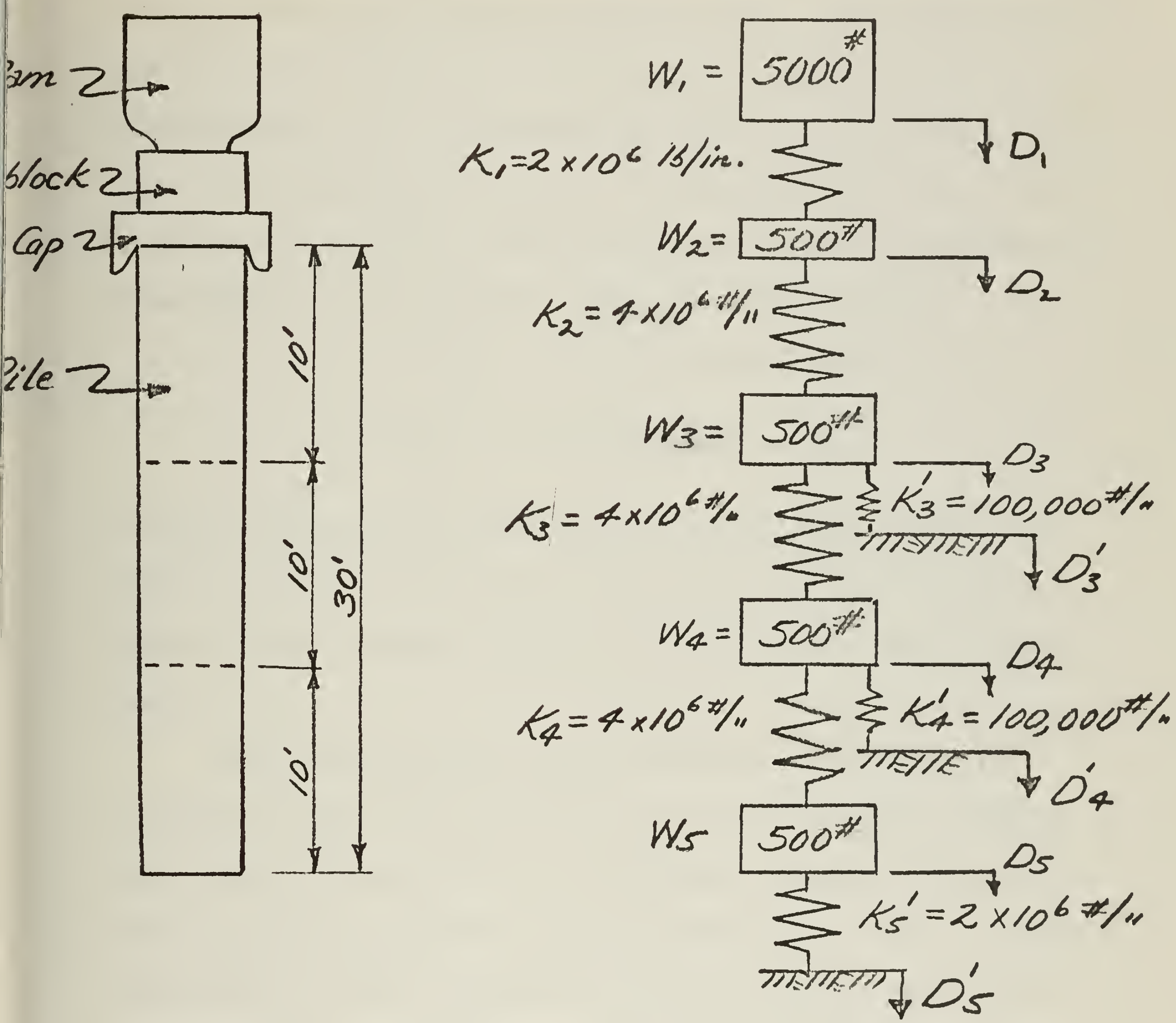


FIGURE (13)

constants K_2, K_3, and K_4 for the pile as computed by equation (6) are each taken to be 4,000,000 pounds per inch. Ground quake will be 0.10 inch. The ultimate ground resistance has been determined to be 220,000 pounds. It is distributed as follows: 10,000 pounds on each of the two top sections of the pile and 200,000 pounds on the bottom 10-foot section of pile. Dividing the ultimate ground resistance by ground quake will give the soil spring constants as follows:

$$K_3' = K_4' = 100{,}000 \text{ pounds per inch}$$

$$\text{and } K_5' = 2{,}000{,}000 \text{ pounds per inch}$$

The time interval for the calculations will be 1/4000 seconds. The damping constants J and J' will both be taken as equal to 0.1 for purposes of this example, although, as noted before, J' may be smaller than J.

The calculations may then be performed by slide rule and recorded in tabular form as shown on the following pages. Although space does not permit here, it is recommended that when making such a table a single large sheet be used. In referring to Table A, on each line values are first computed for D_1, D_2, D_3, D_4, D_5; then C_1, C_2, C_3, C_4, C_5; etc., until all computations have been completed. The next line is approached in the same order, and so on.

It may be of interest to note from Table A that C_1 reaches a first maximum at time interval 8; restitution takes place from time interval 9 through 22. Recompression occurs from time interval 23 through 27 at which time a second $C_{1\,max}$ is reached. Restitution continues until the end of the calculations.

[illegible] C_1, C_2, and C_3 [illegible]

$$[illegible]$$

[illegible]

[illegible]

[illegible]

[illegible]

TABLE A

Time Interval	$D_1 = d_1 + 0.003\ v_1$	$C_1 = D_1 - D_2$	$F_1 = (2x10^6)(C_1)$, except during restitution	$Z_1 = -F_1$	$V_1 = v_1 + \frac{Z_1}{621,000}$
0	0	0	0	0	10.00
1.	0.03	0.03	60,000	- 60,000	9.9035
2.	0.06	0.057	114,000	-114,000	9.7202
3.	0.0892	0.0784	156,800	-156,800	9.468
4.	0.1176	0.0934	186,800	-186,800	9.167
5.	0.1451	0.1025	205,000	-205,000	8.837
6.	0.1716	0.1073	214,600	-214,600	8.491
7.	0.1971	0.1092	218,400	-218,400	8.139
8.	0.2215	0.1096	219,200	-219,200	7.786
9.	0.2449	0.1091	215,200	-215,200	7.440
10.	0.2672	0.1081	207,200	-207,200	7.106
11.	0.2885	0.1069	197,600	-197,600	6.788
12.	0.3088	0.1058	188,800	-188,800	6.484
13.	0.3283	0.1053	184,800	-184,800	6.186
14.	0.3469	0.1051	183,200	-183,200	5.891
15.	0.3646	0.1049	181,600	-181,600	5.599
16.	0.3814	0.1043	176,800	-176,800	5.314
17.	0.3973	0.10334	169,120	-169,120	5.042
18.	0.4124	0.10224	160,320	-160,320	4.784
19.	0.4267	0.10164	155,520	-155,520	4.534
20.	0.4403	0.10238	161,440	-161,440	4.274
21.	0.45312	0.10479	180,720	-180,720	3.984
22.	0.46507	0.10855	210,800	-210,800	3.644
23.	0.47600	0.11266	225,320	-225,320	3.281
24.	0.48584	0.11687	233,740	-233,740	2.905
25.	0.49456	0.12095	241,900	-241,900	2.516
26.	0.50211	0.1243	248,600	-248,600	2.116
27.	0.50846	0.12612	252,240	-252,240	1.710
28.	0.51376	0.12580	249,680	-249,680	1.309
29.	0.51769	0.12294	226,800	-226,800	0.944
30.	0.52052	0.11925	197,280	-197,280	0.626
31.	0.52240	0.11684	178,000	-178,000	0.340
32.	0.52342	0.11697	179,040	-179,040	0.052
33.	0.52357	0.11913	196,320	-196,320	-0.264
34.	0.52278	0.12142	214,640	-214,640	-0.610
35.	0.52095	0.12185	218,080	-218,080	-0.962
36.	0.51806	0.11971	200,960	-200,960	-1.284
37.	0.51421	0.11586	170,160	-170,160	-1.558
38.	0.50954	0.11247	143,040	-143,040	-1.789
39.	0.50417	0.11129	133,600	-133,600	-2.004

TABLE A (cont'd.)

Time Interval	$D_2 = d_2 + .003\ v_2$	$C_2 = D_2 - D_3$	$F_2 = 4x10^6 C_2$	$Z_2 = F_1 - F_2$	$V_2 = v_2 + \frac{Z_2}{62,100}$
1	0	0	0	60,000	0.965
2	0.003	0.003	12,000	102,000	2.605
3	0.01081	0.0102	40,800	116,000	4.475
4	0.02423	0.02123	84,920	101,880	6.115
5	.04257	0.03357	134,280	70,720	7.255
6	.06434	0.0445	178,000	36,600	7.845
7	.08788	0.05208	208,320	10,080	8.007
8	.11190	0.0557	222,800	-3,600	7.949
9	.13575	0.05655	226,200	-11,000	7.772
10	.15905	0.05595	223,800	-16,600	7.505
11	.18157	0.05517	220,680	-23,080	7.134
12	.20297	0.05417	216,680	-27,880	6.684
13	0.22302	0.05262	210,480	-25,680	6.270
14	.24183	0.05053	202,120	-18,920	5.965
15	0.25972	0.04825	193,000	-11,400	5.781
16	0.27706	0.0465	186,000	- 9,200	5.633
17	0.29396	0.0459	183,600	-14,480	5.400
18	0.31016	0.0468	187,200	-26,880	4.967
19	0.32506	0.04948	197,920	-42,400	4.285
20	0.33792	0.05303	212,120	-50,680	3.469
21	0.34833	0.05667	226,680	-45,960	2.729
22	0.35652	0.05980	239,200	-28,400	2.272
23	0.36334	0.06244	249,760	-24,440	1.878
24	0.36897	0.06359	254,360	-20,620	1.546
25	0.37361	0.06275	251,000	- 9,100	1.400
26	0.37781	0.06046	241,840	+ 6,760	1.509
27	0.38234	0.05741	229,640	+22,600	1.873
28	0.38796	0.05635	225,400	+24,280	2.264
29	0.39475	0.05808	232,320	- 5,520	2.175
30	0.40127	0.06092	243,680	-46,400	1.429
31	0.40556	0.06208	248,320	-70,320	0.298
32	0.40645	0.05977	239,080	-60,040	-0.669
33	0.40444	0.05466	218,640	-22,320	-1.028
34	0.40136	0.04941	197,640	+17,000	-0.754
35	0.39910	0.04671	186,840	+31,240	-0.251
36	0.39835	0.04636	185,440	+15,520	-0.001
37	0.39835	0.04917	196,680	-26,520	-0.428
38	0.39707	0.0508	203,200	-60,160	-1.398
39	0.39288	0.04769	190,760	-57,160	-2.320

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE A (cont'd.)

Time Interval	$D_3 = d_3+0.003\ v_3$	$C_3 = D_3 - D_4$	$F_3 = 4x10^6 C_3$	$D_3 - Q$
1	0	0	0	
2	0	0	0	
3	0.0006	0.0006	2,400	-
4	0.0030	0.0029	11,600	-
5	0.0090	0.00823	32,920	-
6	0.0198	0.01693	67,720	-
7	0.0358	0.02809	112,360	-
8	0.0562	0.0396	158,400	-
9	0.0792	0.04883	195,320	-
10	0.1031	0.05424	216,960	0.0031
11	0.1264	0.05547	221,880	0.0264
12	0.1488	0.05402	216,080	0.0488
13	0.1704	0.05175	207,000	0.0704
14	0.1913	0.05003	200,120	0.0913
15	0.21147	0.04965	198,600	0.11147
16	0.23056	0.05095	203,800	0.13056
17	0.24800	0.0539	215,600	0.14800
18	0.26332	0.05791	231,640	0.16332
19	0.27558	0.06122	244,880	0.17558
20	0.28489	0.06289	251,560	0.18489
21	0.29166	0.06249	249,960	0.19166
22	0.29672	0.06151	246,040	0.19672
23	0.30090	0.05803	232,120	0.20090
24	0.30538	0.05557	222,280	0.20538
25	0.31086	0.05455	218,200	0.21086
26	0.31735	0.05181	207,240	0.21735
27	0.32493	0.05893	235,720	0.22493
28	0.33161	0.06166	246,640	0.23161
29	0.33667	0.06228	249,120	0.23667
30	0.34035	0.06096	243,840	0.24035
31	0.34348	0.05898	235,920	0.24348
32	0.34668	0.05751	230,040	0.24668
33	0.34978	0.05670	226,800	0.24978
34	0.35195	0.05571	222,840	0.25195
35	0.35239	0.05363	214,520	0.25239
36	0.35200	0.05139	205,560	0.25200
37	0.34918	0.04744	189,760	0.24918
38	0.34627	0.04419	156,760	0.24627
39	0.34519	0.04466	178,640	0.24519

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE A (cont'd.)

Time Interval	D_3'	$D_3 - D_3'$	$K_3'(D_3-D_3')$	$1 + J'v_3$	$R_3 = K_m'(D_3-D_3') \cdot (1 + J'v_3)$	$Z_3 = F_2-F_3-R_3$	$V_3 = v_3 + \frac{Z_3}{62,100}$
1					0	0	0
2	0	0	0	0	0	12,000	0.1931
3	0	0.0006	60	1.0193	60	38,340	0.8091
4	0	.0030	300	1.0809	324	72,996	1.985
5	0	0.0090	900	1.1985	1,079	100,281	3.601
6	0	0.01980	1,980	1.3601	2,690	107,590	5.336
7	0	0.03580	3,580	1.5336	5,490	90,470	6.795
8	0	0.0562	5,620	1.6795	9,430	54,970	7.680
9	0	0.0792	7,920	1.7680	14,000	16,880	7.952
10	0.0031	0.10	10,000	1.7952	17,952	-11,112	7.773
11	0.0264	0.10	10,000	1.773	17,730	-18,930	7.458
12	0.0488	0.10	10,000	1.7458	17,458	-16,858	7.186
13	0.0704	0.10	10,000	1.7186	17,186	-13,706	6.965
14	0.0913	0.10	10,000	1.6965	16,965	-14,965	6.724
15	0.11147	0.10	10,000	1.6724	16,724	-22,324	6.364
16	0.13056	0.10	10,000	1.6364	16,364	-34,164	5.814
17	0.14800	0.10	10,000	1.5814	15,814	-47,814	5.044
18	0.16332	0.10	10,000	1.5044	15,044	-59,484	4.087
19	0.17558	0.10	10,000	1.4087	14,087	-61,047	3.103
20	0.18489	0.10	10,000	1.3103	13,103	-52,543	2.257
21	0.19166	0.10	10,000	1.2257	12,257	-35,537	1.686
22	0.19672	0.10	10,000	1.1686	11,686	-18,526	1.394
23	0.20090	0.10	10,000	1.1394	11,394	+ 6,246	1.494
24	0.20538	0.10	10,000	1.1494	11,494	+20,586	1.826
25	0.21086	0.10	10,000	1.1826	11,826	+20,974	2.163
26	0.21735	0.10	10,000	1.2163	12,163	+22,437	2.525
27	0.22493	0.10	10,000	1.2525	12,525	-18,605	2.225
28	0.23161	0.10	10,000	1.2225	12,225	-33,465	1.686
29	0.23667	0.10	10,000	1.1686	11,686	-28,486	1.227
30	0.24035	0.10	10,000	1.1227	11,227	-11,387	1.044
31	0.24348	0.10	10,000	1.1044	11,044	+ 1,356	1.066
32	0.24668	0.10	10,000	1.1066	11,066	- 2,026	1.033
33	0.24978	0.10	10,000	1.1033	11,033	-19,193	0.724
34	0.25195	0.10	10,000	1.0724	10,724	-35,924	0.146
35	0.25239	0.10	10,000	1.0146	10,146	-37,826	-0.464
36	0.25239	0.09961	9,961	0.9536	9,490	-29,610	-0.941
37	0.25239	0.09679	9,679	0.9059	8,760	- 1,840	-0.971
38	0.25239	0.09388	9,388	0.9029	8,440	+38,000	-0.359
39	0.25239	0.09280	9,280	0.9641	8,942	+ 3,178	-0.308

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE A (cont'd.)

Time Interval	$D_4 = d_4 + 0.003\ v_4$	$C_4 = D_4 - D_5$	$F_4 = 4x10^6 C_4$	$D_4 - Q$
1				
2				
3	0	0	0	
4	0.00012	0.00012	480	-
5	0.00077	0.00075	3,000	-
6	0.00287	0.00269	10,760	-
7	0.00771	0.0069	27,600	-
8	.01660	0.01382	55,280	-
9	.03037	0.02329	93,160	-
10	0.04886	0.03376	135,040	-
11	0.07093	0.04313	172,520	-
12	0.09479	0.04978	199,120	-
13	0.11865	0.05365	214,600	0.01865
14	0.14127	0.05636	225,440	0.04127
15	0.16182	0.05975	239,000	0.06182
16	0.17961	0.06402	256,080	0.07961
17	0.19410	0.06663	266,520	0.09410
18	0.20541	0.06668	266,720	0.10541
19	0.21436	0.06477	259,080	0.11436
20	.22200	0.06219	248,760	0.12200
21	0.22917	0.06008	240,320	0.12917
22	0.23521	0.05518	220,720	0.13521
23	0.24287	0.05865	234,600	0.14287
24	0.24981	0.05475	219,000	0.14981
25	0.25631	0.05603	224,120	0.15631
26	0.26194	0.05696	227,840	0.16194
27	0.26600	0.05670	226,800	0.16600
28	0.26995	0.05627	225,080	0.16995
29	0.27439	0.05654	226,160	0.17439
30	0.27939	0.05745	229,800	0.17939
31	0.28450	0.05834	233,360	0.18450
32	0.28919	0.05854	234,160	0.18917
33	0.29308	0.05777	231,080	0.19308
34	0.29624	0.05626	225,040	0.19624
35	0.29876	0.05440	217,600	0.19876
36	0.30061	0.05243	209,720	0.20061
37	0.30174	0.04898	195,920	0.20174
38	0.30208	0.04641	185,640	0.20208
39	0.30053	0.04358	174,320	0.20053

TABLE A (cont'd.)

Time Interval	D_4'	$D_4 - D_4'$	$K_4'(D_4-D_4')$	$1 + J'v_4$	$R_4 = K_4'(D_4-D_4') \cdot (1 + J'v_4)$	$Z_4 = F_3-F_4-R_4$	$V_4 = v_4 + \frac{Z_4}{62,100}$
1							
2							
3						2,400	0.03865
4	0	0.00012	12	1.003865	12.05	11,108	0.21765
5	0	0.00077	77	1.0218	78.75	29,841	0.699
6	0	0.00287	287	1.0699	307	56,653	1.613
7	0	0.00771	771	1.1613	895	83,865	2.963
8	0	0.01660	1,660	1.2963	2,145	100,975	4.590
9	0	0.03037	3,037	1.459	4,420	97,740	6.163
10	0	0.04886	4,886	1.6163	7,900	74,020	7.356
11	0	0.07093	7,093	1.7356	12,300	37,060	7.953
12	0	0.09479	9,479	1.7953	17,000	-40	7.952
13	0.01865	0.10	10,000	1.7952	17,952	-25,550	7.541
14	0.04127	0.10	10,000	1.7541	17,541	-42,861	6.851
15	0.06182	0.10	10,000	1.6851	16,851	-57,251	5.929
16	0.07961	0.10	10,000	1.5929	15,929	-68,209	4.830
17	0.09410	0.10	10,000	1.4830	14,830	-65,750	3.770
18	0.10541	0.10	10,000	1.3770	13,770	-48,850	2.984
19	0.11436	0.10	10,000	1.2984	12,984	-27,184	2.547
20	0.12200	0.10	10,000	1.2547	12,547	- 9,747	2.390
21	0.12917	0.10	10,000	1.2390	12,390	- 2,750	2.346
22	0.13521	0.10	10,000	1.2346	12,346	+12,974	2.555
23	0.14287	0.10	10,000	1.2555	12,555	-15,035	2.313
24	0.14981	0.10	10,000	1.2313	12,313	- 9,033	2.1678
25	0.15631	0.10	10,000	1.21678	12,168	-18,088	1.8768
26	0.16194	0.10	10,000	1.18768	11,877	-32,477	1.3548
27	0.16600	0.10	10,000	1.13548	11,355	- 2,435	1.3156
28	0.16995	0.10	10,000	1.13156	11,316	+10,244	1.4806
29	0.17439	0.10	10,000	1.14806	11,481	+11,479	1.6656
30	0.17939	0.10	10,000	1.16656	11,666	+ 2,374	1.7038
31	0.18450	0.10	10,000	1.17038	11,704	- 9,144	1.5568
32	0.18917	0.10	10,000	1.15568	11,557	-15,677	1.3048
33	0.19308	0.10	10,000	1.13048	11,305	-15,585	1.0538
34	0.19624	0.10	10,000	1.10538	11,054	-13,254	0.8403
35	0.19876	0.10	10,000	1.08403	10,840	-13,920	0.6163
36	0.20061	0.10	10,000	1.0616	10,616	-14,776	0.378
37	0.20174	0.10	10,000	1.0378	10,378	-16,538	0.112
38	0.20208	0.10	10,000	1.0112	10,112	-38,992	- .515
39	0.20208	0.09845	9,845	0.9485	9,340	- 5,020	-0.596

TABLE A (cont'd.)

ime nter- val	$D_5 =$ $d_5 +$ $0.003\ v_5$	D_5-Q	D'_p	$D_5-D'_p$	(K'_5) $(D_5-D'_p)$	$1+J\ v_5$	$R_5 =$ $(K'_5)(D_5-D'_p)\cdot$ $(1+Jv_5)$	$Z_5 =$ F_4-R_5	$V_5 =$ $v_5 +$ $\frac{Z_5}{62,10}$
1									
2									
3									
4	0		0				0	480	0.007
5	0.00002	-	0	0.00002	40	1.0008	40.03	2,960	0.055
6	0.00019	-	0	0.00019	380	1.00544	382	10,378	.222
7	.00086	-	0	0.00086	1,720	1.0224	1,693	25,907	.639
8	.00278	-	0	0.00278	5,560	1.0639	5,920	49,360	1.434
9	.00708	-	0	0.00708	14,160	1.1434	16,190	76,970	2.674
10	.01510	-	0	0.01510	30,200	1.2674	38,250	96,790	4.234
11	.02780	-	0	0.02780	55,600	1.4234	79,100	93,420	5.737
12	.04501	-	0	0.04501	90,020	1.5737	141,700	57,420	6.662
13	.06500		0	0.06500	130,000	1.6662	216,200	- 1,600	6.636
14	.08491	-	0	0.08491	169,820	1.6636	282,300	-56,860	5.721
15	0.10207	0.0207	0.0207	0.10	200,000	1.5721	314,420	-75,420	4.508
16	0.11559	0.01559	0.01559	0.10	200,000	1.4508	290,160	-34,080	3.959
17	0.12747	0.02747	0.02747	0.10	200,000	1.3959	279,180	-12,660	3.755
18	0.13873	0.03873	0.03873	0.10	200,000	1.3755	275,100	- 8,380	3.620
19	.14959	0.04959	0.04959	0.10	200,000	1.3620	272,400	-13,320	3.406
20	.15981	0.05981	0.05981	0.10	200,000	1.3406	268,120	-19,360	3.094
21	0.16909	0.06909	0.06909	0.10	200,000	1.3094	261,880	-21,560	2.747
22	0.17733	0.07733	0.07733	0.10	200,000	1.2747	254,940	-34,220	2.296
23	.18422	0.08422	0.08422	0.10	200,000	1.2296	245,920	-11,320	2.114
24	0.19506	0.09506	0.09506	0.10	200,000	1.2114	242,280	-23,280	1.739
25	0.20028	0.10028	0.10028	0.10	200,000	1.1739	234,780	-10,660	1.567
26	0.20498	0.10498	0.10498	0.10	200,000	1.1567	231,340	- 3,500	1.511
27	0.20931	0.10931	0.10931	0.10	200,000	1.1511	230,220	- 3,420	1.456
28	0.21368	0.11368	0.11368	0.10	200,000	1.1456	229,120	- 4,040	1.391
29	0.21785	0.11785	0.11785	0.10	200,000	1.1391	227,820	- 1,660	1.364
30	0.22194	0.12194	0.12194	0.10	200,000	1.1364	227,280	+ 2,520	1.405
31	0.22616	0.12616	0.12616	0.10	200,000	1.1405	228,100	+ 5,260	1.490
32	0.23063	0.13063	0.13063	0.10	200,000	1.1490	229,800	+ 4,360	1.560
33	0.23531	0.13531	0.13531	0.10	200,000	1.1560	231,200	-120	1.558
34	0.23998	0.13998	0.13998	0.10	200,000	1.1558	231,160	- 6,120	1.460
35	0.24436	0.14436	0.14136	0.10	200,000	1.1460	229,200	-11,600	1.273
36	0.24818	0.14818	0.14818	0.10	200,000	1.1273	225,460	-15,740	1.527
37	0.25276	0.15276	0.15276	0.10	200,000	1.1527	230,540	-34,620	0.969
38	0.25567	0.15567	0.15567	0.10	200,000	1.0969	219,380	-33,740	0.425
39	0.25695	0.15695	0.15695	0.10	200,000	1.0425	208,500	-34,180	-0.125

It may also be noted that permanent ground displacement was not recorded until it exceeded the value assigned ground quake which was 0.10 in this problem. The time intervals at which these occurred were

for D_3' , time interval 10

D_4' , time interval 13

D_5' , time interval 15

The calculations were terminated when all the velocities became negative; effects after this are considered to be secondary and of little importance for the purposes of this study.

A graphical presentation of the results is shown in figures (14) through (16) inclusive.

The reason that a time interval of 0.00025 seconds was chosen may not be clear. Generally, it may be said that the best time interval to use is one that produces a completely stable calculation. This means that when the results are plotted there should be no sharp wiggles or peaks. If such irregularities are present, either an error has been made in the calculations or else the time interval chosen is too large. Once the unit lengths of the pile have been chosen, the time interval must be selected to correspond. Choosing too small a time interval would result in an excessive amount of calculation with only a very limited increase in accuracy.

Each spring in the figure (13) has a critical time interval which is the time that is required for a stress wave to traverse the spring and its associated weight. Stress waves travel in both directions

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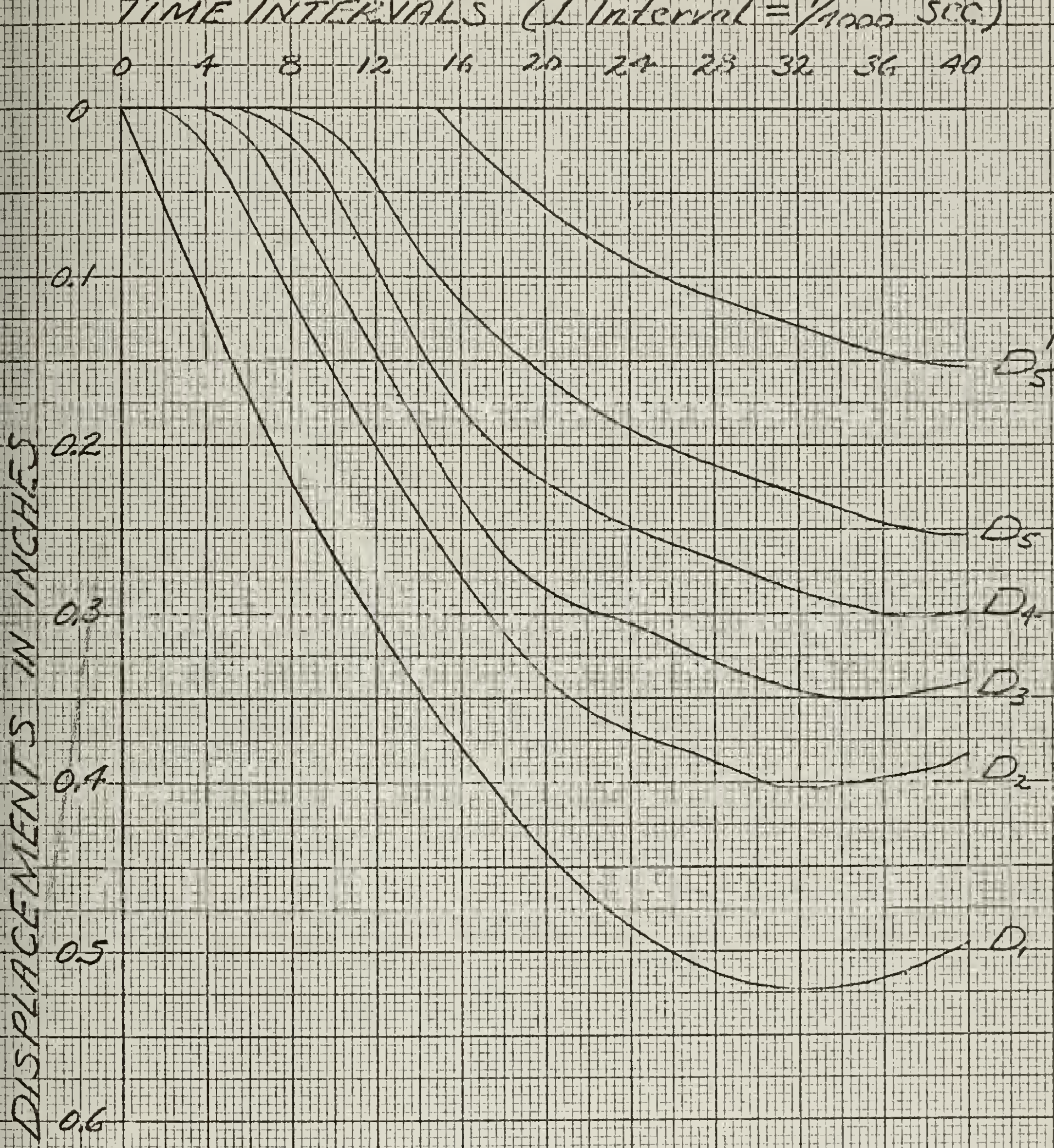
[illegible]

[illegible]

[illegible]

[illegible]

[illegible]



DISPLACEMENTS
vs
TIME
EXAMPLE PROBLEM
FIGURE (14)

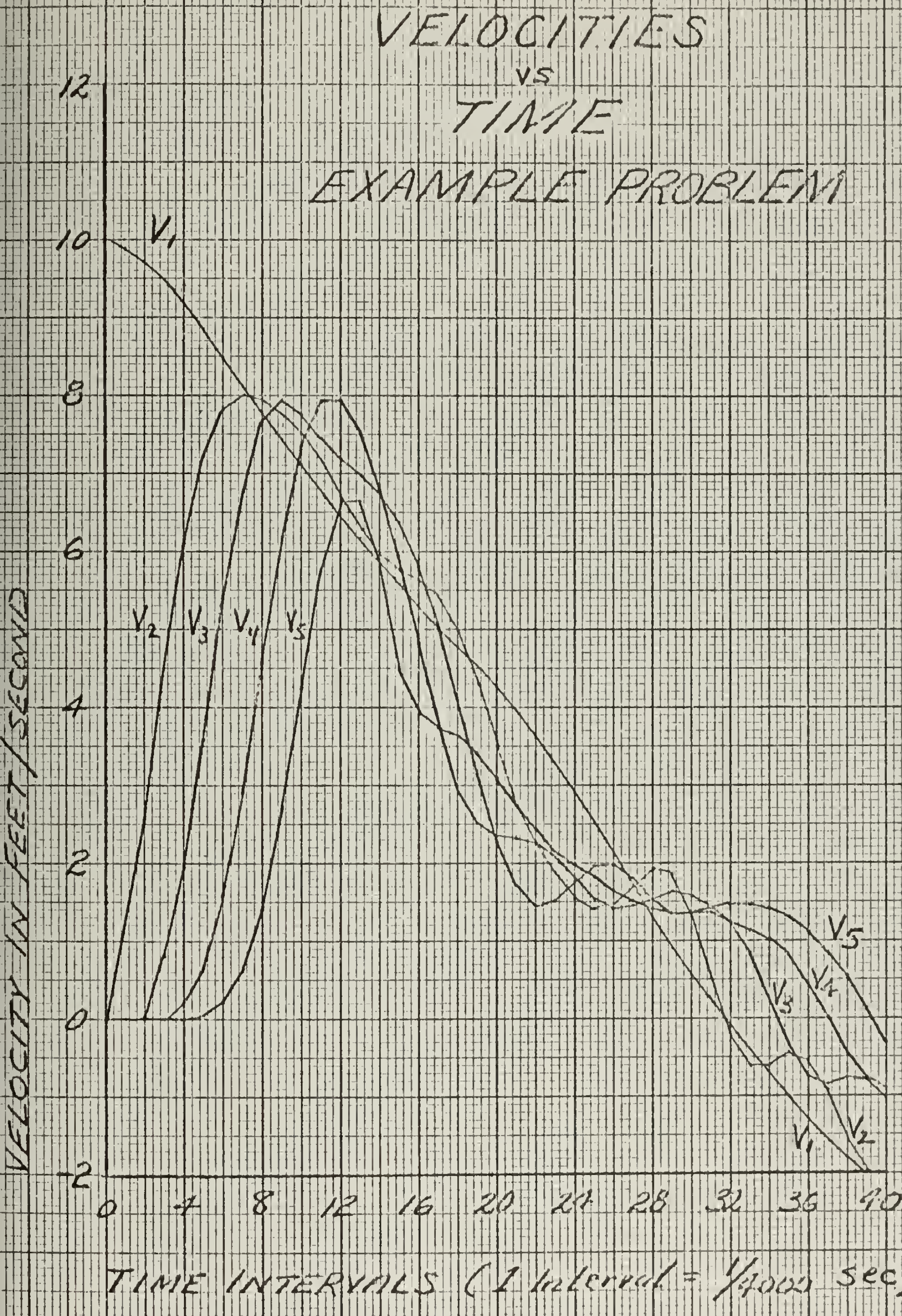

VELOCITIES
VS
TIME
EXAMPLE PROBLEM
VELOCITY IN FEET / SECOND
12
10
8
6
4
2
0
-2
V1
V2
V3
V4
V5
0
4
8
12
16
20
24
28
32
36
40
TIME INTERVALS (1 Interval = 1/4000 sec)

FIGURE (15)

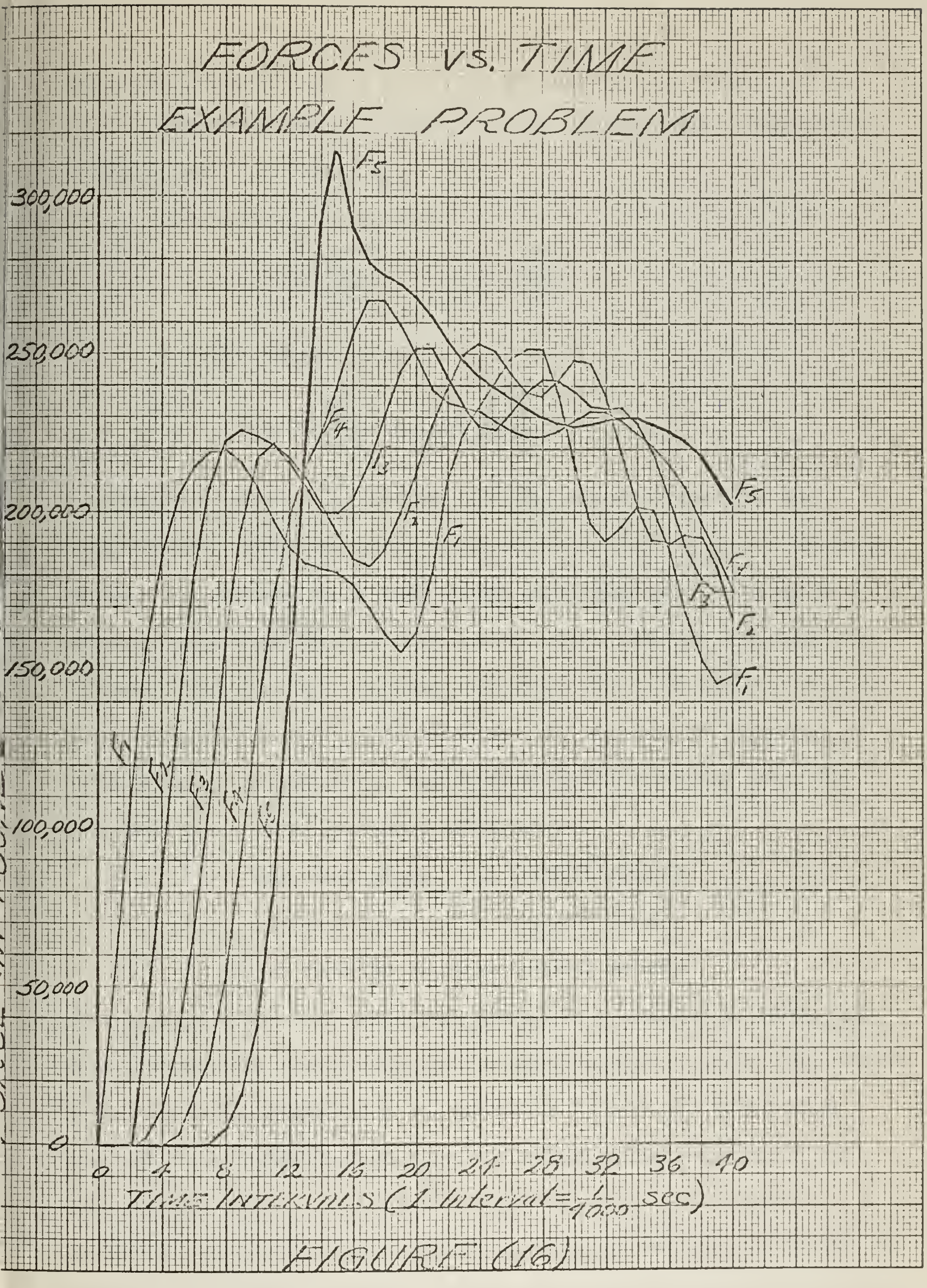

FORCES vs. TIME
EXAMPLE PROBLEM
300,000
250,000
200,000
150,000
100,000
50,000
0
0
4
8
12
16
20
24
28
32
36
40
TIME INTERVALS (1 Interval = 1/1000 sec)

FIGURE (16)

at a speed of $\sqrt{E/\rho}$ where ρ is the mass per unit of volume. The following equations may be derived for the critical time interval for spring K_m which is called T_m.

For wave motion downward:

$$T_m = \sqrt{\frac{W_{m+1}}{12gK_m}} = \frac{1}{19.648}\sqrt{\frac{W_{m+1}}{K_m}}$$

For wave motion upward:

$$T_m = \sqrt{\frac{W_m}{12gK_m}} = \frac{1}{19.648}\sqrt{\frac{W_m}{K_m}} .$$

The smallest value of T_m obtained by using these formulas is the critical one. About one half of the critical value should be selected as the time interval to be used in any particular problem so as to prevent instability from arising due to other factors not included in the above formulas such as quake, damping, and coefficients of restitution. When dividing the pile into ten-foot sections for purposes of mathematical representation, the recommendations of Smith (ref. 28) have been found to be generally satisfactory:

For steel piles -- 0.00025 seconds

For wood piles -- 0.00025 seconds

For concrete piles -- 0.00033 seconds

During some of the test cases run, instability resulted in using this time interval for wood piles, and recomputations were made using smaller time intervals. Instability can generally be detected by plotting the results as noted previously or by comparing the velocity of the pile cap and the pile tip with the velocity of the ram at impact. If either of these velocities exceeds twice the ram velocity at impact, instability is probably present. The velocity check should not be relied upon alone, however, to establish the stability of the calculations.

[illegible]

$$[illegible]$$

[illegible]

$$[illegible]$$

[illegible]

[illegible]

[illegible]

IV. COMPUTER ADAPTATION

A. General

As shown by the illustrative example, pile driving may be analyzed by the wave equation using a numerical solution method and a slide rule. This approach is a valid one and will produce satisfactory results. The main problem, of course, is the amount of time and work required to make the calculations. The example problem was selected as being about as simple as possible, with convenient figures to deal with numerically, and still illustrate all aspects of the calculations. Nevertheless, it required about twelve hours of manual computation to get the final set. It should also be noted that an error made at the beginning of a solution will carry throughout the calculations. The usual problem encountered in practice would probably take several man-days to complete.

Fortunately, there is a quicker and better way of making these calculations. Use may be made of high-speed digital computers which can solve up to 800 such practical problems in about 5 minutes, depending upon the computer used. As a consequence, use of the wave equation for pile driving analysis has become a practical matter.

Although it is not the purpose here to go into the details of programming, it may be well to mention some of the broader aspects for those totally unfamiliar with computers. For a computer to be effective in solving engineering-type problems, the problem-solving procedures must be presented to the computer in a language which it can "understand." The computer's basic language consists of elementary instructions such

[illegible]

[illegible]

[illegible]

[illegible]

as add or subtract. The problem-solving procedure must be translated, then, into instructions which the computer can obey. This translation is called programming and can be carried out entirely by a person, or the computer can assist in the process by use of a compiler.

A compiler is a large set of computer instructions which can accept a problem-solving procedure, written in a form resembling the procedure, and produce from it the proper elementary machine instructions that will solve the problem. The algebraic compiler system used for the programs here is called Fortran, which stands for formula translation. A procedure to be followed in solving a problem with Fortran is specified by a series of statements, of which there are several types. One type specifies arithmetic operations; another calls for reading a data card or printing results; a third specifies the sequence in which the statements are to be executed; and a fourth type is made up of statements which provide information about the procedure without themselves causing any action. All of these statements taken together form a source program. The programs included in the appendices of this writing are source programs. When punched on cards and put through the machine, it is translated by the Fortran system into simple sets of machine instructions which is called an object program. For those who wish to pursue the subject further, reference (21) provides in the space of 83 pages all the information necessary for a person to learn to develop programs of his own.

B. Description of Programs Developed

1. Basic Programs

A basic program in Fortran language for the IBM 7090 computer was prepared, and it follows the method of calculation illustrated in

the example problem. This program requires that input data be supplied to indicate the number and size of the weights in the mathematical model, the time increment to be used, the spring constants of the pile, the spring coefficients of the soil, the coefficients of restitution of the capblock and cushion block (if used), the velocity of the ram at the instant of impact, the value of ground quake, values of point and side damping, whether or not the pile cap can transmit tension, and whether or not side resistance is present. Output of the basic program includes the input data for purposes of identification and checking, and then for each time increment the force and resistance applied to each block of the model together with displacement and velocity of each block at that particular instant. The permanent set of the tip of the pile is also recorded. The program is arranged to stop the computer after 300 time-intervals or before if

(a) the velocity of the pile cap exceeds twice the velocity the ram had at the instant of impact;

(b) the velocity of the pile tip exceeds twice the velocity the ram had at the instant of impact;

(c) the velocities of all the blocks are simultaneously negative or equal to zero. Velocities toward the top of the pile are taken to be negative.

The basic program can be used to study stresses, velocities, accelerations, and permanent set of the pile as they vary with time. The program may be run with one or more sets of data at a time. This program in its entirety, together with instructions on how to use it, is included as appendix A.

Having developed this program, the question was how best to use it in trying to determine if correlations could be achieved with available field data on driving records, load tests, and borings. Of the variables that must be provided as input data, the greatest uncertainty was with the values to assign ground quake, point and side damping, and distribution of resistance along the sides of the pile.

2. Vary RU Program.

Correlation was approached in two different ways. The first method was particularly suited to the very few cases in which careful and complete field tests were reported and analyzed comprehensively. In these cases, reasonably accurate estimates of the variables except ground quake and damping factors could be made. Another program was developed for this approach which was basically the original program, but automatically for the one set of input data varies ground quake, ultimate ground resistance, and damping factors over a wide range of values. The output includes for each variation of basic information indicated the ultimate ground resistance, the blows per inch at final set, maximum tension and compression at the head, mid-length, and tip of the pile, the time interval the calculations stopped and why, and the original input data. This program in its entirety is included with instructions on how to use it as appendix B.

3. Researcher Program.

The second method of approaching correlation was more of a statistical approach, and it was designed to try and glean as much information as possible from published field data which were not as

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comprehensively reported as data used in the first method of approach. References (15) and (16) were used extensively in this attempt. Certain pile sections, lengths, and driving hammers were chosen because more reasonably good data were available for them than for other types. Another computer program was developed around the original program and designed to systematically produce in quantity data from which curves of ultimate ground resistance vs. number of blows per inch could be plotted for various values of damping constants and ground quake and side resistance. This program, together with instructions for its use, is included as appendix C. It is designed to be run with one or more sets of data. Its output includes for each of the above automatic variations of input, the ultimate ground resistance, values of soil spring coefficients, the permanent set, the blows per inch at the final set, maximum tension and compression at the head, mid-length, and tip of the pile, the time interval the calculations stopped and why, and the original input data. Thus about eighty complete problem solutions are obtained for each set of data run. Ordinarily three sets of data can be run at a time in this manner and take no more than five minutes' IBM 7090 computer time. Manual solution would require at least one man-year for just one set of data. Running time on a 709 computer would be 20 to 25 minutes, but only about a minute if the 7094 is used.

4. Validation of Computer Programs

Before using any computer program, great care must be taken to prove that it is correct and that all of its features behave as they are intended to. Making a manual solution is the only way to be certain that the program is correctly written. To prove the original, or

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[illegible]

[illegible]

[illegible]

basic, program, the data of the illustrative example presented herein was used as input, and the solution compared with the manual solution. A tabulation of the results of the two solutions for displacements of the ram in each time interval are found as Table B. It can be readily seen that the results compare as closely as one would expect considering that a slide rule was used in the manual computation. As a further check of the program, the illustrative problem presented by E. A. L. Smith (ref. 28) was computed using the basic program, and the results plotted. They compared in every detail with Smith's solution.

As additional programs were developed around this basic program, they were tested with both of these illustrative examples and verified to insure that subsequent work would be based on sound programs.

5. Computer Program for the Hiley Dynamic Formula

The Hiley type formula is thought by many to be one of the better dynamic formulas in general use today. Although not accepted by all, it is becoming more popular in the United States and has been included in some building codes. In order to facilitate comparison of results obtained by the wave equation and those obtained by using the Hiley formula, a computer program was developed for it also. This program requires that input data be provided to give the hammer data; pile material; the pattern, either rectangular or triangular, of side resistance distribution; coefficient of restitution; weight of the pile; area of the pile at the top, mid-length, and bottom; the ground quake; the modulus of elasticity of the pile material;

Table B

COMPARISON OF COMPUTER SOLUTION WITH MANUAL SOLUTION FOR ILLUSTRATIVE EXAMPLE

	Displacement of Block 1	
Time Interval	Computer Results	Manual Solution
1	0.030	0.030
2	0.060	0.060
3	0.089	0.089
4	0.117	0.118
5	0.145	0.145
6	0.171	0.172
7	0.197	0.197
8	0.221	0.221
9	0.245	0.245
10	0.267	0.267
11	0.288	0.288
12	0.309	0.309
13	0.328	0.328
14	0.347	0.347
15	0.364	0.365
16	0.381	0.381
17	0.397	0.397
18	0.412	0.412
19	0.426	0.426
20	0.440	0.440
21	0.453	0.453
22	0.465	0.465
23	0.476	0.476
24	0.485	0.485
25	0.494	0.495
26	0.502	0.502
27	0.508	0.508
28	0.513	0.514
29	0.517	0.518
30	0.520	0.520
31	0.522	0.522
32	0.523	0.523
33	0.523	0.524
34	0.522	0.523
35	0.522	0.521
36	0.517	0.518
37	0.513	0.514
38	0.508	0.509
39	0.502	0.504

END OF CALCULATIONS

and the pile length in feet. The program will automatically range over ultimate ground resistances from 20 to 380 tons in increments of 20 tons, writing out the set and blows per inch corresponding to each ground resistance as well as repeating the input data for each case. It computes for each value of the ground resistance the set associated with all the load carried by the point, one half of the load carried by the point, and none of the load carried by the point. Programming the Hiley formula may not be justified under usual circumstances, but it was done in this case as a convenience. The program and instructions for its use are included as Appendix D. It should be noted that the Hiley formula is discriminatory to long, heavy piles and is subject to the same general limitations as any dynamic formula.

V. SOIL ENGINEERING CONSIDERATIONS

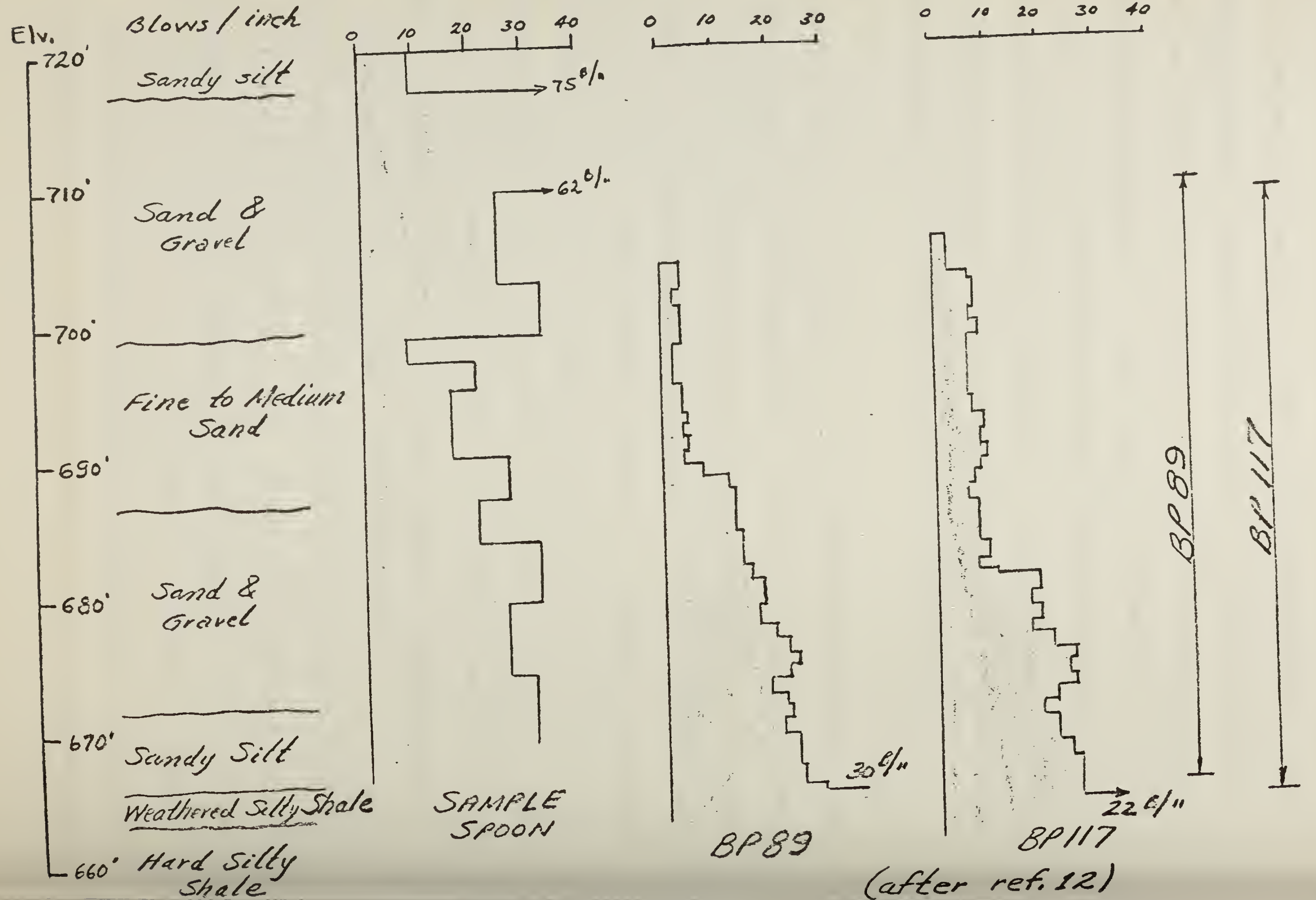
As has been mentioned in the development of this method of pile driving analysis, values must be assigned to the dynamic soil properties of ground quake and damping. The distribution of forces acting on the pile tip and sides must also be evaluated to complete the information required for a solution. Furthermore, in order to approach correlation between dynamic driving resistance and static resistance, the manner in which frictional resistance along the sides of the pile acts in both the static and the dynamic cases must be explored. The subject of load testing must also be investigated insofar as the way in which it is done and the interpretation of results affect correlations.

A. Side Friction Under Static Loading

1. Granular Materials.

Some recent work has been done on load transfer in the case of end bearing piles in which two instrumented H piles were tested (ref. 12). The piles were 14BP89 and 14BP117 and were driven approximately 40 feet through granular soil varying from sandy silt to sand and gravel. The piles were end bearing on weathered silty shale. In spite of the fact that the piles were short and relatively stiff, they transferred as much as one third of their load to the surrounding soil by friction. Figure 17 shows the soil conditions and driving records.

Upward shear along the pile develops as the pile shortens elastically under the applied static load at the top of the pile.



Elv.
720'
710'
700'
690'
680'
670'
660'
Blows / inch
Sandy silt
Sand & Gravel
Fine to Medium Sand
Sand & Gravel
Sandy Silt
Weathered Silty Shale
Hard Silty Shale
0 10 20 30 40
75 B/"
62 B/"
SAMPLE SPOON
0 10 20 30
30 B/"
BP89
0 10 20 30 40
22 B/"
BP117
BP89
BP117

(after ref. 12)

This movement of the pile causes distortion of the surrounding soil resulting in development of a frictional force resisting the displacement of the pile. Generally, it would be assumed that the greatest side resistance would be developed near the top of the pile and would diminish with depth. This action may be visualized by thinking of the pile as a spring embedded in the ground with a load applied at the top of the spring. One might also expect a nearly linear relationship between load applied and load transferred as long as no relative movement occurs between the soil and the pile. However, a loading is reached at which the soil begins to slip along the pile. This zone of shear failure begins at the top of the pile and progresses downward. If the pile tip never moved, the entire applied load would be carried by friction along the pile sides, but even in the case of progressive downward failure, the failure zone would never extend down to the tip. In the actual case the pile tip does move downward as the applied load is increased so that it accepts a portion of it.

The test results reported in reference 1 were presented from a viewpoint of over-all transfer of load to the soil by friction. Taking the data presented, and dividing each of the test piles into sections about 5 feet long, Table C has been prepared in an effort to show the variation in friction force on each section of the pile as the test load was increased. In each of the test piles, section 1 is the top section. A study of this table will show the validity of the description of the interaction between pile and soil, i.e., the frictional forces have a tendency to develop more at the top of

Table C

FRICTION FORCE ON SECTION
(IN TONS)

Increment of Total Applied Load	0-75 Tons		75-150 Tons		150-225 Tons		225-300 Tons	
	Change	Total (at 75)	Change	Total (at 150)	Change	Total (at 225)	Change	Total (at 300)
Section of 14BP89 Pile and Soil Description								
1--Sand & Gravel	8	8	2	10	5	15	4	19
2--Sand & Gravel	7	7	2	9	4	13	2	15
3--Fine to Med.Sand	6	6	1	7	-2	5	3	8
4--Fine to Med.Sand	0	0	3	3	4	7	1	8
5--Sand & Gravel	5	5	2	7	-3	4	4	8
6--Sand & Gravel	4	4	0	4	1	5	8	13
7--Sand & Gravel	3	3	-1	2	5	7	5	12
8--Sandy Silt, tip in weathered shale	1	1	5	6	0	6	4	10
TOTAL FRICTION FORCE		34		48		62		93
TOTAL FORCE ON TIP		41		102		163		207
Section of 14BP117 Pile and Soil Description								
1--Sand & Gravel	4	4	-1	3	-3	0	0	0
2--Sand & Gravel	4	4	1	5	-3	2	0	2
3--Fine to Med.Sand	0	0	4	4	-4	0	2	2
4--Fine to Med.Sand	2	2	3	5	-5	0	1	1
5--Sand & Gravel	3	3	1	4	13	17	11.5	28.5
6--Sand & Gravel	3	3	1	4	4	8	-1.5	6.5
7--Sand & Gravel	6.5	6.5	3.5	10	-1	9	5	14
8--Sandy Silt, tip in weathered shale	0.5	0.5	1.5	2	4	6	-4	2
TOTAL FRICTION FORCE		23		37		42		54
TOTAL FORCE ON TIP		52		113		183		246

the pile. It also shows that local failure extends over a wide range of total applied load. For example, as the applied load is increased from 75 to 150 tons, the friction forces on section 7 of the 14 BP 89 is decreased by one ton; from 150 to 225 tons' applied load, local failure occurs at sections 3 and 5. After this loading the granular structure of the soil seems to have been readjusted by the shearing forces and displacements so as to have an increased load-carrying capacity.

In the case of the 14 BP 117 pile, as the applied load is increased from 75 to 150 tons, local failure has taken place on section 1 only. As the applied load is increased to 225 tons, failure continues on section 1 and is expanded to include sections 2, 3, 4, and 7. Upon further increase of the applied load to 300 tons, it is evident that failure has generally progressed down to and including section 4 even though there is a very slight increase in friction force on the top half of the pile. Sections 6 and 8 are experiencing failure while sections 5 and 7 are attempting to assume the load. Even at this depth it is interesting to note that section 5, the uppermost of these two sections, has assumed an appreciably higher part of the load.

Both of these piles were designed for 150-ton static loads. When test-loaded to 150 tons, it is noted that the general shape of side frictional resistance was triangular with the base of the triangle at the top of the pile and the apex toward the lower sections. In the case of the 14 BP 117 at a test load of 150 tons, however, the general shape of the frictional resistance acting along the length of the pile was generally of a rectangular shape down almost to the pile tip. Thee difference may be explained by considering the loads required for a

unit axial shortening (spring constant) of each of the pile shapes. From the expression $K = \frac{AE}{L}$, and the fact that the length and modulus of elasticity of each pile is the same, their spring constants' ratio will vary as their area ratio or 34.44/26.19. It can be seen that the 14 BP 117 is 1.32 times as "stiff" as the 14 BP 89. Since the over-all dimensions of the piles, the applied load, and the surrounding soil are essentially the same, this relative "stiffness" must account for the difference found in the pattern of frictional resistances developed. As a consequence of this, it is interesting to note that the 14 BP 117 was nearer total failure at a test load of 300 tons than was the lighter 14 BP 89 pile.

In summary, the following factors may be important in considering the static behavior of piles in granular material:

a. The time which has elapsed since driving. In some cases a quick condition may have been caused during driving which will disappear as pore pressures dissipate. Relaxation may also occur.

b. The general manner in which load is transferred to the soil may vary considerably and is generally dependent upon the "stiffness" of the pile in resisting axial deformations as related to the strength characteristics of the soil. The less "stiff" piles tend to give up their loads to the soil at the upper strata while the very "stiff" piles tend to carry their load by end bearing.

c. Failure of piles in granular materials is often not too well defined in the case of friction piles. Small local failures along the pile may result in a decrease in load carrying capacity, but then the soil particles in undergoing shear may assume different packings and

become denser with the result that their strength has been restored or even increased over its original value. The pile load may be increased while these readjustments are taking place until a load is finally applied which is sufficient to push the pile through the soil continuously. With each of these readjustments the soil takes on different properties causing the pile to undergo variations in its bearing capacities.

d. Unusual conditions which may be pertinent to a particular problem such as a quick condition or hydrostatic uplift.

2. Cohesive Materials.

As in the case of piles in granular material, some research work has also been done with instrumented piles in cohesive materials. H. B. Seed and L. C. Reese report in reference (2) that several 6-inch-diameter pipe piles from 20 to 22 feet long were driven approximately 15 feet into a stratum of soft, saturated clay. Electric strain gages were installed on one of these piles. Test loadings to failure were made at various intervals after driving as follows:

Interval After Driving	Failure Load (pounds)
3 hours	1070
1 day	2800
3 days	3200
7 days	5400
14 days	5800
23 days	6100
33 days	6200

This data show that the ultimate bearing capacity increased very rapidly with time for the first week, but that thereafter the increase was very gradual.

Most of the load was removed from the pile by side friction with only about 10-15% of the applied load reaching the pile tip. The increase in supporting capacity was reflected by an increase in the load transfer to soil at all depths as shown by Table D which was derived from published data (ref. 26). The pile was divided into three-foot sections with section 1 being at the top of the pile. From this table it can be seen that the resistance is generally increasing with depth for the first 9 to 12 feet of embedment, and then it tends to become constant with depth for the final 3 to 6 feet of embedment. It is interesting to contrast this behavior with that already described for granular materials.

In this series of tests relationships were established for undisturbed soil and fully remolded soil. Tests of soil samples taken next to the pipe wall showed that the strength loss caused by disturbance of driving the pile was about 60% to 70% of that which the soil would have lost due to complete remolding. The clay next to the pile wall subsequently gained strength, and at the end of the test period it had a strength of 60% higher than the strength of the undisturbed soil. This increase in soil strength appeared to parallel the decrease in excess hydrostatic pressure indicating a relationship between the two. For all values of time at all depths the failure of the friction pile did not occur in the soil, but at the interface between pile and soil.

In another series of tests made by Vey (ref. 35), with H piles, he found that in some cases the friction force reversed itself when the applied load was removed and held the stress in the pile. He also

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[illegible]

[illegible]

Table D

FRICTION FORCE ON SECTION
(IN POUNDS)

Test 7--33 Days After Driving

Increment of Total Applied Load / Section of 6" Pipe Pile and Soil Description	0-2000 lbs. Change	0-2000 lbs. Total (for 2000)	2000-4000 lbs. Change	2000-4000 lbs. Total (for 4000)	4000-6000 lbs. Change	4000-6000 lbs. Total (for 6000)
1--Soft sat. clay	110	110	0	110	50	160
2--Soft sat. clay	290	290	270	560	150	710
3--Soft sat. clay	480	480	400	880	420	1300
4--Soft sat. clay	480	480	520	1000	500	1500
5--Soft sat. clay	475	475	525	1000	560	1560
TOTAL FRICTION FORCE		1835		3550		5230
TOTAL FORCE ON TIP		165		450		770

First Loading to Failure (3 hrs. after Driving)	0-1070 lbs. Change	0-1070 lbs. Total (for 1070)
1--Same soil	0	0
2--	200	200
3--	0	0
4--	330	330
5--	280	280
TOTAL FRICTION FORCE		810
TOTAL FORCE ON TIP		260

found that adherence would increase between the soil and the pile with time, and reconsolidation of the disturbed clay would increase its shear strength.

The findings of these two series of tests regarding the effect of pile driving on clay cannot, of course, be assumed to be valid under all conditions. Tschebotarioff has shown in reference (34) that the remolding effect caused by driving pipe piles closed end in 100 feet of varved clay can be great. Remolded clay layers can lead to the development of negative friction with the result that end bearing piles may become overloaded. It is generally understood that displacement piles should not be driven in highly sensitive clays; therefore this part of the problem will not be discussed further.

In summary, it may be said that the static behavior of piles driven in cohesive material may depend upon:

a. The time which has elapsed since driving. In some cases the increase in bearing capacity for even short periods of time may be great; in other cases a loss of bearing capacity may result.

b. The friction force acting on the pile tends to increase with depth. It is believed that the load transfer would decrease with depth if the shear strength of the clay were sufficient to accept the loads at the upper strata and if there were a motivating force available to cause these upper strata to come into intimate contact with the pile. Since neither of these conditions usually exists, the load transfer is found to generally increase with depth.

c. Failure of piles in clay may be more dramatic than those in granular materials. In the case of a pile not resting on an extremely

hard strata such as rock, once the shear strength or adhesion has been exceeded, the pile may begin to punch its way through the supporting strata and continue downward until the load is reduced or the pile encounters a firmer strata.

B. Dynamic Factors Involved in the Pile-Soil Interaction

The main obstacle in attempting to relate the penetration record obtained during driving of a pile to the static load carrying capacity is that of obtaining a reasonably accurate estimate of the dynamic resistance to penetration between the pile and soil. The problem is the same whether using the wave equation or a dynamic pile driving formula.

The dynamic resistance to penetration is related to one or more of the following factors:

a. The static resistance.

b. The increase in strength observed to occur in the soil due to rapid loading.

c. Frictional resistance between the pile and soil.

d. Viscous resistance developed as the pile penetrates the soil.

The basic strength of the soil, whether due to cohesion or internal friction, constitutes the static load capacity of the pile and soil. At least, this represents the limiting value since the pile may not develop the full shear strength of the soil when acting as a friction pile. The value of the static resistance can be thought of as the base value of the dynamic resistance in the idealized case

where the strength properties of the soil are not changed by the process of pile driving. This is rarely the case, and usually the driving of the pile acts to decrease or increase the natural strength of the soil.

Several investigators have studied the effect of rapid loading times on the compressive strength of the soil (ref. 4, 11, 25, 37), and some discussion of the results of such investigations appears below. It seems logical to assume that in the early time intervals of pile penetration most of the dynamic resistance is a result of this type of phenomenon, the magnitude of which would be directly related to the static strength of the soil.

Frictional resistance in nearly all cases develops between the pile sides and soil as a result of the penetration of the pile. The nature and magnitude of this frictional resistance depends upon the type of soil, moisture content, type of pile and pile material. If the adhesion or friction acting between pile and soil is greater than the shear strength of the soil, the frictional resistance may be developed within the soil itself. The dynamic frictional resistance would be of about the same value as the static resistance since the dynamic coefficient of friction between the pile and soil materials is about equal to the static coefficient of friction (ref. 24). Friction as a dynamic factor will be neglected since there is some doubt as to the extent that actual relative movement takes place between the pile and soil at the pile-soil interface.

No published work has been found that deals with viscous forces as developed by penetration into soils. One investigator

attributes part of the influence of high rates of strain on the increase in compressive strength of soil to viscous action (ref. 37). Since viscosity is defined as a property of internal friction (ref. 2), it will be considered as included as one of the factors contributing to the increase of compressive strength experienced by a soil subjected to high rates of strain.

During the driving of a pile, it is assumed that the resistance developed at the tip and along the sides can be expressed in terms of the resistance developed under static loading. This is to say that the dynamic resistance to driving is composed of the static resistance plus an increment of resistance that develops under dynamic loading and is expressed as a percentage of the static value. The problem is to determine what form the relationship between static and dynamic resistance takes and how to quantitize it.

Another problem, which will be taken up later, is how much of the static resistance is present during driving. It cannot be generally assumed that all of the static resistance is present during driving, and Chellis points out ways in which this may vary (ref. 6). On the other hand, dynamic resistance may develop in soil strata that offer only very small or negligible static resistance.

In the computer solution, the total resistance to driving at the pile point is represented by the expression:

$$R = (D_m - D_m') K' (1 + Jv) \qquad (15)$$

where $(D_m - D_m')$ is the "elastic" strain of the soil with a limiting value of Q,

K' is the spring constant of the soil,

v is the pile tip velocity at any instant, and

J is the damping constant.

This relationship is shown by Smith (ref. 28).

Two questions arise with respect to this mathematical representation of the dynamic resistance to driving:

(a) Does this equation approximate the actual form of the pile resistance; i.e., does the resistance-time relationship approximate that of the actual case?

(b) What values of J should be assigned for different soil and driving conditions?

Smith proposed the use of a value of 0.15 for J; however, since the purpose of his published work did not include the correlation of results with load tests, he gave no basis for the use of this value. Some preliminary attempts to correlate wave equation results with the results of pile load tests and driving records using values of 0.15 and 0.05 for J and J' indicated that this value for J was not appropriate for clay soils.

In an attempt to answer the questions posed above, a search of the literature was made for any experimental work that might lead to a confirmation of the nature of the dynamic resistance as well as to a determination of more appropriate values for the coefficients J and J'.

Several research programs over the past 15 years have investigated the influence of time of loading, or strain rate, on the compressive strength of soils. All such projects have used rapid loading in triaxial and unconfined compression tests, or dynamically loaded footing models as the method of investigation. The application of results from

such tests to pile driving must be approached with caution and should be considered only as indicative of the general nature of the dynamic resistance to pile driving and of the general order of magnitude for values of the coefficients of damping.

Before describing the tests made by individual investigators, several terms will be defined:

(1) Time of loading: The time from the beginning of a test to that at which the maximum force is achieved. Figure (21) shows this quantity graphically.

(2) Strain rate: The rate, expressed in per cent per second, of strain imposed on the sample at the time the compressive strength is reached.

(3) Transient load: A dynamically applied load in an unconfined compression or triaxial test that reaches a maximum in times of less than 30 seconds and as short as 0.005 second.

(4) Static test: For comparative purposes, a test that is made with a time of loading of ten minutes or more, unless otherwise indicated.

(5) Strength ratio: The ratio of the compressive strength for a given time of loading to the strength for a static test.

(6) Strain-rate effect: Same as strength ratio.

2. Work at Harvard

In a test program sponsored by the Panama Canal in 1948, Casagrande and Shannon made a series of static and transient unconfined and triaxial tests on soft to medium clays and a soft rock (ref. 4). Triaxial tests of the vacuum type were also made on a dry sand. Times

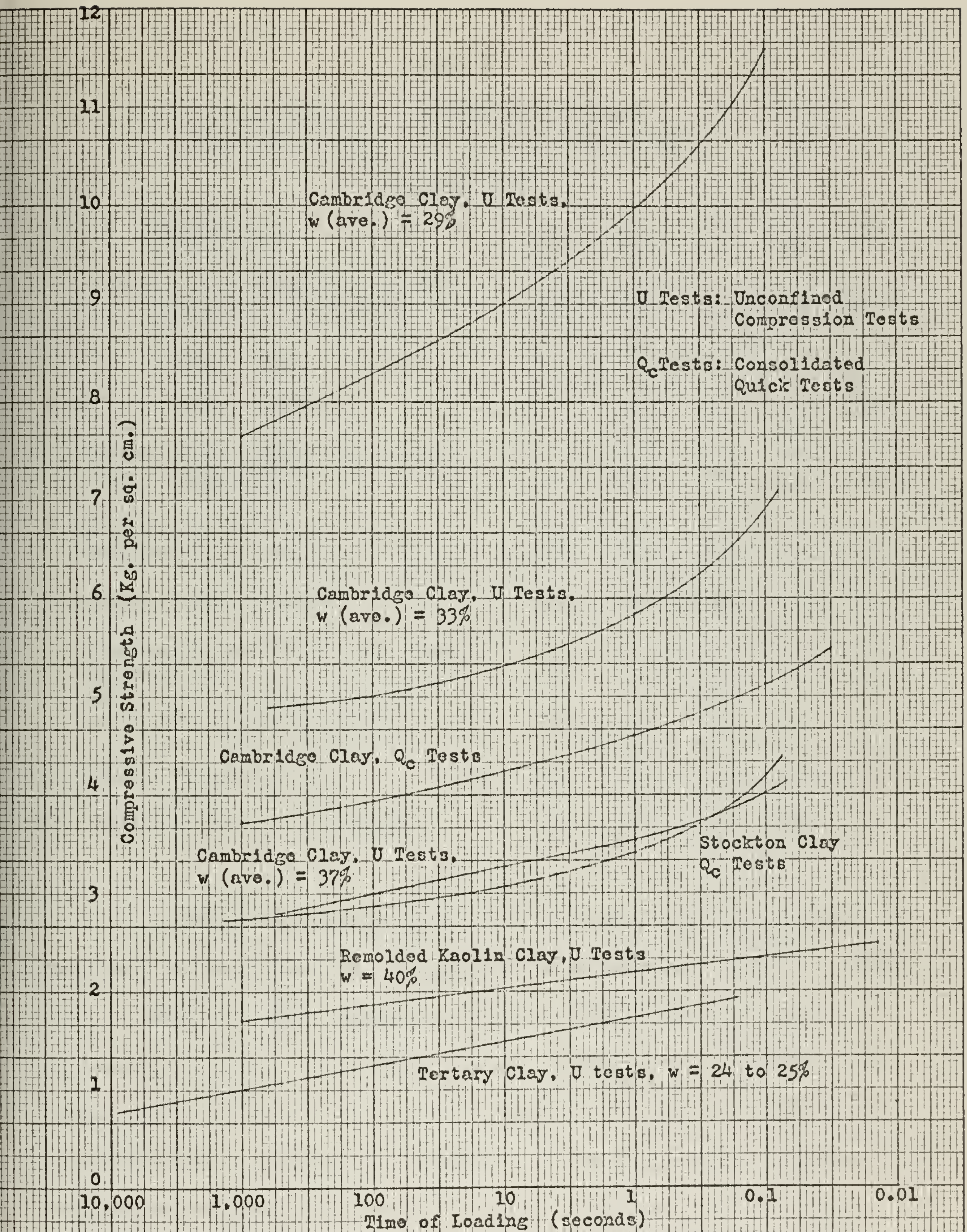
of loading for the transient tests ranged from 0.01 seconds to 30 seconds.

In all the tests on clays, the transient compressive strength was greater than the compressive strength as determined by static tests. The transient strengths, taken at a time of loading of 0.02 second, were from 1.4 to 2.6 times the static compressive strength. The tests also showed that the increase of the fast transient compressive strength over the static strength is greatest for specimens at the highest water content and least for the specimens at lowest water content. (Figure (17-A) is a summary of the average results from the clay tests.

Vacuum-type triaxial tests were performed on a clean, medium sand with a void ratio of 0.62 and at a minor principal stress of 0.3 kg/cm^2. The test results indicated that the compressive strength of a dry sand in fast transient tests is about 15% greater than the static compressive strength.

3. Work of Whitman

Whitman measured the increase in compressive strengths of soils using a triaxial machine, and testing at high rates of strain (ref. 37). Table E shows his results for several types of cohesive soils. The last column of this table (S-R) is the ratio of the failure load at a strain rate of 1000 per cent per second to the failure load at a strain rate of 0.03 per cent per second. It is observed that the strain-rate effect varies from 1.3 to 2.0 for these soils. In his tests, Whitman achieved times of loading as short as .005 second. Confining pressures up to 100 p.s.i. were used.



Effect of Time of Loading on the Compressive Strengths of some Clay Soils

Figure (17-A) After ref. 4

Table E

INCREASE OF COMPRESSIVE STRENGTH
IN COHESIVE SOILS DUE TO HIGH STRAIN RATES

(from ref. 37)

DESCRIPTION	P.I.	P.L.	w	qu	S-R
1. Normally consolidated, sensitive ocean sediment, undisturbed	63	49	92	0.3	2.0
2. Very plastic clay, remolded	27	38	48	7	1.6
3. Plastic clay, remolded	17	11	16	10	1.7
4. Medium soft, slightly sensitive clay, undisturbed	24	26	27	10	2.0
5. Slightly plastic clay loam, remolded	23	22	21	13	1.4
6. Plastic clay, remolded	27	38	44	15	1.7
7. Moderately sensitive silty clay, undisturbed	21	22	35	22	1.6
8. Impervious compacted fill	17	11	12	25	1.8
9. Tough compacted fill	41	21	26	35	2.0
10. Stiff, dry clay, undisturbed	23	30	20	250	1.3

In comparing results of tests on clay with and without confining pressure, Whitman observed two different time effects. The first effect can be likened to that encountered in an extremely viscous fluid, and is associated with a continuous, plastic deformation of the soil. The second effect is associated with the formation of discontinuities such as shear planes and splits, and occurs when the soil sample is tested to failure without (or small values of) confining pressure. From these results, he concluded that when cohesive samples were subjected to a

lateral pressure, the strain-rate effect was the result of viscous phenomena, and was the same regardless of the magnitude of strain. The same effect occurred in very plastic soils regardless of confining pressure.

In dry or moist sands an increase in compressive strength of 10-15 per cent was observed. Only limited tests were conducted with saturated sands, and figure (18) shows results for saturated Ottawa sand and a well graded sand. Whitman observed that the strain-rate effect in saturated sand must result from differences between the pore water pressures in slow and rapid tests. When sand is tested at constant volume, pore water migrates to the central part of the sample. As speeds of deformation increase, more energy must be expended to overcome resistance to the flow of water.

4. Work of Seed and Lundgren

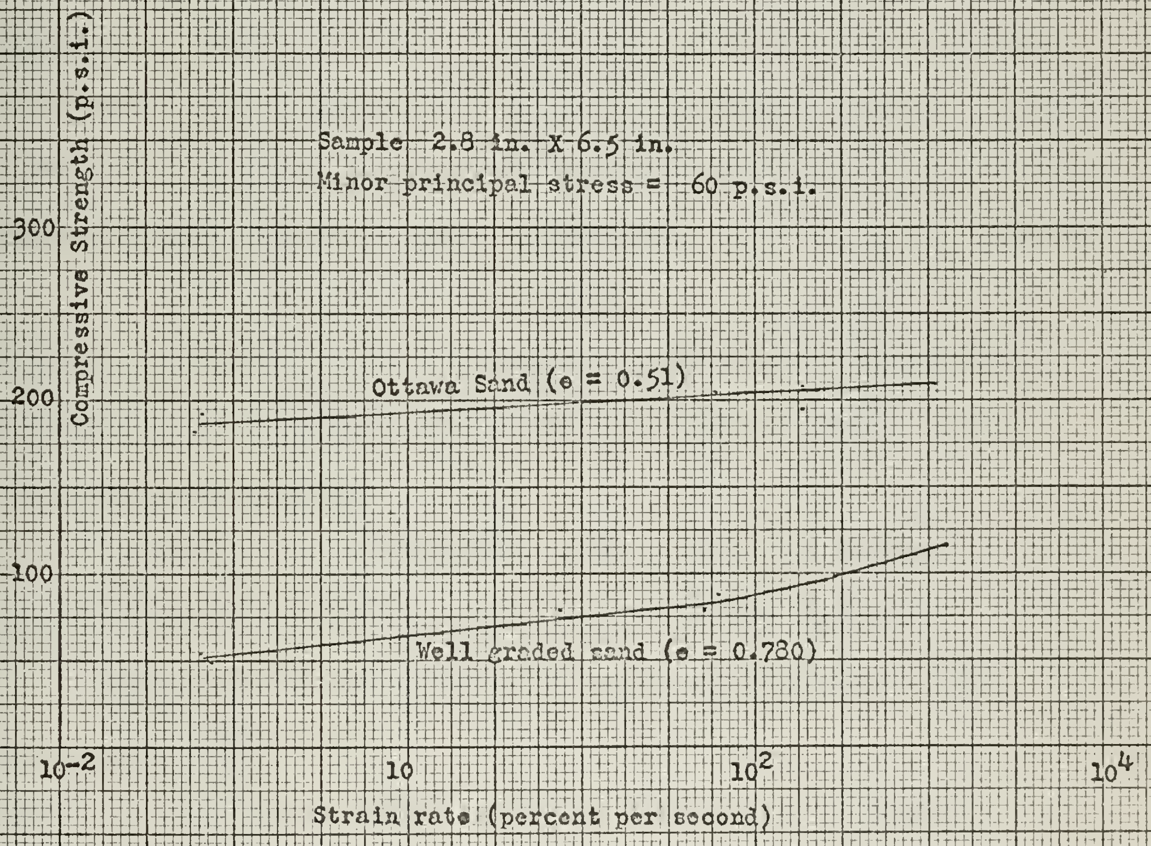
Seed and Lundgren investigated the effect of transient loadings on the strength and deformation characteristics of saturated sands (ref. 25). Their tests were also made with the triaxial apparatus, and three different categories of tests were conducted:

a. Static tests in which the loading times were about 10 minutes;

b. Slow transient tests with loading times of about 4 seconds;

c. Rapid transient tests with loading times of about .02 second.

In the rapid transient tests a constant rate of deformation of 40 inches per second was used. All tests were conducted with a confining pressure of 2 kg/sq.cm., and the samples were tested at various void ratios.



Relationship Between Compressive Strength and Strain Rate For Saturated Sand.

Figure (18) After ref.37

Results of the tests on dense, fine sand indicated that the compressive strength of a sample subjected to rapid transient loading was about 40 per cent greater than that obtained by static drained tests. Due to the high rate of loading the rapid tests were actually undrained tests, since drainage could not occur within the very short time of the test. About half of the increase in strength is attributed to a dilatancy effect and half to the high rate of loading alone.

It is a well known fact that when a dense sand is sheared it increases in volume. If shear occurs under conditions that prevent the movement of pore water, such as in the undrained triaxial test or in very rapid loading, the increase in volume will result in a decrease in pore pressure and an increase in strength, as shown by the equation

$$s = (\sigma - u)\tan\phi$$

in which the value of u (pore pressure) is decreased. These results are summarized in figure (19) and apply to fine and coarse sands for very short loading times.

If the results of the tests made by Seed and Lundgren are applied in equation (15) the following results are obtained:

Dynamic Resistance (R_D) = 1.40 x Static Resistance (R_S)

or, $R_D = R_S(1 + JV)$

where J is the damping factor and V the velocity of deformation.

$$1.40\, R_S = R_S(1 + JV)$$

$$J = \frac{1.40-1.00}{V}$$

From figure (20), the load curve from one test, the velocity of deformation is estimated to be about 35 inches per second, and

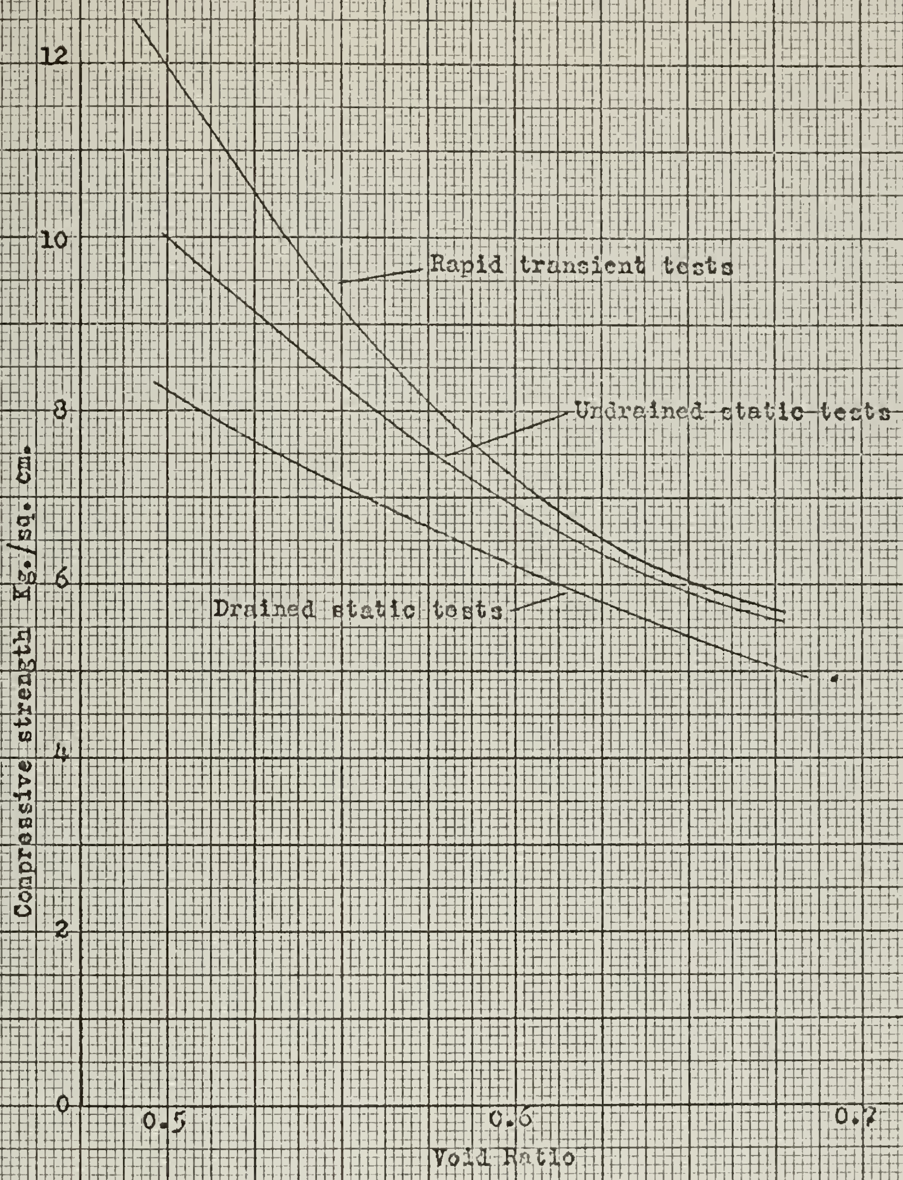
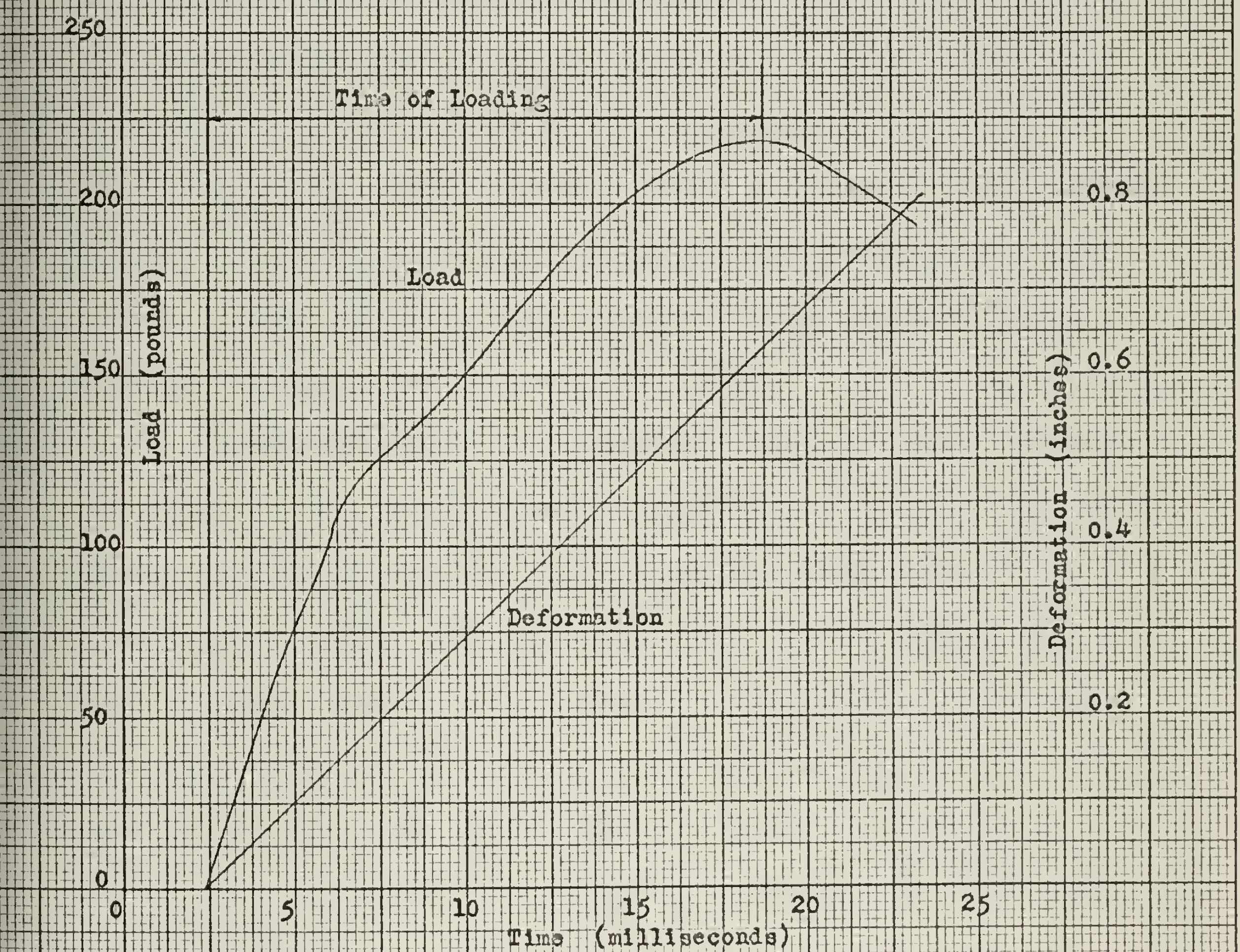


Figure (19)

Effect of transient loading and dilatancy on compressive strength at various void ratios. (From Ref. 25)



Deformation and Load vs. Time for tests in Saturated Sand

Figure (20) After ref. 25

substituting this value in the last equation gives

$$J = .133$$

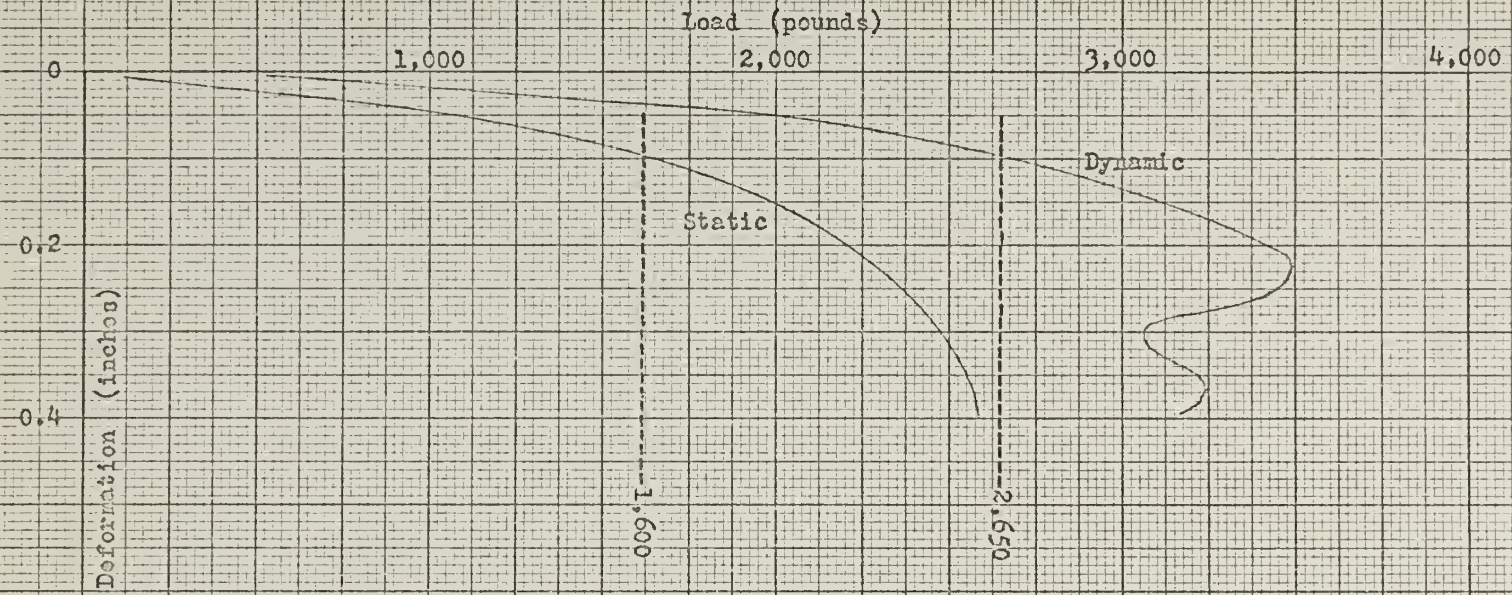
for the case of rapid loading in saturated sand. This value agrees closely with the value of 0.15 for J suggested by Smith (ref. 28).

5. Work of Cunny and Sloan

Some preliminary tests were conducted by the Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi in which small footings were dynamically loaded in a test tank filled with clay or dry sand (ref. 11). In these tests a loading machine capable of applying a load in a time of from 3 to 150 milliseconds was programmed to apply a preset load during a predetermined loading time. As a control, static tests were also performed on the same soil under identical conditions except for loading time.

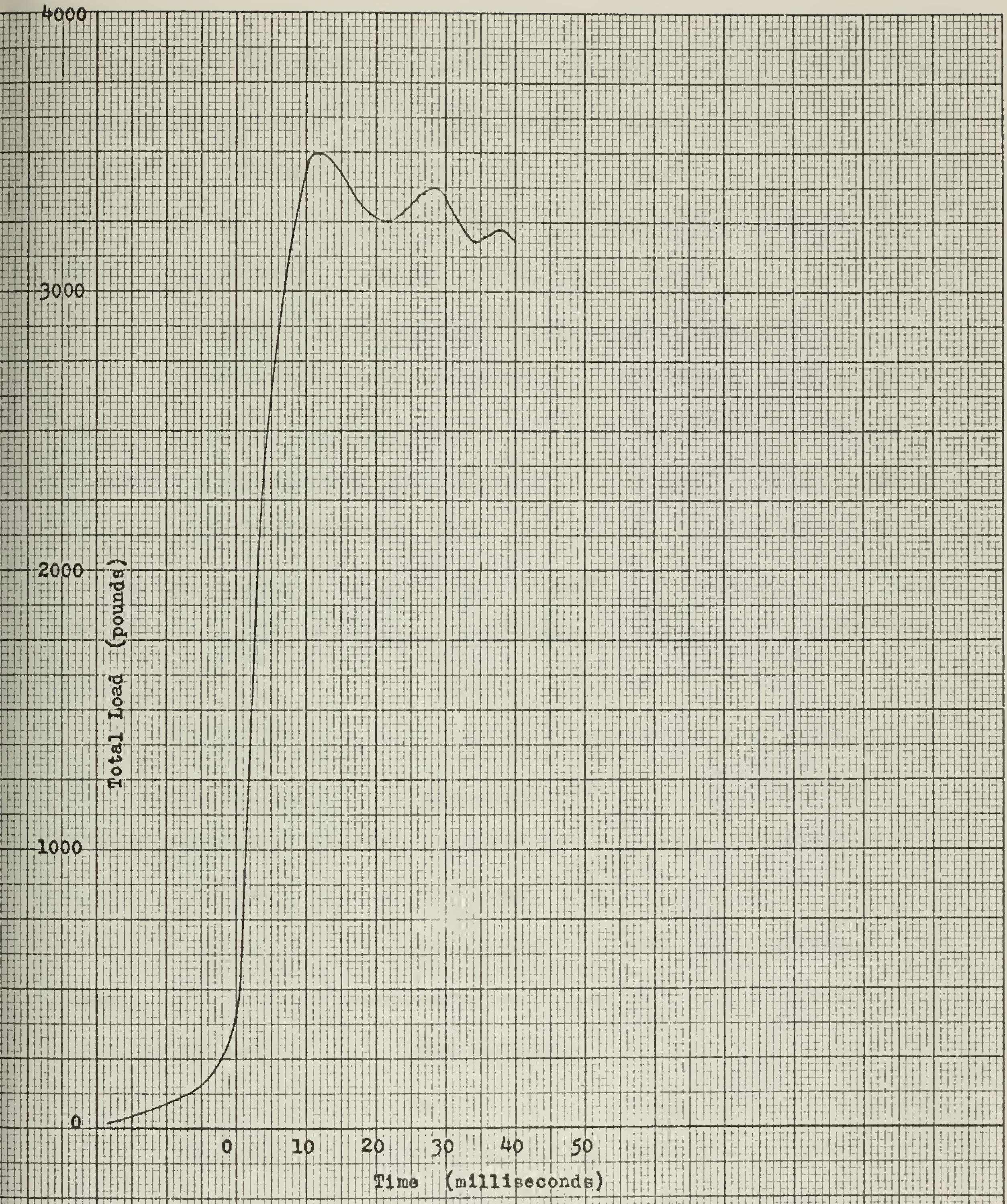
Dynamic and static tests were conducted on a sand compacted to a relative density of 96 per cent. Figure (21) shows the relationship between load and deformation for the static and one dynamic test on a 9-inch-square footing. The load time curve for the dynamic test is shown in figure (22). From this curve it is seen that the loading time was 11 milliseconds and the maximum load reached was 3500 pounds. Since the maximum value of the load was limited by the setting of the machine, it is not necessarily the full strength of the soil under dynamic loading.

By relating the values of static and dynamic loading at equal deformations, it is possible to arrive at an indication of the value of the damping factor under these conditions of loading. At a deformation



Load - Deformation Relationship for Static and Rapid Loading Tests on 9 inch square Footing Model.

Figure (21) After ref. 11



Load vs. Time for Rapid Loading Test on a 9 inch square Model Footing in Sand.

Figure (22) After ref. 11

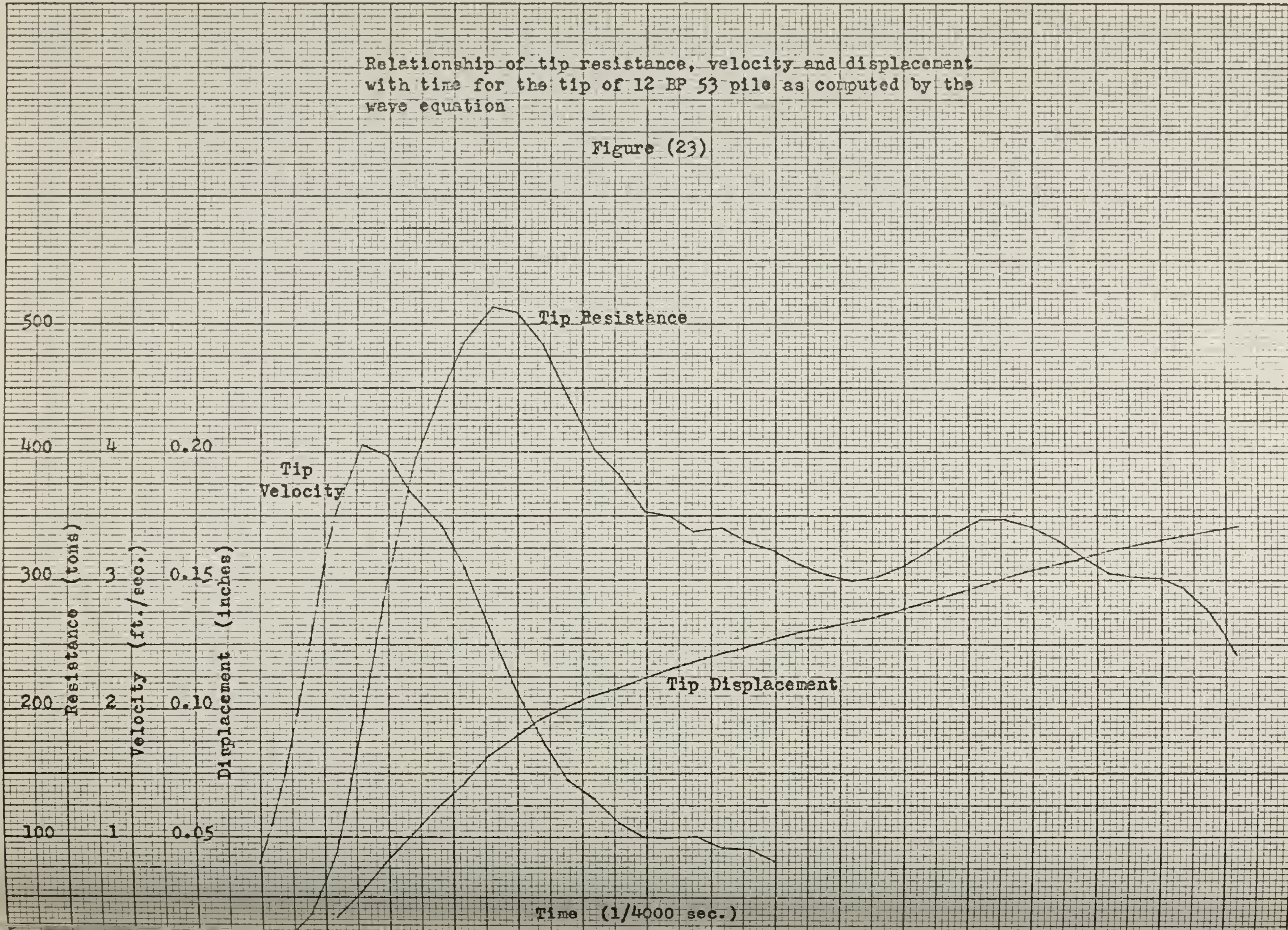
of 0.10 inch the static load is found to be 1600 pounds and the dynamic load 2650 pounds from the load-deformation curve of figure (21). The velocity of deformation at this point is estimated to be about 40 inches per second, or 3.33 feet per second. The corresponding value of J using equation (15) as before is found to be 0.197.

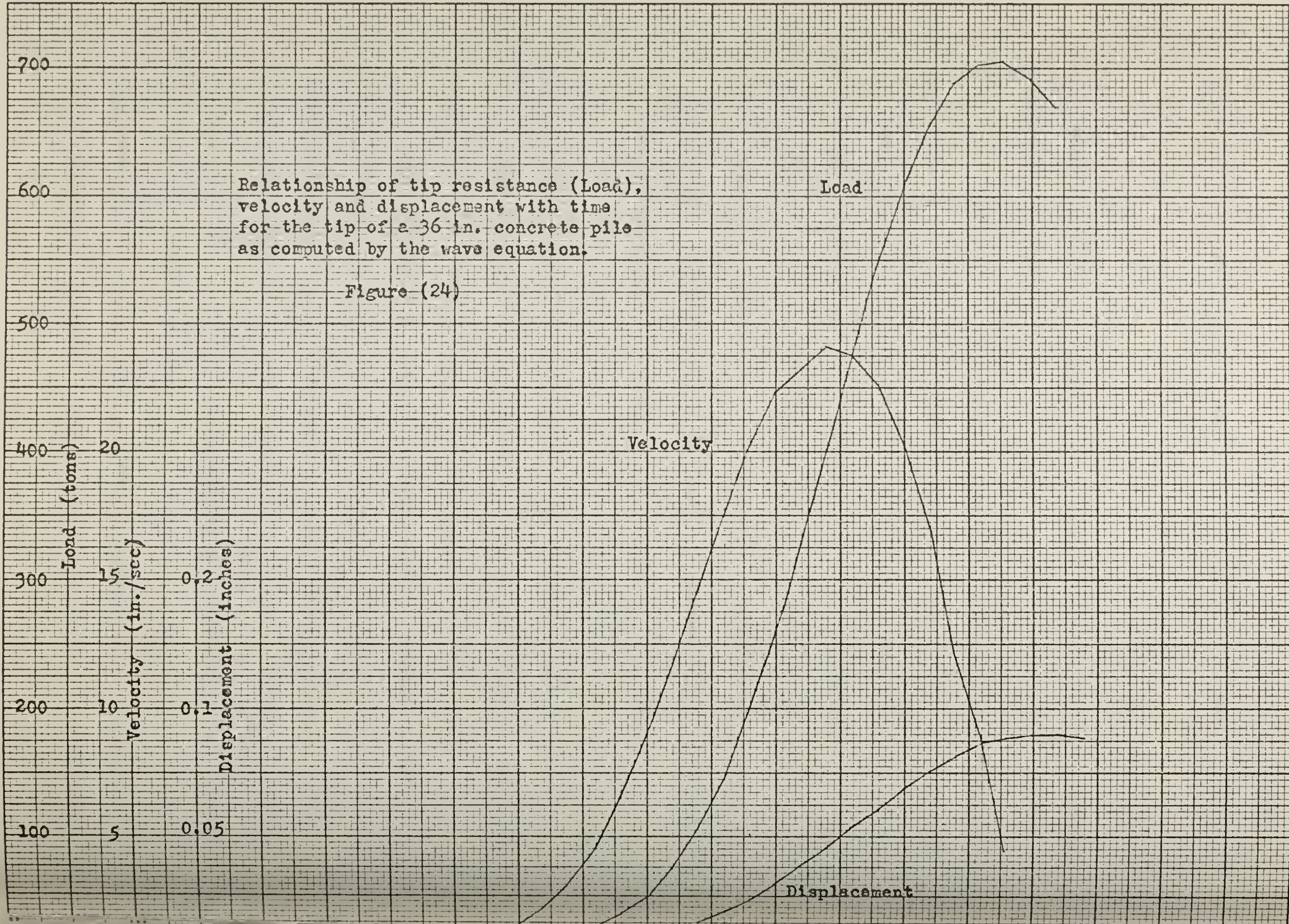
6. Comparison of Dynamic Tests with Computer Solutions

In order to determine whether the dynamic tests described above are qualitatively comparable with the equations used for computation of the total resistance, an analysis has been made of several sets of computer results in the same terms as the data obtained from the laboratory results. This analysis could only be carried to the point of maximum deformation, since the program was established to stop at this point.

Figures (23) and (24) are plots of the computer results for pile tip resistance-time, deformation-time, and load-deformation relationships for a 53-lb. H pile 40 feet long and a concrete pile 36 inches in diameter and 176 feet long. The H pile is driven with a Vulcan #1 hammer and the concrete pile is driven with a Raymond 4/0 hammer. The tip resistance-time curve for the H pile, figure (24), compares closely in shape with same curve for the dynamic laboratory test of the 9-inch-square footing shown in figure (22). This would tend to indicate that the load-time relationship resulting from equation (15) is a fair representation of the actual dynamic load mechanism, at least with respect to a dynamic test on a small footing.

The pile tip velocity-time curves in figures (23) and (24) show a velocity of about 2 feet per second at the time of maximum


Relationship of tip resistance, velocity and displacement with time for the tip of 12 BP 53 pile as computed by the wave equation
Figure (23)
Tip Resistance
Tip Velocity
Tip Displacement
500
400
300
200
100
Resistance (tons)
4
3
2
1
Velocity (ft./sec.)
0.20
0.15
0.10
0.05
Displacement (inches)
Time (1/4000 sec.)



Relationship of tip resistance (Load), velocity and displacement with time for the tip of a 36 in. concrete pile as computed by the wave equation.

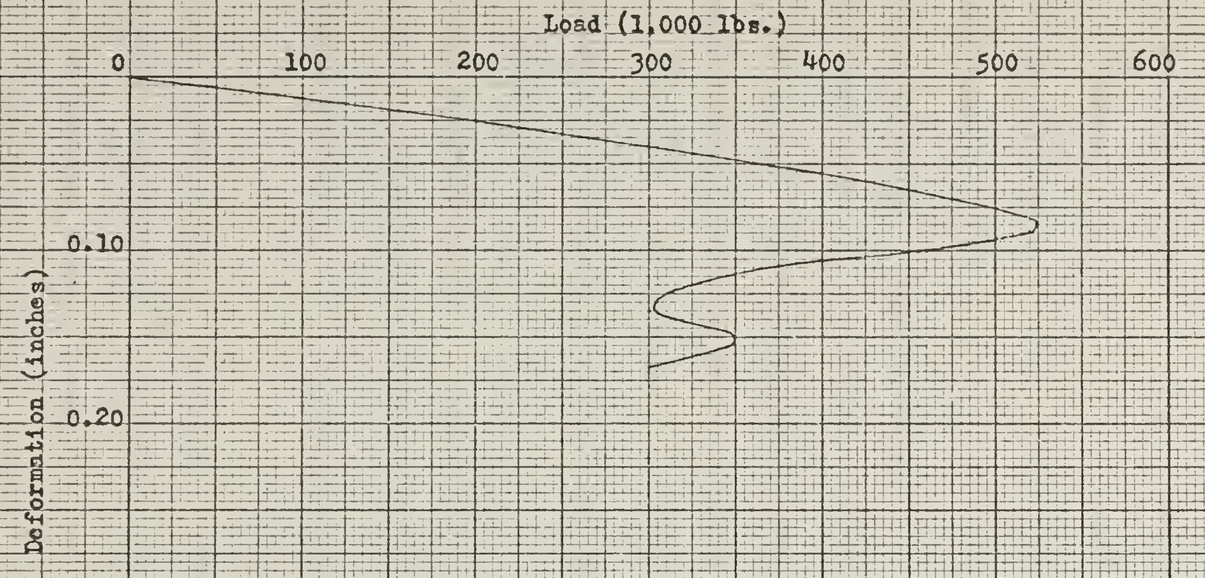
Figure (24)

resistance for the 40-foot H pile and a velocity of about one foot per second for the 176-foot concrete pile. If it is accepted that most of the increase of dynamic resistance over the static resistance is due to the increase in compressive strength of the soil due to rapid loading, and if this increase amounts to a factor of 1.0 for clays and 0.4 for sands, then the damping coefficient must have a value such that when multiplied by the velocity of deformation at the time of maximum load results in values of 1.0 for clays and 0.4 for sand. Figures (23) and (24) indicate that tip velocities at the time of maximum resistance are between 1.0 and 2.0 feet per second, and based on these values, the damping factor (J) would have a value of 0.5 to 1.0 for clay and 0.1 to 0.4 for sand.

If load vs. deformation is plotted from the information available in the curves of load and deformation vs. time for the 12 BP 53 pile shown in figure (23), a curve shown in figure (25) is obtained. This curve is very similar in shape to the curve of dynamic load vs. deformation from rapid load tests performed on sand in the laboratory as shown in figure (21). The fact that these curves are similar in shape tends to indicate that the variation of load with deformation in the wave equation solution is of the same form as that occurring in laboratory load tests. From this it is concluded, tentatively, that the computation of total driving resistance at the tip by the equation

$$R = (D_p - D_p') K' (1 + Jv)$$

is a valid representation of the actual physical process of rapid penetration of the soil by the pile.



Load vs. Deformation From Wave Equation Solution
For a 12 BP 53 Pile and Vulcan Number 1 Hammer

Figure (25)

7. Ground quake, which has been defined as the limit of elastic deformation of the soil, is normally thought of as a static concept, but it is an essential element in wave equation computations. The elastic ground compression occurring during static loading is not necessarily the same that occurs during pile driving, and no attempt has been made to relate the two. Therefore, quake as used in this investigation is a dynamic property.

A method of measuring quake during pile driving is described by Chellis (ref. 6). Essentially, this method measures the total elastic compression of the pile and soil. Pile compression alone is computed from the formula $C_2 = R_u L/AE$, where R_u is the ultimate bearing load on the pile, L is the length of the pile, A is the cross-sectional area, and E is the modulus of elasticity. This value is subtracted from the measured compression of pile and soil to obtain the value of quake. This method of measuring quake assumes that the pile is in compression simultaneously along its entire length, which is somewhat erroneous in view of the longitudinal stress wave concept.

In extensive studies made of pile driving, Hiley made measurements of quake and reported that quake varies considerably for different conditions of ground and may average 0.1 inch in firm gravel, up to 0.2 inch in firm clay and may have values as high as 0.5 inch in peaty material (ref. 17). With respect to driving conditions, Hiley summarized his observations as follows:

Driving Conditions	Quake	Stress in Pile
Easy driving	0.0 - 0.10	500 psi
Medium driving	0.10 - 0.20	1000 psi
Hard driving	0.10 - 0.30	1500 psi
Very hard driving	0.05 - 0.20	2000 psi

It was also stated that quake may be as much as double these values if soft ground is immediately below the pile tip. These values for quake are used as a limiting range for investigations of load tests.

Chapter VI

CORRELATION OF WAVE EQUATION RESULTS WITH PILE LOAD TESTS

A. Methods of Approach and Effects of the Various Parameters

Although there are many load tests published, there are comparatively few which have not left out some vital part of the information required for a thorough evaluation of the total pile driving and loading process. Partly due to this limitation, two general methods of approaching the problem of correlating wave equation computations with load test data were tried.

The first method was particularly applicable to those tests for which very complete information was published. In these cases it was thought that most of the variables could be fairly accurately evaluated so that the investigation could be limited to a trial and error approach in which the principal unknowns could be varied over a wide range of values and tried in various combinations. Generally this means that ground quake and damping constants and side friction were varied with all other factors kept constant for the case in question.

The second method of approach was adopted to try and derive as much information as possible from published load test data which were not reported as completely as those investigated by the first method of approach. It also considered desirable to run a series of data to study more fully the effects of the various parameters on the resistance-set relationship for particular piles and hammers. It was thought that both of these objectives could be met by selecting

a particular pile type for which a number of reasonably good test data were available. Furthermore, it was desired that the pile meet the following requirements:

(a) Be constructed of a material of which definite information was available as to its properties.

(b) Be constructed of a material which of itself would least affect the soil around it and vice versa.

(c) Have uniform properties throughout its length.

(d) The pile as driven would be required to sustain the static loads to be imposed upon it.

The reason for these requirements was to eliminate as many of the unknowns from the problem as possible. The steel H pile and steel pipe pile each satisfied these requirements better than other types. Because there were more usable load tests readily available for steel H piles, particularly the 12-inch H bearing pile weighing 53 pounds per foot, this shape was selected. It had the added advantage that most of the records available indicated that one type of hammer had been used, the Vulcan Number One; however, tests were available with other size hammers so that a further check of the method could be made.

The main effort has been directed toward establishing correlation for each load test case within a limiting range of values of each of the variables involved. Correlation of wave equation computations with the actual pile load test involves the evaluation of five different variable factors. These are:

(a) Ground quake;

(b) Point damping factor;

(c) Side damping factor;

(d) Distribution of resistance between point and sides of the pile;

(e) Per cent of static resistance acting during driving.

These factors have been discussed in above sections with the exception of the last named variable, the per cent of static resistance acting during driving, which will be discussed below.

In correlating theoretical calculations with load tests and the corresponding driving records, all variables except one must be held constant while varying one factor at a time. From practical considerations it was not possible to treat all five factors alike as variables. Since little is known of the nature of the side damping factor and since the effect of side damping in most cases is relatively small compared to point damping, the side damping factor was eliminated as a separate influence by assuming it equal to one third of the point damping factor for most cases. Smith indicated his belief that side damping would be less than point damping for a given soil, and he recommended a value which was one third of the point damping coefficient (ref. 28).

Although up to five different distributions of resistance were investigated for each correlation case, this factor was assumed known when it came to the actual correlation using the first method. For the other method, distribution was treated within a range of values. The various distributions of resistance used for investigation are:

(a) End bearing

(b) 75 per cent end bearing and 25 per cent side friction

(c) 50 per cent end bearing and 50 per cent side friction

(d) 25 per cent end bearing and 75 per cent side friction

(e) All side friction

Since the pile is generally divided into sections 5 to 10 feet long for purposes of calculation, the side friction acting on the lower section of the pile is indistinguishable from the point bearing resistance, except for damping, and adds to it for all practical purposes.

Three methods of distributing side friction along the embedded length of the pile have been used. The method used most, and the simplest, assigns a uniform value of resistance for a unit length of pile regardless of depth. This method is termed rectangular distribution. In another method the side friction is distributed in a manner so that the unit resistance is directly proportional to the depth, and is called triangular distribution. Both of these methods are included in the computer program, and selection of the applicable method is made by use of a control statement in the program. Except for very long piles, the difference in effect on the resistance-set relationship of the two methods is small, so the first method was generally used. The third method of distributing frictional resistance is by estimating the actual resistance developed in each of the soil strata that the pile penetrates.

The per cent of static resistance acting during driving is a factor that is influenced by time effects. In a sensitive clay, pile driving remolds the soil resulting in a reduction in strength during

[illegible]

[illegible]

[illegible]

[illegible]

and immediately after driving of the pile. If the load test is conducted after the soil has had time to regain its strength, the value of the static resistance should be reduced when making the pile-driving analysis. Another type of time effect may result when a pile is driven into a sand layer through overlying layers of soft or medium clay. When the pile is test loaded, resistance initially developed in the clay may be transferred by plastic flow to the tip and lost from the standpoint of static load-carrying capacity. During driving, however, resistance in the upper clay layers will be present and will appear to result in heavy damping. Another effect is the liquefaction of saturated silts due to a quick condition being created by the driving action and resulting in greatly reduced resistance to penetration of the pile. The consideration of the effects of such factors in the calculations is very approximate and without complete soil data cannot be accounted for.

In correlating wave equation computations with load tests in which sufficient data was available on which to base a reasonable estimate of the distribution of load between point bearing and side friction, the approach was to determine the various values of quake and damping that produced a resistance-set curve that included the point representing the ultimate load and final penetration of the test pile. Hypothetically, there would be an infinite number of such curves, but a limiting range of values was used for these variables that was determined on the basis of Smith's work and the experimental work on the dynamic properties of soil discussed above. The range of values for quake and damping investigated were:

[illegible]

[illegible]

(a) Quake

(1) sand 0.05 to 0.20

(2) clay 0.05 to 0.30

(b) Damping

(1) sand 0.10 to 0.20

(2) clay 0.40 to 1.00

These ranges of values were not rigidly adhered to, and in some instances outside values were investigated. The load test information, input data used for the computer program for each load test, and results of the correlation attempts are included in tables F to BB. The accompanying figures are the ultimate resistance versus set (in blows per inch) curves drawn from the computer results and used as the basis for correlation.

Similar curves used as the basis of correlation by the second method are included in appendix E. The length of piles used for the calculations was chosen to generally fit the load test data available, but since the effect of length was not clearly established the choice of 40-, 80- and 120-foot piles was somewhat arbitrary. Calculations were made for 12 BP 53 piles in these lengths using a Vulcan Number One hammer with a hardwood capblock, a pile cap weighing 700 pounds, for all permutations of the ranges of ground quake, point and side damping factors and side friction distributions of a rectangular shape. For each combination of the variables, set in blows per inch was computed for each value of ground resistance ranging from 40 to and including 280 tons in 20-ton increments. The results were plotted so that on any graph there are five curves, one for each percentage of side

resistance assumed. For any one curve of ultimate resistance vs. set, all other variables are constant.

These results show the following generalized effects of changing the variables indicated below:

(a) Ground quake. Increasing ground quake increases the set in blows per inch, i.e. harder driving, for a given ultimate ground resistance for all values of side resistance including the point bearing case.

(b) Increasing both the point and side damping increases the set in blows per inch for a given ultimate ground resistance for all values of side friction, including zero.

(c) Side distribution--increases in per cent of side friction cause dramatic decreases in set in blows per inch for a given ultimate ground resistance.

(d) Increased pile length--here the results are mixed. In the zero friction or end bearing case, the ultimate resistance tends to become the same for 40-, 80-, and 120-foot cases after about 24 blows per inch. Before that point is reached, the 80- and 120-foot piles have essentially the same curve of ultimate resistance vs. set, and the 40-foot pile for a given resistance within this range tends to have an appreciably decreased set in blows per inch. For about 50% friction, the results may be practically identical for all three lengths. For higher values of friction the longer piles may have decreased set for given resistance.

(e) Increased pile weight may result in decreased set in blows per inch for a given ultimate resistance; however, in the lower ranges

of resistance the difference may be slight. For example, for a side resistance of 25 per cent and all factors the same for two 45-foot H piles driven with a Vulcan 0 hammer, the difference in set for a 14 BP 89 and a 14 BP 117 is less than 2 blows per inch. The values are 8 and 6.6 respectively.

(f) Increased hammer weight, but with the same stroke, results in decreased set in blows per inch for a given resistance and all other factors constant.

(g) A softer capblock results generally in increased blows per inch for a given resistance, but the effect is really quite variable. Comparisons were made for an ultimate ground resistance of 100 tons for the following piles driven with a Vulcan Number One hammer:

1) Concrete Piles--18 inches square with lengths of 40, 80, and 120 feet. It was assumed that head packing would be used, but this same soft material was assumed for the capblock also and the results compared with capblocks of hardwood and micarta. There was no appreciable difference in sets for any of these conditions for any of the concrete piles.

2) Wood Piles--for the 80- and 120-foot lengths, the difference was slight for the three capblocks. For the 40-foot pile the difference between a micarta and a hardwood capblock were slight, but the very soft capblock made about 10 blows per inch difference in the results.

3) Steel piles--the capblock had essentially no effect on the 80- and 120-foot length piles, but the very soft capblock made about 2 blows per inch difference with the 40-foot pile.

The effects produced by varying some of these parameters are not surprising, but in some instances they may be, especially if one's judgment is based on results produced by popular dynamic formulas. For example, the results produced by the Hiley formula may indicate very slight differences between the cases when side friction is present and when it is totally absent. On the other hand, it is interesting to note that when side friction is a large percentage of the ultimate resistance to driving, the set vs. ultimate resistance curves may tend toward the curve produced by the Engineering News formula. The foregoing results produced by the wave equation method of analysis would seem to bear out its general suitability under widely varying conditions of driving equipment, pile type, and ground conditions.

As can be seen by examining the following case studies, some of the data upon which to base correlation is rather complete, and some is not. The cases studied include various size piles, assorted pile driving equipment, in soils varying from gravel to clay, piles constructed of concrete and steel, different ratios of embedded length to driven length, and for varying combinations of soil strata. It is apparent that for many of the cases correlation can be obtained in several ways by various combinations of values for ground quake, point and side damping, and side friction. Because these factors are not now known with a high degree of certainty, the problem becomes one of searching for a pattern within the likely ranges of the variables for the various types of soils encountered.

In some of the following cases it was necessary to estimate the values of Q and J which produce correlation, since the range of

values used in the computations did not extend far enough to include the load test point within computed load-set curves. The values for Q and J so estimated are based on the curves nearest to the load test and the trend of variation between curves. It is believed that the estimated values are reasonably accurate considering the nature and accuracy of the basic data.

Before examining the case studies in detail, the manner of identifying the set versus resistance curves is noted:

Curve A -- 0% friction, i.e., an end bearing pile with no resistance to driving offered by the soil along the pile sides.

Curve B -- 25% friction, i.e., 75% of resistance to driving offered by the soil at the pile point and 25% along the pile sides.

Curve C -- 50% friction, i.e., 50% of resistance to driving offered by the soil at the pile point and 50% along the pile sides.

Curve D -- 75% friction, i.e., 25% of resistance to driving offered by the soil at the pile point and 75% along the pile sides.

Curve E -- 100% friction. All resistance to driving distributed along the pile sides.

TABLE F

Case Number 2001

Reference: (18)

Pile: Prestressed concrete, 36 in. diameter, 176 ft. long

Hammer: Raymond 3/0, 12500 lb. ram, 24 in. stroke

Embedded length: 40 ft.

Soil: Soft to medium clay with point in stiff clay.

Final penetration: 20 blows per inch

Load test: Tested to failure at 190 tons including the weight of pile, tested one week after driving

Input data:

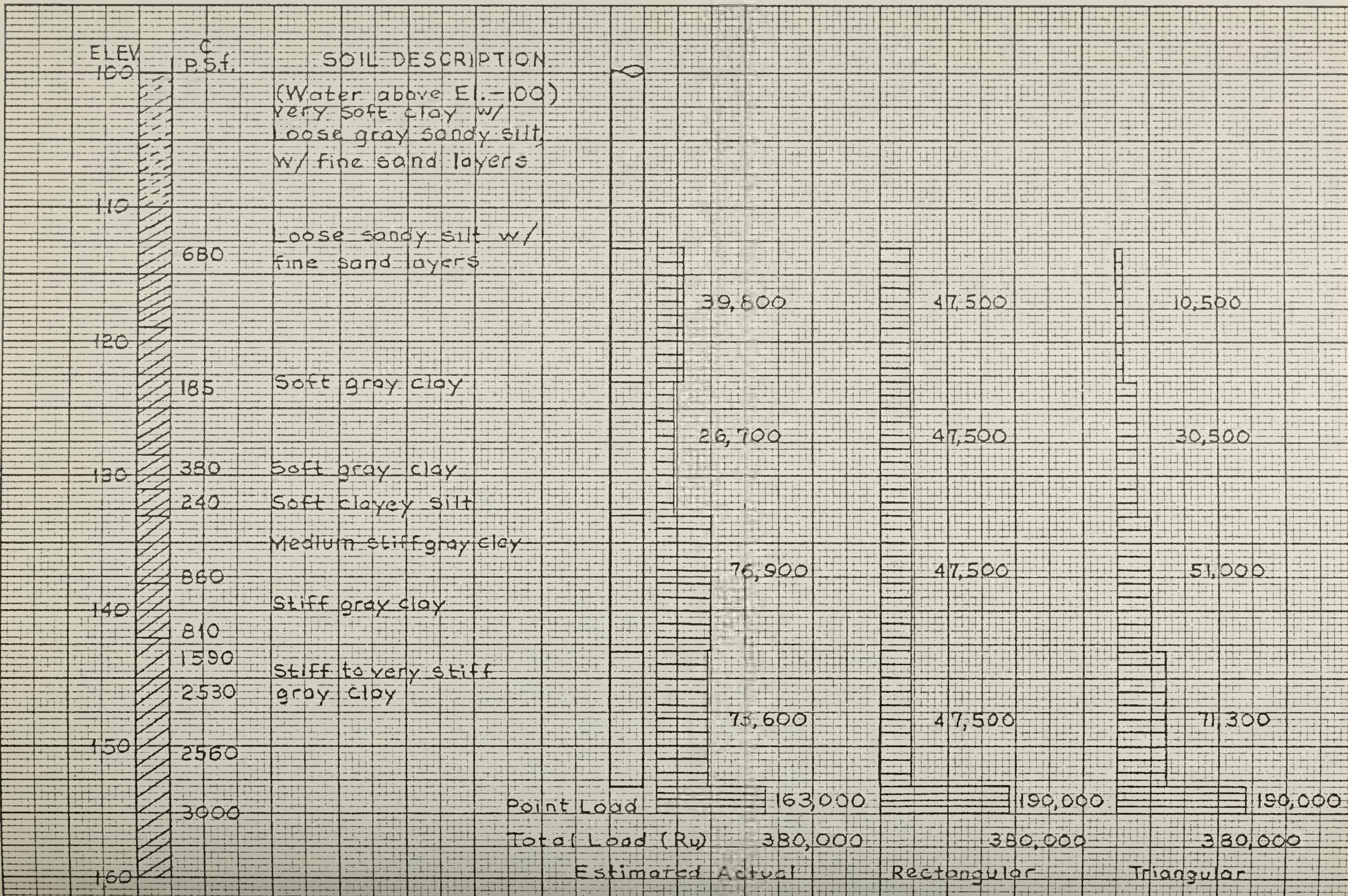
W(1)	12,500	Ram
W(2)	8,800	(includes follower and driving cap)
W(3)-W(20)	4,955	(based on 9.775 ft. segments)
V	10.15	(ram velocity at impact for 2-ft. drop)
K(1)	10,600,000	(capblock spring constant)
K(2)	2,180,000	(cushion and pile)
K(3) –	-	(Pile)
K(19)	20,750,000	
K'(m)	Soil spring constant depends on value of ultimate resistance which is varied.	

Distribution of resistance: 50% end bearing and 50% friction, based on computation of point capacity by the formula: P = 9AC. Friction considered to have a rectangular distribution.

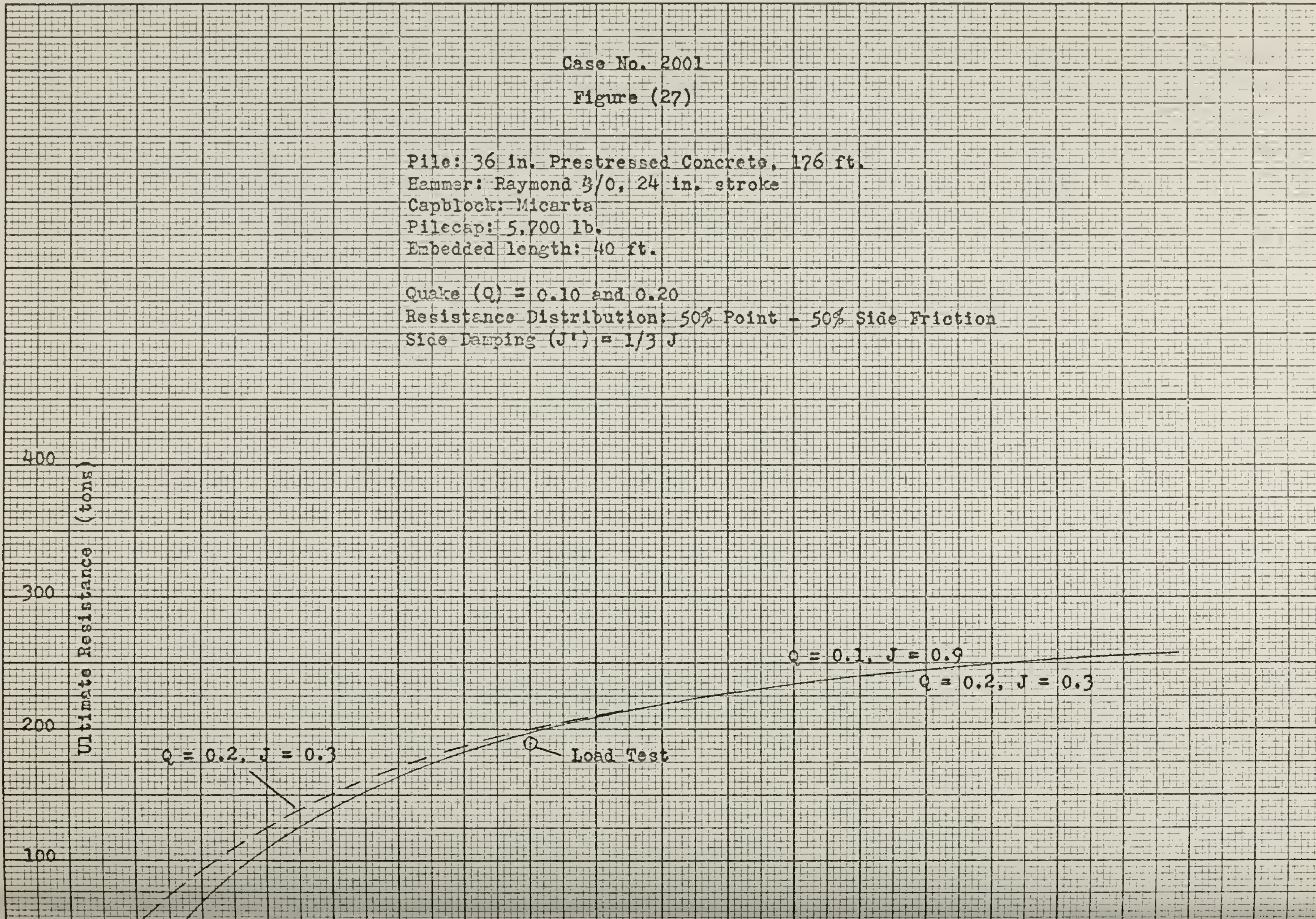
Results: See figures (27) and (28)

Correlation is obtained for the following values of Q and J:

Q	J	J'
0.1	0.9	0.3
0.2	0.3	0.1



SOIL PROFILE AND RESISTANCE DISTRIBUTION FOR LOAD TEST



Case No. 2001
Figure (27)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 3/0, 24 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
Quake (Q) = 0.10 and 0.20
Resistance Distribution: 50% Point - 50% Side Friction
Side Damping (J') = 1/3 J
Ultimate Resistance (tons)
400
300
200
100
Q = 0.1, J = 0.9
Q = 0.2, J = 0.3
Load Test
Q = 0.2, J = 0.3

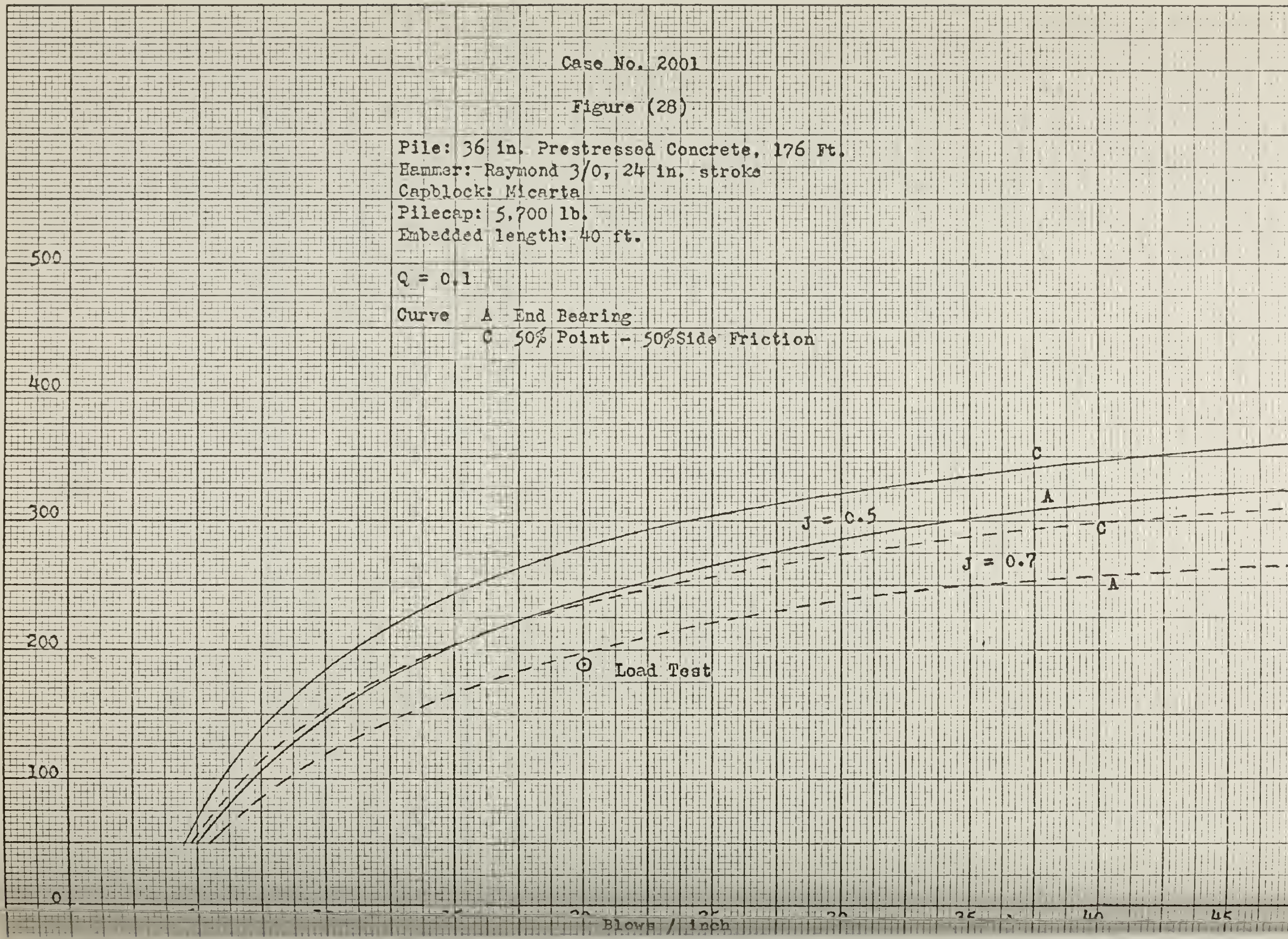

Case No. 2001
Figure (28)
Pile: 36 in. Prestressed Concrete, 176 Ft.
Hammer: Raymond 3/0, 24 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
Q = 0.1
Curve A End Bearing
C 50% Point - 50%Side Friction
J = 0.5
J = 0.7
C
A
C
A
Load Test
500
400
300
200
100
0
45
Blows / inch

TABLE G

Case Number 2002

Reference: (18)

Pile: Prestressed concrete, 36 in. diameter, 176 ft. long.

Embedded length: 40 ft.

Hammer: Raymond 4/0, 15,000 lb. ram, 34 in. stroke.

Soil: Soft to medium clay with point in stiff clay.

Final penetration: Refusal, last 3/4 in. at 99 blows.

Load test: Tested to failure at 230 tons including the weight of pile.

Input data:

W(1)	15,000
W(2)	8,800
V	12.08

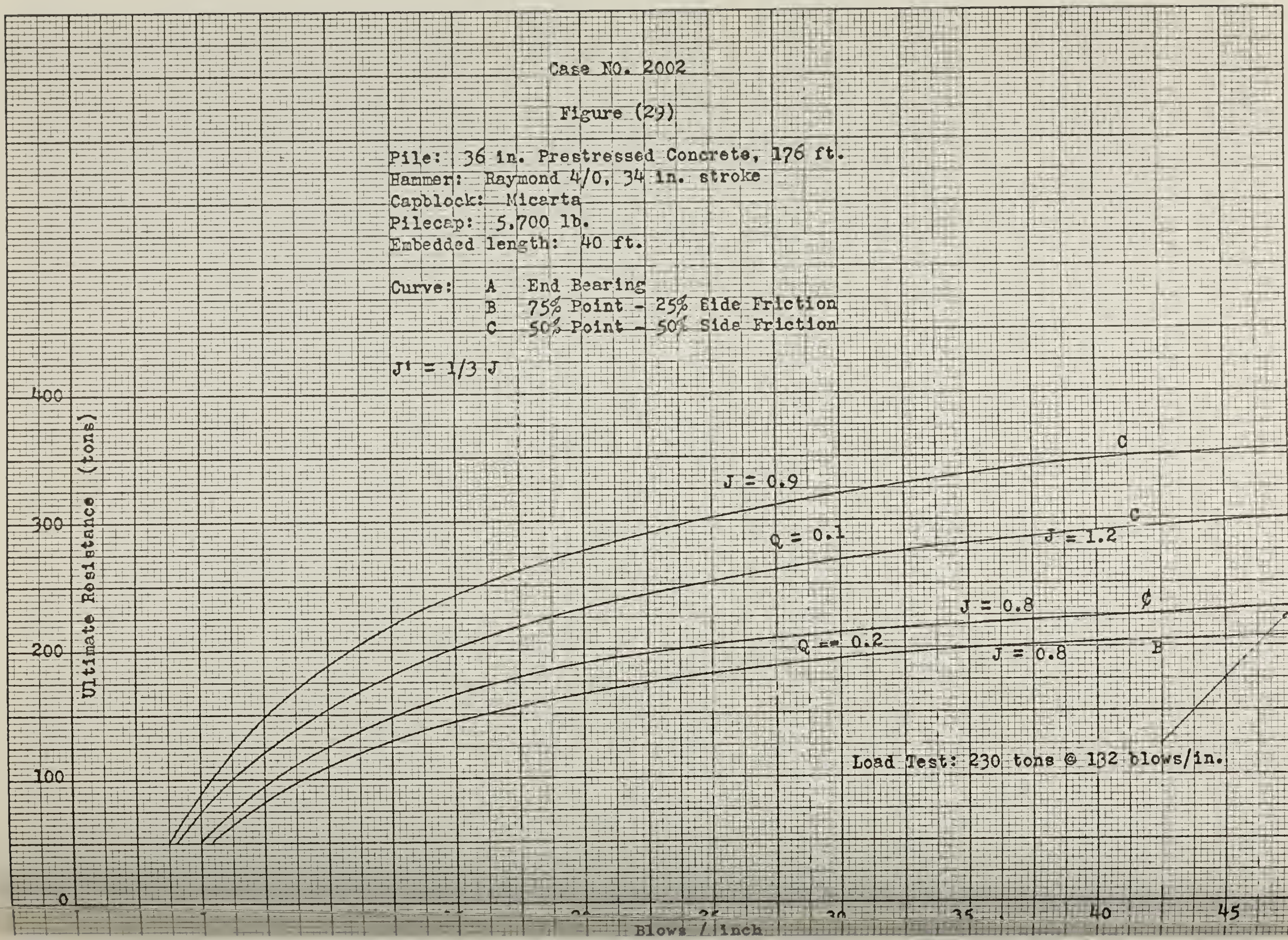
(Rest of input data same as for case 2001)

Distribution of resistance: Estimated between 75% and 50% point bearing.

Results: See figures (29) and (30).

Correlation is obtained for the following values of Q and J:

Q	J	J'
0.2	0.8	0.27



Case NO. 2002
Figure (29)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 4/0, 34 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
Curve: A End Bearing
B 75% Point - 25% Side Friction
C 50% Point - 50% Side Friction
J' = 1/3 J
J = 0.9
Q = 0.1
J = 1.2
J = 0.8
Q = 0.2
J = 0.8
C
C
B
Load Test: 230 tons @ 132 blows/in.
Ultimate Resistance (tons)
0
100
200
300
400
30
35
40
45
Blows / inch

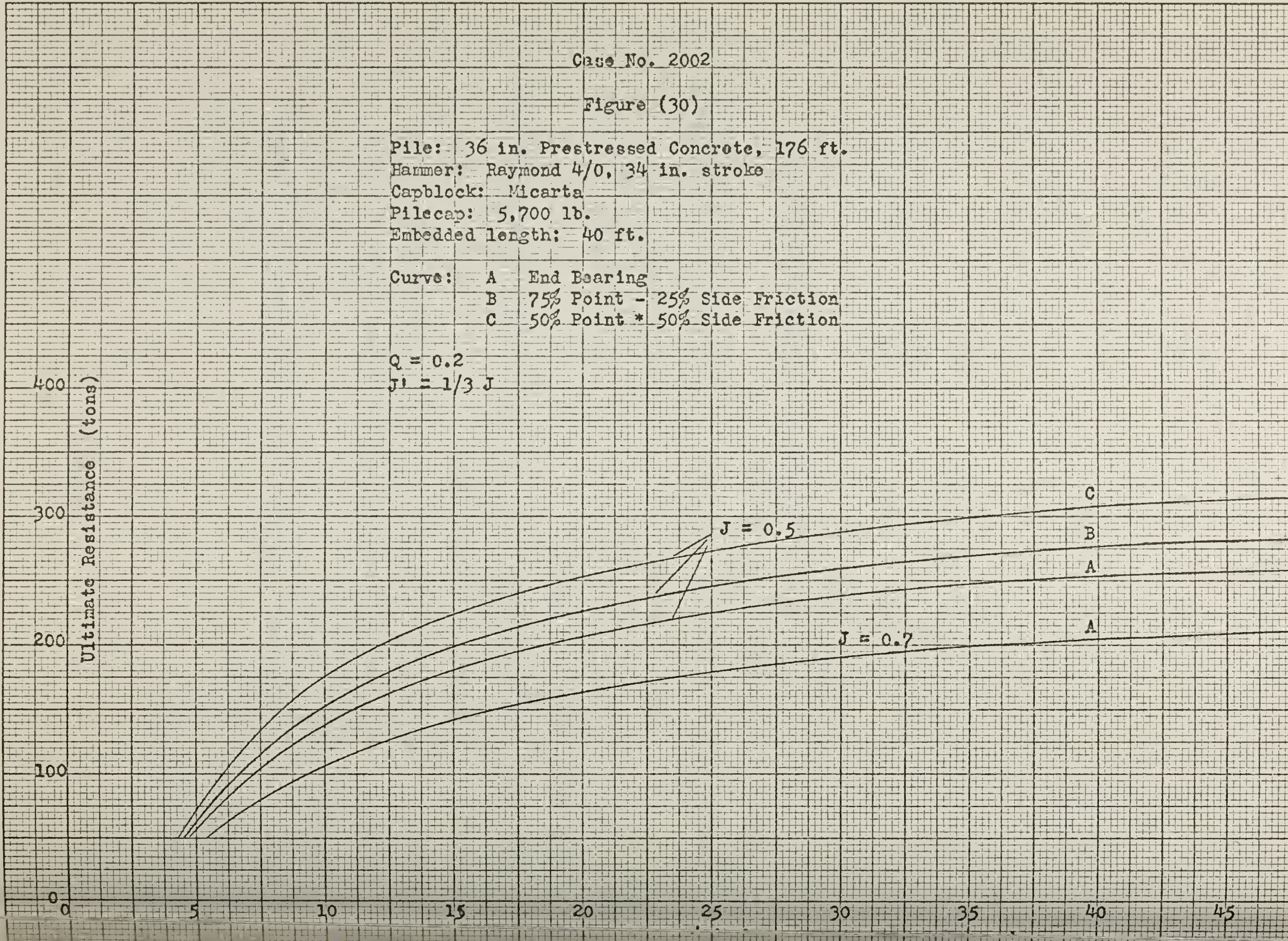

Case No. 2002
Figure (30)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 4/0, 34 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
Curve: A End Bearing
B 75% Point - 25% Side Friction
C 50% Point * 50% Side Friction
Q = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
0
100
200
300
400
0
5
10
15
20
25
30
35
40
45
J = 0.5
J = 0.7
C
B
A
A

TABLE H

Case Number 2003

Reference: (18)

Pile: Prestressed concrete, 36 in. diameter, 176 ft. long.

Embedded length: 40 ft.

Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke

Soil: Soft to medium clay with point in stiff clay

Final penetration: 10 blows per inch

Load test: Tested to failure at 165 tons including weight of pile.

Input data: W(1) 12,500

V 12.9

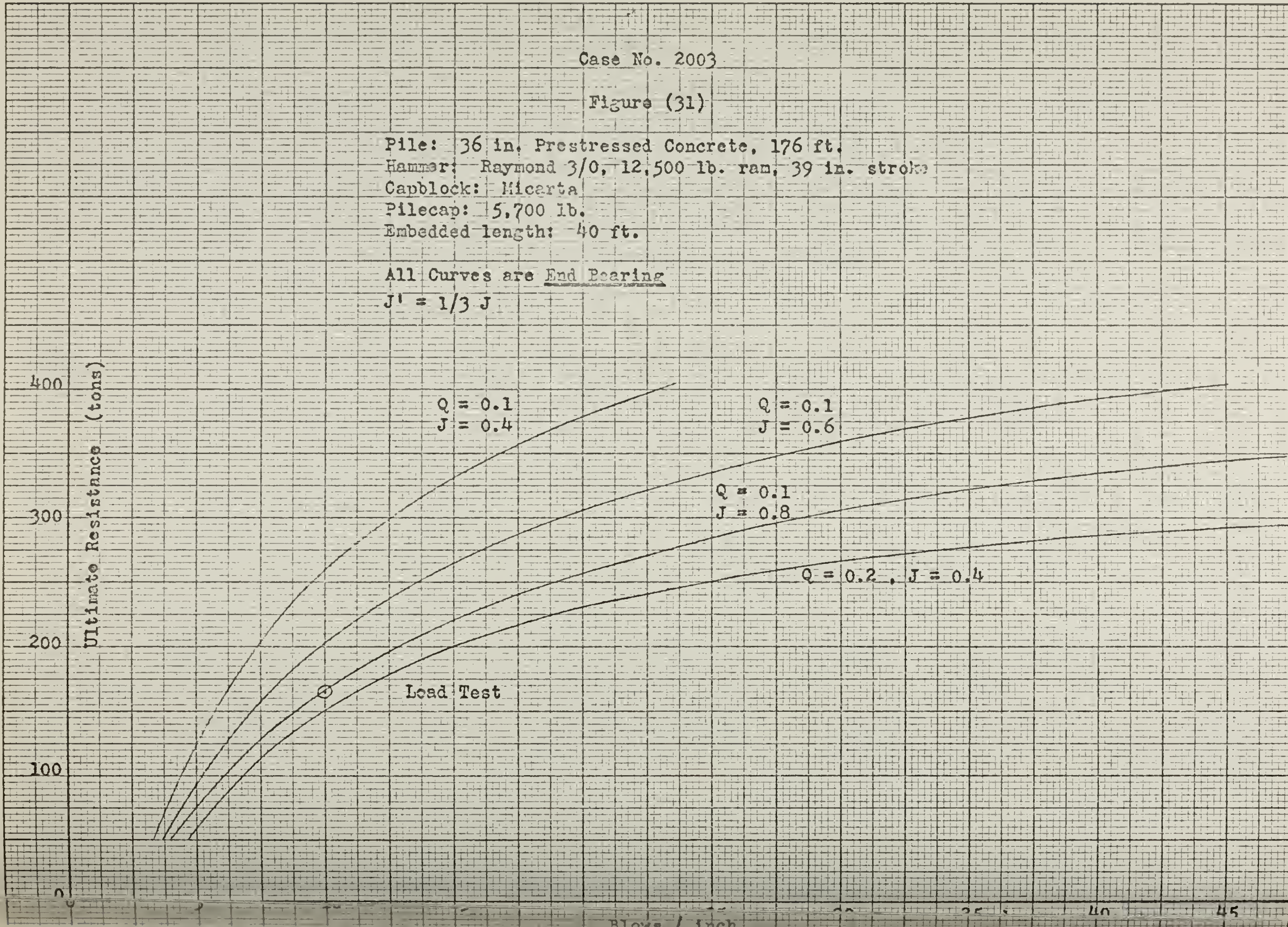
(Rest of input data same as for case 2001)

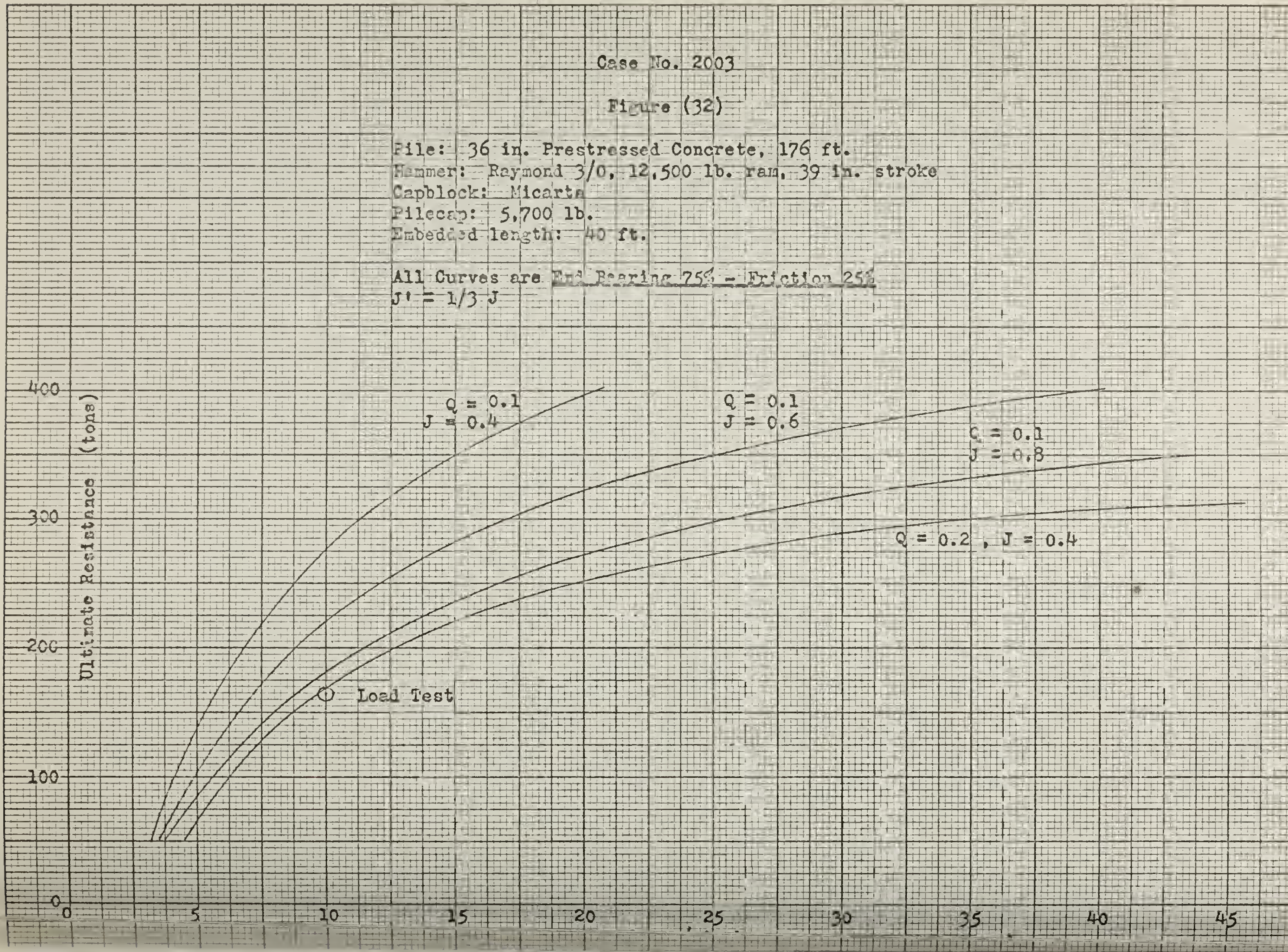
Distribution of resistance: Estimate 50% end bearing and 50% friction

Results: See figures (31) through (34).

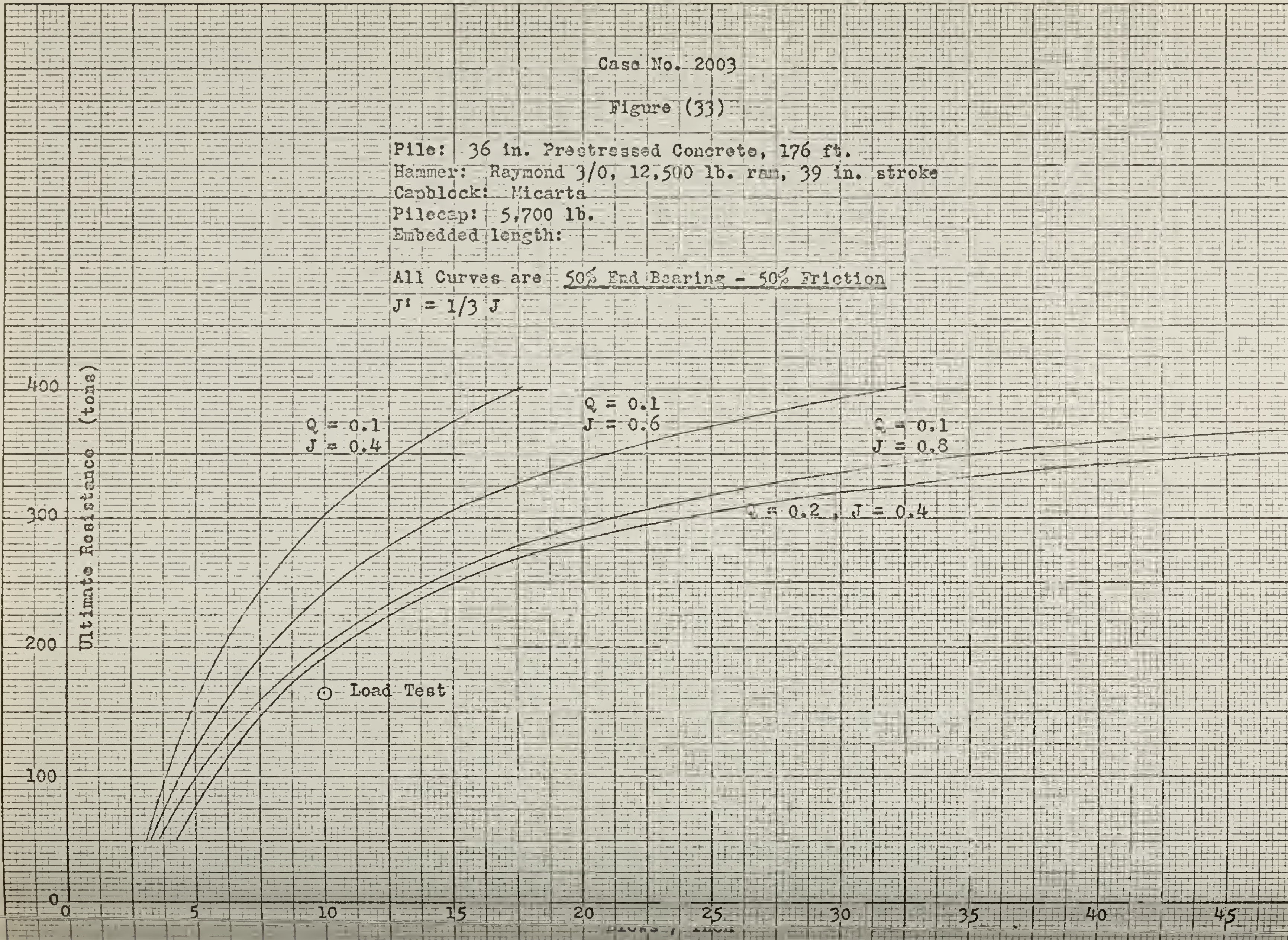
Correlation obtained for the following values of Q and J:

Q	J	J'
0.1	1.0	0.33
0.2	0.5	0.17


Case No. 2003
Figure (31)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
All Curves are End Bearing
J' = 1/3 J
Q = 0.1
J = 0.4
Q = 0.1
J = 0.6
Q = 0.1
J = 0.8
Q = 0.2 , J = 0.4
Load Test
Ultimate Resistance (tons)
400
300
200
100
0
40
45
Blows / inch



Case No. 2003
Figure (32)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 40 ft.
All Curves are End Bearing 75% - Friction 25%
J' = 1/3 J
Ultimate Resistance (tons)
Q = 0.1
J = 0.4
Q = 0.1
J = 0.6
Q = 0.1
J = 0.8
Q = 0.2 , J = 0.4
Load Test
0
100
200
300
400
0
5
10
15
20
25
30
35
40
45


Case No. 2003
Figure (33)
Pile: 36 in. Prestressed Concrete, 176 ft.
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length:
All Curves are 50% End Bearing - 50% Friction
J' = 1/3 J
Q = 0.1
J = 0.4
Q = 0.1
J = 0.6
Q = 0.1
J = 0.8
Q = 0.2 , J = 0.4
Load Test
Ultimate Resistance (tons)
0
100
200
300
400
0
5
10
15
20
25
30
35
40
45

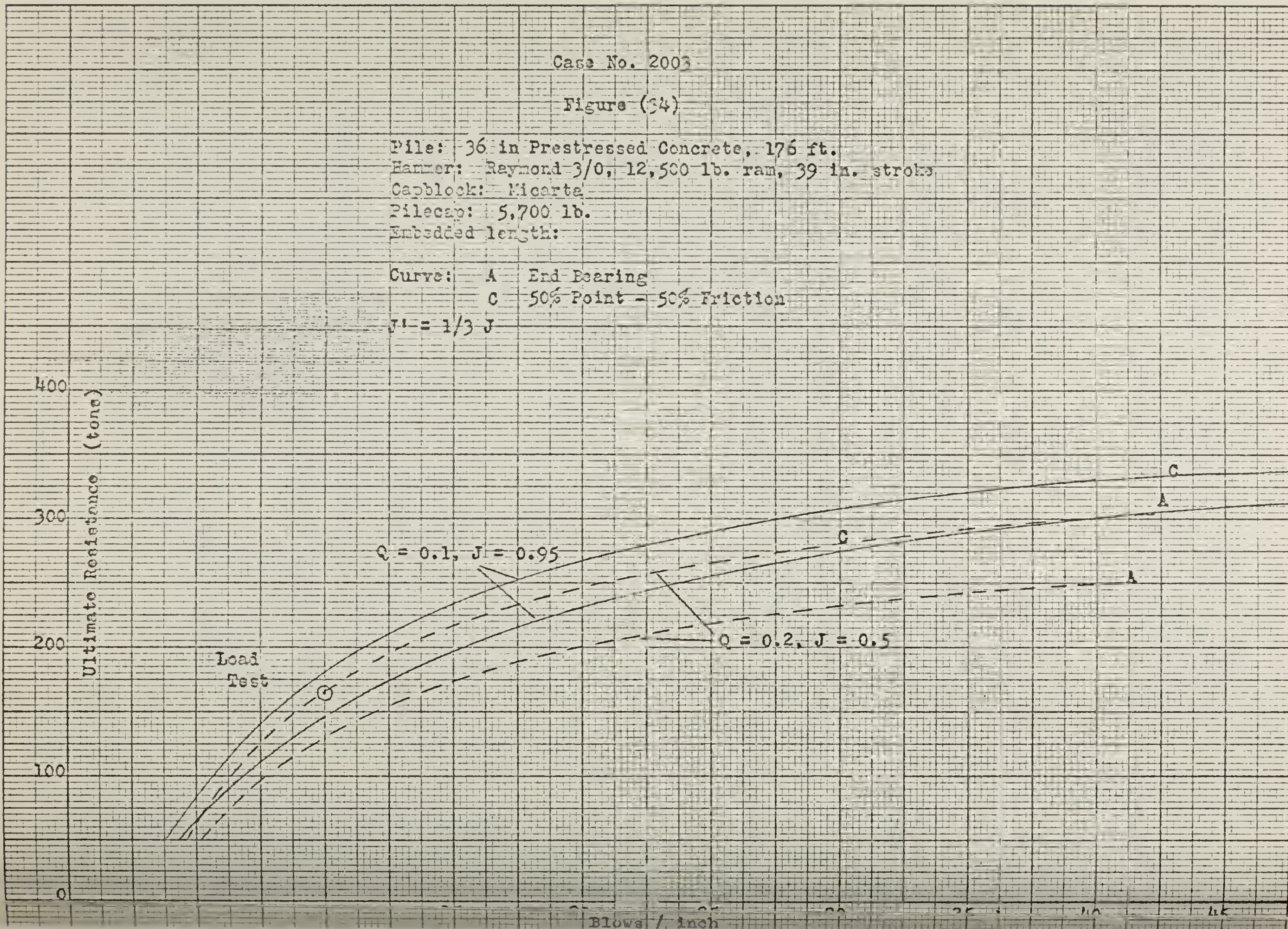

Case No. 2003
Figure (34)
Pile: 36 in Prestressed Concrete, 176 ft.
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length:
Curve: A End Bearing
C 50% Point - 50% Friction
J' = 1/3 J
Ultimate Resistance (tons)
400
300
200
100
0
Q = 0.1, J = 0.95
Q = 0.2, J = 0.5
Load Test
C
A
C
A
Blows / inch

TABLE I

Case Number 2004

Reference: (18)

Pile: Prestressed concrete, 36 in. diameter, 80 ft. long.

Embedded length: 43.5 ft.

Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke.

Soil: Medium and stiff clay with point in very stiff clay.

Final penetration: 20 blows per inch.

Load test: Tested to failure at 365 tons.

Input data:

W(1)	12,500
W(2)	8,800
W(3) - W(10)	5,070
V	12.9
K(1)	10,600,000
K(2)	4,020,000
K(3) - K(9)	20,300,000
e_1	0.8
e_2	0.5

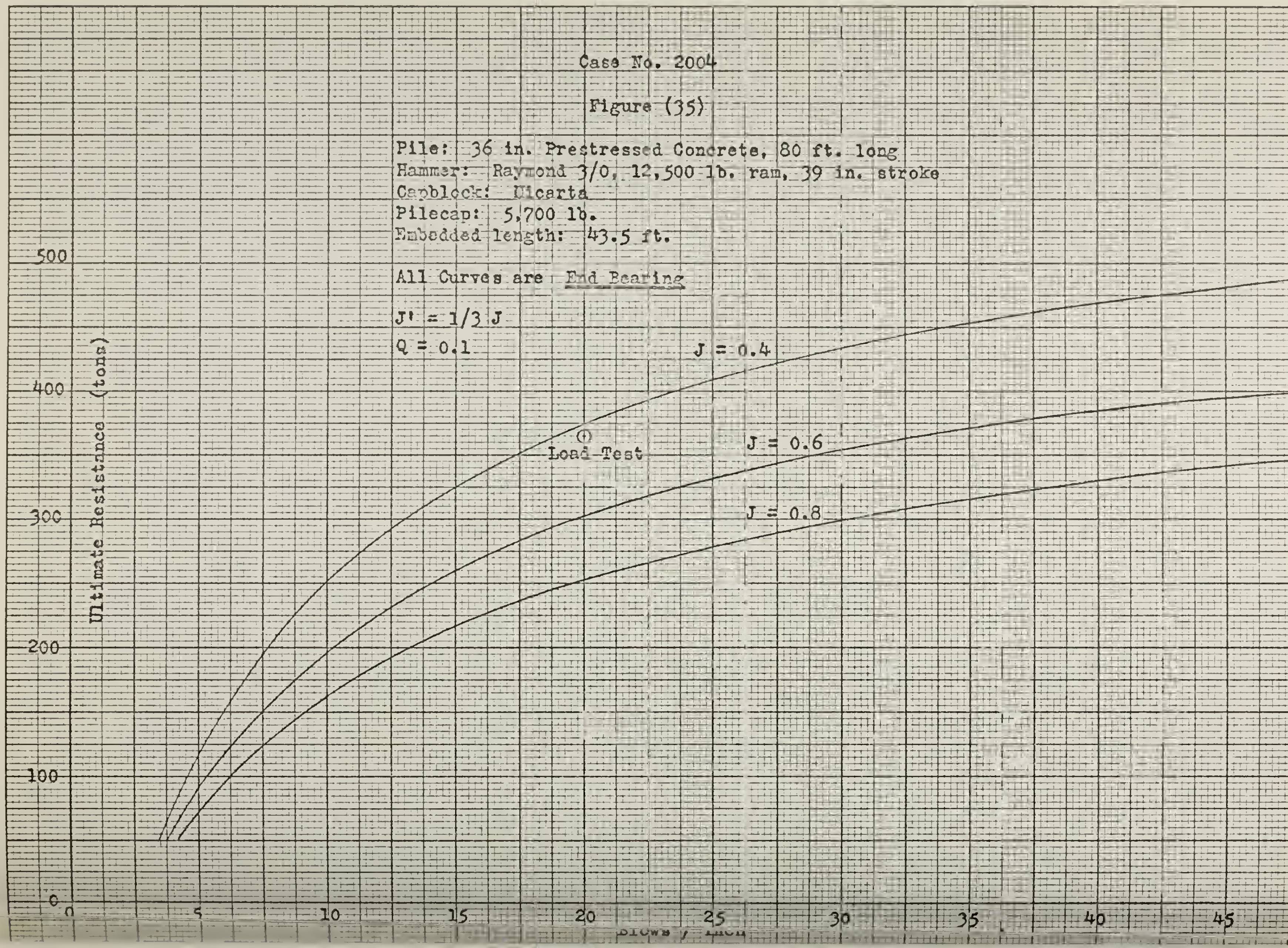
Distribution of resistance: From computations based on load test and soil data, it is estimated that 87% of the load was carried by the pile point at failure. Correlation should be made for distributions between 100 per cent and 75 per cent of the load carried by the point.

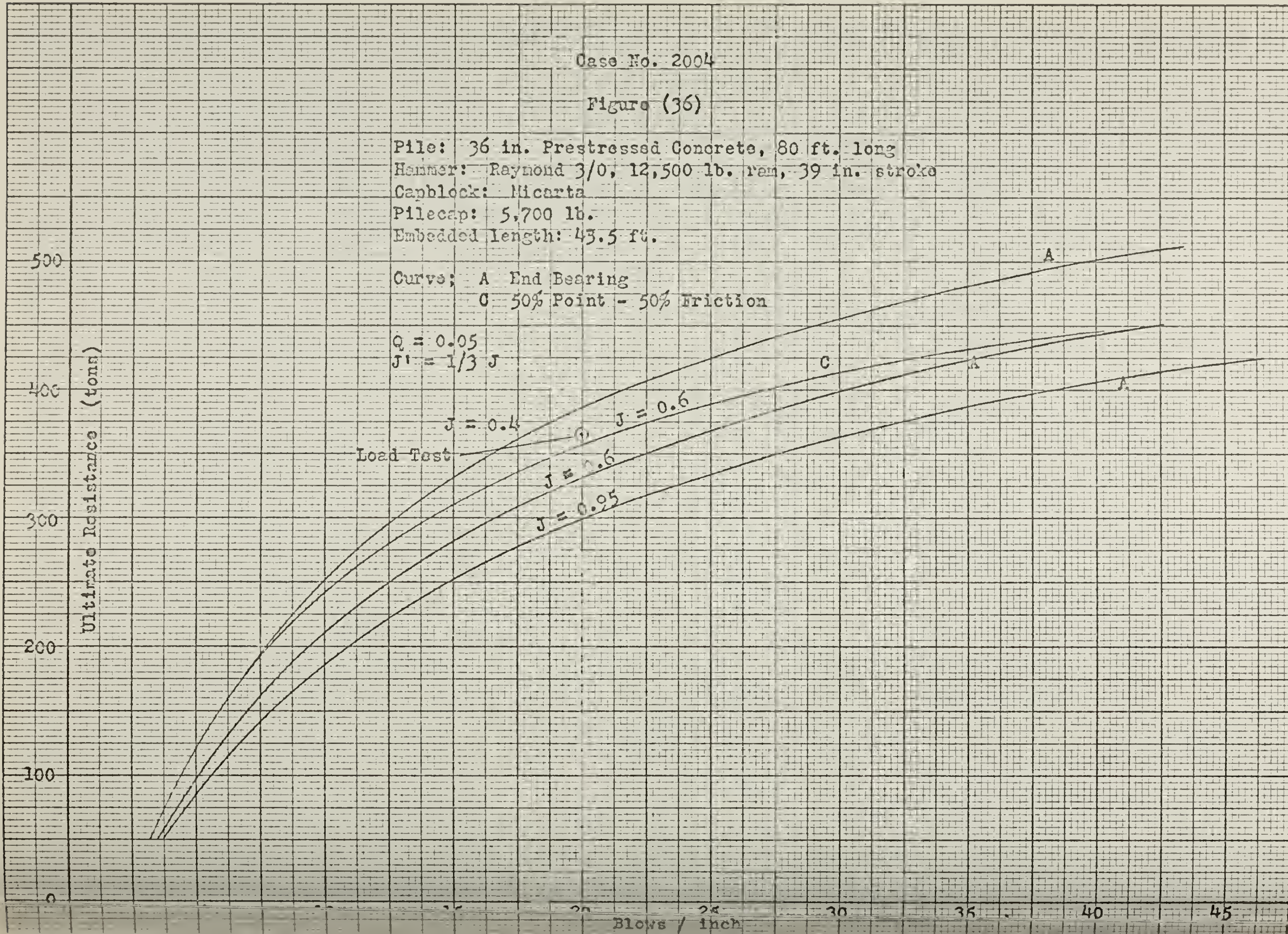
Results: See figures (35) through (37)

Correlation is obtained for the following values of Q and J:

Q	J	J'
0.1	0.45	0.15
.05	0.6	0.2

Remarks: In this case, a hole 24 ft. in depth and 42 in. diameter and below that for a depth of 27 ft., a hole 35 in. diameter was pre-excavated for the pile. The effect of the 35 in. pre-excavation is uncertain, but appears to have considerably reduced the dynamic resistance to driving.


Case No. 2004
Figure (35)
Pile: 36 in. Prestressed Concrete, 80 ft. long
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 43.5 ft.
All Curves are End Bearing
J' = 1/3 J
Q = 0.1
J = 0.4
J = 0.6
J = 0.8
Load Test
Ultimate Resistance (tons)
0
100
200
300
400
500
0
5
10
15
20
25
30
35
40
45


Case No. 2004
Figure (36)
Pile: 36 in. Prestressed Concrete, 80 ft. long
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 43.5 ft.
Curve; A End Bearing
C 50% Point - 50% Friction
Q = 0.05
J' = 1/3 J
J = 0.4
J = 0.6
J = 0.6
J = 0.95
Load Test
A
C
A
A
Ultimate Resistance (tons)
Blows / inch
0
100
200
300
400
500
30
35
40
45

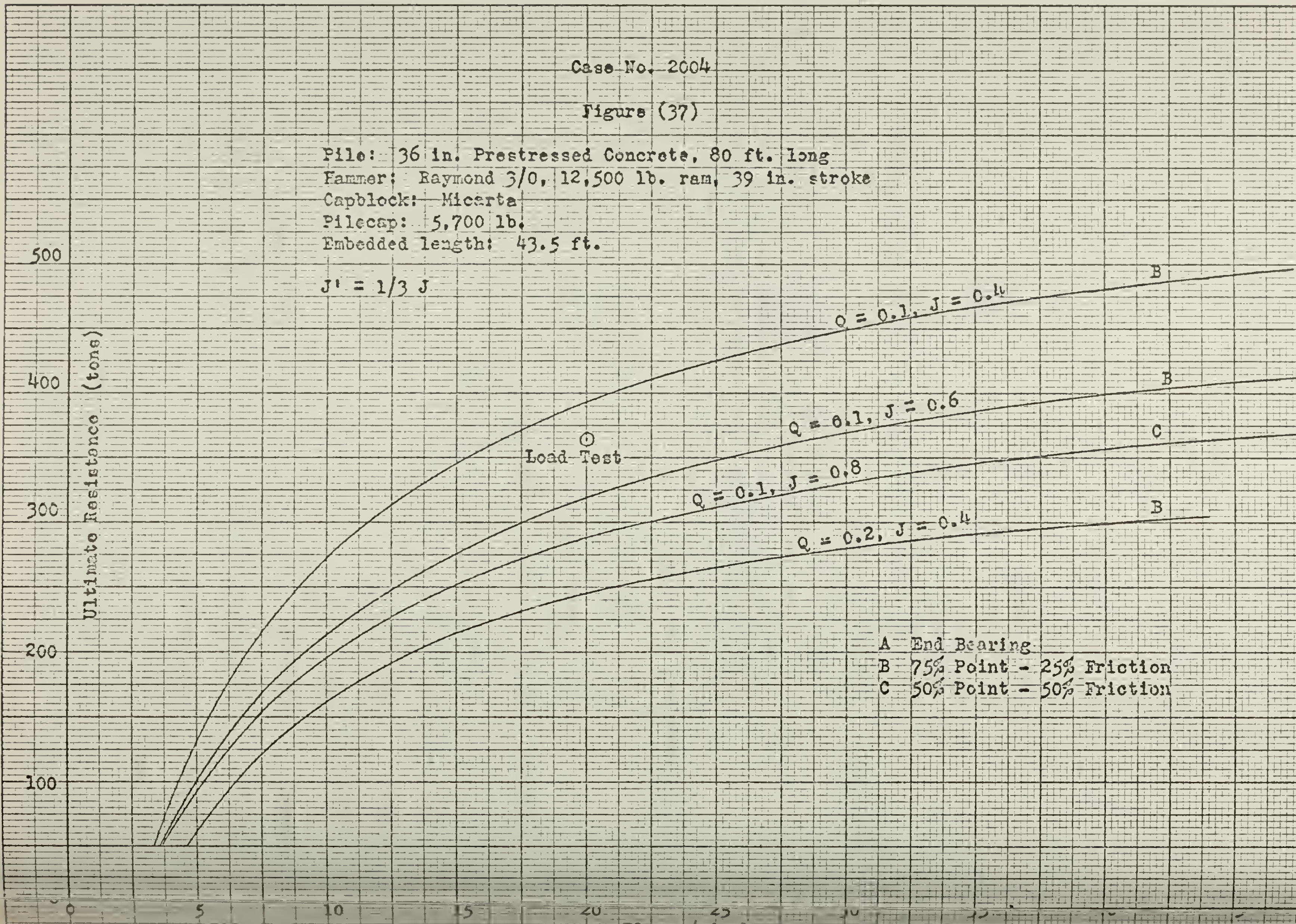

Case No. 2004
Figure (37)
Pile: 36 in. Prestressed Concrete, 80 ft. long
Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke
Capblock: Micarta
Pilecap: 5,700 lb.
Embedded length: 43.5 ft.
J' = 1/3 J
Ultimate Resistance (tons)
500
400
300
200
100
0
5
10
15
20
Q = 0.1, J = 0.4
B
Q = 0.1, J = 0.6
B
C
Load Test
Q = 0.1, J = 0.8
Q = 0.2, J = 0.4
B
A End Bearing
B 75% Point - 25% Friction
C 50% Point - 50% Friction

TABLE J

Case Number 2005

Reference: (18), Test 7-I

Pile: Prestressed concrete, 36 in. diameter, 68 ft. long.

Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke

Embedded length: 31 ft.

Soil: Stiff clay.

Final penetration: 40 blows per inch.

Load test: Tested to failure at 260 tons.

Input data: Same as case 2004, except:

W(3) - W(9)	4,930
K(3) - K(8)	20,900,000

Distribution of resistance:

From calculations based on load test and soil data it is estimated that 43% of the load was carried by the point of the pile failure. For correlation, 50% of the load is considered to be carried by the point. Friction is considered to be distributed over the lower 31 ft. of the pile.

Results: See figure (38).

Correlation is obtained for the following values of Q and J:

Q	J	J'
0.10	1.0	0.33
0.20	0.5	0.17

Case No. 2005

Figure (38)

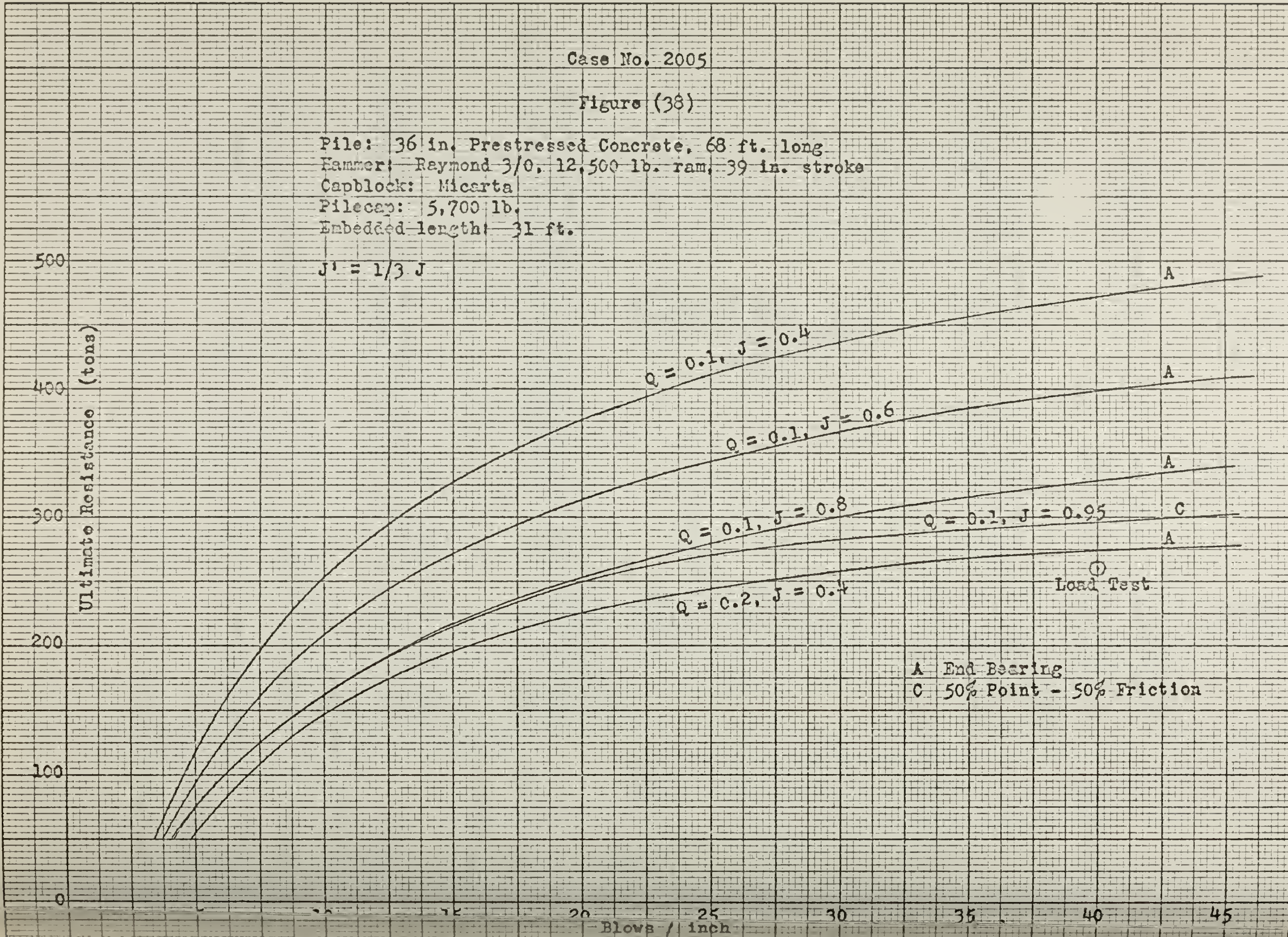


TABLE K

Case Number 6006

Reference: (1)

Pile: Prestressed concrete, 54 in. diameter, 96 ft. long.

Hammer: Raymond 3/0, 12,500 lb. ram, 39 in. stroke, wood capblock.

Embedded length: 60 ft.

Soil: Soft, medium and stiff clay and silty clay with some sand with pile of point in dense, fine, gray sand. See figure (39) for soil profile.

Final penetration: 45 blows last 3/4 in. (60 blows per inch).

Load Test: Tested to 420 tons, failure had not occurred. Bayliss calculated a minimum ultimate load capacity of 498 tons based on load test data and soil characteristics using static formulas (ref. 1).

Input data:

W(1)	12,500
W(2)	6,000
W(3) - W(12)	6,200
V	12.9
K(1)	2,715,000
K(2)	2,025,000
K(3) - K(11)	27,200,000
e_1	0.50
e_2	0.50
T	0.00033

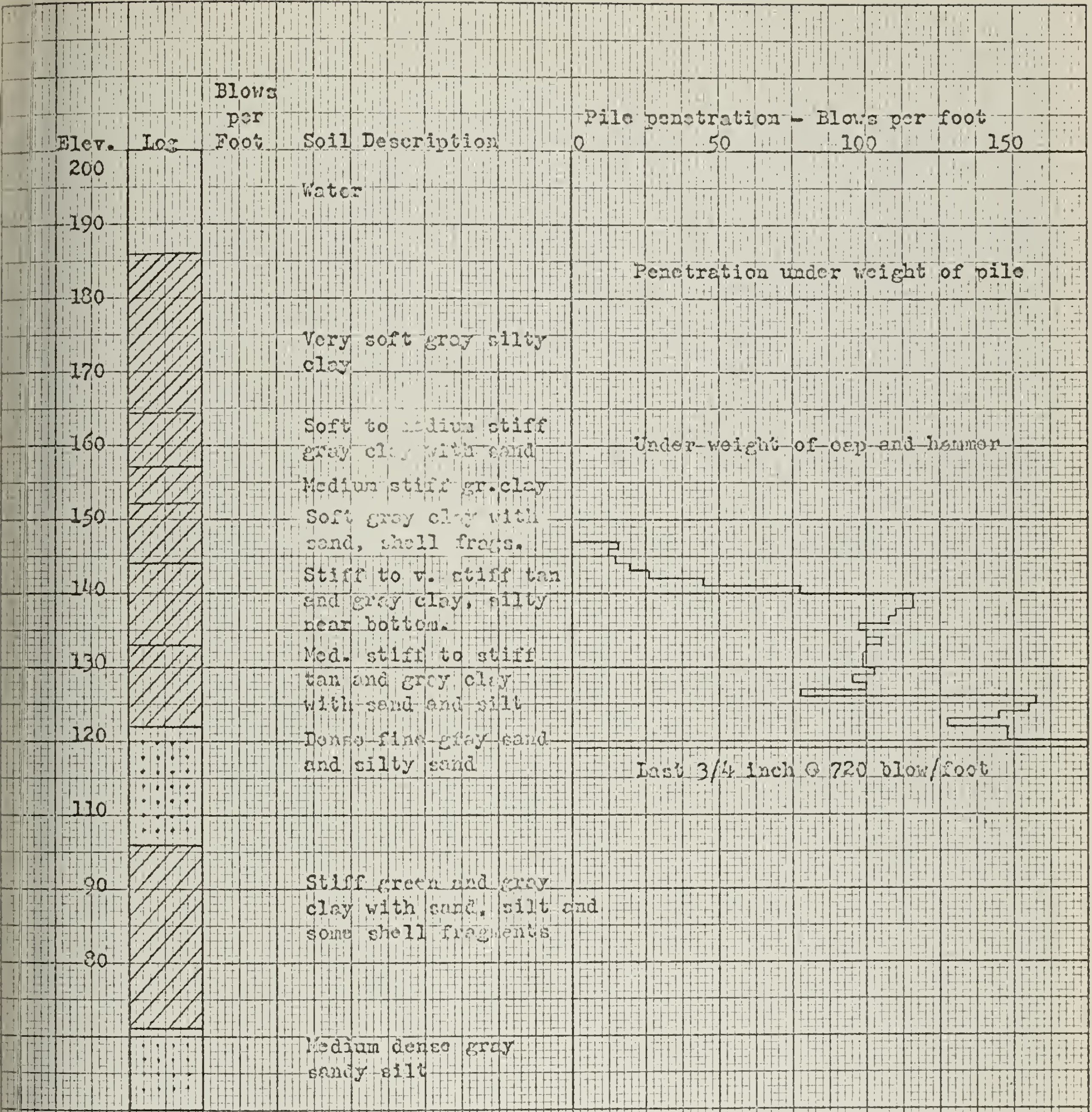
Distribution of resistance:

Based on Bayliss' computations (ref. 1) it is estimated that 50 per cent of the load was carried as end bearing with friction distributed over the lower 60 ft. of the pile.

Results: See figures (39) through (42).

Correlation is obtained for the following values of Q and J:

Q	J	J'	
0.1	0.15	0.05	
0.15	0.10	0.05	
0.05	.2	.05	extrapolated

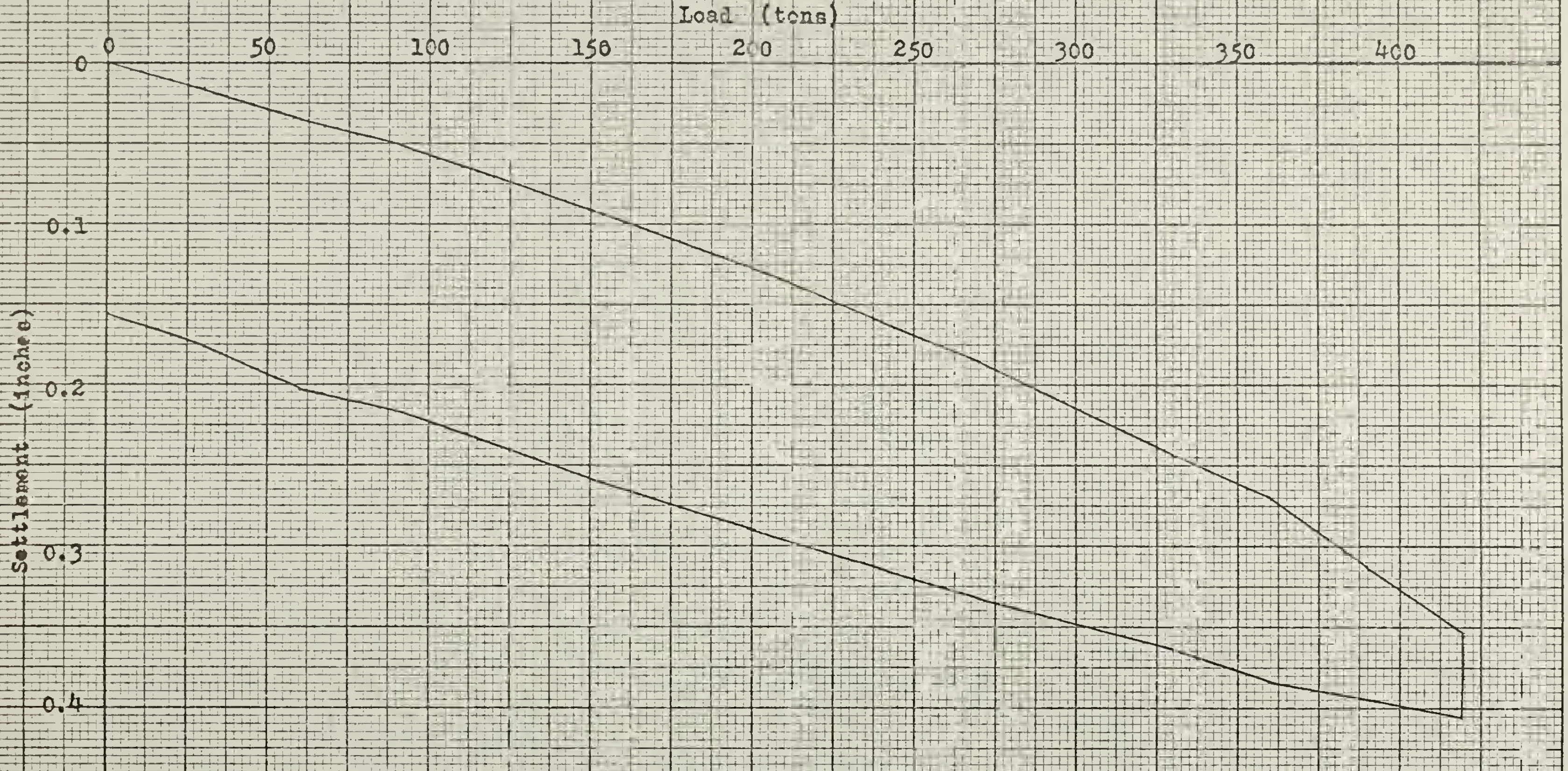


Soil Profile and Driving Record for 54 inch Concrete Pile at Lake Pontchartrain, La. (From ref. 1)

Figure (39)

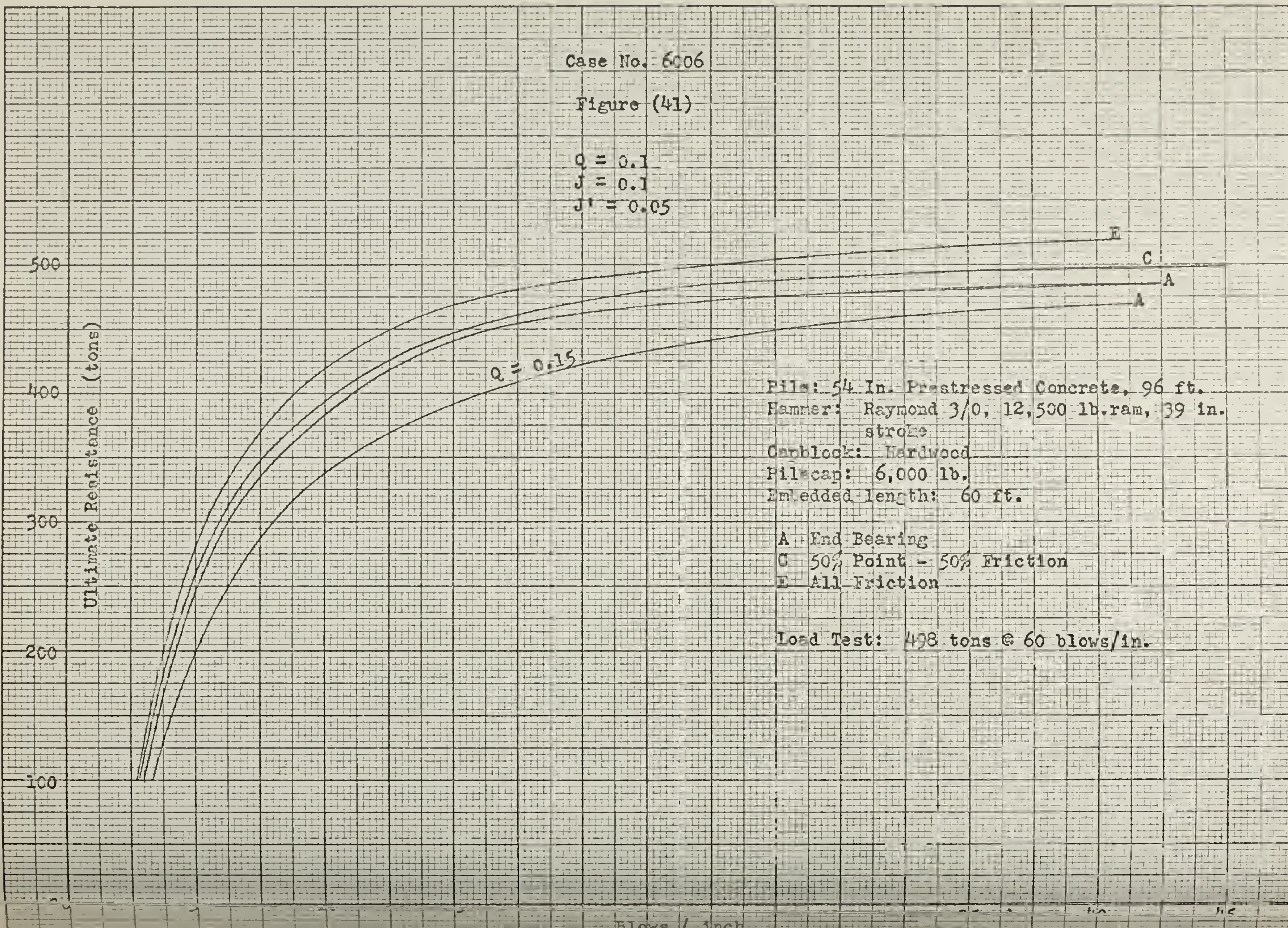
Case No. 6006

Figure (40)



Load Test on 54 in. Diameter Prestressed Concrete Pile

(Data from ref. 1)



Case No. 6006
Figure (41)
Q = 0.1
J = 0.1
J' = 0.05
Ultimate Resistance (tons)
100
200
300
400
500
Q = 0.15
E
C
A
A
Pile: 54 In. Prestressed Concrete, 96 ft.
Hammer: Raymond 3/0, 12,500 lb.ram, 39 in. stroke
Capblock: Hardwood
Pilecap: 6,000 lb.
Embedded Length: 60 ft.
A End Bearing
C 50% Point - 50% Friction
E All Friction
Load Test: 498 tons @ 60 blows/in.

Case No. 6006

Figure (42)

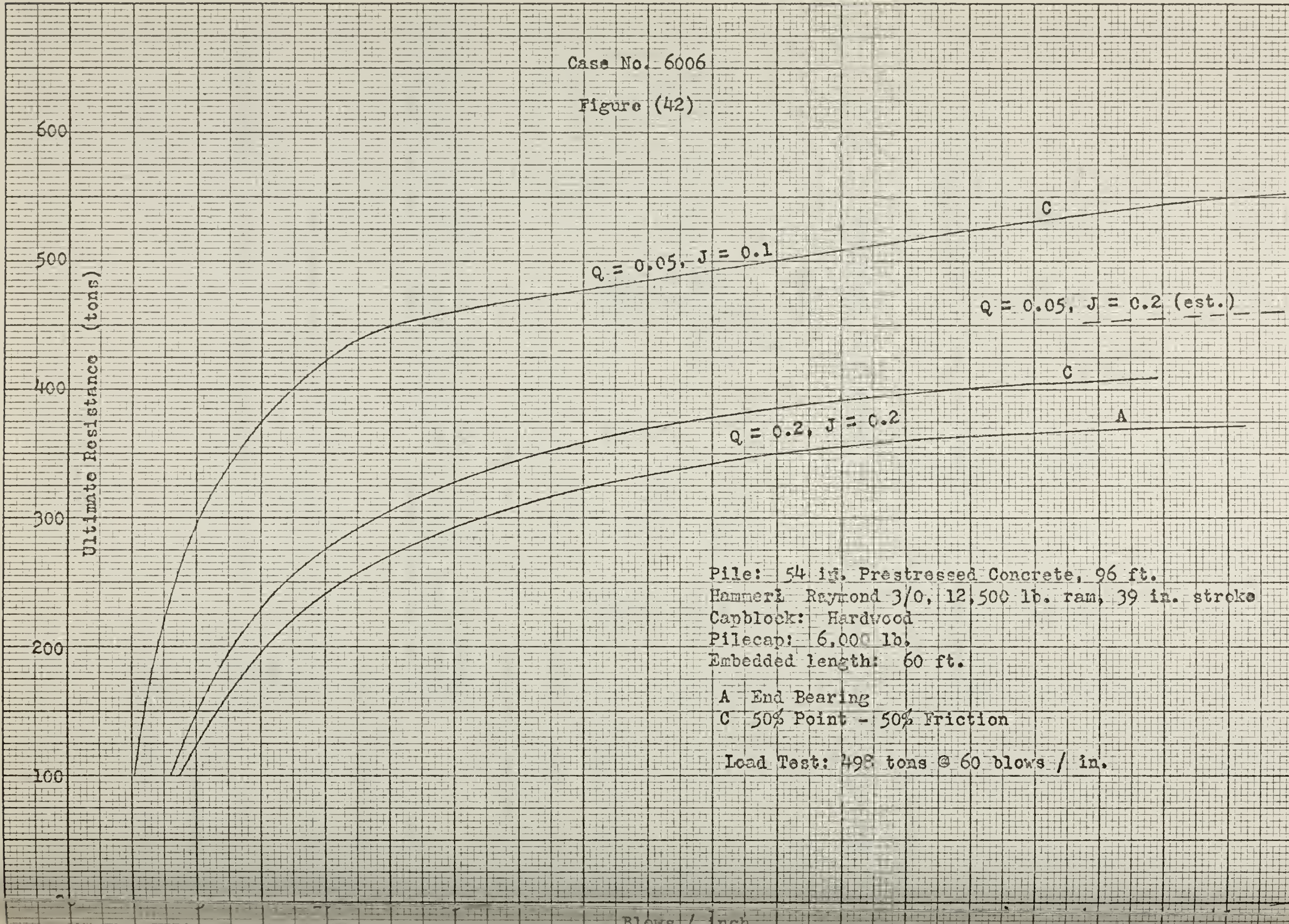


TABLE L

Case Number 6007

Reference: (19)

Pile: Pipe pile, 18 in. diameter, 3/8 in. wall thickness, 80 ft. long.

Hammer: Vulcan No. 1, 5,000 lb. ram, 36 in. stroke.

Embedded length:

Soil: Predominantly plastic clays to a depth of 80 ft. underlain by uniformly graded fine sand having a relative density of approximately 80 per cent. Pile tip stopped 5 ft. above sand.

Final penetration: 2 blows per inch.

Load test: Tested to failure at 81 tons.

Input data:

W(1)	5,000
W(2)	1,000
W(3) - W(12)	562
V	12.4
K(1)	2,715,000
K(2) - K(11)	6,520,000
e_1	0.50
e_2	1.0
T	0.00025

Distribution of resistance: Load carried as friction.

Results: See figure (43).

Correlation is obtained for the following values of Q and J:

Q	J	J'
0.1	0.15	0.05

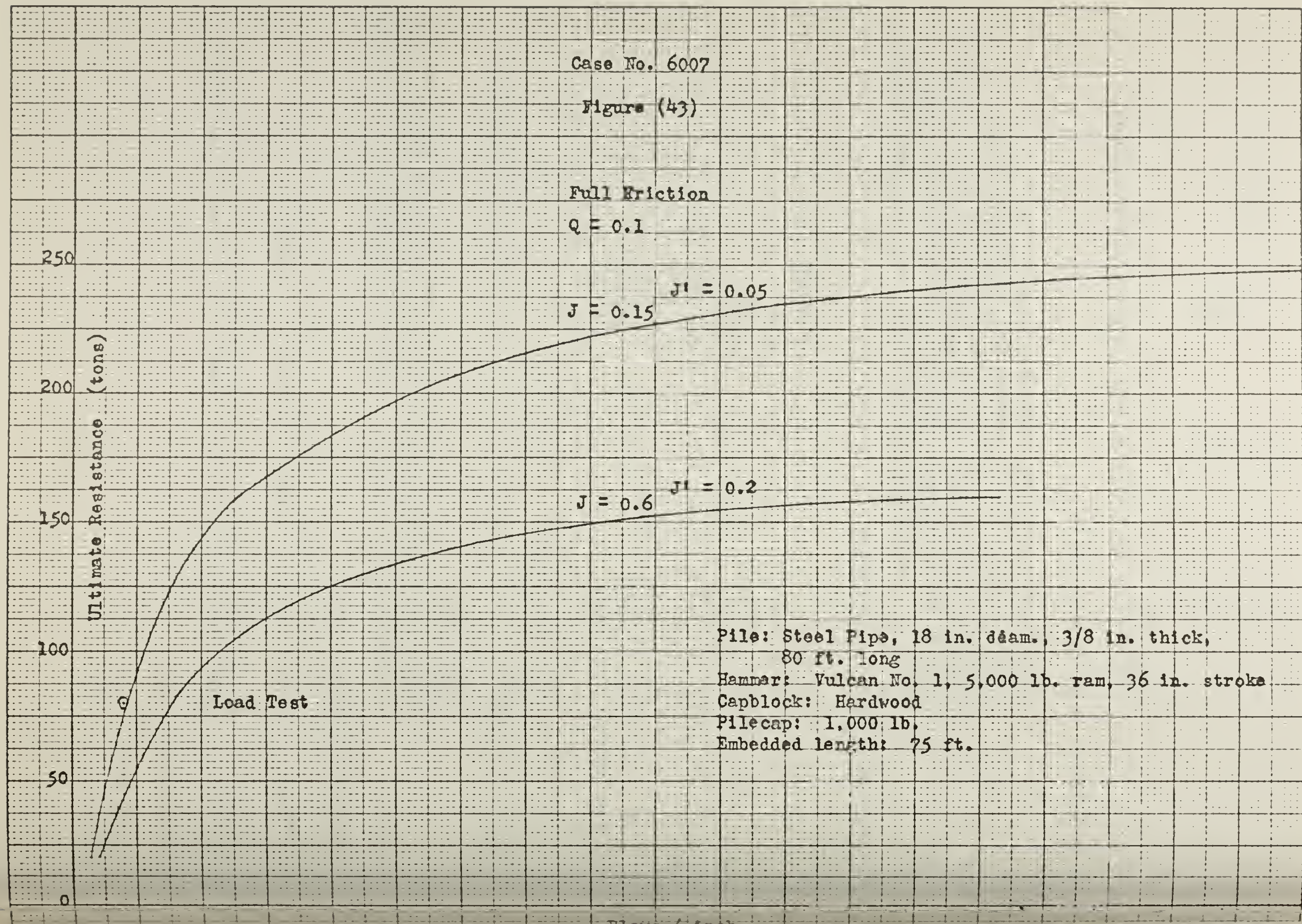

Case No. 6007
Figure (43)
Full Friction
Q = 0.1
J' = 0.05
J = 0.15
J' = 0.2
J = 0.6
Ultimate Resistance (tons)
250
200
150
100
50
0
Load Test
Pile: Steel Pipe, 18 in. diam., 3/8 in. thick,
80 ft. long
Hammer: Vulcan No. 1, 5,000 lb. ram, 36 in. stroke
Capblock: Hardwood
Pilecap: 1,000 lb.
Embedded length: 75 ft.

TABLE M

Case Number 6008

Reference: (19)

Pile: Steel pipe pile, 18 in. diameter, 3/8 in wall thickness, 96 ft. long.

Hammer: Vulcan No. 1, 5,000 lb. ram, 36 in. stroke.

Embedded length: 75 ft.

Soil: Plastic clays to a depth of 80 ft. underlain by a uniformly graded fine sand having a relative density of approximately 80 per cent. The pile tip was driven about 5 ft. into the sand layer.

Final penetration: 226 blows for the last 6 inches (37.7 blows/in.)

Load test: Tested to failure at 244 tons.

Input data:

W(1)	5,000
W(2)	1,000
W(3) - W(12)	676
V	12.4
K(1)	2,715,000
K(2) - K(11)	5,420,000
e_1	0.50
e_2	1.0
T	0.00025

Distribution of resistance:

Based on two different load tests, one with the pile tip in sand and the other (case 6007) with the tip 5 feet above the sand layer, it was determined that 159 tons or 65 per cent of the load was carried by the point (the lower 5 ft.). It is assumed the frictional load is distributed over the embedded length.

Results: See figure (44).

Correlation is obtained with the following values of Q and J:

Q	J	J'
0.05	0.10	0.03
0.10	0.05	0.05 (estimated)

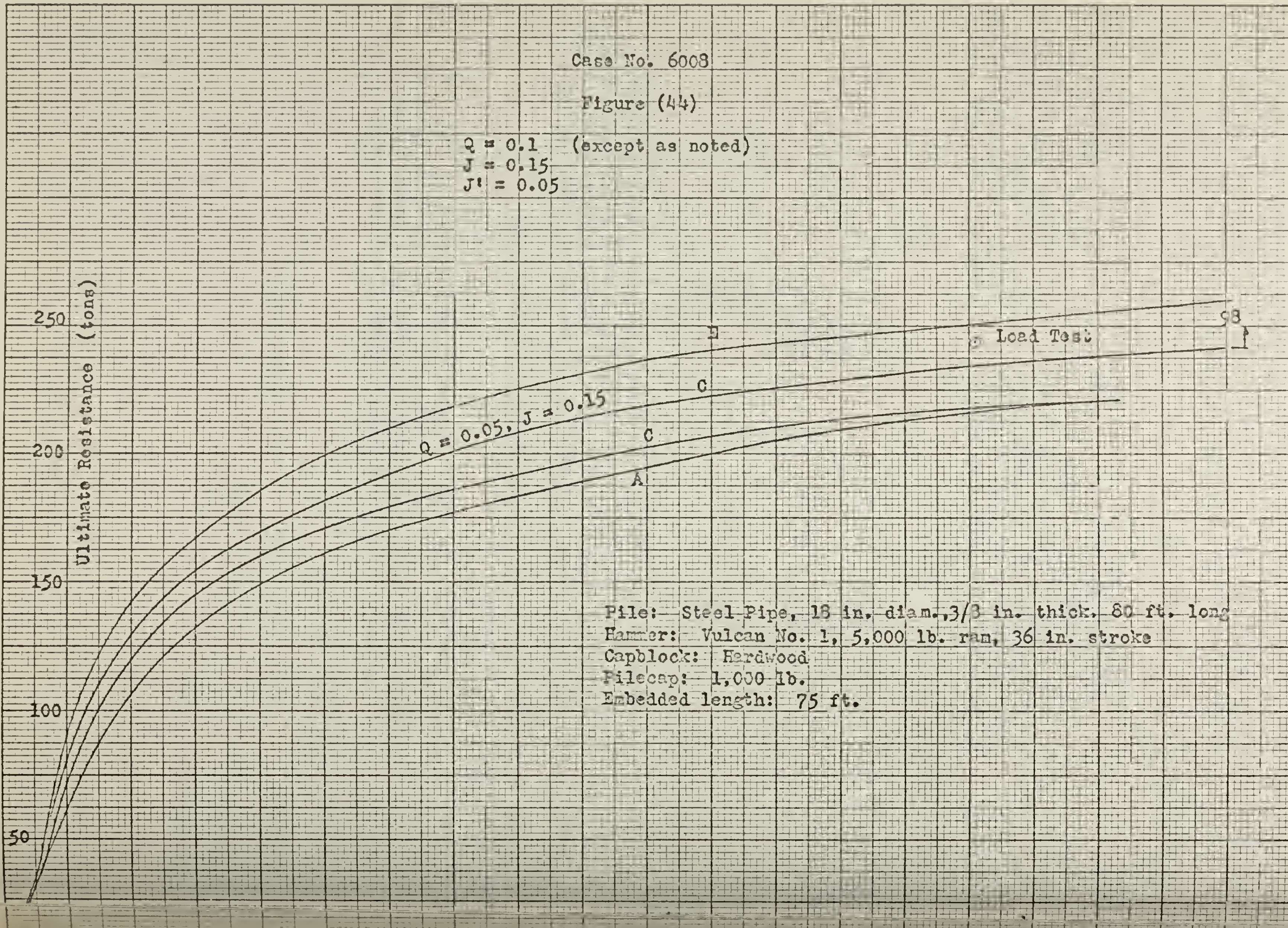

Case No. 6008
Figure (44)
Q = 0.1 (except as noted)
J = 0.15
J' = 0.05
Ultimate Resistance (tons)
250
200
150
100
50
D
C
C
A
Q = 0.05, J = 0.15
Load Test
98
Pile: Steel Pipe, 18 in. diam., 3/8 in. thick, 80 ft. long
Hammer: Vulcan No. 1, 5,000 lb. ram, 36 in. stroke
Capblock: Hardwood
Pilecap: 1,000 lb.
Embedded length: 75 ft.

TABLE N

Case Number 6011

Reference: (20)

Pile: 14-inch H pile weighing 73 lb./ft. and 81 ft. long.

Hammer: Vulcan OR, 9,300 lb. ram, 39 in. stroke.

Embedded length: 80 feet.

Soil: Alternating strata of silts, sandy silts, and silty sands with some interspersed clay strata for a total thickness of about 50 to 60 feet. Clean sands from 40 to 60 feet thick lie beneath the silts. The pile penetrated 32 feet into the sand layer.

Final penetration: 28 blows per foot (2.3 blows/inch)

Load test: The pile buckled under a test load of 344 tons; at this point the pile had reached a gross settlement of 1.2 inches. From the load-settlement diagram it appears that the pile capacity had been reached, based on the usual criteria.

Input data:

W(1)	9,300
W(2)	700
W(3) - W120	648
V	12.9
K(1)	2,715,000
K(2) - K(11)	7,270,000
e_1	0.50
e_2	1.0
T	0.00025

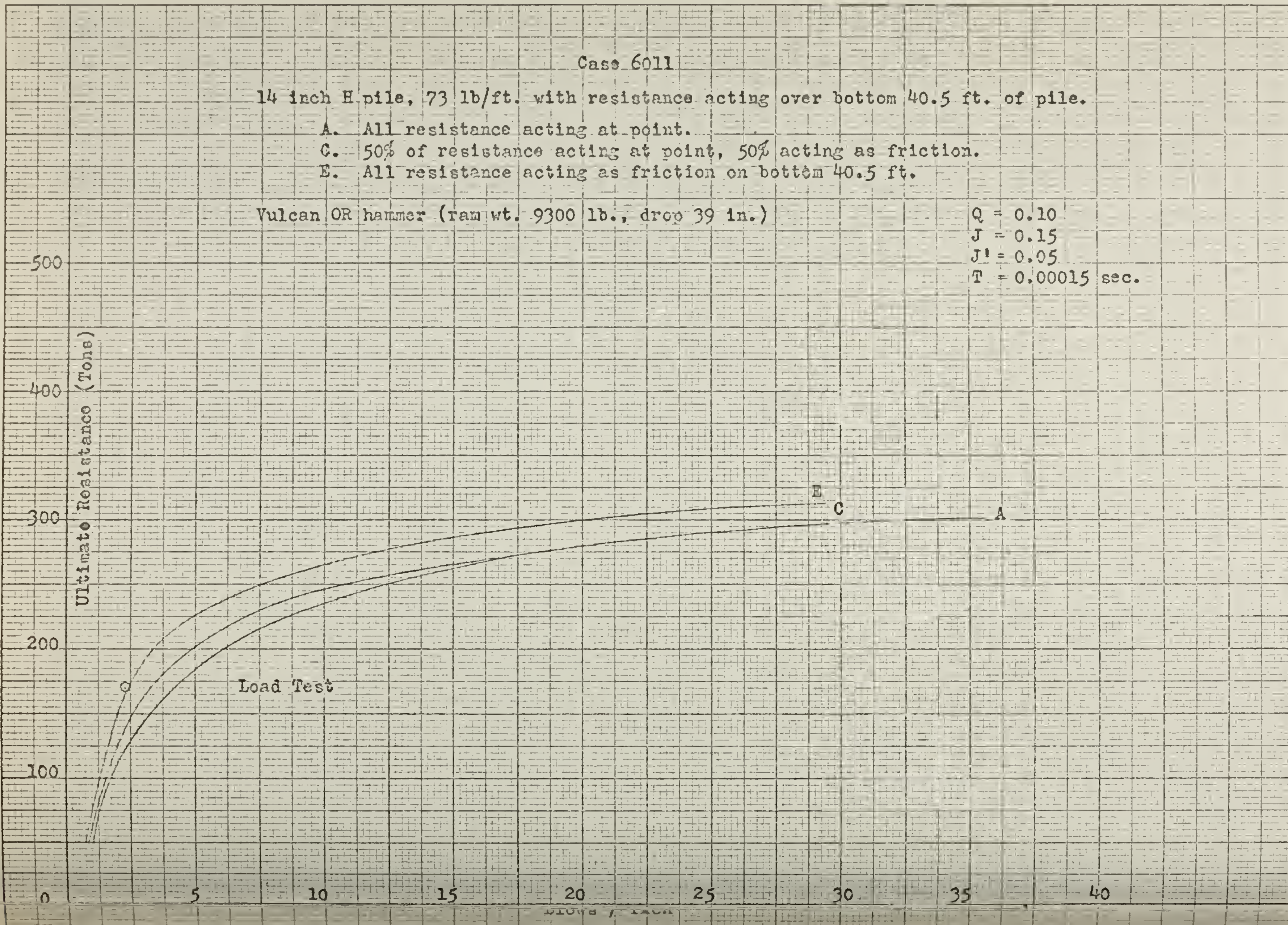
Distribution of resistance:

This pile was equipped with strain rods in order that the distribution of load on the pile could be determined during the load test. The high value of set (low value of blows per inch) for this pile tends to indicate the possibility of liquefaction in the silt layers. Assuming this to be the case, correlation is made on the basis of the load carried to the sand layer which was 170 tons.

Results: See Figures (45) and (46).

Correlation is obtained for the following values of Q and J:

Q	J	J'
.1	.15	.05


Case 6011
14 inch H pile, 73 lb/ft. with resistance acting over bottom 40.5 ft. of pile.
A. All resistance acting at point.
C. 50% of resistance acting at point, 50% acting as friction.
E. All resistance acting as friction on bottom 40.5 ft.
Vulcan OR hammer (ram wt. 9300 lb., drop 39 in.)
Q = 0.10
J = 0.15
J' = 0.05
T = 0.00015 sec.
Ultimate Resistance (Tons)
0
100
200
300
400
500
5
10
15
20
25
30
35
40
Load Test
E
C
A

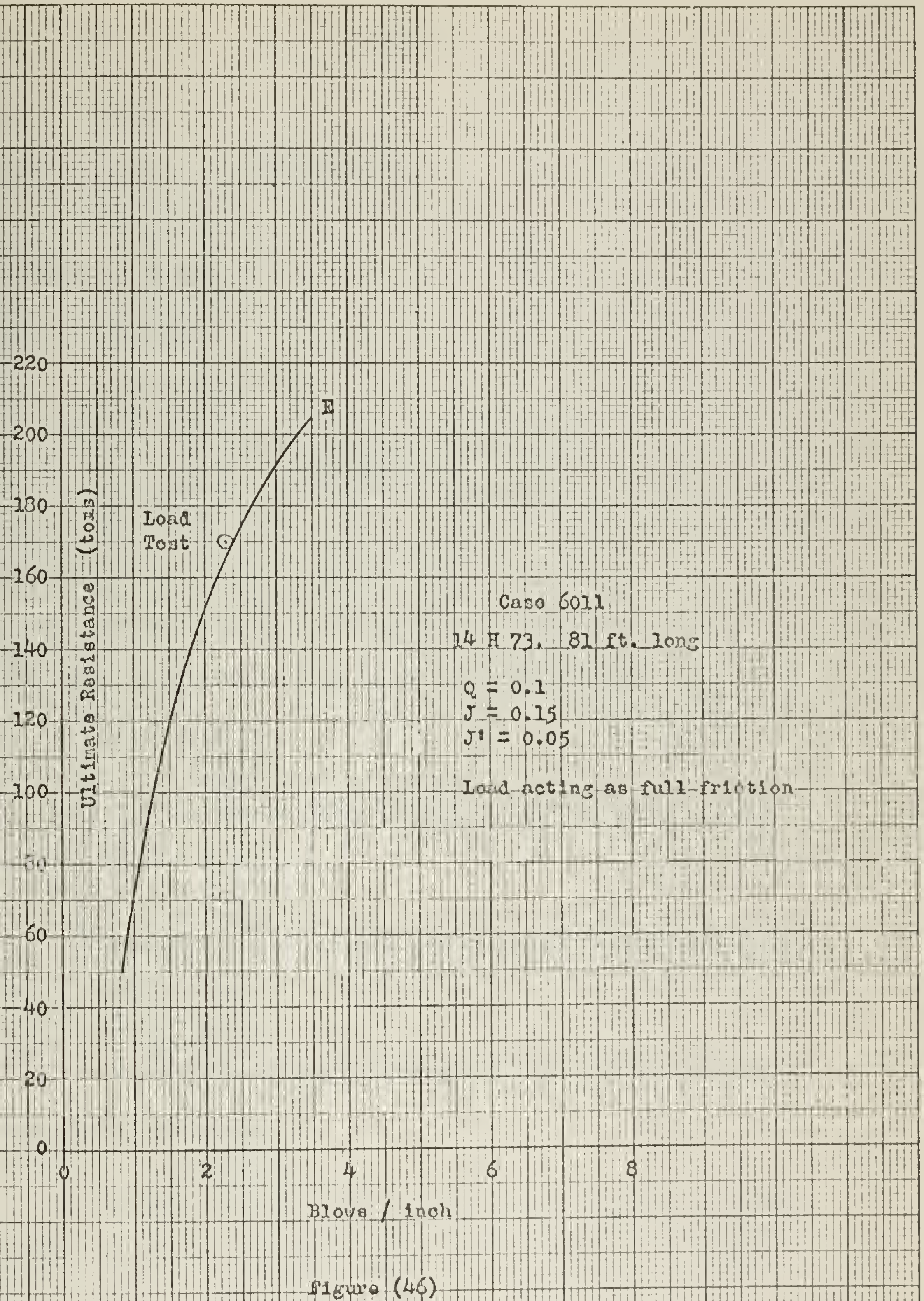

Ultimate Resistance (tons)
220
200
180
160
140
120
100
80
60
40
20
0
E
Load
Test
Case 6011
14 H 73, 81 ft. long
Q = 0.1
J = 0.15
J' = 0.05
Load acting as full-friction
0
2
4
6
8
Blows / inch

Figure (46)

TABLE O

Case: 31

Source: Page 31, reference (16).

Pile: 65' 12BP53

Embedment: 55.4 feet.

Hammer: Vulcan #1

Soil:
10' silty sand
14' loam
26' fine sand, silty sand, and clay
10' fine to medium sand
10' fine to medium sand and gravel
then fine sand

Final penetration: 1.63 blows per inch.

Load test results: 45 tons 17 days after driving.

Results of calculations by the wave equation method:

a. If Q = 0.10
J = 0.10
J'= 0.03
correlation is obtained with a side friction of 2%.

b. If Q = 0.10
J = 0.20
J'= 0.067
correlation is obtained with a side friction of 17%.

c. If Q = 0.20
J = 0.10
J'= 0.03
correlation is obtained with a side friction of 2%.

d. If Q = 0.20
J = 0.20
J'= 0.067
correlation is obtained with a side friction of 32%.

Comments: Hiley Formula gives ultimate resistance of 52 tons.

TABLE DD

Case: 34

Source: Page 34, reference (16).

Pile: 12BP53

Embedment: 47 feet.

Hammer: Vulcan #1

Soil: 35' silty clay
then sand

Final penetration: 4.57 blows per inch.

Load test results: 100 tons, but ultimate resistance not reached.

Results of calculations by the wave equation method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation cannot be obtained for these values for a failure load of 100 tons; however, these are the results:

122 tons for no side friction
146 tons for 25% side friction
185 tons for 50% side friction

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with no side friction.

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with no side friction.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 25%.

Comments: It is not known just what the ultimate resistance would have been if the load test had been carried all the way to failure.

The driven length of the pile was not given; 57' was assumed.

Hiley Formula gives ultimate resistance of 96 tons.

TABLE P

Case: 35

Source: Page 35, reference (16).

Pile: 12BP53

Embedment: 54.4 feet.

Hammer: Vulcan #1

Soil:
15' sandy, silty, clay and loam
10' sand
10' sandy loam
24' sand
then sandy loam

Final penetration: 4.57 blows per inch.

Load test results: 100 tons, but ultimate resistance not reached.

Results of calculations by the wave equation method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation cannot be obtained for these values for a failure load of 100 tons; however, these are the results:

113 tons for no side friction
141 tons for 25% side friction
182 tons for 50% side friction

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 8% for a 100-ton ultimate resistance. Other results are:

114 tons for 25% side friction
151 tons for 50% side friction

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 2% for a 100-ton ultimate resistance. Other results are:

122 tons for a 25% side friction
163 tons for a 50% side friction

Case 35 (continued):

d. If $Q = 0.20$
$J = 0.20$
$J' = 0.067$
correlation is obtained with a side friction of 26% for a 100-ton ultimate resistance. Other results are:

103 tons at 25% side friction
129 tons at 50% side friction

Comments: It is not known just what the ultimate resistance would have been if the load test had been carried all the way to failure.

The driven length of the pile was not given; 65' was assumed.

Hiley Formula gives ultimate resistance of 92 tons.

TABLE Q

Case: 36

Source: Page 36, reference (16)

Pile: 12BP53

Embedment: 49.8 feet.

Hammer: Vulcan #1

Soil: 15' sandy, silty, clay and loam
then sand

Final penetration: 4.15 blows per inch.

Load test results: 100 tons, but ultimate resistance not reached.

Results of calculations by the wave equation method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation cannot be obtained for these values for a failure load of 100 tons. Other results are:

108 tons for no side friction
135 tons for 25% side friction
178 tons for 50% side friction

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 7% for a 100-ton ultimate resistance. Other results are:

115 tons for 25% side friction
143 tons for 50% side friction

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 7% for a 100-ton ultimate resistance. Other values are:

115 tons for 25% side friction
152 tons for 50% side friction

Case 36 (continued):

d. If $Q = 0.20$
$J = 0.20$
$J' = 0.067$
correlation is obtained with a side friction of 32% for a 100-ton ultimate resistance. Other values are:

91 tons at 25% side friction
121 tons at 50% side friction

Comments: It is not known just what the ultimate resistance would have been if the load test had been carried all the way to failure.

The driven length of the pile was not given; 60' was assumed.

Hiley Formula gives ultimate resistance of 92 tons.

TABLE R

Case: 67

Source: Page 67, reference (16)

Pile: 114' 10H42

Embedment: 95 feet

Hammer: Steam, double acting, 950 pounds with stroke of 16 inches.

Soil: Silt and fine sand, pile hit hard strata

Final penetration: 32 blows per inch

Load test results: 60 tons

Results of Calculations by the wave equation:

a. If Q = 0.10
J = 0.15
J' = 0.05
correlation cannot be obtained; however, 100 tons was the calculated value for no side friction.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation cannot be obtained; however, 88 tons was the calculated value for no side friction.

c. If Q = 0.20
J' = 0.05
J = 0.15
correlation is obtained with a side friction of 15%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 40%.

Comments: Time between driving and load testing is unknown.

Hiley Formula gives an ultimate resistance of 25 tons.

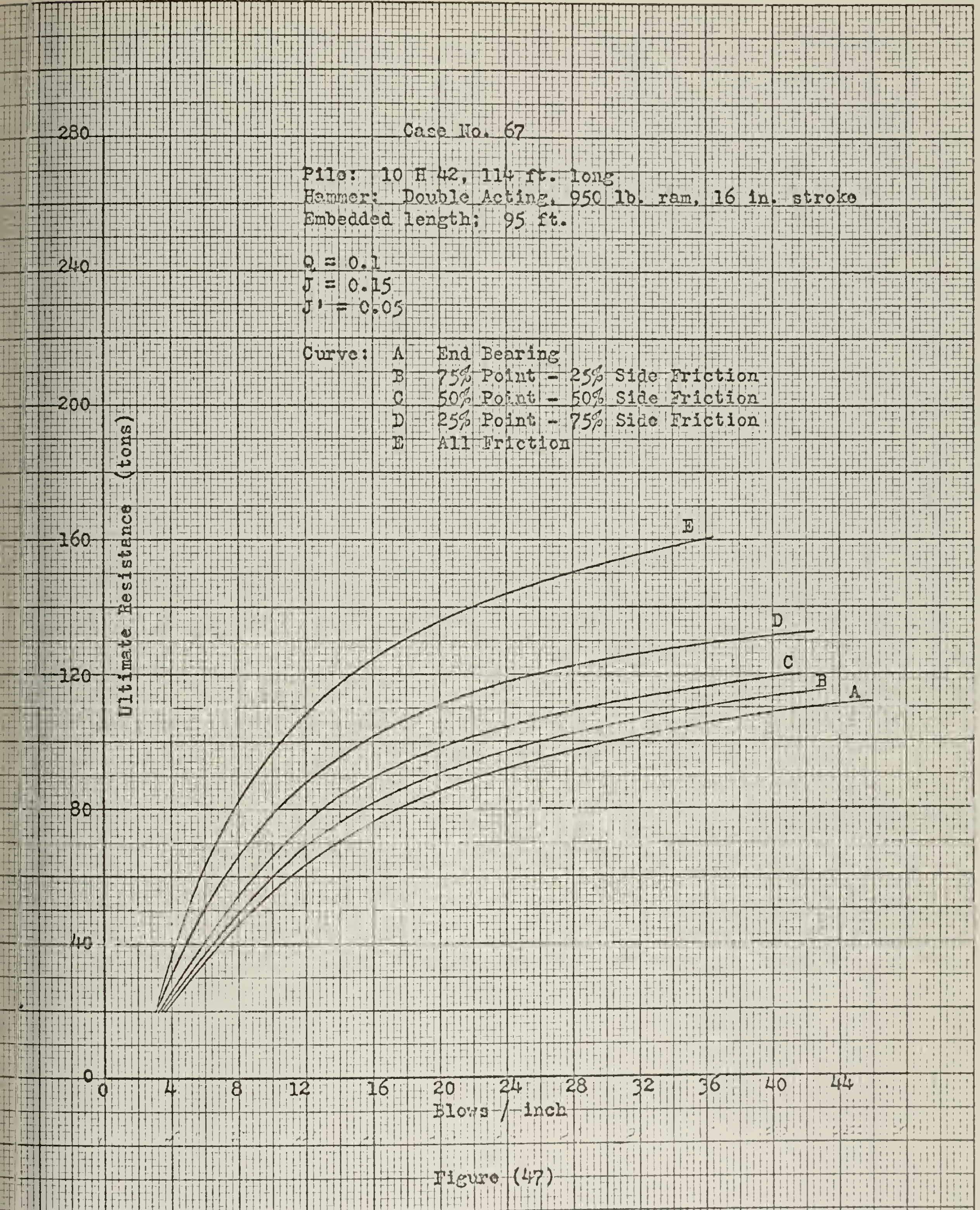
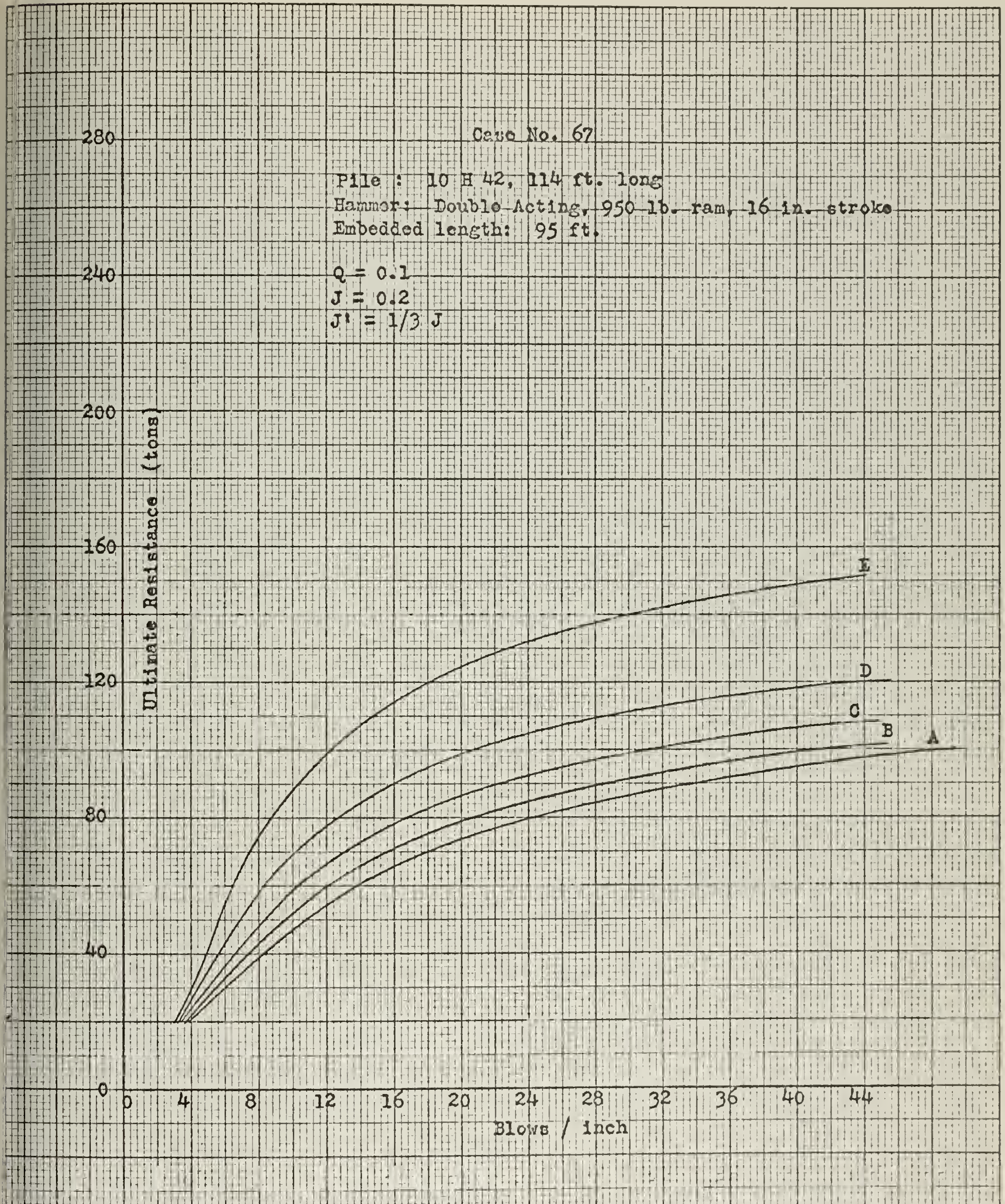

Case No. 67
Pile: 10 H 42, 114 ft. long
Hammer: Double Acting, 950 lb. ram, 16 in. stroke
Embedded length; 95 ft.
Q = 0.1
J = 0.15
J' = 0.05
Curve: A End Bearing
B 75% Point - 25% Side Friction
C 50% Point - 50% Side Friction
D 25% Point - 75% Side Friction
E All Friction
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
4
8
12
16
20
24
28
32
36
40
44

Figure (47)



Case No. 67
Pile : 10 H 42, 114 ft. long
Hammer: Double-Acting, 950 lb. ram, 16 in. stroke
Embedded length: 95 ft.
Q = 0.1
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
4
8
12
16
20
24
28
32
36
40
44
Blows / inch
E
D
C
B
A

Figure (48)

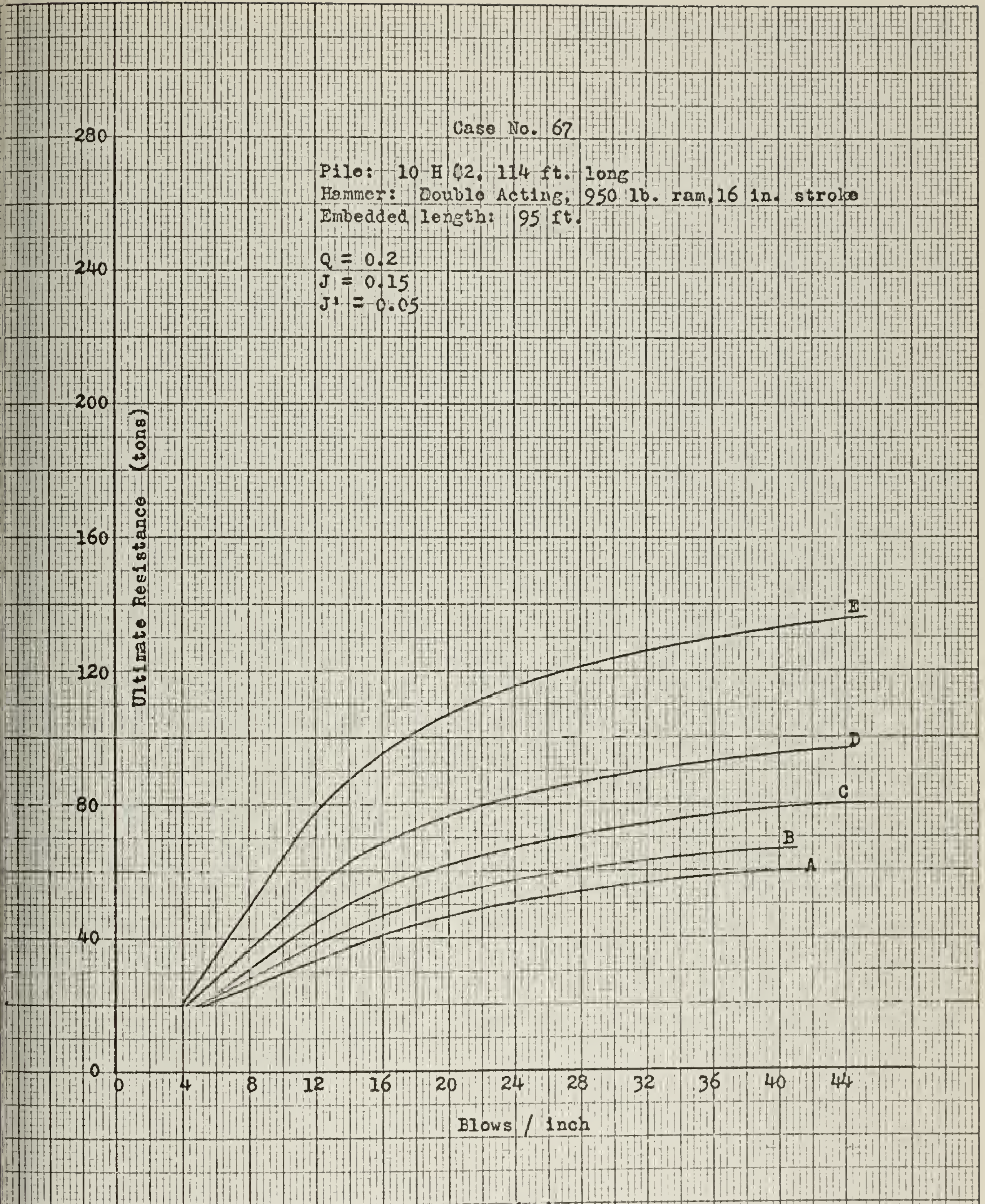

Case No. 67
Pile: 10 H 42, 114 ft. long
Hammer: Double Acting, 950 lb. ram, 16 in. stroke
Embedded length: 95 ft.
Q = 0.2
J = 0.15
J' = 0.05
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
4
8
12
16
20
24
28
32
36
40
44
Blows / inch
A
B
C
D
E

Figure (49)

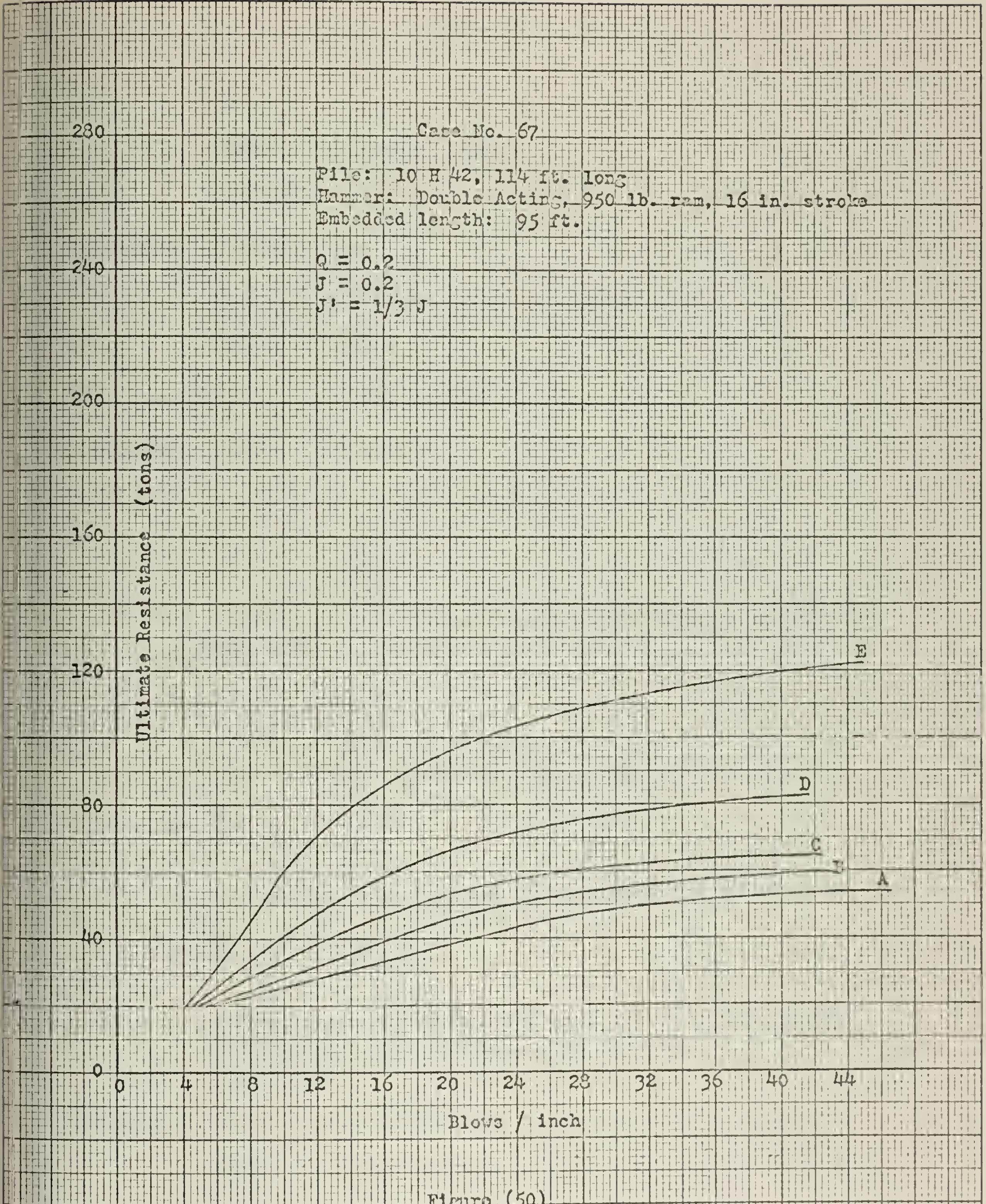

Case No. 67
Pile: 10 H 42, 114 ft. long
Hammer: Double Acting, 950 lb. ram, 16 in. stroke
Embedded length: 95 ft.
Q = 0.2
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
4
8
12
16
20
24
28
32
36
40
44
Blows / inch
A
B
C
D
E

Figure (50)

TABLE S

Case: 71

Source: Page 71, reference (16)

Pile: 20' 10H42

Embedment: 19 feet

Hammer: Gravity, 3,150-pound ram and 5-foot drop

Soil: Sand and Gravel

Final penetration: 2 blows per inch

Load test results: 64 tons

Results of Calculations by the Wave Equation:

a. If $Q = 0.10$
$J = 0.15$
$J' = 0.05$
correlation is obtained at a side friction of both 75% and 100%.

b. If $Q = 0.10$
$J = 0.20$
$J' = 0.067$
correlation is obtained for a side friction of 100%.

c. If $Q = 0.20$
$J = 0.15$
$J' = 0.05$
correlation is obtained for a side friction of 100%.

d. If $Q = 0.20$
$J = 0.20$
$J' = 0.067$
correlation cannot be obtained; however, the ultimate resistance is calculated to be 60 tons for 100% friction, or 40 tons for the end bearing case.

Comments: Time between driving and load testing is unknown.

Hiley equation gives an ultimate resistance of 80 tons.

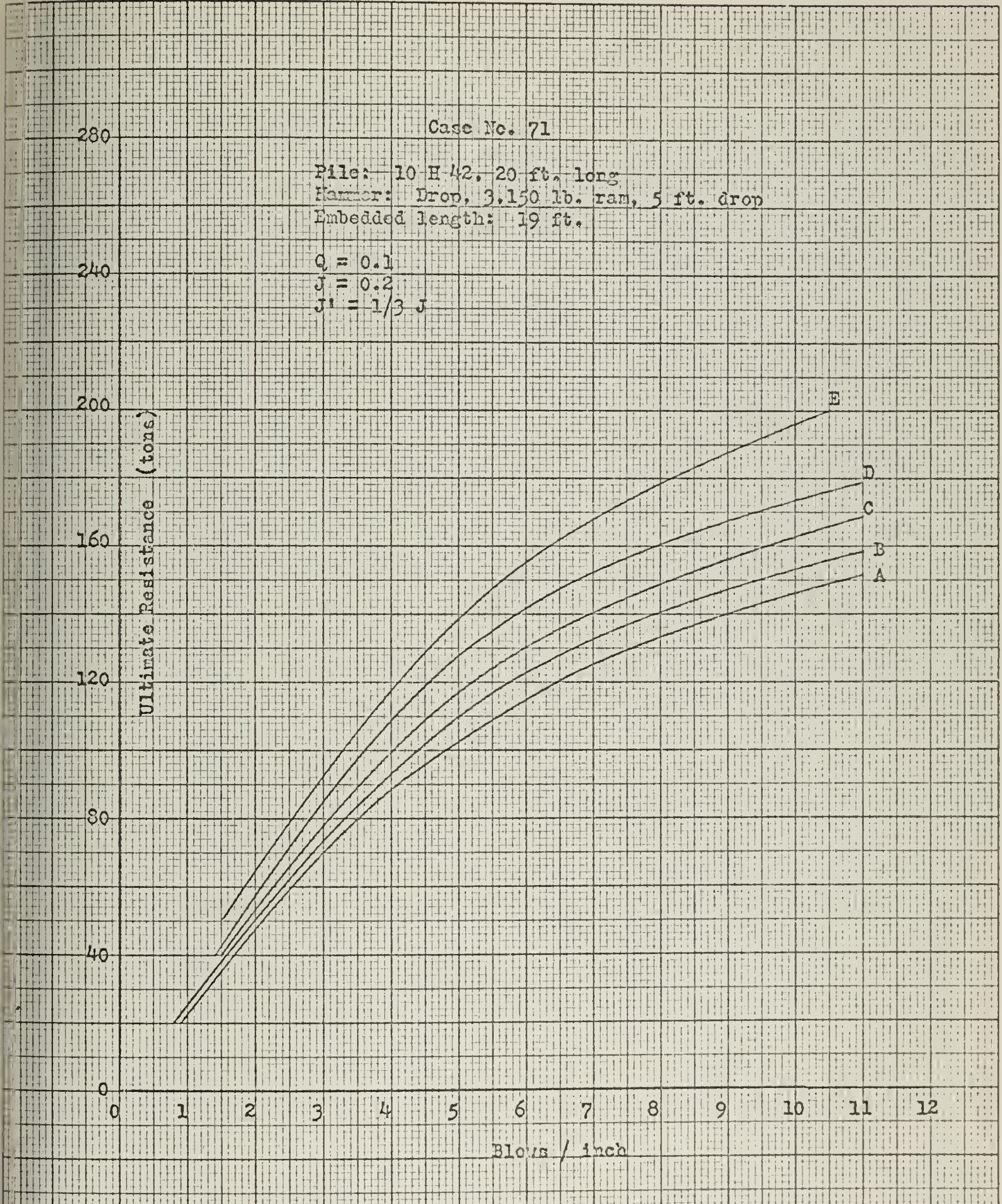

Case No. 71
Pile: 10 H 42, 20 ft. long
Hammer: Drop, 3,150 lb. ram, 5 ft. drop
Embedded length: 19 ft.
Q = 0.1
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
1
2
3
4
5
6
7
8
9
10
11
12
Blows / inch
A
B
C
D
E

Figure (51)

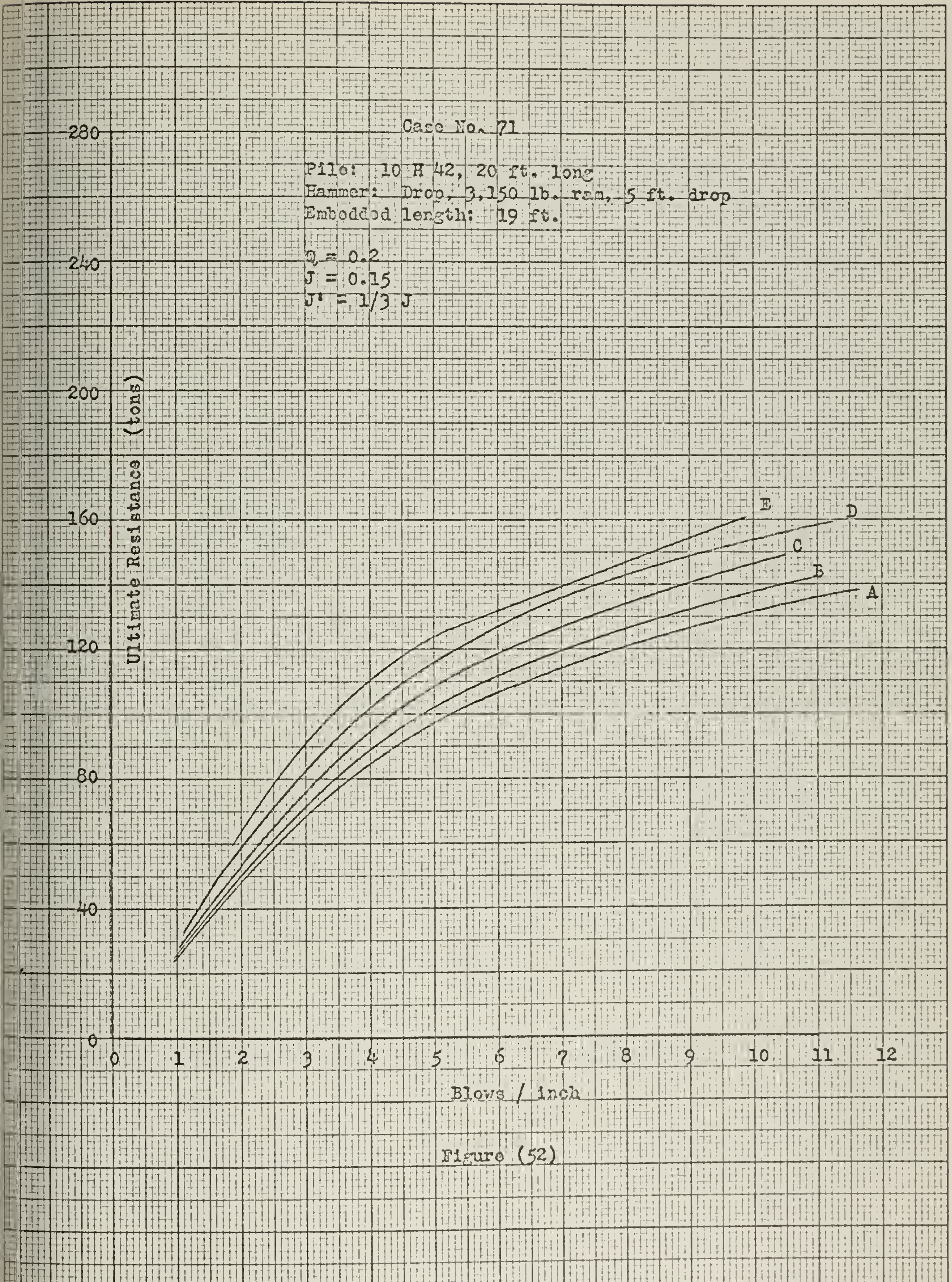

Case No. 71
Pile: 10 H 42, 20 ft. long
Hammer: Drop, 3,150 lb. ram, 5 ft. drop
Embedded length: 19 ft.
Q = 0.2
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
1
2
3
4
5
6
7
8
9
10
11
12

Figure (52)

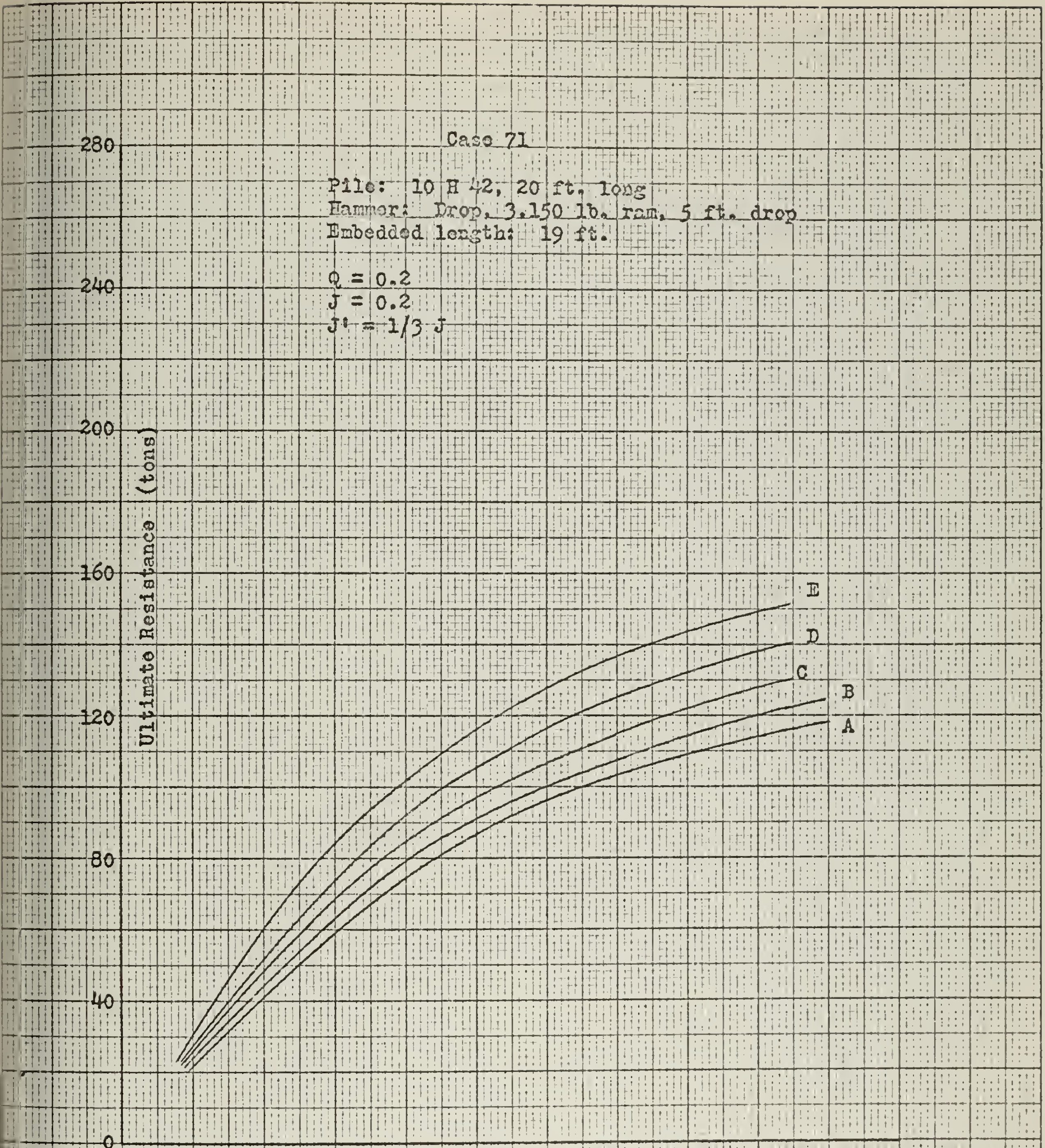

Case 71
Pile: 10 H 42, 20 ft. long
Hammer: Drop, 3,150 lb. ram, 5 ft. drop
Embedded length: 19 ft.
Q = 0.2
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
1
2
3
4
5
6
7
8
9
10
11
12
Blows / inch
E
D
C
B
A

Figure (53)

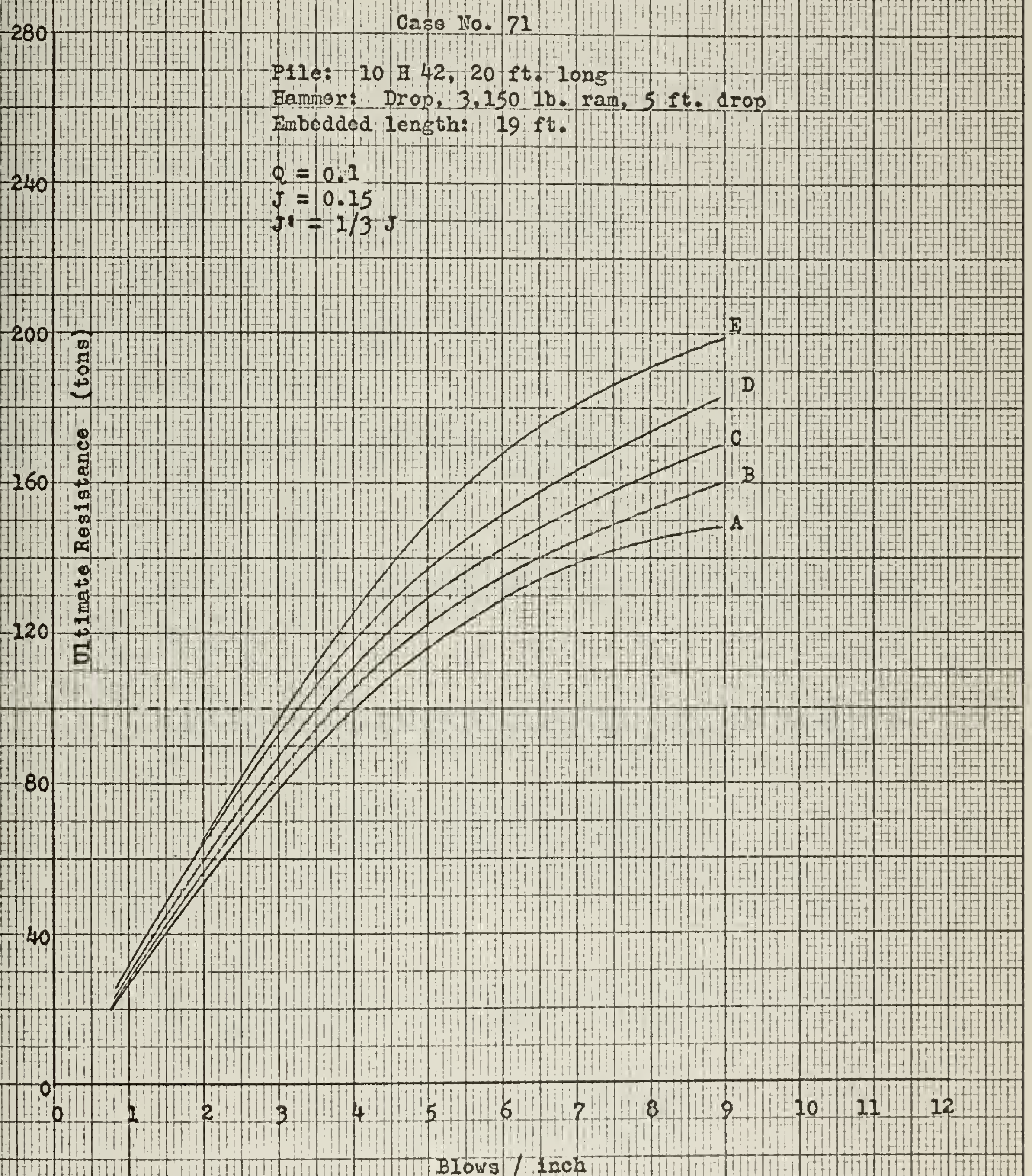

Case No. 71
Pile: 10 H 42, 20 ft. long
Hammer: Drop, 3,150 lb. ram, 5 ft. drop
Embedded length: 19 ft.
Q = 0.1
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
1
2
3
4
5
6
7
8
9
10
11
12

Figure (54)

TABLE T

Case: 75

Source: Page 75, reference (16)

Pile: 30' 10H42

Embedment: 23 feet

Hammer: Drop--3000-pound ram, 5-foot drop

Soil: Fine Sand

Final Penetration: 5 blows per 2 inches

Load Test Results: 90 tons

Results of Calculations by the Wave Equation:

a. If Q = 0.10
   J = 0.15
   J' = 0.05
   correlation is obtained for 100% side friction.

b. If Q = 0.10
   J = 0.20
   J' = 0.067
   correlation cannot be obtained for a failure load of 90 tons; however, these are the results:

   55 tons for no side friction
   64 tons for 50% side friction
   70 tons for 100% side friction

c. If Q = 0.20
   J = 0.15
   J' = 0.05
   correlation cannot be obtained for a failure load of 90 tons; however, these are the results:

   54 tons for no side friction
   60 tons for 50% side friction
   60 tons for 100% side friction

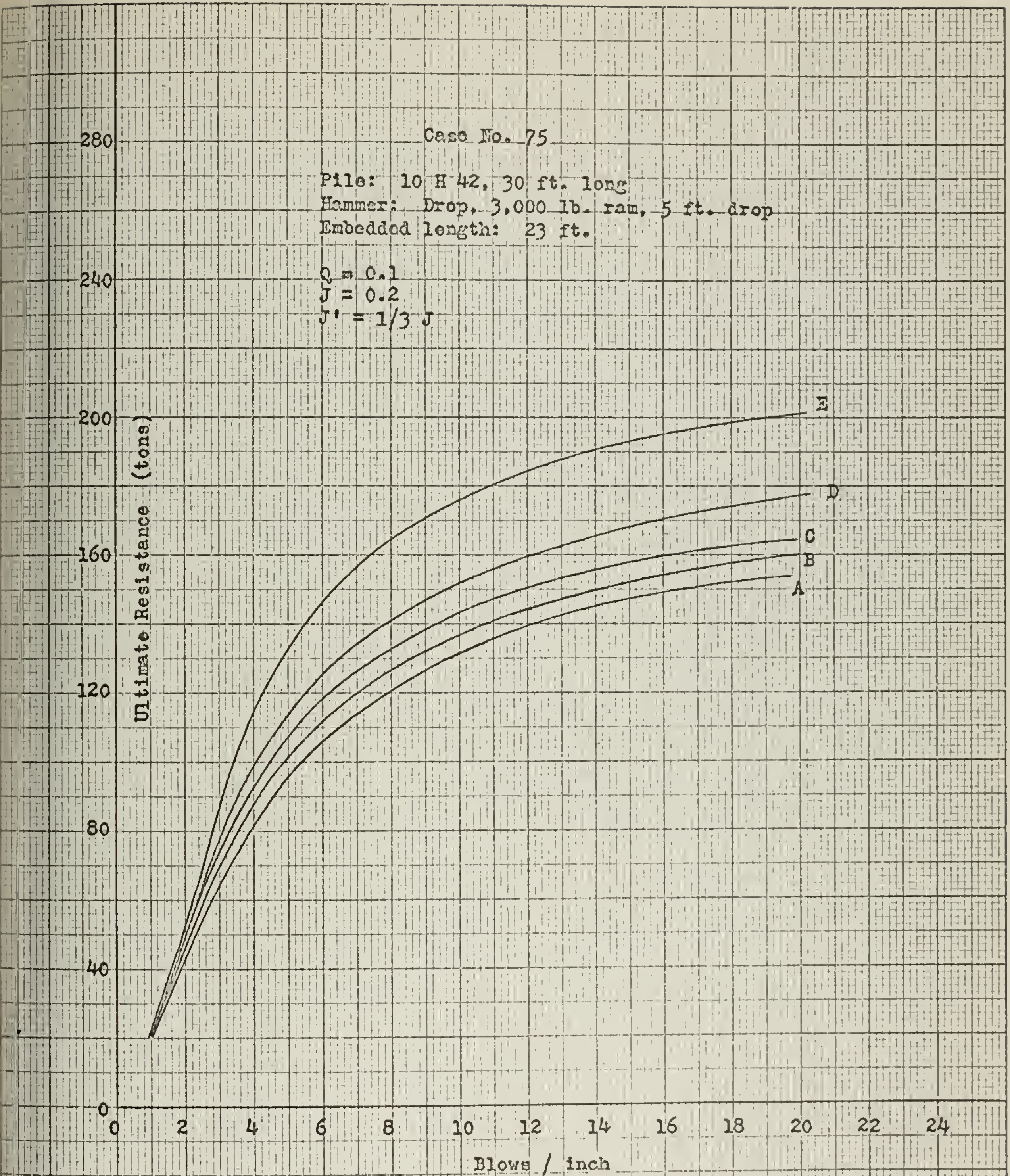
d. If Q = 0.20
   J = 0.20
   J' = 0.067
   correlation cannot be obtained for a failure load of 90 tons; however, these are the results:

TABLE T (cont'd.)

48 tons for no side friction
55 tons for 50% side friction
60 tons for 100% side friction

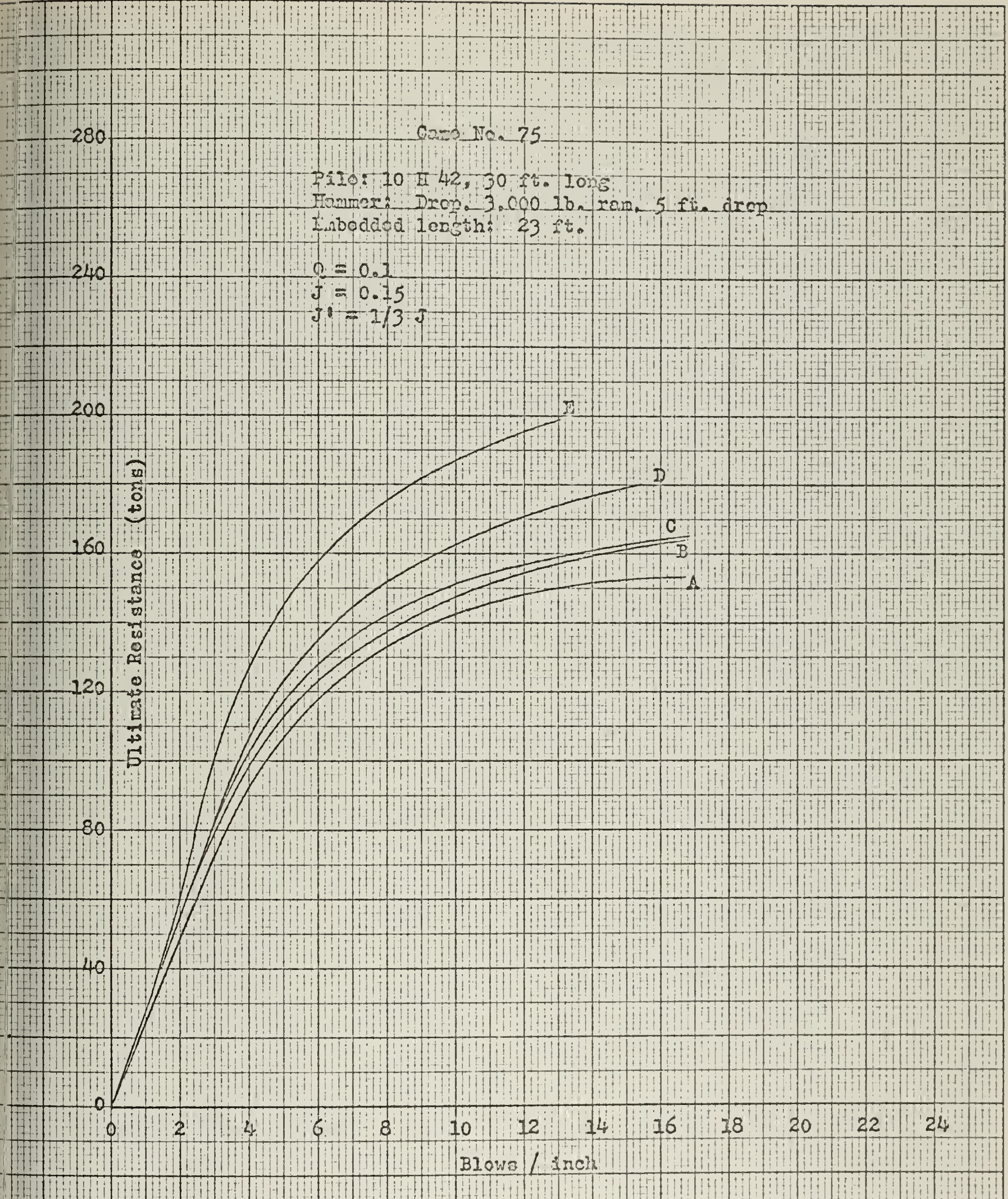
Comments: Time between driving and load testing unknown.

Hiley formula gives Ru of 80 tons.



Case No. 75
Pile: 10 H 42, 30 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.1
J = 0.2
J' = 1/3 J
E
D
C
B
A
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch

Figure (55)



Case No. 75
Pile: 10 H 42, 30 ft. long
Hammer: Drop. 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.1
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
2
4
6
8
10
12
14
16
18
20
22
24

Figure (56)

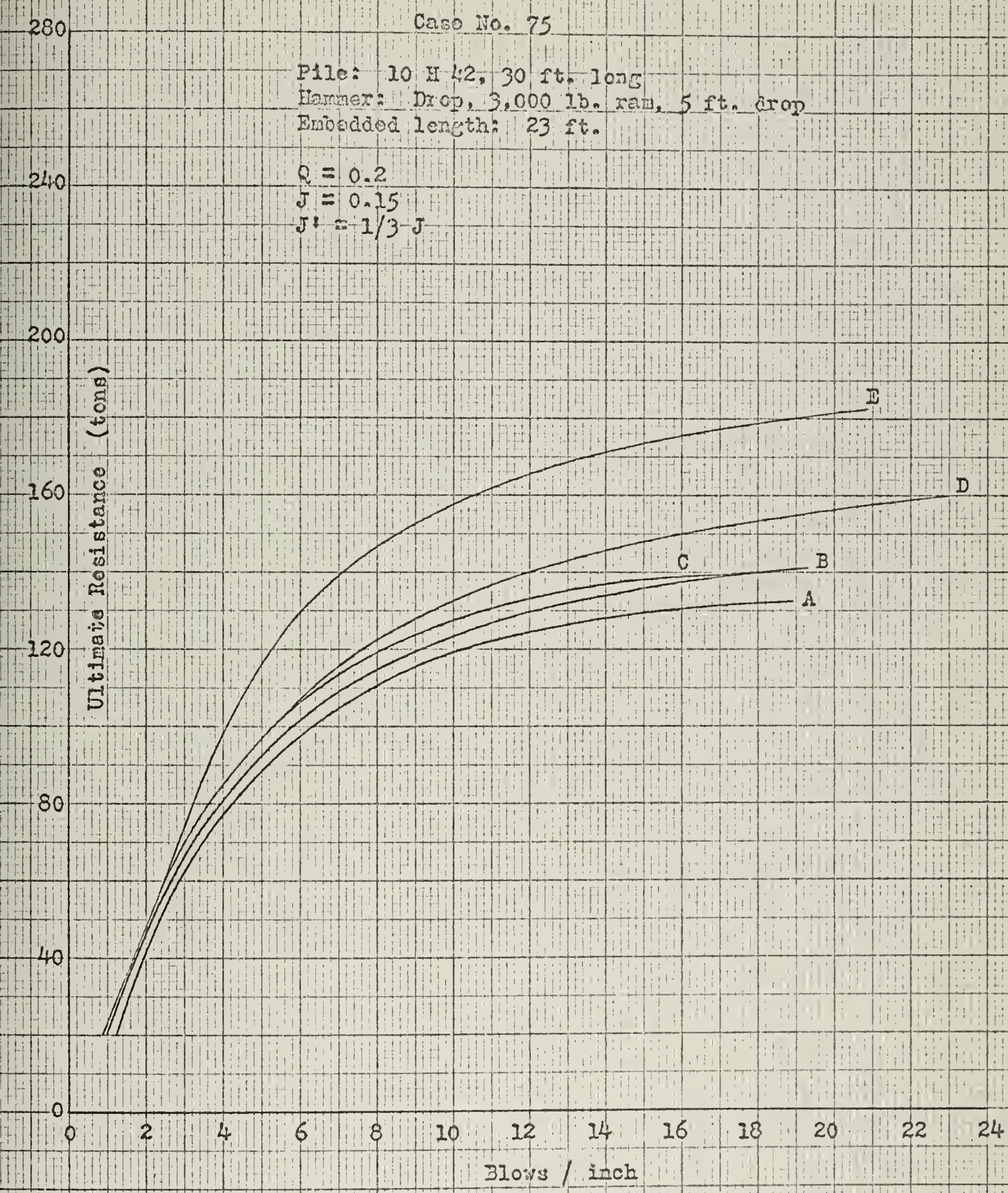

Case No. 75
Pile: 10 H 42, 30 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.2
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E

Figure (57)

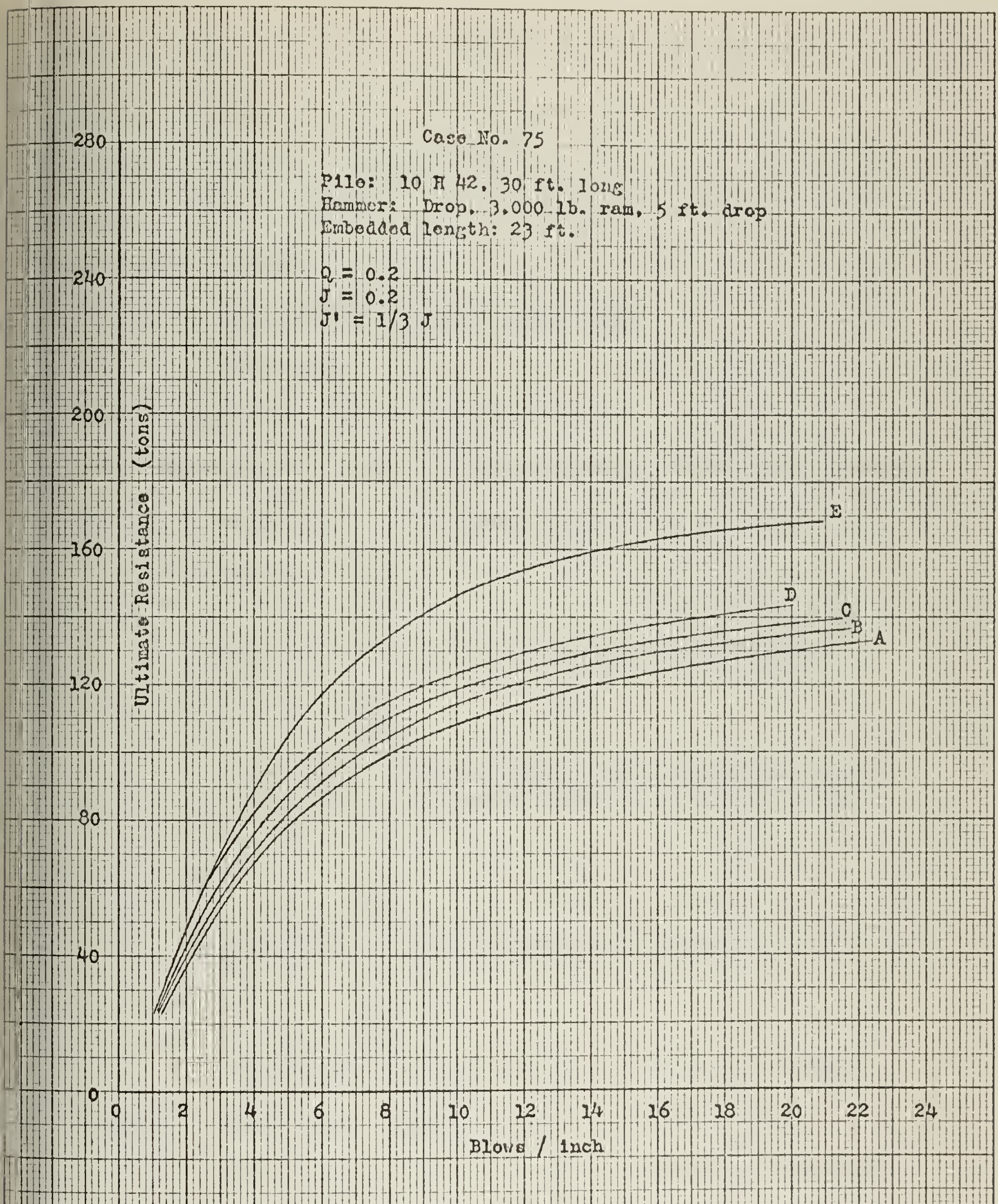

Case No. 75
Pile: 10 H 42, 30 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.2
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24

Figure (58)

TABLE U

Case: 76

Source: Page 76, reference (16)

Pile: 40' 10H42

Embedment: 34 feet

Hammer: Drop--3000-pound ram, 5-foot drop

Soil: Fine Sand

Final Penetration: 2 blows per inch

Load Test Results: 75 tons on second loading, 50 tons first loading

Results of Calculations by the Wave Equation:

a. If Q = 0.10
J = 0.15
J' = 0.05
correlation is obtained with side friction of 25%.

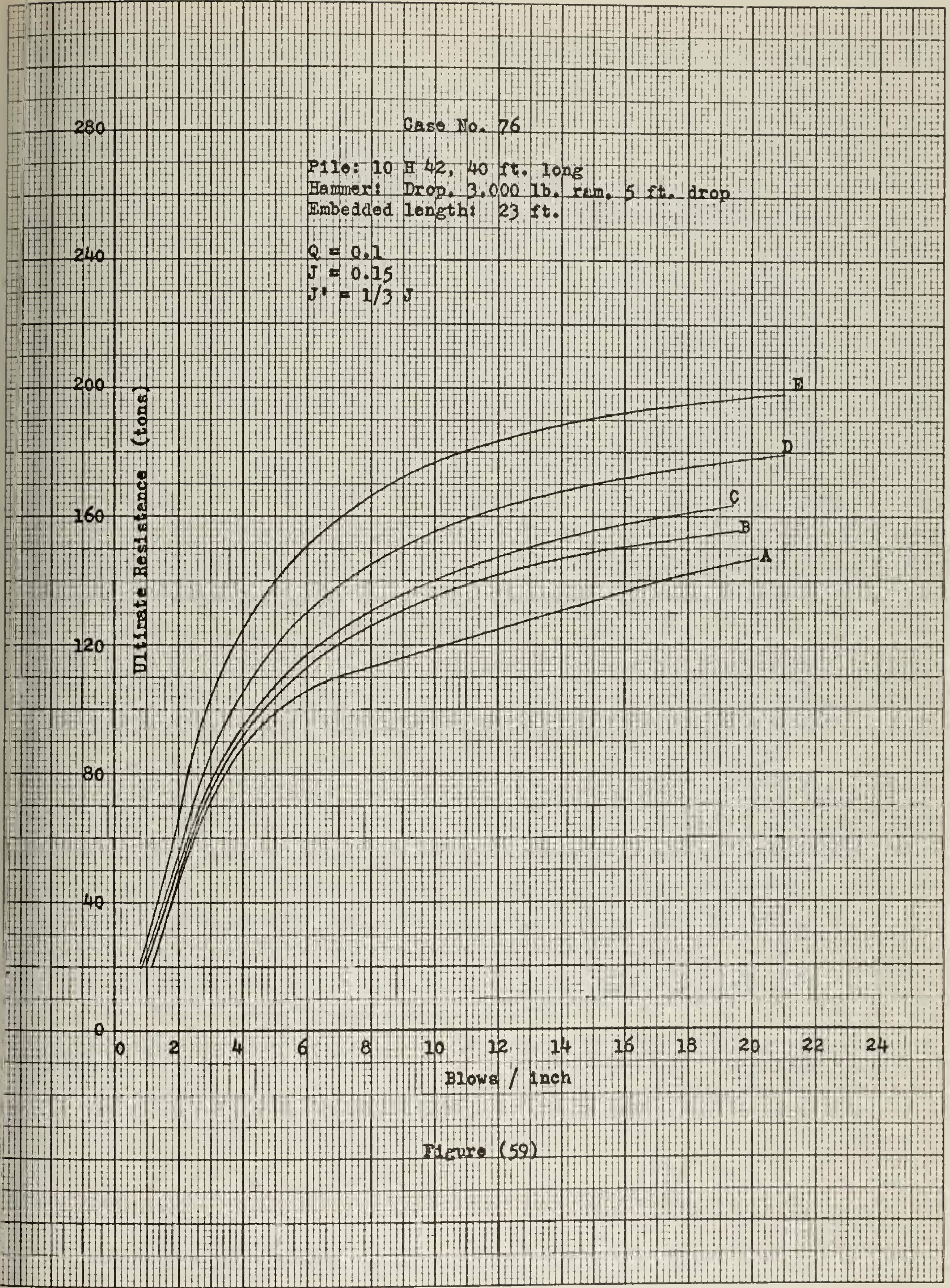
b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained for a side friction of 35% (the difference in this case between 25% and 50% friction is only 1 ton).

c. If Q = 0.20
J = 0.15
J' = 0.05
correlation is obtained with a side friction of 75%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 90%.

Comments: Time between driving and load testing unknown.

Hiley formula gives an ultimate resistance of 60 tons.



Case No. 76
Pile: 10 H 42, 40 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.1
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24

Figure (59)

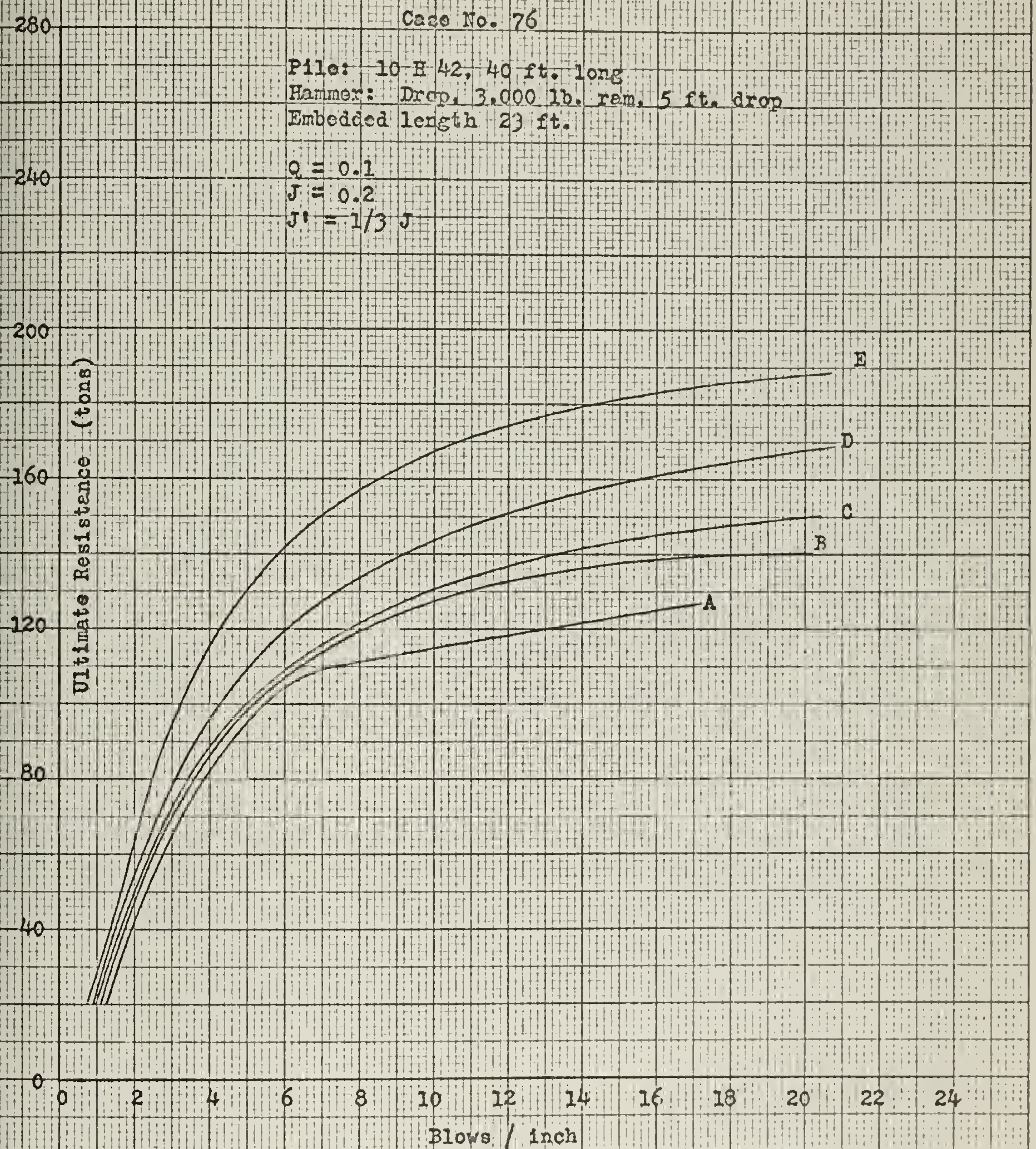

Case No. 76
Pile: 10 H 42, 40 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length 23 ft.
Q = 0.1
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
A
B
C
D
E

Figure (60)

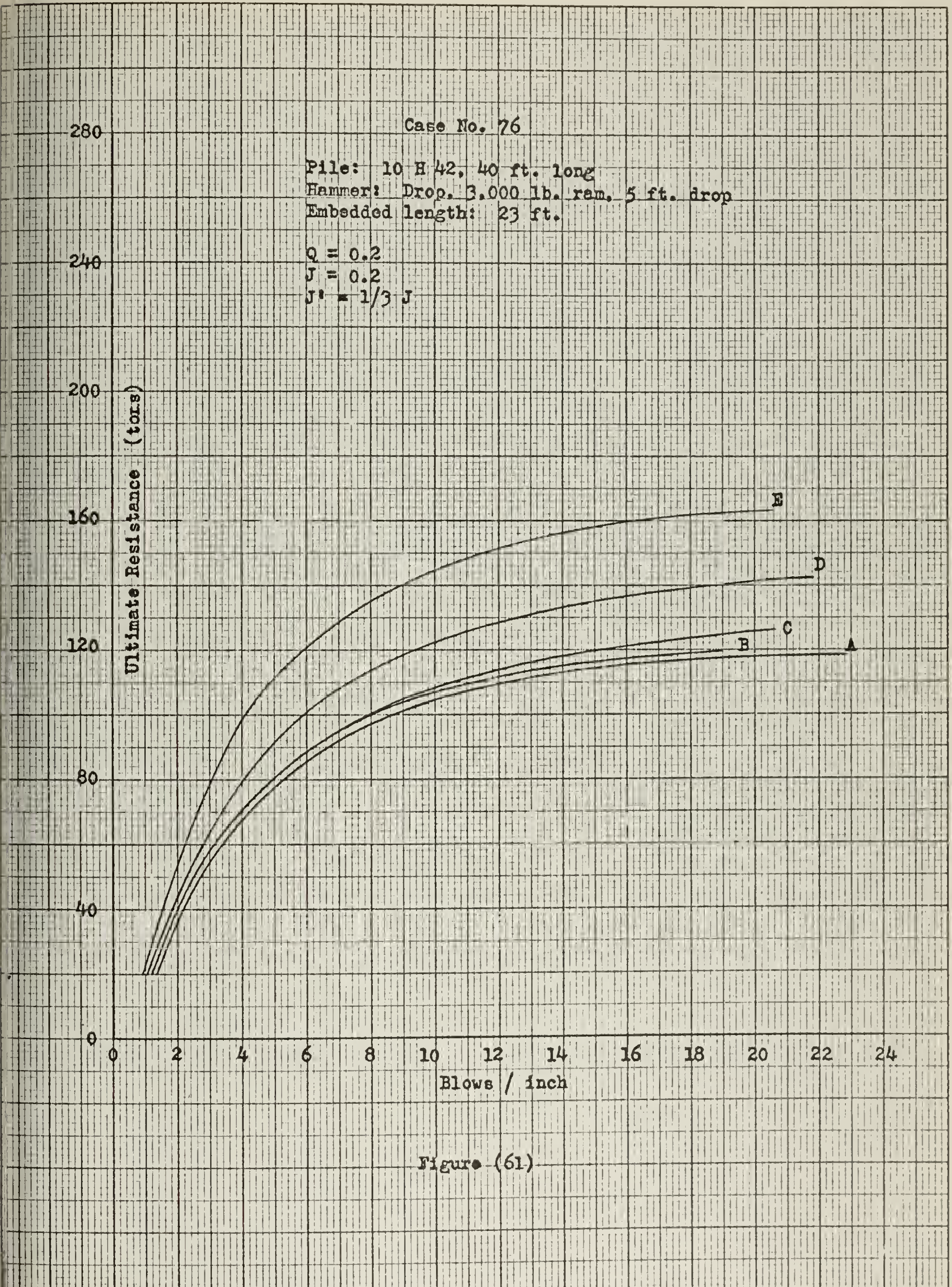

Case No. 76
Pile: 10 H 42, 40 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.2
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
A
B
C
D
E

Figure (61)

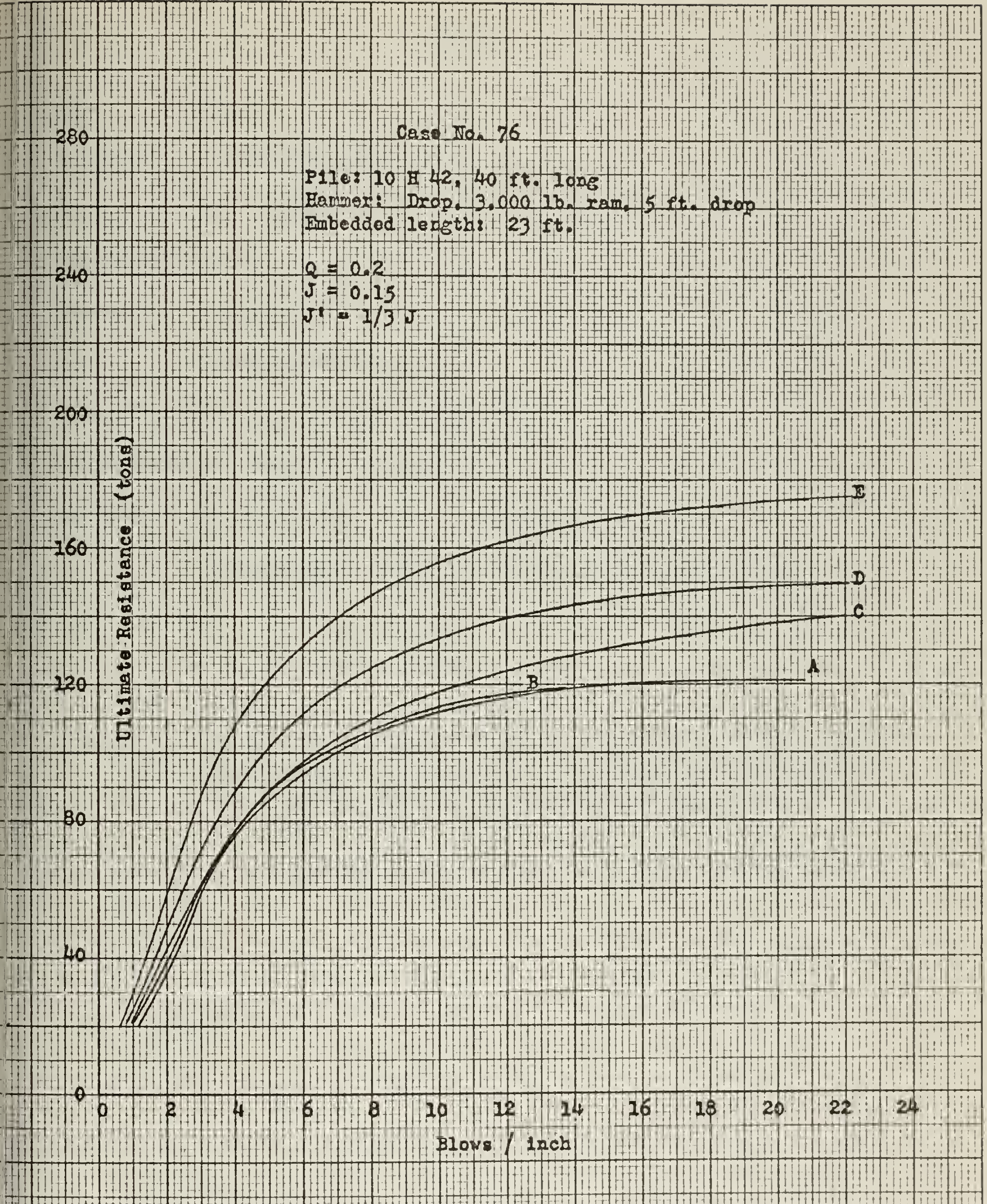

Case No. 76
Pile: 10 H 42, 40 ft. long
Hammer: Drop, 3,000 lb. ram, 5 ft. drop
Embedded length: 23 ft.
Q = 0.2
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E

Figure (62)

TABLE V

Case: 89

Source: Reference (12)

Pile: 45' 14BP89

Embedment: 44'

Hammer: Vulcan 0

Soil: Same as case # (1)

Final Penetration: 30 blows per inch

Results of Load Test: 300 tons

Results of Calculations by the Wave Equation Method:

Correlation is obtained with 25% friction and

$$Q = 0.20$$
$$J = 0.20$$
$$J' = 0.067$$

Comment: This pile was tested 5 days after driving. The 300-ton load appears to be close to the ultimate resistance of the ground. Load increments were 75 tons, 150 tons, 225 tons, 225 tons (repeated), and 300 tons.

Hiley formula gives ultimate resistance of 230 tons.

TABLE W

Case: 113

Source: Page 113, reference (16)

Pile: 40' precast concrete weighing 2.7 tons

Embedment: 31.5'

Hammer: Vulcan #1

Driving Cap: 0.525 tons

Soil: 18' coarse sand
26' fine, yellow sand

Final Penetration: 10 blows per 2.05 inches

Load Test Results: 100 tons

Results of Calculations by the Wave Equation:

a. If Q = 0.10
J = 0.15
J' = 0.05
correlation is obtained for 50% friction.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 85%.

c. If Q = 0.20
J = 0.15
J' = 0.05
correlation is obtained with a side friction of 75%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 95%.

Comments: Time between driving and load testing is unknown.

Results using the Hiley equation are 122 tons.

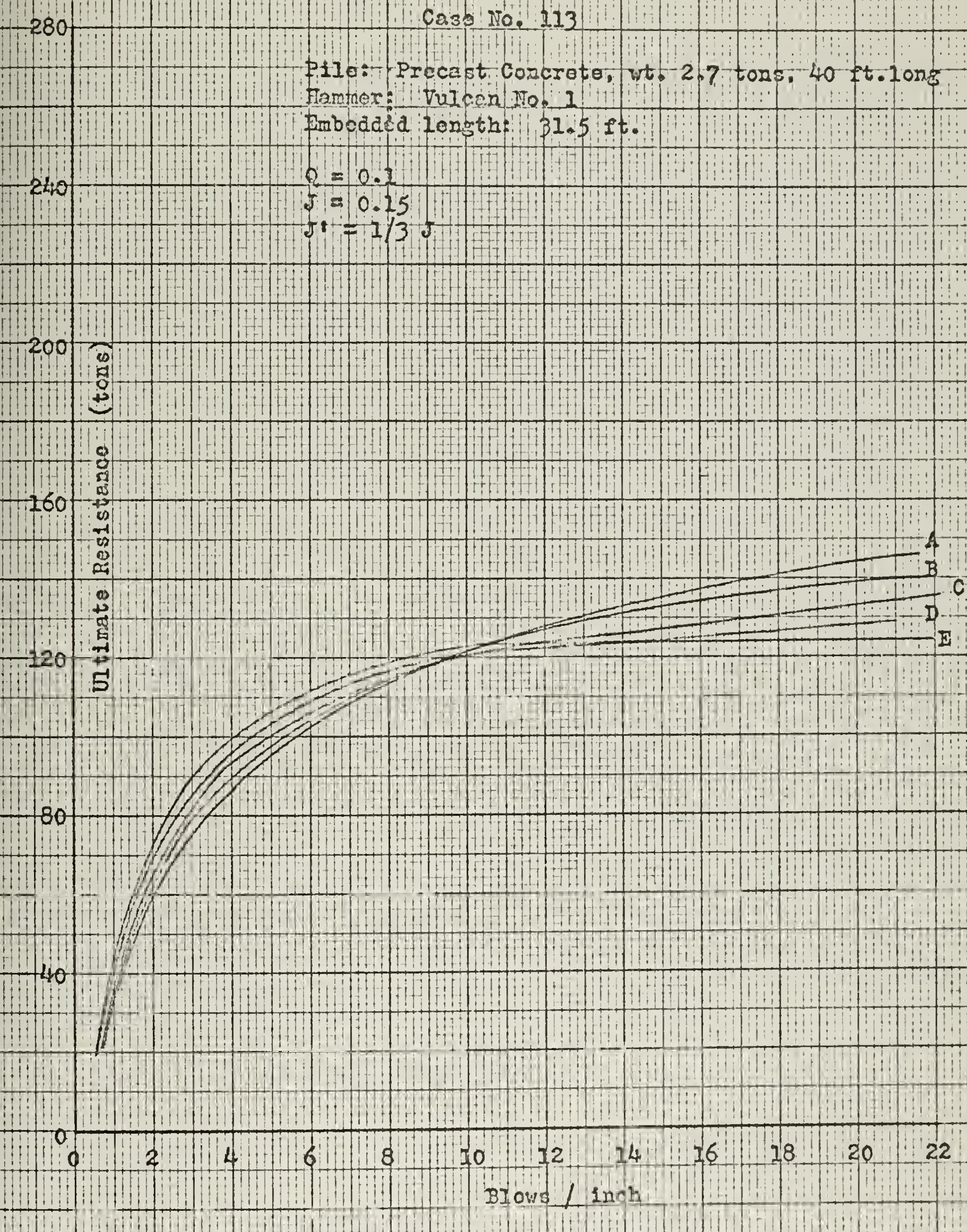

Case No. 113
Pile: Precast Concrete, wt. 2.7 tons, 40 ft.long
Hammer: Vulcan No. 1
Embedded length: 31.5 ft.
Q = 0.1
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22

Figure (63)

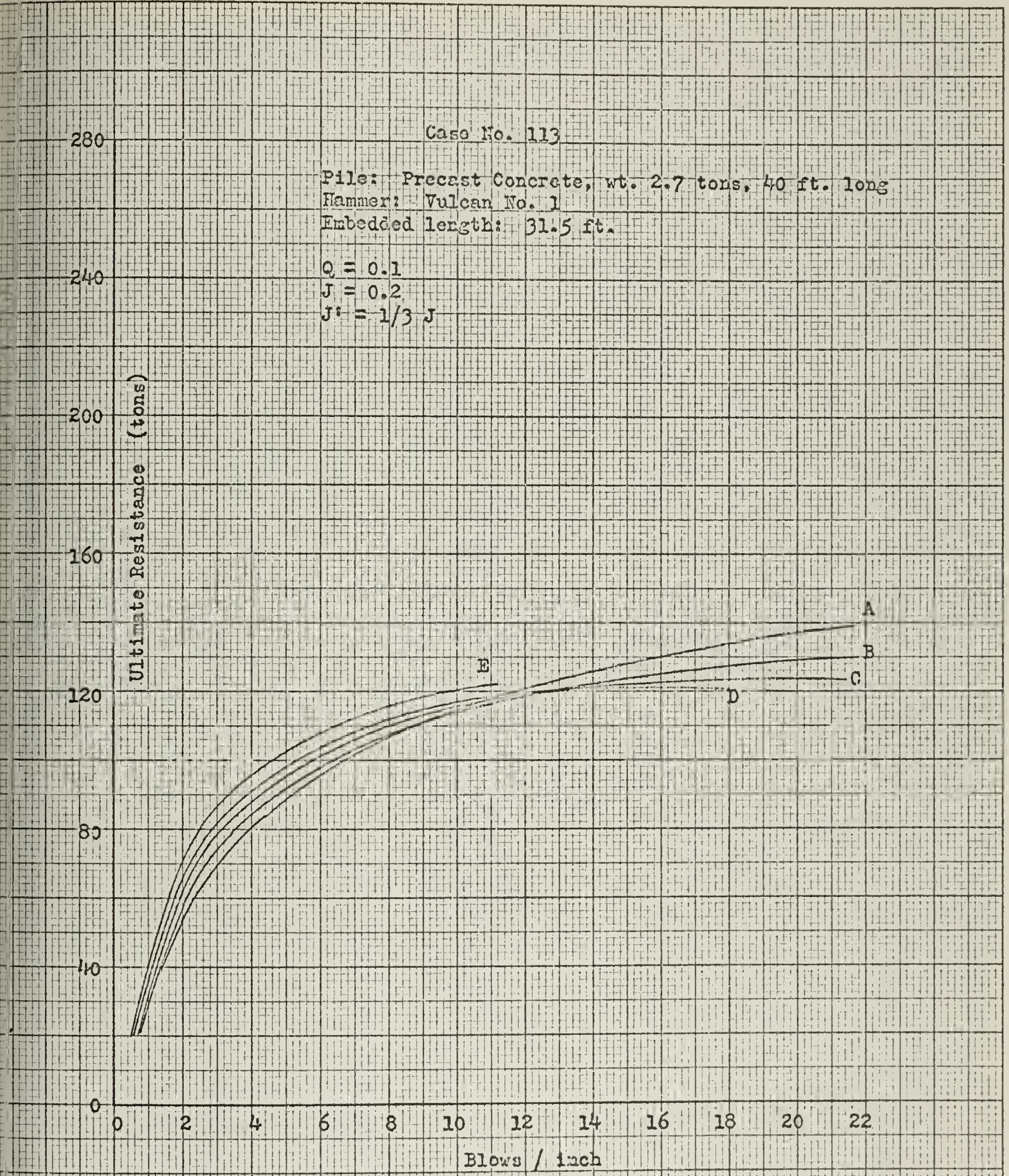

Case No. 113
Pile: Precast Concrete, wt. 2.7 tons, 40 ft. long
Hammer: Vulcan No. 1
Embedded length: 31.5 ft.
Q = 0.1
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22

Figure (64)

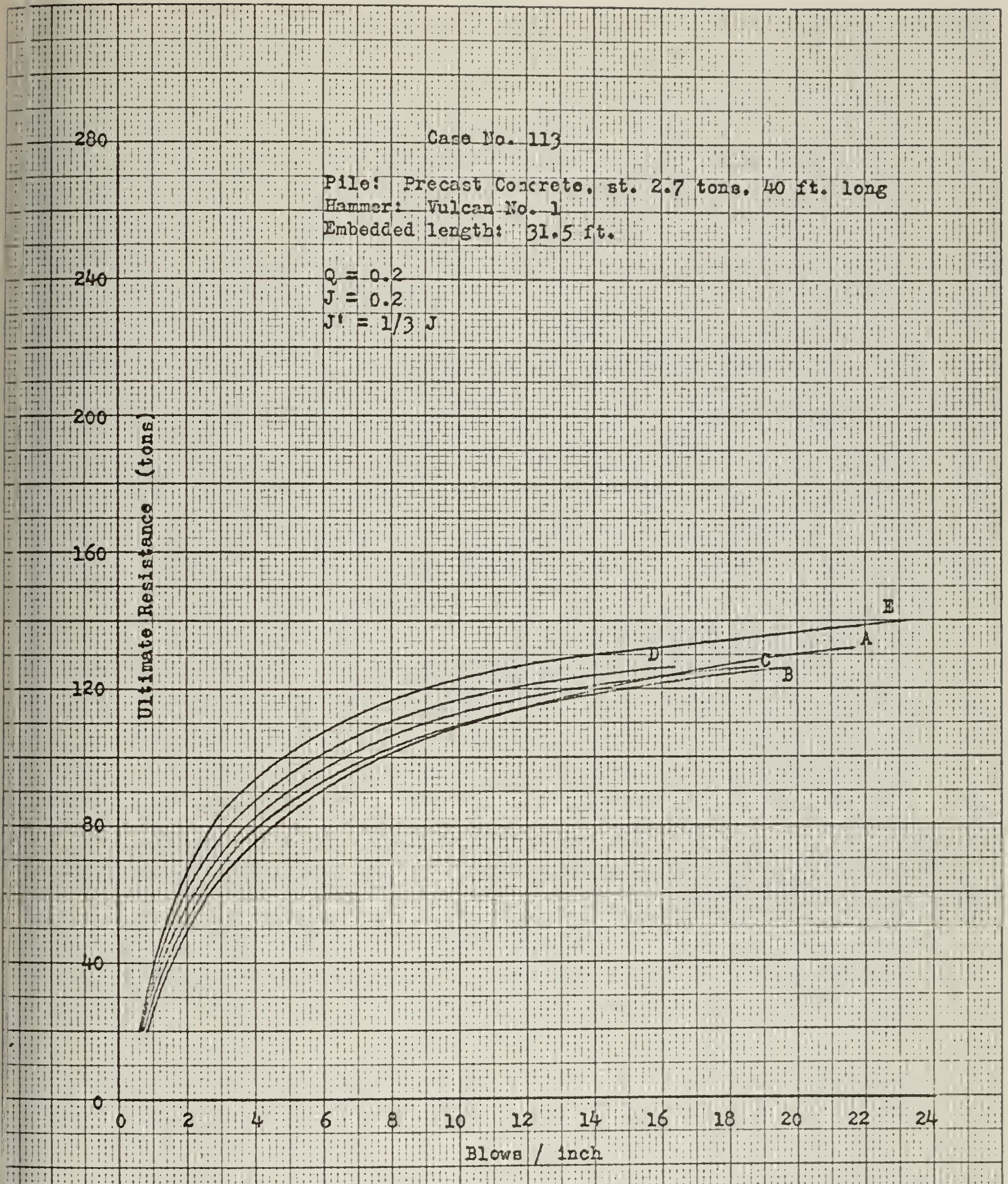

Case No. 113
Pile: Precast Concrete, st. 2.7 tons. 40 ft. long
Hammer: Vulcan No. 1
Embedded length: 31.5 ft.
Q = 0.2
J = 0.2
J' = 1/3 J
Ultimate Resistance (tons)
Blows / inch
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E

Figure (65)

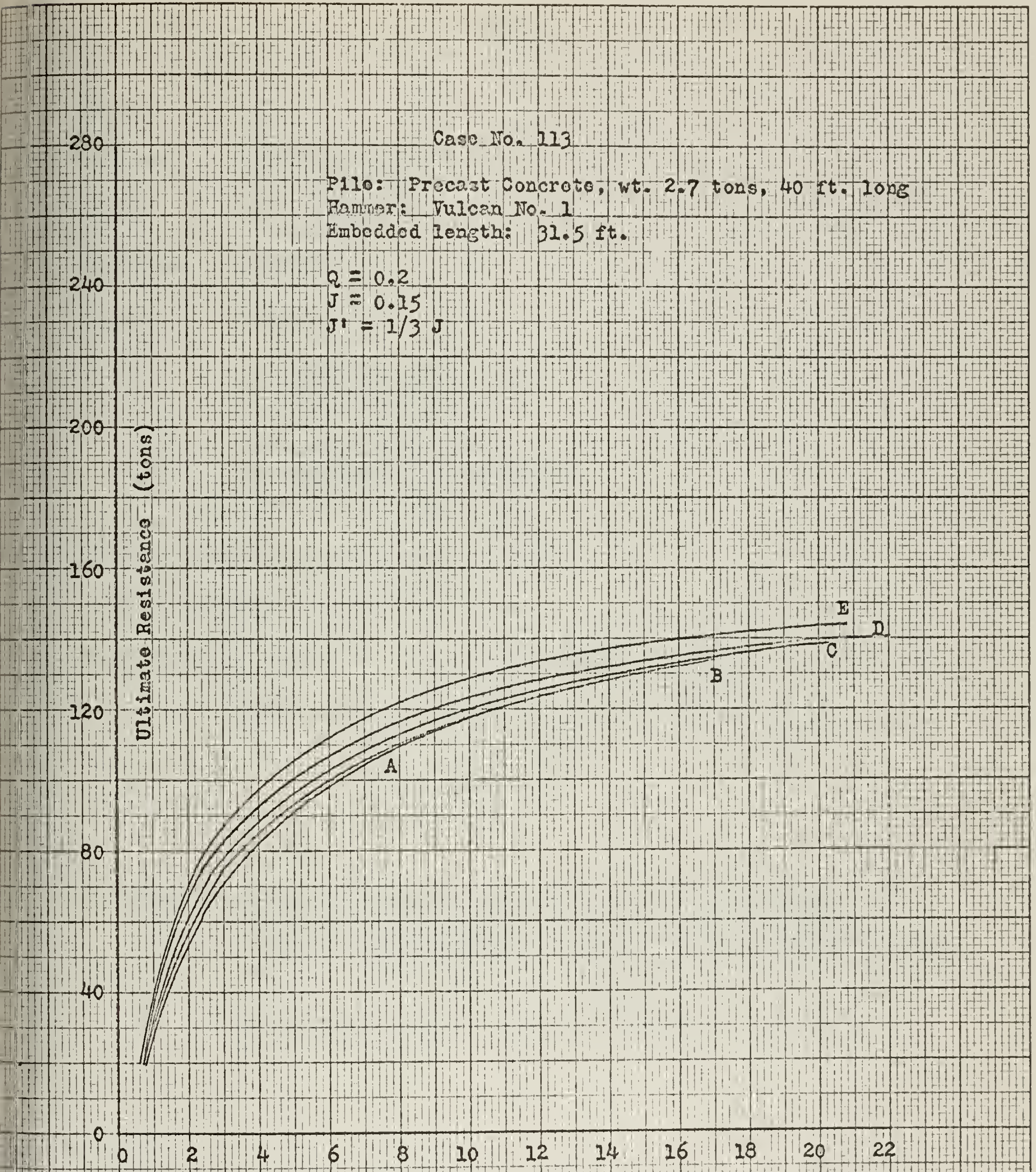

Case No. 113
Pile: Precast Concrete, wt. 2.7 tons, 40 ft. long
Hammer: Vulcan No. 1
Embedded length: 31.5 ft.
Q = 0.2
J = 0.15
J' = 1/3 J
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
Blows / inch
A
B
C
D
E

Figure (66)

TABLE X

Case: 117

Source: Reference (12)

Pile: 45' 14BP117

Embedment: 44 feet

Hammer: Vulcan 0

Soil:
9' sand & gravel
12' fine to medium sand
15' sand and gravel
6' sandy silt
2' weathered, silty shale
then hard, silty shale

Final Penetration: 22 blows per inch

Results of Load Test: 300 tons

Results of Calculations by the Wave Equation Method:

Correlation is obtained with 25% friction and

$$Q = 0.20$$
$$J = 0.20$$
$$J' = 0.067$$

Comments: This pile was test loaded 2 days after driving. The 300-ton load appears to be close to the ultimate resistance of the ground. Load increments were 75 tons, 150 tons, 225 tons, 225 tons (repeated), and 300 tons.

Hiley formula gives ultimate resistance of 140 tons.

TABLE Y

Case: 144

Source: Page 144, reference (16)

Pile: 120' 12BP53

Embedment: 81.5'

Hammer: Vulcan #1

Soil:
5' clay loam
11' sandy, silty, clay
12' medium sand
5' coarse sand
39' medium clay
10' stiff clay
10' very stiff clay
10' medium clay
11' hard clay with pebbles
then medium clay

Final Penetration: 8.0 blows per inch

Load Test Results: 117 tons ultimate resistance

Results of Calculations by the Wave Equation Method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation cannot be obtained for a failure load of 117 tons. At full point bearing (0% side friction) the ultimate resistance is calculated to be 132 tons.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 5%.

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 0%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 20%.

Comments: Hiley Formula gives ultimate resistance of 90 tons.

TABLE Z

Case: 145

Source: Page 145, reference (16)

Pile: 106' 12BP53

Embedment: 105'

Hammer: Vulcan #1

Soil: Same as case # 144

Final Penetration: 10.3 blows per inch

Load Test Results: 182 tons ultimate resistance

Results of Calculations by the Wave Equation Method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 19%.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 35%.

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 36%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 47%.

Comment: Hiley Formula gives ultimate resistance of 86 tons.

TABLE AA

Case: 146

Source: Page 146, reference (16)

Pile: 100' 12BP53

Embedment: 97.1'

Hammer: Vulcan #1

Soil: Same as Case #144

Final Penetration: 4.75 blows per inch

Load Test Results: 91 tons

Results of Calculations by the Wave Equation Method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation cannot be obtained for a failure load of 91 tons. At full point bearing (0% side friction) the ultimate resistance is calculated to be 110 tons.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 2%.

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained at 2% side friction.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 17%.

Comment: Hiley Formula gives ultimate resistance of 72 tons.

TABLE BB

Case: 161

Source: Page 161, reference (16).

Pile: 50' 12BP53

Embedment: 32 feet

Hammer: Vulcan #1

Soil:
3' fill
25' coarse sand
18' medium sand
15' fine sand
9' medium clay
20' very stiff clay
22' medium clay
then very stiff clay

Final Penetration: 4.83 blows per inch.

Load Test Results: 167 tons

Results of Calculations by the Wave Equation Method:

a. If Q = 0.10
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 35%.

b. If Q = 0.10
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 55%.

c. If Q = 0.20
J = 0.10
J' = 0.03
correlation is obtained with a side friction of 53%.

d. If Q = 0.20
J = 0.20
J' = 0.067
correlation is obtained with a side friction of 70%.

Comment: Hiley Formula gives ultimate resistance of 96 tons.

C. Discussion of Results.

Each of the cases correlated by the first method are discussed below:

1. Case number 2001, table (F). This and the next two cases are 36-inch diameter prestressed concrete piles driven at Lake Maricaibo, Venezuela. The load carrying capacity of the pile tip was calculated using the equation, P = 9AC, and a value of cohesion (C) of 2560 p.s.f., giving a capacity of 163,000 pounds. This value is 42 per cent of the total load carried by the pile at failure, and the assumption that the load is carried one half in friction and one half by point bearing would be reasonable. If the load carried as friction is considered to be evenly distributed on the embedded length of the pile, each ten-foot section would develop a resistance equal to 47,500 pounds. If the distribution of the side friction resistance were assumed to vary directly as the depth, the distribution would be as shown in figure (26), which also compared both distributions with that estimated from the shear strength of the soil. In the calculated values the full value of the shear strength of the soft clay layers was assumed to be effective and a reduced value of 800 p.s.f. was used for the stiff clay layer between elevations -153 and -143 ft. This value is consistent with Nordlund's estimate (ref. 23) and the findings of others that have investigated the friction values of piles in stiff clay (ref. 33, 38). Comparing the load distribution in figure (26), it appears that the rectangular distribution is as good a representation of the estimated actual load distribution as the triangular method would be. From figure (27) it is seen that values of Q of 0.1 and

J of 0.9, and Q of 0.2 and J of 0.3 produce nearly identical curves that give a resistance value slightly higher (200 tons vs. 190 tons) for the set of 20 blows per inch.

<u>Case 2002</u>. This is the same pile as case 2001 redriven to refusal with a heavier hammer. The load capacity was not increased a great deal by the redriving, going only from 190 tons to 230 tons. Correlation was obtained only with a quake of 0.20 using a value of J of 0.80 as shown in figure (29). This is quite a departure from the value of J obtained in the previous case and would tend to indicate that for harder driving, quake increased. Correlation could not be obtained for a quake of 0.1 with values of damping as high as 1.2.

<u>Case 2004</u>. The tip of the pile in this case penetrated into clay having a shear strength of 10,000 p.s.f. as determined from unconfined compression tests. Results of correlation tend to indicate a smaller value of quake than experiencedin the previous cases. Figure (38) shows the results from wave equation computations.

<u>Case 2003</u>. In this case correlation was obtained with values of Q and J of the same approximate values as in case 2001 as indicated in figures (34) and (35). Figures (31) to (33) are included as a matter of interest, and show that correlation could be obtained with different distributions of resistance.

<u>Case 2005</u>. This pile is of the same type as the previous one but was driven into a somewhat softer clay having a shear strength of about 3000 p.s.f. Figure (39) is the basis of correlation, and indicates values of J in the range more-or-less expected.

Case 6006. The pile in this case was not loaded to failure, but, from analysis of the load test and driving records, the engineer supervising this test estimated the value of side friction and point bearing resistance developed under static loading. From this the ultimate load carrying capacity of the pile was computed to be a minimum of 498 tons. This value was used in correlating the load test with wave equation computations. The correlation for a value of Q of 0.20 was obtained by extrapolation, but it is believed sufficiently accurate considering the general order of accuracy of the other variables and data. Figure (40) is the load settlement curve obtained from the load test, and figures (41) and (42) show the basis for correlation.

Cases 6007 and 6008. These are two pipe piles driven through 80 feet of plastic clay underlain by a thick sand stratum. One pile was driven about 5 feet into the sand and the other was stopped 5 feet above the sand layer. The object of the load tests was to determine the load carried by the point in sand. Correlation for the pile that extends into the sand (case 6008) was obtained in the lower end of the expected range of values for the point damping factor and quake, with values of 0.05 and 0.10 respectively. No attempt was made to account for the fact that the pile itself is largely embedded in clay. For the pile stopped above the sand layer, correlation was obtained for values of Q of 0.10, J of 0.15 and J' of 0.05. Since the load is carried entirely by friction, all of the damping is due to J', the side damping factor. These values are not consistent with the expected range of values for clay, and indicate that the value of quake is smaller

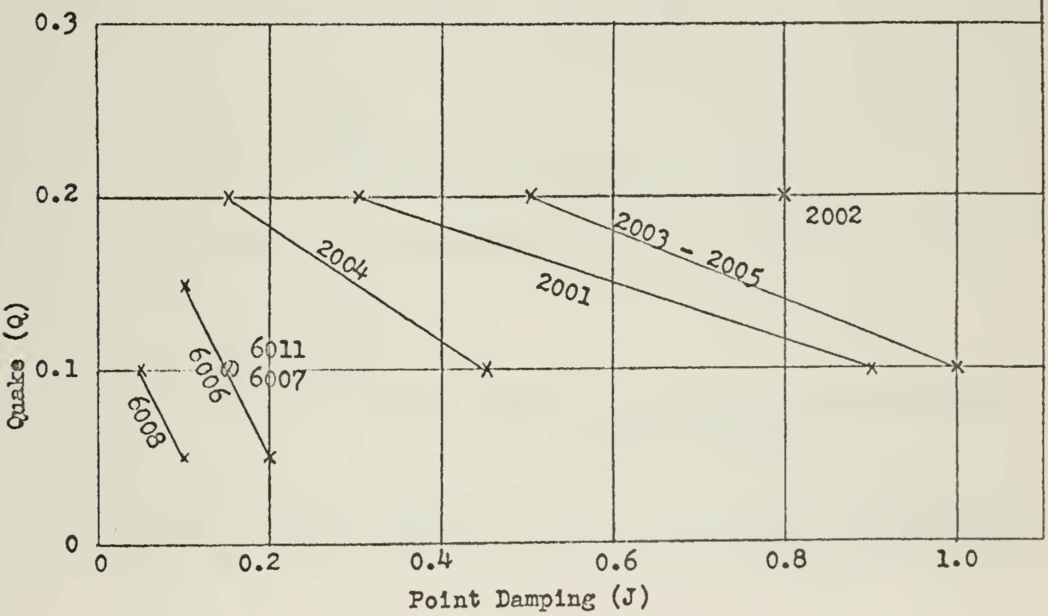
than expected, which would be consistent with the easy driving conditions, that the relationship between J and J' is in error, or that side damping does not develop when a steel pipe pile is driven through plastic clay. With respect to case 6008, pile point in sand, it is possible that the clay was somewhat sensitive and regained strength during the period before the load test was made. If the sensitivity of the clay were such that one half of its strength were lost due to remolding and then regained after a period of time, the ultimate resistance during driving would be reduced by about 40 tons. This would produce correlation at a value of Q of 0.10 and J of 0.15.

Case 6011. The high load capacity developed by this pile and the low resistance to penetration indicate that a quick condition in the overlying silt layers occurred. If this were the case, the resistance to driving would be developed on the portion of the pile embedded in sand with no resistance occurring in the silt layers. Correlation is obtained on this basis as shown in figures (45) and (46).

The results of correlation attempts for these cases are summarized in figure (67). The piles driven with the tip in sand are closely grouped and fall within a range of values for J of 0.05 to 0.2, and for values of Q of 0.05 to 0.10. This result tends to confirm Smith's recommended value for J of 0.15, for piles driven in sand.

Results of correlation for piles driven in clay are spread over a range of values for J of 0.15 to about 1.4, and for values of Q of 0.1 and 0.2. Case 6007, which has a J value for correlation of 0.15, is a 100% friction pile, and the point damping factor (J) has no influence. This possibly indicates that the one-third relationship

Case	Test Load	Blows per in.	% Load at point	Values of J for Q = 0.05	Q = 0.1	Q = 0.2	Soil
2001	190	20	50		0.9	0.3	Clay
2002	230	132	50 - 75			0.8	Clay
2003	165	10	50		1.0	0.5	Clay
2004	365	20	50 - 25		0.45	0.15	Clay
2005	260	40	100 - 75		1.0	0.5	Clay
6006	498	60	50	0.2	0.15		Sand
6007	81	2	0		0.15		Sand
6008	244	38	75 - 50	0.1	0.05		Sand
6011	170	2	100 - 75		0.15		Sand



Summary of Correlation Results - First Method

Figure (67)

between J and J' is too gross an assumption in cases where all or most of the load is carried by friction. In general, the results in clay are consistent with values of J expected, based on dynamic tests made on soils in the laboratory.

D. Summary of Correlation Results for Second Method of Approach

For the cases studied, the following values of ground quake, point damping, and side damping seem to be appropriate for the soil types indicated. The percentages of side friction given are for steel H piles unless otherwise indicated. Furthermore, the principal case studies used in arriving at the values are shown:

1. Coarse Sand: (Case Numbers 113 and 161)

   Q = 0.10
   J = 0.15
   J' = 0.05
   Side Friction is 35%.

2. Sand and Gravel Mixed: (Case Number 71)

   Q = 0.10
   J = 0.15
   J' = 0.05
   Side Firction is between 75% and 100%.

3. Fine Sand: (Case numbers 75, 76, and 113)

   Q = 0.15
   J = 0.15
   J' = 0.05
   Side Friction is 100% (use 50% for straight-sided concrete pile).

4. Sand layers combined with layers or strata of clay or loam or both, but sand layers predominate: (Case numbers 31, 34, 35, 36, 144, 145, 146, and 161 apply)

   Q = 0.20
   J = 0.20
   J' = 0.067
   Side Friction is 25%

5. Silt and Fine Sand underlain by Hard Strata: (Case number 67)

   Q = 0.20
   J = 0.20
   J' = 0.067
   Side Friction is 40%.

6. Sand and Gravel Underlain by Firm Strata such as Shale: (Case numbers 89 and 117)

   Q = 0.10 to 0.20
   J = 0.15 to 0.20
   J' = 0.05 to 0.067
   Side friction of 25%.

Using the average values of Q, J, and J' and side friction obtained for the various types of soil conditions, the variations of calculated ultimate resistances from load test results for the specific cases will be investigated.

Hiley equation results are also shown with this comparison in Table CC.

It should be noted that the values of ultimate resistance shown to be calculated by the Hiley formula were obtained by using values of C_1, C_2, and C_3 recommended by Chellis (ref. 5) for use in the absence of specific information. No attempt was made to determine what changes in these coefficients would be necessary to achieve closer agreement with load test results. Comparison of Hiley formula ultimate resistance calculations has therefore been restricted to those case studies where complete data were not provided. Whether someone having experience with the Hiley formula could have justified use of different values for these coefficients and thereby achieved closer agreement with load tests is not known.

TABLE CC

COMPARISON OF WAVE EQUATION SOLUTION WITH HILEY FORMULA AND LOAD TESTS

Case Number	Ultimate Resistance: Computed by Wave Equation (tons)	Computed by Hiley Formula (tons)	Load Test (tons)	Per Cent Error: Wave Equation %	Hiley Formula %
31	40	52	45	-11.1	-15.5
34	100	96	100	0.0	- 4.0
35	103	92	100	3.0	- 8.0
36	91	92	100	- 9.0	- 8.0
67	60	25	60	0.0	-58.4
71	64	80	64	0.0	25.0
75	90	80	90	0.0	-11.1
76	50	60	50	0.0	20.0
89	300	230	300	0.0	-23.3
117	300	140	300	0.0	-53.3
113	100	122	100	0.0	22.0
144	122	90	117	4.3	-23.0
145	133	86	182	-26.9	-52.7
146	97	72	91	6.6	-20.8
161	103	96	167	-38.3	-42.5

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It must be remembered that the number of cases investigated here is small, and the values of point and side damping, ground quake, and side friction reported as giving good correlation may be replaced by more accurate values as more driving records and load tests are analyzed and as more basic information about the dynamic properties of soils becomes known. Furthermore, side resistance was taken as rectangular, or uniform distribution, along the side of the piles with about the top 10 feet of embedment discounted in each case. Different side distribution patterns would, of course, alter the values found above somewhat.

On the other hand, the values reported are within the ranges that one might anticipate, and they do give good correlation as well as providing results generally more accurate than predictions from the Hiley formula.

Chapter VII

RECOMMENDATIONS FOR FURTHER RESEARCH

No attempt will be made here to make an exhaustive and detailed list of further research possibilities, but several will be discussed briefly. Probably as many have already occurred to the reader, and certainly many more will present themselves to anyone beginning work in this general area. The main factors requiring further evaluation are ground quake, point and side damping, and distribution of side frictional resistance. Specifically, the recommendations are as follows:

A. Construct an apparatus in which model piles may be loaded both statically and dynamically. If economically feasible, the model pile may be instrumented and recording pressure cells embedded in the soil. Also it would be desirable to be able to enclose the entire soil-pile system so that a variable confining pressure could be applied. With this set-up the model pile could be encased to avoid side friction effects in the initial experiments. The model pile could then be struck by a small drop hammer and the set determined. This procedure could be repeated for different sized hammers and strokes, and the results compared with computer results for the test set-ups. The ultimate resistance could be determined from test loading statically an identical pile in the same controlled soil. In this way values of quake and damping might be bracketed more closely for various soil-pile systems.

B. Obtaining a picture of the interaction between pile and soil may prove useful. It is thought that this may be done for both

the static and dynamic cases. A photosensitive material could be used for a model pile which would be embedded in another photosensitive material representing the ground. The stress waves propagated by striking the model pile at its head by a small drop hammer could be captured in successive time intervals by using a high-speed camera. Even though such a study may give only qualitative results, it may assist in understanding how much of the ground is acting with a pile and the order of magnitude of stresses within this zone. Such information should prove helpful in evaluating both ground quake and the distribution of side frictional resistance.

C. Study the problem from both an analytical and statistical viewpoint. Seek the cooperation of major construction corporations, government bodies, and others in obtaining as many good load tests as possible. For example, it is understood that the University of Michigan or the Bureau of Public Roads is on the verge of publishing a number of very carefully controlled tests. Run a wide range of curves for various pile and hammer combinations as was done herein for the 53-pound bearing pile, and see if dynamic soil properties cannot be deduced from them which will give good correlation. If sufficient driving records and load tests become available, it may be advisable to write a computer program which will assist in matching up wave equation results with field data.

D. A wide range of wave equation solutions may be run off for the common soil sampling rigs with a view toward correlating the information with the soil data and with pile driving and testing records. In this connection it should be noted that the pipe section

of the sampler should be broken down into one-foot increments and the time intervals of the calculations proportionately reduced in order to obtain stable calculations. It should also be kept in mind that since lateral support of the long pipe section is lacking in this case, some energy will be lost in transverse vibrations. How seriously this will affect the results is not known.

E. The computer program included herein may be modified so that it can also handle the case of a long hammer, which should be represented as several weights in the mathematical model, and the case where the ram strikes the pile other than at its top. A. E. L. Smith illustrates how the mathematical model for these cases may be represented (ref. 28).

F. More detailed study should be made of pile tip velocities at the time of maximum tip force, as determined from the computer results, for a wide range of pile types, lengths, and types of hammers in order to establish a more valid basis for relating results of rapid loading tests made in the laboratory with pile driving action.

G. With respect to the computer program, investigation should be made into the factors that cause instability in the computations. In a few computer results for this work, instability was encountered unexpectedly. These instances involved light piles, driven with relatively heavy hammers, or low values of ultimate resistance relative to the normal capacity of the pile. Investigations should be made to determine additional criteria for use in making up the input data so as to avoid instability.

As previously indicated, these are but a few of the possibilities for further work; however, it is hoped that these listed will be sufficient to stimulate the interest of potential investigators.

CONCLUSION

The prediction of a pile's ultimate static bearing capacity from its dynamic behavior during driving has been an elusive goal of civil engineers for many years. Hundreds of ingenious empirical dynamic formulas have been devised which may give good results under particular conditions of driving equipment, pile, and soil combinations; but the difficulty has been in selecting the proper formula to obtain reliable results. Furthermore, these dynamic formulas have limited ranges of applicability since each ignores important aspects of the problem. Because of these limitations, their validity as a general method of approaching the problem at hand is open to question. Civil engineers have recognized these shortcomings and have tended to treat the dynamic formulas as crude guides to the ultimate bearing capacity of a pile in granular soils, recognizing their complete inadequacy for piles driven in cohesive soils.

The application of the wave equation to the pile driving problem, as envisioned here, represents an attempt to obtain a general method of analysis which will assist in predicting the ultimate bearing capacity of a pile from its observed behavior under the last hammer blow during driving subject to the limitations enumerated in paragraph E below. The wave equation method of analysis is also useful for studying stresses occurring during driving and for selecting appropriate pile driving equipment to meet economically specific field conditions. The major contribution by A. E. L. Smith, a pioneer in this work, was in devising a mathematical representation

of the pile driving problem suitable for digital computer solution. He then developed such a computer program, and his published work which followed emphasized the stress determination problem. He did, however, suggest that the method could be used in relating the static bearing capacity of a pile to its dynamic behavior, and he offered values of damping and ground quake which he believed to be accurate enough to make use of the method practical.

To examine the possibility of correlating pile driving records with load tests by this method of analysis has been the principal objective of this thesis. Before making detailed case studies, it was necessary to examine the validity of the method itself, to review the soil engineering aspects of the problem, and to develop the necessary computer programs. These findings, together with the results from 24 case studies, are as follows:

A. Validity of the mathematical model used. The model devised by A. E. L. Smith has been accepted as correct by those commenting on his work, but it had not been proven to be equivalent to the wave equation. The formal proof of this equivalency has been provided in Chapter III.

B. Resistance to driving. It is shown in Chapter V that the expression used for the resistance of a pile to driving corresponds approximately to published experimental findings of others on the resistance developed in a soil due to high rates of loading.

C. Computer programs. Although A E. L. Smith developed computer programs for the wave equation method of analysis, they were not available as he had not published them. Computer programs are

published herewith which were developed independently. Their validity is shown by comparing with both a manual solution for a sample problem which utilizes all parts of the program and with results published by A. E. L. Smith. Complete instructions for using these programs are provided in Appendices A, B, and C so that someone unfamiliar with computer work may utilize them.

The computer program developed for the Hiley formula is also subject to the limitations inherent in an empirical dynamic pile driving formula.

D. Ground damping, quake, and side friction. From published experimental work by others, approximate values of damping to be used with the wave equation method of analysis for some soils have been derived in Chapter V. Starting with a range of quake reported by others, a range of values of damping were found which caused the solution of the wave equation method to correlate with load tests. This range of values of damping was compatible with the values derived from the published experimental work. Using the values of quake and damping thus established, values of uniformly distributed side frictional resistance were determined for various soil and pile combinations.

E. Usefulness of the wave equation method for predicting static bearing capacity. Attempts to correlate 24 pile driving records with load tests have shown surprising results. It appears that the wave equation method of pile driving analysis may become an accurate method which may be used together with other factors in predicting the ultimate static bearing capacity of a pile, as indicated by the results

summarized in figure (67) and table (CC). Among these other factors which must be evaluated are the relaxation or set-up of the soil with time after driving, the group effect of piles, development of negative friction, consolidation of a clay layer beneath a pile tip, long-term changes in water table, corrosion or deterioration of the pile from other causes, and any other special problems which may be associated with a particular construction project.

Furthermore, the problem has been treated as one-dimensional; i.e., lateral vibrations have not been included in the analysis. This should not, however, be a serious limitation on the method for most pile driving situations since a large proportion of the pile will be embedded as final penetration is reached. Hysteresis losses in the pile have not been included in the analysis, but their neglect is considered justified at this stage of development of the wave equation method. Since good correlation was possible using values derived from the published dynamic soil tests, it appears that the wave equation method is well suited for analysis of any size or type of pile being driven by any hammer in not only granular soils, but also cohesive soils.

F. The dynamic interaction between pile and soil is incompletely understood and offers a fruitful area for further research. Even a qualitative picture of stress transfer should prove helpful. Additional experimental work to isolate the parameters affecting ground quake, point damping, side damping, and distribution of point and side resistance to driving is needed as well as the evaluation of these factors.

G. Correlation with many driving records and load tests should be made before taking the values reported herein as correct. Future investigators may use freely the computer programs and plotted results of calculations already made, and presented here, to facilitate such work.

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APPENDIX A

DETAILS OF THE "BASIC COMPUTER PROGRAM" WHICH USES THE WAVE EQUATION

General: This program is written in Fortran language for the IBM 7090 computer, but it can be used equally well with the IBM 709 or IBM 7094 computer. It has as its objective the detailed analysis of stresses, velocities, accelerations, displacements, and permanent set of the pile as these factors vary with time. For details of the theory and the mathematical model, refer to chapter III.

Termination: Analysis is terminated automatically when the velocities of all the blocks of the mathematical model are simultaneously negative or equal to zero. Negative velocities are taken as being in the direction from the bottom of the pile toward its top. Termination is also scheduled when the permanent set of the pile becomes constant. Effects beyond either of these times are considered to be of secondary importance for purposes of this study. Generally these conditions will be met before calculations have been made for 300 time intervals, and in most cases much sooner. The time interval is usually taken as 0.00025 or 0.00033 seconds because of the high velocity of stress wave travel in materials used for piles; therefore, this program investigates what happens during the first 0.1 second or less after the hammer hits the pile. In the usual case the pile will achieve its permanent set well within this time.

The program will also stop automatically if the velocity of either the pile cap or pile tip exceeds twice the velocity the ram had at the moment of impact. This feature is designed to help detect

instability (see Chapter III, C) in the calculations, but should not be relied upon entirely. The surest way of detecting instability is by plotting the displacements and velocities of each block in the model against time and then examining the curves so obtained for very sharp peaks or discontinuities. Should such irregularities be present, the time interval may be reduced and the program rerun and rechecked for stability. The importance of this check cannot be over-emphasized.

Should the program run the full 300 time increments, no indication is given in the output as to why the program stopped. In such a case the results should be examined especially carefully, for in all probability either instability was present or the program stopped before the final permanent set was reached. In such a case the variable DP may be redimensioned to a larger value (second statement above statement number 207 in the beginning of the program), the statement immediately after statement number 500 should be changed to read DO 501 I = 1,X where X will be the new dimension of variable DP, and the statement immediately after statement number 302 should be similarly changed to read DO 101 N = 3,X. In all other cases of program termination the reason for stopping will be printed out automatically in plain language; for example, "VELOCITY OF PILE CAP EXCEEDED TWICE THE RAM VELOCITY."

<u>Preparation of Input Data</u>: All data is punched on standard data cards in the exact order indicated below and then placed immediately after the * DATA card at the end of the source program.

CARD ORDER	DESCRIPTION
1	Case number--any number up to 10 places may be assigned, and it should be punched within the first 10 places on the card. Blanks will be read as zeros; use no decimal points.
2	If there is another case with more data following this set of data, punch +1 in spaces 9 and 10 on this card; if this is the last case for this run, punch -1 in places 9 and 10.
3	In the first 4 places of this card punch the number of weights in the mathematical model. Within spaces 5 through 8 punch the number of weights in the mathematical model minus 1. Use no decimals.
4	Punch within the first 10 spaces the decimal form of the time interval to be used in seconds.
The next M cards	M is the number of weights in the mathematical model. One card is to be used for each weight. Punch the value in pounds of each weight in the first 10 spaces of each card. The first card is for weight 1 (the ram) and then consecutively down to the pile tip. Use a decimal point and the number can be located anywhere within the first 10 spaces.
The next (M-1) cards	Punch within the first 10 spaces of each of the cards the spring constants beginning with the capblock and extending to the top of the pile tip section of the model. The values are to be in pounds per inch. Use a decimal point and the number can be located anywhere within the first 10 spaces.
The next M cards	Punch within the first 10 spaces of each card the soil spring coefficients beginning with block 1. At least the first 2 or 3 will be zero normally. Use a decimal point and the number in pounds per inch can be placed anywhere within the first 10 spaces.
The next card	Punch with a decimal point anywhere within the first 10 spaces on this card the value of the capblock coefficient of restitution.
The next card	Punch with a decimal point anywhere within the first 10 spaces on this card the value of the coefficient of restitution of the head packing

	in the case of a concrete pile. For all other cases without head packing punch 1.00.
The next card	Punch with a decimal point anywhere within the first 10 spaces the velocity of the ram at the instant of impact in feet per second.
The next card	Punch with a decimal point anywhere within the first 10 spaces the value of ground quake in inches.
The next card	Punch with a decimal point anywhere within the first 10 spaces the value of point damping to be used.
The next card	Punch with a decimal point anywhere within the first 10 spaces the value of side damping to be used.
The next card	If the pile cap is allowed to transmit tension, enter a +1 in spaces 9 and 10; if not, enter a -1 there. Use no decimals.
The next card	If side friction is present, enter a +1 in spaces 9 and 10; if not, enter a -1 there. Use no decimals.

Running the Program: The data deck of cards just described is placed immediately after the source program deck and is ready to be turned over to a machine operator for processing.

Results: The computer will clearly write out in the output the input data; the number of each time interval; and identify the displacements, velocities, permanent sets, forces, and ground resistances acting on each block. The results are printed in decimal form with a multiplier notation. For example, 0.2207 E06 means 0.2207×10^6 or 220,700.

The Program: The complete program is presented so that any who wish to use it may do so freely. Each line of the program is punched on a separate card and must be assembled in the exact order shown. Anyone preparing data cards from this program should verify painstakingly

that no mistakes have crept in during punching, for the slightest discrepancy can lead to totally erroneous results. For that reason those preparing cards from the program given here are cautioned to try it out with test cases before relying on the results.

The notation used in the program is the same as was used in the derivation of the basic equations except as dictated by Fortran requirements. The more important variations in terminology are:

S	instead of	K	for spring constants
SP	instead of	K'	for soil spring coefficients
RES1	instead of	e_1	for coefficient of restitution for the capblock
RES2	instead of	e_2	for coefficient of restitution for head packing
Z	instead of	J	for point damping factor
ZP	instead of	J'	for side damping factor

Additionally, the distinction between the present time interval, the last time interval, and the second previous time interval is made by use of the letters L, M, and S respectively. For example, in the case of velocities,

VL	is used instead of	V
VM	is used instead of	v
VS	is used instead of	v^*

Other variables introduced in the program are defined as they occur within the program or have been explained under preparation of data above.

```
*ID   REESE          C2165   CIVIL ENGR   G WAVE EQ   5  50
*     XEQ
*     LABEL
C     WAVE EQUATION
      DIMENSION DS(100), DM(100), DL(100),VS(100), VM(100), VL(100)
      DIMENSION DPSM(100), DPSL(100)
      DIMENSION S(100), W(100), DP(300), R(100), SP(100),CS(100)
      DIMENSION CM(100), CL(100),FS(100), FM(100), FL(100),DPSS(100)
207   DO 500 I = 1,100
      DS(I) = 0.0
      DM(I) = 0.0
      DL(I) = 0.0
      VS(I) = 0.0
      VM(I) = 0.0
      VL(I) = 0.0
      DPSM(I) = 0.0
      DPSL(I) = 0.0
      S(I) = 0.0
      W(I) = 0.0
      R(I) = 0.0
      SP(I) = 0.0
      CS(I) = 0.0
      CM(I) = 0.0
      CL(I) = 0.0
      FS(I) = 0.0
      FM(I) = 0.0
      FL(I) = 0.0
500   DPSS(I) = 0.0
      DO 501 I = 1,300
501   DP(I) = 0.0
      READ INPUT TAPE 5,27,NCASE,ITURN
 27   FORMAT (I10/I10)
     0 READ INPUT TAPE 5, 70,M,IM1,T,(W(I),I=1,M),(S(I),I=1,IM1)
     1  ,(SP(I),I=1,M), RES1, RES2, V,Q,Z,ZP,CAP,RUB
 70   FORMAT (2I4/(F10.4))
      WRITE OUTPUT TAPE 6,30,NCASE
 30   FORMAT (12H CASE NUMBER I5)
      WRITE OUTPUT TAPE 6,28,(I,W(I),S(I),SP(I),I=1,M)
 28   FORMAT (5H DATA//58H    M                   W                   S
     1          SP/(I4,3F20.2))
      WRITE OUTPUT TAPE 6,29,RES1,RES2,ZP,Z,Q,V
 29   FORMAT (36H COEF. OF REST. OF CAPBLOCK (RES1) = F4.2/
     1 36H COEF. OF REST. OF PILECAP (RES2) =  F4.2/
     2 36H SIDE DAMPING FACTOR (ZP) =          F4.2/
     3 36H POINT DAMPING FACTOR (Z) =          F4.2/
     4 36H GROUND QUAKE (Q) =                  F4.2/
     5 36H INITIAL RAM VELOCITY (V) =          F6.2)
      WRITE OUTPUT TAPE 6,31,T,CAP,RUB
 31   FORMAT (36H TIME INTERVAL =                    F8.6/
```

```
  1 36H CAP =                                F6.2/
  2 36H RUB =                                F6.2)
    WRITE OUTPUT TAPE 6,1
1   FORMAT (84H            M         DL              VL              DP
  1               FL             R//)
    DS(1) = V*12.*T
    VS(1) =     V +(-DS(1)*S(1))*T*32.2/W(1)
    VS(2) = (DS(1)*S(1))*T*32.2/W(2)
    DM(1) = DS(1) + VS(1)*12.*T
    DM(2) = VS(2)*12.*T
    CM(2) = DM(2)
    VM(1) = VS(1) -((DM(1)-DM(2))*S(1))*T*32.2/W(1)
    VM(2) = VS(2)+((DM(1)-DM(2))*S(1)-(DM(2)-DM(3))*S(2))*T*32.2/W(2)
    VM(3) = VS(3) +((DM(2)-DM(3))*S(2)-R3)*T*32.2/W(3)
    CM(1) = DM(1) - DM(2)
    CHECK = 2.* V
    WRITE OUTPUT TAPE 6,301,DS(1), VS(1)
301 FORMAT (8H N =   1,6H     1 2E15.4//)
    WRITE OUTPUT TAPE 6,302,DM(1),VM(1),DM(2),VM(2)
302 FORMAT (8H N =   2,6H     1 2E15.4/14H             2 2E15.4//)
    DO 101 N = 3, 300
    WRITE OUTPUT TAPE 6,303,N
303 FORMAT (5H = I3)
    MLESS1 = M-1
    DO 130 I = 1,M
130 DL(I) = DM(I) + VM(I)*12.*T
    DO 131 I = 1,MLESS1
131 CL(I) = DL(I) - DL(I+1)
 13 IF(DL(M)) 12,12,14
 12 DE = 0.0
    GO TO 24
 14 IF(DL(M) - Q) 16,16,18
 16 DE = 0.0
    GO TO 24
 18 VALUE = DL(M) - Q
    IF (VALUE-DE) 22,22,20
 20 DE = VALUE
    GO TO 24
 22 DE = DE
 24 DP(N) = DE
    R(M) = (DL(M)-DP(N))*SP(M)*(1.+Z*VM(M))
    IF (DP(N)) 25,25,26
 26 IF (DP(N) - DP(N-1)) 197,197,25
 25 CONTINUE
  3 VAL = S(1)*CL(1)
    IF(CL(1)-CM(1)) 5,5,4
  4 FL(1) = VAL
    GO TO 33
  5 CL(1) = CM(1)
    FL(1)=VAL/(RES1**2)-(1./(RES1**2)-1.)*S(1)*CL(1)
 33 VAL2 = S(2)*CL(2)
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      IF(CL(2)-CM(2)) 35,35,34
 34   FL(2) = VAL2
      GO TO 37
 35   CL(2) = CM(2)
      FL(2) = VAL2/(RES2**2)-(1./(RES2**2)-1.)*S(2)*CL(2)
 37   IF(CAP) 38,36,36
 38   IF(FL(2)) 39,36,36
 39   FL(2) = 0.0
 36   CONTINUE
  7   IF(FL(1)) 8,133,133
  8   FL(1) = 0.0
133   DO 132 I = 3,MLESS1
132   FL(I) = CL(I)*S(I)
 42   IF(RUB)49,49,44
 44   DO 48 I = 3,MLESS1
      DPSL(I) = DPSM(I)
      CHANGE = DL(I) - Q
      SUM + DL(I) + Q
      IF(DPSL(I)-CHANGE)45,46,46
 45   DPSL(I) = CHANGE
 46   IF(DPSL(I)-SUM)48,48,47
 47   DPSL(I) = SUM
 48   R(I) = (DL(I) - DPSL(I))*SP(I)*(1.+ZP*VM(I))
 49   DO 50 I = 1,M
      VL(I) = VM(I) + (FL(I-1) - FL(I) - R(I))*T*32.2/W(I)
 50   CONTINUE
      DO 100 I = 1,M
703   WRITE OUTPUT TAPE 6,304,I,DL(I),VL(I),DPSL(I),FL(I),R(I)
304   FORMAT (11H           I3,5E15.4)
100   CONTINUE
      DO 75 K = 1,M
      X = 1.
      IF (VL(K))     75,75,800
 75   CONTINUE
      GO TO 199
800   CONTINUE
      IF(VL(2) - CHECK)99,99,190
 99   IF(VL(M)-CHECK)102,102,194
102   DO 98 K = 1,M
      DS(K) = DM(K)
      DM(K) = DL(K)
      DL(K) = 0.0
      VS(K) = VM(K)
      VM(K) = VL(K)
      VL(K) = 0.0
      CS(K) = CM(K)
      CM(K) = CL(K)
      CL(K) = 0.0
      FS(K) = FM(K)
      FM(K) = FL(K)
      FL(K) = 0.0
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```
      DPSS(K) = DPSM(K)
      DPSM(K) = DPSL(K)
 98   DPSL(K) = 0.0
      WRITE OUTPUT TAPE 6,305,DP(N)
305   FORMAT (14H              E15.4)
101   CONTINUE
      GO TO 196
190   WRITE OUTPUT TAPE 6,191,N
191 0 FORMAT (64H VELOCITY OF PILE CAP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I3)
      GO TO 196
194   WRITE OUTPUT TAPE 6,195,N
195 0 FORMAT (64H VELOCITY OF PILE TIP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I3)
      GO TO 196
197   WRITE OUTPUT TAPE 6,198,N
198   FORMAT (26H DP BECAME CONSTANT AT N = I3)
      GO TO 196
199   WRITE OUTPUT TAPE 6,200,N
200   FORMAT (52H ALL VL WERE SIMULTANEOUSLY NEGATIVE OR ZERO AT N = I4)
196   IF(ITURN)205,205,207
205   CALL EXIT
      END
*     DATA
```

APPENDIX B

DETAILS OF THE "VARY Ru PROGRAM" WHICH USES THE WAVE EQUATION

General. This program is identical to the BASIC PROGRAM except that it provides the capability of varying certain input data in order to obtain wave equation solutions for a range of input variables. The program provides for a maximum of 500 time cycles, instead of 300, to allow for the use of smaller time increments and a greater number of pile segments. Otherwise, the termination features are the same as for the BASIC PROGRAM.

Program Features. The program is designed with the following features that are different from or in addition to those of the basic program.

1. The ultimate resistance (Ru) is varied over a preselected range of values in any increment desired. The set is computed for each value of Ru. The initial value of Ru, the increment of increase of Ru and the number of times the increment is applied are provided as input data.

2. The effective length of embedment of the pile can be varied and is determined by input data. This is the length of the pile over which the resistance to driving acts.

3. Five patterns of resistance distribution between point bearing and side friction can be applied for computation. These patterns vary from full point bearing to full side friction with intermediate distributions of 75, 50, and 25 per cent point bearing with the balance distributed as side friction.

4. The method of distributing side friction can be rectangular or triangular as desired. Input data determine the selection of the method.

5. The capability of recycling the entire program up to three times with the new values of any variable is provided. This feature was used to provide new values of Q and J, although other variables could be used. This capability is provided with a control statement, and the new values of the variables to be changed must be put into the program itself, a somewhat awkward arrangement.

6. Any number of sets of data can be used in a single run, as provided for in the input data. This is also a feature of the basic program.

Input Data. The input data is the same as for the basic program except for the additional data required.

CARD ORDER	DESCRIPTION
2a	After the second card of the basic program, areas of the pile head, center and tip in square inches are punched on one card in the first three ten-column fields. Values are punched with a decimal point.
After the last card of the basic program	In the first four columns punch 1 to indicate triangular distribution of resistance on the side or -1 to indicate rectangular distribution.
Next card	In the first 10 columns punch the initial value of Ru in pounds, and in the second 10-column field the increment of Ru in pounds. A decimal point must be used. These are variables RU1 and ADD.
Next card	In the first four columns punch the number representing the number of times the increment of Ru is to be applied; this establishes the maximum value of Ru. In the second four columns (5 - 8)

punch the number of blocks in the mathematical model to which ground resistance is not to be applied; this establishes the effective embedded length of the pile. These variables are ICYCLE and NOLOAD and are punched without a sign or decimal point.

Next card — A number of 1 through 5 is punched in each of the first three four-column fields for the variables LA, LB, and LC respectively. These numbers provide the values for the indexing parameters of a DO statement that determines the distribution of resistance between point and side.

Next and last card — A number with a value between 1 through 4 is punched in the first four columns. This number controls the number of times the program is to be repeated with new variables. No sign or decimal is used. This variable is called IRES. If the number 1 is used the program will cycle the full three times, and if 4 is used it will stop after all computations using the initial data are made.

Output Data. In addition to listing all input data, the following output is provided:

1. Value of Ru in pounds for the particular computation.
2. Set in blows per inch.
3. Compression and tension stresses at the head, center and tip of the pile, in pounds per square inch.
4. The number of time cycles run.

Use of the Program. This program is designed to provide the data necessary to plot Ultimate Resistance vs. Set curves for various values of Q, J, and J', and for one or more of the five different distributions of resistance between point and side. The values of Q, J, and J' are initially provided in the input data. If additional values are desired for computations, they are provided through the use of the "computed-go-to" statement following statement number 600. The

particular values must be placed in the program following statements 700, 701, and 702. This feature could easily be expanded if desired. If it is desired to use all five distributions of resistance between point and side, the values of LA, LB, and LC must be 1, 5, and 1 respectively. When the index variable of the DO statement (five statements after 777) takes different values, the distribution of resistance is as indicated below:

Value	Distribution
1	All point bearing
2	75 per cent point bearing, 25 per cent friction
3	50 per cent point bearing, 50 per cent friction
4	25 per cent point bearing, 75 per cent friction
5	All side friction

By assigning proper values to LA, LB, and LC, the desired distributions are obtained in accordance with the operation of a DO statement as explained in reference (21).

```
*ID     REESE          C2165    CIVIL ENGR    G WAVE EQ 15   250   VARY RU
*       XEQ
*       LABEL
C       WAVE EQUATION VARY RU 101
        DIMENSION DS(100), DM(100), DL(100),VS(100), VM(100), VL(100)
        DIMENSION DPSM(100), DPSL(100)
        DIMENSION S(100), W(100), DP(500), R(100), SP(100),CS(100)
        DIMENSION CM(100), CL(100),FS(100), FM(100), FL(100),DPSS(100)
207     READ INPUT TAPE 5,27,NCASE,ITURN
 27     FORMAT (I10/I10)
        READ INPUT TAPE 5,71,AREA1,AREA2,AREA3
 71     FORMAT(3F10.4)
      0 READ INPUT TAPE 5, 70, M,IM1,T,(W(I),I=1,M),(S(I),I=1,IM1)
      1  ,(SP(I),I=1,M), RES1, RES2, V,Q,Z,ZP,CAP,RUB
 70     FORMAT (2I4/(F10.4))
        READ INPUT TAPE 5,604,MUD
604     FORMAT(I4)
        READ INPUT TAPE 5,72,RU1,ADD,ICYCLE,NOLOAD
 72     FORMAT (2F10.4/2I4)
        READ INPUT TAPE 5,73,LA,LB,LC,IRES
 73     FORMAT (3I4/I4)
        WRITE OUTPUT TAPE 6,30,NCASE
 30     FORMAT (12H CASE NUMBER I5)
        WRITE OUTPUT TAPE 6,28,(I,W(I),S(I),SP(I),I=1,M)
 28     FORMAT (5H DATA//58H    M                   W                   S
      1          SP/(I4,3F20.2))
        WRITE OUTPUT TAPE 6,29,RES1,RES2,ZP,Z,Q,V
 29     FORMAT (36H COEF. OF REST. OF CAPBLOCK (RES1) = F4.2/
      1 36H COEF. OF REST. OF PILECAP (RES2) =  F4.2/
      2 36H SIDE DAMPING FACTOR (ZP) =          F4.2/
      3 36H POINT DAMPING FACTOR (Z) =          F4.2/
      4 36H GROUND QUAKE (Q) =                  F4.2/
      5 36H INITIAL RAM VELOCITY (V) =          F6.2)
        WRITE OUTPUT TAPE 6,605,MUD
605     FORMAT (6H MUD = I6)
        WRITE OUTPUT TAPE 6,31,T,CAP,RUB
 31     FORMAT (36H TIME INTERVAL =                     F8.6/
      1 36H CAP =                              F6.2/
      2 36H RUB =                              F6.2//)
        WRITE OUTPUT TAPE 6,608,RU1,ADD,ICYCLE,NOLOAD
608     FORMAT (5H RU = E20.4,10H     ADD = E20.4/9H ICYCLE = I6,
      1 13H     NOLOAD = I6)
        WRITE OUTPUT TAPE 6,609,LA,LB,LC,IRES
609     FORMAT (5H LA = I4,10H     LB = I4,10H     LC = I4,10H     IRES =
      1  I4)
        CHECK = 2.* V
777     CONTINUE
        RU = RU1
        DO 600 IAM = 1,ICYCLE
```

```
      ITEM = M - NOLOAD
      AITEM = ITEM
      DO 602 J = LA,LB,LC
      FMAX1 = 0.0
      FMAX2 = 0.0
      FMAX3 = 0.0
      FTEN1 = 0.0
      FTEN2 = 0.0
      FTEN3 = 0.0
      DO 500 I = 1,100
      DS(I) = 0.0
      DM(I) = 0.0
      DL(I) = 0.0
      VS(I) = 0.0
      VM(I) = 0.0
      VL(I) = 0.0
      DPSM(I) = 0.0
      DPSL(I) = 0.0
      R(I) = 0.0
      SP(I) = 0.0
      CS(I) = 0.0
      CM(I) = 0.0
      CL(I) = 0.0
      FS(I) = 0.0
      FM(I) = 0.0
      FL(I) = 0.0
500   DPSS(I) = 0.0
      DO 501 I = 1,500
501   DP(I) = 0.0
      DS(1) = V*12.*T
      VS(1) =     V +(-DS(1)*S(1))*T*32.2/W(1)
      VS(2) = (DS(1)*S(1))*T*32.2/W(2)
      DM(1) = DS(1) + VS(1)*12.*T
      DM(2) = VS(2)*12.*T
      CM(2) = DM(2)
      VM(1) = VS(1) -((DM(1)-DM(2))*S(1))*T*32.2/W(1)
      VM(2) = VS(2)+((DM(1)-DM(2))*S(1)-(DM(2)-DM(3))*S(2))*T*32.2/W(2)
      VM(3) = VS(3)+((DM(2)-DM(3))*S(2)-R(3))*T*32.2/W(3)
      CM(1) = DM(1) - DM(2)
      AJ = J
      PART = (5.-AJ)/4.
      SP(M) = PART*RU/Q
      SIDE = RU - PART*RU
      LOAD = NOLOAD + 1
      IF(MUD)650,651,652
650   DO 601 N = LOAD,IM1
601   SP(N) = SIDE/(AITEM*Q)
      SP(M) = SP(M) + SIDE/(AITEM*Q)
      GO TO 651
652   DO 603 N = 3,IM1
      NT = N-2
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      NT1 = 2*NT - 1
      IM2 = M-2
      ANT1 = NT1
      AIM2 = IM2
603   SP(N) = SIDE*ANT1/((AIM2**2)*Q)
      SP(M) = SP(M) + SIDE*(2.*AIM2-1.)/((AIM2**2)*Q)
651   CONTINUE
      DO 101 N = 3,500
      MLESS1 = M-1
      DO 130 I = 1,M
130   DL(I) = DM(I) + VM(I)*12.*T
      DO 131 I = 1,MLESS1
131   CL(I) = DL(I) - DL(I+1)
13    IF(DL(M)) 12,12,14
12    DE = 0.0
      GO TO 24
14    IF(DL(M) - Q) 16,16,18
16    DE = 0.0
      GO TO 24
18    VALUE = DL(M) - Q
      IF (VALUE-DE) 22,22,20
20    DE = VALUE
      GO TO 24
22    DE = DE
24    DP(N) = DE
      R(M) = (DL(M)-DP(N))*SP(M)*(1.+Z*VM(M))
      IF (DP(N)) 25,25,26
26    IF (DP(N) - DP(N-1)) 197,197,25
25    CONTINUE
3     VAL = S(1)*CL(1)
      IF(CL(1)-CM(1)) 5,5,4
4     FL(1) = VAL
      GO TO 33
5     CL(1) = CM(1)
      FL(1)=VAL/(RES1**2)-(1./(RES1**2)-1.)*S(1)*CL(1)
33    VAL2 = S(2)*CL(2)
      IF(CL(2)-CM(2)) 35,35,34
34    FL(2) = VAL2
      GO TO 37
35    CL(2) = CM(2)
      FL(2) = VAL2/(RES2**2)-(1./(RES2**2)-1.)*S(2)*CL(2)
37    IF(CAP) 38,36,36
38    IF(FL(2)) 39,36,36
39    FL(2) = 0.0
36    CONTINUE
7     IF(FL(1)) 8,133,133
8     FL(1) = 0.0
133   DO 132 I = 3,MLESS1
132   FL(I) = CL(I)*S(I)
      IF(FL(2))213,211,210
213   IF(FTEN1-FL(2))211,211,214
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214   FTEN1 = FL(2)
      GO TO 211
210   IF(FMAX1-FL(2))215,211,211
215   FMAX1 = FL(2)
211   CONTINUE
      MC = M/2 + 1
      IF(FL(MC))216,217,218
216   IF(FTEN2-FL(MC))217,217,219
219   FTEN2 = FL(MC)
      GO TO 217
218   IF(FMAX2-FL(MC))220,217,217
220   FMAX2 = FL(MC)
217   CONTINUE
      IF(FL(IM1))221,222,224
221   IF(FTEN3-FL(IM1))222,222,223
223   FTEN3 = FL(IM1)
      GO TO 222
224   IF(FMAX3-FL(IM1))225,222,222
225   FMAX3 = FL(IM1)
222   CONTINUE
 42   IF(RUB)49,49,44
 44   DO 48 I = 3,MLESS1
      DPSL(I) = DPSM(I)
      CHANGE = DL(I) - Q
      SUM = DL(I) + Q
      IF(DPSL(I)-CHANGE)45,46,46
 45   DPSL(I) = CHANGE
 46   IF(DPSL(I)-SUM)48,48,47
 47   DPSL(I) = SUM
 48   R(I) = (DL(I) - DPSL(I))*SP(I)*(1.+ZP*VM(I))
 49   DO 50 I = 1,M
      VL(I) = VM(I) + (FL(I-1) - FL(I) - R(I))*T*32.2/W(I)
 50   CONTINUE
      DO 75 K = 1,M
      X = 1.
      IF (VL(K))     75,75,800
 75   CONTINUE
      GO TO 199
800   CONTINUE
      IF(VL(2) - CHECK)99,99,190
 99   IF(VL(M)-CHECK)102,102,194
102   DO 98 K = 1,M
      DS(K) = DM(K)
      DM(K) = DL(K)
      DL(K) = 0.0
      VS(K) = VM(K)
      VM(K) = VL(K)
      VL(K) = 0.0
      CS(K) = CM(K)
      CM(K) = CL(K)
      CL(K) = 0.0
      FS(K) = FM(K)
```

```
      VM(K) = FL(K)
      FL(K) = 0.0
      DPSS(K) = DPSM(K)
      DPSM(K) = DPSL(K)
 98   DPSL(K) = 0.0
101   CONTINUE
      GO TO 196
190   WRITE OUTPUT TAPE 6,191,N
191 0 FORMAT (64H VELOCITY OF PILE CAP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I3)
      GO TO 196
194   WRITE OUTPUT TAPE 6,195,N
195 0 FORMAT (64H VELOCITY OF PILE TIP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I13)
      GO TO 196
197   WRITE OUTPUT TAPE 6,198,N
198   FORMAT (26H DP BECAME CONSTANT AT N = I3)
      GO TO 196
199   WRITE OUTPUT TAPE 6,200,N
200   FORMAT (52H ALL VL WERE SIMULTANEOUSLY NEGATIVE OR ZERO AT N = I4)
196   CONTINUE
      WRITE OUTPUT TAPE 6,607,RU
607   FORMAT (27H ULTIMATE RESISTANCE (RU) = E15.4)
      WRITE OUTPUT TAPE 6,330,Q,W(1),V
330   FORMAT (4H Q = E10.2,7H W(1) = E15.4,4H V = E15.4)
      WRITE OUTPUT TAPE 6,606,(SP(L),L=1,M)
606   FORMAT (17H NEW VALUES OF SP/(8E15.4))
      WRITE OUTPUT TAPE 6,305,DP(N)
305   FORMAT (7H SET = E10.4)
      BLOW = 1./DP(N)
      WRITE OUTPUT TAPE 6,805,BLOW
805   FORMAT (66H
    1 BLOWS/IN= E10.3)
      FTEN1 = FTEN1/AREA1
      FTEN2 = FTEN2/AREA2
      FTEN3 = FTEN3/AREA3
      WRITE OUTPUT TAPE 6,307,FTEN1,FTEN2,FTEN3
307 0 FORMAT(30H MAXIMUM TENSION AT HEAD =     E15.4/
    1 30H MAXIMUM TENSION AT CENTER =  E15.4/
    2 30H MAXIMUM TENSION AT TIP =     E15.4)
      FMAX1 = FMAX1/AREA1
      FMAX2 = FMAX2/AREA2
      FMAX3 = FMAX3/AREA3
      WRITE OUTPUT TAPE 6,306,FMAX1,FMAX2,FMAX3
306 0 FORMAT (26H MAXIMUM FORCE AT HEAD =   E15.4/
    1 26H MAXIMUM FORCE AT CENTER = E15.4/
    2 26H MAXIMUM FORCE AT TIP =    E15.4//)
602   CONTINUE
      RU = RU + ADD
600   CONTINUE
```

```
      GO TO (700,701,702,703),IRES
700   CONTINUE
      Z = 0.20
      ZP = 0.067
      IRES = 2
      GO TO 777
701   CONTINUE
      Q = 0.20
      IRES = 3
      GO TO 777
702   CONTINUE
      Z = 0.15
      ZP = 0.05
      IRES = 4
      GO TO 777
703   CONTINUE
      IF(ITURN)205,205,207
205   CALL EXIT
      END
*     DATA
```

APPENDIX C

DETAILS OF THE "RESEARCHER COMPUTER PROGRAM" WHICH USES THE WAVE EQUATION

General: This program is written in Fortran language for the IBM 7090 computer, but it can be used equally well with the 709 or 7094 computer. It has as its objective the systematic production of data from which set vs. ultimate ground resistance curves can be plotted for any case. Its chief advantage lies in the fact that for every set of data run, the program automatically begins with a value of ground resistance of 20 tons and finds the set associated with all the load carried by the pile tip, 75% of load carried by the pile tip, 50% of load carried by the pile tip, 25% of load carried by the pile tip, and finally all the load carried by friction along the sides of the pile. It then assumes an increase of ground resistance of 20 tons and repeats the calculations. It automatically assigns new values of ground resistance in this manner up to and including 280 tons. Essentially, then, it progresses through the basic program five times at each value of ultimate ground resistance, or 70 times for each set of data. It also writes out the maximum compression and tension stresses in pounds per square inch for each of these 70 conditions at the head, mid-length, and tip of the pile. The side friction distribution may be chosen as rectangular or triangular in shape. In both of these cases the top 10 feet of the pile is considered free of frictional resistance.

Termination: Termination of each of the 70 conditions for each set of data is programmed exactly like the basic program. The reason for each termination is written out, and the computer immediately moves

[illegible]

[illegible]

[illegible]

to the next set of conditions. No over-all termination between sets of data should occur.

Stability: Besides the checks to detect instability in the calculations as described under the basic program, a further check is obtained when plotting set vs. ultimate ground resistance. This plot should also produce smooth curves. If not, a set of data for each suspicious point can be prepared and run with the basic program to obtain a detailed analysis so that evaluation can be made of the validity of the results. Reduced time intervals can be used to eliminate the instability, or smaller pile lengths in the mathematical model together with reduced time intervals may be used.

Preparation of Input Data: All data is punched on standard cards exactly as described in the basic program except that two additional cards are required for this program.

The first extra card is inserted between cards 2 and 3. In the first 10 spaces of this new card are punched, with a decimal point, the cross sectional area of the pile at its top, within spaces 11 through 20 is punched in the same manner the pile cross-sectional area at mid-length of the pile, and similarly the area of the pile tip in spaces 21 through 30.

The second extra card is included as the very last card of data. If side distribution is rectangular, punch -1 in spaces 3 and 4; if side distribution is to be triangular, punch +1 in spaces 3 and 4.

In all other respects, data is prepared exactly as described under instructions for the "basic computer program."

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

Running the program: The data deck of cards just described is placed immediately after the program deck (after the * DATA card) and is then ready to be processed by a machine operator.

Results: The input data will be repeated in the output information. The time cycle that the calculations were terminated for the first condition and the reason therefor will first appear. The ultimate ground resistance; the soil spring coefficients; set; and the maximum tension and compression at the pile head, mid-length, and tip will be written out for each condition. The results are printed in decimal form with a multiplier notation. For example, 0.1511 E01 means $0.1511 \times 10^1 = 1.511$.

The Program: The complete program is presented so that any who wish to use it may do so freely. Each line of the program is punched on a separate card and must be assembled in the exact order given below. Extreme care must be used in punching to prevent errors from creeping in, and for that reason it is recommended that test cases be run to check the program before relying on its results.

The notation used is as described under this section of instructions for the "basic computer program."

```
*ID   REESE          C2165   CIVIL ENGR   G WAVE EQ   4 100    RESEARCHER
*     XEQ
*     NOBIN
*     LABEL
C     WAVE EQUATION  VARY RU
      DIMENSION DS(100), DM(100), DL(100),VS(100), VM(100), VL(100)
      DIMENSION DPSM(100), DPSL(100)
      DIMENSION S(100), W(100), DP(300), R(100), SP(100),CS(100)
      DIMENSION CM(100), CL(100),FS(100), FM(100), FL(100),DPSS(100)
207   READ INPUT TAPE 5,27,NCASE,ITURN
 27   FORMAT (I10/I10)
      READ INPUT TAPE 5,71,AREA1,AREA2,AREA3
 71   FORMAT(3F10.4)
    0 READ INPUT TAPE 5, 70, M,IM1,T,(W(I),I=1,M),(S(I),I=1,IM1)
    1  ,(SP(I),I=1,M), RES1, RES2, V,Q,Z,ZP,CAP,RUB
 70   FORMAT (2I4/(F10.4))
      READ INPUT TAPE 5,604,MUD
604   FORMAT(I4)
      WRITE OUTPUT TAPE 6,30,NCASE
 30   FORMAT (12H CASE NUMBER I5)
      WRITE OUTPUT TAPE 6,28,(I,W(I),S(I),SP(I),I=1,M)
 28   FORMAT (5H DATA//58H    M                    W                   S
    1          SP/(I4,3F20.2))
      WRITE OUTPUT TAPE 6,29,RES1,RES2,ZP,Z,Q,V
 29   FORMAT (36H COEF. OF REST. OF CAPBLOCK (RES1) = F4.2/
    1 36H COEF. OF REST. OF PILECAP (RES2) =  F4.2/
    2 36H SIDE DAMPING FACTOR (ZP) =          F4.2/
    3 36H POINT DAMPING FACTOR (Z) =          F4.2/
    4 36H GROUND QUAKE (Q) =                  F4.2/
    5 36H INITIAL RAM VELOCITY (V) =          F6.2)
      WRITE OUTPUT TAPE 6,605,MUD
605   FORMAT (6H MUD = I6)
      WRITE OUTPUT TAPE 6,31,T,CAP,RUB
 31   FORMAT (36H TIME INTERVAL =                   F8.6/
    1 36H CAP =                              F6.2/
    2 36H RUB =                              F6.2//)
      CHECK = 2.* V
      RU = 40000.
       DO 600 I = 1,14
      ITEM = M-3
      AITEM = ITEM
      DO 602 J = 1,5
      FMAX1 = 0.0
      FMAX2 = 0.0
      FMAX3 = 0.0
      FTEN1 = 0.0
      FTEN2 = 0.0
      FTEN3 = 0.0
      DO 500 I = 1,100
```

```
      DS(I) = 0.0
      DM(I) = 0.0
      DL(I) = 0.0
      VS(I) = 0.0
      VM(I) = 0.0
      VL(I) = 0.0
      DPSM(I) = 0.0
      DPSL(I) = 0.0
      R(I) = 0.0
      SP(I) = 0.0
      CS(I) = 0.0
      CM(I) = 0.0
      CL(I) = 0.0
      FS(I) = 0.0
      FM(I) = 0.0
      FL(I) = 0.0
500   DPSS(I) = 0.0
      DO 501 I = 1,300
501   DP(I) = 0.0
      DS(1) = V*12.*T
      VS(1) =     V +(-DS(1)*S(1))*T*32.2/W(1)
      VS(2) = (DS(1)*S(1))*T*32.2/W(2)
      DM(1) = DS(1) + VS(1)*12.*T
      DM(2) = VS(2)*12.*T
      CM(2) = DM(2)
      VM(1) = VS(1) -((DM(1)-DM(2))*S(1))*T*32.2/W(1)
      VM(2) = VS(2)+((DM(1)-DM(2))*S(1)-(DM(2)-DM(3))*S(2))*T*32.2/W(2)
      VM(3) = VS(3)+((DM(2)-DM(3))*S(2)-R(3))*T*32.2/W(3)
      CM(1) = DM(1) - DM(2)
      AJ = J
      PART = (5.-AJ)/4.
      SP(M) = PART*RU/Q
      SIDE = RU - PART*RU
      IF(MUD)650,651,652
650   DO 601 N = 4,IM1
601   SP(N) = SIDE/(AITEM*Q)
      SP(M) = SP(M) + SIDE/(AITEM*Q)
      GO TO 651
652   DO 603 N = 3,IM1
      NT = N-2
      NT1 = 2*NT - 1
      IM2 = M-2
      ANT1 = NT1
      AIM2 = IM2
603   SP(N) = SIDE*ANT1/((AIM2**2)*Q)
      SP(M) = SP(M) + SIDE*(2.*AIM2-1.)/((AIM2**2)*Q)
651   CONTINUE
      DO 101 N = 3, 300
      MLESS1 = M-1
      DO 130 I = 1,M
130   DL(I) = DM(I) + VM(I)*12.*T
```

```
      DO 131 I = 1,MLESS1
131   CL(I) = DL(I) - DL(I+1)
 13   IF(DL(M)) 12,12,14
 12   DE = 0.0
      GO TO 24
 14   IF(DL(M) - Q) 16,16,18
 16   DE = 0.0
      GO TO 24
 18   VALUE = DL(M) - Q
      IF (VALUE-DE) 22,22,20
 20   DE = VALUE
      GO TO 24
 22   DE = DE
 24   DP(N) = DE
      R(M) = (DL(M)-DP(N))*SP(M)*(1.+Z*VM(M))
      IF (DP(N)) 25,25,26
 26   IF (DP(N) - DP(N-1)) 197,197,25
 25   CONTINUE
  3   VAL = S(1)*CL(1)
      IF(CL(1)-CM(1)) 5,5,4
  4   FL(1) = VAL
      GO TO 33
  5   CL(1) = CM(1)
      FL(1)=VAL/(RES1**2)-(1./(RES1**2)-1.)*S(1)*CL(1)
 33   VAL2 = S(2)*CL(2)
      IF(CL(2)-CM(2)) 35,35,34
 34   FL(2) = VAL2
      GO TO 37
 35   CL(2) = CM(2)
      FL(2) = VAL2/(RES2**2)-(1./(RES2**2)-1.)*S(2)*CL(2)
 37   IF(CAP) 38,36,36
 38   IF(FL(2)) 39,36,36
 39   FL(2) = 0.0
 36   CONTINUE
  7   IF(FL(1)) 8,133,133
  8   FL(1) = 0.0
133   DO 132 I = 3,MLESS1
132   FL(I) = CL(I)*S(I)
      IF(FL(2))213,211,210
213   IF(FTEN1-FL(2))211,211,214
214   FTEN1 = FL(2)
      GO TO 211
210   IF(FMAX1-FL(2))215,211,211
215   FMAX1 = FL(2)
211   CONTINUE
      MC = M/2 + 1
      IF(FL(MC))216,217,218
216   IF(FTEN2-FL(MC))217,217,219
219   FTEN2 = FL(MC)
      GO TO 217
218   IF(FMAX2-FL(MC))220,217,217
220   FMAX2 = FL(MC)
```

```
217   CONTINUE
      IF(FL(IM1))221,222,224
221   IF(FTEN3-FL(IM1))222,222,223
223   FTEN3 = FL(IM1)
      GO TO 222
224   IF(FMAX3-FL(IM1))225,222,222
225   FMAX3 = FL(IM1)
222   CONTINUE
 42   IF(RUB)49,49,44
 44   DO 48 I = 3,MLESS1
      DPSL(I) = DPSM(I)
      CHANGE = DL(I) - Q
      SUM = DL(I) + Q
      IF(DPSL(I)-CHANGE)45,46,46
 45   DPSL(I) = CHANGE
 46   IF(DPSL(I)-SUM)48,48,47
 47   DPSL(I) = SUM
 48   R(I) = (DL(I) - DPSL(I))*SP(I)*(1.+ZP*VM(I))
 49   DO 50 I = 1,M
      VL(I) = VM(I) + (FL(I-1) - FL(I) - R(I))*T*32.2/W(I)
 50   CONTINUE
      DO 75 K = 1,M
      X = 1.
      IF (VL(K))     75,75,800
 75   CONTINUE
      GO TO 199
800   CONTINUE
      IF(VL(2) - CHECK)99,99,190
 99   IF(VL(M)-CHECK)102,102,194
102   DO 98 K = 1,M
      DS(K) = DM(K)
      DM(K) = DL(K)
      DL(K) = 0.0
      VS(K) = VM(K)
      VM(K) = VL(K)
      VL(K) = 0.0
      CS(K) = CM(K)
      CM(K) = CL(K)
      CL(K) = 0.0
      FS(K) = FM(K)
      FM(K) = FL(K)
      FL(K) = 0.0
      DPSS(K) = DPSM(K)
      DPSM(K) = DPSL(K)
 98   DPSL(K) = 0.0
101   CONTINUE
      GO TO 196
190   WRITE OUTPUT TAPE 6,191,N
191 0 FORMAT (64H VELOCITY OF PILE CAP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I3)
      GO TO 196
```

```
194     WRITE OUTPUT TAPE 6,195,N
195 0 FORMAT (64H VELOCITY OF PILE TIP EXCEEDED TWICE THE RAM VELOCITY W
    1 HEN N WAS I3)
        GO TO 196
197     WRITE OUTPUT TAPE 6,198,N
198     FORMAT (26H DP BECAME CONSTANT AT N = I3)
        GO TO 196
199     WRITE OUTPUT TAPE 6,200,N
200     FORMAT (52H ALL VL WERE SIMULTANEOUSLY NEGATIVE OR ZERO AT N = I4)
196     CONTINUE
        WRITE OUTPUT TAPE 6,607,RU
607     FORMAT (27H ULTIMATE RESISTANCE (RU) = E15.4)
        WRITE OUTPUT TAPE 6,606,(SP(L),L=1,M)
606     FORMAT (17H NEW VALUES OF SP/(8E15.4))
        WRITE OUTPUT TAPE 6,305,DP(N)
305     FORMAT (7H SET = E10.4)
        BLOW = 1./DP(N)
        WRITE OUTPUT TAPE 6,805,BLOW
805     FORMAT(66H
    1 BLOWS/IN= E10.3)
        FTEN1 = FTEN1/AREA1
        FTEN2 = FTEN2/AREA2
        FTEN3 = FTEN3/AREA3
        WRITE OUTPUT TAPE 6,307,FTEN1,FTEN2,FTEN3
307 0 FORMAT(30H MAXIMUM TENSION AT HEAD =     E15.4/
    1 30H MAXIMUM TENSION AT CENTER =  E15.4/
    2 30H MAXIMUM TENSION AT TIP =     E15.4)
        FMAX1 = FMAX1/AREA1
        FMAX2 = FMAX2/AREA2
        FMAX3 = FMAX3/AREA3
        WRITE OUTPUT TAPE 6,306,FMAX1,FMAX2,FMAX3
306 0 FORMAT (26H MAXIMUM FORCE AT HEAD =   E15.4/
    1 26H MAXIMUM FORCE AT CENTER = E15.4/
    2 26H MAXIMUM FORCE AT TIP =    E15.4//)
602     CONTINUE
        RU = RU + 40000.
600     CONTINUE
        IF(ITURN)205,205,207
205     CALL EXIT
        END
*       DATA
```

APPENDIX D

DETAILS OF THE "HILEY FORMULA COMPUTER PROGRAM"

General: This program is written in Fortran Language for the IBM 7090 computer, but may be used equally well with the 709 or 7094 computer. It has as its objective the systematic production of data of set vs. ultimate ground resistance using the Hiley type formula. The program will take given pile and hammer information and beginning with an ultimate ground resistance of 5 tons will range in increments of 5 tons until an ultimate ground resistance of 240 tons is reached. At each value of ground resistance the set is computed and written out. The program is arranged so that any type of hammer or pile can be used. The coefficients for use with the Hiley formula are as recommended by Chellis (ref. 5) and are built into the program except as indicated under input data preparation section below.

Termination: No conditions for termination are built into this program. If extensive use were planned for this program, an "if" statement should be added so that it will cause the calculations to terminate when the computed set becomes negative.

Input Data Preparation: All data is punched on standard data cards in the exact order indicated below. Upon completion of punching the data deck, it is placed immediately after the * DATA card of the source program.

CARD ORDER	DESCRIPTION
1	If another set of data follows this set for another case, punch +1 in spaces 9 and 10; if not, punch -1. Use no decimals.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]	[illegible]
[illegible]	[illegible]

2	If the hammer is double acting, punch +1 in spaces 9 and 10; if not, punch -1. Use no decimals.
3	If the pile is steel, punch +1 in spaces 9 and 10; if wood, -1 in spaces 9 and 10; if concrete, punch 0 in space 10. Use no decimals.
4	If triangular side distribution is desired, punch +1 in spaces 9 and 10; if rectangular side distribution is desired, punch -1 in spaces 9 and 10.
5	In the first 10 spaces punch the hammer efficiently using a decimal point.
6	In the first 10 spaces punch the weight of the ram in pounds using a decimal point.
7	In the first 10 spaces punch the coefficient of restitution using a decimal point.
8	In the first 10 spaces punch the weight of the pile including the shoe and driving cap for drop hammers and single acting steam hammers. Weight of pile including weight of anvil in case of double acting or differential acting steam hammers. All in pounds and with a decimal point.
9	In the first 10 spaces punch the cross sectional area in square inches of the pile at its head or the area of the capblock if one is used. Use a decimal point.
10	Same as card 9 but for the cross sectional pile area at mid-length of the pile.
11	Same as card 9 but for the cross sectional pile area at the pile tip.
12	Punch in the first 10 spaces the ground quake in inches, using a decimal point.
13	Punch in the first 10 spaces the modulus of elasticity in pounds per square inch using a decimal point.
14	Punch in the first 10 spaces the pile length in feet using a decimal point.

15 Punch in the first 10 spaces the height of free fall of the ram, in inches, for drop hammers

or

the normal (shortest) stroke of ram in inches for single acting steam hammers

or

for double acting, differential acting steam and diesel hammers use the rated energy per blow in foot pounds as published by the manufacturers.

Running the Program: The data deck of cards just described is placed immediately after the source program deck and is ready for delivery to a machine operator for processing.

Results: In the output will be repeated the original input information, values of the ultimate ground resistance, per cent of load carried by the point, and the set. The results are printed in decimal form with a multiplier so that 0.6192 E00, for example, means 0.6192×10^0, or 0.6192.

The Program: The complete program is presented so that any who wish to use it may do so. Each line of the program should be punched on a separate card and assembled in the exact order indicated. Anyone preparing cards from this print-out is cautioned to verify the program with test cases before relying on the results obtained.

```
*ID   REESE          C2165   CIVIL ENGR   G HILEY      7  150
*     XEQ
*     NOBIN
*     LABEL
C     HILEY FORMULA PROGRAM
  206 READ INPUT TAPE 5,20,ITURN,IEQUIP,IMAT,MUD
   20 FORMAT(I10)
      READ INPUT TAPE 5,21,E,WR,RES,WP,APH,APAV,APT,Q,ELAS,PL,H
   21 FORMAT (F10.4)
      IF(IMAT) 1,3,5
    1 WRITE OUTPUT TAPE 6,2
    2 FORMAT (12H WOODEN PILE)
      GO TO 7
    3 WRITE OUTPUT TAPE 6,4
    4 FORMAT (14H CONCRETE PILE)
      GO TO 7
    5 WRITE OUTPUT TAPE 6,6
    6 FORMAT(11H STEEL PILE)
    7 IF(IEQUIP) 8,12,10
    8 WRITE OUTPUT TAPE 6,9
    9 FORMAT(34H SINGLE ACTING OR DROP HAMMER USED)
      GO TO 12
   10 WRITE OUTPUT TAPE 6,11
   11 FORMAT(26H DOUBLE ACTING HAMMER USED)
   12 IF(MUD) 13,205,15
   13 WRITE OUTPUT TAPE 6,14
   14 FORMAT(33H SIDE DISTRIBUTION IS RECTANGULAR)
      GO TO 17
   15 WRITE OUTPUT TAPE 6,16
   16 FORMAT (32H SIDE DISTRIBUTION IS TRIANGULAR)
   17 WRITE OUTPUT TAPE 6,18,E
   18 FORMAT(13H EFFICIENCY= E10.4)
      WRITE OUTPUT TAPE 6,19,WR
   19 FORMAT (16H WEIGHT OF RAM= E10.4)
      WRITE OUTPUT TAPE 6,22,RES
   22 FORMAT (29H COEFFICIENT OF RESTITUTION= E10.4)
      WRITE OUTPUT TAPE 6,23,WP
   23 FORMAT (13H WT OF PILE= E10.4)
      WRITE OUTPUT TAPE 6,24,APH
   24 FORMAT (26H AREA PILE HEAD IN SQ IN= E10.4)
      WRITE OUTPUT TAPE 6,25,APAV
   25 FORMAT (20H AVERAGE PILE AREA= E10.4)
      WRITE OUTPUT TAPE 6,26,APT
   26 FORMAT (16H AREA PILE TIP= E10.4)
      WRITE OUTPUT TAPE 6,27,Q
   27 FORMAT (8H QUAKE= E10.4)
      WRITE OUTPUT TAPE 6,28,ELAS
   28 FORMAT (17H YOUNGS MODULUS= E10.4)
      WRITE OUTPUT TAPE 6,29,PL
   29 FORMAT (20H PILE LENGTH IN FT= E10.4)
      IF (IEQUIP) 31,35,33
```

```
 31 WRITE OUTPUT TAPE 6,32,H
 32 FORMAT (24H HAMMER DROP IN INCHES= E10.4)
     GO TO 35
 33 WRITE OUTPUT TAPE 6, 34, H
 34 FORMAT (40H RATED ENERGY OF HAMMER IN FOOT POUNDS= E10.4)
 35 RU = 10000.
    DO 204 J = 1,48
    WRITE OUTPUT TAPE 6,30,RU
 30 FORMAT (20H THE VALUE OF RU IS E10.4//)
    P1 = RU/APH
    P2 = RU/APAV
    P3 = RU/APT
    DO 203 I = 1,3
    IF(I-1) 60,40,42
 40 PLMOD = PL
    WPC = WP/2.
    WRITE OUTPUT TAPE 6,41
 41 FORMAT (29H ALL OF LOAD CARRIED BY POINT)
    GO TO 60
 42 IF(I-2) 60,44,52
 44 WRITE OUTPUT TAPE 6, 47
 47 FORMAT(31H ONE HALF LOAD CARRIED BY POINT)
    IF(MUD) 46,60,48
 46 PLMOD = 0.75 *PL
    WPC = WP
    GO TO 60
 48 PLMOD = 4. * PL / 6.
    WPC = WP
    GO TO 60
 52 IF(I-3) 60, 54, 60
 54 WRITE OUTPUT TAPE 6, 55
 55 FORMAT(29H ALL LOAD CARRIED BY FRICTION)
    IF(MUD) 56,60,58
 56 PLMOD = PL/2
    WPC = WP
    GO TO 60
 58 PLMOD = 2. * PL / 3.
    WPC = WP
 60 C3 = Q
    IF(IMAT) 70,80,90
 70 C1= 0.00010 * P1
    C2 = 0.00008*P2*PLMOD*1500000./ELAS
    GO TO 100
 80 C1= 0.00025 * P1
    C2 = 0.000004*P2*PLMOD*3000000./ELAS
    GO TO 100
 90 C1 = 0.00008 * P1
    C2 = 0.0000004*P2*PLMOD*30000000./ELAS
100 DP=(((E*WR*H)*(WR+WPC*RES**2))/((WR+WPC)*RU))-(C1+C2+C3)/2.
    IF(IEQUIP) 104,203,102
102 DP = DP + (C1+C2+C3)/2.
```

```
      DP = 12.*DP
      DP = DP -(C1+C2+C3)/2.
  104 BLOW = 1./DP
      WRITE OUTPUT TAPE 6,201,DP
  201 FORMAT(8H SET IS E20.4)
      WRITE OUTPUT TAPE 6,202,BLOW
  202 FORMAT(20H BLOWS PER INCH ARE E20.4/)
  203 CONTINUE
  204 RU = RU + 10000.
      IF (ITURN) 205,205,206
  205 CALL EXIT
      END
*     DATA
```

APPENDIX E

This appendix includes some of the graphs of computer solutions which were used both for correlation with actual pile cases and for studying the effect of varying the size of hammer, length of pile, ground quake, point damping, and side damping. On each sheet several curves are drawn for various values of frictional resistance expressed as a percentage of the total ultimate bearing capacity of the pile and indicated as follows:

Curve A -- End bearing; no side friction

Curve B -- 75 per cent end bearing; 25 per cent side friction.

Curve C -- 50 per cent end bearing; 50 per cent side friction.

Curve D -- 25 per cent end bearing; 75 per cent side friction.

Curve E -- 100% friction.

Rectangular side distribution was used. The description of the driving equipment, pile, and soil properties for which the computations were made appear on each sheet, but for convenience of use in future correlation work an index is included which summarizes this information for each sheet. The figure numbers and page numbers are identical in this appendix.

INDEX OF FIGURES IN APPENDIX E

1. Piles Driven by a Vulcan #1 Hammer

(80-foot 12BP53 Piles cont'd.)

Point Damping	Side Damping	Ground Quake	Figure Number
0.10	0.033	0.20	E21
0.20	0.067	0.20	E22
0.30	0.10	0.20	E23
0.40	0.133	0.20	E24
0.50	0.167	0.20	E25
0.10	0.033	0.30	E26
0.20	0.067	0.30	E27
c. 120-foot 12BP53 Piles			
0.0	0.0	0.10	E28
0.10	0.033	0.10	E29
0.20	0.067	0.10	E30
0.30	0.10	0.10	E31
0.40	0.133	0.10	E32
0.50	0.167	0.10	E33
0.10	0.033	0.20	E34
0.20	0.067	0.20	E35
0.30	0.10	0.20	E36
0.40	0.133	0.20	E37
0.50	0.167	0.20	E38
0.10	0.033	0.30	E39
0.20	0.067	0.30	E40
d. 18-inch-square concrete pile 40 feet long:			
0.15	0.0	0.10	E41

2. Piles Driven with a Vulcan O Hammer

a. 12BP53 Piles 80 feet long:

Point Damping	Side Damping	Ground Quake	Figure Number
0.20	0.067	0.10	E42
0.40	0.133	0.10	E43

b. 12BP53 Piles 45 feet long:

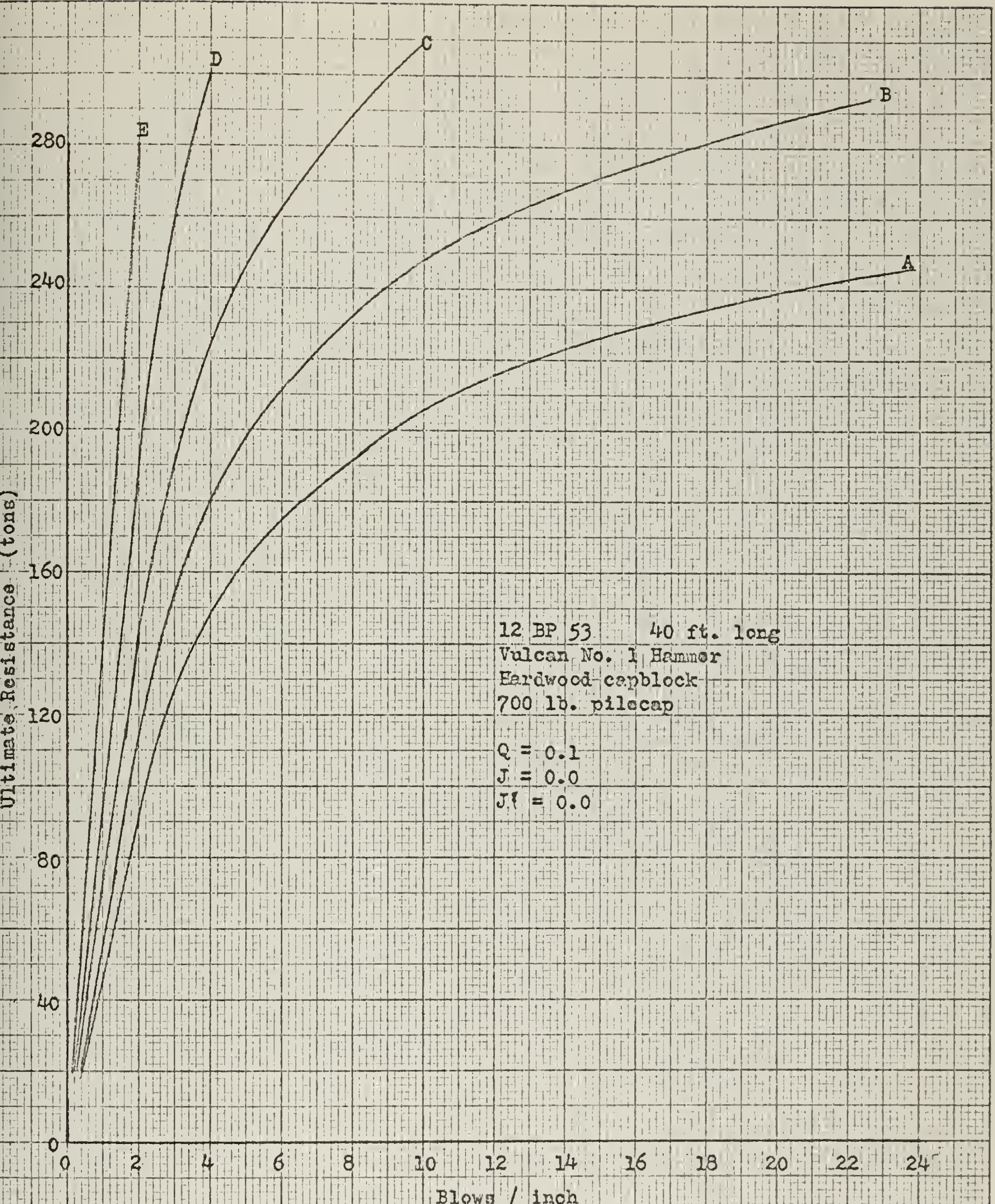
Point Damping	Side Damping	Ground Quake	Figure Number
0.10	0.033	0.10	E44

c. 14BP89 Piles 45 feet long:

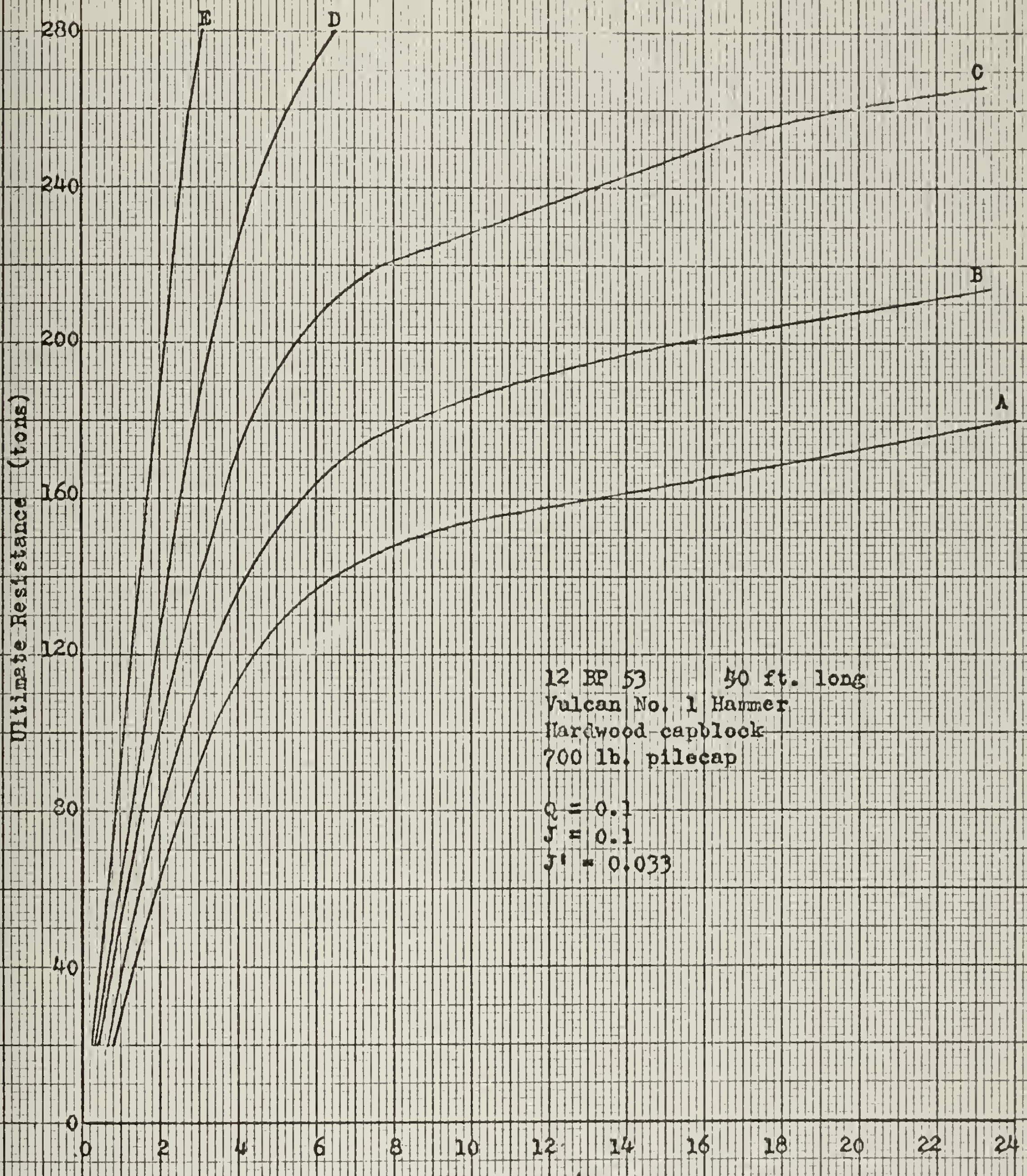
Point Damping	Side Damping	Ground Quake	Figure Number
0.15	0.05	0.10	E45
0.20	0.067	0.10	E46
0.20	0.067	0.20	E47

d. 14BP117 Piles 45 feet long:

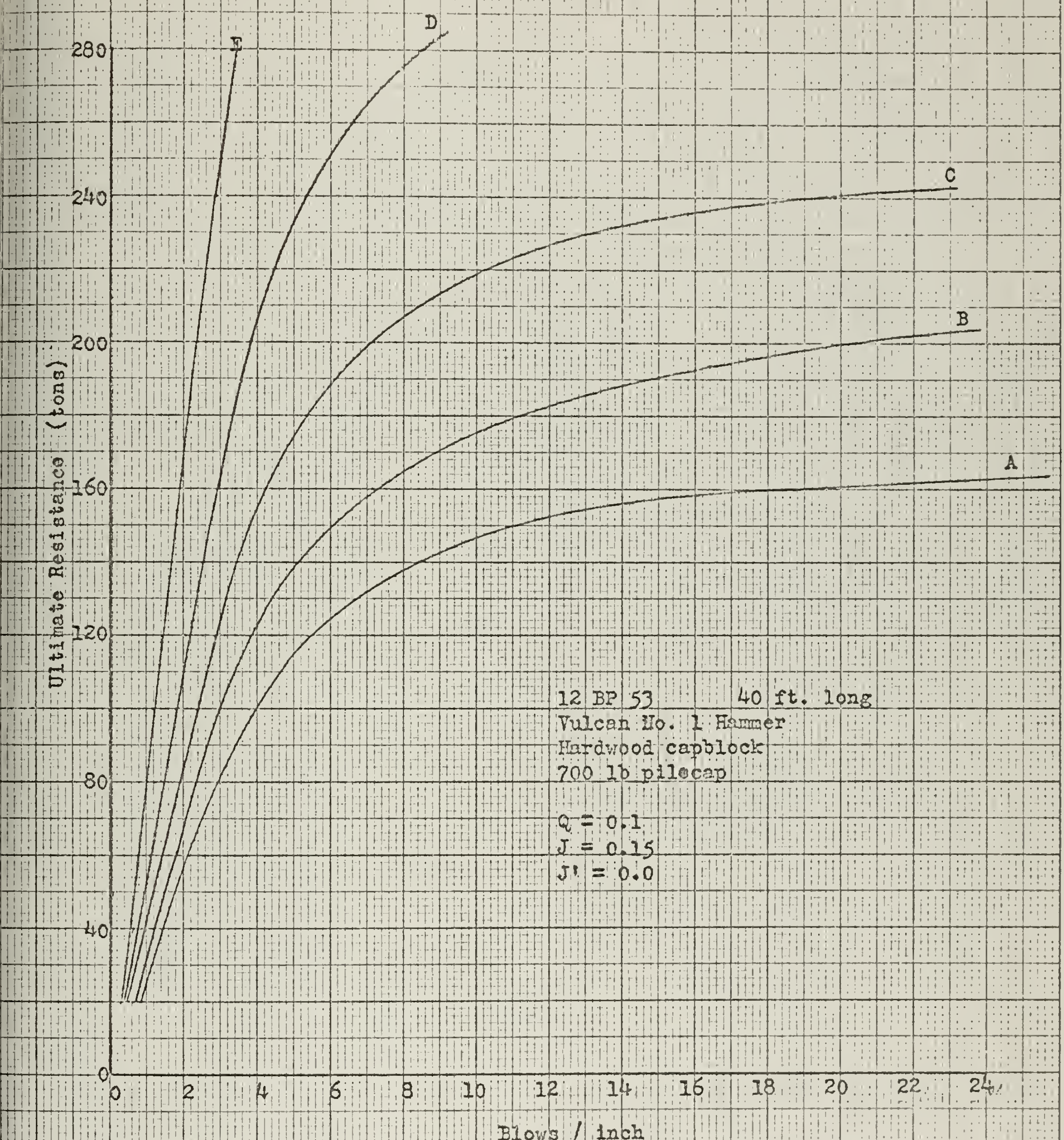
Point Damping	Side Damping	Ground Quake	Figure Number
0.15	0.05	0.10	E48
0.20	0.067	0.20	E49

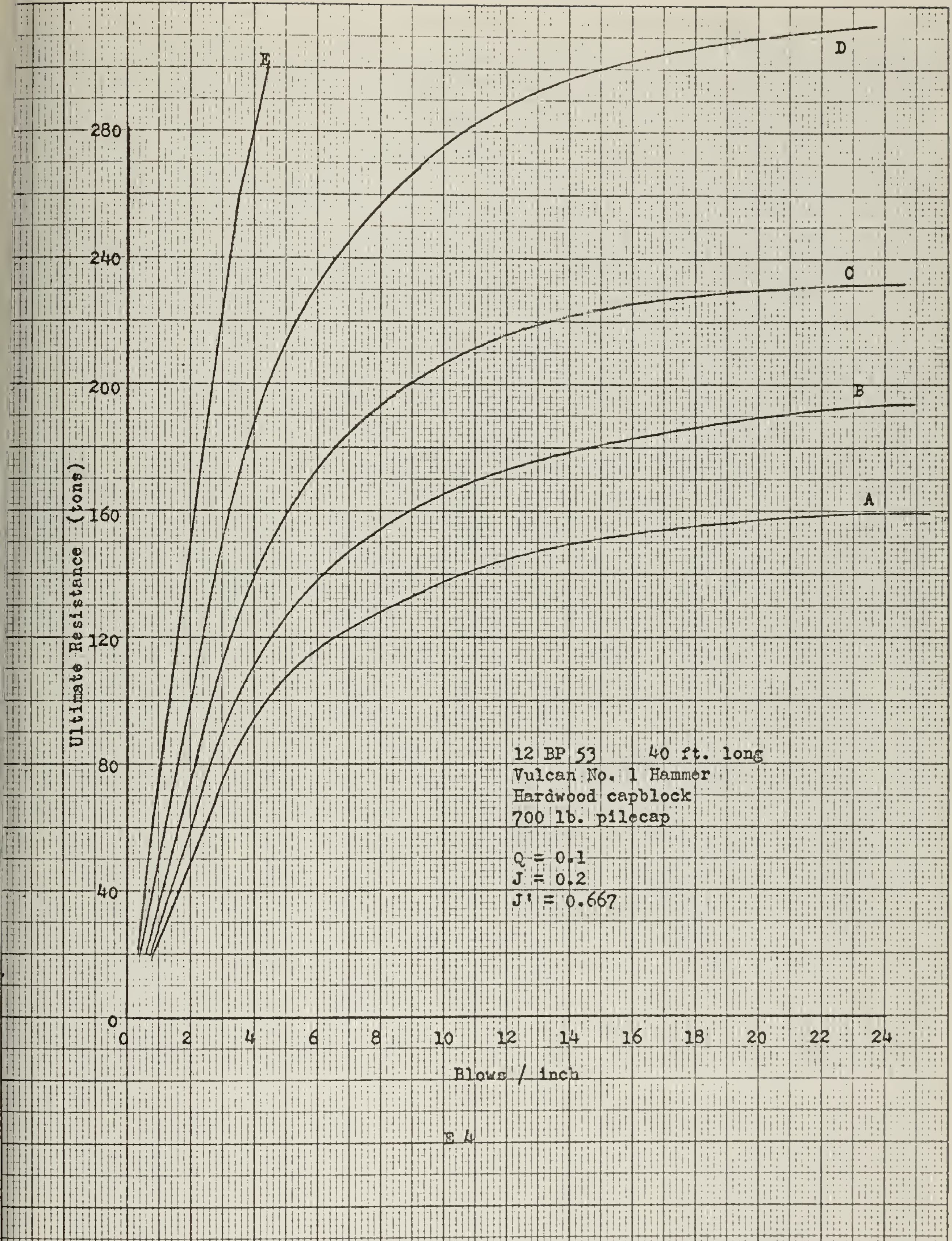

A
B
C
D
E
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Ultimate Resistance (tons)
Blows / inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.0
J' = 0.0

E 1

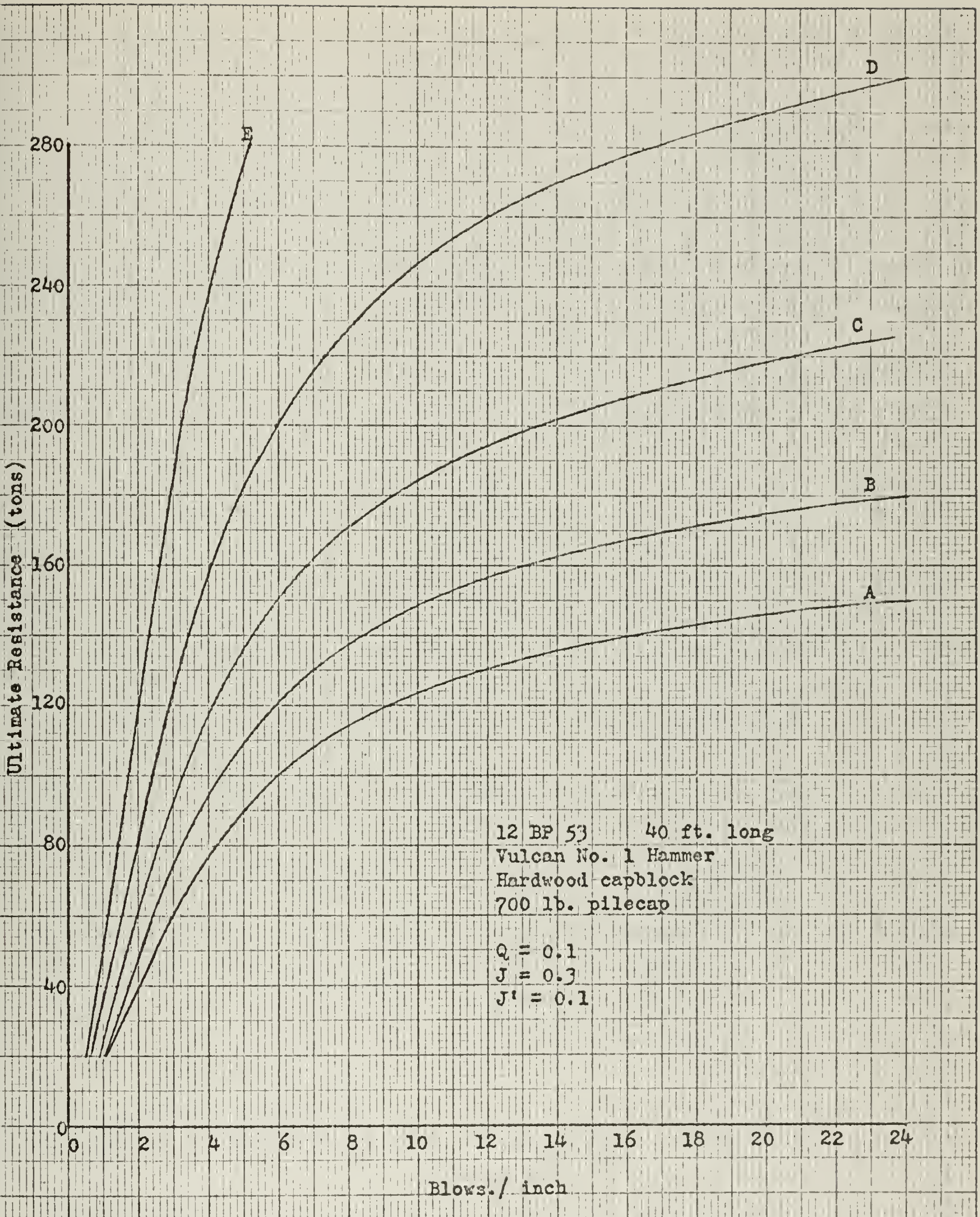


280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
E
D
C
B
A
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.1
J' = 0.033

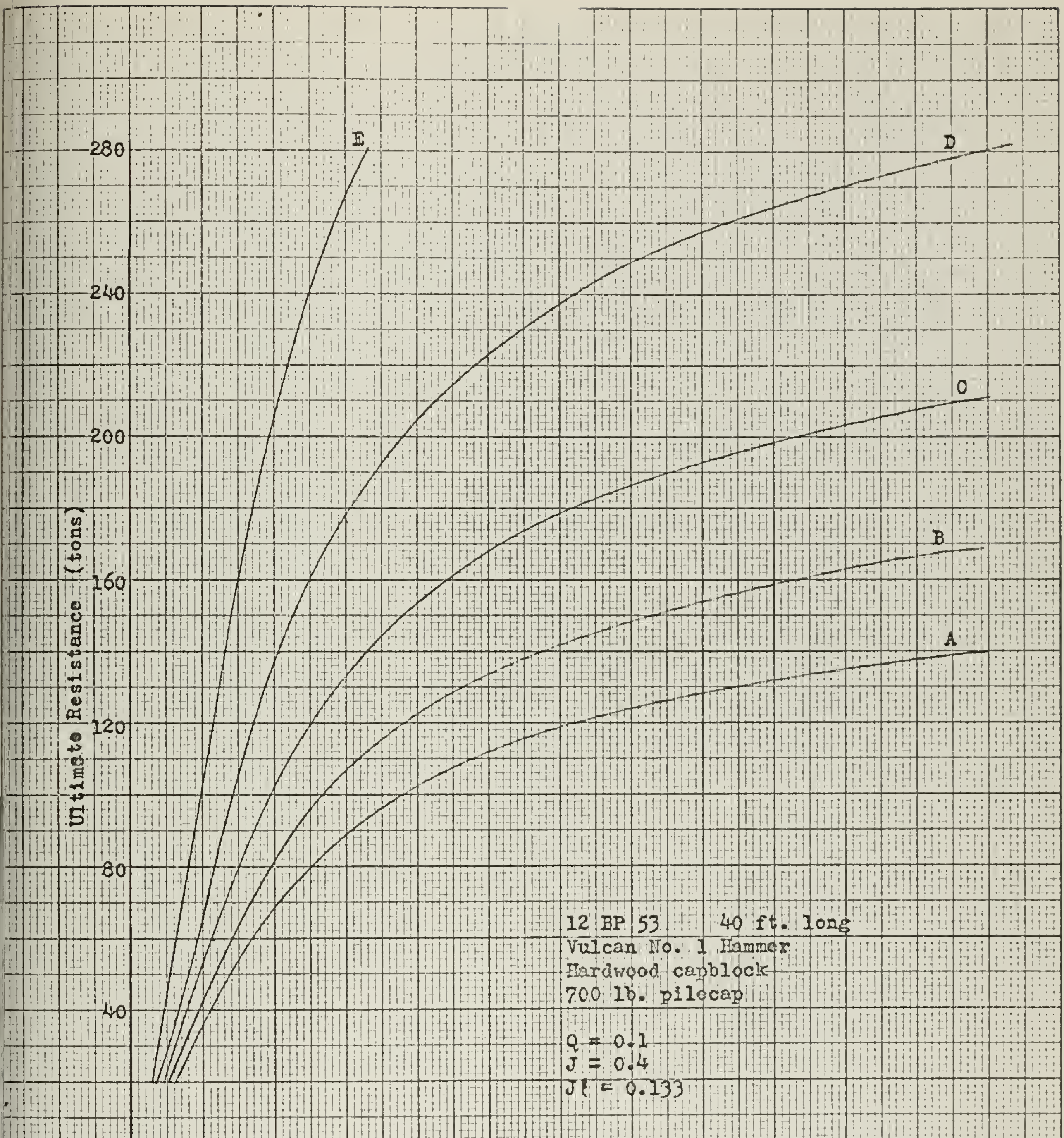

Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
E
D
C
B
A
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb pilecap
Q = 0.1
J = 0.15
J' = 0.0



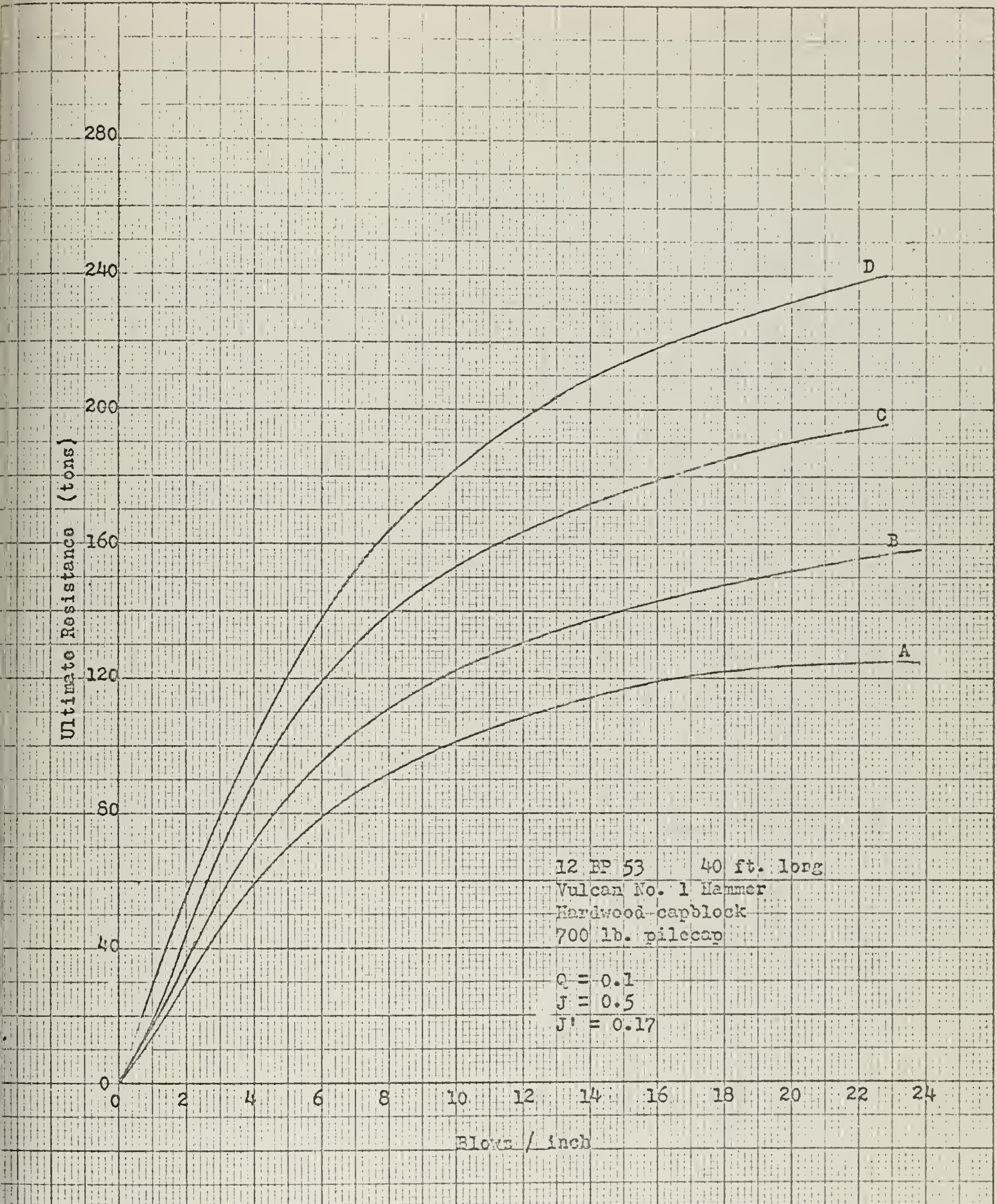
E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.2
J' = 0.667

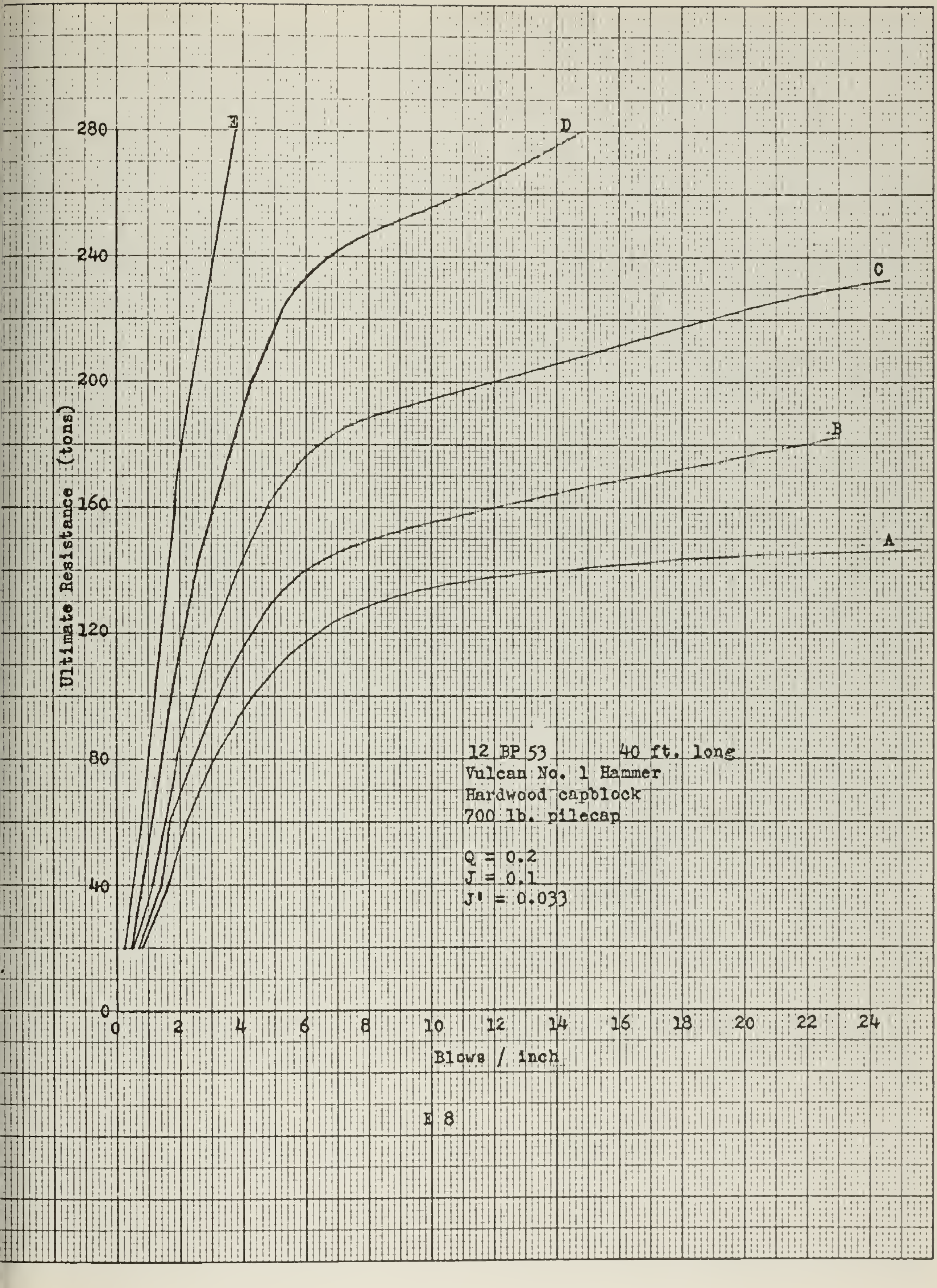


Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
Blows / inch
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.3
J' = 0.1


Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
Blows / inch
0
2
4
6
8
10
12
14
16
18
20
22
24
E
D
C
B
A
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.4
J' = 0.133

E 6


Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
D
C
B
A
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.5
J' = 0.17


Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
Blows / inch
0
2
4
6
8
10
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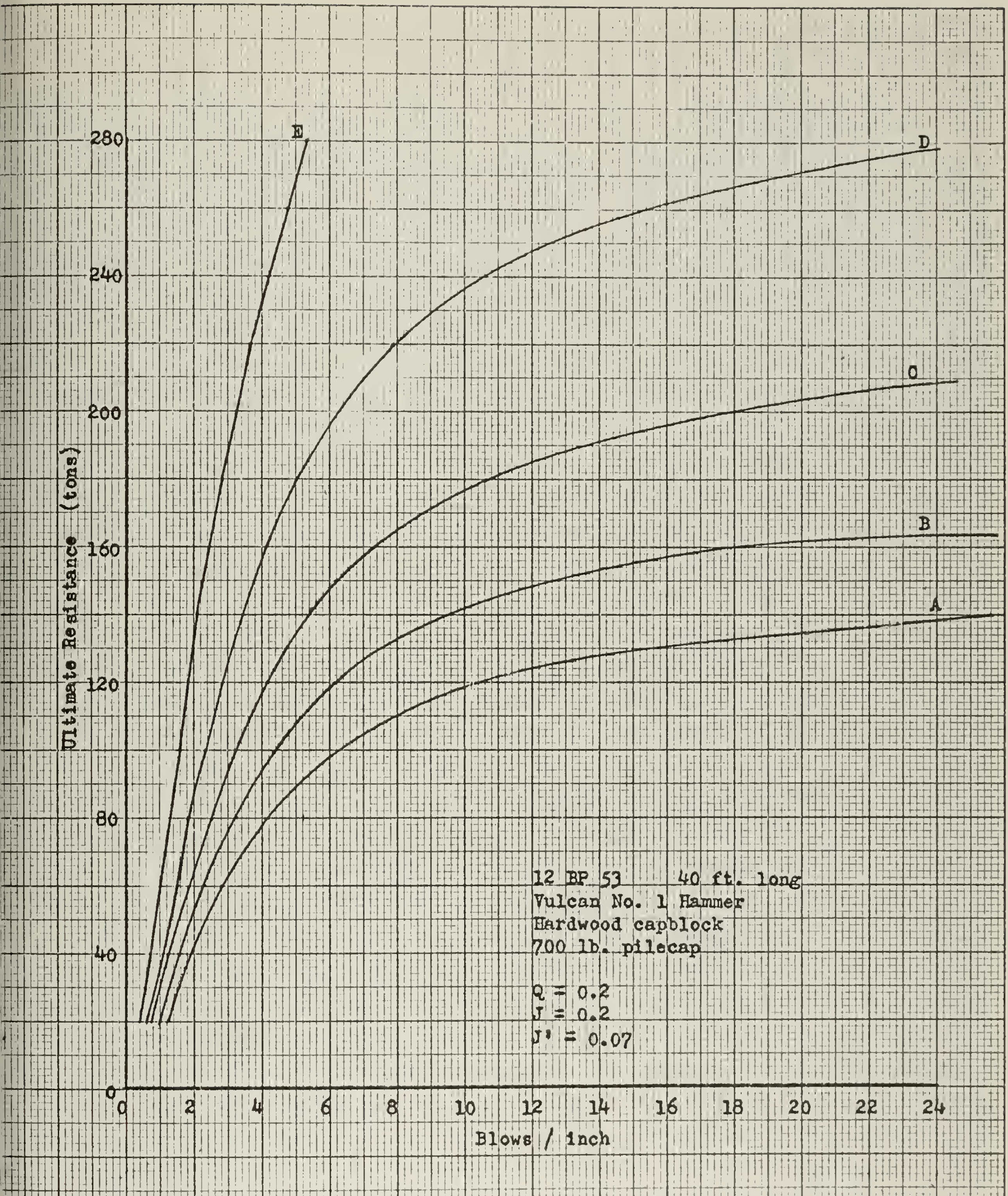
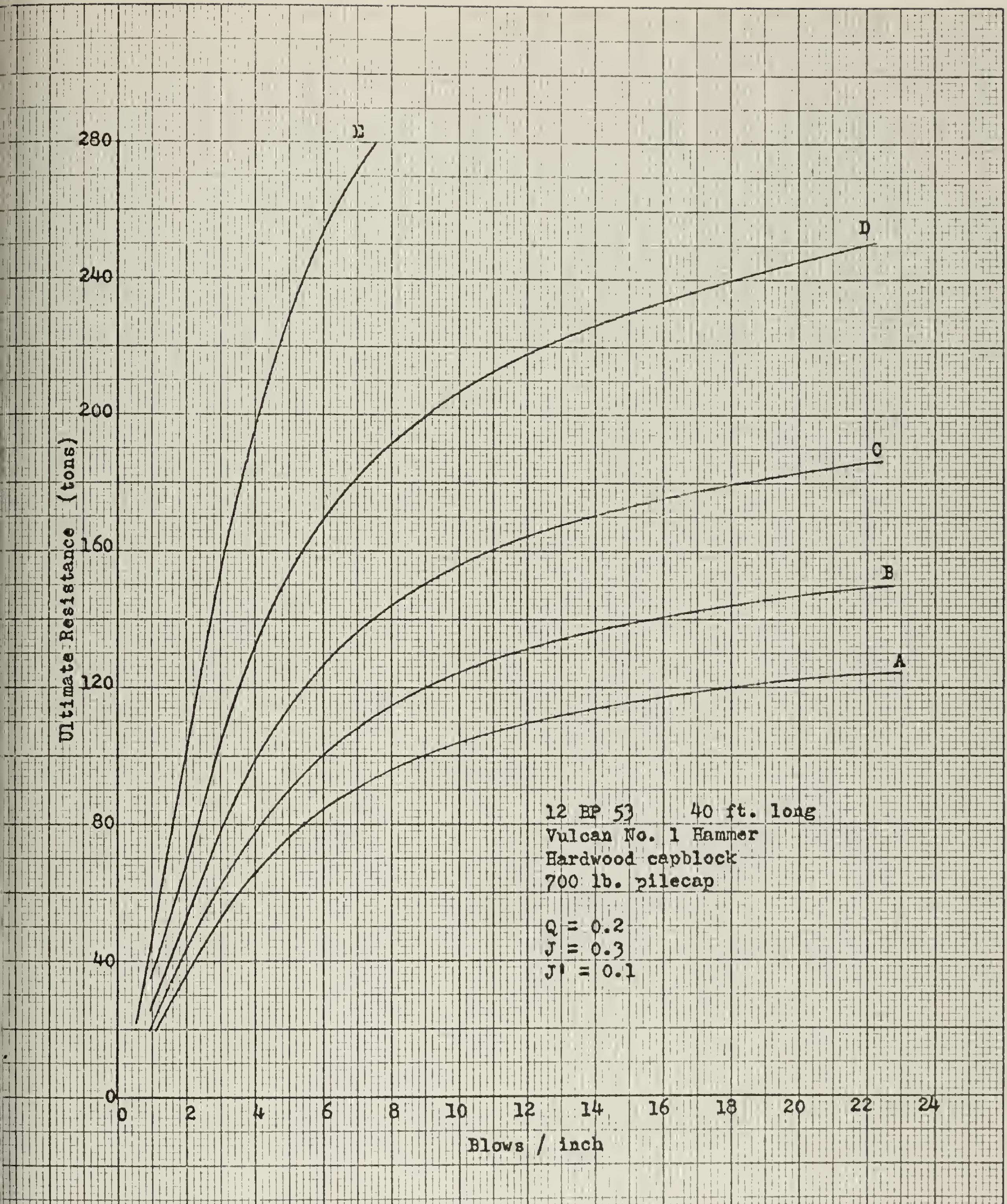
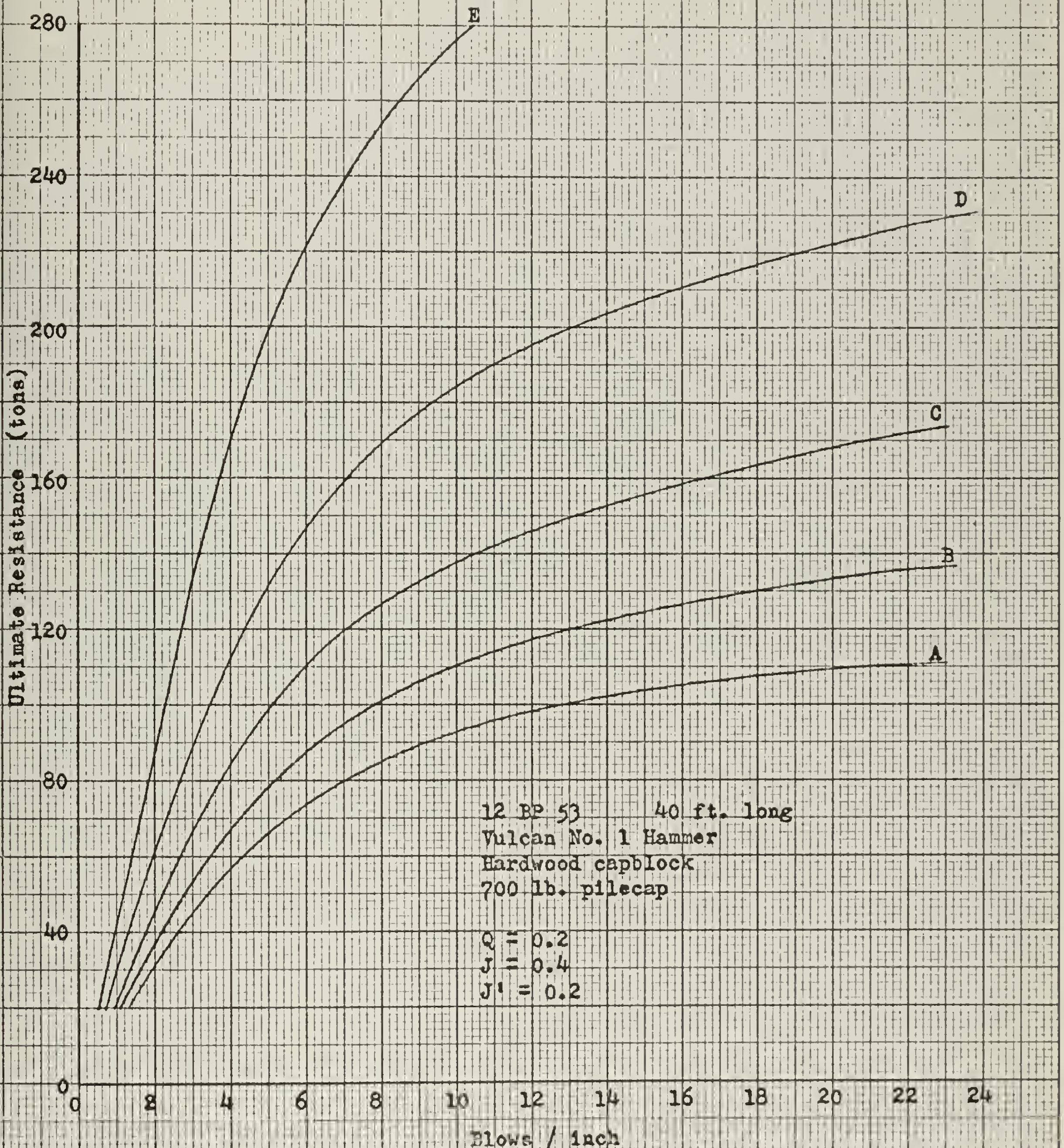
14
16
18
20
22
24
A
B
C
D
E
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.1
J' = 0.033

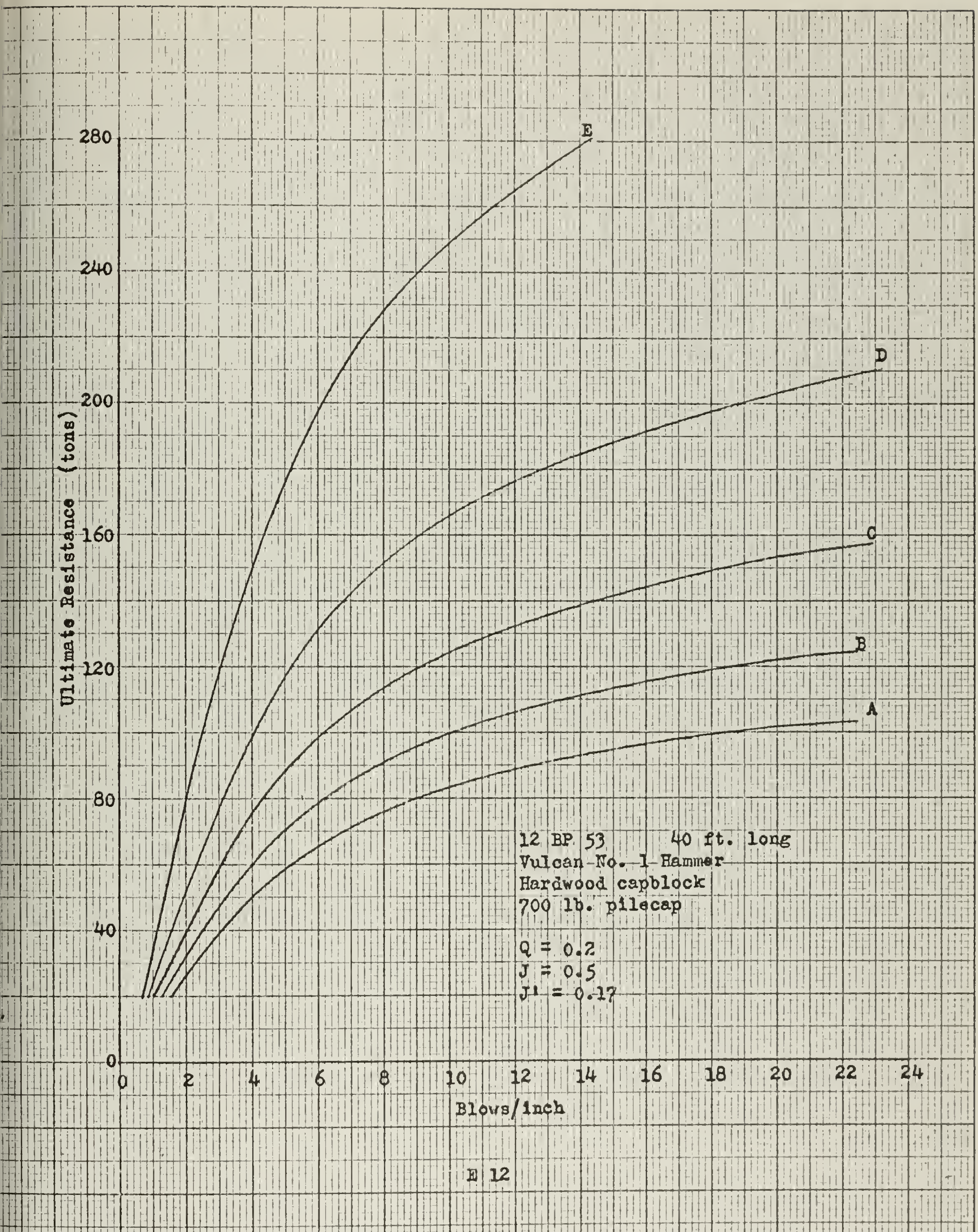

E
D
C
B
A
Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.2
J' = 0.07



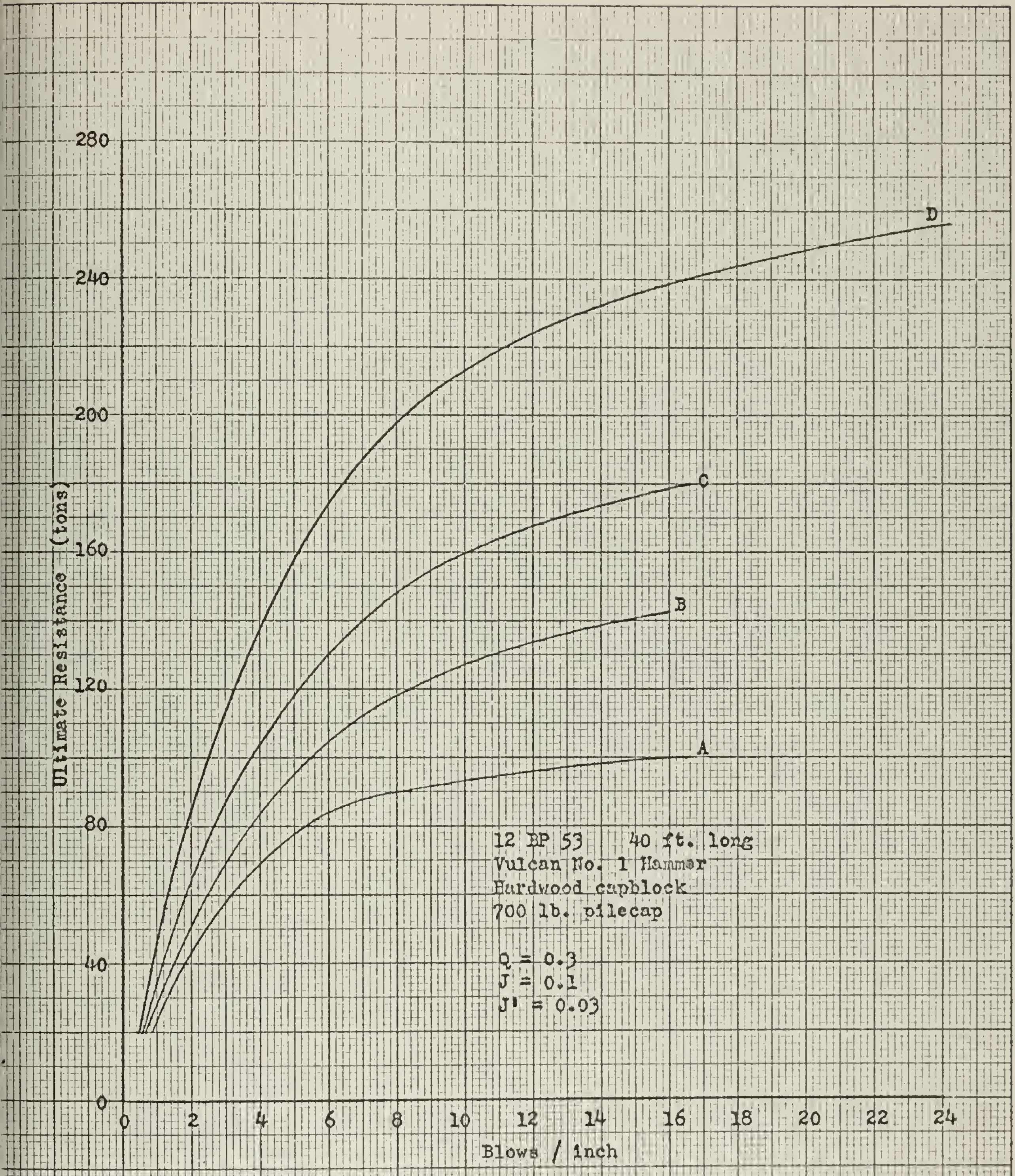
E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.3
J' = 0.1


280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
E
D
C
B
A
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.4
J' = 0.2

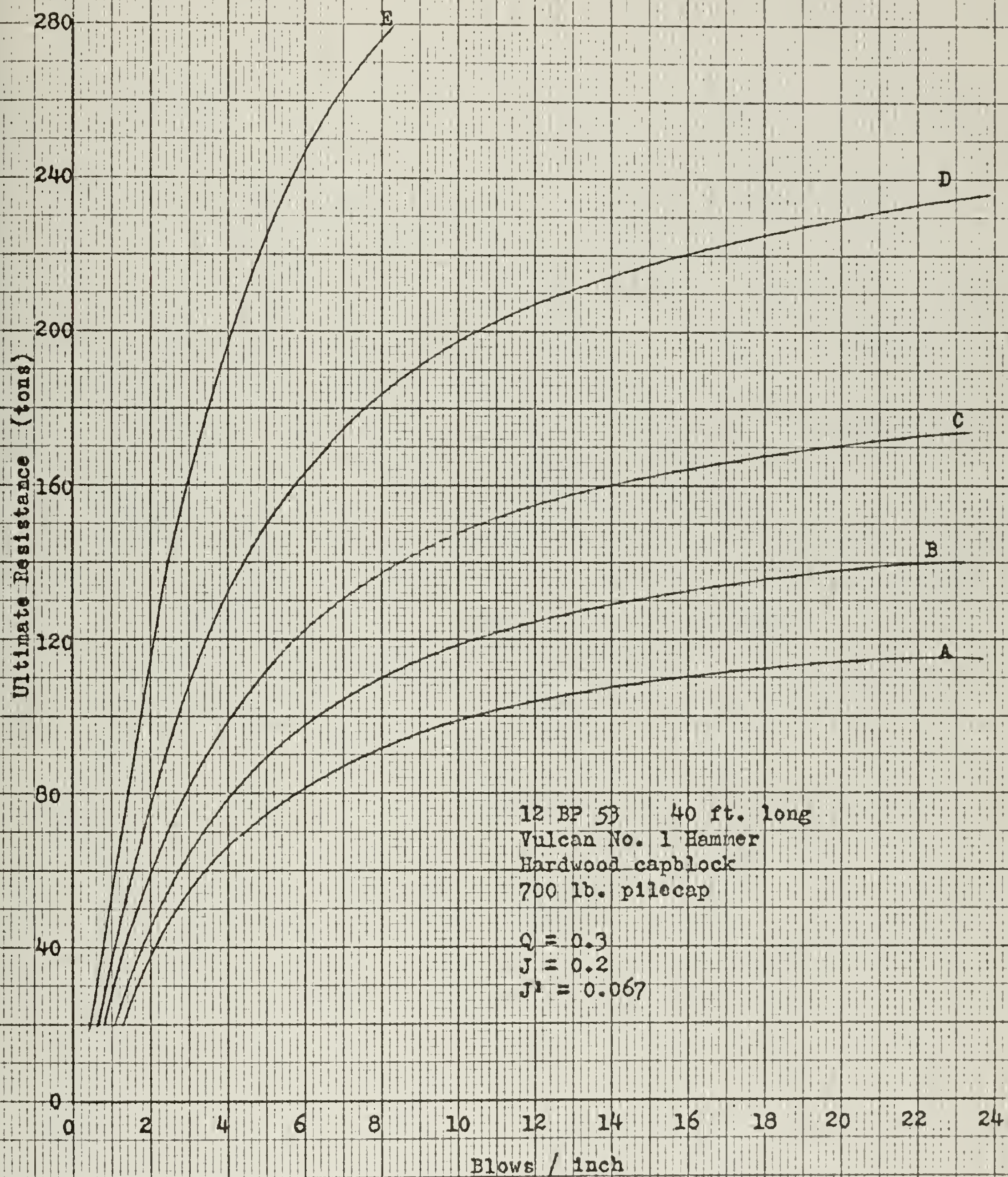
E 11



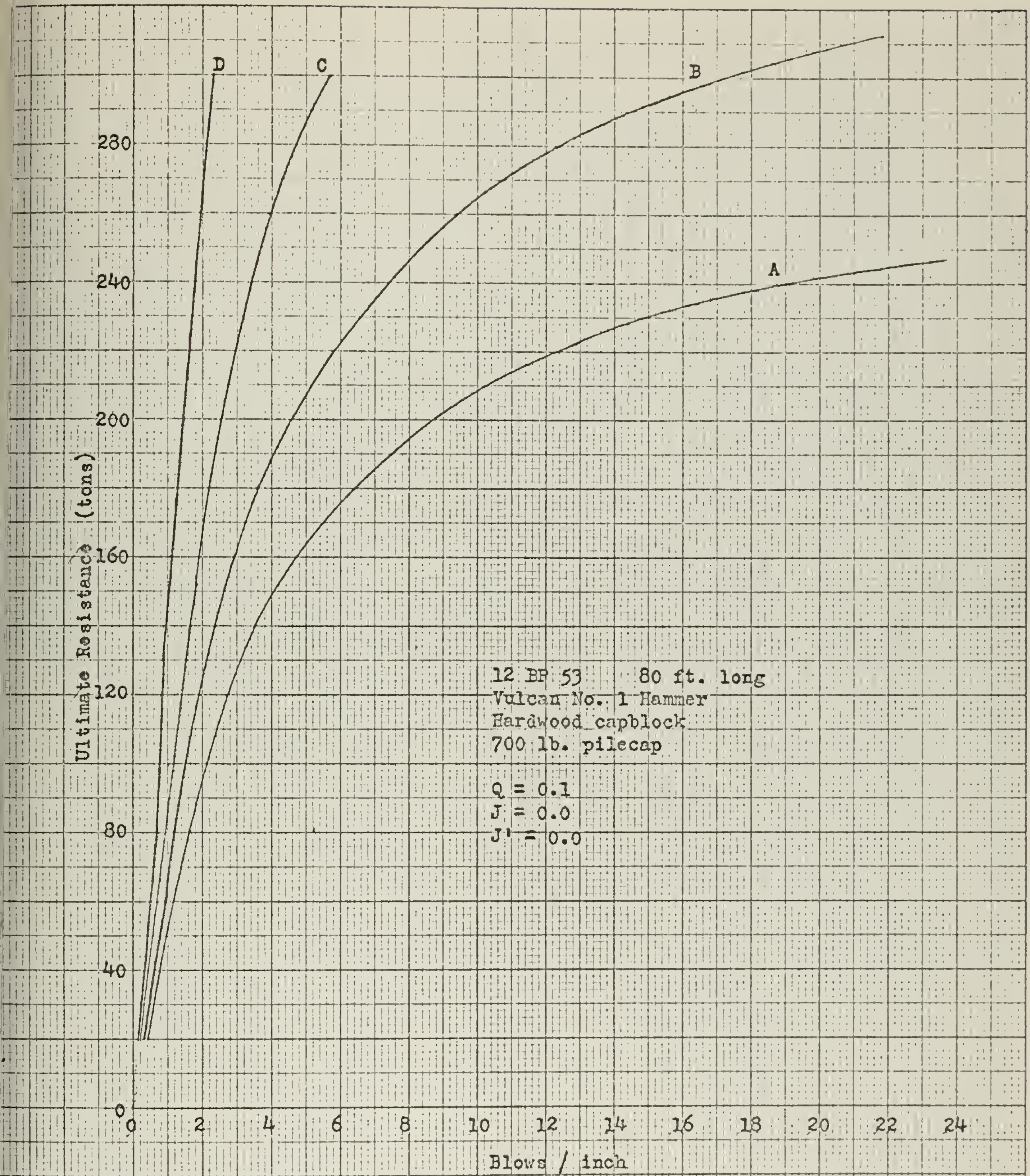
E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows/inch
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.5
J' = 0.17


Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0 2 4 6 8 10 12 14 16 18 20 22 24
Blows / inch
A
B
C
D
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.3
J = 0.1
J' = 0.03

E 13

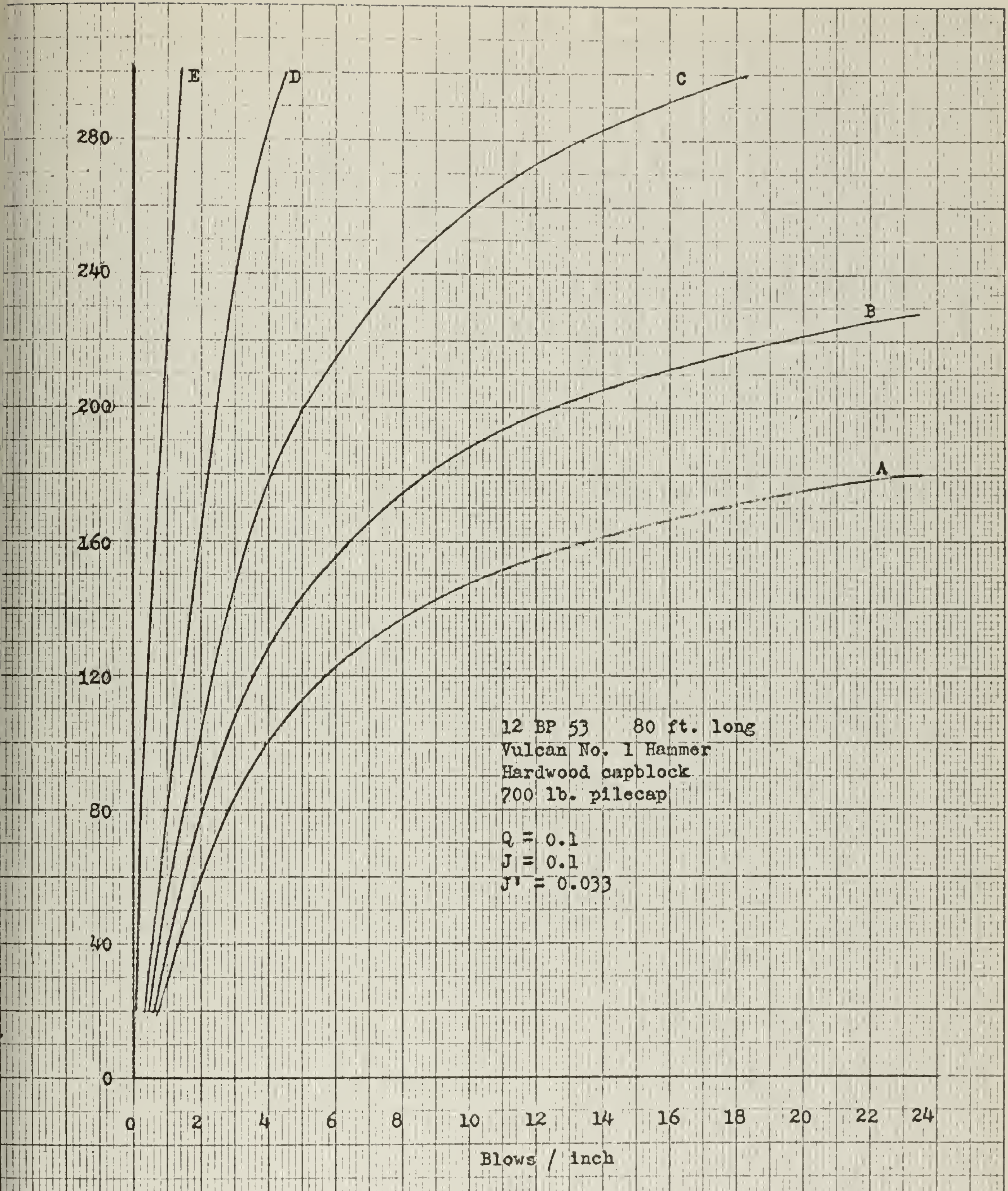


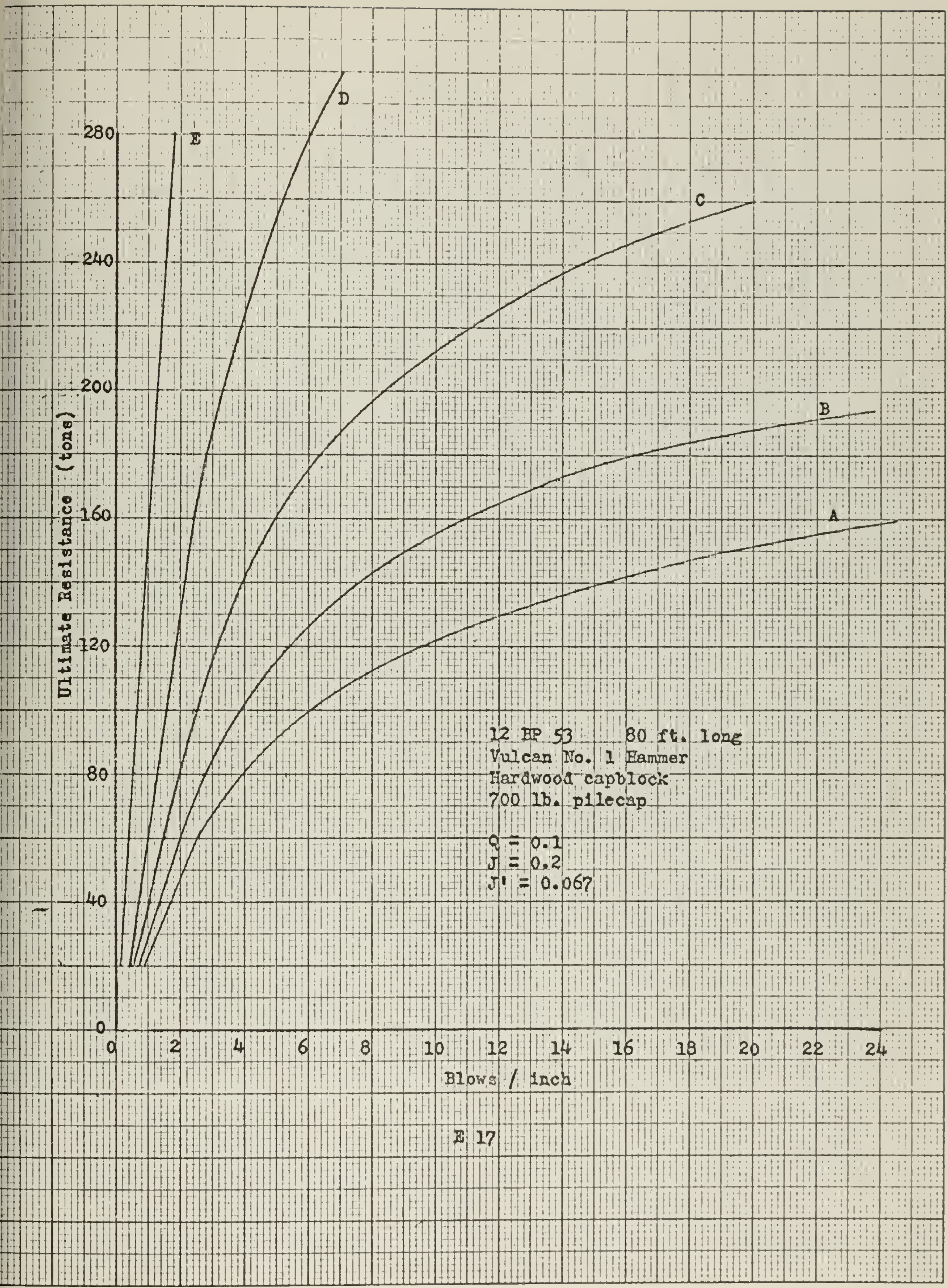
Ultimate Resistance (tons)
Blows / inch
A
B
C
D
E
12 BP 53 40 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.3
J = 0.2
J' = 0.067



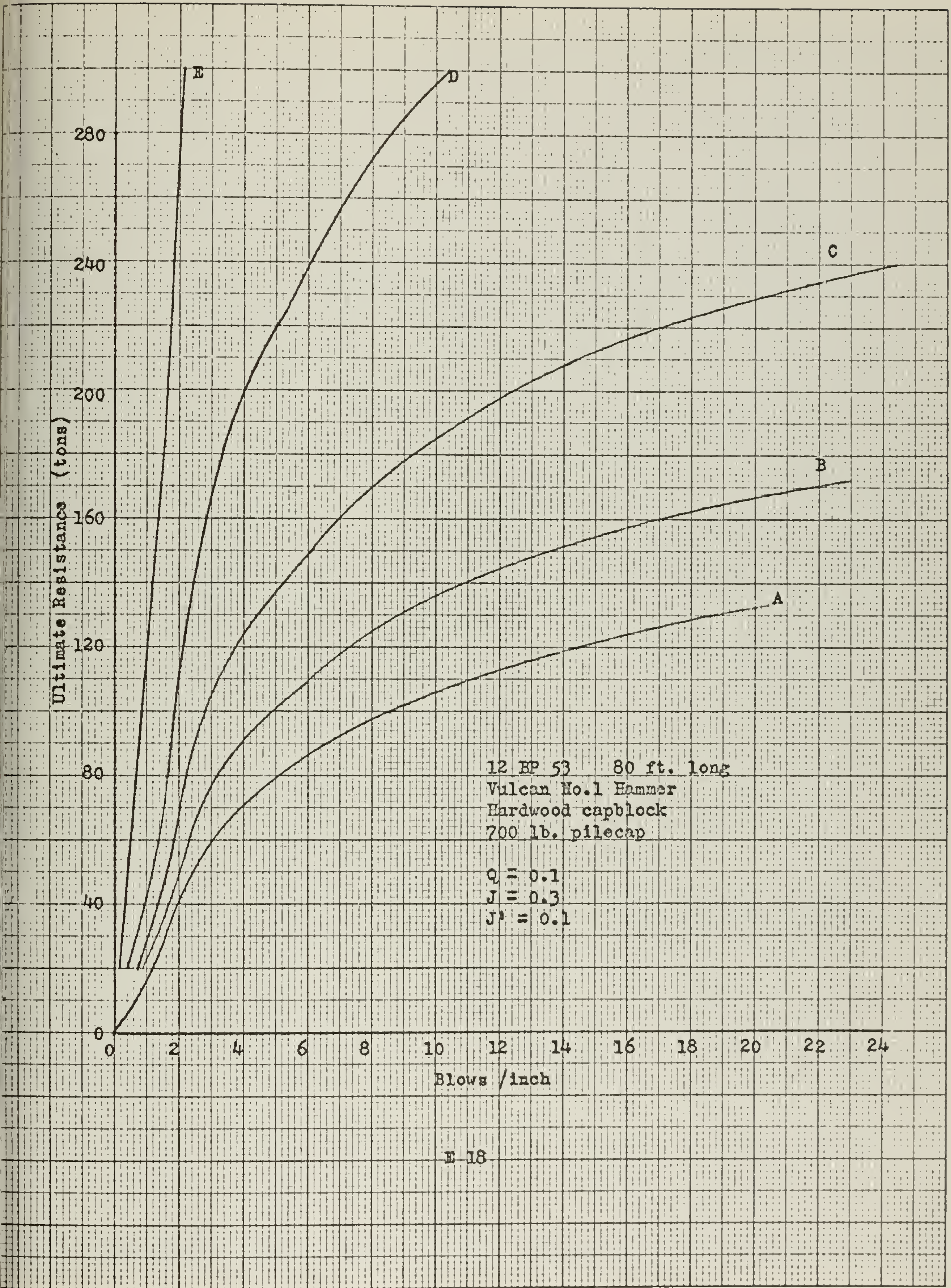
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.0
J' = 0.0

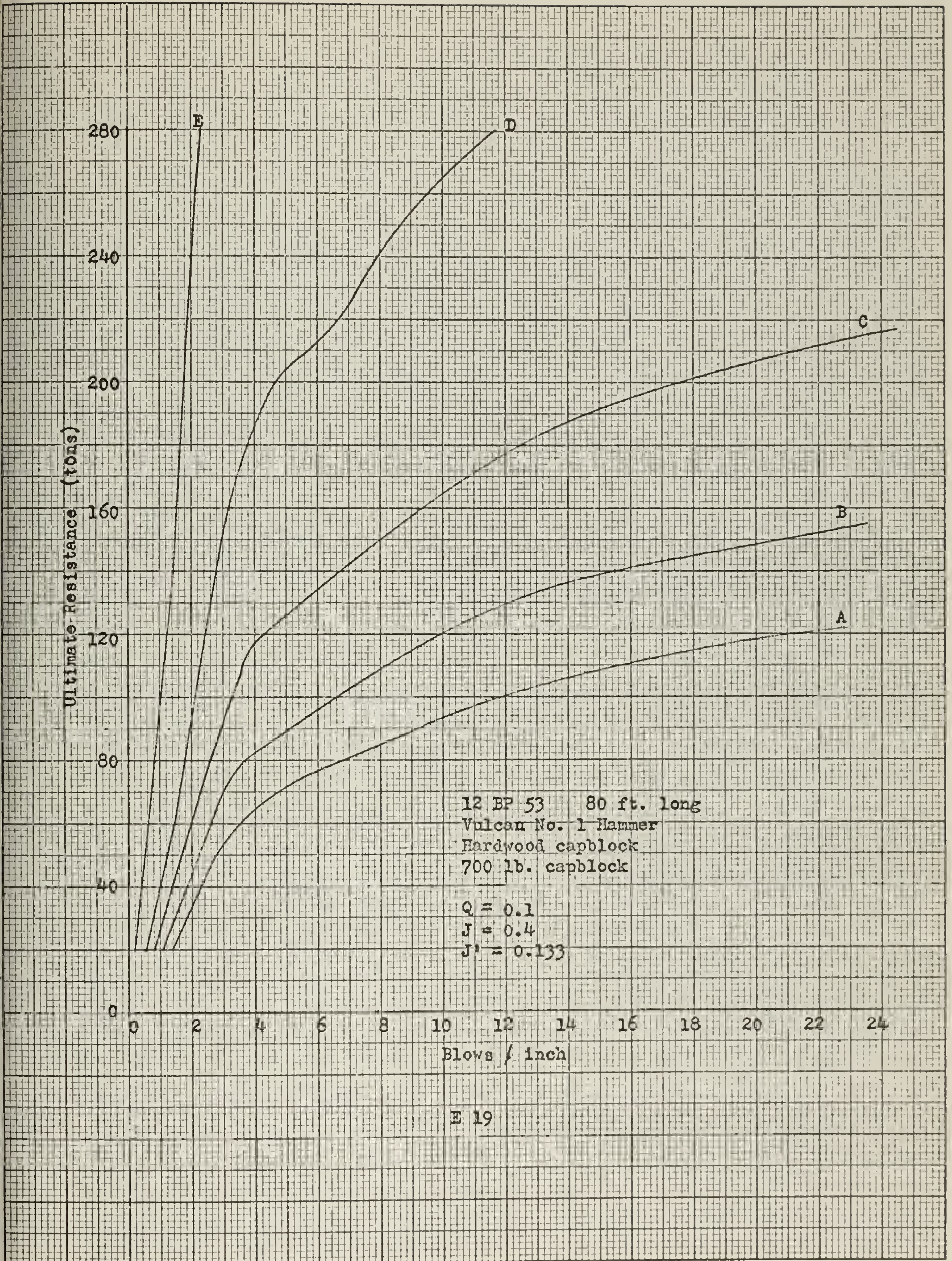
E 15


E
D
C
B
A
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.1
J' = 0.033

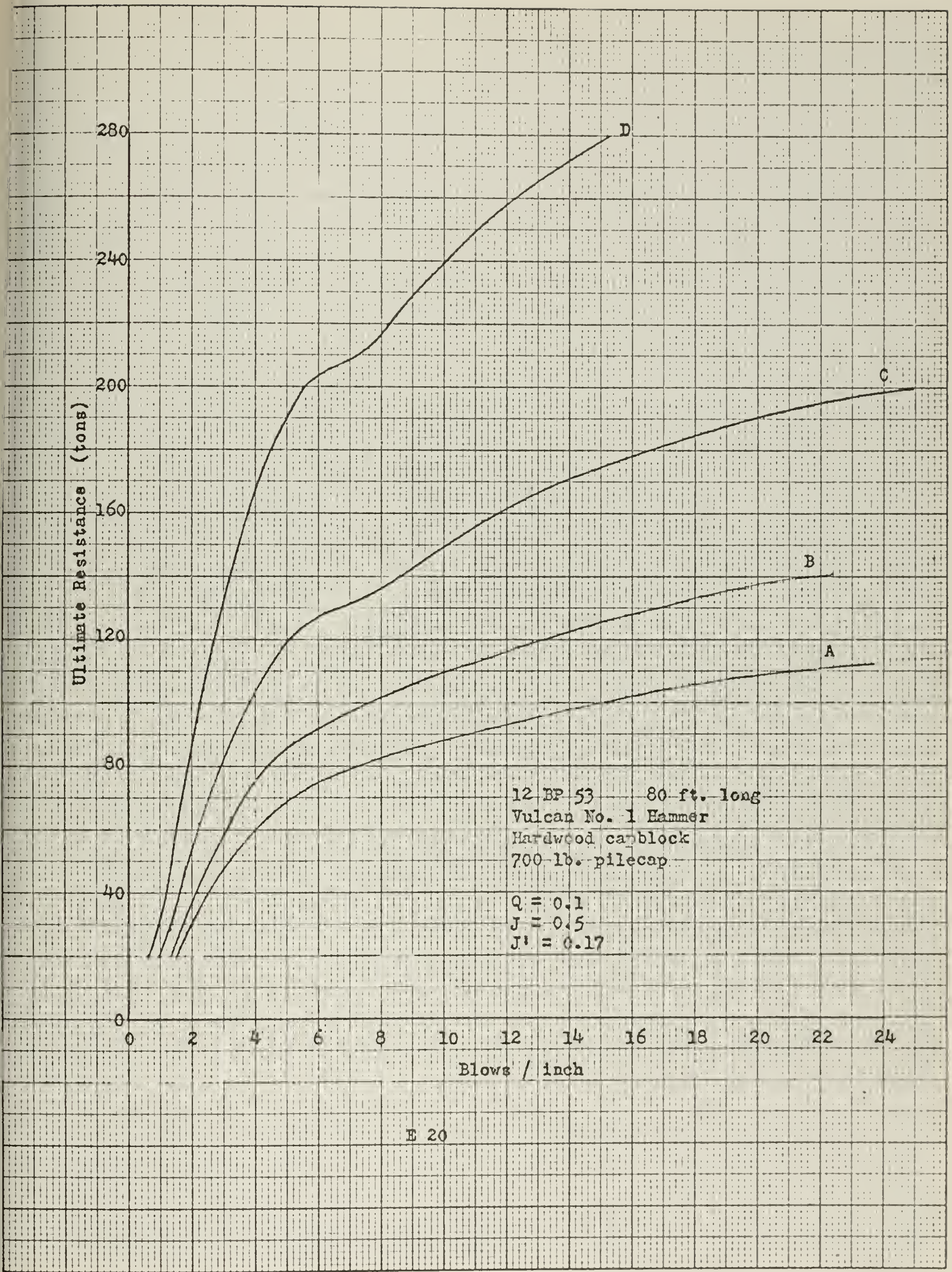

Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
E
D
C
B
A
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.2
J' = 0.067

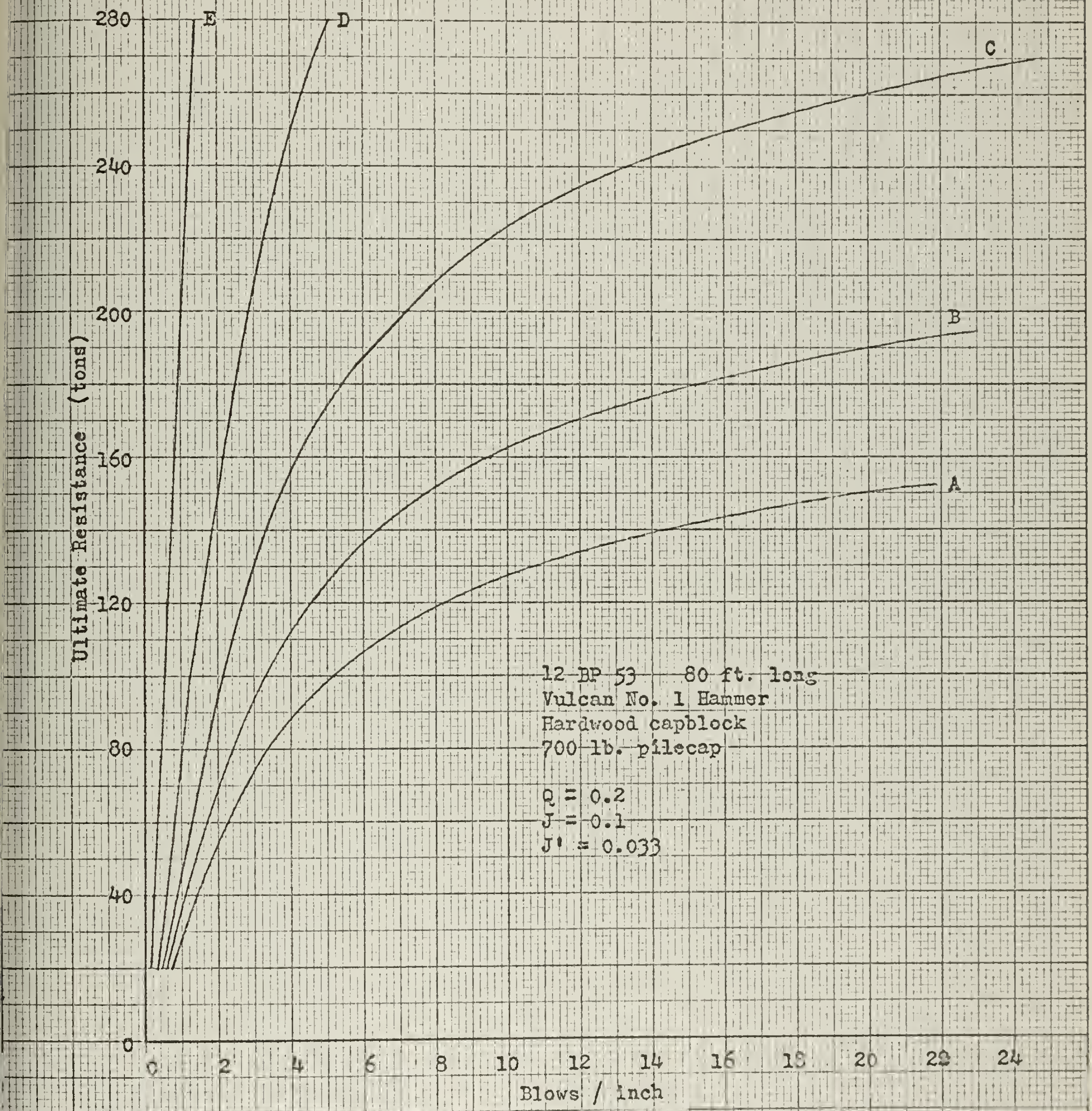
E 17


E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows /inch
12 BP 53 80 ft. long
Vulcan No.1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.3
J' = 0.1

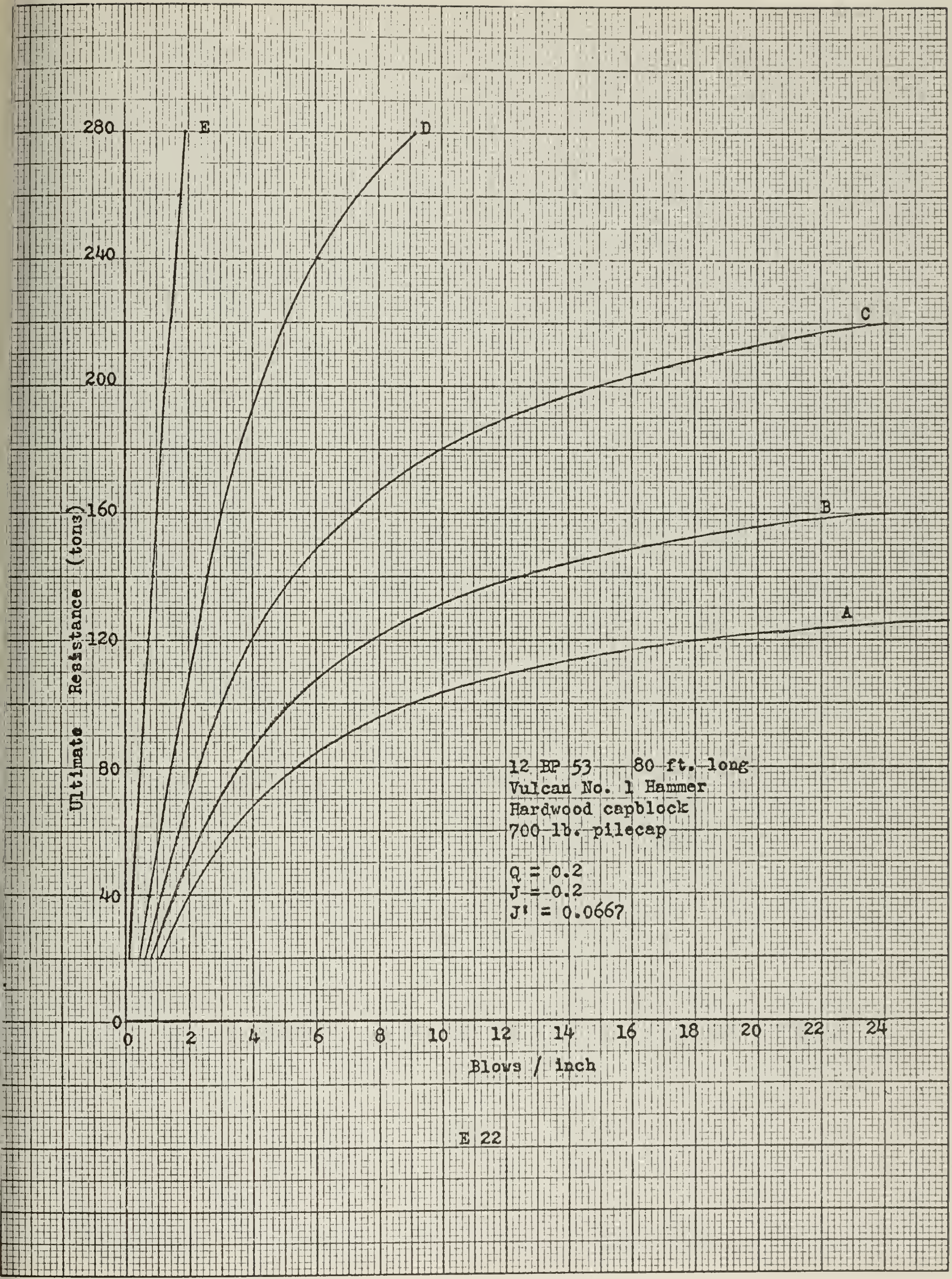

280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
E
D
C
B
A
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. capblock
Q = 0.1
J = 0.4
J' = 0.133

E 19


D
C
B
A
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.5
J' = 0.17

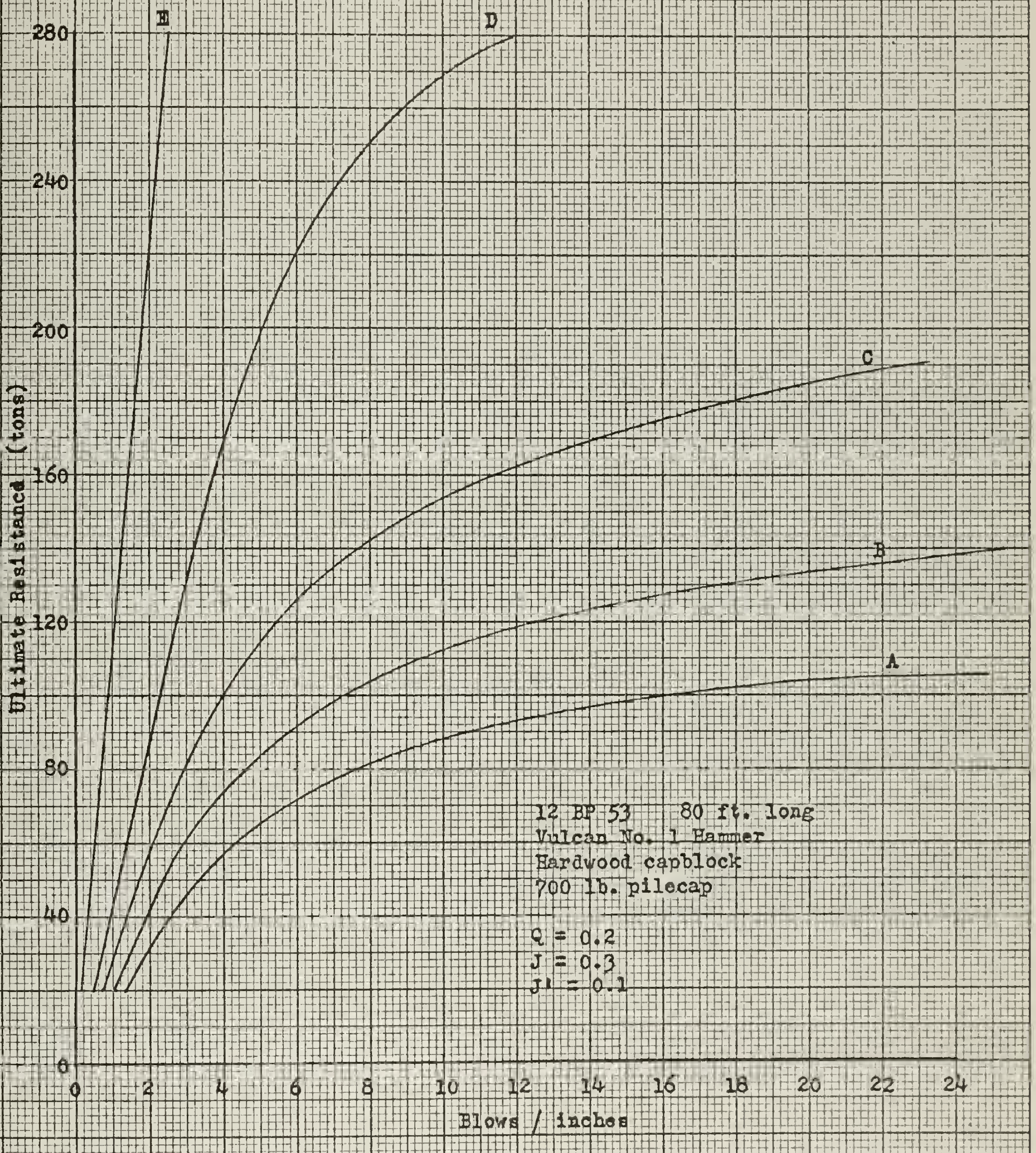


Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
E
D
C
B
A
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.1
J' = 0.033
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch

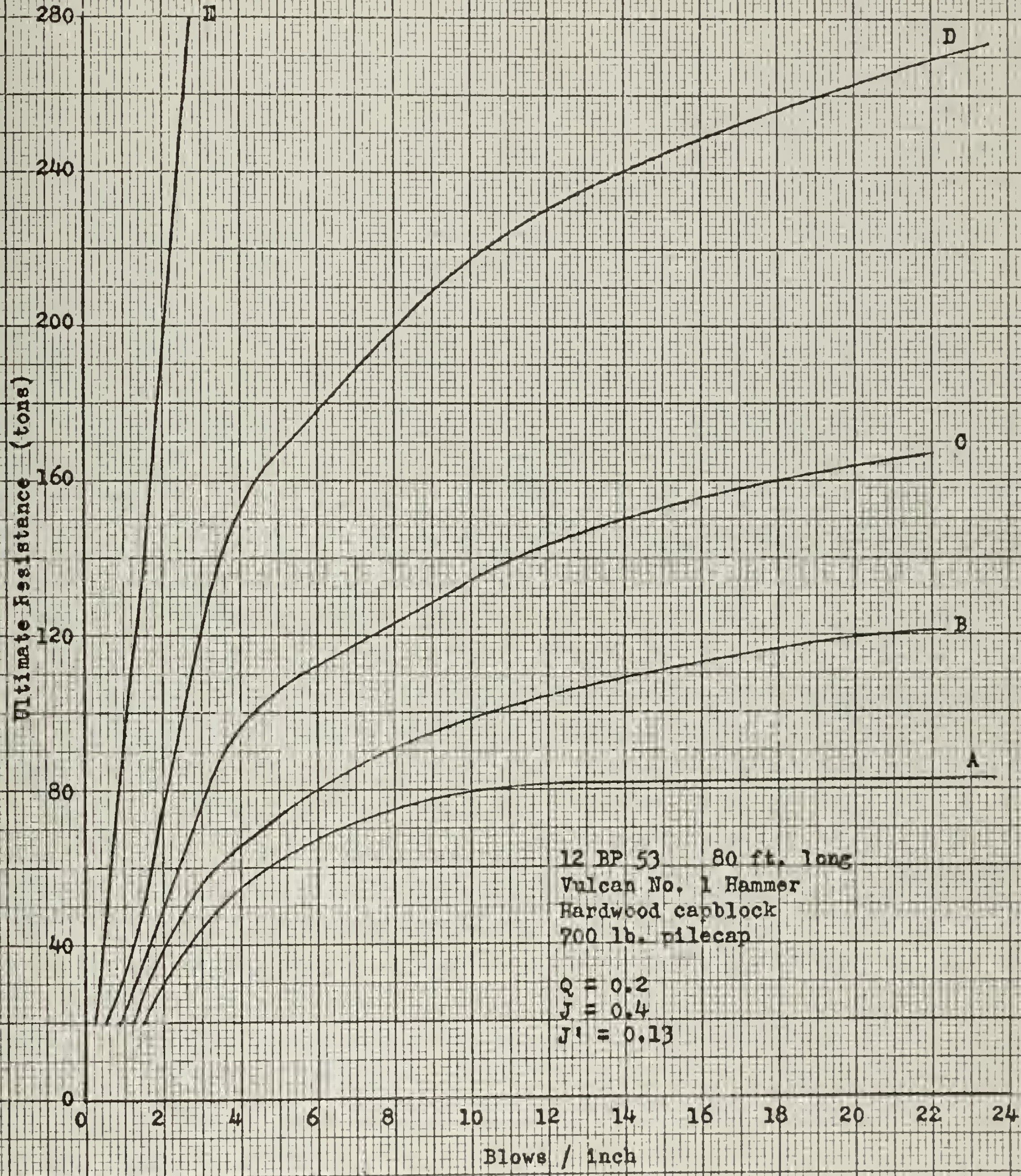


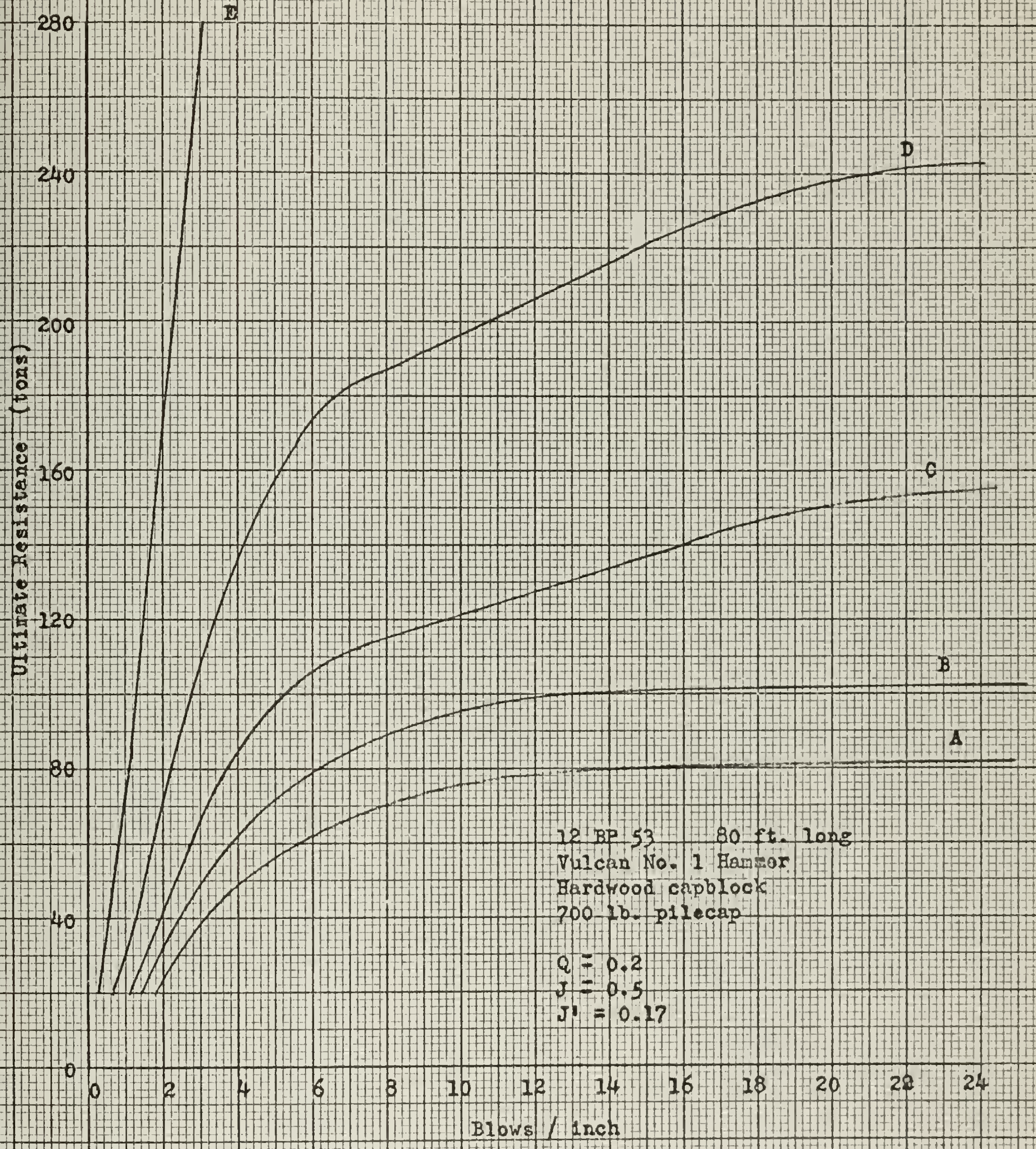
E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.2
J' = 0.0667

E 22

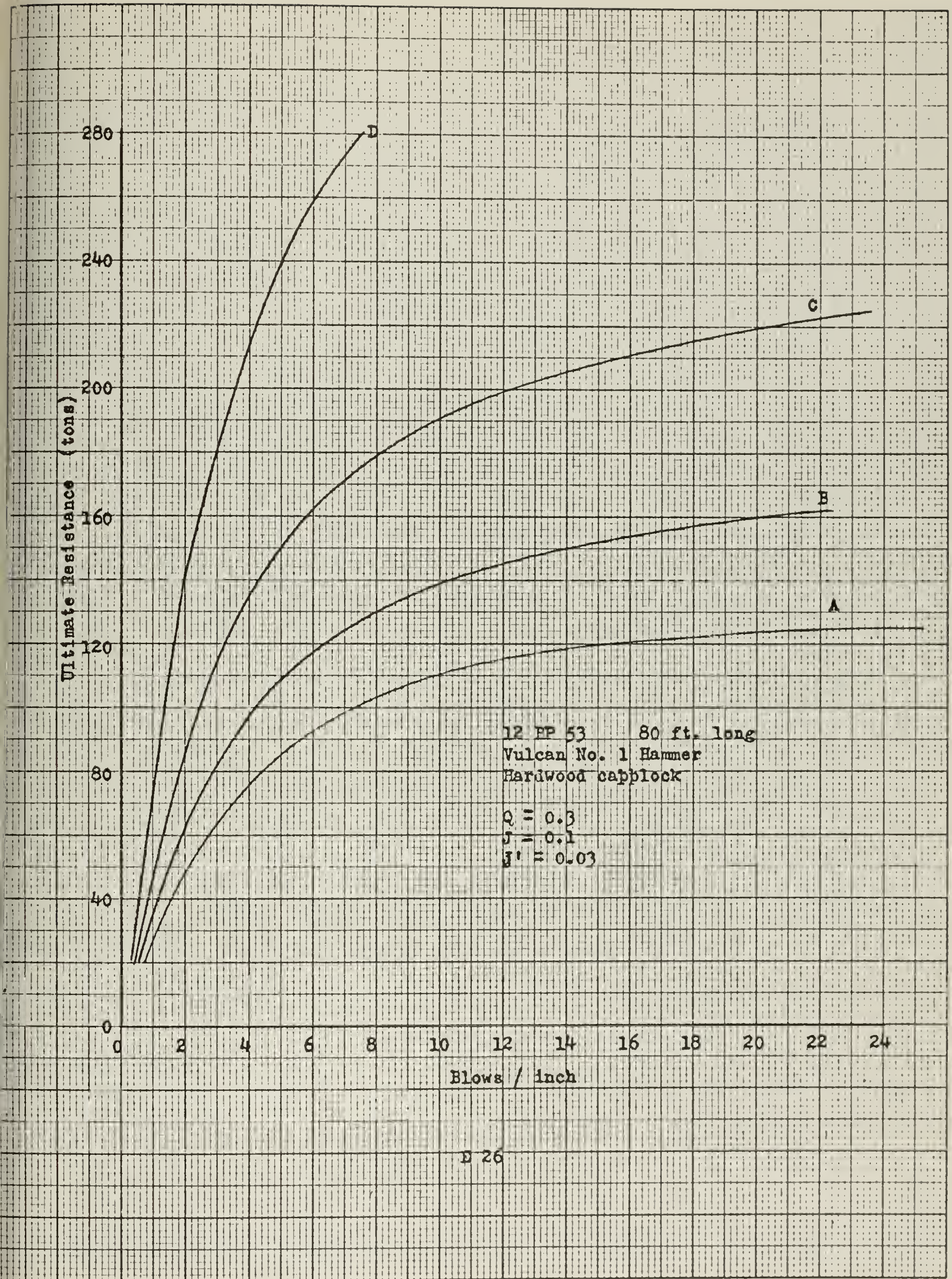

E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inches
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.3
J' = 0.1

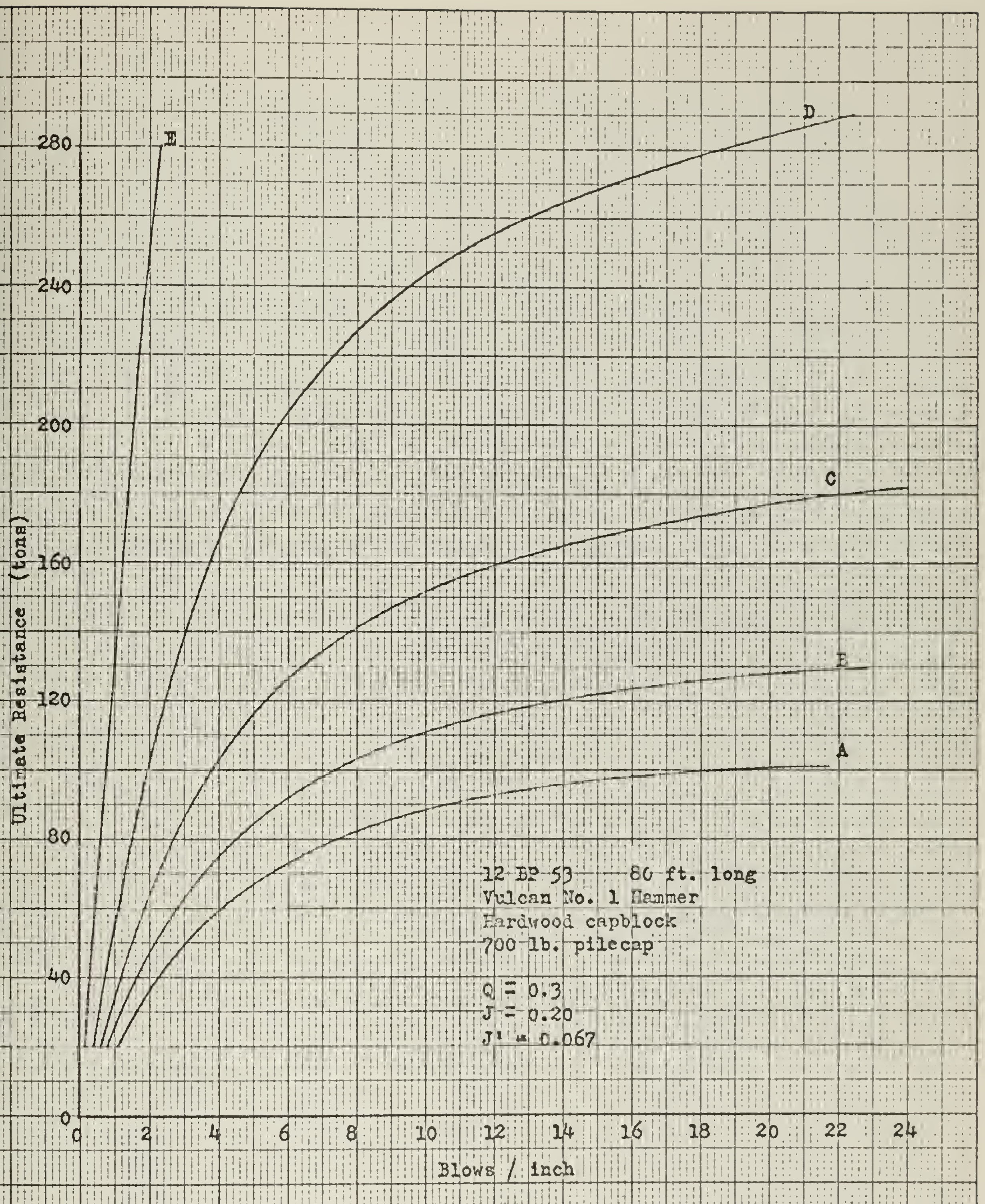
E 23

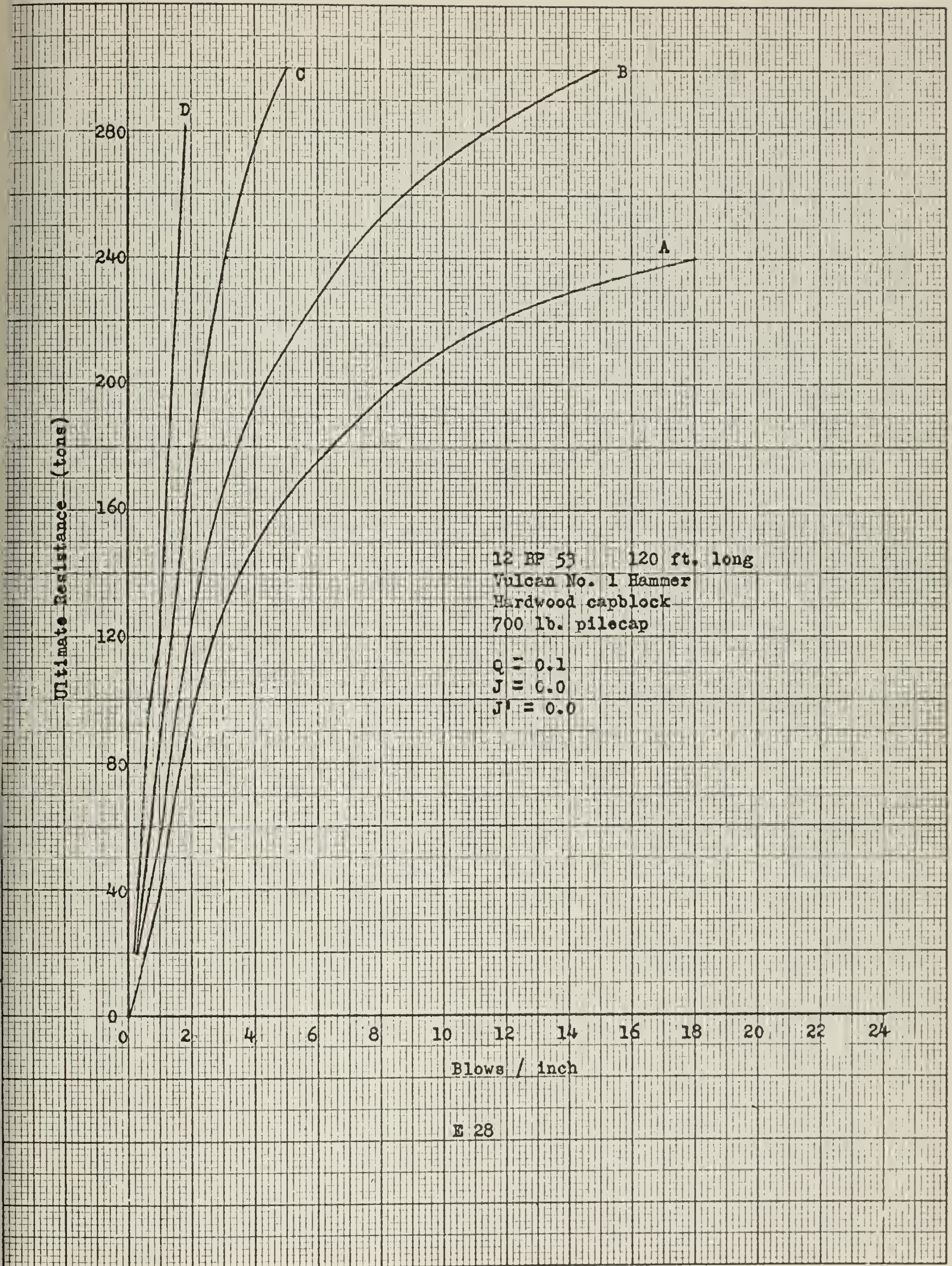

Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
E
D
C
B
A
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.4
J' = 0.13
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch

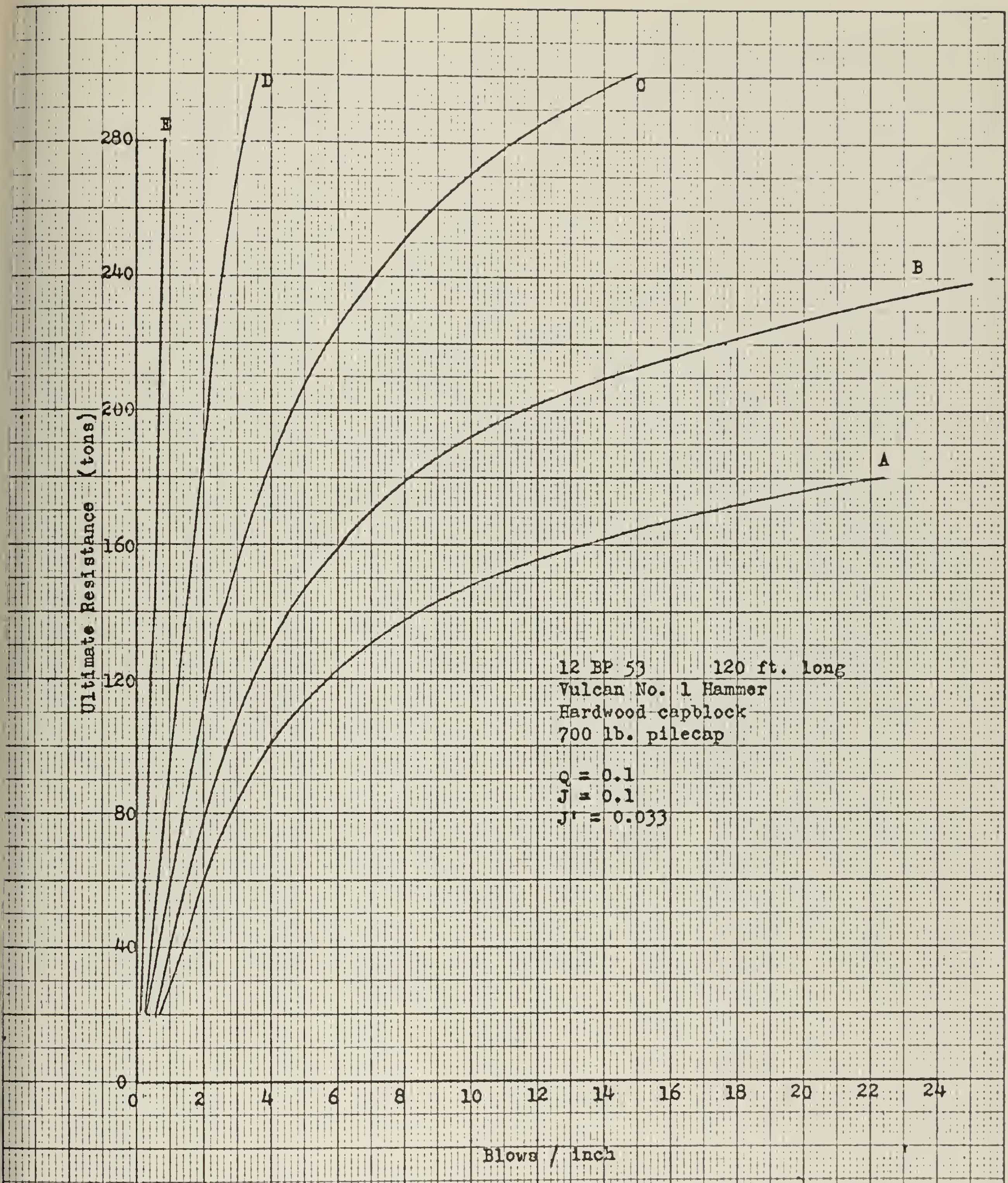

E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.5
J' = 0.17

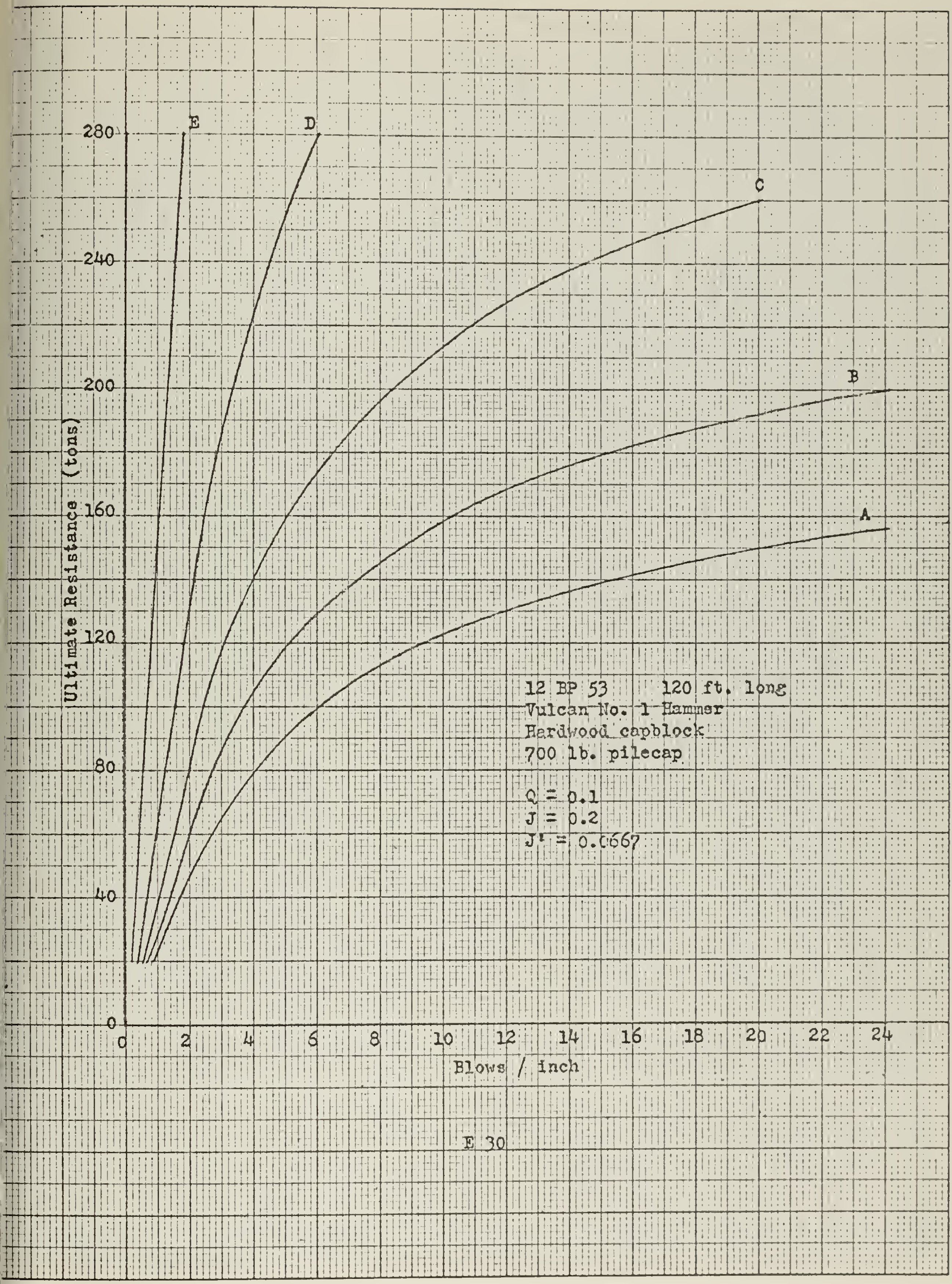
E 25


D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
Q = 0.3
J = 0.1
J' = 0.03

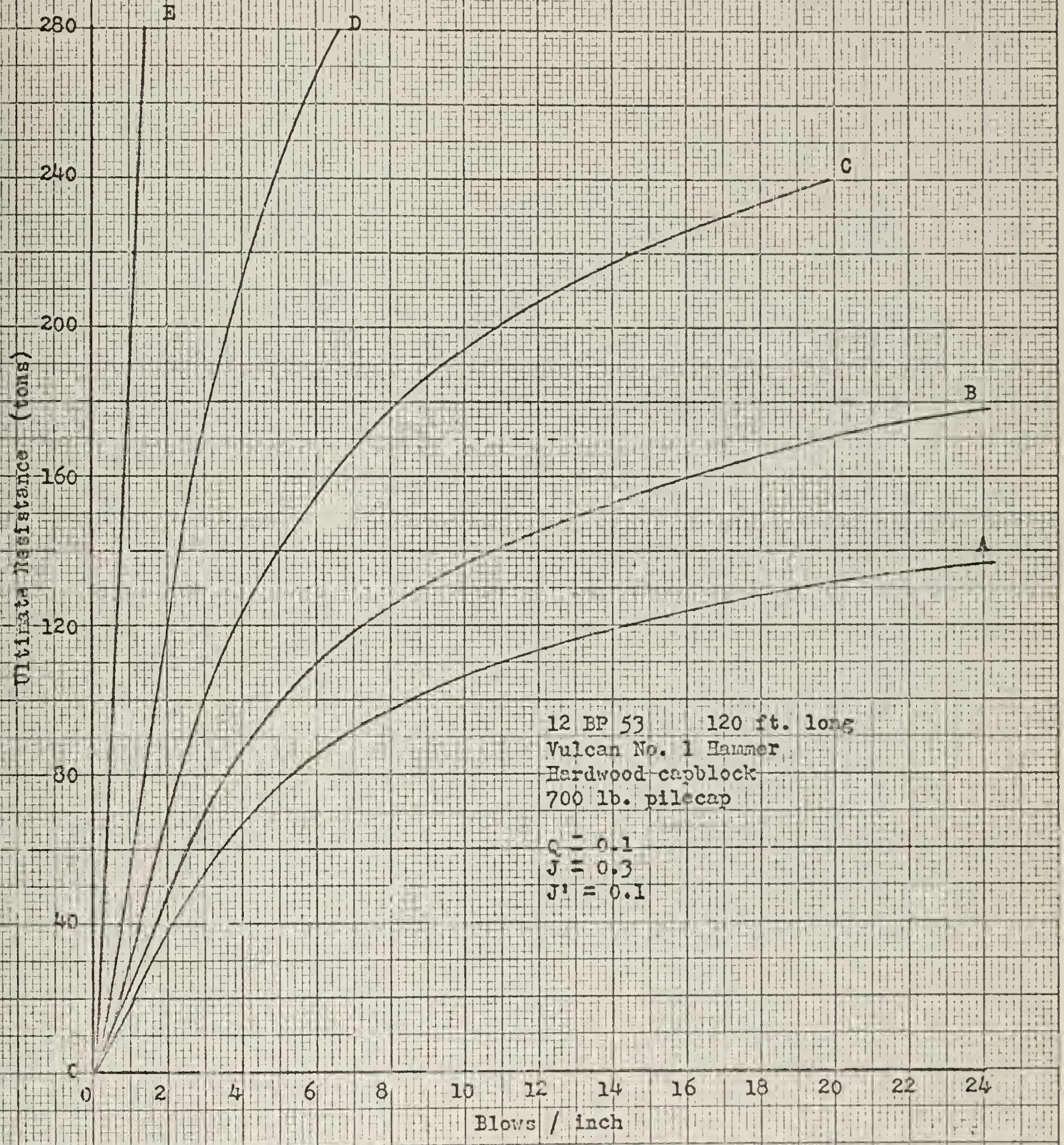

E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.3
J = 0.20
J' = 0.067


Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
D
C
B
A
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.0
J' = 0.0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch

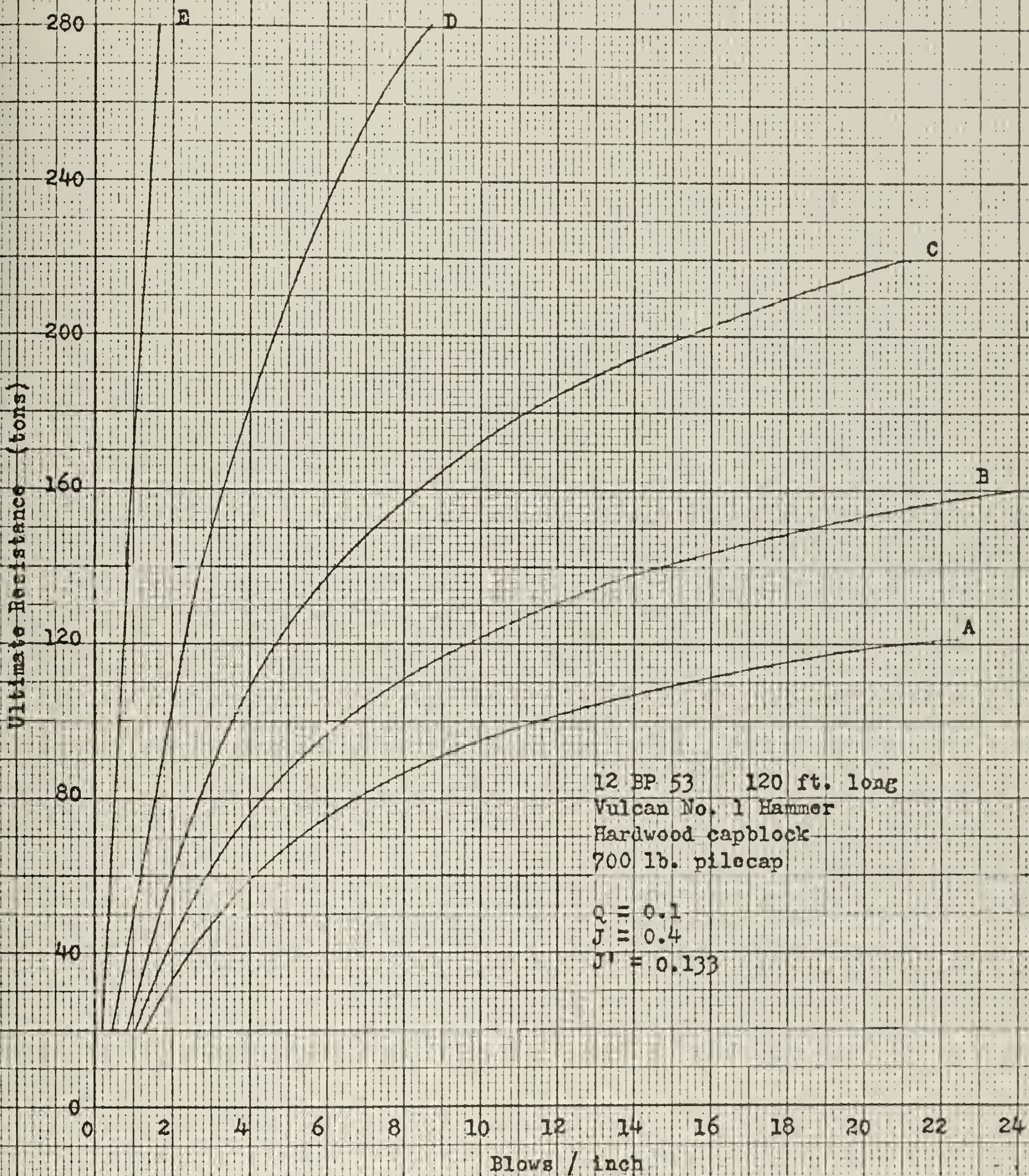

Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
A
B
C
D
E
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.1
J' = 0.033

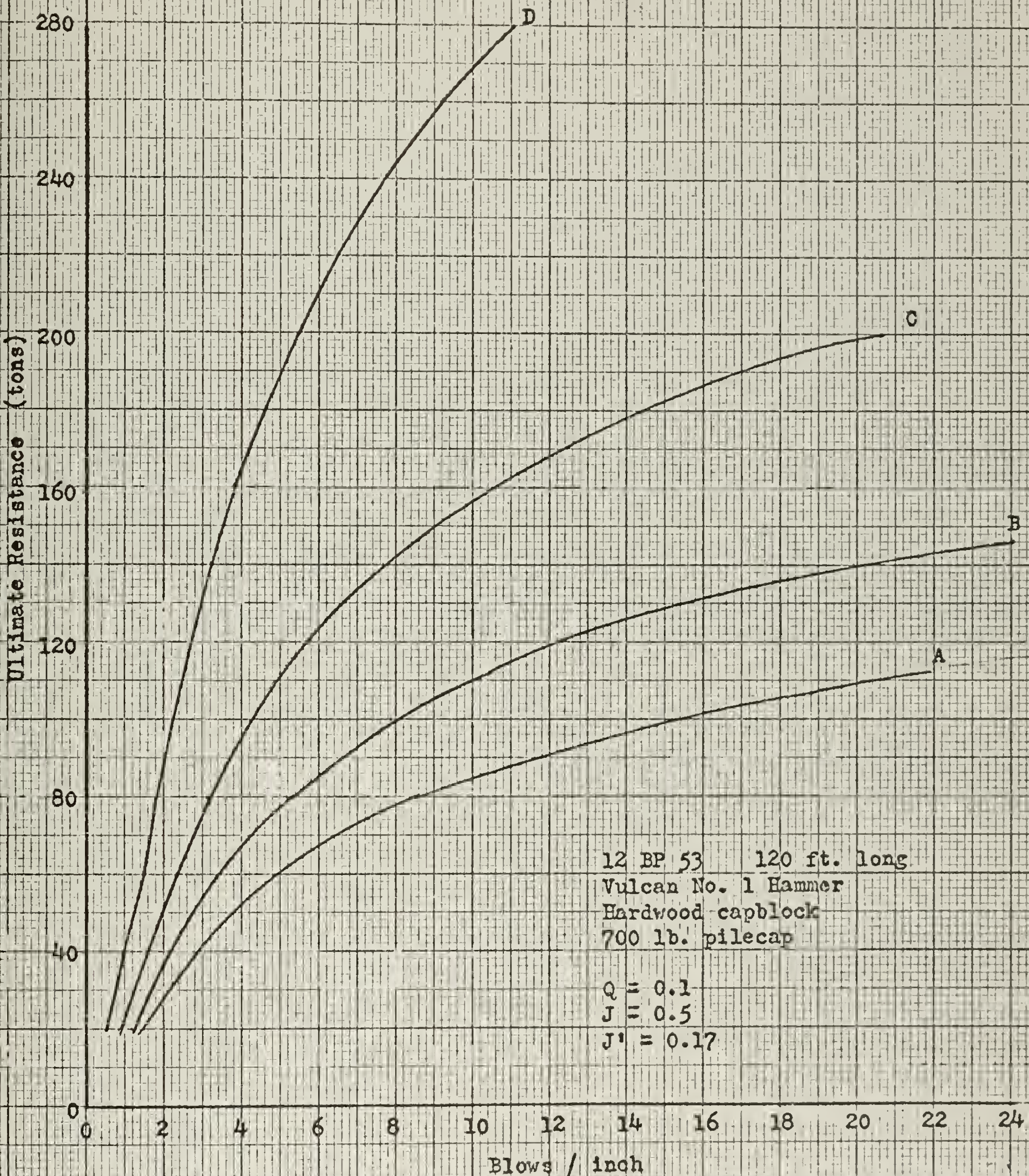

E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.2
J' = 0.0667

E 30

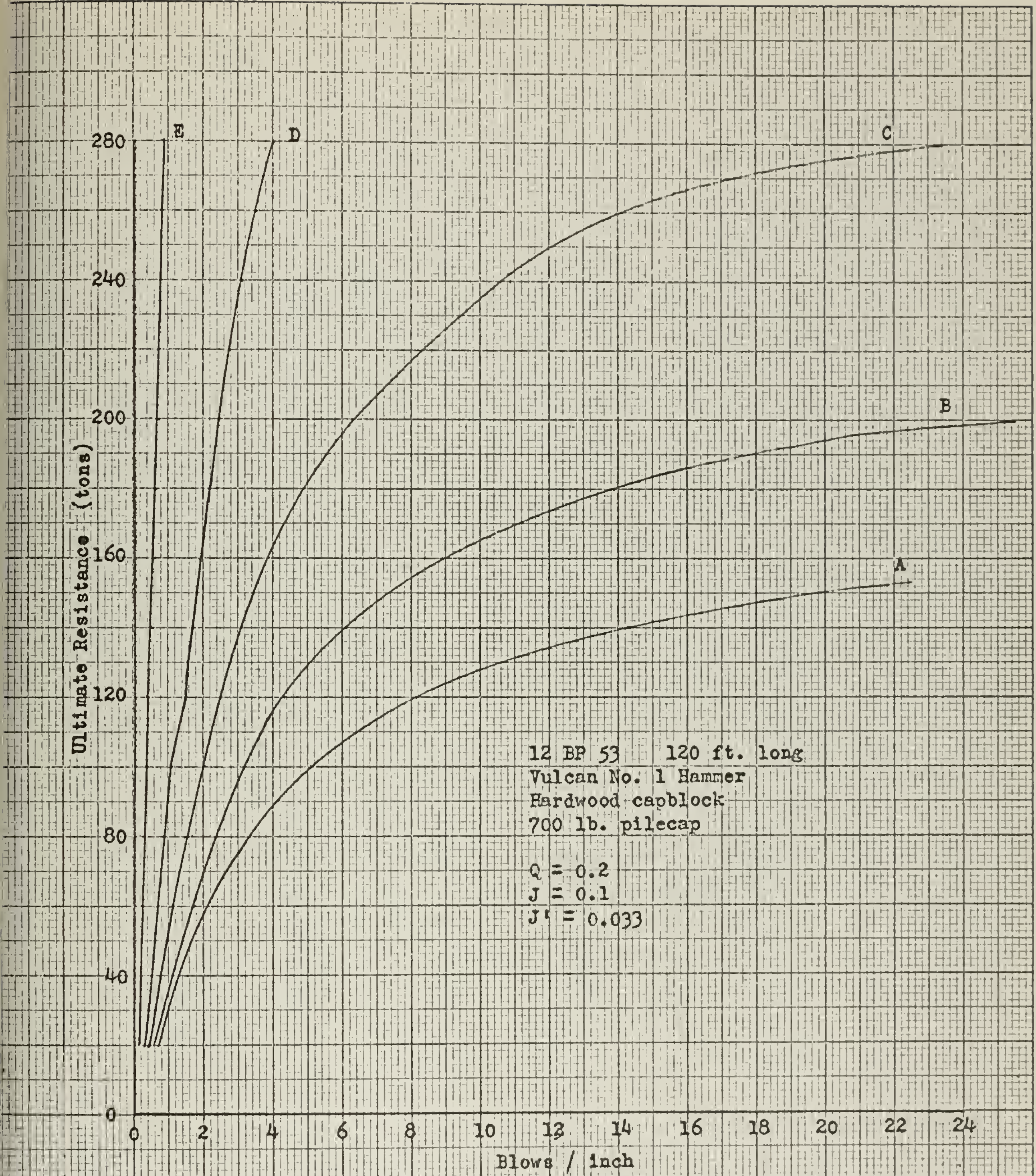


Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
E
D
C
B
A
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.3
J' = 0.1


Ultimate Resistance (tons)
Blows / inch
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.4
J' = 0.133

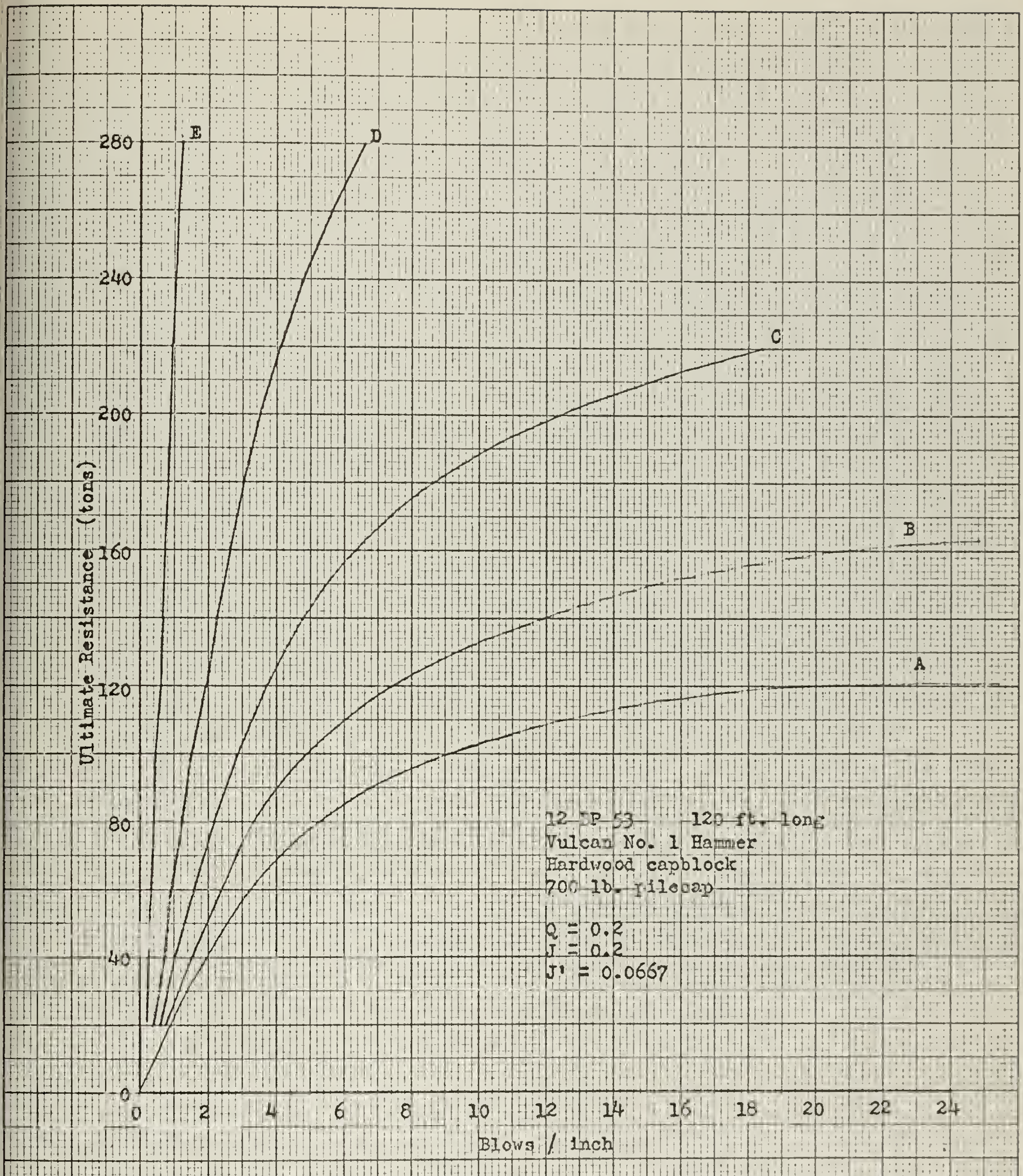


Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
D
C
B
A
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.5
J' = 0.17



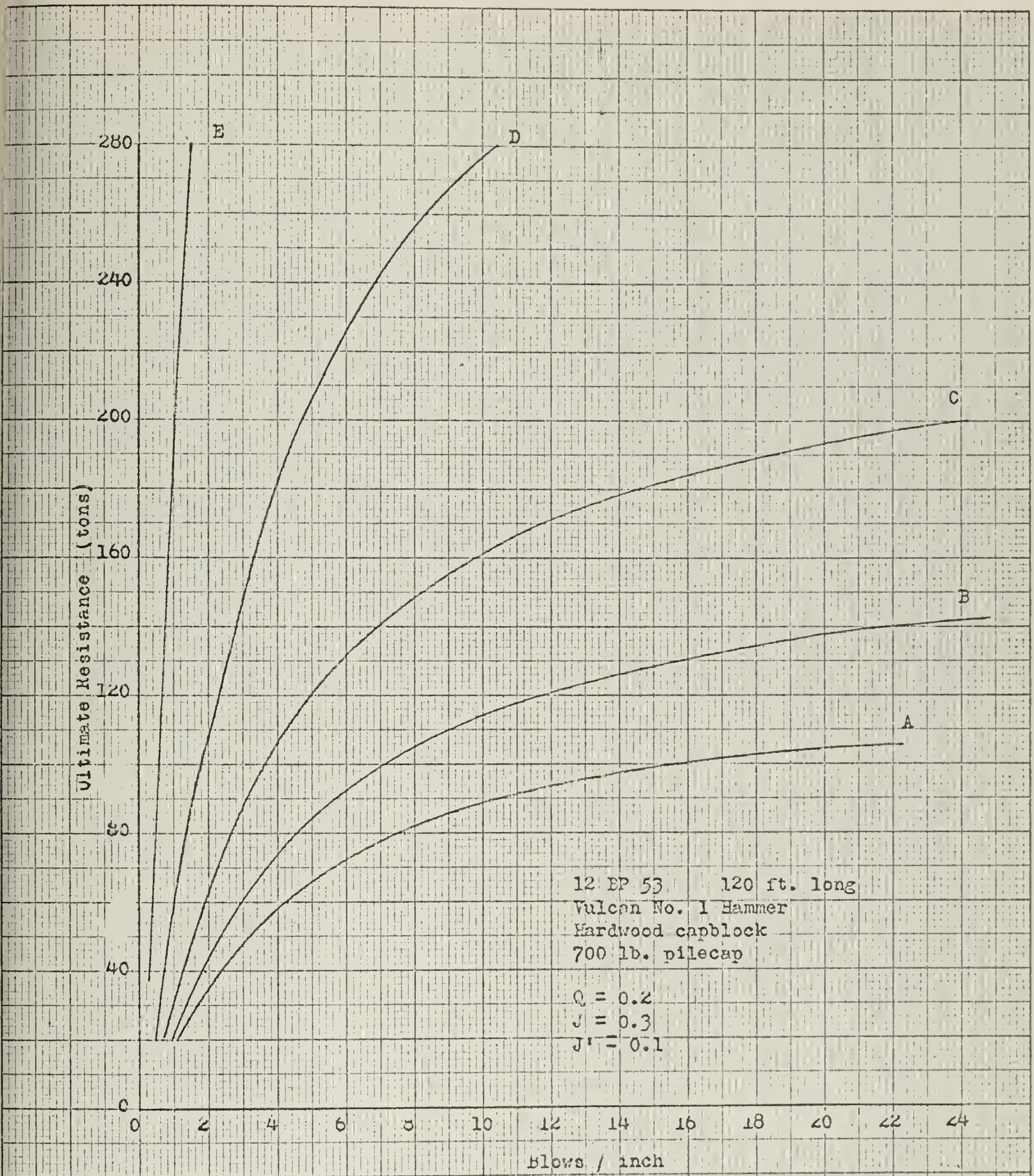
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
Blows / inch
0
2
4
6
8
10
12
14
16
18
20
22
24
A
B
C
D
E
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.1
J' = 0.033

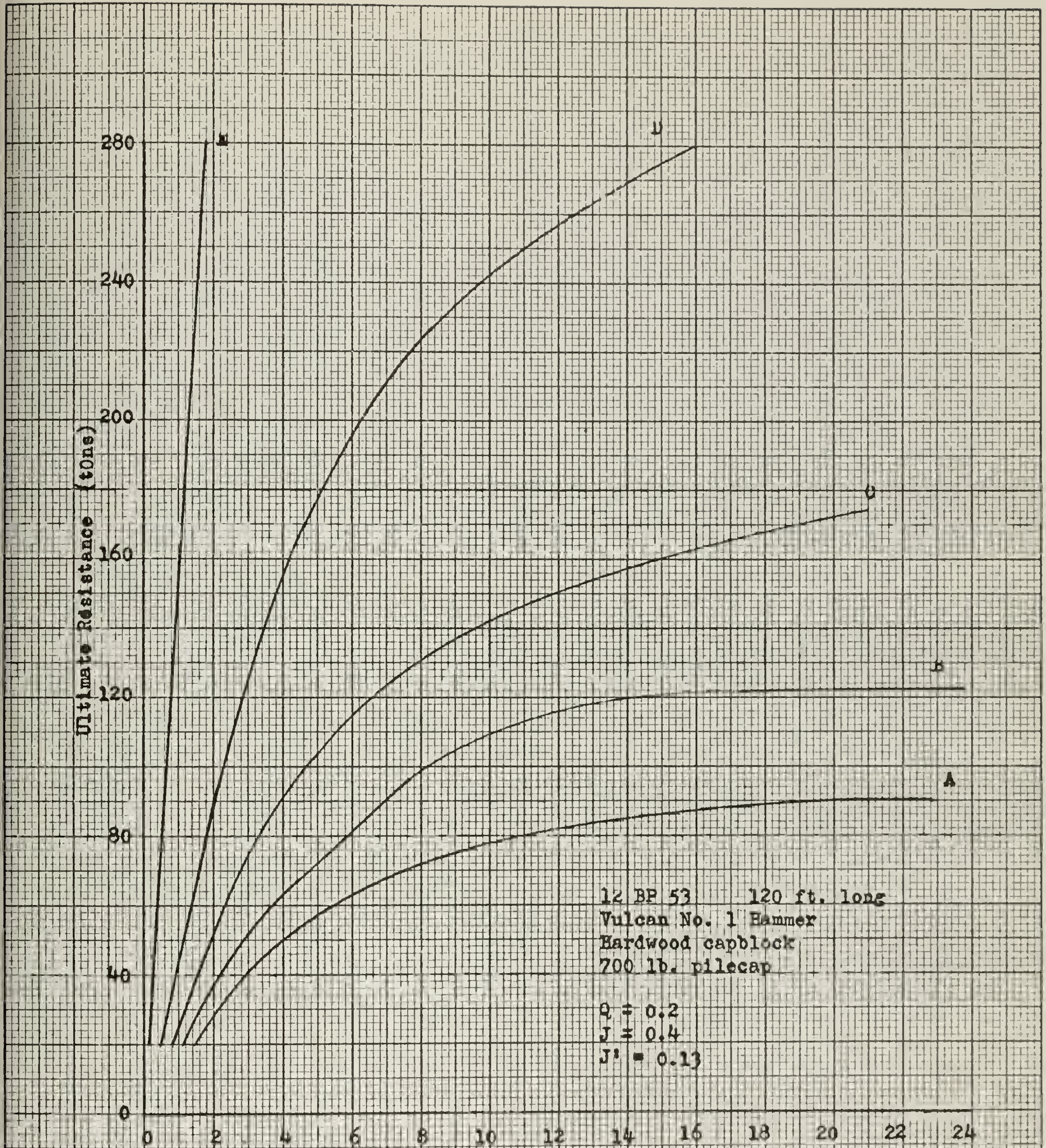
E 34



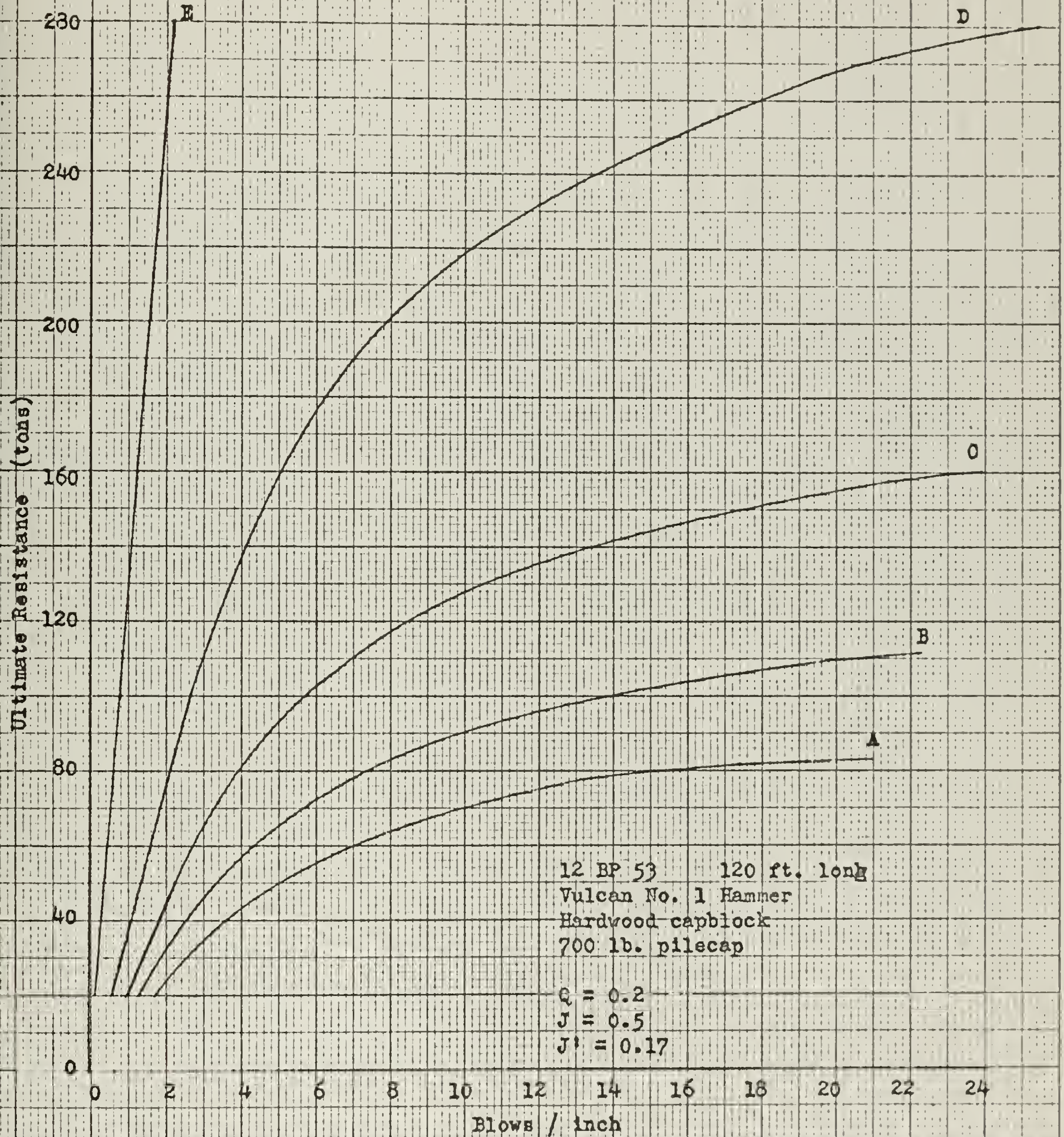
E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.2
J' = 0.0667

E 35

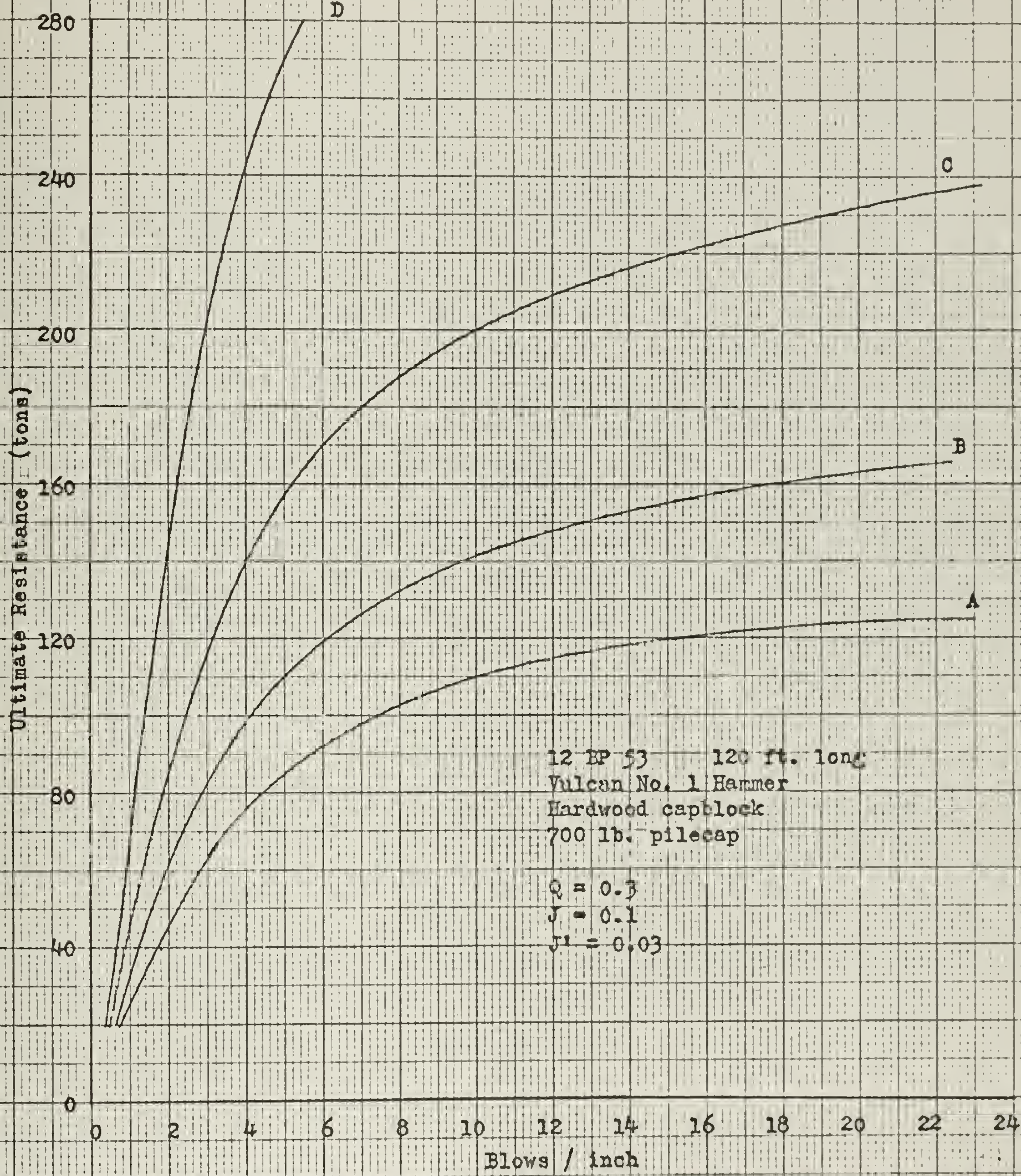

Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0 2 4 6 8 10 12 14 16 18 20 22 24
Blows / inch
E
D
C
B
A
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.3
J' = 0.1



Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0 2 4 6 8 10 12 14 16 18 20 22 24
Blows / inch
A
B
C
D
E
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.4
J' = 0.13

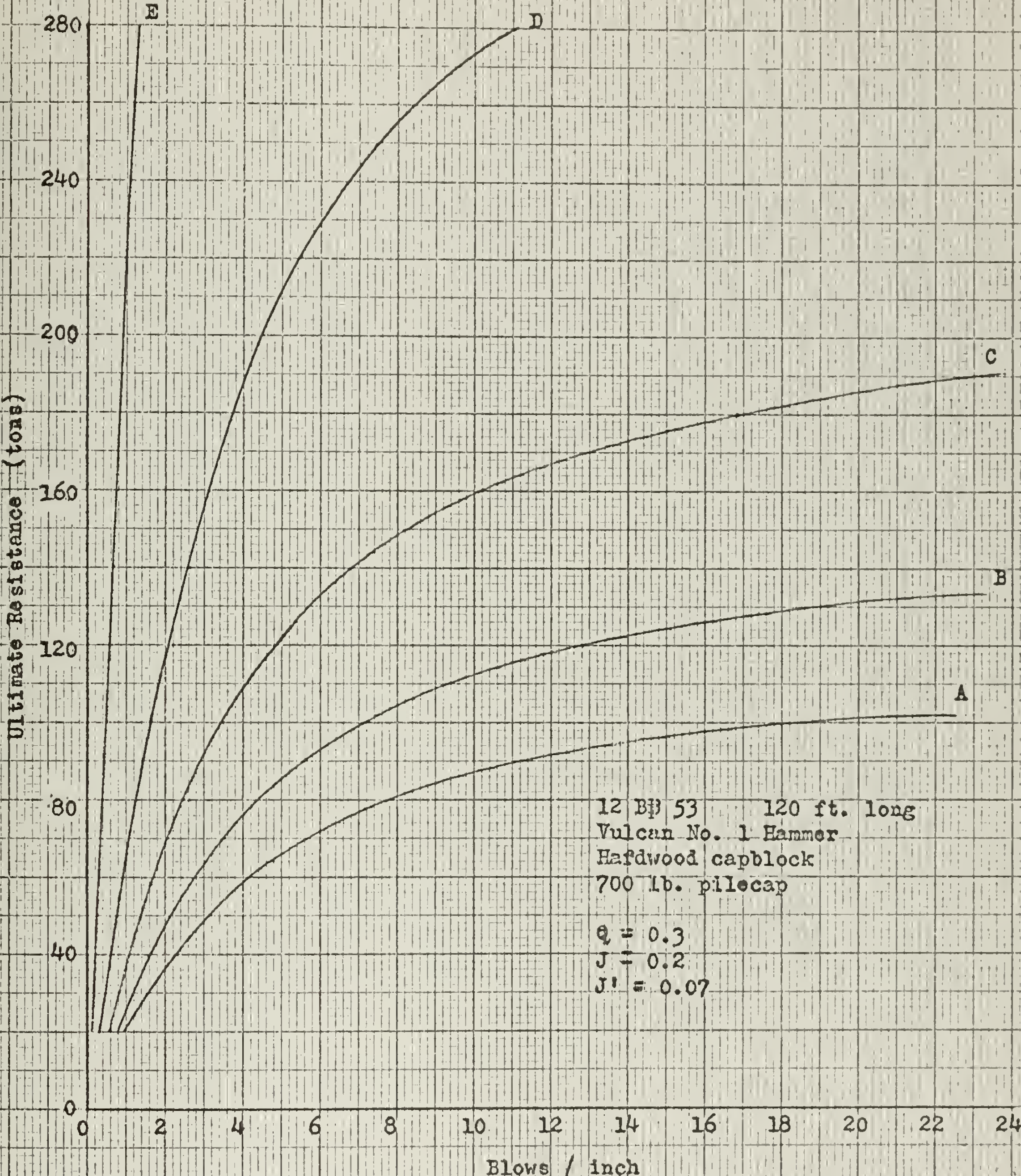


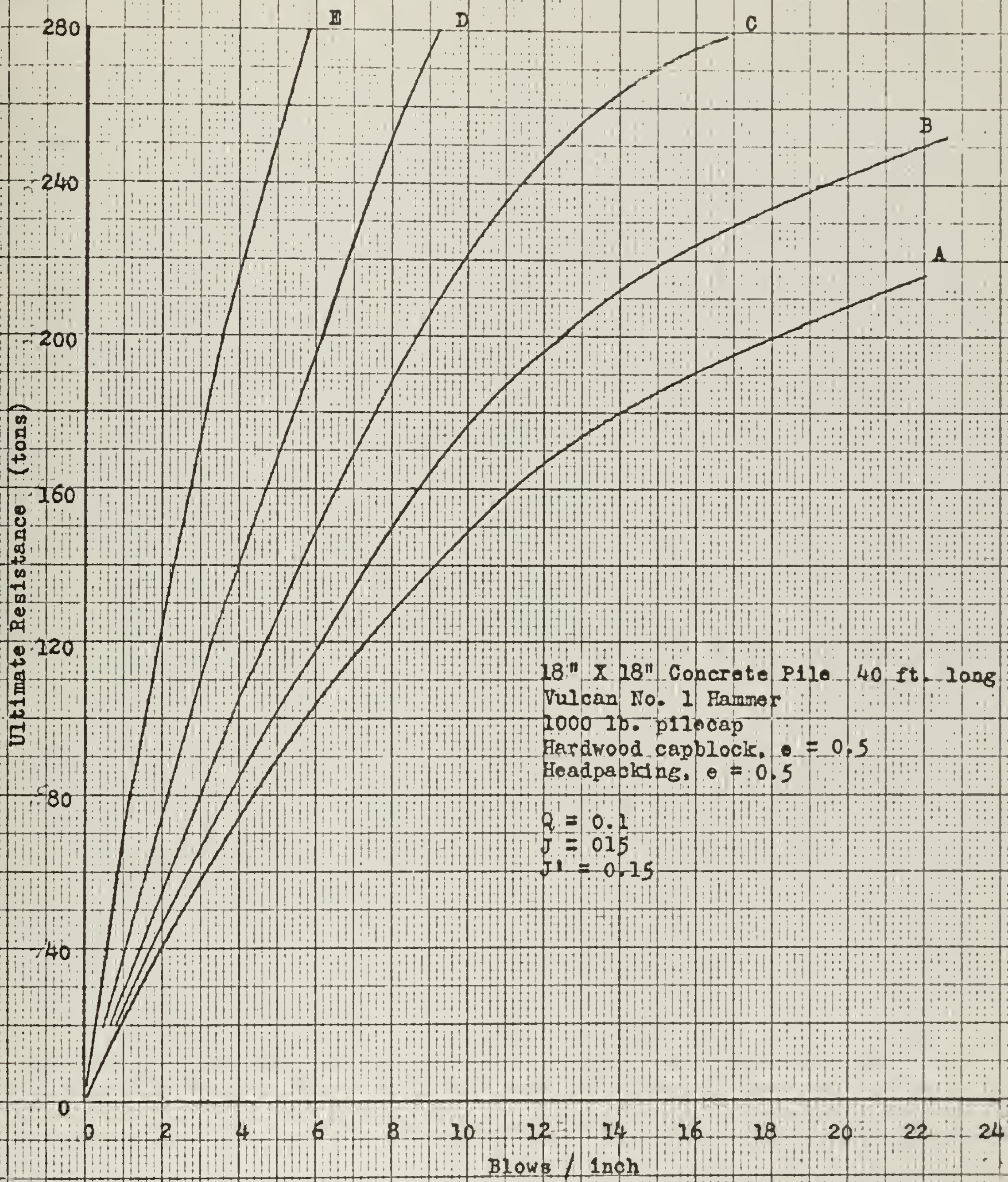
Ultimate Resistance (tons)
0
40
80
120
160
200
240
280
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
A
B
C
D
E
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.2
J = 0.5
J' = 0.17

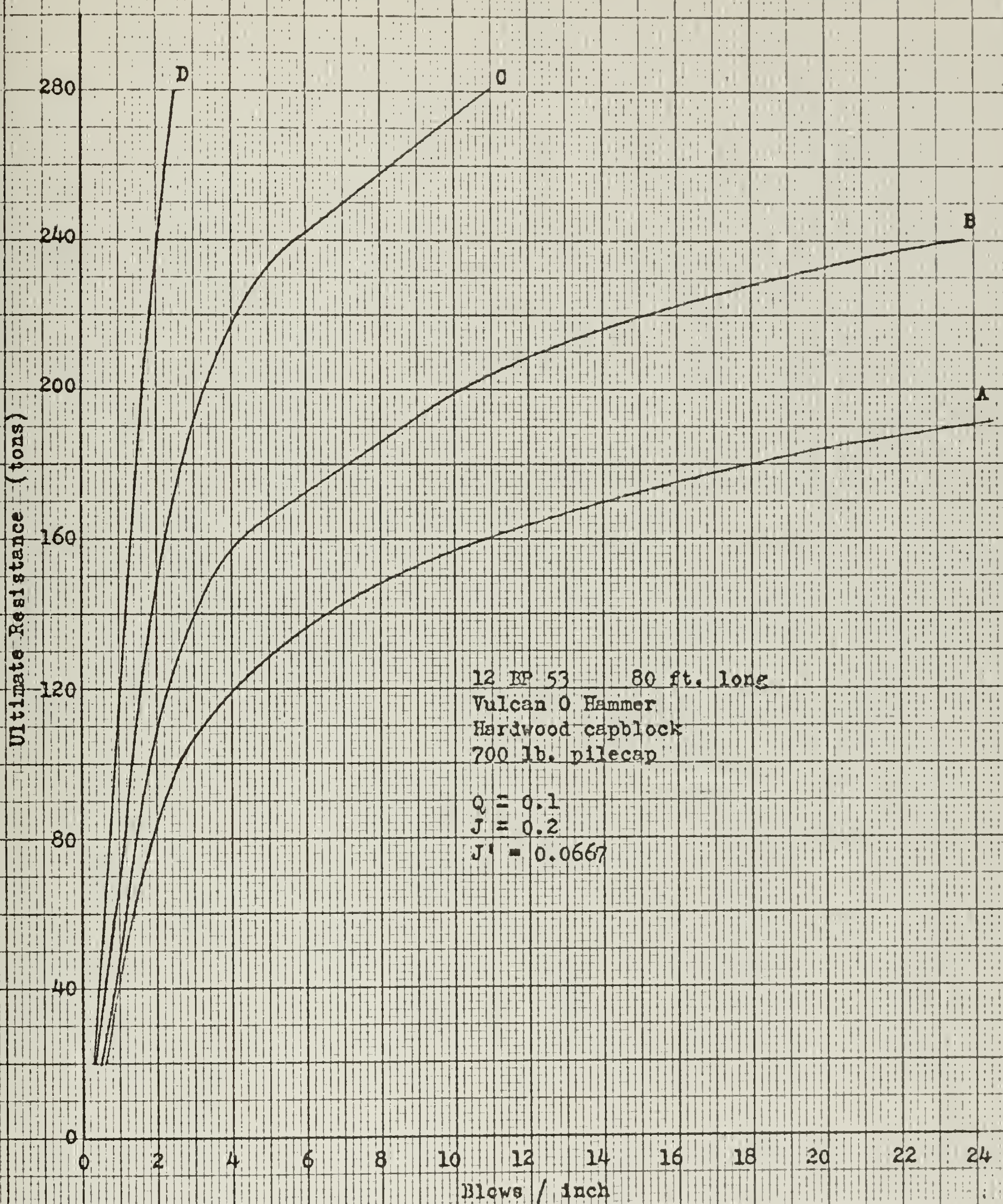


Ultimate Resistance (tons)
280
240
200
160
120
80
40
0
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
D
C
B
A
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.3
J = 0.1
J' = 0.03

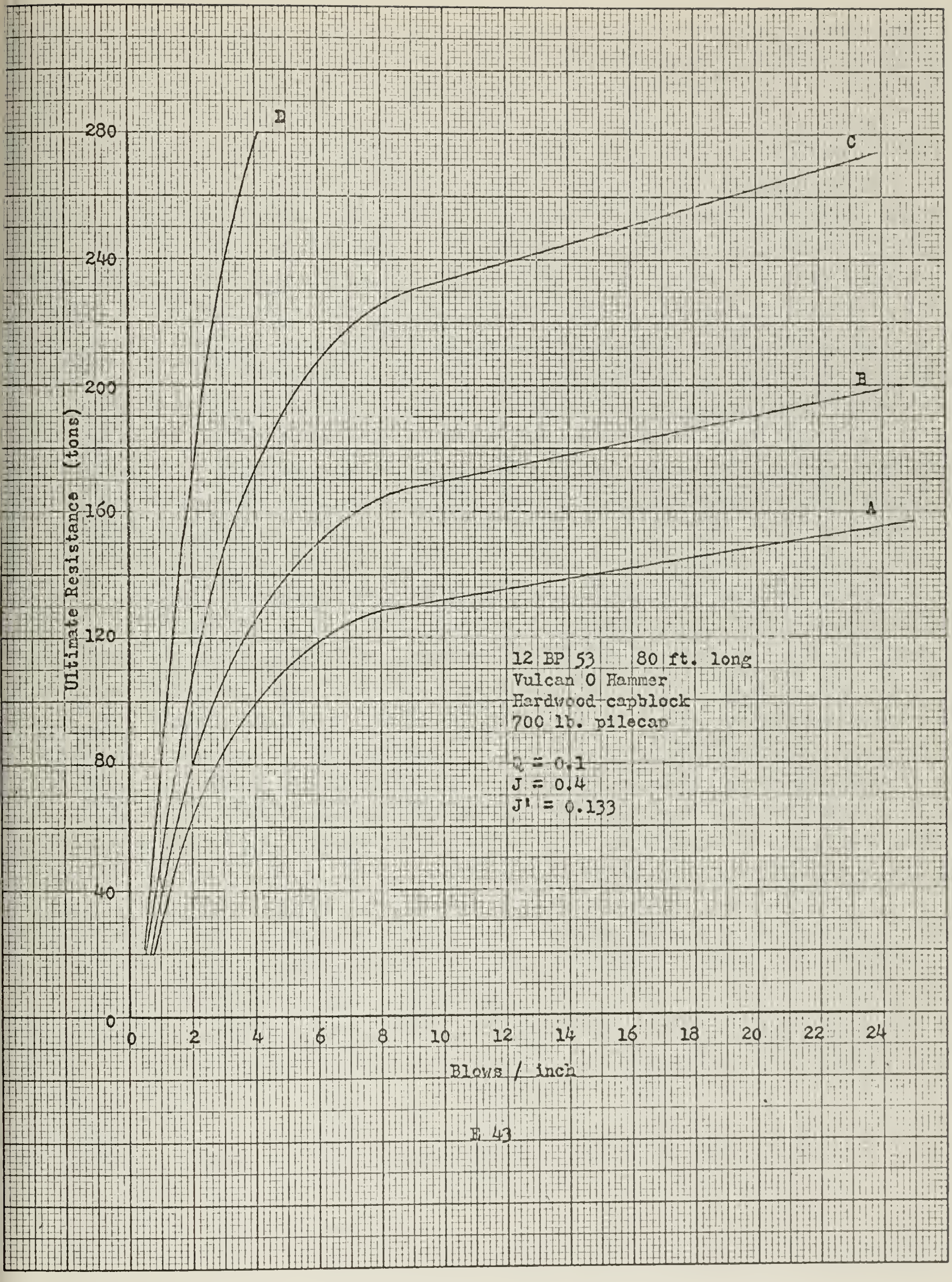
E 39


E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 120 ft. long
Vulcan No. 1 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.3
J = 0.2
J' = 0.07

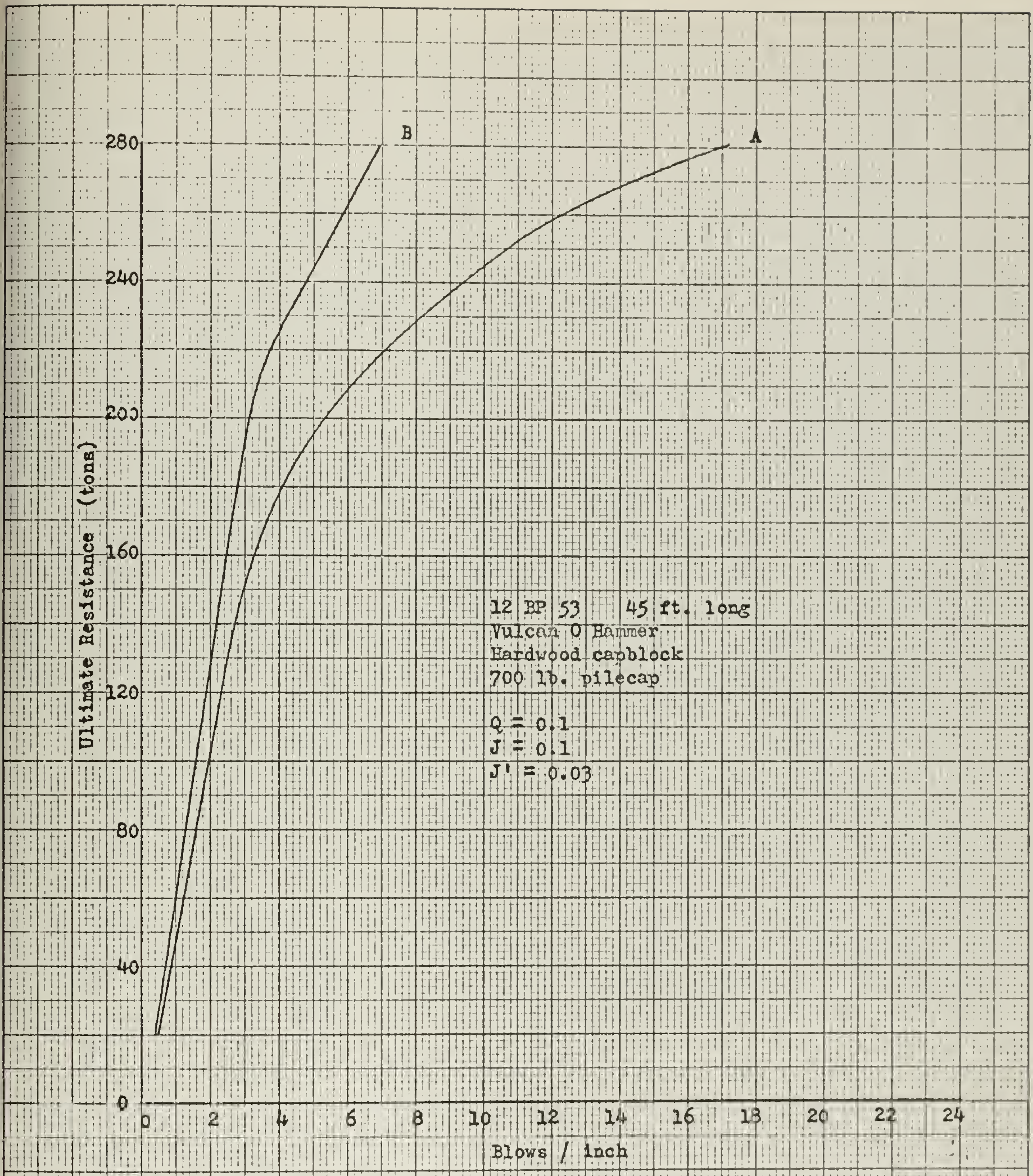

E
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
18" X 18" Concrete Pile 40 ft. long
Vulcan No. 1 Hammer
1000 lb. pilecap
Hardwood capblock, e = 0.5
Headpacking, e = 0.5
Q = 0.1
J = 015
J' = 0.15


D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.2
J' = 0.0667

E 42

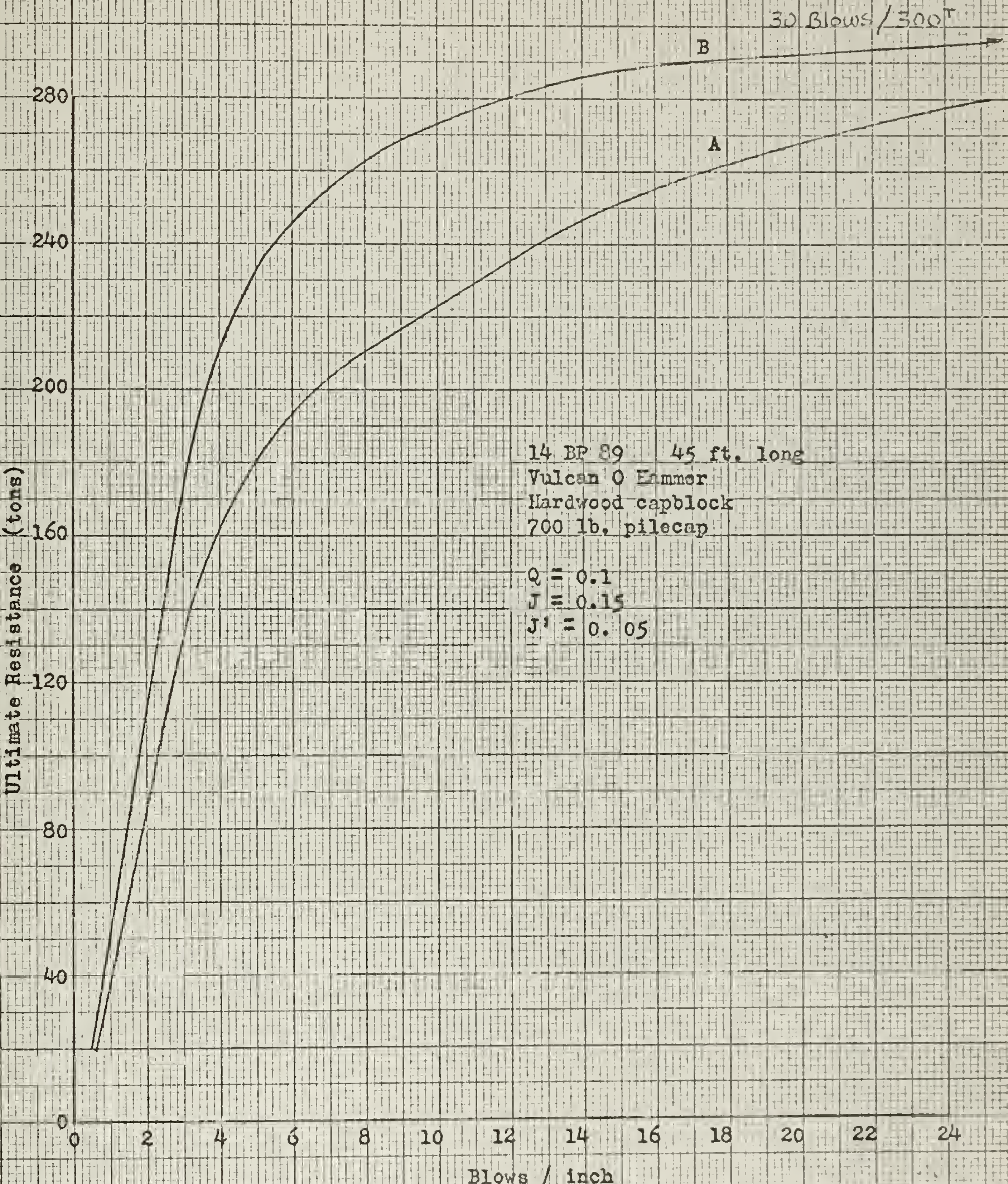


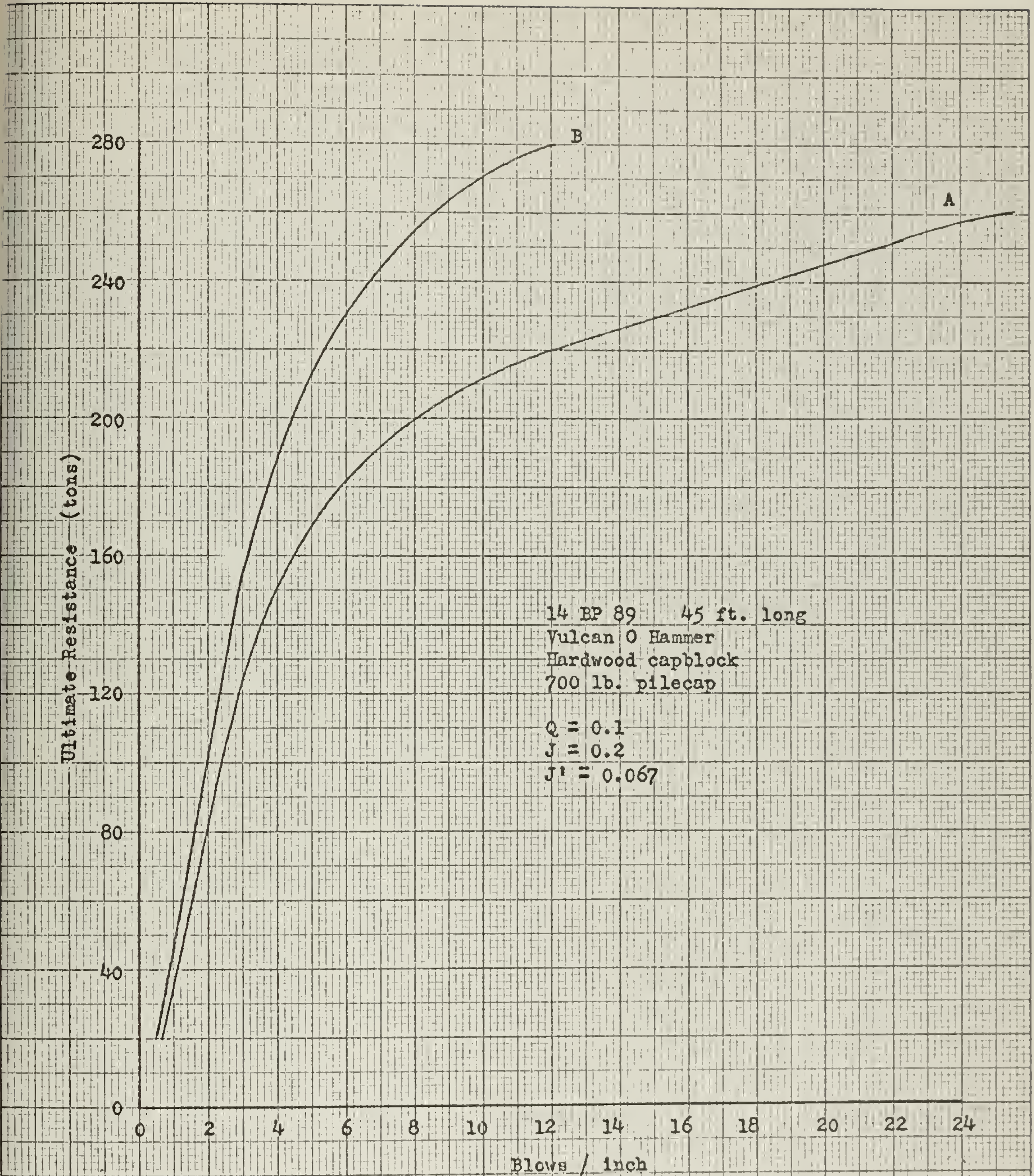
D
C
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 80 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.4
J' = 0.133



B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
12 BP 53 45 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.1
J' = 0.03

E 44


30 Blows/300T
B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
14 BP 89 45 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.15
J' = 0. 05


B
A
280
240
200
160
120
80
40
0
Ultimate Resistance (tons)
0
2
4
6
8
10
12
14
16
18
20
22
24
Blows / inch
14 BP 89 45 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.2
J' = 0.067

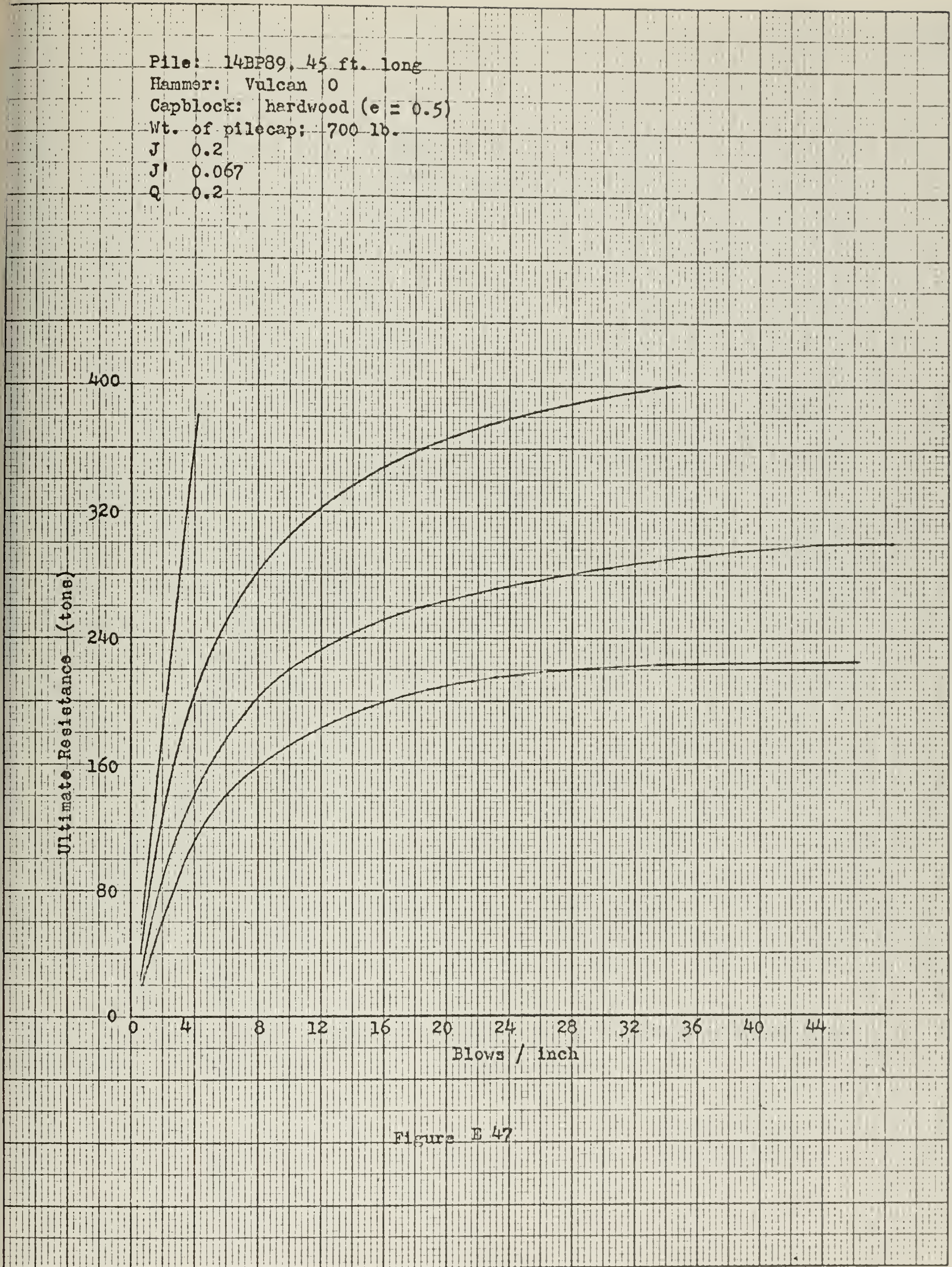
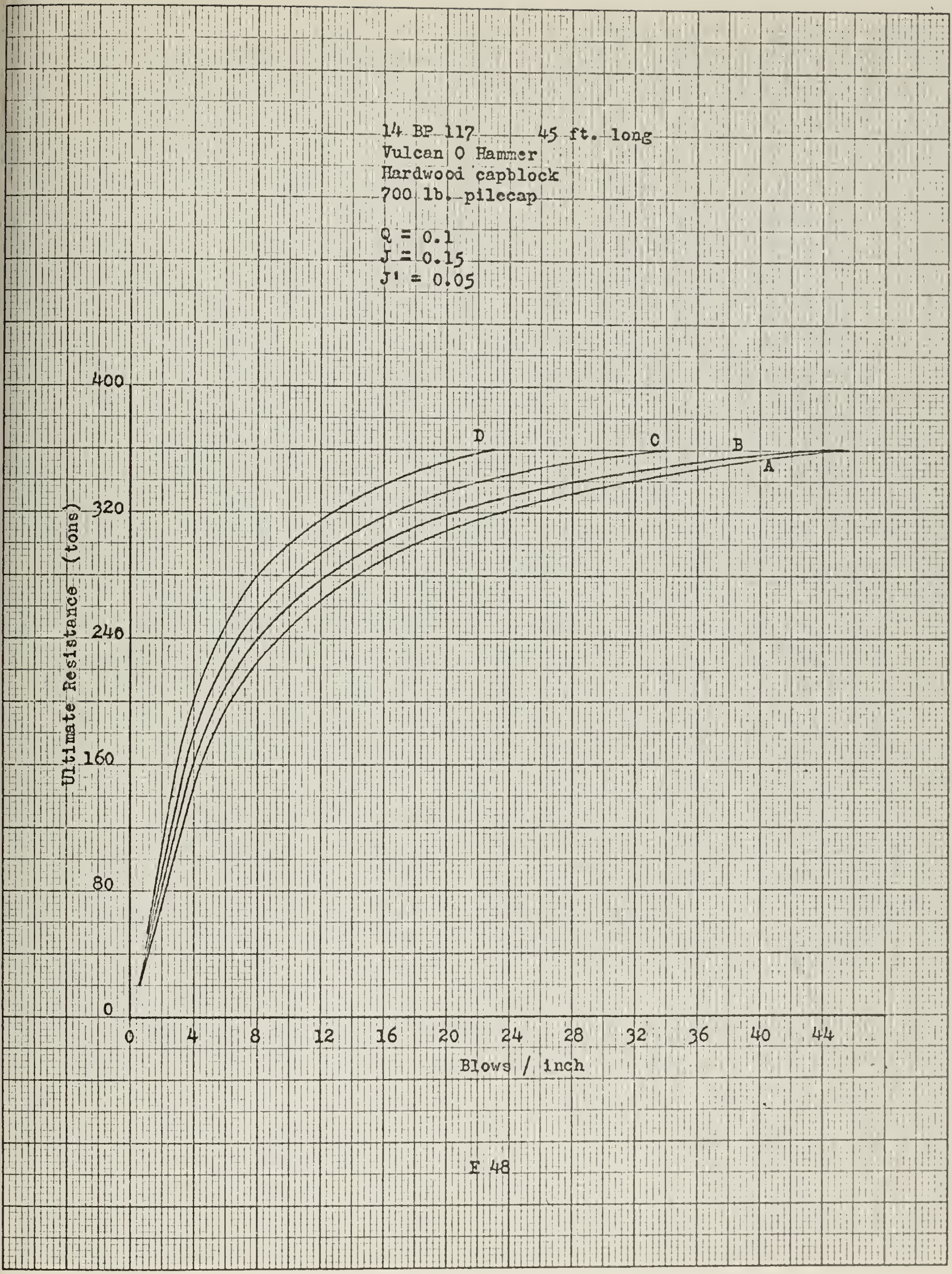

Pile: 14BP89, 45 ft. long
Hammer: Vulcan 0
Capblock: hardwood (e = 0.5)
Wt. of pilecap: 700 lb.
J 0.2
J' 0.067
Q 0.2
Ultimate Resistance (tons)
0
80
160
240
320
400
0
4
8
12
16
20
24
28
32
36
40
44
Blows / inch

Figure E 47


14 BP 117 45 ft. long
Vulcan 0 Hammer
Hardwood capblock
700 lb. pilecap
Q = 0.1
J = 0.15
J' = 0.05
D
C
B
A
Ultimate Resistance (tons)
0
80
160
240
320
400
0
4
8
12
16
20
24
28
32
36
40
44
Blows / inch

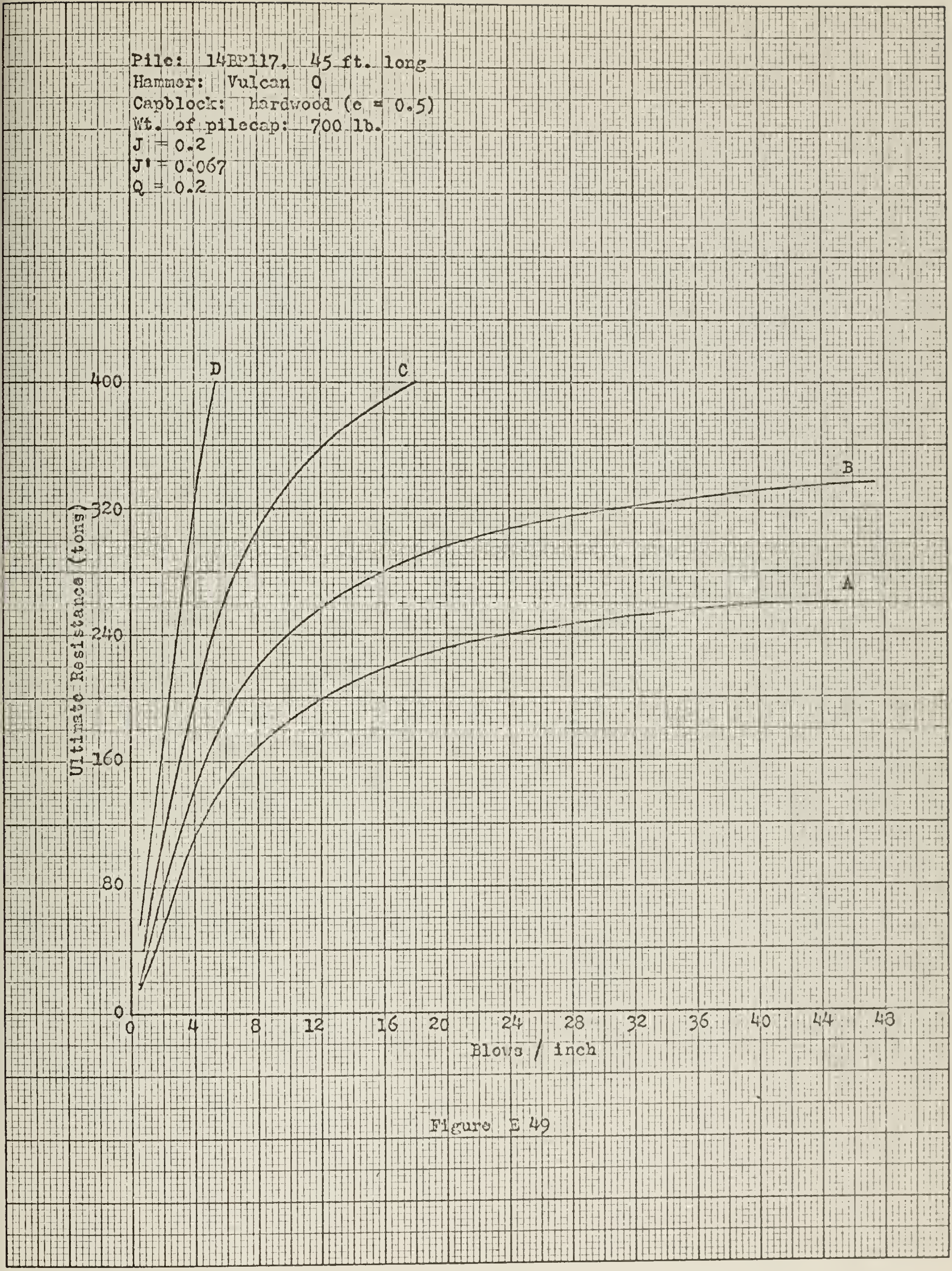

Pile: 14BP117, 45 ft. long
Hammer: Vulcan 0
Capblock: hardwood (c = 0.5)
Wt. of pilecap: 700 lb.
J = 0.2
J' = 0.067
Q = 0.2
D
C
B
A
Ultimate Resistance (tons)
400
320
240
160
80
0
0
4
8
12
16
20
24
28
32
36
40
44
48
Blows / inch

Figure E 49