WAVE EQUATION ANALYSES OF FULL-SCALE TEST PILES USING MEASURED FIELD DATA

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ABSTRACT

A procedure for obtaining soil damping constants for use in wave equation analyses is presented. Field data consisting of static bearing capacity and static and dynamic pile forces, obtained from two full-scale test piles in clay and one in sand, are correlated with the predicted results obtained from wave equation analyses. Thirty-seven additional non-instrumented full-scale test piles are analyzed. Soil damping constants are determined according to soil type and are shown to be valid for both initial driving and redriving at a later date.

KEY WORDS: Wave Equation Analyses, Soil Damping Constants, Dynamic and Static Pile Tests, Predicted Bearing Capacity, Piles in Clay, Piles in Sand

SUMMARY

The information presented in this report was developed during the fifth year of Research Study 2-5-67-125 which is a six-year cooperative research study entitled "Bearing Capacity for Axially Loaded Piles" sponsored jointly by the Texas Highway Department and the Federal Highway Administration. A procedure for obtaining soil damping constants for use in wave equation analyses using measured field data from full-scale pile tests was developed in this study. The validity of the procedure was shown through correlation of measured dynamic and static field data with results obtained from wave equation analyses of three full-scale instrumented test piles. Two of the piles were embedded in clay and the third in saturated sand.

Results of this study indicate the feasibility of determining soil damping constants using the one-dimensional wave equation to analyze field driving and load test data. Although results showed soil damping values to be variable, the concept of using soil damping constants based on soil type for pile bearing predictions was verified. The increase in bearing capacity resulting from soil "set-up" was shown to be predictable from the increased dynamic driving resistance during redriving at a later date. Soil damping constants at initial driving were shown to be valid at redriving.

Records of 37 additional full-scale load test piles were analyzed by the wave equation method to determine soil damping constants for other soil types. Predicted bearing capacities were compared with load test results adjusted for soil set-up in clays. The results were encouraging in light of the multiple load test procedures employed and the problem of evaluating soil set-up in clays for piles which were load tested from 5 to 78 days after initial driving.

IMPLEMENTATION STATEMENT

The findings presented in this report represent the most comprehensive and current information available concerning wave equation analyses of foundation piles. The soil damping constants given for various soil types are the most representative values based on available field data and may be used for routine pile investigations. These damping constants are valid for both time-of-driving and redriving data.

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INTRODUCTION

Nature of the Problem. - The problem of predicting load bearing capacities for single piles has proven to be a troublesome one for engineers. Chellis (4) lists four basic methods of predicting single pile capacities:

1. Static methods relating soil shear strengths as determined by soils investigations to side frictional forces and point bearing force on the pile;
2. Dynamic formulas relating the driving resistance (set per blow) during driving to the static load capacity of the pile;
3. Field load tests of single piles; and
4. The one-dimensional wave equation which provides a completely general method of solution and accounts for the known and assumed material constants of the entire hammer-pile-soil system.

The advantages and disadvantages of the first three methods and associated limitations are widely documented in the literature (38,39,7). A static analysis is only as reliable as the soils investigation and the application of sound engineering judgment and experience in the evaluation of associated engineering

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properties. In most cases the shear strength of the soil undergoes such drastic changes during the pile driving operation that it becomes very difficult, if not impossible, to evaluate soil strength characteristics after driving is completed. Estimating soil strength gained or lost due to soil "set-up" or "relaxation" is only one of the serious drawbacks in applying the static analysis approach to pile bearing prediction. Dynamic formulas are useful in such cases as extrapolating field load test results of driven and load tested piles to untested piles of similar characteristics and driving conditions. Over four hundred and fifty such formulas have been placed on file (35). Cummings (6) has shown many dynamic formulas to be improperly derived and inapplicable to the pile driving problem. Others (5) have also recognized the many omissions and simplifying assumptions of dynamic formulas. While field load tests are desirable to predict pile capacities, the costs in time and money involved are often prohibitive. In some offshore piling operations the magnitude of required loads makes field load tests unfeasible and impractical. The one-dimensional wave equation approach to pile driving analysis, in general, appears more and more to be the most promising, combining economy with a general method of solution applicable to most situations.

Present Status of the Problem. - Issacs (20) is credited with first noting the applicability of the one-dimensional wave equation to the pile driving problem. In 1931, Fox (9) proposed that an exact solution be used for pile driving analysis which was later verified by Granville et al. (13) thru correlation of experimental studies with results obtained by Fox's exact solution. However, the complexity of the solution and the lack of electronic computers necessitated many simplifying assumptions for even the simplest of cases. In 1940, Cummings (6) reviewed previous work using the wave equation noting the long and complicated mathematical expressions requiring laborious numerical calculations. The wave equation's applicability to the pile driving problem was again proposed by Smith in 1950 (36) but gained little popularity until 1960 when he (35) published a summary of the numerical solution to the one-dimensional wave equation including a mathematical model to simulate pile-soil interaction. Smith also recommended a number of material constants to account for the dynamic action of the hammer-pile-soil system based on his many years of experience in the field. These constants were considered adequate for use until more accurate values could be established. According to Smith the total soil resistance mobilized during dynamic loading could be described as follows:

$$R_{u_{dynamic}} = R_{u_{static}} [1 + (J \text{ or } J') V] \qquad [1]$$

where R_u = dynamic or static soil resistance, pounds;

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

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

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

The use of a numerical solution to the one-dimensional wave equation has been well documented by Forehand and Reese (8), Samson et al. (33) and Lowery et al. (22). While necessity dictated initial research to be placed on derivation of computer programs based on Smith's numerical solution (8,30), the investigation of induced pile stresses during driving (15,19,29,31), the significance of various parameters on the solution (14,24,33), the characteristics of hammer, cushion and pile properties (16,17 18,22,24,32), and prediction of pile bearing capacity were also considered. As early as 1963, Forehand and Reese (8) studied the dynamic action of soils during driving and recommended soil parameter values for correlation of predicted capacities with static load test results for the limited number of cases considered. By trial and error, Lowery et al. (22) correlated wave equation results with field load test results for several full scale piles and recommended the use of J = 0.10 for sands, 0.30 for clays and weighted J values for composite soils of sand and clay. J' was set equal to J/3 for all cases. Percent errors based on field load tests adjusted for soil "set-up" varied from -43 to +36 percent for sand supported piles, -32 to +156 percent for clay supported piles and -32 to +27 percent for piles supported by both sand and clay. These results clearly indicated the need for further development and refinement of soil damping constants for reliable field use.

Since 1967 considerable research has been undertaken to determine representative damping constants for various types of soils (3,12,21,27,28). This research consisted of studies on specially prepared soil samples and model pile tests in the laboratory and in the field. Gibson and Coyle (12) obtained constant damping values by modifying Smith's equation as follows:

$$R_{u_{dynamic}} = R_{u_{static}} \left[1 + (J \text{ or } J')\ V^N\right] \quad 0<N\leq 1.0 \qquad [2]$$

where N = a power to which the velocity, V, must be raised for J or J' to be a constant.

For piles in clay, Korb and Coyle (21) recommended values of N = 0.35 for J' = 1.25 and N = 1.0 for J = 0.15 for use with equation [2].

Using equation [2] with the values recommended by Korb and Coyle, Bartoskewitz and Coyle (2) compared predicted bearing capacities with load test results for two full-scale instrumented piles in clay and found the predicted capacities to be approximately 30 percent low. Using N = 0.35, exact agreement resulted at J' = 0.535 and J' = 0.67 for the two piles, respectively. Van Reenen et al. (40) determined the friction damping parameter, J', to be fairly constant at N = 1.0 for the same two test piles and recommended values of

J = 0.15 and J' = 0.20 for piles in clay using Smith's original equation [1]. To date no procedure for determining soil damping constants from full-scale test piles has been developed beyond the stage of trial and error.

Objectives. - Using wave equation analyses of full-scale test piles and measured field data, the objectives of this study are:

1. To develop a procedure for obtaining soil damping constants to be used in wave equation prediction of pile bearing capacity at time of driving and after soil "set-up" has taken place.
2. To determine the validity of material constants for pile driving components above the pile head.

INSTRUMENTED TEST PILES AND FIELD DATA

Port Arthur Test Piles. - During November, 1969, two instrumented test piles were driven and load tested as part of the Intracoastal Canal Bridge on State Highway 87, south of Port Arthur, Texas. Both piles were 16-in. OD, 3/8-in. wall thickness, closed end steel pipe piles driven by a Link Belt 520 diesel hammer. Test pile No. 1 (PA 1) had a total length of 67 ft and an embedded length of 64 ft. Test pile No. 2 (PA 2) had a total length of 78 ft and was embedded 74 ft. Both piles were statically load tested using the Quick-Load Test Method (11) of the Texas Highway Department employed within two hours after initial driving and again at eleven days. The piles were redriven approximately 5 ft upon completion of the eleven day load tests. Strain gages at the pile head and tip were used to determine the total static soil resistance (RUT) and the point-bearing resistance (RUP) during each load test. Dynamic force-time data for each gage location was also recorded during initial driving and final redriving. Such pertinent information as soil properties, pile instrumentation, data recording equipment and detailed static load test data was first reported by Perdue and Coyle (26) and later by Sulaiman (37). Soil profiles, location of strain gages, load settlement curves, load versus depth curves, driving records and summaries of input data used in wave equation analyses are presented in detail in Appendix III. Table 1 summarizes static load test results as reported by Bartoskewitz and Coyle (2).

The predominant soil formation at Port Arthur was found to be Beaumont Clay overlain by Recent river and beach deposits. These underconsolidated deposits extended to an approximate depth of 20 ft for test pile No. 1 and 30 ft for test pile No. 2 as depicted in the soil profiles of Appendix III. The Beaumont Clay is overconsolidated by desiccation as a result of past weathering and exhibits numerous joints and fissures. The soil classified as a CH using the Unified Soil Classification System. The geological history of the area can be found in a report by Sellards et al. (34). The water table was approximately 4 ft below the ground surface.

Final blow counts of 14.5 blows per foot for PA 1 at initial driving (PA 1-Initial) and 16 blows per foot for PA 2 at initial driving (PA 2-Initial) were reported by Bartoskewitz and Coyle (2). However, a closer inspection of the driving records in Tables III-1 and III-3 of Appendix III shows that both piles were driven the last few feet with a cushion consisting of twelve 1/4-in. douglas fir disks. Therefore, the driving records were extrapolated to obtain blow counts of 16 blows per foot for PA 1-Initial and 18 blows per foot for PA 2-Initial during the last foot of driving. The final blow

TABLE 1. - SUMMARY OF STATIC LOAD TEST RESULTS FOR TEST PILES

Test Pile No.	Static Soil Resistance (RUT), tons		Point-Bearing Resistance (RUP), tons		Soil "Set-Up"	$\frac{RUP}{RUT}$	
	Initial Test	Final Test	Initial Test	Final Test	$\frac{\text{Col (3)}}{\text{Col (2)}}$	Initial Test	Final Test
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
PA 1	46.2	100.0	9.0	5.0	2.16	0.195	0.050
PA 2	50.1	122.0	8.0	10.0	2.43	0.160	0.082
CC	138.2	161.1	106.4	112.2	1.166	0.768	0.696

counts for PA 1-Final (final redriving) and PA 2-Final (final redriving) were determined by averaging the relatively constant blow counts encountered once the piles were broken loose and moving relative to the soil. PA 1-Final was determined to have a blow count of 72 blows per foot and PA 2-Final a blow count of 200 blows per foot as shown in Tables III-2 and III-4 of Appendix III.

Corpus Christi Test Pile. - A third instrumented test pile was driven and statically load tested near Corpus Christi, Texas during May, 1971 as part of Park Road 22 at the Intra-coastal Waterway. The test pile was a 16-in. square prestressed concrete pile 38 ft long and driven to an embedded depth of 28.7 ft by a Delmag D-22 diesel hammer. The test pile was load tested using the Quick-Load Test Method upon completion of initial driving and again at ten days. The pile was redriven approximately 4 ft upon completion of the ten day load test. The soil profile indicated the test pile to be almost entirely embedded in saturated sand with the water table at ground surface. The soil classified as SM-SP using the Unified Soil Classification System. A similar set of measurements was obtained for the Corpus Christi test pile as previously discussed for the Port Arthur test piles. Strain gages were employed near the pile head and tip to measure RUT and RUP during static load tests and to obtain dynamic force-time data during initial driving (CC-Initial) and final redriving

(CC-Final). A soil profile, location of strain gages, load settlement curves, load versus depth curves, driving records, and summaries of input data used in wave equation analyses are presented in detail in Appendix III. A summary of static load test results is included in Table 1. As noted on the load settlement curves in Fig. III-9 of Appendix III, the 10 day load test was not carried to completion due to a reaction beam flexure in the loading system. However, since the test pile was embedded almost entirely in sand where soil "set-up" should be small to negligible and since the 10 day load settlement curve closely follows the seven day curve, the seven day values of 161.1 tons (RUT) and 112.2 tons (RUP) were used as final load test results.

The blow count over the last foot of initial driving was determined to be 48 blows per foot as shown in Table III-5 of Appendix III. The relatively constant blow count at final redriving after pile movement had taken place was used as the final blow count and averaged to be 84 blows per foot as shown in Table III-6 of Appendix III.

INVESTIGATION OF DYNAMIC PEAK FORCES

General. - The general procedure followed in this investigation was to vary the stiffness of the first pile segment or the combined stiffness of the cushion plus first pile segment so that wave equation predicted dynamic force-time data agreed closely with experimental (measured) force-time data at the head of each pile. Throughout the investigation the value of Q, the elastic deformation of the soil, was held constant for both friction and point bearing. The value of Q = 0.10-in. as recommended by Smith (35), Lowery et al. (22) and others (2,21) was used.

Wave equation analyses in this study employed the computer program developed by Lowery et al. (24) for solving the one-dimensional wave equation using Smith's numerical method. The program was run using Fortran IV G on the IBM 360/65, OS 360 facilities of the Data Processing Center, Texas A&M University.

Port Arthur Test Piles. - Van Reenen et al. (40) reported that at initial driving of Port Arthur test piles using equation [2], curves of J' versus N converged to an approximate value of J' = 0.20 at N = 1.0 and therefore recommended values of J' = 0.20 and J = 0.15 (after Smith) for wave equation prediction of bearing capacity of friction piles in clay at time of driving. Using the same procedure van Reenen et al.

(40) further determined that at final redriving the curves converged to an approximate value of J' = 0.44 at N = 1.0 and concluded that the larger soil damping value at final redriving was due to increased RUT values resulting from soil consolidation or "set-up". Van Reenen et al. (40) gave no consideration to the dynamic pile forces occurring during initial driving and final redriving.

Since Bartoskewitz and Coyle (2) determined by parameter studies of J and J' that the accuracy of predicted pile bearing capacities for friction piles in clay was not significantly influenced by the point damping parameter, J, a wave equation analysis was conducted for each Port Arthur test pile at initial driving and final redriving using J' = 0.20 and J = 0. Additionally, a wave equation analysis was carried out for each Port Arthur test pile at final redriving using J' = 0.44 and J = 0. A pile segment length of 5 ft, a ram division of one and a uniform side load distribution were used for all of the above wave equation analyses. Partial input data for the analyses are given in Figs. 1 and 2 and complete input data are presented in Tables III-7, III-8, III-9 and III-10 of Appendix III. The results shown in Figs. 1 and 2 clearly verify the findings of van Reenen et al. (40). Results are tabulated in Table 2 showing percent error in each case.

In order to investigate the dynamic peak forces during initial driving and final redriving, two foot pile segment

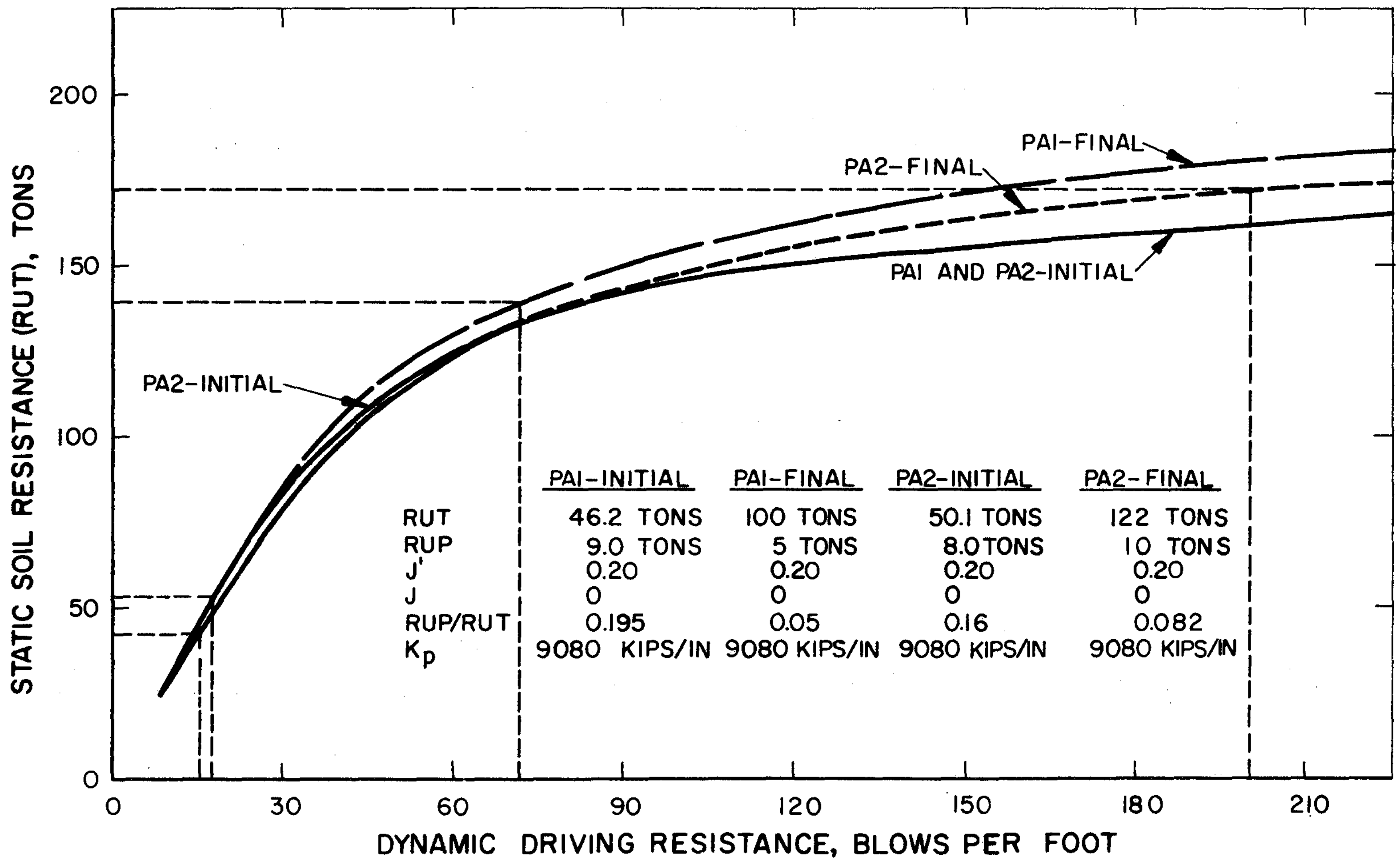

FIG. 1.— RUT vs. BLOW COUNT CURVES FOR PORT ARTHUR TEST PILES USING CALCULATED STIFFNESS VALUE

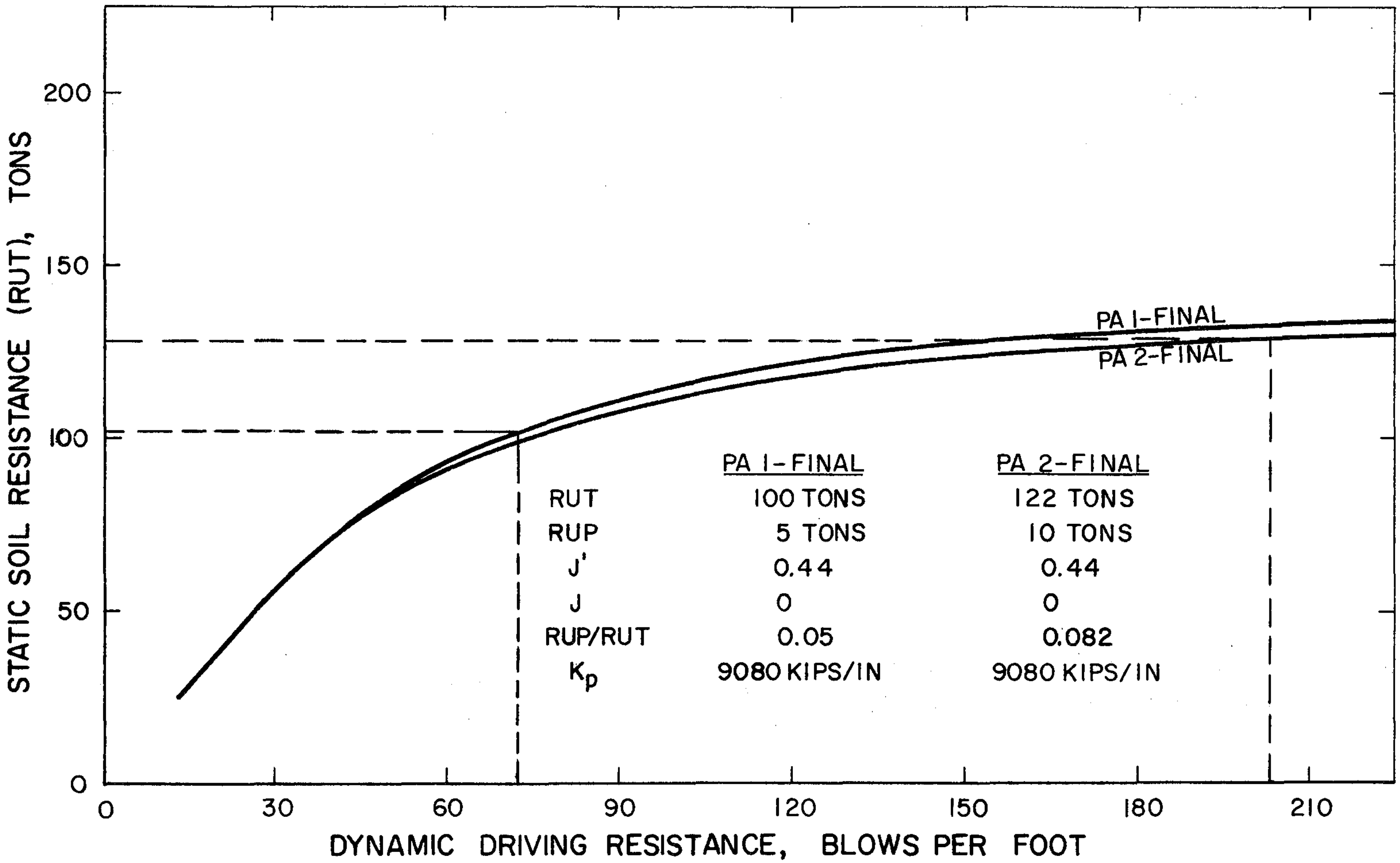

FIG. 2.- RUT vs. BLOW COUNT CURVES FOR PORT ARTHUR TEST PILES USING CALCULATED STIFFNESS VALUE (FINAL REDRIVING)

TABLE 2. - SUMMARY OF PREDICTED CAPACITIES FOR PORT ARTHUR TEST PILES

Pile No.	J' (seconds per foot)	J (seconds per foot)	Capacity by load test (RUT_{LT}), tons	Capacity by wave equation (RUT_{WE}), tons	% Error $\left(\frac{RUT_{WE}-RUT_{LT}}{RUT_{LT}}\right)$ (100)
PA 1-Initial	0.20	0	46.2	45.0	-2.6
PA 2-Initial	0.20	0	50.1	52.5	+4.8
PA 1-Final	0.20	0	100.	138.	+38.0
PA 2-Final	0.20	0	122.	172.	+41.0
PA 1-Final	0.44	0	100.	102.	+2.0
PA 2-Final	0.44	0	122.	128.	+4.9

lengths were used to obtain computed dynamic peak forces within one foot or less of the strain gage locations. Additionally, the ram was broken into 3 segments (NR = 3) to lower the dynamic peak force at the head of each pile. Lowery et al. (24) determined that when a steel ram longer than 6 ft impacts directly on a steel anvil, dividing the ram into segments has a significant effect on the dynamic peak forces in the pile. The Link Belt 520 has a ram 6.67 ft long. The greatest reduction in peak forces for all cases considered by Lowery et al. was of the order of 25 percent using a point bearing pile with soil parameters of RUT = 500 kips, Q = 0.10-in. and J = 0.15 sec/ft. Since the Port Arthur test piles were friction piles with little to no point damping and a smaller total load (RUT), the effect of dividing the ram into segments should be less significant. This was verified to be the case by comparing results using ram divisions of one (NR = 1) and three (NR = 3). The reduction in dynamic peak forces at each gage location using NR = 1 and NR = 3 was of the order of 5 percent or less.

Table 3 is a summary of dynamic peak forces as computed by the wave equation using NR = 3 and determined experimentally (measured) for each gage location for Port Arthur test piles. Experimental data for gage No. 2 of PA 1 and gage No. 3 of PA 2 were not recorded in the field. For all gage locations and all cases the wave equation results were much higher than the

TABLE 3. - COMPARISON OF COMPUTED AND EXPERIMENTAL DYNAMIC PEAK FORCES FOR PORT ARTHUR TEST PILES

Pile No.	Gage No.	Experimental dynamic peak force (kips)	Computed dynamic peak force using J'=0.20, J=0 (kips)	Computed dynamic peak force using J'=0.44, J=0 (kips)
PA 1-Initial	1	182.4	525.3	
	3	147.5	487.1	
	4	55.5	413.1	
	5	28.5	195.7	
PA 1-Final	1	294.4	492.3	492.6
	3	180.6	357.0	322.9
	4	82.5	328.8	264.4
	5	56.6	171.1	149.2
PA 2-Initial	1	215.0	533.2	
	2	190.8	500.6	
	4	117.8	437.4	
	5	36.0	194.8	
PA 2-Final	1	240.1	492.2	538.6
	2	273.8	518.6	512.5
	4	122.3	342.8	274.3
	5	23.0	163.4	140.2

experimental results. The computed rise times to the peak forces in all cases were also of much shorter duration than those measured. These discrepancies showed the need for a closer investigation of input parameters.

Prior to initial driving of the Port Arthur test piles, a 16-in. OD, 3/8-in. wall thickness, 2 ft high load cell was bolted to each pile head. An adapter section of pipe possessing the same dimensions as the load cell and pile was then bolted to the top of the load cell. The pile driving adapter of the hammer was placed directly on top of this adapter section. The purpose of the load cell was to increase the reliability of static and dynamic measurements at the pile head and to provide a means of calibration for the jack used in conducting static load tests. From the viewpoint of a wave equation analysis this merely extended the pile length since the load cell and adapter section were of the same dimensions as the pile. Since steel on steel is the net result with no cushion between the hammer adapter and the added adapter section, the stiffness of the first pile segment (K_p) was calculated to be 9,080 kips/in. for a five foot segment and 22,700 kips/in. for a two foot segment.

The initial adapter section was 3 ft long. It was used to drive and load test PA 1-Initial before being shortened by approximately 1 1/2 ft by using a cutting torch in the field. This shortened section was then used to drive and load test PA 1-Final, PA 2-Initial and PA 2-Final. An inspection of the original 3 ft adapter section revealed that it had been beveled on the inside when cut, resulting in a reduced contact area. The contact area of the shortened adapter section was estimated to be less than 10 percent of its original area. Full contact area was not developed during initial driving or final redriving as a result of deformation or yielding as evidenced by the total absence of any crushed or deformed portion of the shortened adapter section after being used to drive and load test PA 1-Final, PA 2-Initial and PA 2-Final. This reduction in contact area greatly reduced the first pile segment stiffness and caused uneven distribution of forces under dynamic loading. Lowery et al. (24) reported that maximum dynamic forces consistently increased with an increase in stiffness. At initial driving the reduced stiffness was still sufficient to drive the piles without significantly affecting the blow counts. However, at final redriving the reduced stiffness had a significant effect on the dynamic driving resistances required to overcome soil resistances and damping and hence the predicted pile capacities. This is verified later in this study.

Because of the poor contact area in all four cases of Port Arthur, the true stiffness of the first pile segment could not be calculated directly. Since a good correlation between experimental and computed force-time data was desirable, the first pile segment stiffness for each Port Arthur case was obtained by trial and error until the computed dynamic peak force at the pile head agreed closely with the experimental dynamic peak force. Other input parameters were held constant. Table 4 lists the stiffness values determined for each case with their associated dynamic peak forces. Complete correlation of force-time data for all gage locations and all cases, as well as prediction of pile bearing capacity using the stiffness values of Table 4, are presented later in this study after soil damping constants have been investigated.

TABLE 4. - SUMMARY OF STIFFNESS VALUES FOR

PORT ARTHUR TEST PILES

Pile No.	First pile segment stiffness, K_p (kips/in.)	Experimental dynamic peak force at pile head (kips)	Computed dynamic peak force at pile head using J'=0.20, J=0, (kips)
PA 1-Initial	75	182.4	187.0
PA 1-Final	350	294.4	292.2
PA 2-Initial	178	215.0	243.2
PA 2-Final	125	240.1	245.4

Corpus Christi Test Pile. - While a calibrated load cell was used during static load tests of the Corpus Christi test pile, it was not used during either the initial driving or final redriving phase. A 5 1/4-in. plywood fir cushion was utilized during initial driving and final redriving to protect the concrete pile from high stress damage. The cushion stiffness (K_c) was calculated to be 1,705 kips/in. using the secant modulus of elasticity recommended by Lowery et al. (22). The cushion stiffness (K_c) combined with the first pile segment stiffness (K_p) calculated to be 1,595 kips/in. for a 5 ft segment length and 1,660 kips/in. for a 2 ft segment length. Since these values are very nearly the same, a combined cushion plus first pile segment stiffness (K_{c+p}) of 1,595 kips/in. was used for both pile segment lengths.

Since the Corpus Christi test pile data had not been previously analyzed, no damping constants were readily available for use in computing dynamic peak forces in the pile at initial driving and final redriving as in the case of Port Arthur test piles. Therefore, wave equation analyses were conducted for Corpus Christi test pile using values of J = 0 and J' = 0.20, 0.30, 0.40 and 0.50 to see if changes in the damping would be significant on the dynamic peak force at the pile head. Two foot pile segments were used with NR = 3 as previously discussed for Port Arthur piles. It was determined from these analyses that the dynamic peak force at the pile head changed less than one percent while peak forces at other gage locations were appreciably reduced with increased values of J'. Since the value of J' = 0.50 resulted in better blow count agreement for CC-Initial and CC-Final, it was used for a comparison of computed and experimental dynamic peak forces. A summary of these results is presented in Table 5.

As shown in Table 5 the computed dynamic peak forces at all gage locations for both CC-Initial and CC-Final were too high. Therefore, the cushion plus first pile segment stiffness was varied by the trial and error procedure employed for Port Arthur test piles in order to obtain better correlation of dynamic peak forces at the pile head. This procedure resulted in a value of 750 kips/in. for the combined cushion plus first pile segment stiffness (K_{c+p}). A summary of stiffness values and their associated dynamic peak forces are tabulated in Table 6. Complete correlation of force-time data for all gage locations using K_{c+p} = 750 kips/in. is presented in a later section of this study after soil damping constants have been investigated.

TABLE 5. - COMPARISON OF COMPUTED AND EXPERIMENTAL DYNAMIC PEAK FORCES FOR CORPUS CHRISTI TEST PILE

Pile No.	Gage No.	Experimental dynamic peak force (kips)	Computed dynamic peak force using J'=0.50, J=0, (kips)
CC-Initial	1	505.6	632.8
	2	504.4	544.0
	3	218.1	291.7
CC-Final	1	517.2	647.1
	2	511.6	539.0
	3	248.0	295.4

TABLE 6. - SUMMARY OF STIFFNESS VALUES FOR CORPUS CHRISTI TEST PILE

Pile	Cushion plus first pile segment stiffness, K_{c+p} (kips/in.)	Experimental dynamic peak force at pile head (kips)	Computed dynamic peak force at pile head using J'=0.50, J=0 (kips)
CC-Initial	1,595	505.6	632.8
CC-Final	1,595	517.2	647.1
CC-Initial	750	505.6	506.1
CC-Final	750	517.2	516.2

Since the dynamic peak forces computed at the head of the pile using K_{c+p} = 1,595 kips/in. were only in the order of 25 percent error and the rise times to the first peaks were not significantly changed using K_{c+p} = 750 kips/in., both stiffness values were considered in the investigation of soil damping constants and pile bearing prediction later in this study.

The significance of breaking the ram into segments was also considered for the Corpus Christi pile. The reduction in peak forces were of the order of 10 percent at the center and 5 percent at the pile head and tip using NR = 1 and NR = 3, respectively.

INVESTIGATION OF SOIL DAMPING CONSTANTS

General. - Having obtained a reasonable agreement of computed and experimental dynamic peak forces at the pile head for each case, the next step in the procedure involved an investigation of soil damping constants resulting in proper blow count and hence bearing capacity prediction.

Port Arthur Test Piles. - Consider PA 1-Initial with K_p = 75 kips/in. as previously established for dynamic peak force agreement at the pile head. If J is held constant at .01 while varying J' at .1, .2, .3, .4 and using 75 kips/in. as the first pile segment stiffness, a plot of J' versus blow count results in four points on the J = .01 curve. If J is now incremented to new values of .09, .17 and .25 and the procedure repeated, a plot J' versus blow count for curves of J = .09, .17 and .25 are developed as illustrated in Fig. 3. By entering Fig. 3 at the proper blow count (16 blows per foot) and intersecting the J = .01 curve, a unique value of J' = .180 is determined. This then is one workable combination of J and J' that results in accurate pile bearing prediction. The same procedure can be applied to the curves of J = .09, .17 and .25 to obtain other workable combinations of J and J'. If the entire procedure is repeated for the other three Port Arthur cases using K_p = 350, 178 and 125 kips/in., respectively,

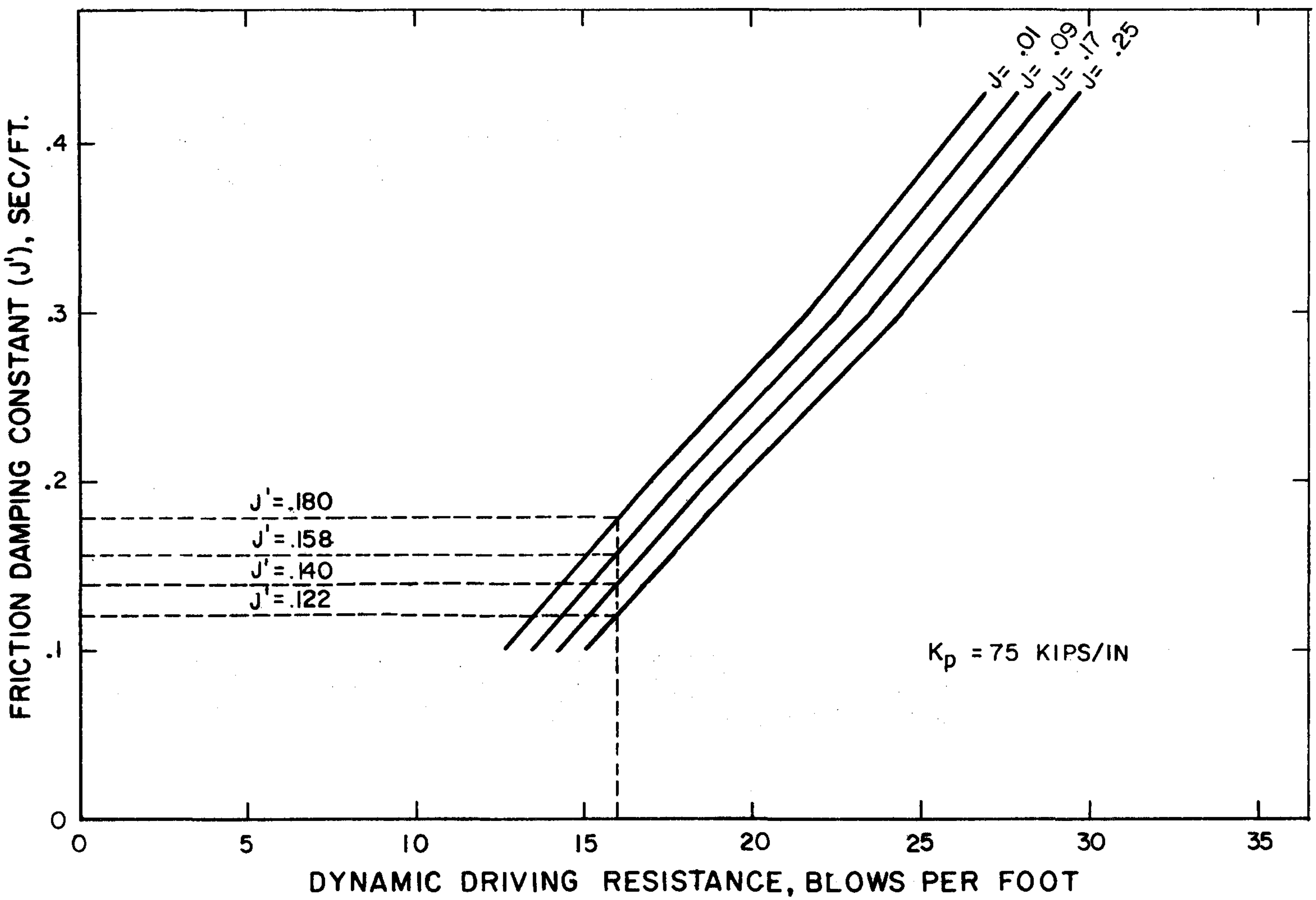

FIG. 3.—FRICTIONAL DAMPING CONSTANT vs. DYNAMIC DRIVING RESISTANCE (PA NO. 1 INITIAL)

workable combinations for all four cases are determined. These workable combinations of J and J' have been plotted in Fig. 4(a) and tabulated in Table 7 including average J versus J' values. Since the damping values for the four cases vary widely and the average J versus J' curve of Fig. 4(a) is a reflection of the average stiffness value as well as average J and J' values, a second set of curves was generated using the average stiffness value of K_p = 182 kips/in. Results are plotted in Fig. 4(b) and summarized in Table 8 including average values.

Figs. 4(a) and 4(b) show that in all cases the piles are relatively insensitive to any change in J and very sensitive to any change in J'. Based on the average J versus J' curves in Figs. 4(a) and 4(b) it appears that J' = .20 is a valid soil damping constant for Port Arthur test piles using any value of J considered. However, it is shown in the next section of this study that the best correlation of computed and experimental force-time data at the pile tips was obtained using J = 0.

Corpus Christi Test Pile. - Using the same procedure as outlined above for the Port Arthur test piles, workable combinations of J and J' were determined for the Corpus Christi test pile at initial driving and final redriving using both the established stiffness value of K_{c+p} = 750 kips/in. and the calculated stiffness value of K_{c+p} = 1,595 kips/in. Results including average values are plotted in Figs. 5(a) and 5(b) and tabulated in Table 9. As expected the Corpus Christi test

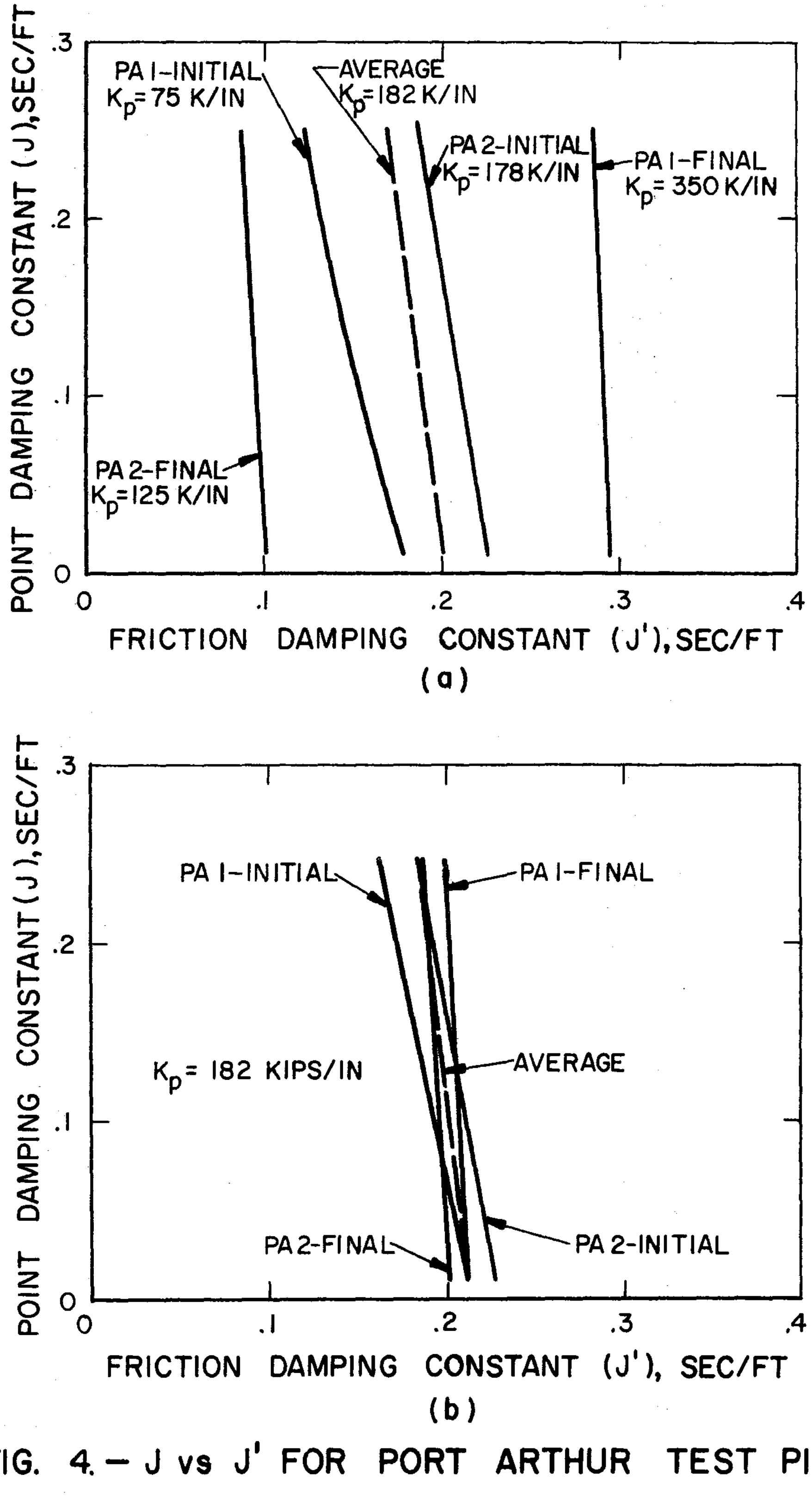

FIG. 4.— J vs J' FOR PORT ARTHUR TEST PILES

TABLE 7. - SUMMARY OF WORKABLE DAMPING VALUES FOR PORT ARTHUR TEST PILES USING ESTABLISHED STIFFNESS VALUES

Pile No.	Stiffness value, K_p, (kips/in.)	J (sec/ft)	J' (sec/ft)
PA 1-Initial	75	.01	.180
		.09	.158
		.17	.140
		.25	.122
PA 1-Final	350	.01	.295
		.09	.291
		.17	.288
		.25	.285
PA 2-Initial	178	.01	.227
		.09	.216
		.17	.200
		.25	.186
PA 2-Final	125	.01	.102
		.09	.097
		.17	.092
		.25	.082
Average (4 cases)	182	.01	.201
		.09	.190
		.17	.180
		.25	.170

TABLE 8. - SUMMARY OF WORKABLE DAMPING VALUES FOR PORT ARTHUR TEST PILES USING AVERAGE ESTABLISHED STIFFNESS VALUE

Pile No.	Stiffness value, K_p, (kips/in.)	J (sec/ft)	J' (sec/ft)
PA 1-Initial	182	.01	.212
		.09	.196
		.17	.178
		.25	.163
PA 1-Final	182	.01	.212
		.09	.206
		.17	.203
		.25	.201
PA 2-Initial	182	.01	.228
		.09	.214
		.17	.200
		.25	.187
PA 2-Final	182	.01	.202
		.09	.198
		.17	.192
		.25	.187
Average (4 cases)	182	.01	.214
		.09	.204
		.17	.193
		.25	.184

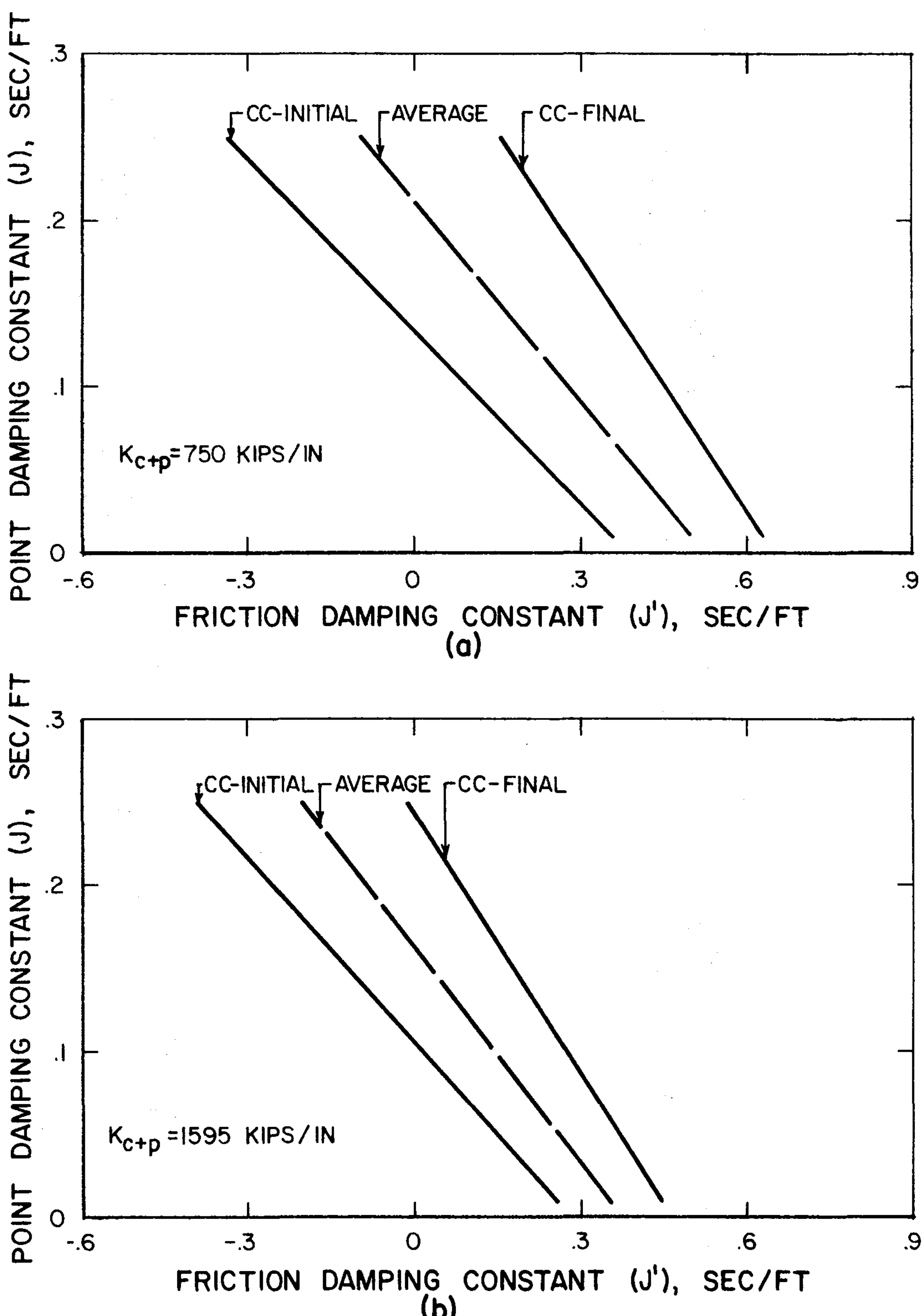

FIG. 5.— J vs. J' FOR CORPUS CHRISTI TEST PILE

TABLE 9. - SUMMARY OF WORKABLE DAMPING VALUES FOR CORPUS CHRISTI TEST PILE USING ESTABLISHED AND CALCULATED STIFFNESS VALUES

Pile No.	Stiffness value, K_{c+p}, (kips/in.)	J (sec/ft)	J' (sec/ft)
CC-Initial	750	.01	.365
		.09	.120
		.17	-.115
		.25	-.340
CC-Final	750	.01	.630
		.09	.466
		.17	.310
		.25	.156
Average	750	.01	.498
		.09	.293
		.17	.098
		.25	-.092
CC-Initial	1,595	.01	.262
		.09	.016
		.17	-.170
		.25	-.385
CC-Final	1,595	.01	.450
		.09	.287
		.17	.127
		.25	-.010
Average	1,595	.01	.356
		.09	.152
		.17	-.022
		.25	-.198

pile was more sensitive to J than J'. The high RUP/RUT ratios are an indication of this and the J versus J' curves verify it.

A comparison of measured static peak forces and dynamic peak forces at the pile tip for initial driving and initial load test and final load test and final redriving revealed that they were approximately the same as shown in Table 10.

TABLE 10. - STATIC AND DYNAMIC PEAK FORCES AT PILE TIP OF CORPUS CHRISTI TEST PILE

Pile No.	Pile tip static peak force (kips)	Pile tip dynamic peak force (kips)
CC-Initial	212.8	218.1
CC-Final	224.4	248.0

The results contained in Table 10 indicate that the damping term in equation [1] must be approximately zero for $R_{u_{static}}$ and $R_{u_{dynamic}}$ to be approximately equal. Therefore entering the average J versus J' curve of Fig. 5(a) for K_{c+p} = 750 kips/in. at J = 0, a value of J' = 0.50 is obtained. As seen in Fig. 5(b) and Table 9 a value of J' = 0.50 should result in slight overdamping for K_{c+p} = 1,595 kips/in. However, since the Corpus Christi test pile is more sensitive to point damping than friction damping, soil damping constants of J = 0 and J' = 0.50 should yield acceptable results of pile bearing prediction on the conservative side. This is fully investigated later in this study in the section on prediction of bearing capacity by wave equation analysis.

CORRELATION OF FORCE-TIME DATA

Having established the stiffness values required for dynamic peak force agreement at the pile head for each test pile and having selected soil damping constants, the next step in the procedure was to correlate the force-time data at each gage location for initial driving and final re-driving. To accomplish this, wave equation analyses were conducted for each test pile using 2 ft pile segments to obtain force-time data within one foot of any gage location. The previously established stiffness values of 75, 350, 178 and 125 kips/in. were used for the Port Arthur piles and 750 kips/in. for the Corpus Christi pile. Soil damping constants used were J = 0 and J' = 0.20 for Port Arthur piles and J = 0 and J' = 0.50 for the Corpus Christi pile. In order to duplicate the soil resistance distribution the static load versus depth curves of Appendix III were used as input for all wave equation analyses.

The first wave equation analysis involved using the hammer-pile-soil system parameters as input and plotting the resulting force-time points at each gage location. The dashed line curves in Figs. 6, 7, 8 and 9 are the resulting force-time plots at gages 1, 3, 4 and 5, respectively, of PA 1-Initial. They are labeled as Hammer Input curves. The curves in Figs. 6, 7, 8 and 9 are the actual e: (measured)

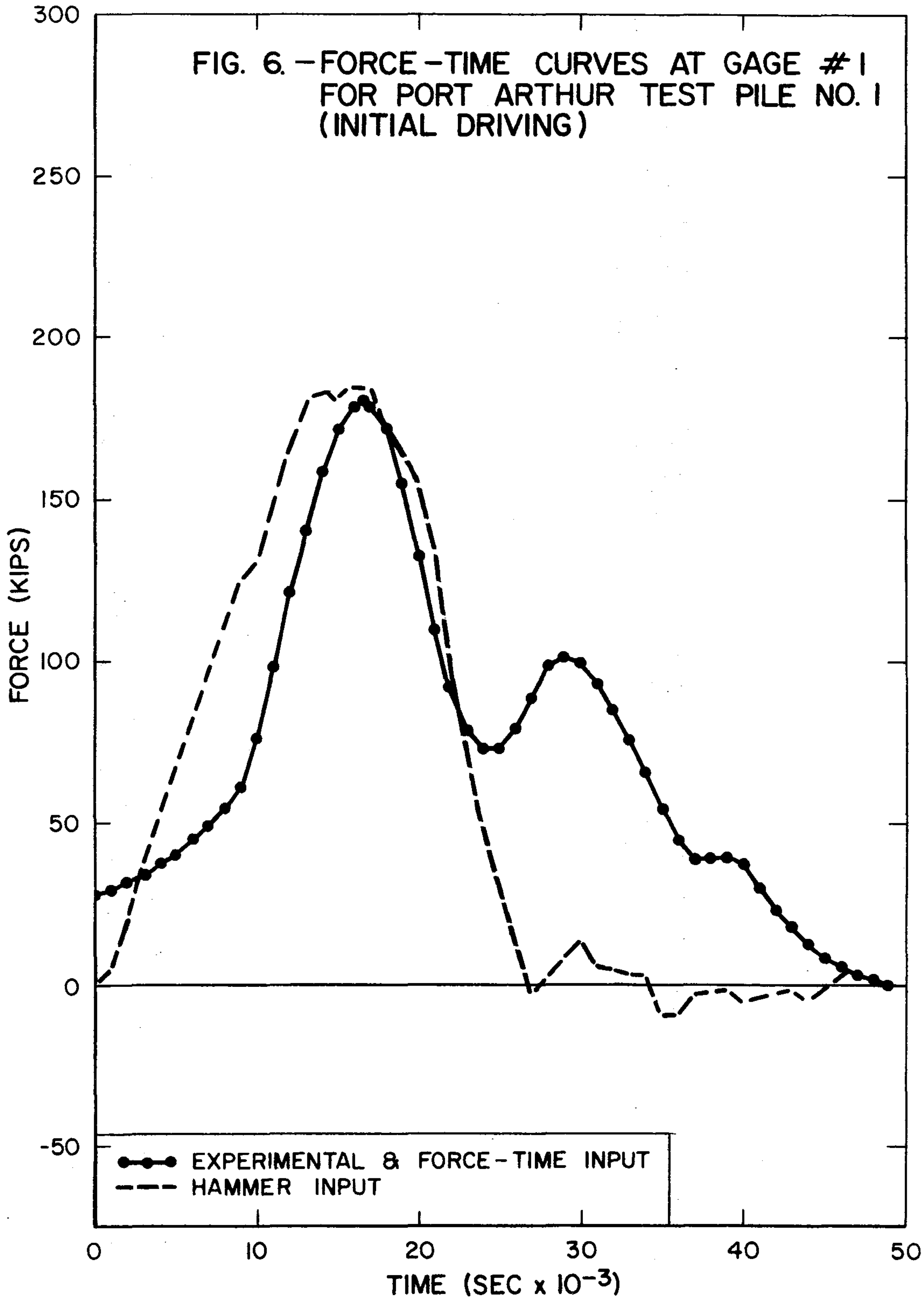

FIG. 6. – FORCE-TIME CURVES AT GAGE #1 FOR PORT ARTHUR TEST PILE NO. 1 (INITIAL DRIVING)

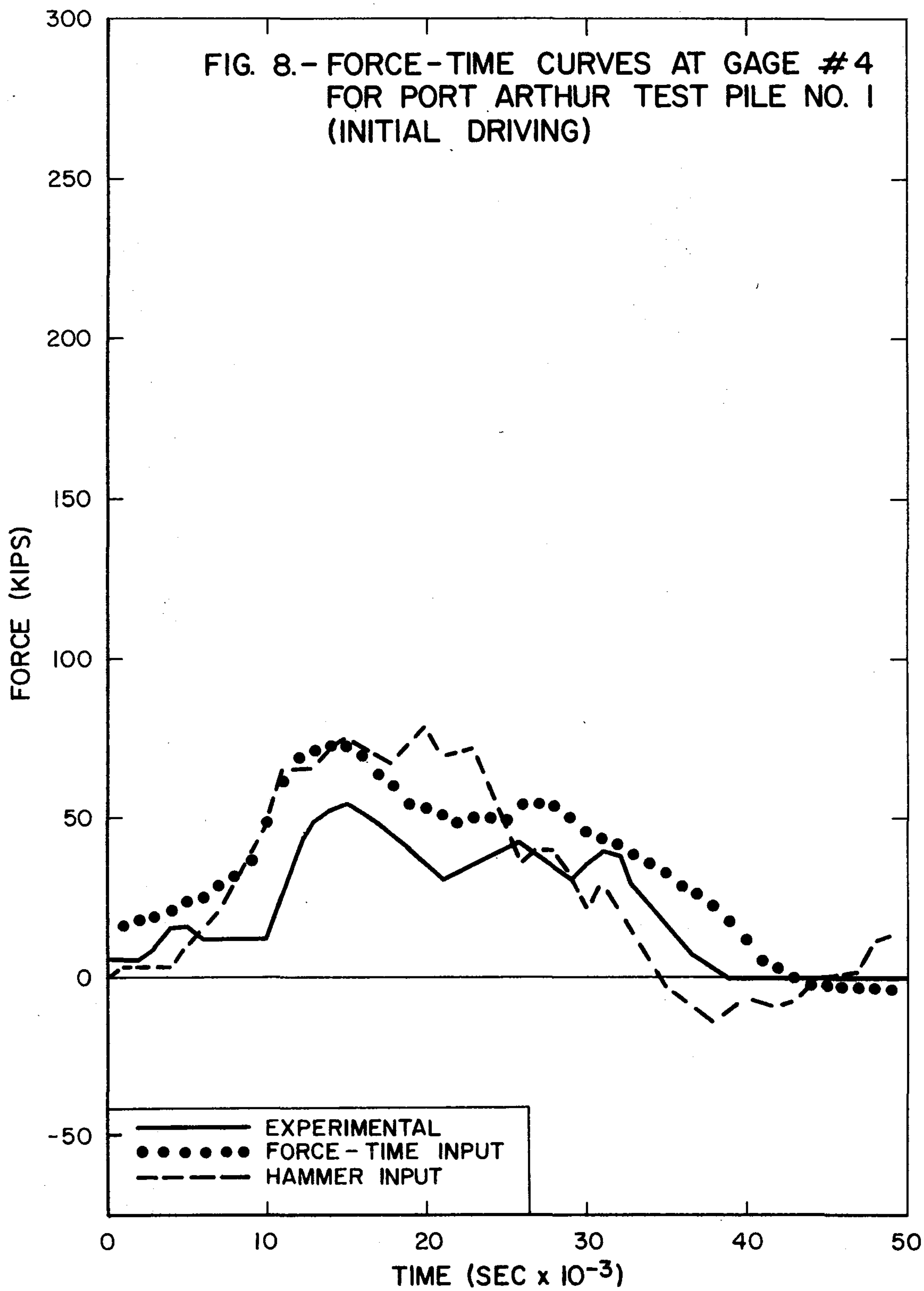
FIG. 8.- FORCE-TIME CURVES AT GAGE #4
FOR PORT ARTHUR TEST PILE NO. I
(INITIAL DRIVING)
FORCE (KIPS)
300
250
200
150
100
50
0
-50
0
10
20
30
40
50
TIME (SEC x 10^-3)
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

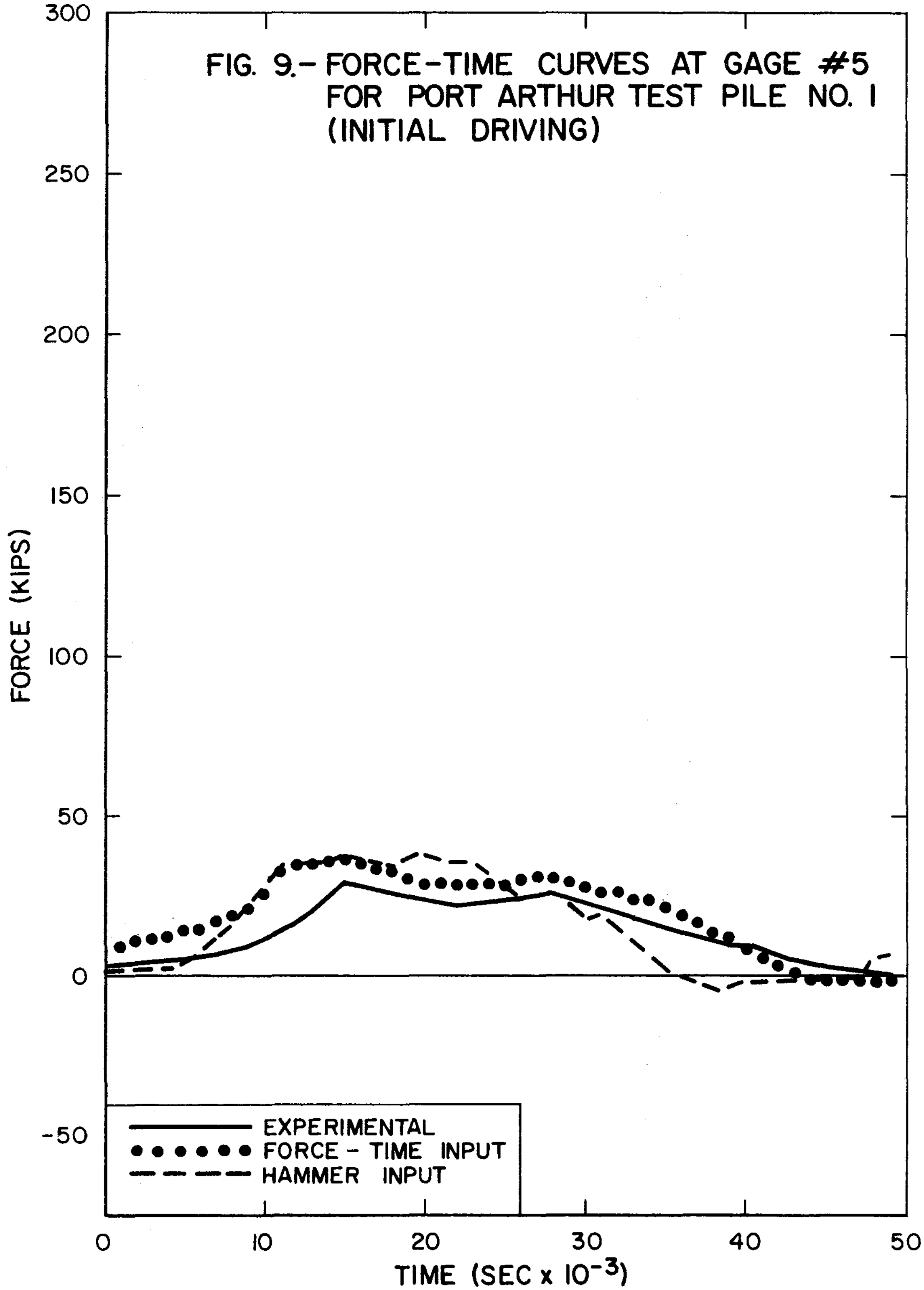

FIG. 9.- FORCE-TIME CURVES AT GAGE #5 FOR PORT ARTHUR TEST PILE NO. 1 (INITIAL DRIVING)

force-time points for PA 1-Initial plotted at the same time iteration as the Hammer Input curves. These curves are labeled Experimental curves. Thus, a comparison of Hammer Input curves with Experimental curves is possible at each gage location.

A second wave equation analysis was performed for each test pile case using as input the experimental (measured) force-time data at the pile head reduced to the same time iteration as the Hammer Input data. All other input parameters were the same as the first analysis discussed above for each case. The resulting force-time points at each gage location are plotted as solid dots in Figs. 6, 7, 8 and 9 for PA 1-Initial. They are labeled as Force-Time Input. Since the hammer and driving accessories were omitted as input for the second analysis, a comparison of Force-Time Input and Experimental curves enables an evaluation of the pile-soil system at each gage location.

Referring to Figs. 6, 7, 8 and 9, the zero time for each set of curves was arbitrarily taken as the zero time computed by the wave equation for the Hammer Input curve with the Force-Time Input and Experimental curves matched approximately to the first compressive peak of the Hammer Input curve. Force-time curves for PA 1-Final, PA 2-Initial, PA 2-Final, CC-Initial and CC-Final are presented in Appendix II. Only the first compressive peak and the reflected peak are shown in most cases with the rebound peak or third peak not shown. Since the wave equation computer program used throughout this study employed the option of computing blow count based on the maximum point displacement governed by the first compressive peak reaching the pile tip, only the first peak is of concern for pile capacity prediction. Thus, as soon as the maximum displacement of the pile tip occurred when velocity of the tip was zero, computations ceased and a set per blow was computed by subtracting the elastic rebound of the soil, Q, from the maximum point displacement. The reciprocal of this set per blow resulted in a blow count in blows per in. which was converted in the program print out to blows per foot.

In general the following observations can be made concerning the correlation of force-time data discussed above and plotted in Figs. 6, 7, 8, 9 and Figs. II-1 thru II-18 of Appendix II:

1. The Force-Time Input curves follow the Experimental curves very closely in shape and amplitude at each gage location, indicating a reasonably good pile-soil model.
2. All three curves correlate well over the first compressive peak resulting in good agreement of dynamic peak forces and pile displacements.
3. The Hammer Input curves do not duplicate the preloading resulting from trapped compressed gases in the hammer that took place before the first compressive peak. Only part of the preloading is

observed for the Force-Time Input and Experimental curves since zero times for these curves were normalized to that of the Hammer-Input curves as previously discussed.

4. The Hammer Input curves indicate higher tensile (negative) forces than were actually measured in the piles.
5. The reflected compressive peaks are, in general, greater than those determined experimentally.
6. The Hammer Input curves damp out in a shorter time oscillating about the time axis while the Force-Time Input and Experimental curves tend to remain in the compressive range longer.
7. The maximum compressive forces occurred near the heads of the piles.
8. Maximum tensile forces occurred near the midpoints of the piles.
9. Higher stiffness values resulted in higher compressive peak forces with accompanying decreases in rise times.
10. The compressive peak forces computed by the wave equation agree well with those determined experimentally.

Summaries of compressive peak forces at each gage location for Port Arthur test piles and the Corpus Christi test pile are presented in Tables 11 and 12, respectively. Referring to Table 11, the Hammer Input dynamic peak force at the pile tip (gage No. 5) using J = 0 was higher than the experimental dynamic peak force for three out of four cases. Any value of J greater than zero would result in even higher computed peak forces at the pile tips. Thus, the previously selected soil damping constants of J = 0 and J' = .20 for Port Arthur test piles yield the best correlation of force time data at the pile tips.

TABLE 11. - SUMMARY OF DYNAMIC PEAK COMPRESSIVE FORCES FOR PORT ARTHUR TEST PILES

Pile No.	Gage No.	Experimental dynamic peak force (kips)	Force-Time Input dynamic peak force using J'=.20, J=0 (kips)	Hammer Input dynamic peak force using J'=.20, J=0 (kips)
PA 1-Initial	1	182.4	182.4	187.0
	3	147.5	141.7	143.6
	4	55.5	73.7	79.2
	5	28.5	36.8	38.8
PA 1-Final	1	294.4	294.4	292.2
	3	180.6	207.0	185.9
	4	82.5	76.4	70.1
	5	56.6	36.3	28.6
PA 2-Initial	1	215.0	215.0	243.3
	2	190.8	196.0	215.9
	4	117.8	90.8	102.4
	5	36.0	31.2	37.2
PA 2-Final	1	240.1	240.1	245.4
	2	273.8	267.1	261.2
	4	122.3	99.0	98.8
	5	23.0	36.3	36.4

TABLE 12. - SUMMARY OF DYNAMIC PEAK COMPRESSIVE FORCES FOR CORPUS CHRISTI TEST PILE

Pile No.	Gage No.	Experimental dynamic peak force (kips)	Force-Time Input dynamic peak force using J'=.50, J=0 (kips)	Hammer Input dynamic peak force using J'=.50, J=0 (kips)
CC-Initial	1	505.6	505.6	506.1
	2	504.4	502.5	486.4
	3	218.1	249.2	247.7
CC-Final	1	517.2	517.2	516.2
	2	511.6	445.4	504.4
	3	248.0	252.1	257.4

PREDICTION OF BEARING CAPACITY BY WAVE EQUATION ANALYSIS

General. - In order to investigate all cases previously discussed for comparison of wave equation predicted bearing capacities with static load test results at initial driving and final redriving, a wave equation analysis was conducted for each case. Five foot pile segments were employed in each analysis using the previously established stiffness value and selected soil damping constants. Uniform side load distributions were used for all cases. As shown by the load versus depth curves of Appendix III, the actual side load distributions for Port Arthur test piles were somewhere between uniform and triangular distributions. Bartoskewitz and Coyle (2) previously reported that the side load distribution was not significant for piles in clay and obtained approximately the same results with either distribution. The load versus depth curves in Appendix III for the Corpus Christi test pile show a uniform side load distribution for CC-Initial and very nearly a uniform distribution for CC-Final.

Port Arthur Test Piles. - RUT versus blow count curves for Port Arthur test piles using the previously established stiffness value for peak force agreement in each case are plotted in Fig. 10. Results using the average stiffness value of 182 kips/in. are presented in Fig. 11. It is interesting

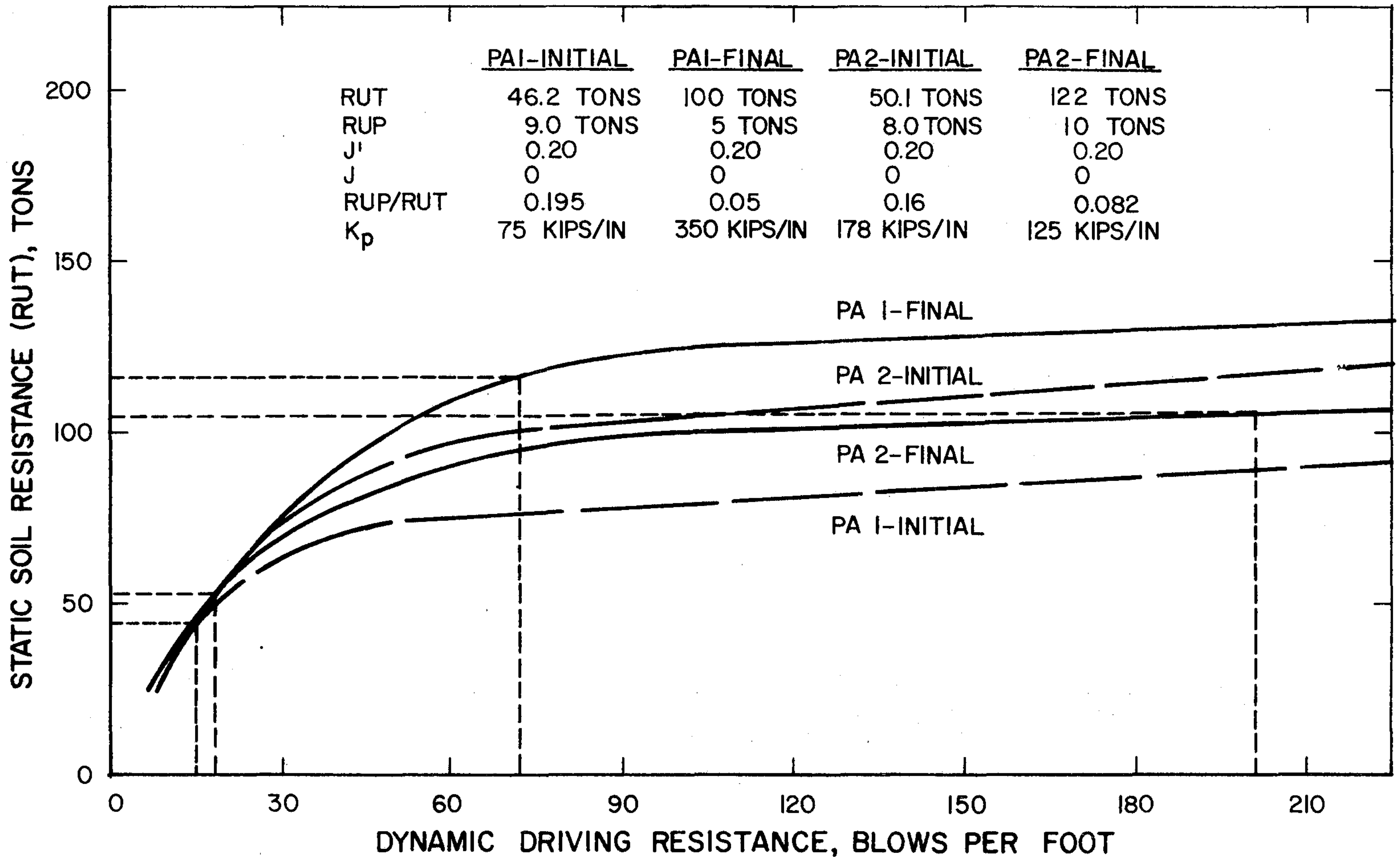

FIG. 10.— RUT vs. BLOW COUNT CURVES FOR PORT ARTHUR TEST PILES USING STIFFNESS VALUES FOR EXPERIMENTAL PEAK FORCE AGREEMENT

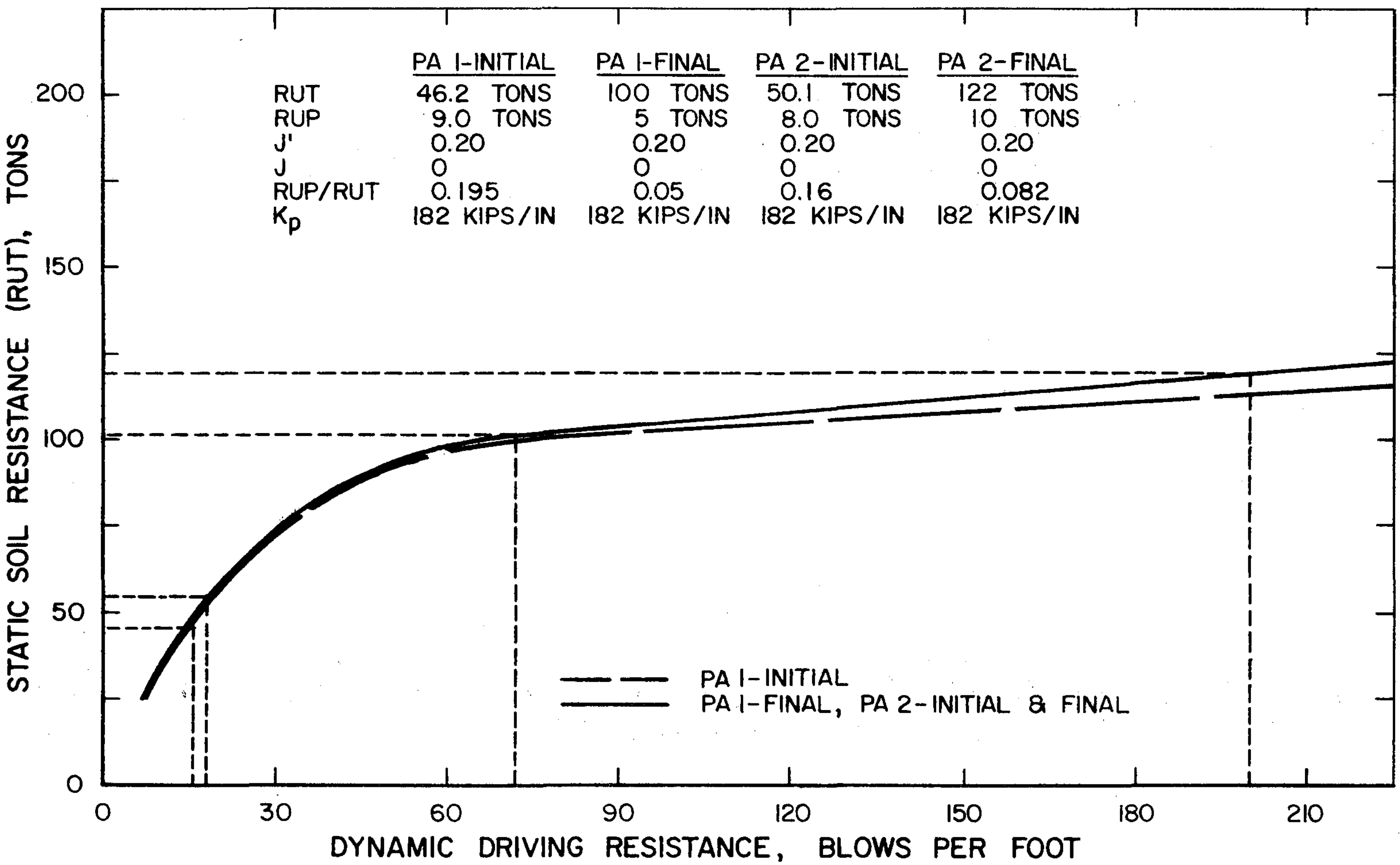

FIG. 11.— RUT vs. BLOW COUNT CURVES FOR PORT ARTHUR TEST PILES USING AVERAGE STIFFNESS VALUE FOR EXPERIMENTAL PEAK FORCE AGREEMENT

to note that the RUT values shown in Fig. 10 vary over a wide range for a given blow count when the established stiffness value is used for each case. However, as shown in Fig. 11 the curves are essentially the same when the average stiffness value is used for all cases. Results for all Port Arthur test cases have been summarized in Table 13 for easy reference showing the percent error in each case.

Corpus Christi Test Pile. - Predicted bearing capacity curves for the Corpus Christi test pile using the previously established stiffness value of 750 kips/in. for peak force agreement are presented in Fig. 12. Similar curves using the calculated stiffness value of 1,595 kips/in. are plotted in Fig. 13. Table 13 summarizes results for all cases including the percent error in each case.

Discussion of Results. - The percent error in each case as shown in Table 13 can be directly related to the use of selected soil damping constants. This is clearly shown by referring to the J versus J' curves of Figs. 4 and 5. For example, considering PA 1-Final (greatest percent error) using a stiffness value of 350 kips/in. and soil damping constants of $J' = 0.20$ and $J = 0$, the error is +16 percent. Fig. 4(a) graphically shows the true friction damping constant to be approximately 0.30 at $J = 0$. Therefore, the pile was underdamped resulting in a lower blow count and hence a higher predicted capacity than determined by the static load test.

TABLE 13. - SUMMARY OF PREDICTED BEARING CAPACITY RESULTS FOR TEST PILES

Pile No.	Stiffness value, K_p or K_{c+p} (kips/in.)	Capacity by load test, RUT_{LT} (tons)	Capacity by wave equation RUT_{WE} (tons)	% Error $\left(\frac{RUT_{WE}-RUT_{LT}}{RUT_{LT}}\right)(100)$
PA 1-Initial	75	46.2	45	-2.6
PA 1-Final	350	100.0	116	+16.0
PA 2-Initial	178	50.1	52	+3.8
PA 2-Final	125	122.0	105	-13.9
PA 1-Initial	182	46.2	47	+1.7
PA 1-Final	182	100.0	101	+1.0
PA 2-Initial	182	50.1	53	+5.8
PA 2-Final	182	122.0	119	-2.5
CC-Initial	750	138.2	133	-3.8
CC-Final	750	161.1	175	+8.6
CC-Initial	1,595	138.2	123	-11.0
CC-Final	1,595	161.1	158	-1.9

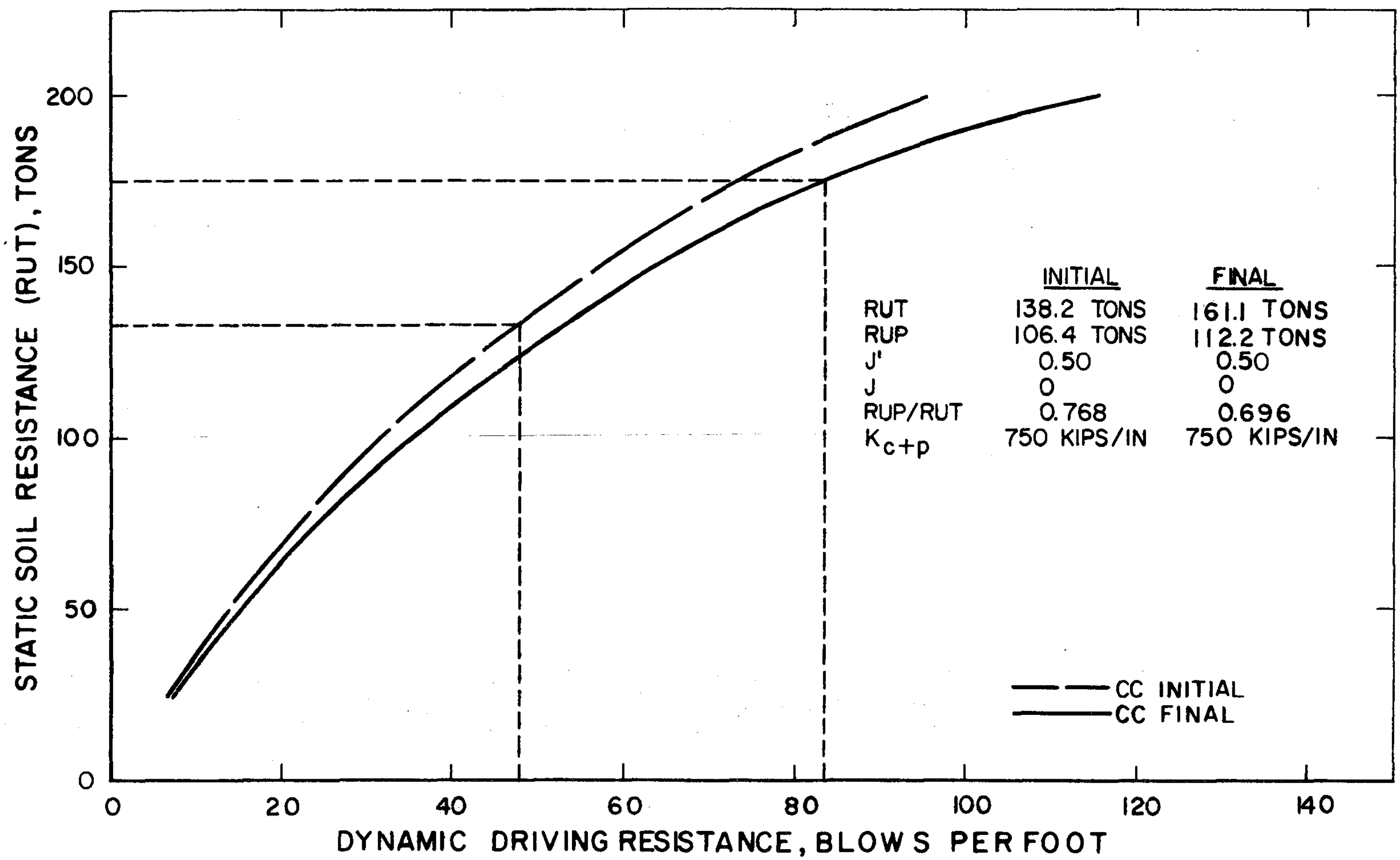

FIG. 12.—RUT vs. BLOW COUNT CURVES FOR CORPUS CHRISTI TEST PILE USING STIFFNESS VALUE FOR EXPERIMENTAL PEAK FORCE AGREEMENT

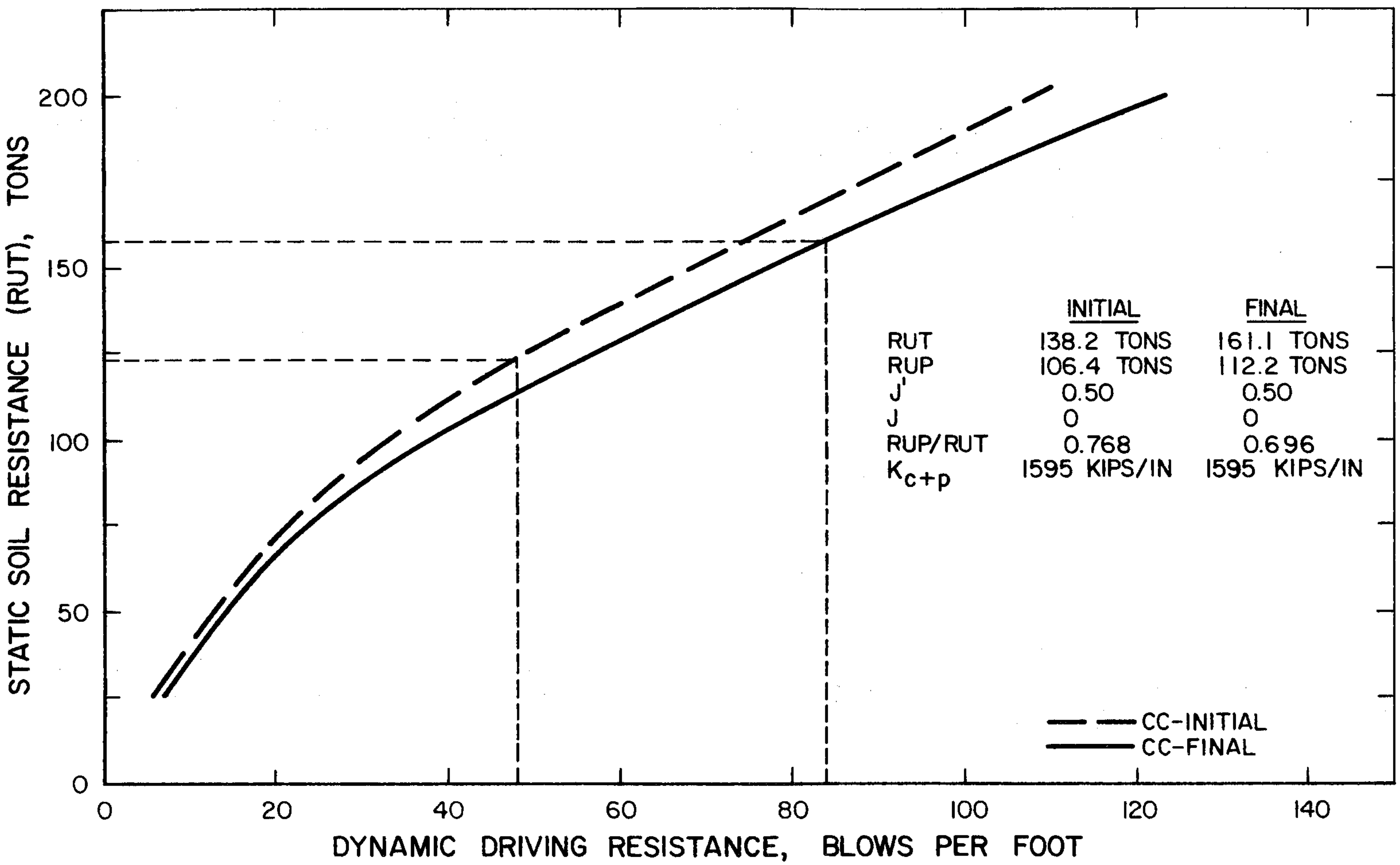

FIG. 13.— RUT vs. BLOW COUNT CURVES FOR CORPUS CHRISTI TEST PILE USING CALCULATED STIFFNESS VALUE

The same procedure of analysis holds true for all cases listed in Table 13.

The fact that three of the Port Arthur curves in Fig. 11 plot as a single curve while the fourth (PA 1-Initial) is very close to the single curve is not surprising. As previously discussed, PA 1-Initial was driven using a 3 ft adapter section of reduced contact area while in the other three cases the piles were driven using the same 1 1/2 ft adapter section. Since this adapter section basically had the same contact area in each case and the piles were driven using the same hammer with little differences in ram velocities and soil conditions, the curves should be approximately the same. The significance of this single curve concept is that the same curve generated under similar input conditions as discussed above can be used for pile bearing prediction at initial driving and at final redriving. This is true so long as the constant blow count value is used once the pile is broken loose and moving relative to the soil. Use of the initial high blow count at final redriving before the pile is broken loose and moving relative to the soil is felt to reflect too large a capacity because a much larger soil resistance is mobilized than experienced in the load test just prior to final redriving. The soil mass involved once the pile is moving under a relatively constant blow count is felt to be more nearly equal to the soil mass involved during any static load test. Thus the dynamic driving resistance at any time appears to reflect any soil "set-up" or "relaxation" as the case may be. This means that in the case of Port Arthur test piles the increase in static resistances from initial driving to final redriving are reflected in the increased dynamic driving resistances, i.e., the higher blow counts at final redriving.

It should be noted that the Corpus Christi curves in Figs. 12 and 13 do not plot as a single curve using either the established stiffness value or the calculated stiffness value. The reason for this may not be apparent at first. The only input difference for CC-Initial and CC-Final curves in Figs. 12 or 13 was the percent of RUT carried as side and point load. The difference between the RUP/RUT ratios was small but the friction damping constant, J', was sufficiently large to exert such influence on the results.

A final comment on the average blow count over the last foot of driving for CC-Initial should be made to explain the difference in percent error for CC-Initial and CC-Final shown in Table 13. The final blow count for CC-Initial over the last 0.20 ft of driving was 54 blows per foot as shown by the blow count record in Table III-5 of Appendix III. If this blow count is considered valid, then higher predicted bearing capacities result for CC-Initial which are more consistent with CC-Final. This can be verified by entering CC-Initial curves in Figs. 12 and 13 at the higher blow count of

54 blows per foot to obtain new values of RUT of 148 tons and 130 tons, respectively.

VALIDITY OF MATERIAL CONSTANTS FOR PILE DRIVING COMPONENTS ABOVE THE PILE HEAD

Throughout this research study the material constants of the pile driving components used were those values recommended by Lowery et al. (22). They include such constants as coefficients of restitution for cushion and capblock materials and steel on steel impact, hammer efficiencies, rated hammer energies and effective ram strokes for determining input ram velocities, secant moduli of elasticity for calculating stiffnesses of cushions and capblocks, explosive diesel forces, ram stiffnesses, anvil weights and pile helmet weights.

The only material constant used differently was the coefficient of restitution of the oak capblock used for initial driving and final redriving of the Corpus Christi test pile. The wood grain was vertical and the coefficient of restitution was assumed to be 0.80 based on a secant modulus of elasticity of 700,000 psi determined by laboratory tests. Since the recommended value of 0.80 for such stiff material as micarta was reported by Lowery et al. (22) as being too high, the value of 0.80 might have also been too high and possibly one cause for the high dynamic peak forces during initial driving and final redriving using the calculated stiffness value of 1,595 kips/in.

Considering the previously justified first pile segment stiffness changes necessary for agreement of dynamic force-time data and pile bearing predictions for Port Arthur test piles, the material constants used appear to be valid producing excellent results for the limited number of cases investigated.

While it was not feasible to fully evaluate the validity of individual material constants integrated into the hammer-pile-soil system due to a lack of experimental data pertaining to each one, the results of force-time data correlation and pile bearing predictions tend to support their validity.

ADDITIONAL CASE STUDIES

General. - In order to verify the use of the soil damping constants selected for the Port Arthur and Corpus Christi test piles and to determine by correlation usable soil damping constants for other type soils, thirty-seven additional load test piles (LTP) were analyzed by the wave equation. Records of eight of the piles were taken from the Arkansas River Navigation Project (10), seventeen from the Michigan Report (25) and twelve from the Texas Gulf Coast (1,23). All but six of the load test piles were analyzed in an earlier report by Lowery et al. (22) using J = 0.30 for clays, 0.10 for sands, and weighted values for composite soils, always setting J' = J/3. Additional information concerning the test piles may be found in References 1, 10, 23 and 25. Information on five of the Texas Gulf Coast piles (Houston LTP 1, 2, 3, 4 and Padre Island LTP 22) may be obtained by contacting the Bridge Division of the Texas State Highway Department in Austin, Texas.

The additional case studies included steel pipe piles (closed and open end), prestressed concrete piles (constant area and tapered), fluted tapered steel pipes and H piles. A good variety of hammers and pile driving accessories was represented as well as variable soil conditions and load distributions.

The piles were separated into three basic groups for analysis; those supported entirely by cohesionless soils (sands and silts), those supported entirely by cohesive soils (clays), and those supported by composite soils (sands, silts and clays). With the exceptions of Belleville LTP 1 and Detroit LTP 1 which were load tested within a few hours after driving, all load test piles were statically load tested from 5 to 78 days after driving. Since the piles were load tested by a variety of testing methods, the selection of ultimate load test values was somewhat difficult. The problem of selecting ultimate load test values coupled with the problem of estimating soil "set-up" in clays appreciably influences any correlation of predicted bearing capacities at time of driving.

Based on the results of soil "set-up" for Port Arthur test piles and Belleville LTP 1 and Detroit LTP 1 of the Michigan report, all friction loads in clay at time of load test were reduced by a factor of 2 to obtain the frictional load at time of driving. Exceptions were Houston LTP 1, 2, 3, 4 and Padre Island LTP 22 which were load tested at 5 days. A factor of 1.5 was applied to frictional side loads in clays for these cases assuming that one half of the increased strength due to soil "set-up" had taken place. No "set-up" factor was applied to silts, sands and soft semi-organic sedimentary deposits (soft peat). The point loads of all piles were held constants at time of load test and time of driving.

Since the Quick-Load Test Method generally terminated at less than 1-in. of total settlement at the pile head, this value was arbitrarily applied to all load test piles as a limiting value for selection of ultimate load test values at time of load test. Where a load test terminated prior to 1-in. of settlement, the greatest applied load was used as the ultimate load test value.

Using the load distributions obtained from strain gages or static analyses as listed in Tables 14, 15 and 17, the soil "set-up" factors for frictional loads in clays were applied to reduce soil resistances at time of load test to soil resistances at time of driving. The load distributions at time of driving are listed in Tables 14, 16 and 18. These load distributions were used as input for wave equation analyses. A complete summary of input data used for the wave equation analysis and the resulting RUT versus blow count curve for each load test pile are presented in Appendix IV. Table 19 summarizes the ultimate load test value (RUT_{LT}) for each pile, the amount of settlement at the pile head at the ultimate load test value, the time between driving and load testing and the ultimate resistance at time of driving (RUT_{DR}).

Additional soil damping constants were obtained by trial and error correlations of wave equation predicted capacities (RUT_{WE}) with ultimate resistances at time of driving (RUT_{DR}). A summary of final soil damping constants used for all piles based on soil type is given in Table 20.

TABLE 14. - LOAD DISTRIBUTION AT TIME OF LOAD TEST AND TIME OF DRIVING FOR PILES SUPPORTED BY COHESIONLESS SOILS

Location	Load Test Pile	Side Resistance: % RUT Sand / 100	Side Resistance: % RUT Silt / 100	Point Resistance: % RUT (type) / 100
Arkansas	1[a]	0.73	0.00	0.27 (sand)
	2[a]	0.70	0.00	0.30 (sand)
	3[a]	0.61	0.00	0.39 (sand)
	4[b]	0.44	0.00	0.56 (sand)
	5[b]	0.50	0.00	0.50 (sand)
	6[a]	0.77	0.00	0.23 (sand)
	7[a]	0.77	0.00	0.23 (sand)
	16[a]	0.70	0.00	0.30 (sand)
Copano Bay	103[b]	0.20	0.20	0.60 (sand & silt)
Muskegon	2[b]	0.85	0.00	0.15 (sand)
	3[b]	0.92	0.00	0.08 (sand)
	4[b]	0.98	0.00	0.02 (sand)
	6[b]	0.75	0.00	0.25 (sand)
	9[b]	0.75	0.00	0.25 (sand)

[a]Distribution determined from strain gages (Reference 10).

[b]Distribution determined from static analysis (References 10,23,25).

TABLE 15. - LOAD DISTRIBUTION AT TIME OF LOAD TEST
FOR PILES SUPPORTED BY COHESIVE SOILS

Location	Load Test Pile	Side Resistance $\frac{\% \text{ RUT Clay}}{100}$	Side Resistance $\frac{\% \text{ RUT Hardpan}}{100}$	Point Resistance $\frac{\% \text{ RUT (type)}}{100}$
Belleville	1[a]	1.00	0.00	0.00 (clay)
Detroit	1[a]	1.00	0.00	0.00 (clay)
	2[a]	0.15	0.00	0.85 (hardpan)
	7[a]	0.26	0.09	0.65 (hardpan)
	8[a]	0.58	0.18	0.24 (hardpan)
	10[a]	0.14	0.05	0.81 (hardpan)
Beaumont	53[b]	0.85	0.00	0.15 (clay)

[a]Distribution determined from static analysis (Reference 25).
[b]Distribution determined from strain gages (Reference 1).

TABLE 16. - LOAD DISTRIBUTION AT TIME OF DRIVING
FOR PILES SUPPORTED BY COHESIVE SOILS

Location	Load Test Pile	Side Resistance $\frac{\% \text{ RUT Clay}}{100}$	Side Resistance $\frac{\% \text{ RUT Hardpan}}{100}$	Point Resistance $\frac{\% \text{ RUT (type)}}{100}$
Belleville	1[a]	1.00	0.00	0.00 (clay)
Detroit	1[a]	1.00	0.00	0.00 (clay)
	2[a]	0.08	0.00	0.92 (hardpan)
	7[a]	0.16	0.05	0.79 (hardpan)
	8[a]	0.47	0.15	0.38 (hardpan)
	10[a]	0.08	0.03	0.89 (hardpan)
Beaumont	53[b]	0.74	0.00	0.26 (clay)

[a]Distribution determined from static analysis (Reference 25).
[b]Distribution determined from strain gages (Reference 1).

TABLE 17. - LOAD DISTRIBUTION AT TIME OF LOAD TEST

FOR PILES SUPPORTED BY COMPOSITE SOILS

Location	Load Test Pile	Side Resistance			Point Resistance
		% RUT Clay / 100	% RUT Sand / 100	% RUT Silt / 100	% RUT (type) / 100
Victoria	35[a]	0.26	0.14	0.00	0.60 (sand)
	40[a]	0.30	0.26	0.00	0.44 (sand)
	45[a]	0.30	0.03	0.03	0.64 (sand & silt)
Chocolate Bayou	40[a]	0.72	0.06	0.05	0.16 (sand)
Houston	1[b]	0.30	0.67	0.00	0.03 (clay)
	2[b]	0.66	0.27	0.00	0.07 (sand)
	3[b]	0.11	0.84	0.00	0.05 (sand)
	4[b]	0.80	0.13	0.00	0.07 (sand)
	30[a]	0.20	0.08	0.10	0.62 (clay& sand)
Belleville	3[b]	0.20	0.14	0.14	0.52 (hardpan)
	4[b]	0.11	0.10	0.10	0.69 (hardpan)
	5[b]	0.11	0.10	0.10	0.69 (hardpan)
	6[b]	0.61	0.17	0.17	0.05 (sand& silt)
Muskegon	7[b]	0.12	0.44	0.00	0.44 (sand)
	8[b]	0.12	0.44	0.00	0.44 (sand)
Padre Island	22[b]	0.68	0.30	0.00	0.02 (clay)

[a]Distribution determined from strain gages (Reference 23).

[b]Distribution determined from static analysis (Reference 25).

TABLE 18. - LOAD DISTRIBUTION AT TIME OF DRIVING
FOR PILES SUPPORTED BY COMPOSITE SOILS

Location	Load Test Pile	Side Resistance			Point Resistance
		% RUT Clay / 100	% RUT Sand / 100	% RUT Silt / 100	% RUT (type) / 100
Victoria	35[a]	0.15	0.16	0.00	0.69 (sand)
	40[a]	0.17	0.30	0.00	0.53 (sand)
	45[a]	0.18	0.03	0.03	0.76 (sand & silt)
Chocolate Bayou	40[a]	0.57	0.09	0.08	0.26 (sand)
Houston	1[b]	0.18	0.79	0.00	0.03 (clay)
	2[b]	0.50	0.41	0.00	0.09 (sand)
	3[b]	0.06	0.88	0.00	0.06 (sand)
	4[b]	0.68	0.21	0.00	0.11 (sand)
	30[a]	0.11	0.09	0.11	0.69 (clay & silt)
Belleville	3[b]	0.11	0.16	0.16	0.57 (hardpan)
	4[b]	0.05	0.11	0.11	0.73 (hardpan)
	5[b]	0.05	0.11	0.11	0.73 (hardpan)
	6[b]	0.44	0.25	0.25	0.06 (sand & silt)
Muskegon	7[b]	0.08	0.46	0.00	0.46 (sand)
	8[b]	0.08	0.46	0.00	0.46 (sand)
Padre Island	22[b]	0.51	0.46	0.00	0.03 (clay)

[a]Distribution determined from strain gages (Reference 23).
[b]Distribution determined from static analysis (Reference 25).

TABLE 19. - SUMMARY OF LOAD TEST INFORMATION

Location	Load Test Pile	Time Between Driving and Load Test (days)	Settlement at RUT_{LT} (in.)	RUT_{LT} (kips)	RUT_{DR} (kips)
Victoria	35	8	0.94	256	223
	40	34	0.20(plunge)	190	162
	45	45	0.50	400	340
Chocolate Bayou	40	19	0.19	238	152
Houston	1	5	0.94	300	270
	2	5	0.98	208	161
	3	5	0.57	380	366
	4	5	0.46	230	170
	30	14	0.26(plunge)	360	324
Copano Bay	103	78	0.65	356	356
Beaumont	53	13	0.42	240	138
Padre Island	22	5	0.26	494	382
Arkansas	1	30	1.00	336	336
	2	32	1.00	446	446
	3	34	1.00	494	494
	4	30	0.73	396	396
	5	24	1.00	556	556
	6	24	0.94	350	350
	7	24	0.80	440	440
	16	23	0.92	336	336
Belleville	1	0	1.00	100	100
	3	50	1.00	360	324
	4	50	1.00	700	661
	5	57	1.00	700	661
	6	51	1.00	420	292
Detroit	1	0	1.00	40	40
	2	8	1.00	300	277
	7	22	1.00	330	272
	8	14	1.00	340	211
	10	23	1.00	400	366
Muskegon	2	26	1.00	200	200
	3	26	1.00	120	120
	4	23	1.00	94	94
	6	18	1.00	480	480
	7	25	1.00	570	536
	8	19	1.00	534	502
	9	25	1.00	484	484

TABLE 20. - SUMMARY OF SOIL DAMPING CONSTANTS

Type Soil	J' (sec/ft)	J (sec/ft)
Clay (CL, CH & Hardpan)	0.20	0
Partially Saturated Sands & Silts	0.05	0
Saturated Sands & Silts	0.50	0
Soft Peat	0.00	0

Detailed results presented in Appendix IV have been summarized in Tables 21, 22 and 23 for each pile group showing the percent error in each case based on the soil resistance at time of driving.

Discussion of Results. - The overall correlation was quite good considering the problems of selecting ultimate load test values and estimating soil "set-up" in clays. The two largest errors were associated with Belleville LTP 1 and Detroit LTP 1 as shown in Table 22. These piles should have shown excellent correlation since soil "set-up" was a known factor. For both of these open end steel pipe piles damping was applied to the inside clay cores as well as to the sides of the piles and predicted values were still much too high. While no definite causes could be found to account for the discrepancies, a heavy load cell used in driving all of the Michigan piles might well have affected results. Both piles carried considerably smaller loads at time of driving than any other

TABLE 21. - SUMMARY OF RESULTS FOR WAVE EQUATION ANALYSES OF PILES SUPPORTED BY COHESIONLESS SOILS

Location	Load Test Pile	RUT_{DR} (Resistance at Time of Driving) (kips)	RUT_{WE} (Predicted Resistance by Wave Equation) (kips)	% Error in Terms of RUT_{DR} $\left(\frac{RUT_{WE}-RUT_{DR}}{RUT_{DR}}\right)(100)$
Arkansas	1	336	310	-8
	2	446	530	+19
	3	494	590	+19
	4	396	340	-14
	5	556	584	+5
	6	350	260	-26
	7	440	430	-2
	16[a]	336	400	+19
Copano Bay	103	356	336	-6
Muskegon	2	200	210	+5
	3	120	170	+42
	4[b]	94	124	+32
	6	480	320	-33
	9	484	334	-31

[a] Jetted 40 ft.

[b] Internally jetted.

TABLE 22. - SUMMARY OF RESULTS FOR WAVE EQUATION ANALYSES OF PILES SUPPORTED BY COHESIVE SOILS

Location	Load Test Pile	RUT_{DR} (Resistance at Time of Driving) (kips)	RUT_{WE} (Predicted Resistance by Wave Equation) (kips)	% Error in Terms of RUT_{DR} $\left(\frac{RUT_{WE}-RUT_{DR}}{RUT_{DR}}\right)(100)$ $\bar{A}$	$\bar{B}$
Belleville	1[a]	100	176	+76	omit
	1[b]	100	92	omit	-8
Detroit	1[a]	40	64	+60	omit
	1[b]	40	50	omit	+25
	2	277	230	-17	-17
	7	272	250	-8	-8
	8	211	246	+16	+16
	10	366	320	-13	-13
Beaumont	53	138	150	+9	+9

[a]The values for these piles were questionable.

[b]Reducing the first pile segment stiffness by the same percent as the average percent reduction for Port Arthur test piles.

TABLE 23. - SUMMARY OF RESULTS FOR WAVE EQUATION ANALYSES OF PILES SUPPORTED BY COMPOSITE SOILS

Location	Load Test Pile	RUT_{DR} (Resistance at Time of Driving) (kips)	RUT_{WE} (Predicted Resistance by Wave Equation) (kips)	% Error in Terms of RUT_{DR} $\left(\frac{RUT_{WE}-RUT_{DR}}{RUT_{DR}}\right)(100)$
Victoria	35	223	206	-8
	40	162	170	+5
	45	340	450	+32
Chocolate Bayou	40	152	150	-1
Houston	1	270	290	+7
	2	161	120	-26
	3	366	430	+17
	4	170	210	+23
	30	324	340	+5
Belleville	3	324	372	+15
	4	661	396	-40
	5	661	362	-45
	6	292	290	-1
Muskegon	7	536	420	-22
	8	502	460	-8
Padre Island	22	382	390	+2

pile of the Michigan report driven in clay. It is interesting to note that when the first pile segment stiffness (K_p) was reduced by the same percent as the average percent reduction for Port Arthur test piles, both piles yielded reasonable results as shown in Table 22 and Figs. IV-9 and IV-14 of Appendix IV.

The two jetted piles of Table 21 resulted in positive percent errors as expected. The longer Muskegon piles (LTP 6, 7, 8 and 9) were driven thru approximately 50 ft of soft peat assumed to carry no frictional load. It should be noted from Tables 21 and 23 that all four piles yielded low predicted values indicating the possibility of some load carrying capacity and soil "set-up" in the peat layer. Considering all 37 piles, 41 percent were within ±10% error, 65 percent within ±20% error and 87 percent within ±35% error in terms of RUT_{DR}. These results show only a small improvement over the results reported by Lowery et al. (22) using 31 of the 37 piles. However, it is felt that the results of this study are more valid for the following reasons:

1. The soil damping constants for clays and saturated sands were obtained using the instrumented test piles and measured field data of this study;
2. A consistent method for selection of ultimate load test values for the 37 piles was employed;
3. A consistent method for estimating soil "set-up" in

clays was used to reduce ultimate load test values to ultimate soil resistances at time of driving; and

4. The additional soil damping constants of Table 20 were obtained by trial and error correlation using several piles to isolate the soil type under consideration.

CONCLUSIONS

The objectives of this research were to develop a procedure for obtaining soil damping constants to be used in wave equation prediction of pile bearing capacity at time of driving and after soil "set-up" has taken place and to determine the validity of material constants for pile driving components above the pile head. It is felt that both of these objectives were accomplished.

The wave equation analyses of Port Arthur and Corpus Christi test piles show the importance of reliable input data, especially as related to driving conditions and cushion or first pile segment stiffness values. The validity of wave equation analyses was shown thru correlation of computed and experimental force-time data and comparisons of predicted pile bearing capacities with load test results.

The following conclusions are made based on the results of this study:

1. Soil damping values are not constant but the use of soil damping constants for various type soils appears to be justified.
2. Soil damping constants at initial driving and final redriving are the same. Thus any increased pilé capacity due to soil "set-up" is reflected at final

redriving by an increased blow count. This should also hold true in reverse, i.e., decreased pile capacity due to soil "relaxation" should be reflected in a reduced blow count at final redriving.

3. A single curve can be used for pile bearing prediction at initial driving and final redriving so long as input values including RUP/RUT ratios do not change appreciably.
4. The pile-soil model proposed by Smith is valid based on correlation of wave equation results using measured force-time input data with experimental data.
5. The material constants for hammers, cushions and piles recommended by Lowery et al. (22) give reasonably good results and can be used in wave equation analyses.
6. Soil damping constants of J = 0 and J' = 0.20 should be used for friction piles in clay.
7. Soil damping constants of J = 0 and J' = 0.50 should be used for piles in saturated sand and silt.
8. The soil damping constants listed in Table 20 for additional case studies should be used until the soil damping constants are verified or refined based on results of instrumented piles.
9. The most critical input parameters for wave equation analyses are ram velocity, cushion or first pile

segment stiffness, load distribution, and soil damping constants.

10. The cushion stiffness or first pile segment stiffness controls the magnitude of the dynamic peak force at the pile head when the ram velocity and soil resistance are held constant.

11. The soil damping constants control the magnitude of dynamic peak forces along the pile when the cushion stiffness or first pile segment stiffness, the ram velocity and soil resistance are held constant.

12. Reliable pile bearing capacity using a wave equation analysis is insured if the experimental and computed peak force at the pile head agree. Matching the peak force at the pile head compensates for any input inaccuracies above the pile head. Since Q and J are held constant, the only variable soil parameters are load distribution and friction damping, J'.

RECOMMENDATIONS

Based on the results of this study the following recommendations are offered:

1. Sufficient instrumented test piles should be driven and load tested in other type soils using the procedure outlined in this study to obtain soil damping constants. This should be accomplished for at least the major classification of soils by the Unified Soils Classification System.
2. Specifications for standardizing cushion materials and their condition should be employed where practical to reduce unknown cushion properties.
3. A ring of micarta or other very stiff material should be used above the pile head for steel piles driven with no cushion to insure full contact area while transmitting dynamic peak forces.
4. Load tests should be conducted as close as possible to the time of initial driving or final redriving to eliminate the variable of soil "set-up" in clays.
5. A simple device for measuring the dynamic peak force at the pile head during initial driving or final redriving should be developed and incorporated into the hammer-pile-soil system. As discussed in the

conclusions, this procedure would eliminate all variables but J' and the load distribution on the pile.

6. Methods of static analyses should be investigated to develop a procedure for determining pile load distributions that produce consistent results in predicting pile bearing capacities by wave equation analyses.

7. Penetrometer test values should be investigated to determine if they can be effectively related to pile load distributions for wave equation analyses.

APPENDIX I. - REFERENCES

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APPENDIX II. - FORCE-TIME CURVES FOR INSTRUMENTED TEST PILES

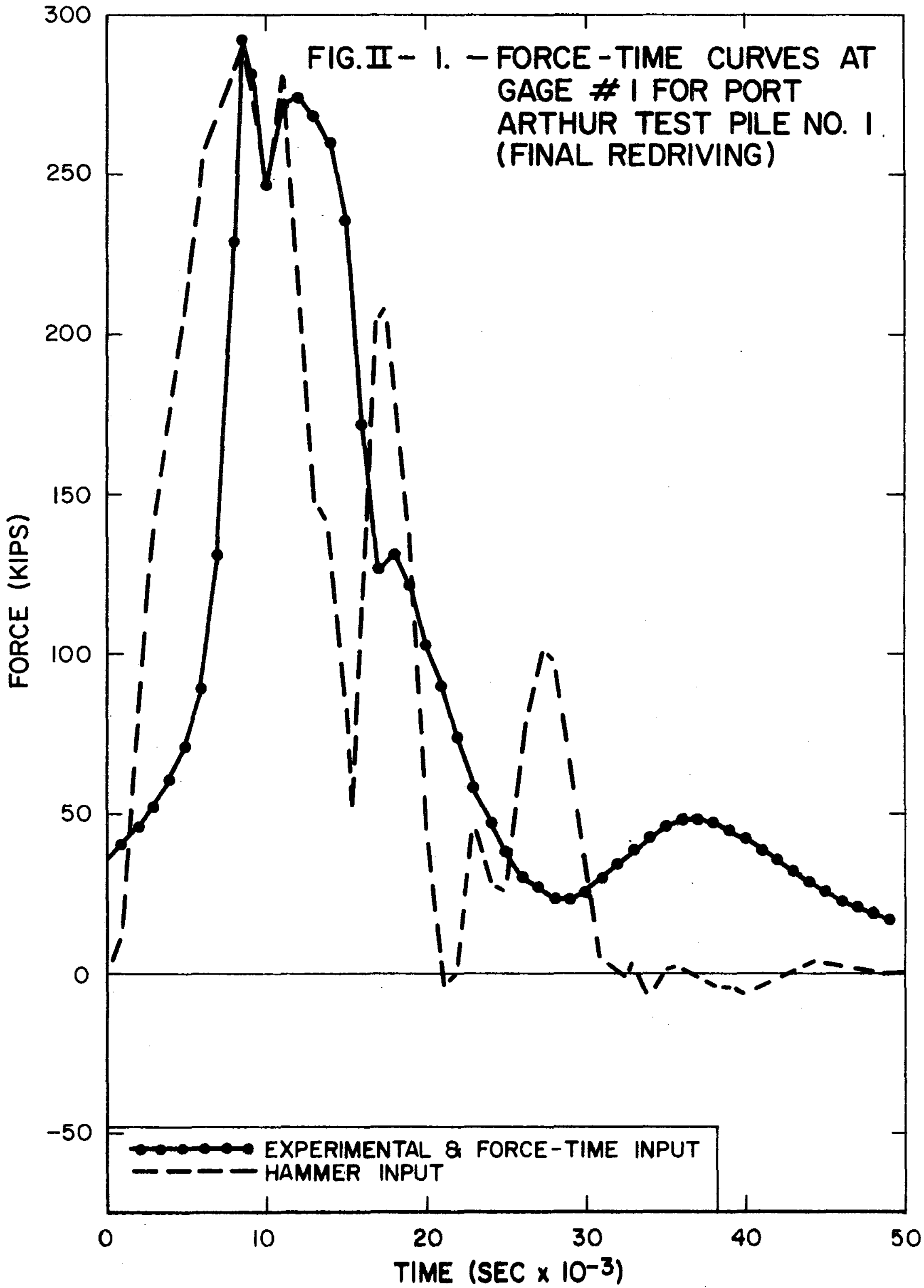

FIG. II - 1. — FORCE-TIME CURVES AT GAGE # 1 FOR PORT ARTHUR TEST PILE NO. 1 (FINAL REDRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
0
10
20
30
40
50
TIME (SEC x 10^-3)
EXPERIMENTAL & FORCE-TIME INPUT
HAMMER INPUT

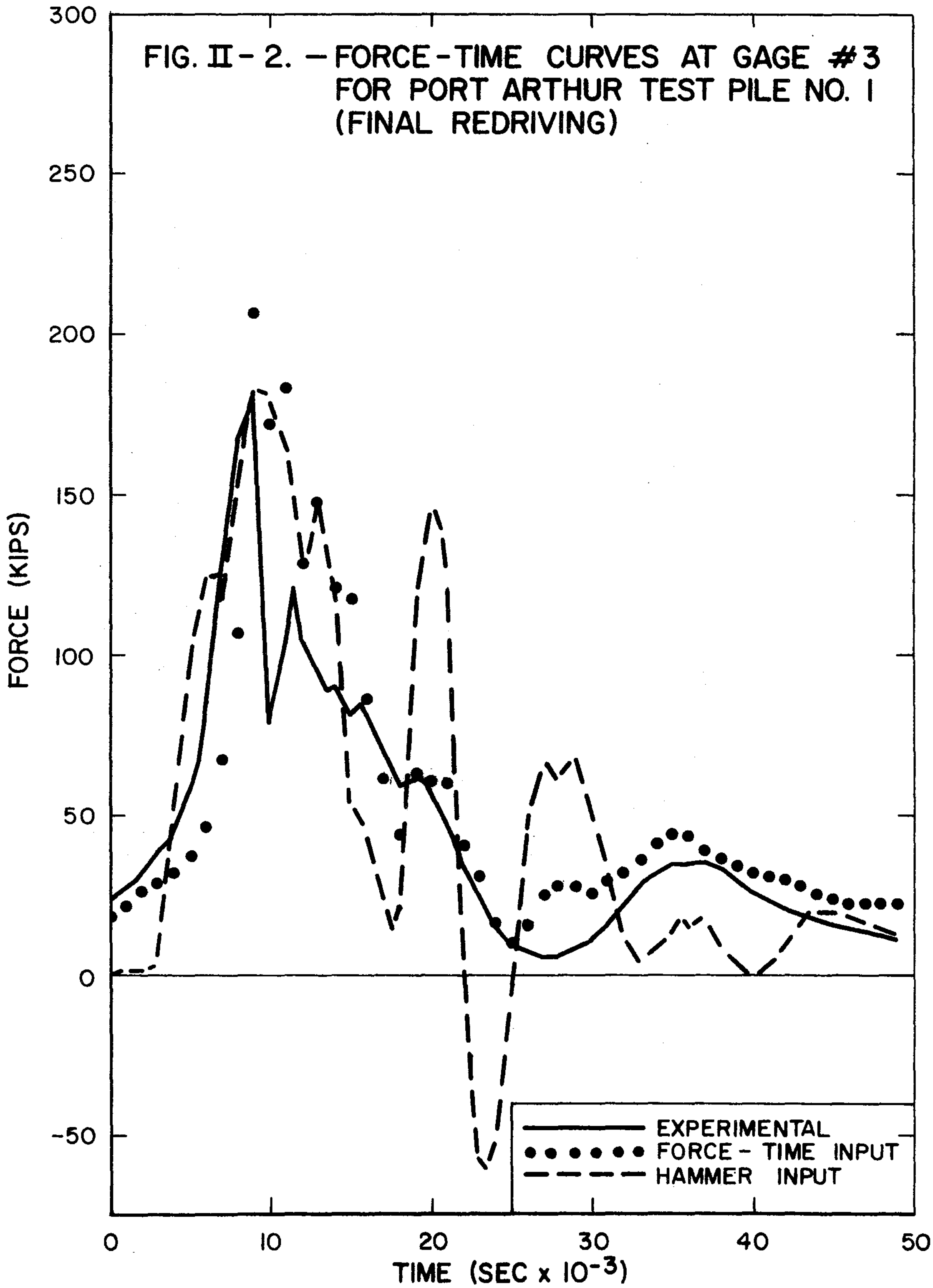
FIG. II - 2. — FORCE-TIME CURVES AT GAGE #3
FOR PORT ARTHUR TEST PILE NO. 1
(FINAL REDRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
0
10
20
30
40
50
TIME (SEC x 10-3)
EXPERIMENTAL
FORCE - TIME INPUT
HAMMER INPUT

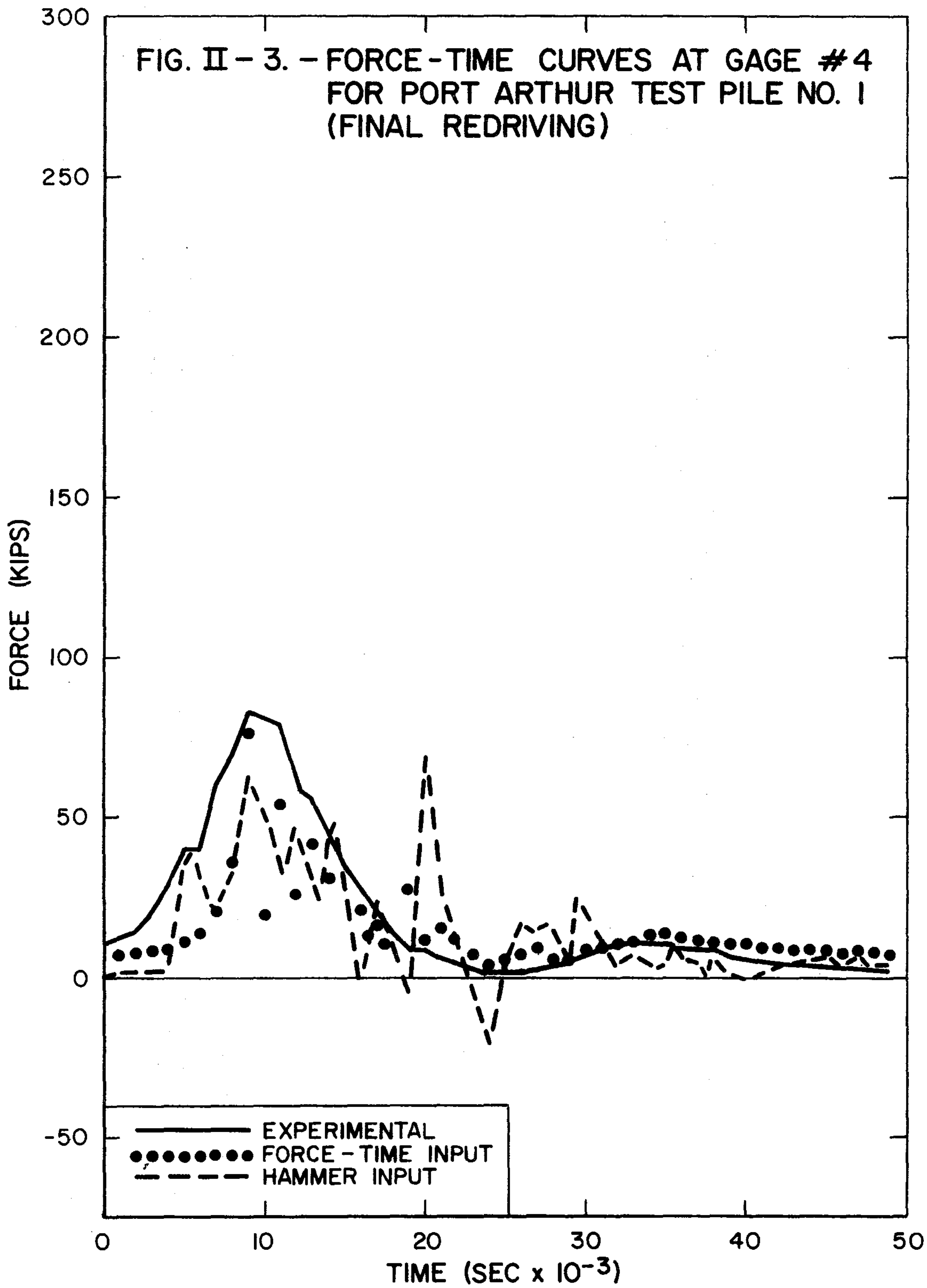
FIG. II - 3. - FORCE-TIME CURVES AT GAGE #4
FOR PORT ARTHUR TEST PILE NO. 1
(FINAL REDRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
EXPERIMENTAL
FORCE - TIME INPUT
HAMMER INPUT
0
10
20
30
40
50
TIME (SEC x 10^-3)

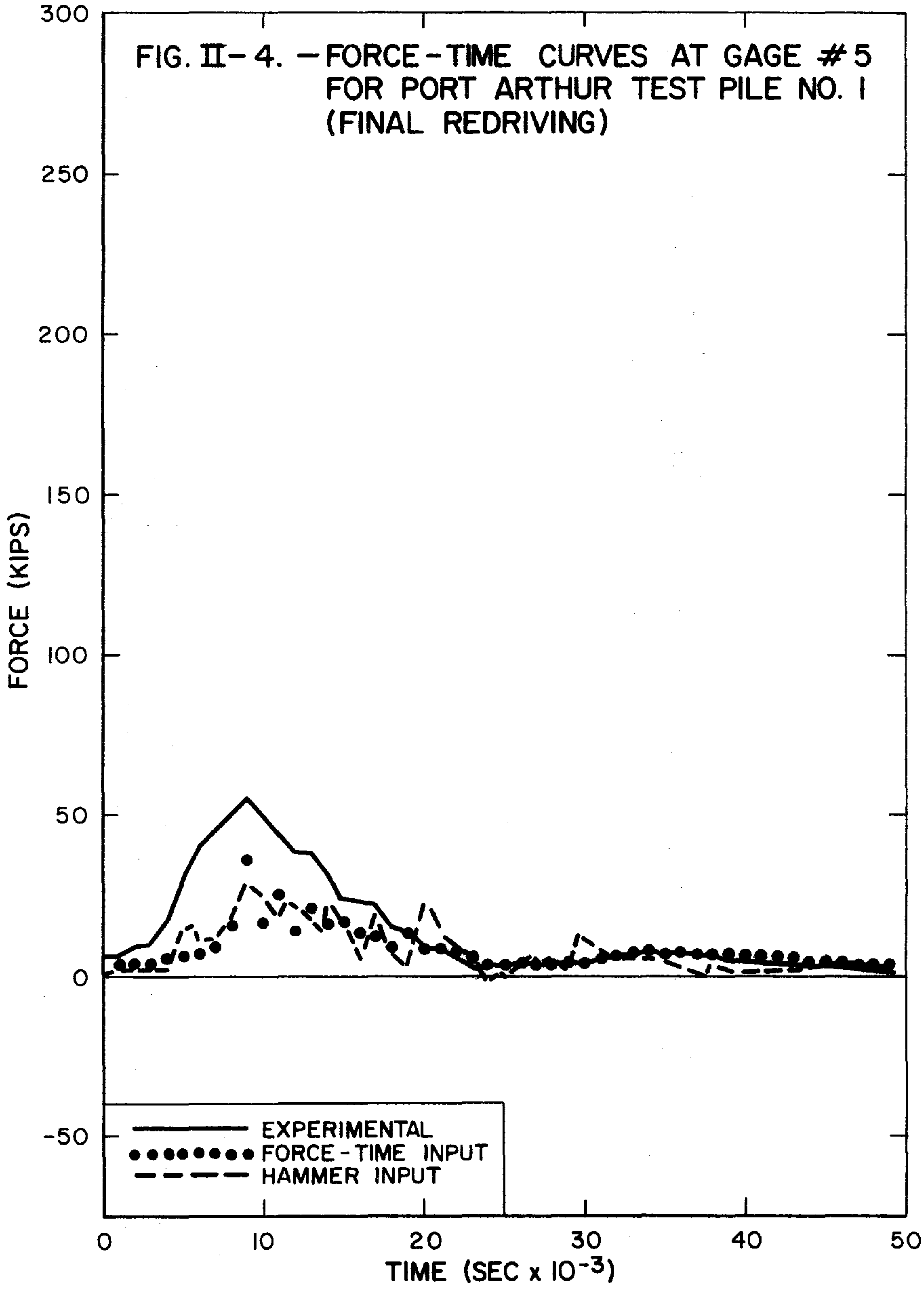
FIG. II-4. — FORCE-TIME CURVES AT GAGE #5
FOR PORT ARTHUR TEST PILE NO. I
(FINAL REDRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT
0
10
20
30
40
50
TIME (SEC x 10^{-3})

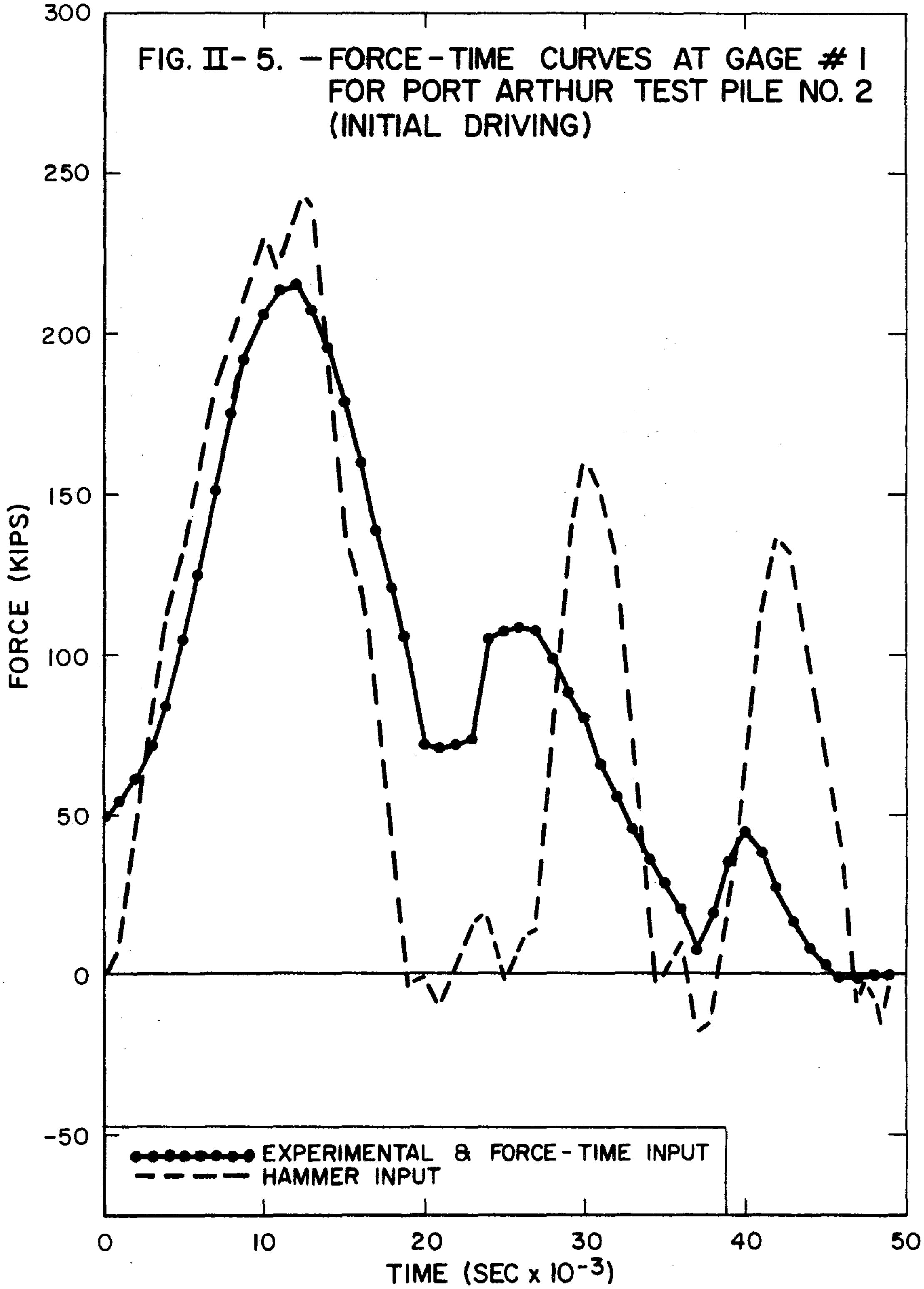

FIG. II-5. — FORCE-TIME CURVES AT GAGE # 1 FOR PORT ARTHUR TEST PILE NO. 2 (INITIAL DRIVING)

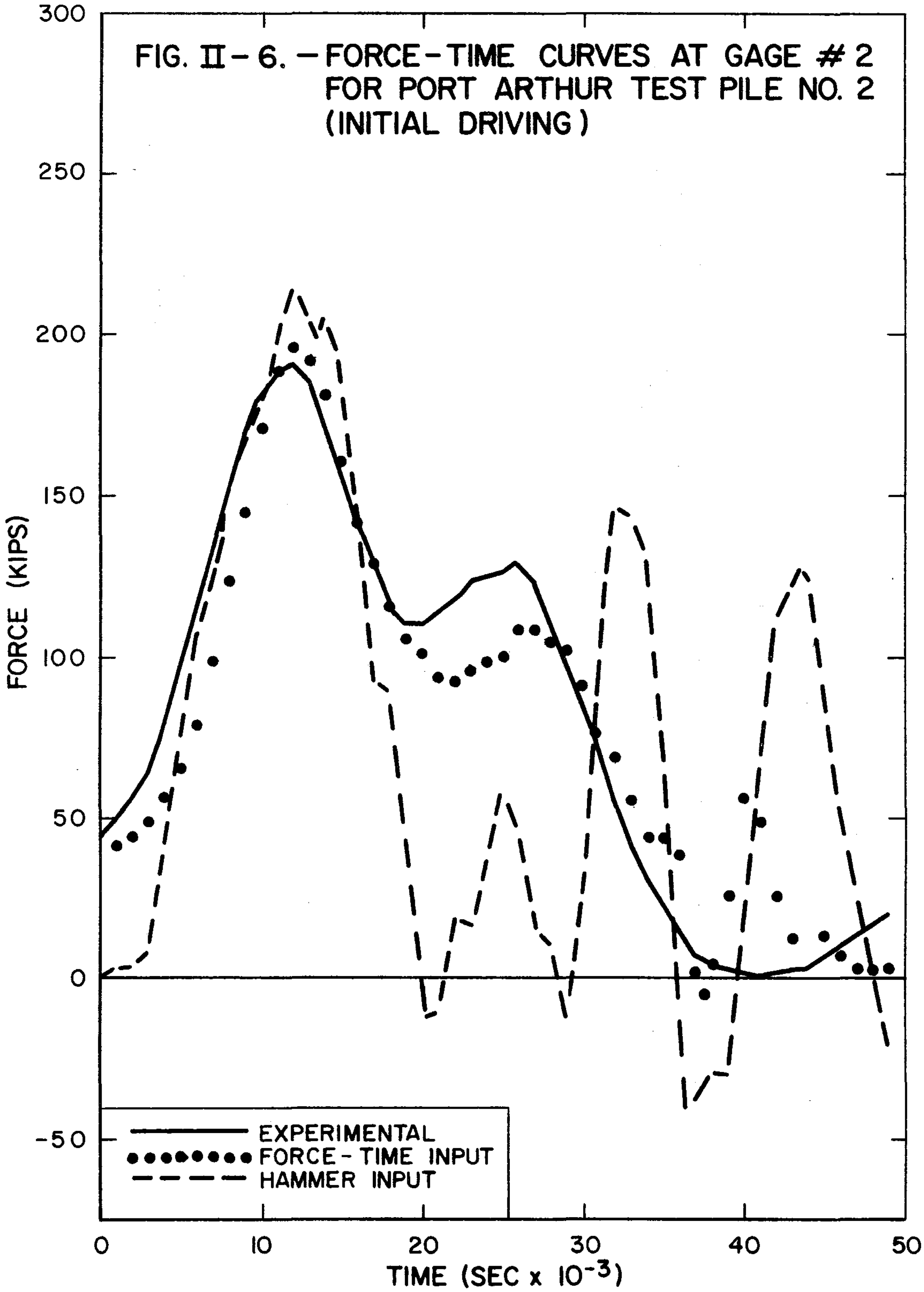
FIG. II-6. — FORCE-TIME CURVES AT GAGE # 2
FOR PORT ARTHUR TEST PILE NO. 2
(INITIAL DRIVING)
FORCE (KIPS)
300
250
200
150
100
50
0
-50
TIME (SEC x 10⁻³)
0
10
20
30
40
50
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

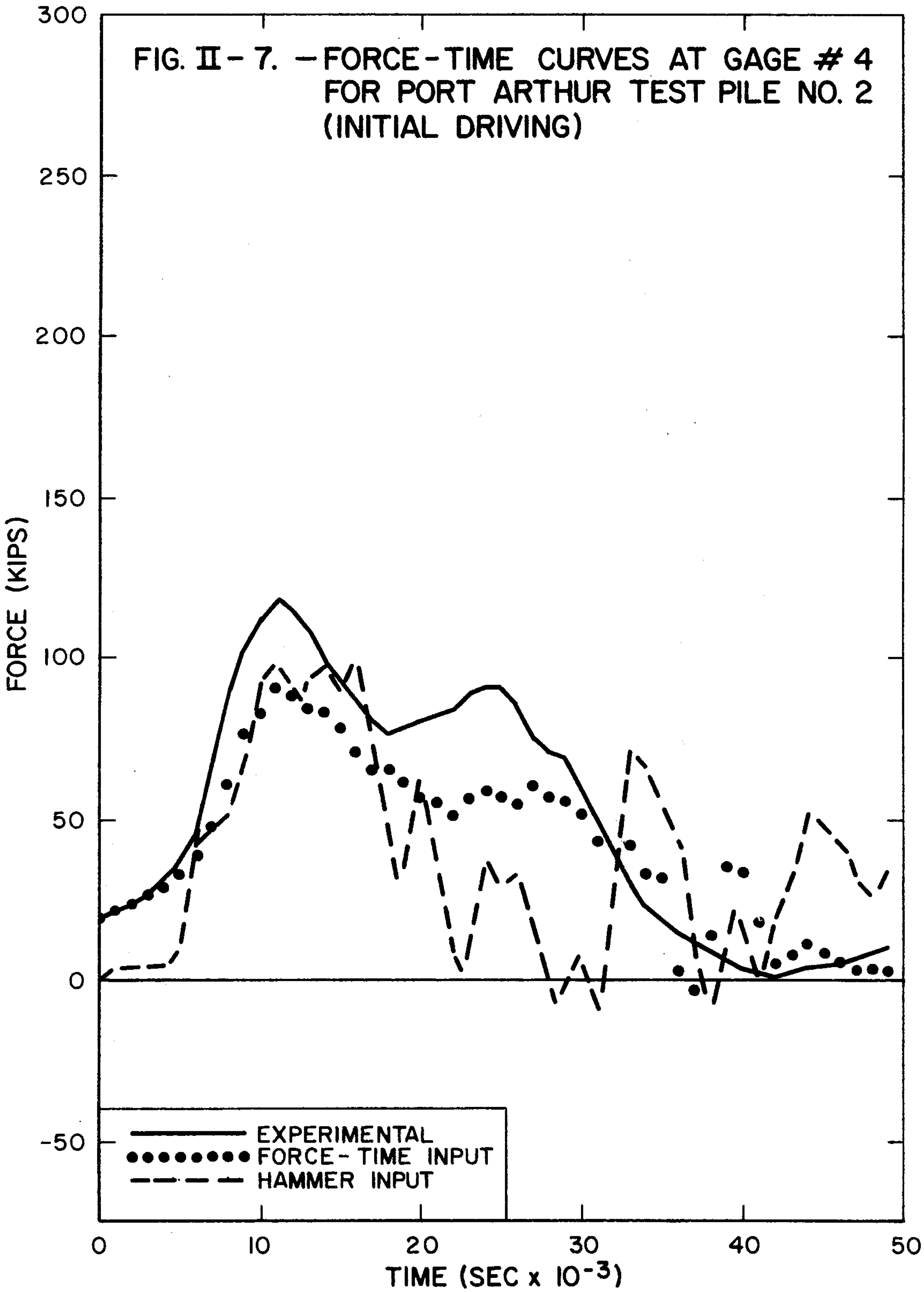
FIG. II - 7. — FORCE-TIME CURVES AT GAGE # 4
FOR PORT ARTHUR TEST PILE NO. 2
(INITIAL DRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
0
10
20
30
40
50
TIME (SEC x 10^{-3})
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

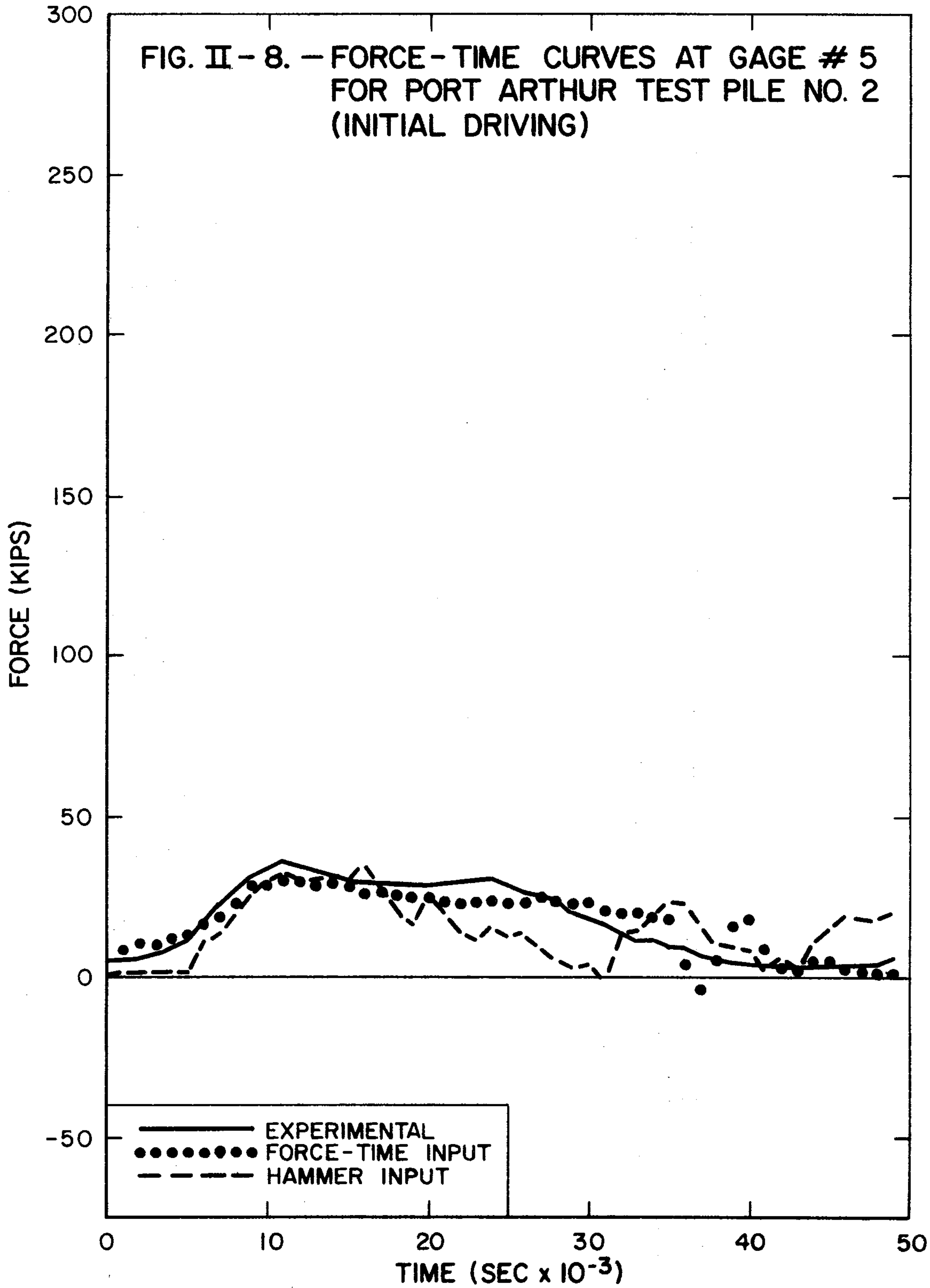
FIG. II - 8. — FORCE-TIME CURVES AT GAGE # 5
FOR PORT ARTHUR TEST PILE NO. 2
(INITIAL DRIVING)
FORCE (KIPS)
300
250
200
150
100
50
0
-50
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT
0
10
20
30
40
50
TIME (SEC x 10^{-3})

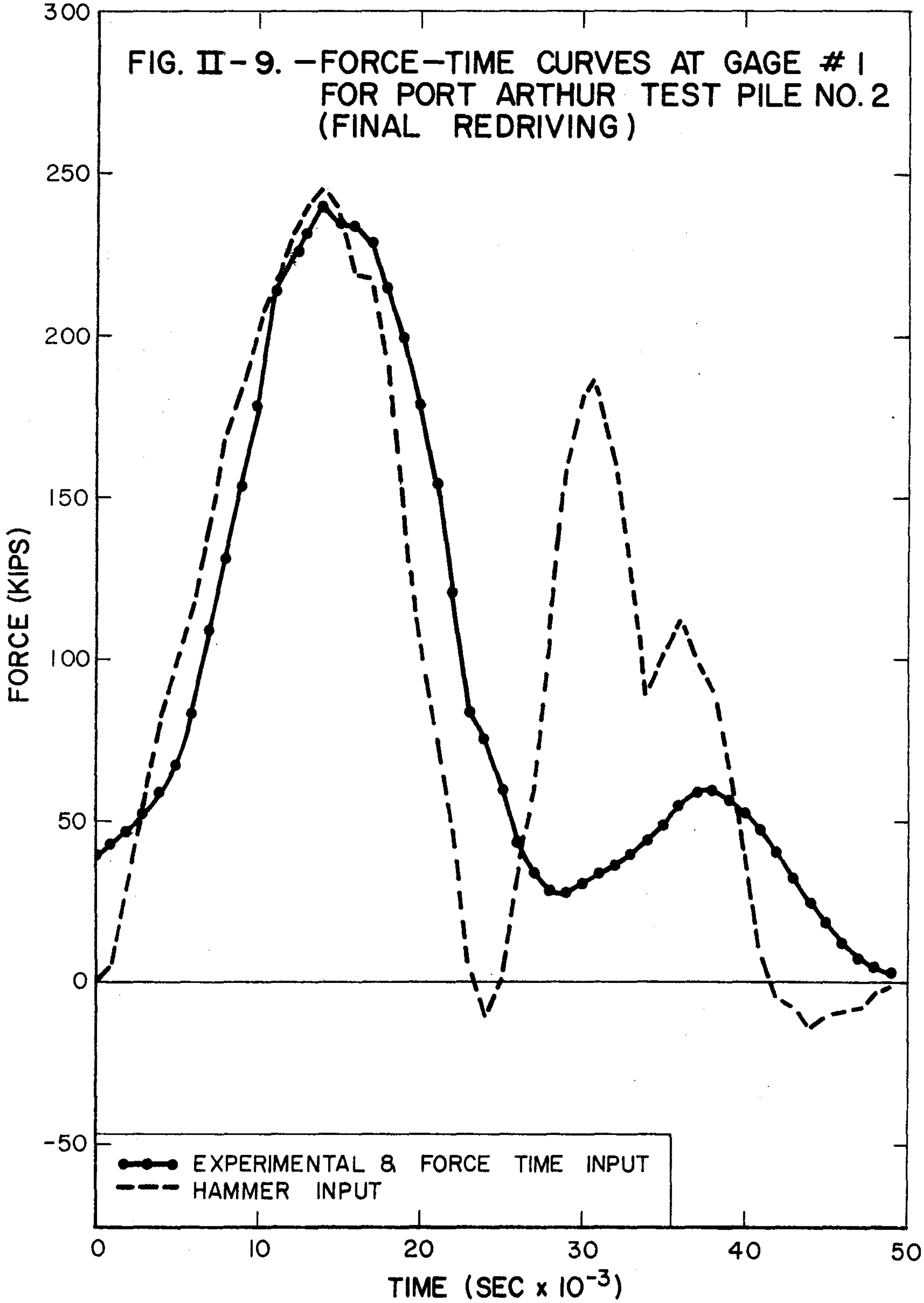
FIG. II-9. —FORCE—TIME CURVES AT GAGE # 1 FOR PORT ARTHUR TEST PILE NO. 2 (FINAL REDRIVING)
300
250
200
150
100
50
0
-50
FORCE (KIPS)
0
10
20
30
40
50
TIME (SEC x 10^{-3})
EXPERIMENTAL & FORCE TIME INPUT
HAMMER INPUT

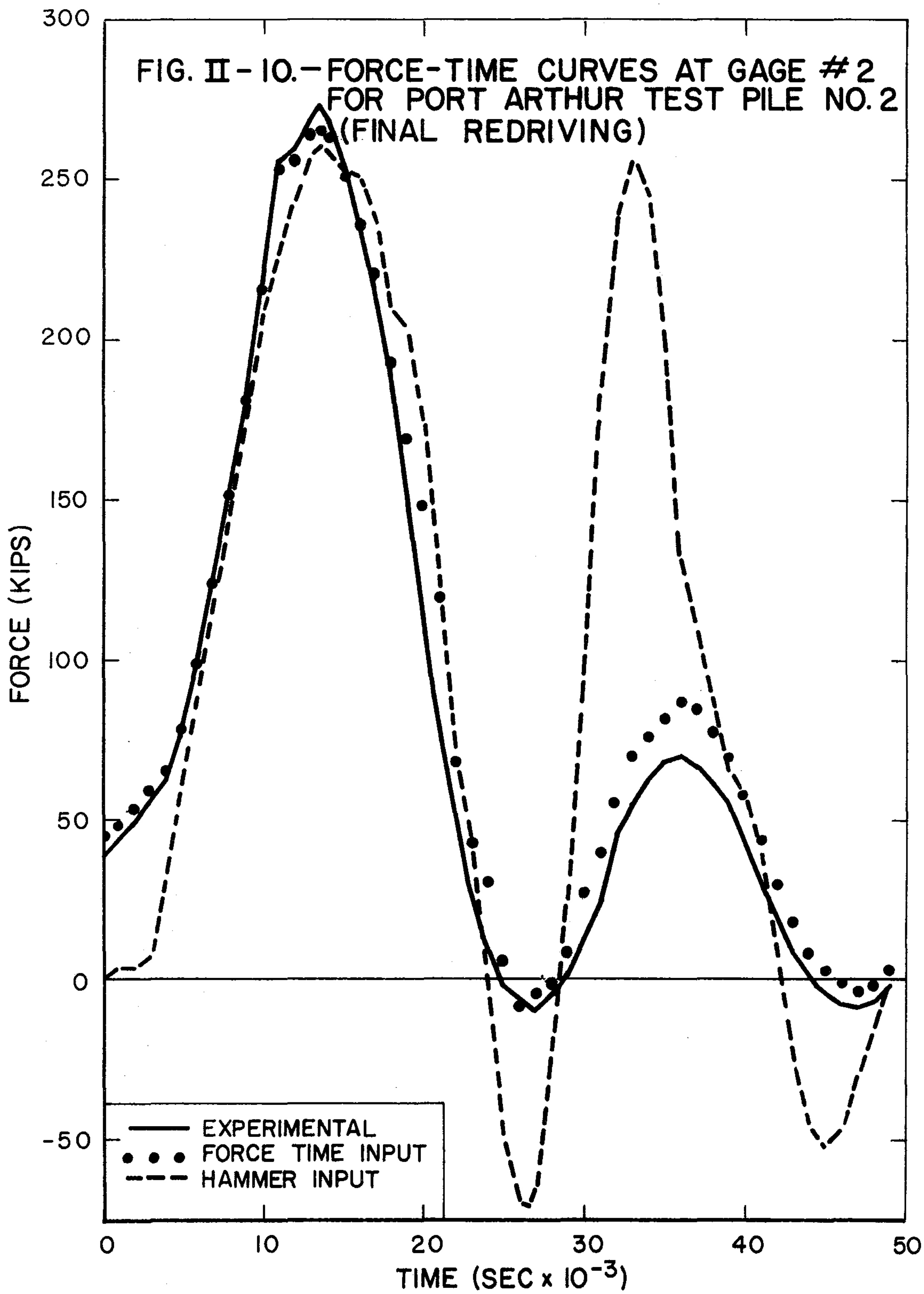
FIG. II - 10.—FORCE-TIME CURVES AT GAGE #2
FOR PORT ARTHUR TEST PILE NO. 2
(FINAL REDRIVING)
FORCE (KIPS)
TIME (SEC x 10^-3)
300
250
200
150
100
50
0
-50
0
10
20
30
40
50
EXPERIMENTAL
FORCE TIME INPUT
HAMMER INPUT

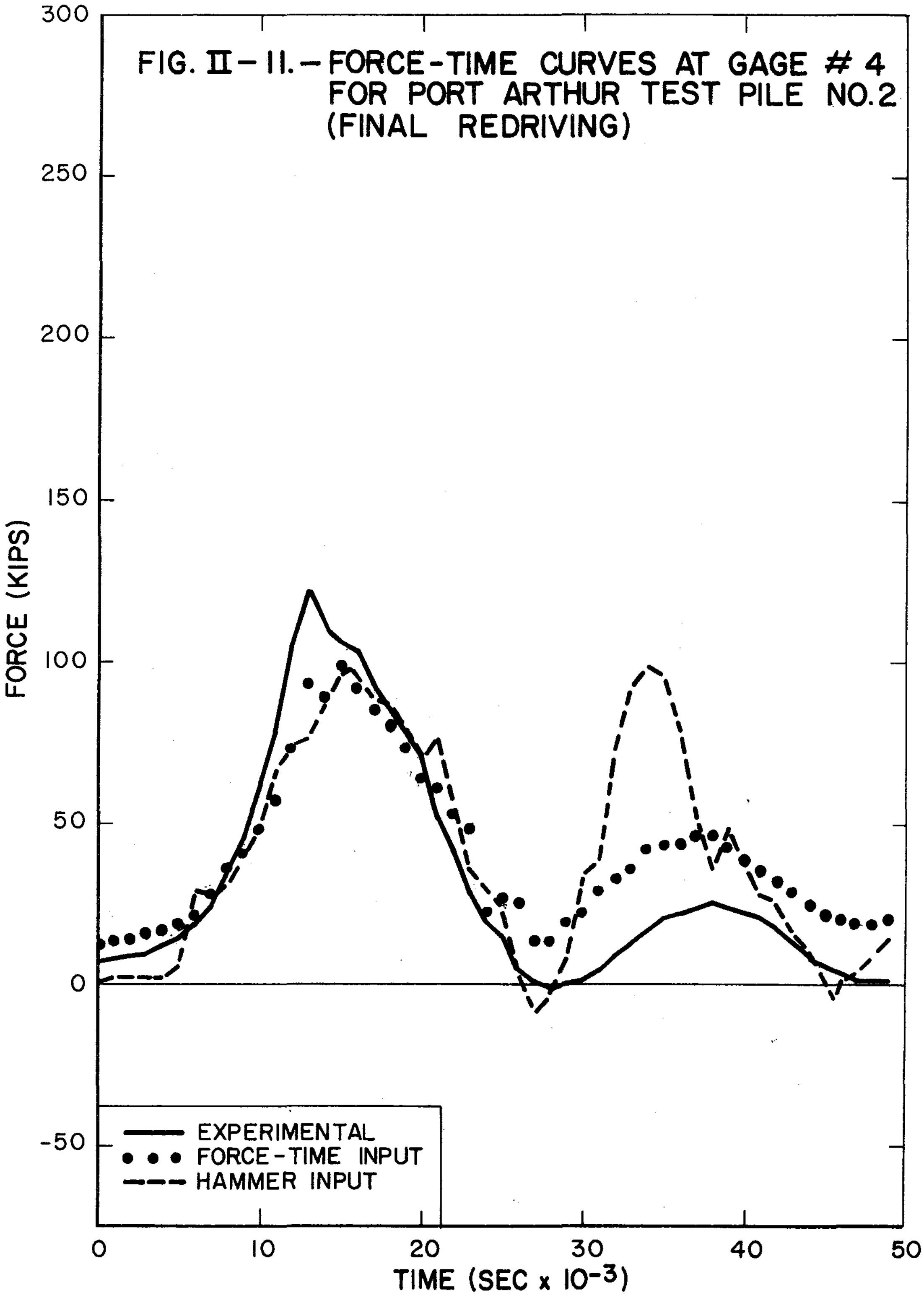

FIG. II-11.—FORCE-TIME CURVES AT GAGE # 4 FOR PORT ARTHUR TEST PILE NO. 2 (FINAL REDRIVING)

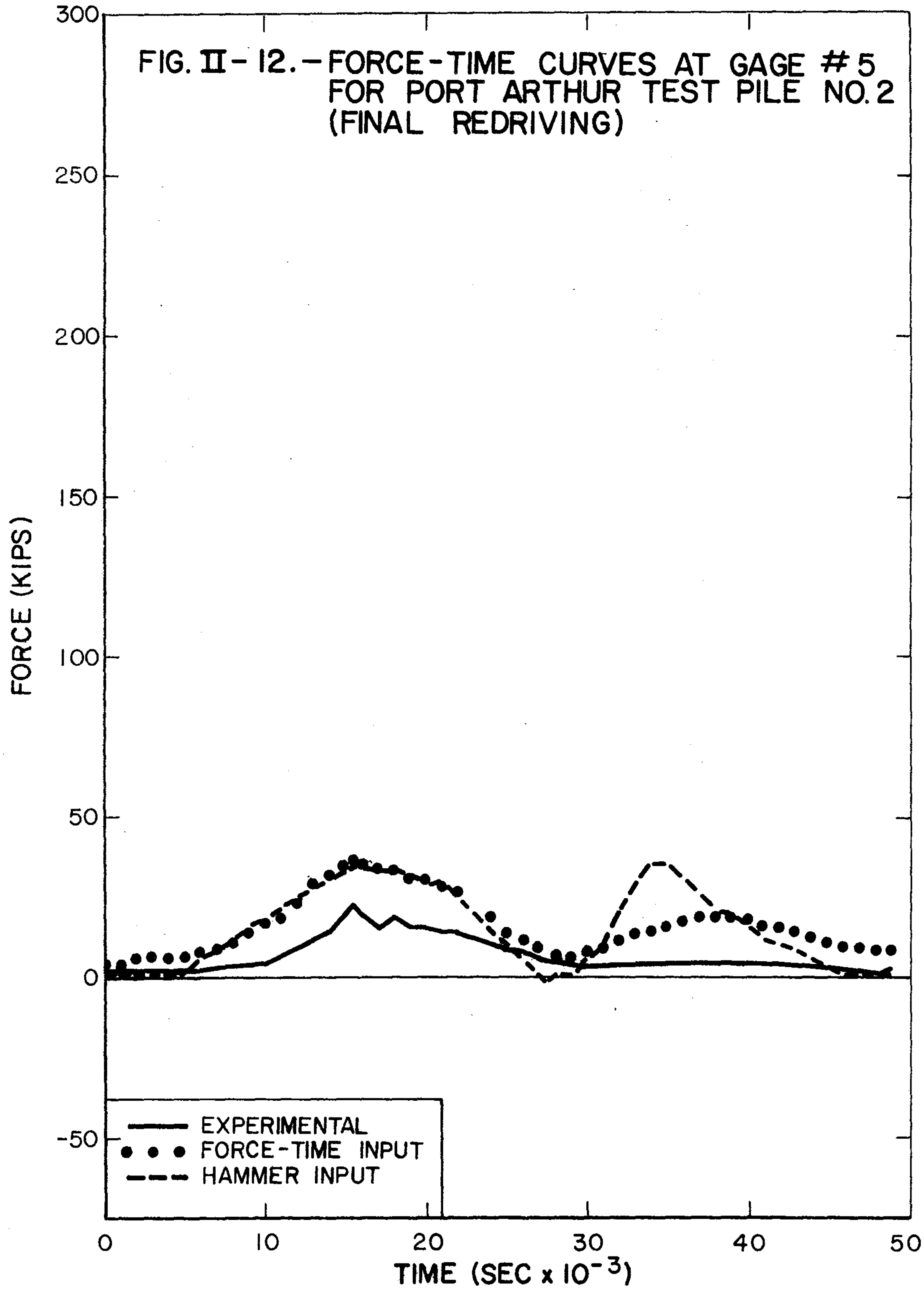
FIG. II - 12. - FORCE-TIME CURVES AT GAGE #5
FOR PORT ARTHUR TEST PILE NO. 2
(FINAL REDRIVING)
FORCE (KIPS)
TIME (SEC x 10^{-3})
300
250
200
150
100
50
0
-50
0
10
20
30
40
50
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

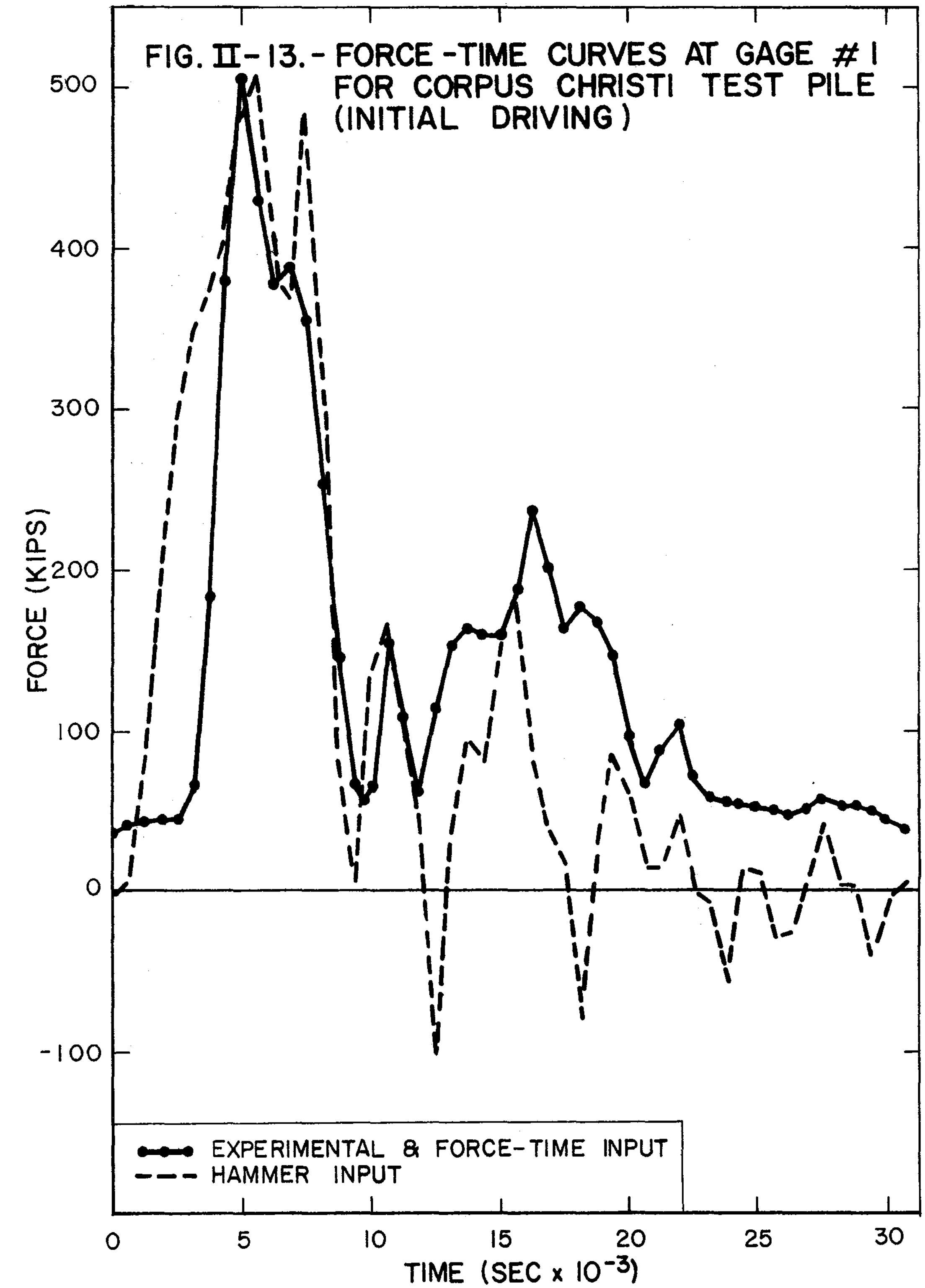
FIG. II-13.- FORCE-TIME CURVES AT GAGE #1 FOR CORPUS CHRISTI TEST PILE (INITIAL DRIVING)
500
400
300
200
100
0
-100
FORCE (KIPS)
0
5
10
15
20
25
30
TIME (SEC x 10^{-3})
EXPERIMENTAL & FORCE-TIME INPUT
HAMMER INPUT

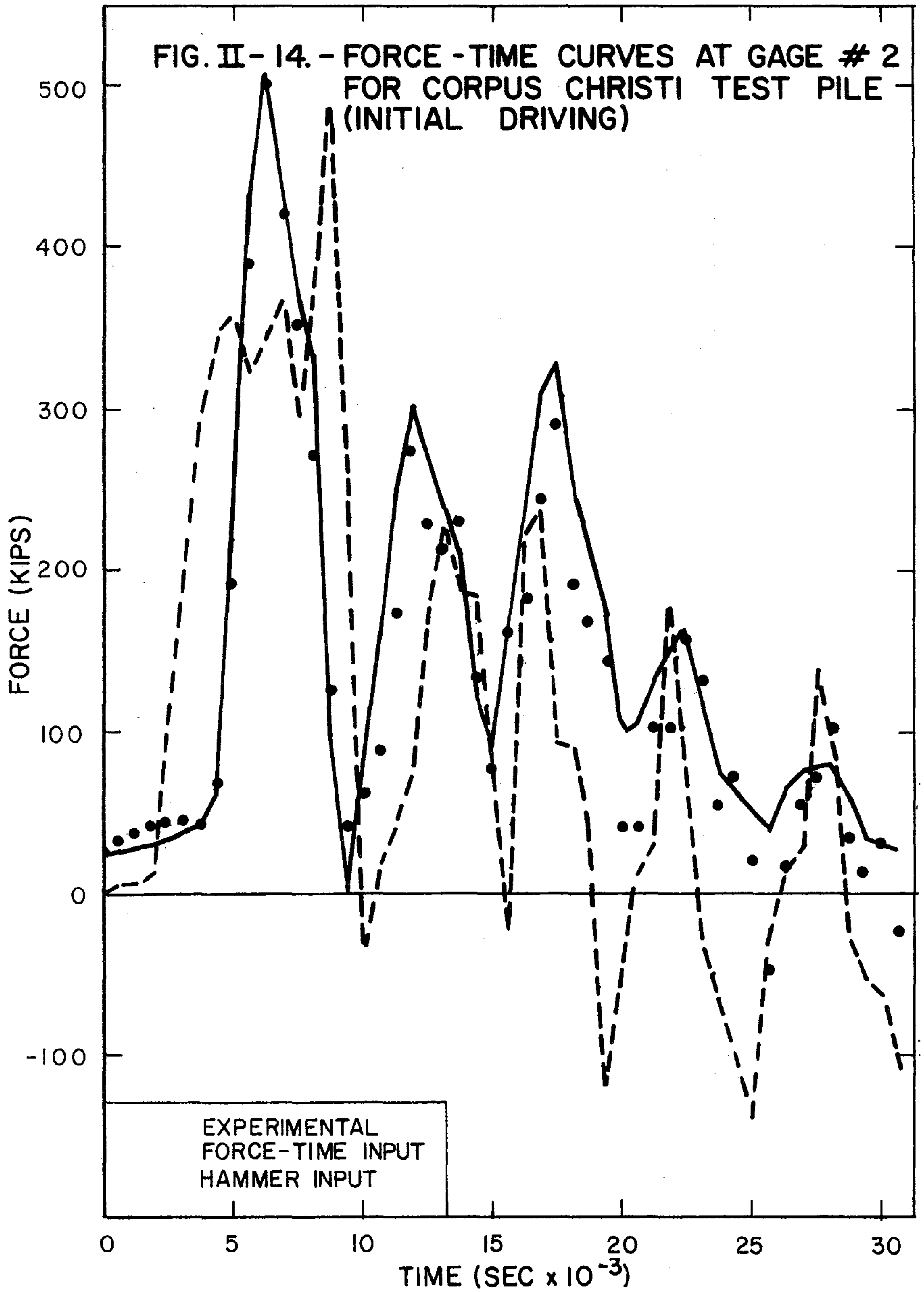
FIG. II-14.-FORCE-TIME CURVES AT GAGE # 2
FOR CORPUS CHRISTI TEST PILE
(INITIAL DRIVING)
FORCE (KIPS)
500
400
300
200
100
0
-100
TIME (SEC x 10^-3)
0
5
10
15
20
25
30
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

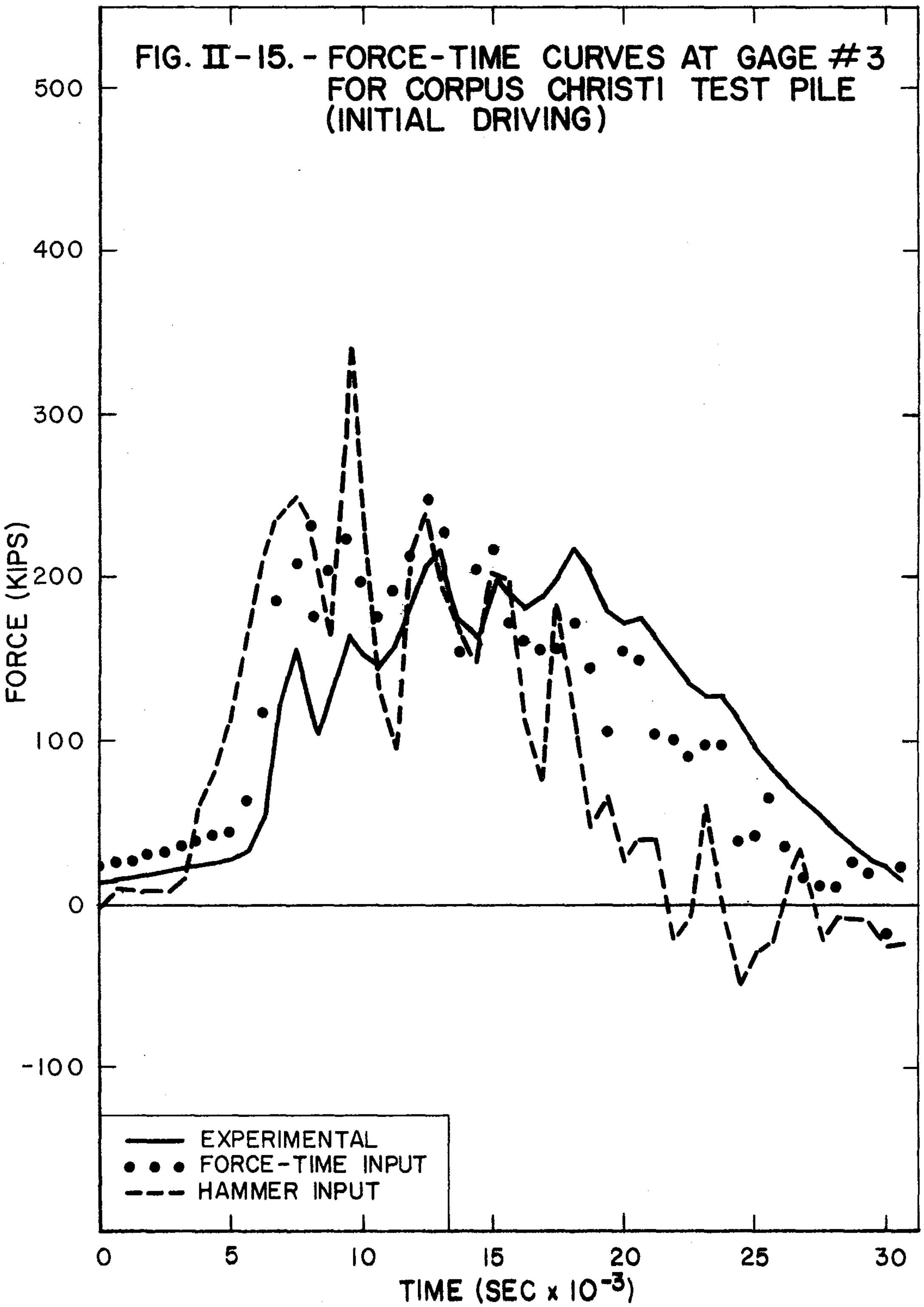
FIG. II-15. - FORCE-TIME CURVES AT GAGE #3
FOR CORPUS CHRISTI TEST PILE
(INITIAL DRIVING)
FORCE (KIPS)
500
400
300
200
100
0
-100
TIME (SEC x 10^{-3})
0
5
10
15
20
25
30
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

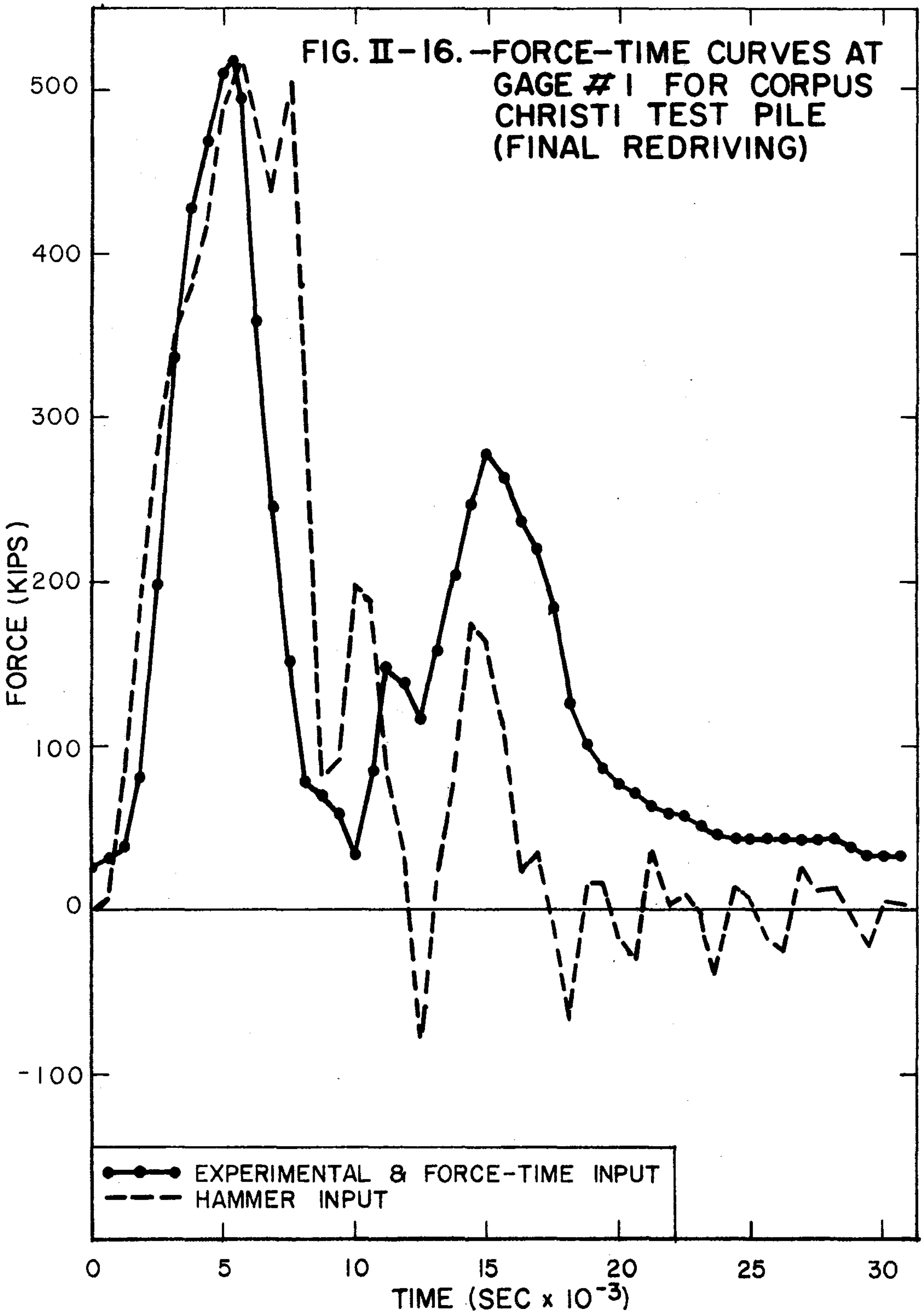
FIG. II-16. -FORCE-TIME CURVES AT GAGE #1 FOR CORPUS CHRISTI TEST PILE (FINAL REDRIVING)
FORCE (KIPS)
500
400
300
200
100
0
-100
EXPERIMENTAL & FORCE-TIME INPUT
HAMMER INPUT
0
5
10
15
20
25
30
TIME (SEC x 10^-3)

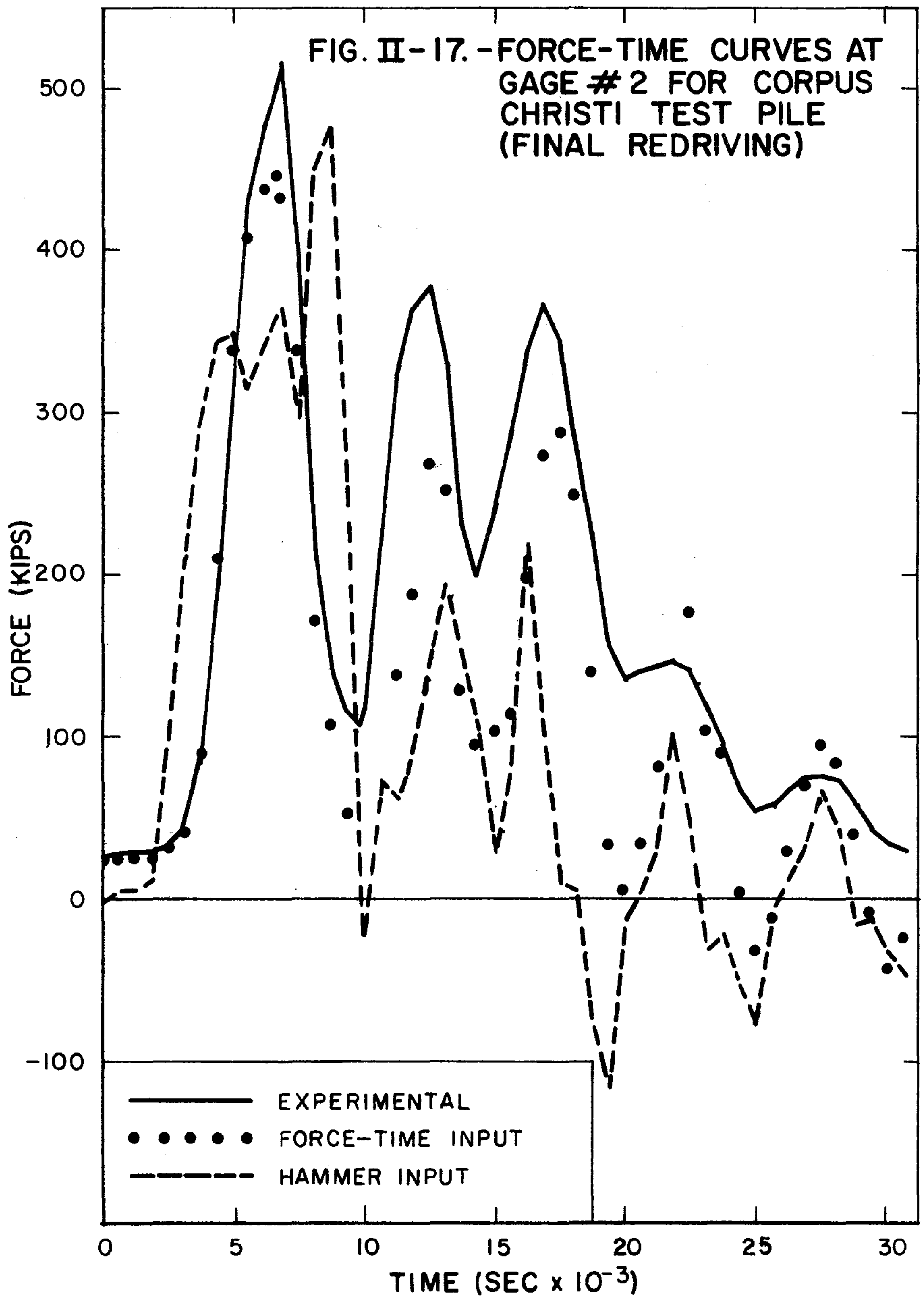
FIG. II-17. -FORCE-TIME CURVES AT GAGE # 2 FOR CORPUS CHRISTI TEST PILE (FINAL REDRIVING)
FORCE (KIPS)
500
400
300
200
100
0
-100
0
5
10
15
20
25
30
TIME (SEC x 10^{-3})
EXPERIMENTAL
FORCE-TIME INPUT
HAMMER INPUT

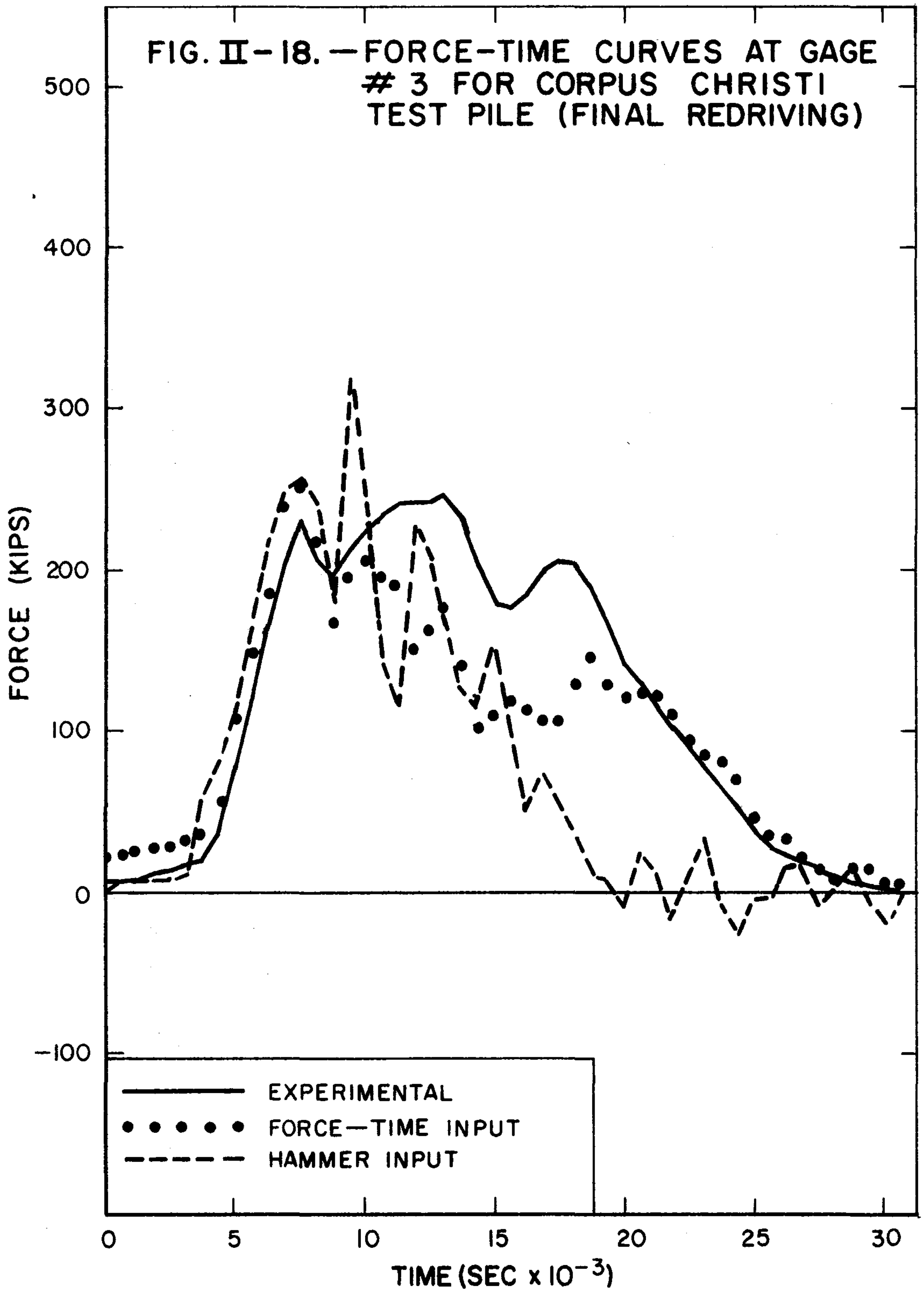

FIG. II-18. —FORCE-TIME CURVES AT GAGE # 3 FOR CORPUS CHRISTI TEST PILE (FINAL REDRIVING)
FORCE (KIPS)
500
400
300
200
100
0
−100
0
5
10
15
20
25
30
TIME (SEC x 10⁻³)
EXPERIMENTAL
FORCE—TIME INPUT
HAMMER INPUT

APPENDIX III. - FIELD DATA AND SUMMARIES OF INPUT DATA FOR INSTRUMENTED TEST PILES

DEPTH (FT.)	SOIL SYMBOL	DESCRIPTION OF STRATUM	DRY DENSITY LB/CU FT.	MOISTURE CONTENT %	LIQUID LIMIT	PLASTIC LIMIT	UNCONFINED SHEAR STR. (PSF)
10 19.5		DARK ORGANIC CLAY	* FIELD VANE SHEAR VALUES				*300 *240 *360 *1000
20 27		WET TAN & GRAY SILTY CLAY	92.6	22.6	78.9	18.2	2820
30 37		WET TAN & GRAY CLAYEY SILT	90.5	30.5	78.4	31.3	1480
40 48		WET GRAY CLAY WITH SOME SHELL	84.5	42.6	85.5	30.7	2510
50 60 62		WET TAN & GRAY SILTY CLAY	105.4	18.4	67.8	18.8	1675
		PLASTIC GRAY CLAY	112.1	16.2	50	17.9	1870

FIG. III-1.- SOIL PROFILE FOR PORT ARTHUR TEST PILE NO. 1

DEPTH (FT.)	SOIL SYMBOL	DESCRIPTION OF STRATUM	DRY DENSITY LB/CU FT.	MOISTURE CONTENT %	LIQUID LIMIT	PLASTIC LIMIT	UNCONFINED SHEAR STR. (PSF)
10 20 30		WET DARK GRAY ORGANIC CLAY	88	* FIELD VANE SHEAR VALUES 32.9	63	21.2	* 240 * 360 * 200 300 748
40 50		WET TAN AND GRAY SILTY CLAY WITH SOME SHELL	103.6	22.5	67.0	21.5	1582
60 67		WET TAN AND GRAY SILTY CLAY	98	23.9	69.5	24.7	1860
70 73		WET TAN AND GRAY SANDY CLAY	105.3	20.6	37.5	17.1	2020
		WET TAN AND GRAY CLAY	100	23.8	65.3	21.9	1720

FIG.III-2.- SOIL PROFILE FOR PORT ARTHUR TEST PILE NO.2

DEPTH, FT.	SOIL SYMBOL	DESCRIPTION OF STRATUM	TEXAS HWY DEPT. PENETRATION TEST, BLOWS 1st 6", 2nd 6"	STANDARD PENETRATION NUMBER N, BLOWS PER FOOT (a)	ESTIMATED ANGLE OF INTERNAL FRICTION, DEGREES (b)	ESTIMATED TOTAL UNIT WEIGHT, LBS. PER CU. FT. (b)	UNIFIED SOIL CLASSIFICATION
0–5		SAND, FINE, SILTY, BLUE-GRAY, P. GRADED, LOOSE, WET TO WATER BEARING	3,2	5	28	98	SM SP
5–10		SAND, FINE, TAN, POORLY GRADED	0,0	0	28	90	SM SP
10–15			8,22	14	31	114	SM SP
15–20		FINE, SILTY SAND, BLUE-GRAY TO TAN, POORLY GRADED, SLIGHTLY COMPACT TO COMPACT, WATER BEARING	14,21	16	32	115	SM SP
20–25			17,16	15	32	114	SM SP
25–30			11,10	10	30	112	SM SP
30–35			23,27	23	34	117	SM SP
35–40			43, 57(4")	58	42	140	SM SP

(a) STANDARD PENETRATION NUMBER N OBTAINED FROM TEXAS HIGHWAY DEPARTMENT TEST DATA.

(b) ESTIMATED FROM STANDARD PENETRATION NUMBER N.

FIG. III-3.- SOIL PROFILE FOR CORPUS CHRISTI TEST PILE

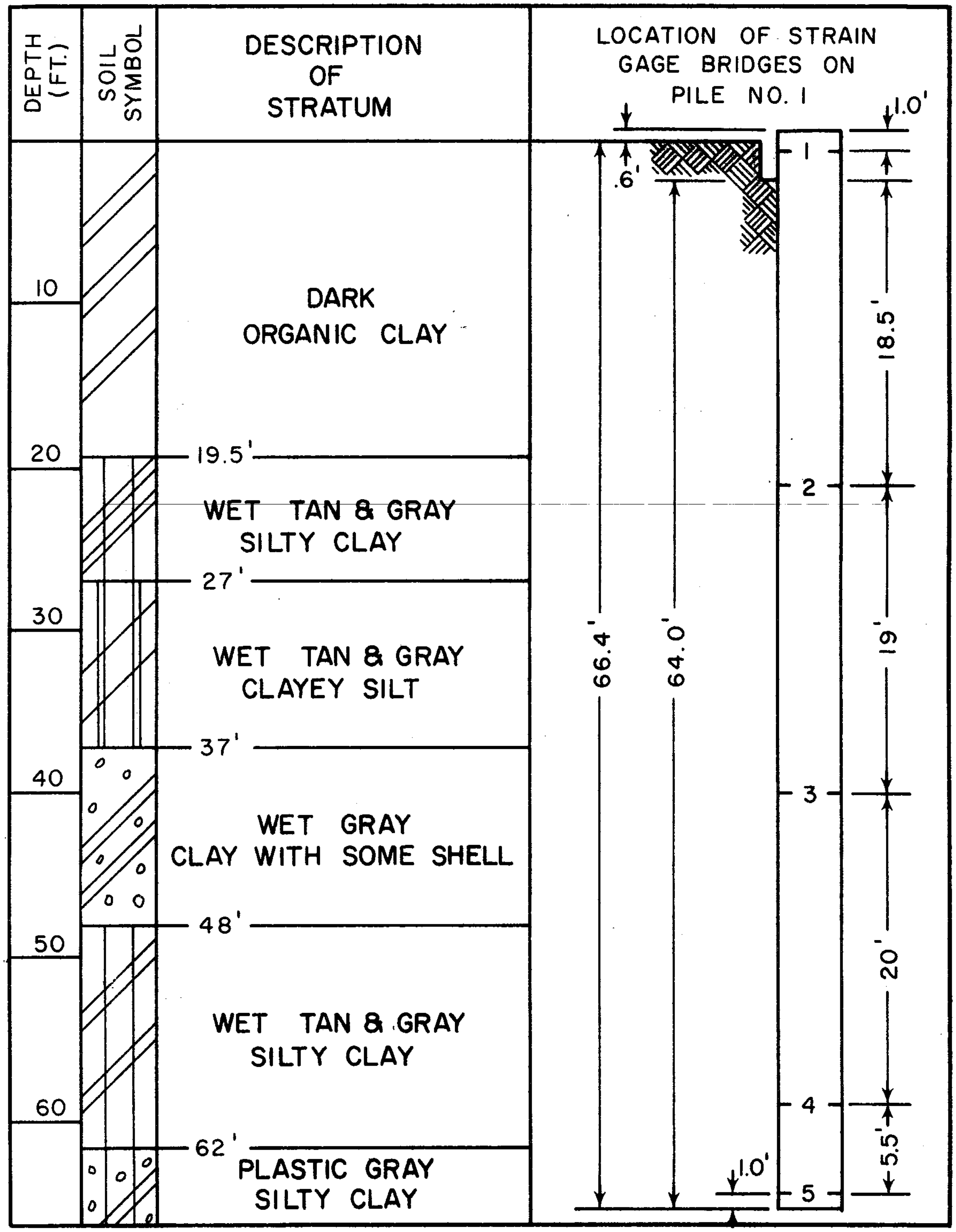

FIG. III-4.- LOCATION OF STRAIN GAGE BRIDGES FOR PORT ARTHUR TEST PILE NO. I

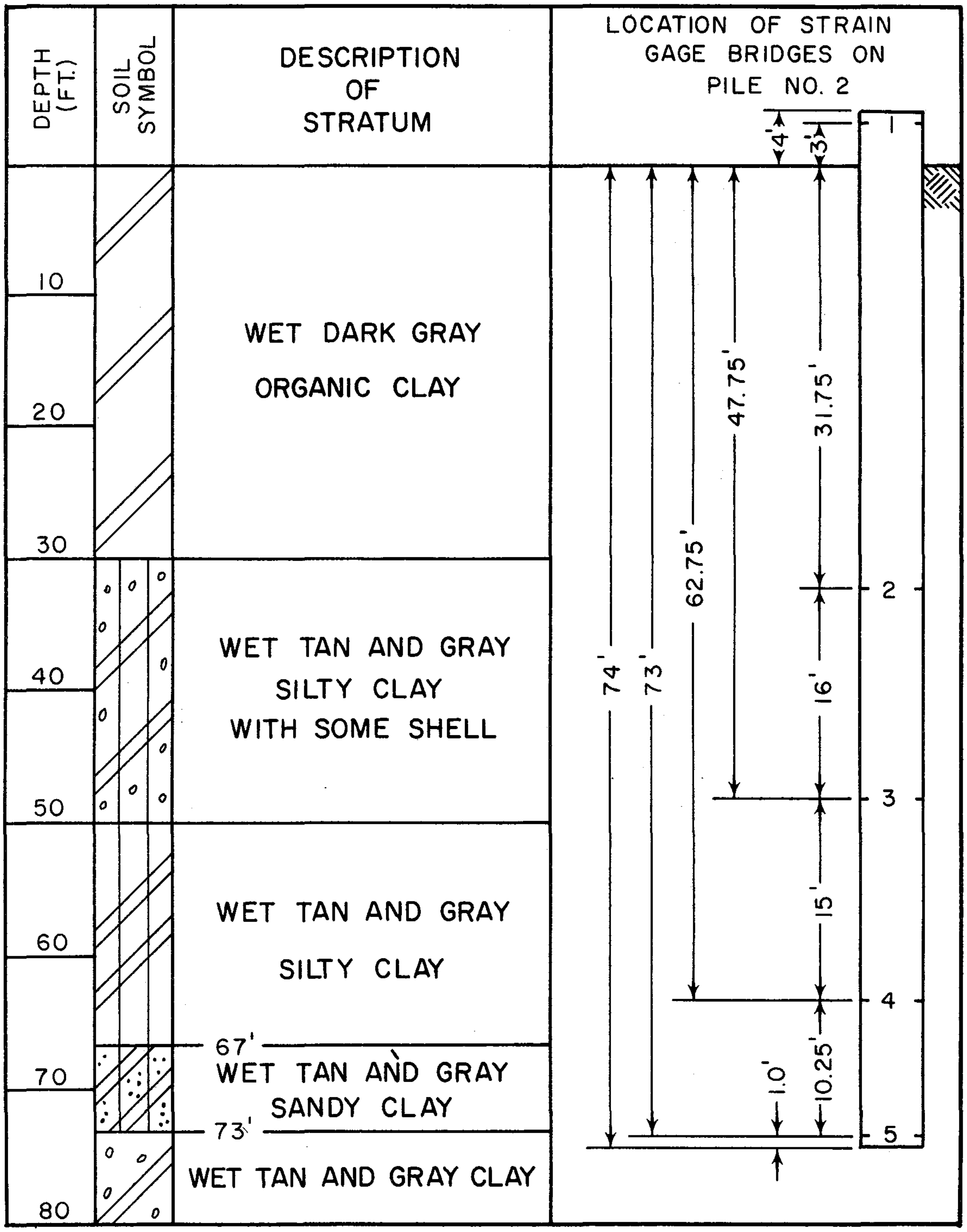

FIG. III-5.- LOCATION OF STRAIN GAGE BRIDGES FOR PORT ARTHUR TEST PILE NO. 2

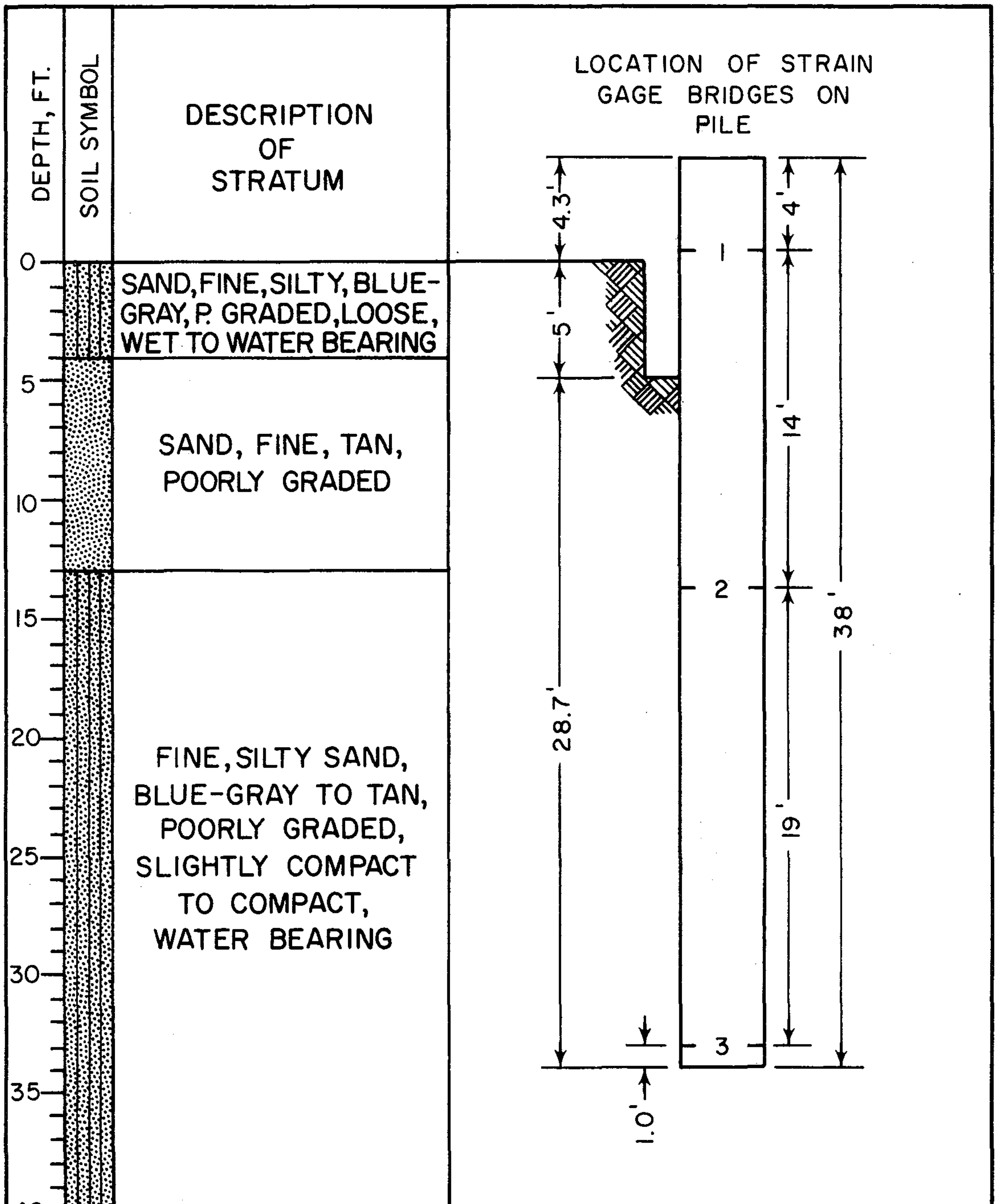

FIG. III-6.- LOCATION OF STRAIN GAGE BRIDGES FOR CORPUS CHRISTI TEST PILE

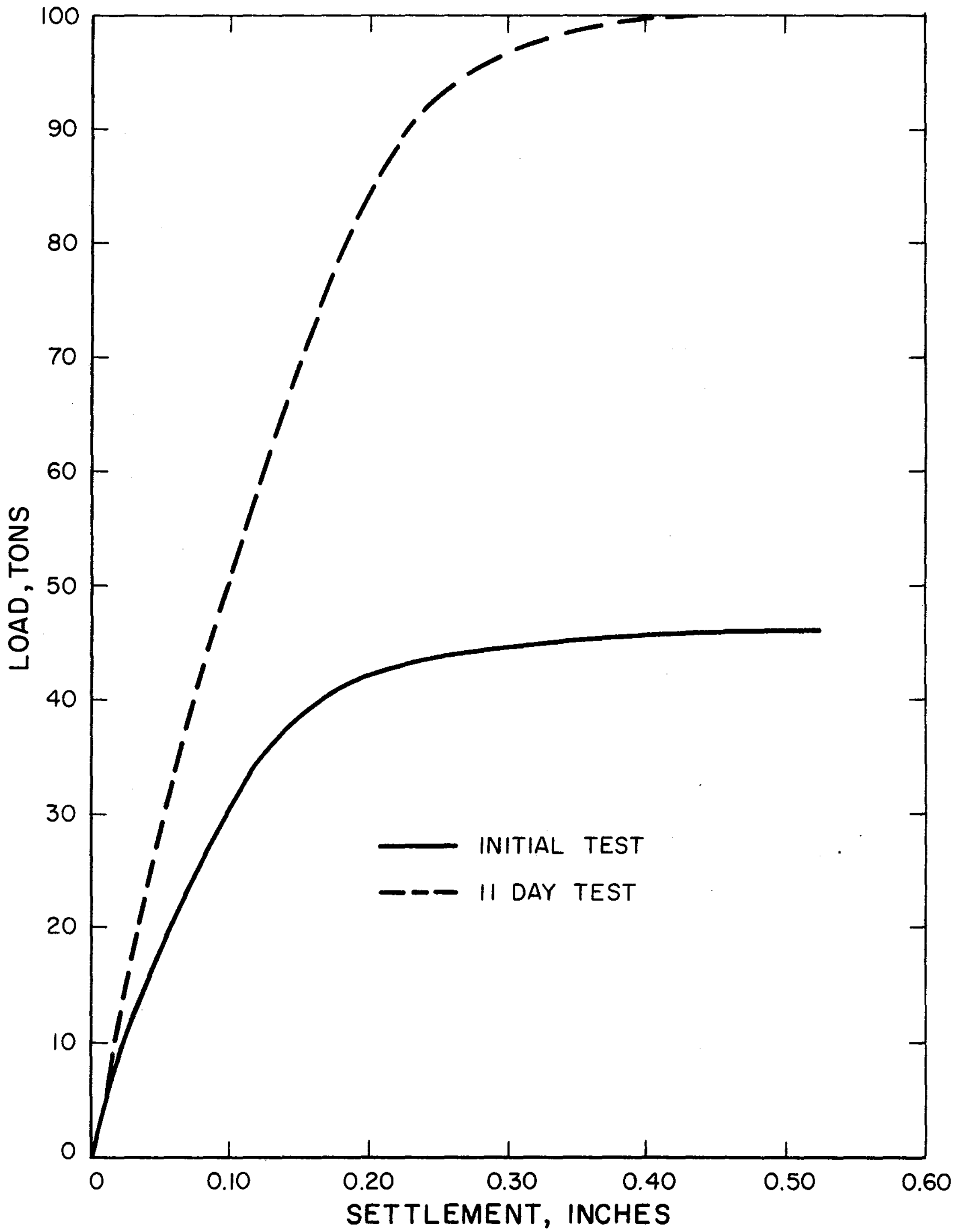

FIG. III-7.- LOAD vs. SETTLEMENT CURVES FOR PORT ARTHUR TEST PILE NO. 1

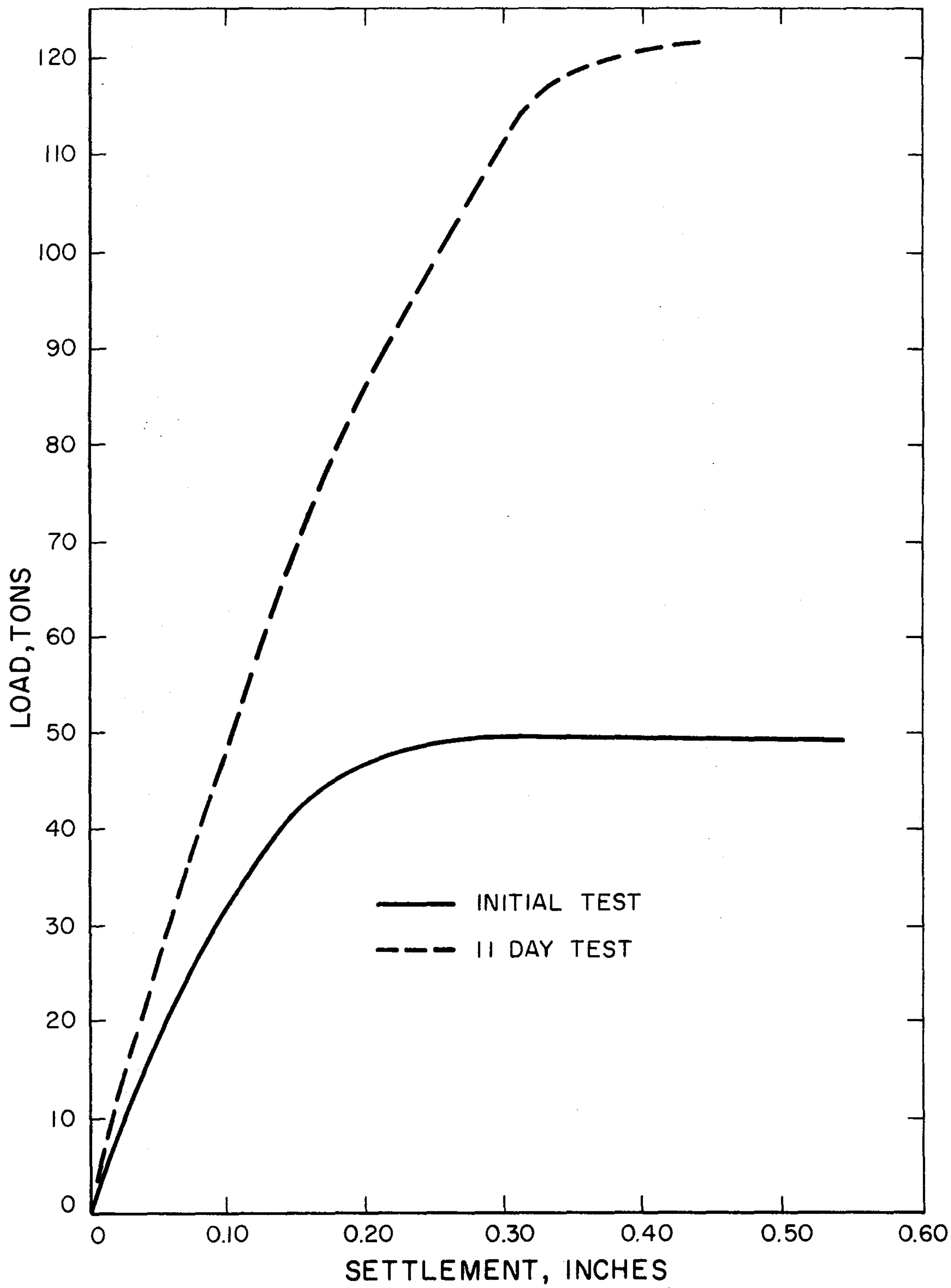

FIG. III-8.- LOAD vs. SETTLEMENT CURVES FOR PORT ARTHUR TEST PILE NO. 2

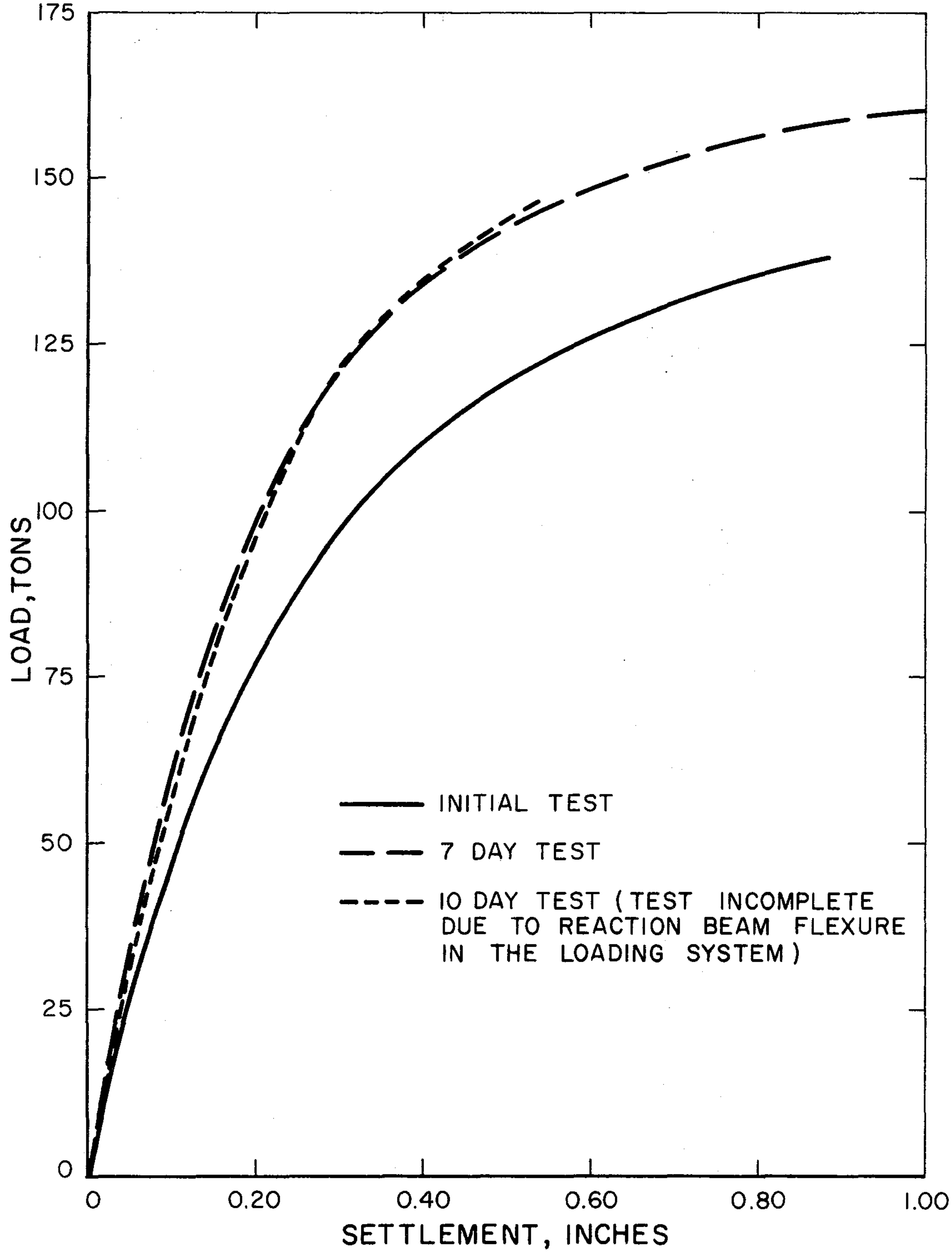

FIG. III-9.- LOAD vs. SETTLEMENT CURVES FOR CORPUS CHRISTI TEST PILE

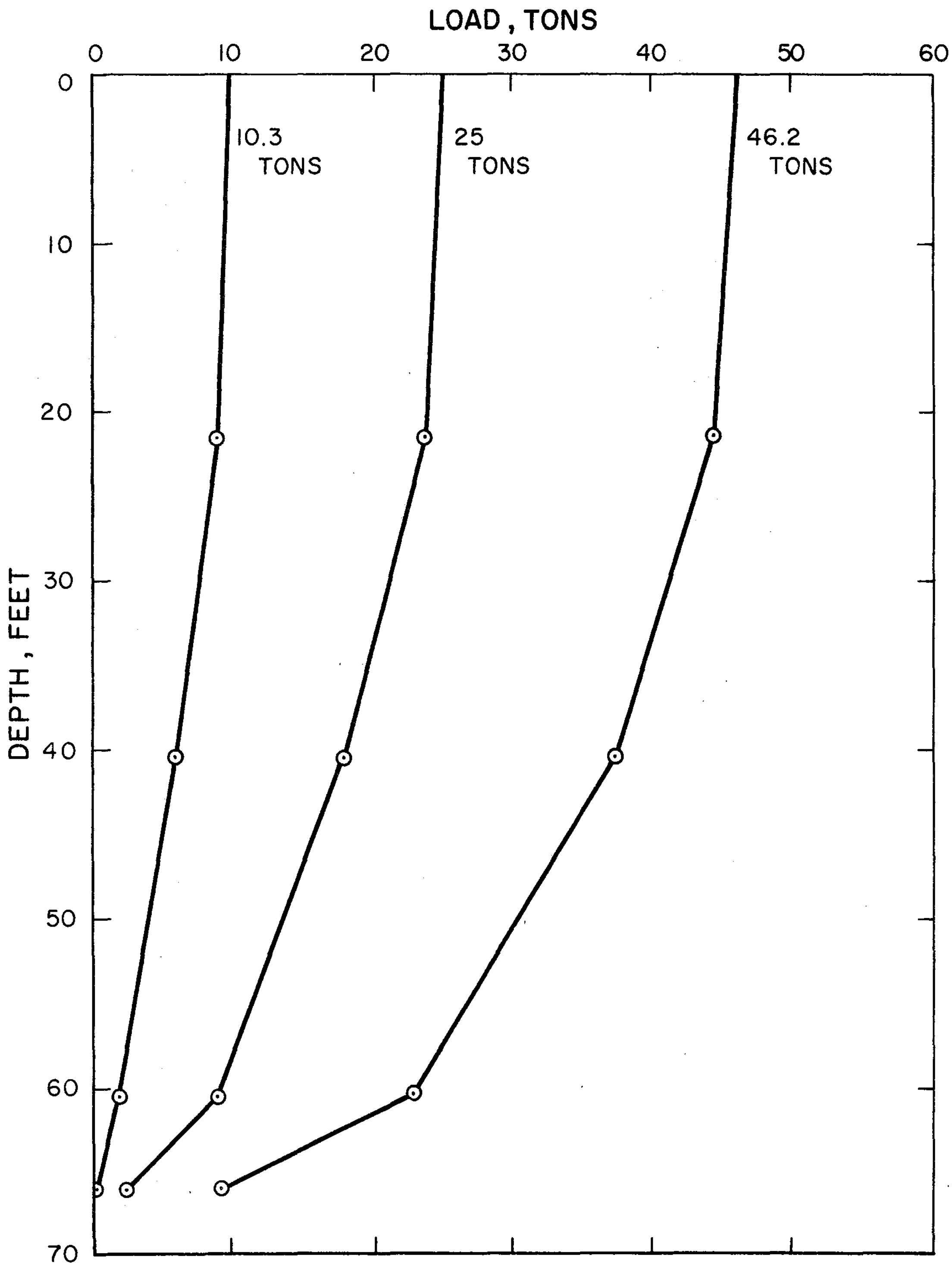

FIG. III - 10. — LOAD vs. DEPTH CURVES FOR PORT ARTHUR TEST PILE NO. I (INITIAL TEST)

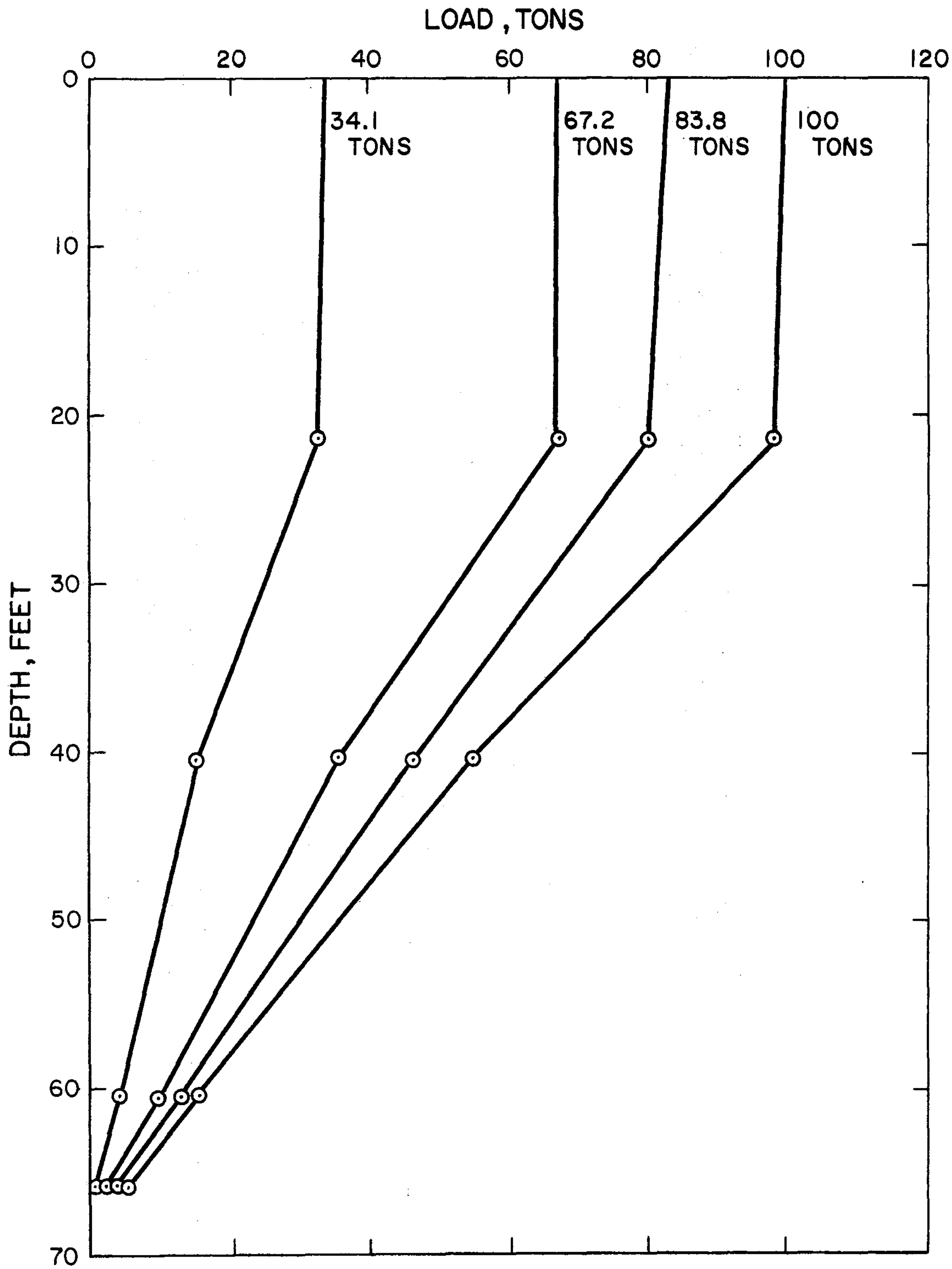

FIG. III-11. – LOAD vs. DEPTH CURVES FOR PORT ARTHUR TEST PILE NO. 1 (11 DAY TEST)

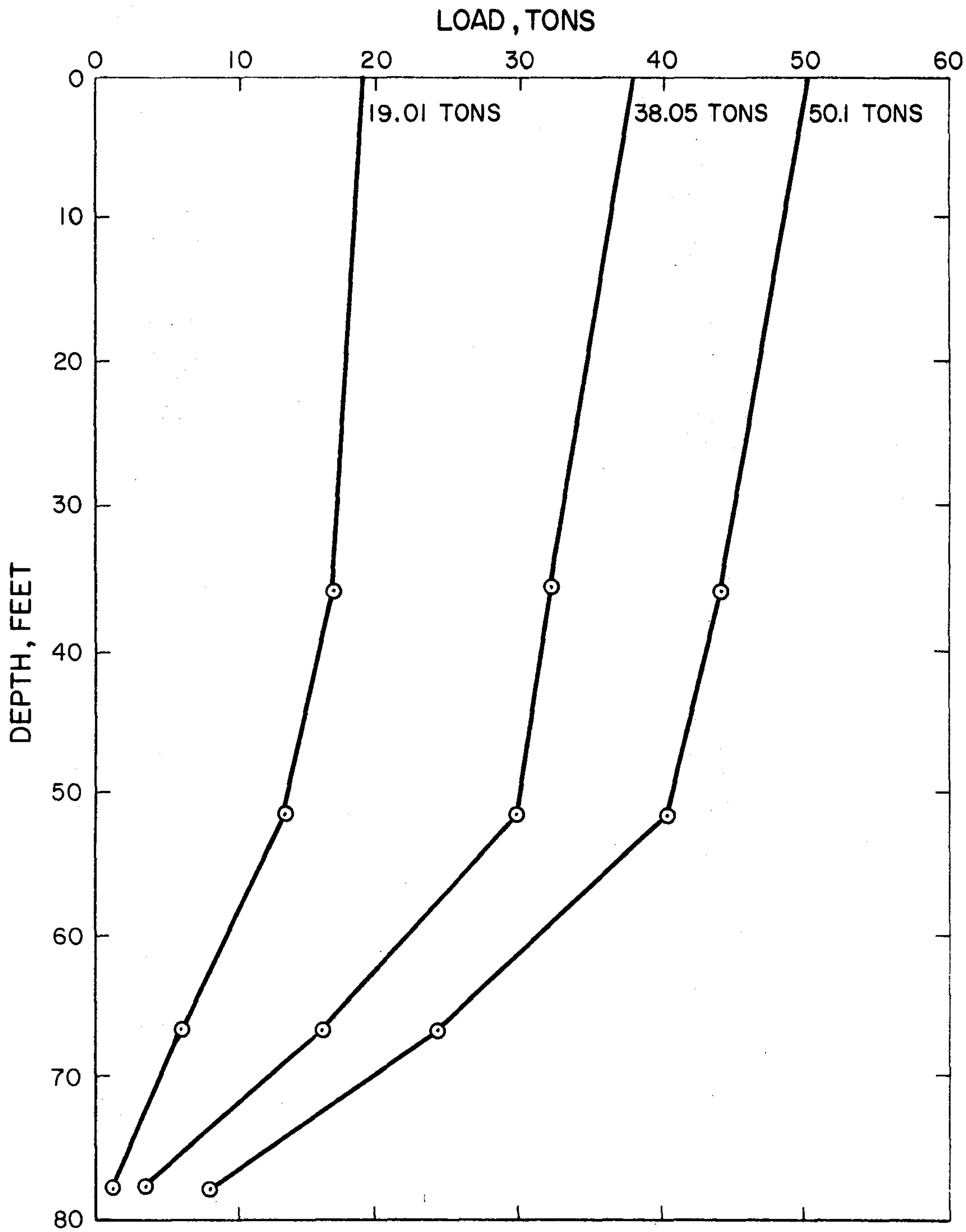

FIG. III - 12.– LOAD vs. DEPTH CURVES FOR PORT ARTHUR TEST PILE NO. 2 (INITIAL TEST)

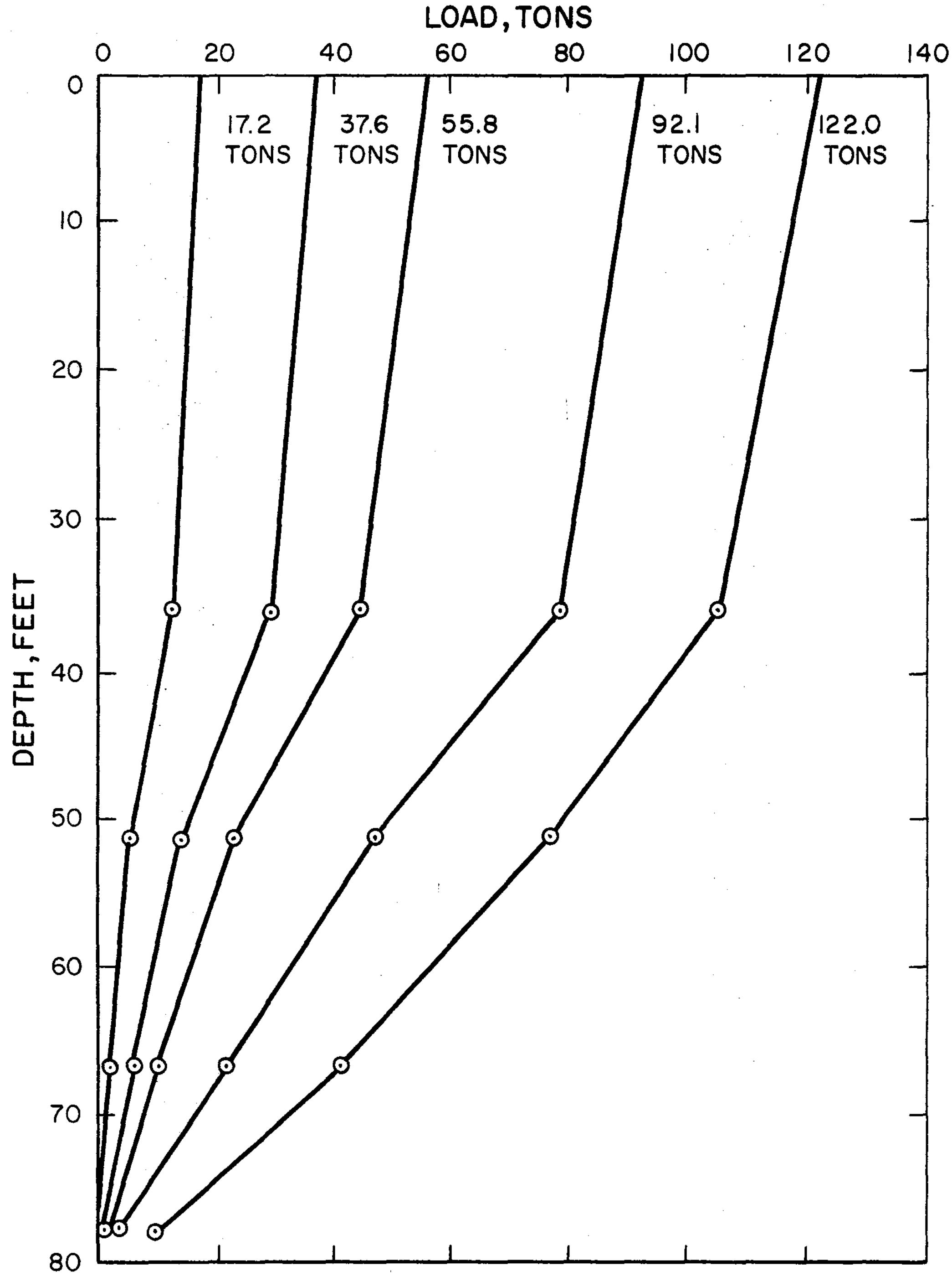

FIG. III - 13. - LOAD vs. DEPTH CURVES FOR PORT ARTHUR TEST PILE NO. 2 (11 DAY TEST)

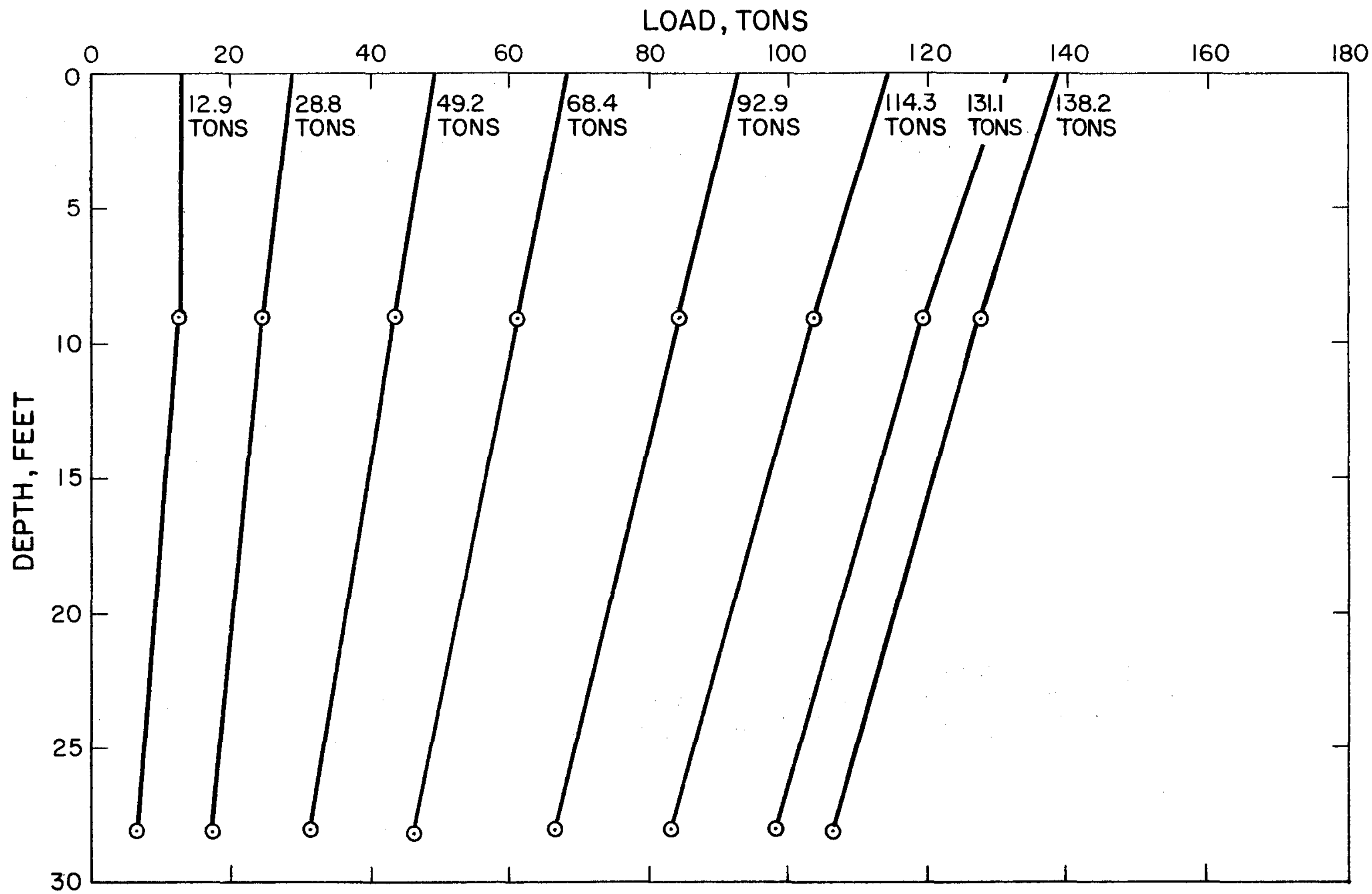

FIG. III-14.— LOAD vs. DEPTH CURVES FOR CORPUS CHRISTI TEST PILE (INITIAL TEST)

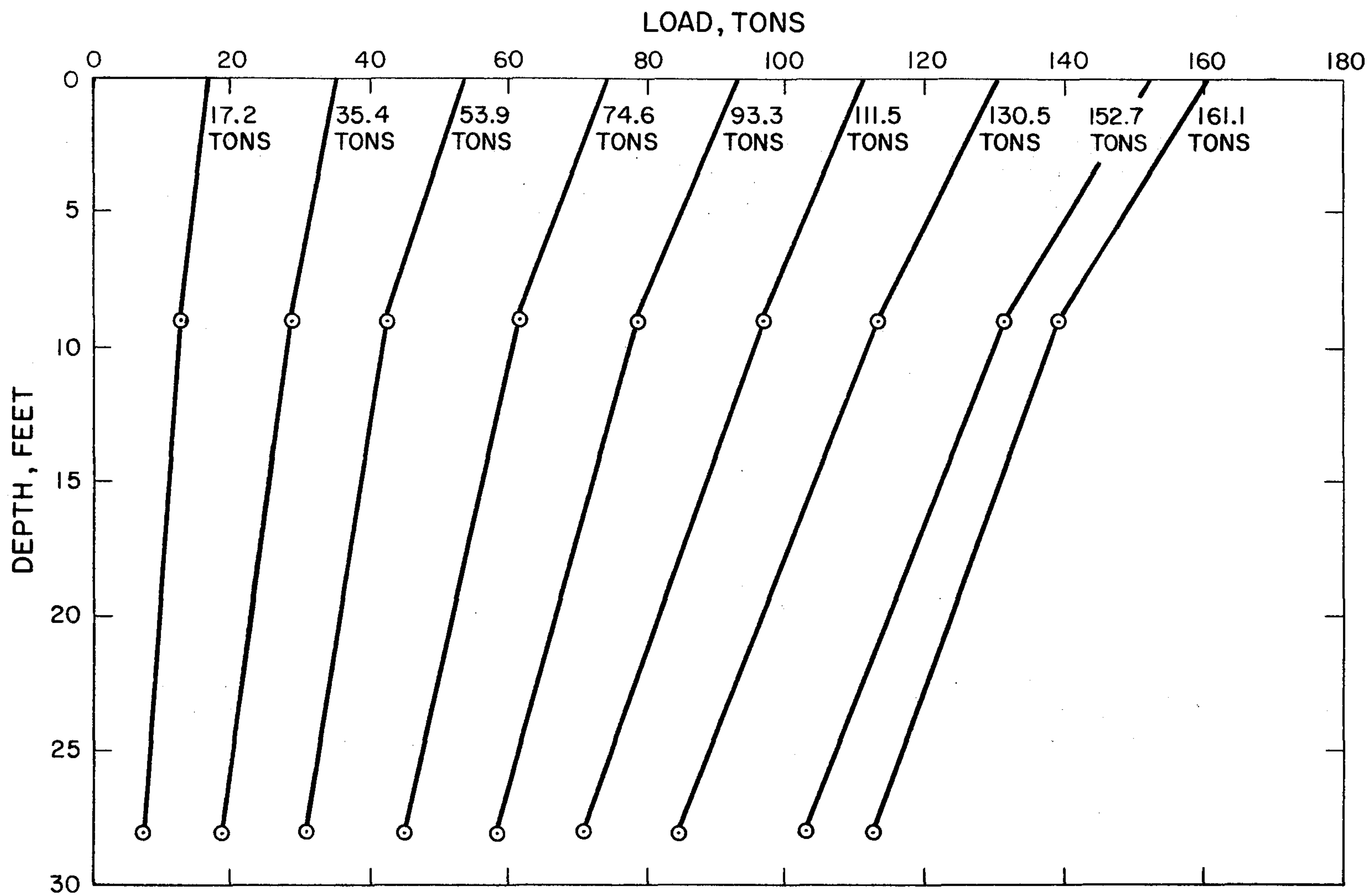

FIG. III - 15.– LOAD vs. DEPTH CURVES FOR CORPUS CHRISTI TEST PILE (7 DAY TEST)

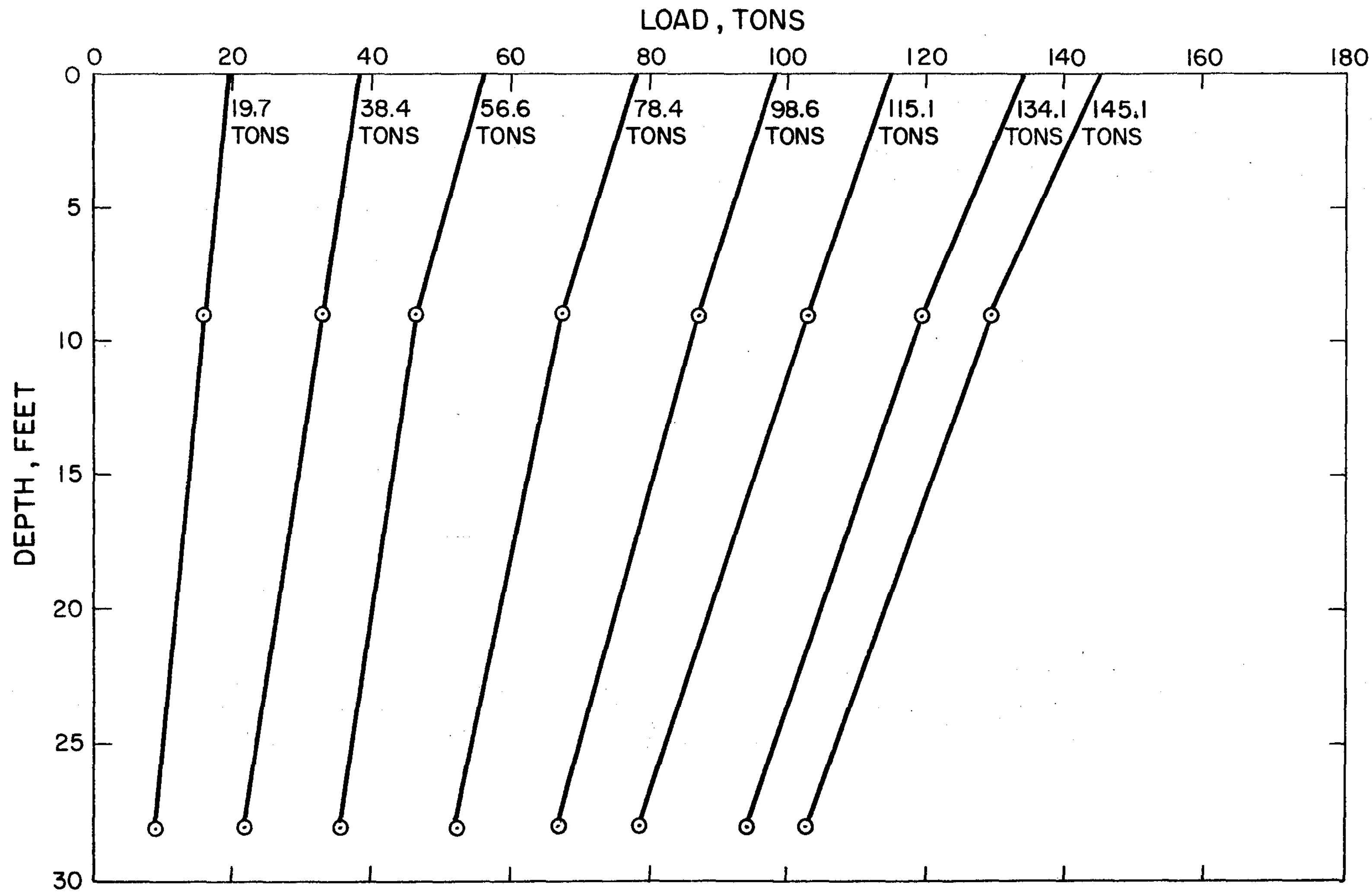

FIG. III-16.– LOAD vs. DEPTH CURVES FOR CORPUS CHRISTI TEST PILE (10 DAY TEST)

TABLE III-1. - PA1-INITIAL PILE DRIVING DATA

Pile tip Elevation (ft)	Depth of Pile in Ground (ft)	Energy of Hammer (ft-lb)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
+3.20 to -18.80	22	Weight of Hammer. Not Within Range of Manu-			
-18.80 to -30.80	34	facturer's Energy Rating Chart for P.S.I.G.			
-31.80	35	15,000	10	12	1.2000
-32.80	36	15,000	10	12	1.2000
-33.80	37	15,000	11	12	1.0909
-34.80	38	15,000	11	12	1.0909
-35.80	39	15,000	11	12	1.0909
-36.80	40	15,000	11	12	1.0909
-37.70	41	15,000	11	12	1.0909
-38.80	42	15,000	10	12	1.2000
-39.80	43	15,000	11	12	1.0909
-40.80	44	15,000	11	12	1.0909
-41.80	45	15,000	11	12	1.0909
-42.80	46	15,000	10	12	1.2000
-43.80	47	15,000	11	12	1.0909
-44.80	48	15,000	11	12	1.0909
-45.80	49	15,000	11	12	1.0909
-46.80	50	16,750	12	12	1.0000
-47.80	51	18,000	13	12	0.9231
-48.80	52	18,000	12	12	1.0000
-49.80	53	18,000	12	12	1.0000
-50.80	54	18,000	12	12	1.0000
-51.80	55	19,125	12	12	1.0000
-52.80	56	19,125	13	12	0.9231
-53.80	57	20,250	14	12	0.8571
-54.80	58	20,250	14	12	0.8571
-55.80	59	20,250	14	12	0.8571
-56.80[a]	60	21,250	15	12	0.8000
-57.80[b]	61	21,250	15	12	0.8000
-58.80[b]	62	19,125	20	12	0.6000
-59.80[b]	63	20,250	18	12	0.6667
-60.55[b]	63.75	20,250	13	9	0.6923

[a]Last blow without cushion. Stopped 1 hour.

[b]Driven with cushion. Final blow count without cushion extrapolated to be 16 blows per foot.

TABLE III-2. - PA1-FINAL PILE DRIVING DATA

Pile tip Elevation (ft)	Depth of Pile in Ground (ft)	Energy of Hammer (ft-lb)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
-60.55[a]	63.75	21,200	36	1	0.0278
-60.72[a]	63.92	19,500	114	2	0.0175
-60.97[b]	64.17	24,250	21	3	0.1429
-61.22[c]	64.42	24,250	18	3	0.1667
-61.47[c]	64.67	24,250	18	3	0.1667
-61.72[c]	64.92	24,250	18	3	0.1667
-61.97[c]	65.17	24,250	18	3	0.1667
-62.22	65.42	24,250	19	3	0.1579
-62.47	65.67	24,250	16	3	0.1875
-62.72	65.92	24,250	18	3	0.1667
-62.97	66.17	24,250	18	3	0.1667
-63.22	66.42	24,250	18	3	0.1667
-63.47	66.67	24,250	17	3	0.1765
-63.72	66.92	24,250	18	3	0.1667
-63.97	67.17	24,250	18	3	0.1667
-64.22	67.42	24,250	18	3	0.1667
-64.47	67.67	24,250	16	3	0.1875
-64.72	67.92	24,250	17	3	0.1765
-64.97	68.17	24,250	18	3	0.1667
-65.22	68.42	24,250	19	3	0.1579

[a]Driven with cushion.

[b]First blow without cushion.

[c]Blow count without cushion averaged to be 72 blows per foot.

TABLE III-3. - PA2-INITIAL PILE DRIVING DATA

Pile tip Elevation (ft)	Depth of Pile in Ground (ft)	Energy of Hammer (ft-lb)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
+3.00 to -28.00	31	Weight of Hammer. Not Within Range of Manufacturer's Energy Rating Chart for P.S.I.G.			
-28.00 to -38.00	41				
-39.00	42	15,000	8	12	1.5000
-40.00	43	15,000	8	12	1.5000
-41.00	44	15,000	8	12	1.5000
-42.00	45	15,000	8	12	1.5000
-43.00	46	15,000	10	12	1.2000
-44.00	47	15,000	9	12	1.3333
-45.00	48	15,000	10	12	1.2000
-46.00	49	15,000	12	12	1.0000
-47.00	50	15,000	12	12	1.0000
-48.00	51	18,000	12	12	1.0000
-49.00	52	19,125	12	12	1.0000
-50.00	53	18,000	11	12	1.0909
-51.00	54	18,000	10	12	1.2000
-52.00	55	18,000	12	12	1.0000
-53.00	56	16,750	12	12	1.0000
-54.00	57	15,000	16	12	0.7500
-55.00	58	16,750	14	12	0.8571
-56.00	59	18,000	16	12	0.7500
-57.00	60	16,750	15	12	0.8000
-58.00	61	16,750	14	12	0.8571
-59.00	62	16,750	15	12	0.8000
-60.00	63	18,000	12	12	1.0000
-61.00	64	20,250	12	12	1.0000
-62.00	65	20,250	13	12	0.9231
-63.00	66	21,250	13	12	0.9231
-64.00	67	21,250	15	12	0.8000
-65.00	68	21,250	16	12	0.7500
-66.00	69	22,250	16	12	0.7500
-67.00[a]	70	22,250	21	12	0.5714
-68.00[b]	71	22,250	23	12	0.5217
-69.00[b]	72	22,250	23	12	0.5217
-70.00[b]	73	22,250	22	12	0.5455
-71.00[b]	74	22,250	22	12	0.5455

[a]Last blow without cushion. Stopped 45 minutes.

[b]Driven with cushion. Final blow count without cushion extrapolated to be 18 blows per foot.

TABLE III-4. - PA2-FINAL PILE DRIVING DATA

Pile Tip Elevation (ft)	Depth of Pile in Ground (ft)	Energy of Hammer (ft-lb)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
-71.00[a]	74.00	Stopped Here on PA2-Initial			
-71.08[a]	74.08	22,750	60	1	0.0167
-71.17[b]	74.17	22,750	44	1	0.0227
-71.25	74.25	22,000	44	1	0.0227
-71.33	74.33	22,750	60	1	0.0167
-71.42	74.42	22,750	50	1	0.0200
-71.50	74.50	22,000	40	1	0.0250
-71.75	74.75	23,500	98	3	0.0306
-72.00[c]	75.00	23,500	50	3	0.0600
-72.25[c]	75.25	23,500	50	3	0.0600
-72.50[c]	75.50	23,500	45	3	0.0667
-72.75[c]	75.75	22,750	45	3	0.0667
-73.00[c]	76.00	22,750	52	3	0.0577
-73.25[c]	76.25	22,000	55	3	0.0545
-73.50[c]	76.50	22,000	48	3	0.0625
-73.75[c]	76.75	22,000	53	3	0.0566
-74.00[c]	77.00	22,000	50	3	0.0600
-74.25	77.25	22,000	50	3	0.0600
-74.50	77.50	22,000	50	3	0.0600
-74.75	77.75	21,200	54	3	0.0556
-75.00	78.00	21,200	58	3	0.0517
-75.25	78.25	23,500	50	3	0.0600
-75.50	78.50	23,500	25	3	0.1200
-75.75	78.75	23,500	31	3	0.0968
-76.00	79.00	23,500	30	3	0.1000
-76.25	79.25	22,750	31	3	0.0968
-76.50	79.50	22,750	35	3	0.0857

[a]Driven with cushion.

[b]First blow without cushion.

[c]Blow count without cushion averaged to be 200 blows per foot.

TABLE III-5. - CC-INITIAL PILE DRIVING DATA

Pile Tip Elevation (ft)	Depth of Pile in Ground (ft)	Stroke of Hammer (ft)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
-6.5	9.5	Approximately 5 ft. alignment hole			
-15.0	18.0	5.00	24	102	4.250
-16.0	19.0	4.50	18	12	0.667
-17.0	20.0	4.50	18	12	0.667
-18.0	21.0	4.50	19	12	0.632
-19.0	22.0	4.75	19	12	0.632
-19.5	22.5	5.00	10	6	0.600
-20.0	23.0	4.50	10	6	0.600
-20.5	23.5	4.50	11	6	0.545
-21.0	24.0	5.00	9	6	0.667
-21.5	24.5	4.75	12	6	0.500
-22.0	25.0	4.75	8	6	0.750
-22.5	25.5	4.75	11	6	0.545
-23.0	26.0	4.75	11	6	0.545
-23.5	26.5	5.00	10	6	0.600
-24.0	27.0	5.00	10	6	0.600
-24.5	27.5	4.75	11	6	0.545
-25.0	28.0	5.00	10	6	0.600
-25.5	28.5	5.00	11	6	0.545
-25.7[a]	28.7	5.00	8	3	0.375
-26.2	29.2	5.25	13	3	0.231
-26.6[b]	29.6	5.00	7	3.75	0.536
-26.9[c]	29.9	5.00	12	4	0.333
-27.1	30.1	5.50	7	2.25	0.321
-27.5[d]	30.5	5.25	16	5	0.313
-28.0	31.0	5.00	18	6	0.333
-28.5	31.5	5.25	20	6	0.300
-28.9[e]	31.9	5.50	14	4.5	0.321
-29.0[e]	32.0	5.50	5	1.5	0.300
-29.3	32.3	5.50	12	3.5	0.292
-29.5[f]	32.5	5.75	9	2.5	0.278
-30.0[f]	33.0	5.75	23	6	0.261
-30.2[f]	33.2	5.75	13	3	0.231
-30.5[f]	33.5	5.50	11	3	0.273
-30.7[f]	33.7	5.50	9	2	0.222

[a] Stopped 33 minutes.

[b] Stopped 10 minutes.

[c] Stopped 7 minutes.

[d] Stopped 6 minutes.

[e] Stopped 2 minutes.

[f] Blow count averaged to be 48 blows per foot.

TABLE III-6. - CC-FINAL PILE DRIVING DATA

Pile Tip Elevation (ft)	Depth of Pile in Ground (ft)	Stroke of Hammer (ft)	Number of Blows	Total Penetration (in.)	Average Penetration (in.)
-30.86	33.9				
-30.9[a]	33.9	6.00	18	1	0.056
-31.0[b]	34.0	6.50	7	1	0.143
-31.1[b,c]	34.1	6.25	8	1	0.125
-31.2[b]	34.2	6.00	6	1	0.167
-31.3[b]	34.3	6.00	7	1	0.143
-31.4[b]	34.4	5.75	7	1	0.143
-31.4	34.4	5.50	7	1	0.143
-31.5	34.5	5.25	8	1	0.125
-31.6	34.6	5.50	9	1	0.111
-31.7	34.7	5.50	8	1	0.125
-31.8	34.8	5.50	8	1	0.125
-31.9	34.9	5.25	9	1	0.111
-32.1	35.1	5.50	19	2	0.105
-32.3	35.3	5.50	33	3	0.091
-32.6[d]	35.6	5.50	31	3	0.097
-32.9	35.9	6.25	33	3	0.091
-33.1	36.1	6.25	32	3	0.094
-33.4	36.4	6.00	34	3	0.088
-33.6[e]	36.6	6.50	44	3	0.068
-33.9	36.9	6.50	39	3	0.077
-34.1	37.1	6.25	41	3	0.073
-34.4	37.4	6.25	46	3	0.065
-34.6	37.6	6.00	43	3	0.070
-34.9	37.9	6.25	54	3	0.056
-35.0	38.0	6.00	46	3	0.065

[a] 9 blows with no explosion. Stopped 2 minutes.

[b] Blow count averaged to be 84 blows per foot.

[c] Stopped 10 minutes.

[d] Stopped 3 minutes.

[e] Stopped 5 minutes.

TABLE III-7. - SUMMARY OF INPUT DATA FOR PA1-INITIAL

Hammer Properties

Type: Link Belt 520

Rated energy: 26,300 ft-lb

Hammer efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 14.70 fps

Ram weight: 5.07 kips

Ram stiffness: 108,500 kips/in.,

e = 0.6 (steel on steel impact)

Anvil weight: 1.179 kips

Adapter weight: 1.05 kips

Capblock: Alternating aluminum and plastic

disks (enclosed). K_c = 18,600 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 3/8-in. wall, closed end steel pipe

Pile length: 67 ft

Embedded length: 64 ft

Segment length: 5 ft or 2 ft

Segment weight: 0.313 kip or 0.1252 kip

Segment stiffness: 9,080 kips/in. or 22,700 kips/in.

TABLE III-7. - SUMMARY OF INPUT DATA FOR
PA1-INITIAL (CONTINUED)

e = 1.0 except for first pile segment (steel on steel impact), e = 0.6

Soil Properties

Type: Uniform clay

RUT: 92.4 kips

RUP: 18.0 kips

Load distribution: 0.805 RUT uniform side load, 0.195 RUT at pile tip

Final blow count: 16 blows per foot

TABLE III-8. - SUMMARY OF INPUT DATA FOR PA1-FINAL

Hammer Properties

Type: Link Belt 520

Rated energy: 26,300 ft-lb

Hammer efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 15.92 fps

Ram weight: 5.07 kips

Ram stiffness: 108,500 kips/in.,

e = 0.6 (steel on steel impact)

Anvil weight: 1.179 kips

Adapter weight: 1.05 kips

Capblock: Alternating aluminum and plastic

disks (enclosed). K_c = 18,600 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 3/8-in. wall, closed end steel pipe

Pile length: 67 ft

Embedded length: 64 ft

Segment length: 5 ft or 2 ft

Segment weight: 0.313 kip or 0.1252 kip

Segment stiffness: 9,080 kips/in. or 22,700 kips/in.

TABLE III-8. - SUMMARY OF INPUT DATA FOR PA1-FINAL (CONTINUED)

e = 1.0 except for first pile segment (steel on steel impact), e = 0.6

Soil Properties

Type: Uniform clay

RUT: 200 kips

RUP: 10 kips

Load distribution: 0.95 RUT uniform side load, 0.05 RUT at pile tip

Final blow count: 72 blows per foot

TABLE III-9. - SUMMARY OF INPUT DATA FOR PA2-INITIAL

Hammer Properties

Type: Link Belt 520

Rated energy: 26,300 ft-lb

Hammer efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 15.16 fps

Ram weight: 5.07 kips

Ram stiffness: 108,500 kips/in.,

e = 0.6 (steel on steel impact)

Anvil weight: 1.179 kips

Adapter weight: 1.05 kips

Capblock: Alternating aluminum and plastic

disks (enclosed). K_c = 18,600 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 3/8-in. wall, closed end steel pipe

Pile length: 78 ft

Embedded length: 74 ft

Segment length: 5 ft or 2 ft

Segment weight: 0.313 kip or 0.1252 kip

Segment stiffness: 9,080 kips/in. or 22,700 kips/in.

TABLE III-9. - SUMMARY OF INPUT DATA FOR

PA2-INITIAL (CONTINUED)

e = 1.0 except for first pile segment (steel on steel impact), e = 0.6

Soil Properties

Type: Uniform clay

RUT: 100.2 kips

RUP: 16.0 kips

Load distribution: 0.84 RUT uniform side load, 0.16 RUT at pile tip

Final blow count: 18 blows per foot

TABLE III-10. - SUMMARY OF INPUT DATA FOR PA2-FINAL

Hammer Properties

Type: Link Belt 520

Rated energy: 26,300 ft-lb

Hammer efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 15.62 fps

Ram weight: 5.07 kips

Ram stiffness: 108,500 kips/in.,

$e = 0.6$ (steel on steel impact)

Anvil weight: 1.179 kips

Adapter weight: 1.05 kips

Capblock: Alternating aluminum and plastic

disks (enclosed). $K_c = 18{,}600$ kips/in., $e = 0.8$

Cushion: None

Pile Properties

Type: 16-in. OD, 3/8-in. wall, closed end steel pipe

Pile length: 78 ft

Embedded length: 74 ft

Segment length: 5 ft or 2 ft

Segment weight: 0.313 kip or 0.1252 kip

Segment stiffness: 9,080 kips/in. or 22,700 kips/in.

TABLE III-10. - SUMMARY OF INPUT DATA FOR PA2-FINAL (CONTINUED)

e = 1.0 except for first pile segment (steel on steel impact), e = 0.6

Soil Properties

Type: Uniform clay

RUT: 244 kips

RUP: 20 kips

Load distribution: 0.918 RUT uniform side load, 0.082 RUT at pile tip

Final blow count: 200 blows per foot

TABLE III-11. - SUMMARY OF INPUT DATA FOR CC-INITIAL

Hammer Properties

Type: Delmag D-22

Rated energy: 39,700 ft-lb

Hammer efficiency (%): 100

Explosive force: 158.7 kips

Ram velocity: 17.1 fps

Ram weight: 4.85 kips

Ram stiffness: 49,700 kips/in.,

e = 0.6 (steel on steel impact)

Anvil weight: 1.576 kips

Helmet weight: 1.3 kips

Capblock: Oak block, 18-in. x 18-in. x 9-in. thick (grain vertical), K_c = 23,800 kips/in., e = 0.8

Cushion: 7 sheets of 3/4-in. plywood fir, K_c = 1,705 kips/in., K_{c+p} = 1,595 kips/in., e = 0.5

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 38 ft

Embedded length: 28.7 ft

Segment length: 5 ft or 2 ft

Segment weight: 1.29 kips or 0.516 kip

Segment stiffness: 23,900 kips/in. or 59,750 kips/in.,

TABLE III-11. - SUMMARY OF INPUT DATA FOR

CC-INITIAL (CONTINUED)

e = 1.0

Soil Properties

Type: Uniform sand (SM-SP)

RUT: 276.4 kips

RUP: 212.8 kips

Load distribution: 0.232 RUT uniform side load,

0.768 RUT at pile tip

Final blow count: 48 blows per foot

TABLE III-12. - SUMMARY OF INPUT DATA FOR CC-FINAL

Hammer Properties

Type: Delmag D-22

Rated energy: 39,700 ft-lb

Hammer efficiency (%): 100

Explosive force: 158.7 kips

Ram velocity: 17.3 fps

Ram weight: 4.85 kips

Ram stiffness: 49,700 kips/in.,

$e = 0.6$ (steel on steel impact)

Anvil weight: 1.576 kips

Helmet weight: 1.3 kips

Capblock: Oak block, 18-in. x 18-in. x 9-in. thick (grain vertical), $K_c = 23{,}800$ kips/in., $e = 0.8$

Cushion: 7 sheets of 3/4-in. plywood fir, $K_c = 1{,}705$ kips/in., $K_{c+p} = 1{,}595$ kips/in., $e = 0.5$

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 38 ft

Embedded length: 28.7 ft

Segment length: 5 ft or 2 ft

Segment weight: 1.29 kips or 0.516 kip

Segment stiffness: 23,900 kips/in. or 59,750 kips/in.,

TABLE III-12. - SUMMARY OF INPUT DATA FOR
CC-FINAL (CONTINUED)

$e = 1.0$

Soil Properties

Type: Uniform soil (SM-SP)

RUT: 322.2 kips

RUP: 224.4 kips

Load distribution: 0.304 RUT uniform side load,
0.696 RUT at pile tip

Final blow count: 84 blows per foot

APPENDIX IV. - SUMMARIES OF INPUT DATA AND RUT VS BLOW COUNT CURVES FOR ADDITIONAL CASE STUDIES

TABLE IV-1. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 1

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. $K_c = 10,125$ kips/in., $e = 0.8$

Cushion: None

Pile Properties

Type: 12.75-in. OD, 0.33-in. wall, closed end steel pipe

Pile length: 55 ft

Embedded length: 53.1 ft

Segment length: 5 ft

Segment weight: 0.295 kips

Segment stiffness: 8,280 kips/in., $e = 1.0$ except for first segment (steel on steel impact), $e = 0.6$

TABLE IV-1. - SUMMARY OF INPUT DATA FOR
ARKANSAS LTP 1 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 336 kips

RUP: 91 kips

Load distribution: 0.73 RUT uniform side load,
0.27 RUT at pile tip

Final blow count: 16 blows/ft

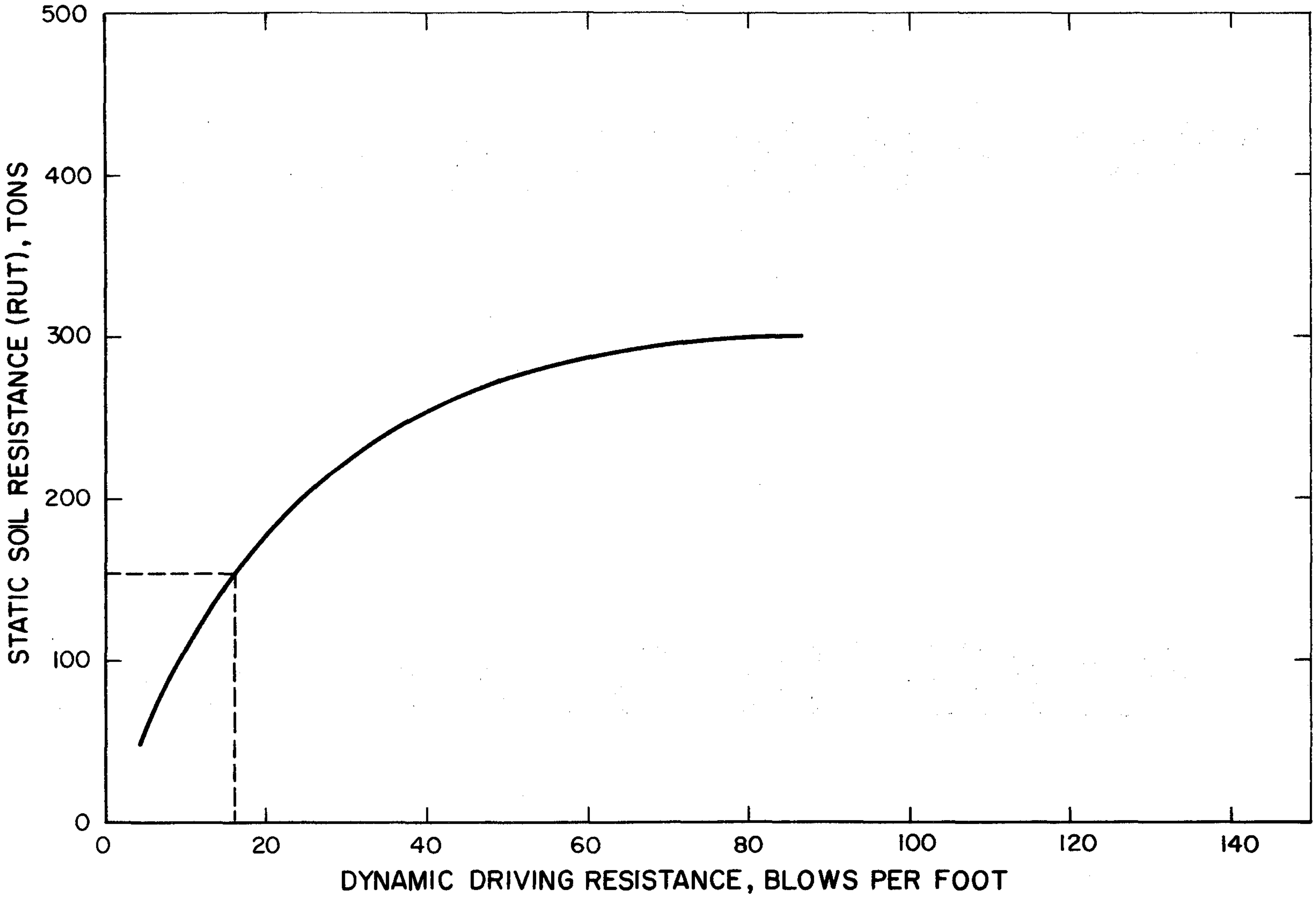

FIG. IV-1. — RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 1

TABLE IV-2. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 2

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. K_c = 10,125 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 0.312-in. wall, closed end steel pipe

Pile length: 55 ft

Embedded length: 52.8 ft

Segment length: 5 ft

Segment weight: 0.410 kips

Segment stiffness: 11,533 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-2. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 2 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 446 kips

RUP: 134 kips

Load distribution: 0.70 RUT uniform side load,

0.30 RUT at pile tip

Final blow count: 38 blows/ft

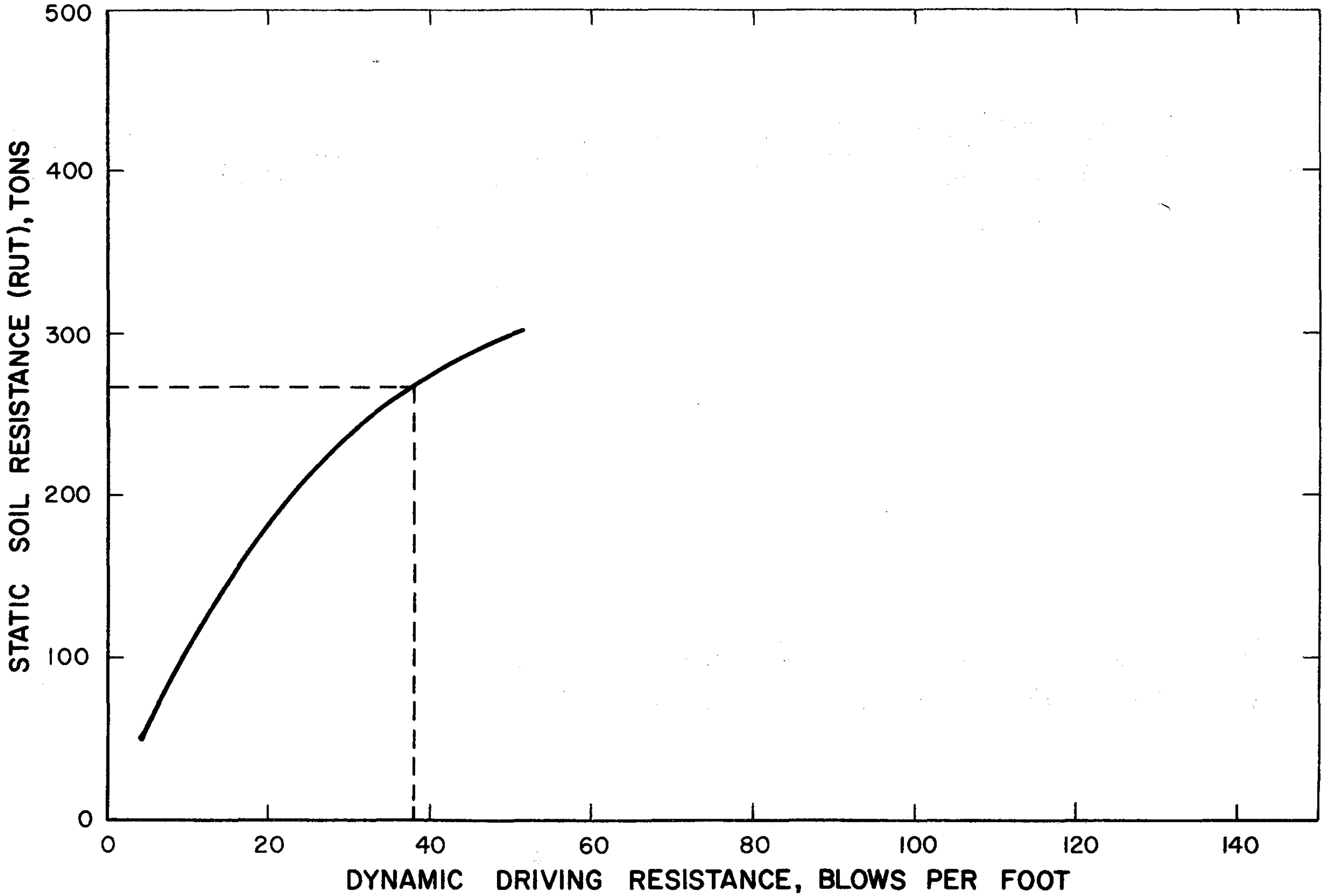

FIG. IV-2.-RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 2

TABLE IV-3. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 3

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. K_c = 10,125 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 20-in. OD, 0.375-in. wall, closed end steel pipe

Pile length: 55 ft

Embedded length: 53 ft

Segment length: 5 ft

Segment weight: 0.468 kips

Segment stiffness: 13,217 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-3. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 3 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 494 kips

RUP: 192 kips

Load distribution: 0.61 RUT uniform side load,

0.39 RUT at pile tip

Final blow count: 44 blows/ft

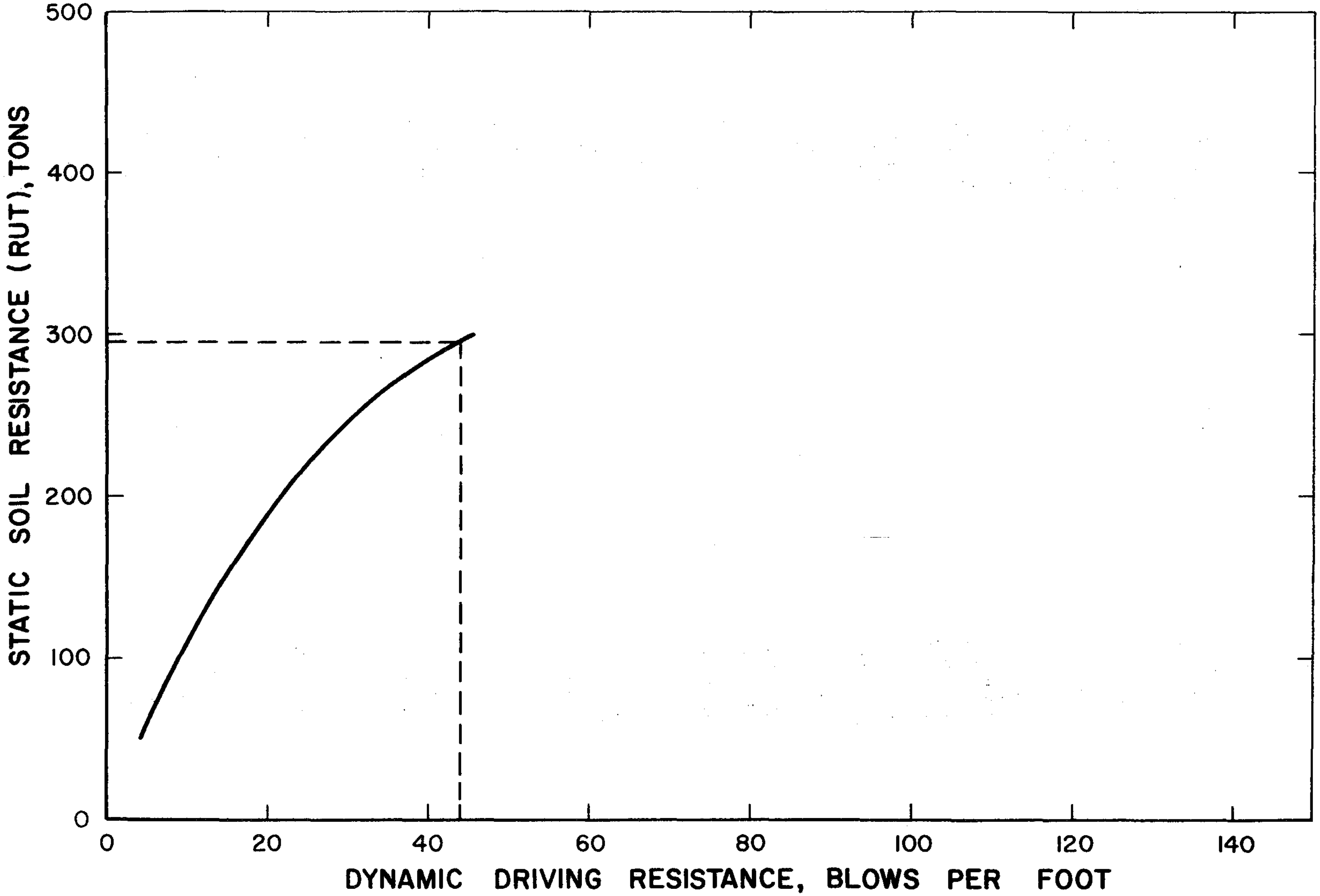

FIG. IV-3.-RUT vs. BLOW CURVE FOR ARKANSAS LTP 3

TABLE IV-4. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 4

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. K_c = 10,125 kips/in. e = 0.8

Cushion: 3 to 5 sheets of 3/4-in. plywood + 1 pad of 1-in. manila rope. K_c = 1,920 kips/in. K_{c+p} = 1,790 kips/in., e = 0.3

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 45 ft

Embedded length: 40.2 ft

Segment length: 5 ft

Segment weight: 1.33 kips

Segment stiffness: 25,800 kips/in., e = 1.0

TABLE IV-4. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 4 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 396 kips

RUP: 222 kips

Load distribution: 0.44 RUT uniform side load,

0.56 RUT at pile tip

Final blow count: 22 blows/ft

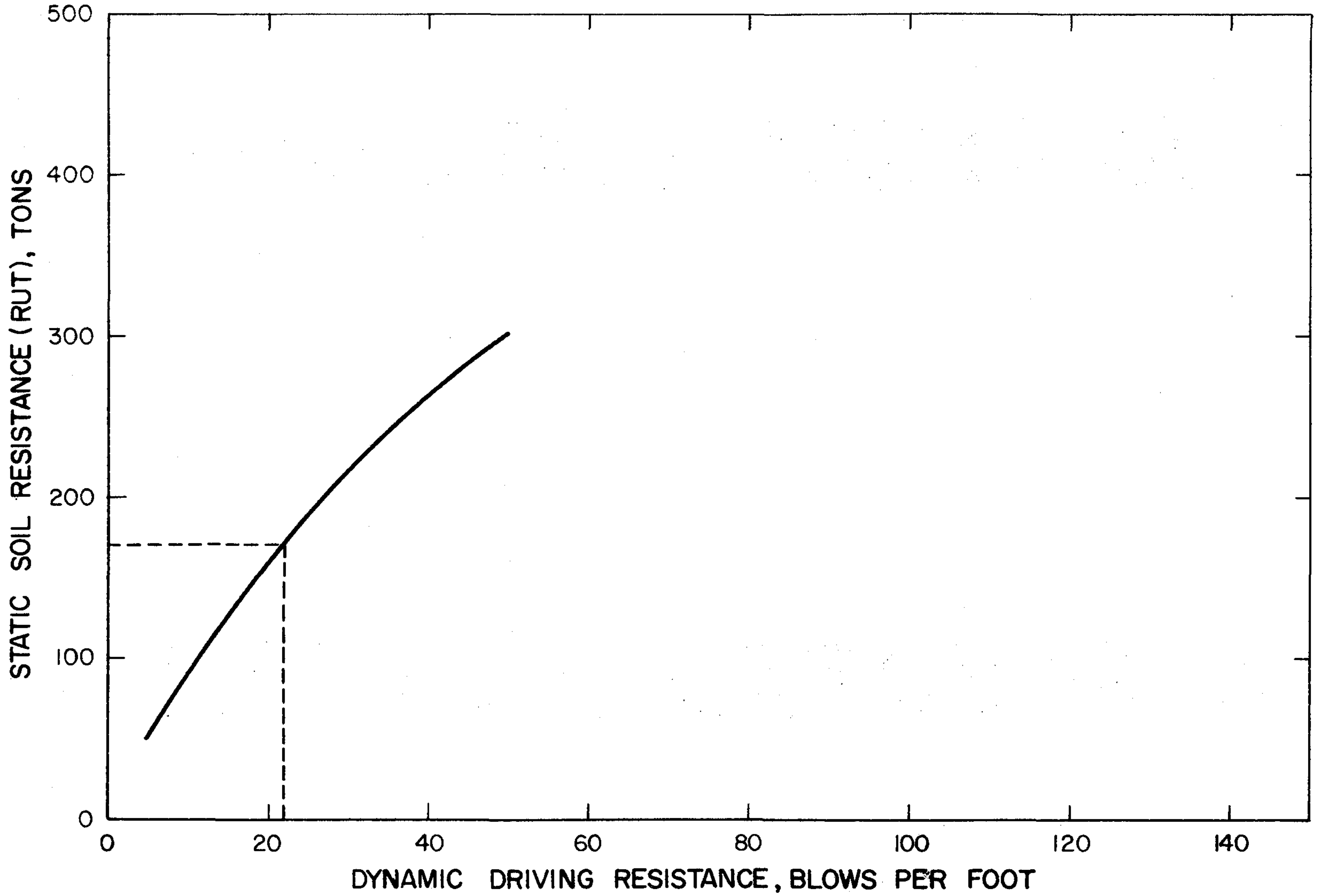

FIG. IV-4. — RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 4

TABLE IV-5. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 5

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. K_c = 10,125 kips/in. e = 0.8

Cushion: 3 to 5 sheets of 3/4-in. plywood + 1 pad of 1-in. manila rope. K_c = 1,920 kips/in. K_{c+p} = 1,790 kips/in., e = 0.3

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 55 ft

Embedded length: 51 ft

Segment length: 5 ft

Segment weight: 1.33 kips

Segment stiffness: 25,800 kips/in., e = 1.0

TABLE IV-5. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 5 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 556 kips

RUP: 278 kips

Load distribution: 0.50 RUT uniform side load,

0.50 RUT at pile tip

Final blow count: 48 blows/ft

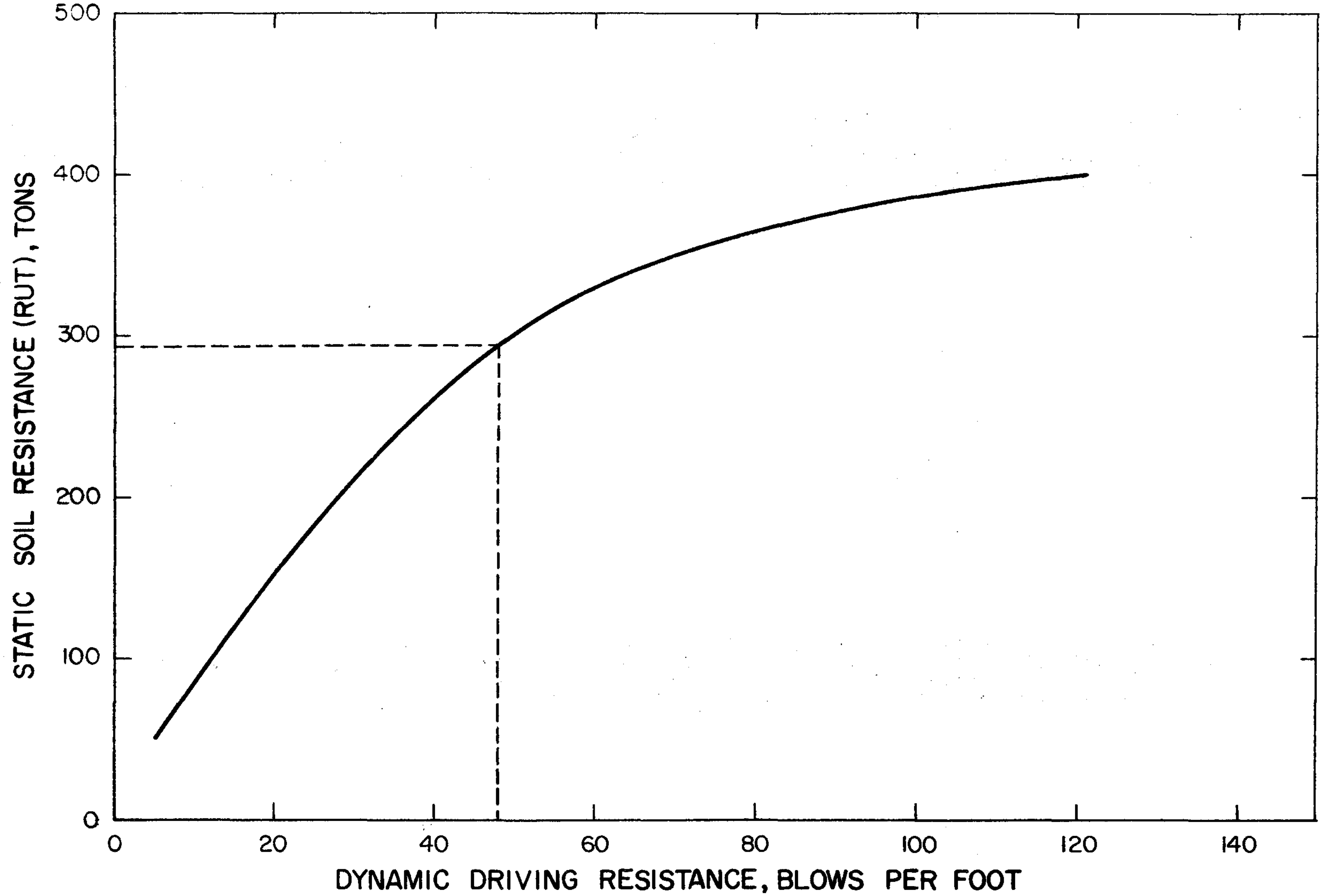

FIG. IV - 5. — RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 5

TABLE IV-6. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 6

Hammer Properties

Type: Vulcan 80C

Rated energy: 24,450 ft-lb

Efficiency (%): 84

Ram velocity: 12.8 fps

Ram weight: 8 kips

Helmet weight: 1.22 kips

Capblock: Alternating 11 aluminum disks 1/2-in. thick and 11 micarta disks 1-in. thick, 14 3/8-in. diameter with 3 1/2-in. bore. K_c = 7,920 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: Steel-H (14BP73)

Pile length: 42 ft

Embedded length: 40 ft

Segment length: 5 ft

Segment weight: 0.44 kips

Segment stiffness: 12,420 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-6. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 6 (CONTINUED)

<u>Soil Properties</u>

Type: Partially saturated sand (SM-SP)

RUT: 350 kips

RUP: 80 kips

Load distribution: 0.77 RUT uniform side load,

0.23 RUT at pile tip

Final blow count: 17 blows/ft

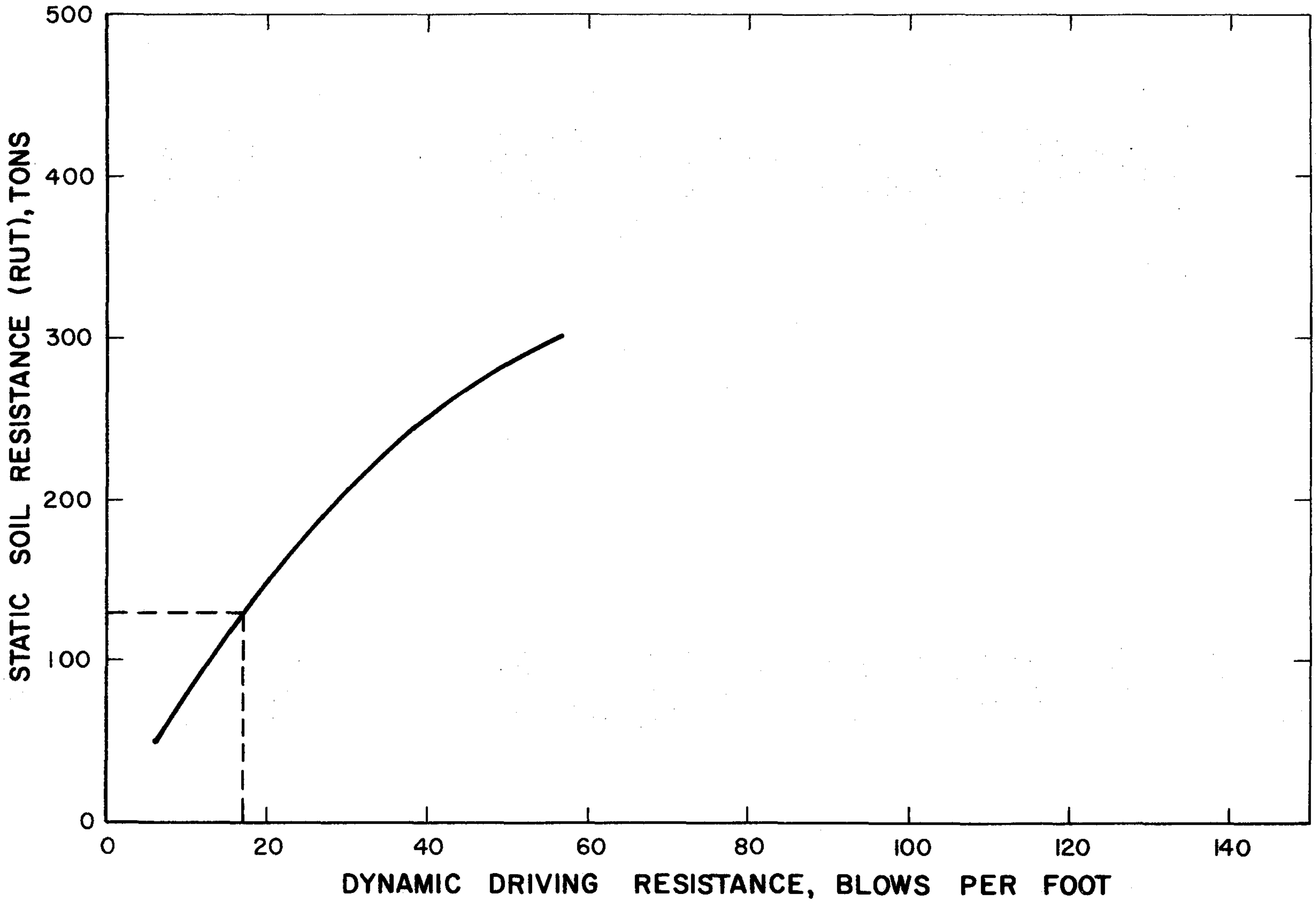

FIG. IV-6.-RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 6

TABLE IV-7. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 7

Hammer Properties

Type: Vulcan 80C

Rated energy: 24,450 ft-lb

Efficiency (%): 84

Ram velocity: 12.8 fps

Ram weight: 8 kips

Helmet weight: 1.22 kips

Capblock: Alternating 11 aluminum disks 1/2-in. thick and 11 micarta disks 1-in. thick, 14 3/8-in. diameter with 3 1/2-in. bore. K_c = 7,920 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: Steel-H (14BP73)

Pile length: 55 ft

Embedded length: 52.1 ft

Segment length: 5 ft

Segment weight: 0.505 kips

Segment stiffness: 14,200 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-7. - SUMMARY OF INPUT DATA FOR

ARKANSAS LTP 7 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 440 kips

RUP: 101 kips

Load distribution: 0.77 RUT uniform side load,

0.23 RUT at pile tip

Final blow count: 31 blows/ft

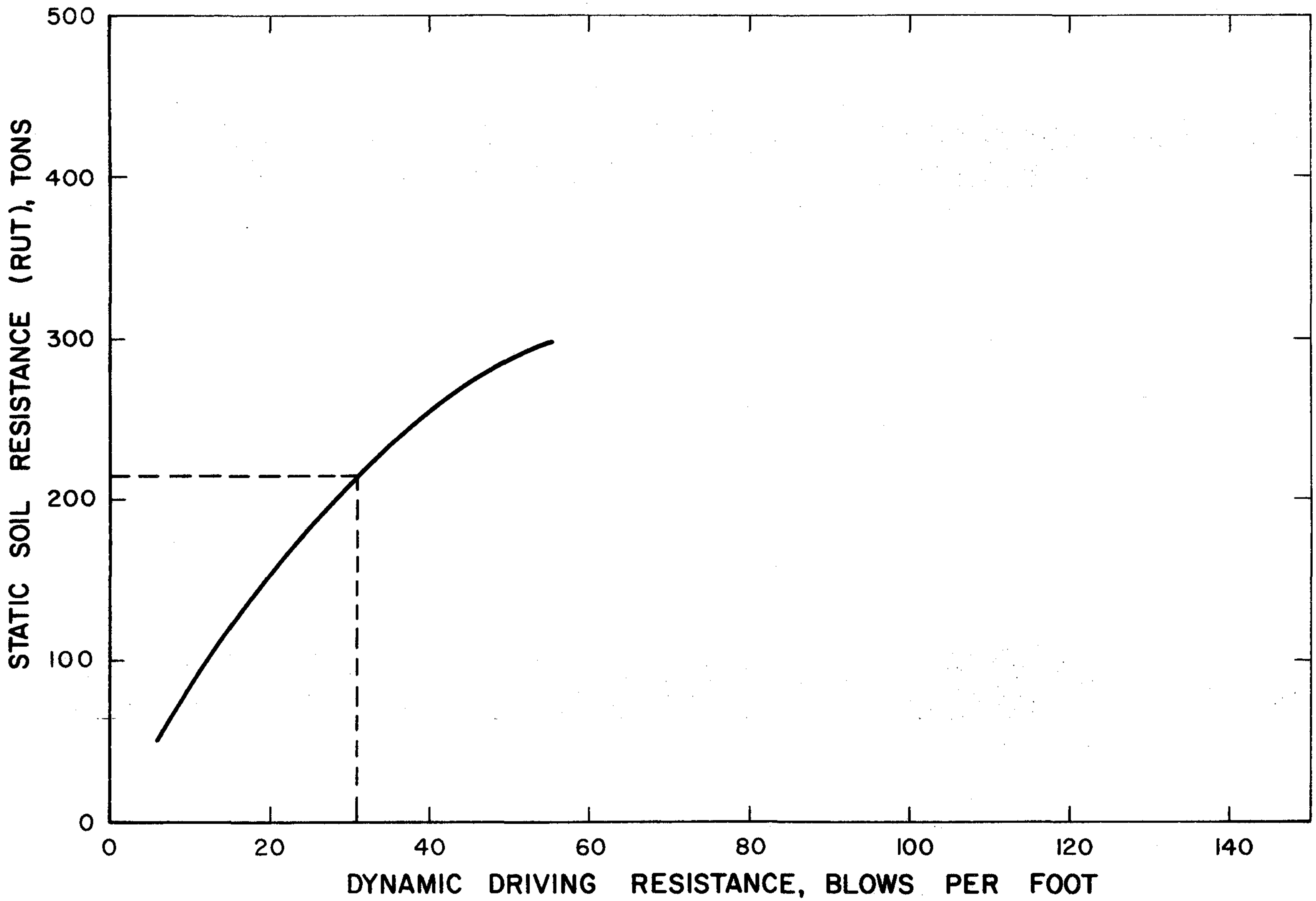

FIG. IV-7.-RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 7

TABLE IV-8. - SUMMARY OF INPUT DATA FOR ARKANSAS LTP 16

Hammer Properties

Type: Vulcan 140C

Rated energy: 36,000 ft-lb

Efficiency (%): 78

Ram velocity: 11.6 fps

Ram weight: 14 kips

Helmet weight: 1.71 kips

Capblock: Alternating 10 aluminum disks 1/2-in. thick and 10 micarta disks 1-in. thick, 17 1/2-in. diameter with 4 1/2-in. bore. K_c = 10,125 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 0.312-in. wall, closed end steel pipe (jetted 40 ft)

Pile length: 55 ft

Embedded length: 52.7 ft

Segment length: 5 ft

Segment weight: 0.337 kips

Segment stiffness: 9,483 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-8. - SUMMARY OF INPUT DATA FOR
ARKANSAS LTP 16 (CONTINUED)

Soil Properties

Type: Partially saturated sand (SM-SP)

RUT: 336 kips

RUP: 101 kips

Load distribution: 0.70 RUT uniform side load,
0.30 RUT at pile tip

Final blow count: 24 blows/ft

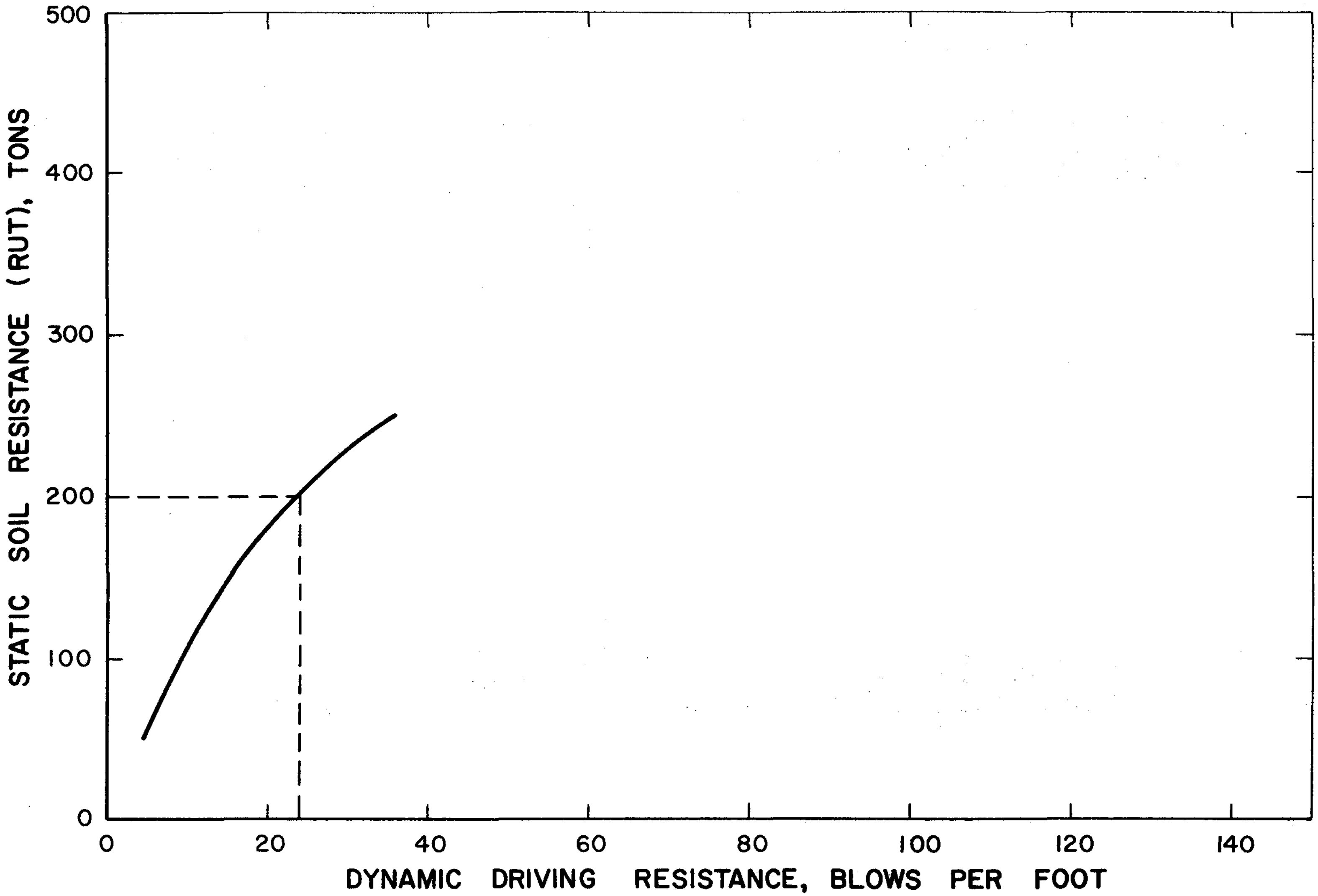

FIG. IV-8.- RUT vs. BLOW COUNT CURVE FOR ARKANSAS LTP 16

TABLE IV-9. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 1

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, open end steel pipe

Pile length: 45.5 ft

Embedded length: 44.4 ft

Segment length: 5 ft

Segment weight: 0.157 kips

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-9. - SUMMARY OF INPUT DATA FOR

BELLEVILLE LTP 1 (CONTINUED)

<u>Soil Properties</u>

Type: Firm clay, pile tip in plastic clay

RUT: 100 kips

RUP: 0 kips

Load distribution: 0.94 RUT uniform side load,

0.06 RUT uniform core load (3.1 ft clay core)

Final blow count: 132 blows/ft

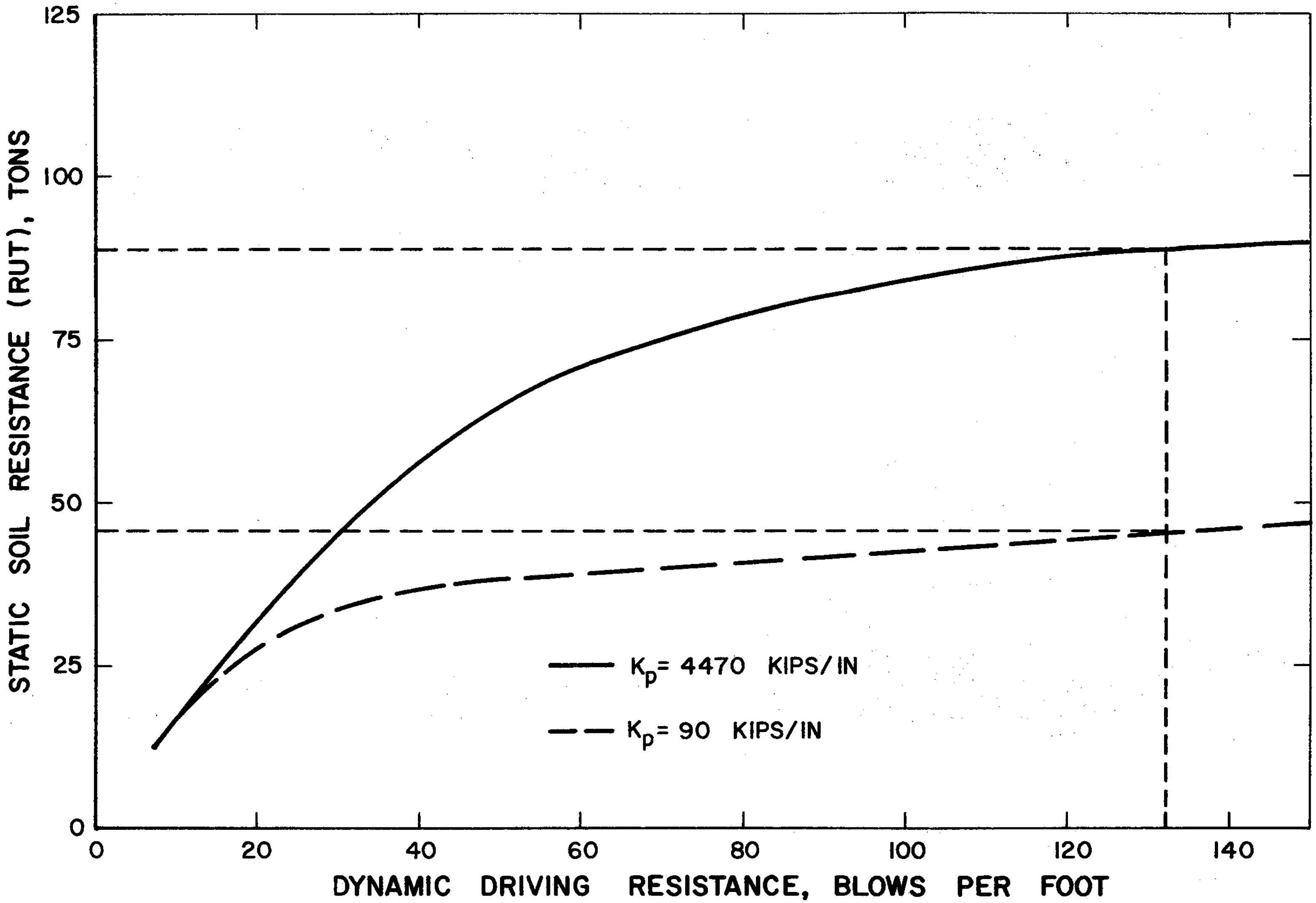

FIG. IV-9.-RUT vs. BLOW COUNT CURVES FOR BELLEVILLE LTP I

TABLE IV-10. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 3

Hammer Properties

Type: MKT DE-30

Rated energy: 22,400 ft-lb

Efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 17.94 fps

Ram weight: 2.8 kips

Ram stiffness: 38,700 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 0.774 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 18 1/2-in. diameter by 2 1/4-in. thick. K_c = 83,600 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, #7 gage, fluted tapered, closed end steel pipe

Pile length: 62 ft

Embedded length: 50.9 ft

Segment length: 5 ft

Segment weight: 0.113 kips

TABLE IV-10. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 3 (CONTINUED)

Segment stiffness: 3,220 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

Soil Properties

Type: 35 ft firm clay, 15 ft partially saturated fine sand and silt

RUT: 324 kips

RUP: 185 kips

Load distribution: 0.11 RUT in firm clay layer, 0.32 RUT in sand and silt layer, 0.57 RUT at pile tip

Final blow count: 696 blows/ft

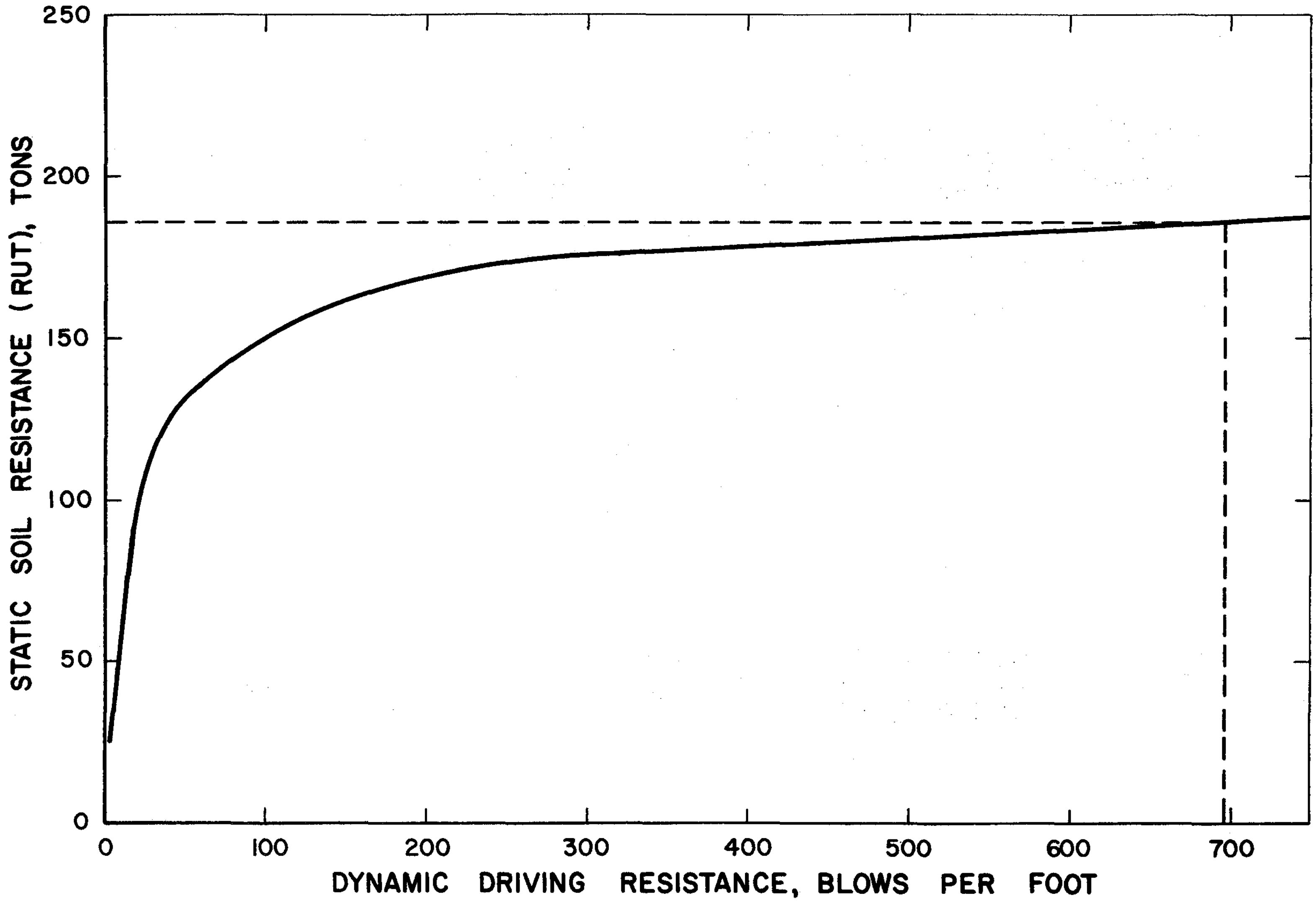

FIG. IV-10.-RUT vs. BLOW COUNT CURVE FOR BELLEVILLE LTP 3

TABLE IV-11. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 4

Hammer Properties

Type: Link Belt 312

Rated energy: 18,000 ft-lb

Efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 15.84 fps

Ram weight: 3.855 kips

Ram stiffness: 142,500 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.188 kips

Helmet weight: 1.381 kips

Capblock: Alternating layers of 5 micarta fiber plates, 11-in. diameter by 1/2-in. thick. K_c = 17,100 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, closed end steel pipe

Pile length: 67.6 ft

Embedded length: 56.5 ft

Segment length: 5 ft

Segment weight: 0.157 kips

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-11. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 4 (CONTINUED)

Soil Properties

Type: 40 ft firm clay, 15 ft partially saturated sand and silt, pile tip in hardpan

RUT: 661 kips

RUP: 483 kips

Load distribution: 0.05 RUT in firm clay layer, 0.22 RUT in fine sand and silt layer, 0.73 RUT at pile tip

Final blow count: 720 blows/ft

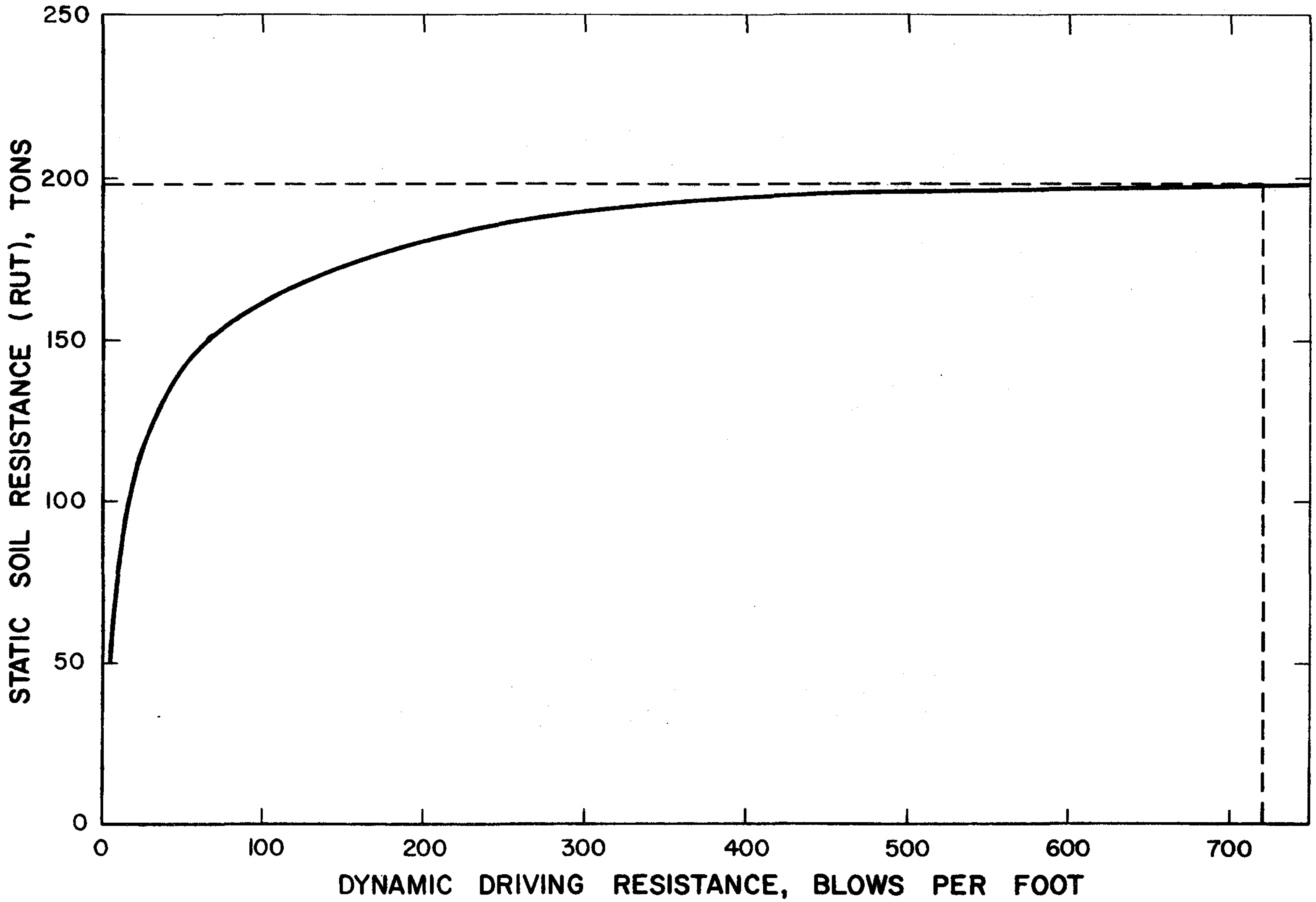

FIG. IV-11.-RUT vs. BLOW COUNT CURVE FOR BELLEVILLE LTP 4

TABLE IV-12. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 5

Hammer Properties

Type: Delmag D-12

Rated energy: 22,600 ft-lb

Efficiency (%): 100

Explosive force: 93.7 kips

Ram velocity: 21 fps

Ram weight: 2.75 kips

Ram stiffness: 31,500 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 0.754 kips

Helmet weight: 1 kip

Capblock: German Oak block (grain vertical), 15-in. x 15-in. x 5-in. thick under steel block, 15-in. x 15-in. x 3-in. thick. K_c = 31,500 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, #7 gage, closed end steel pipe

Pile length: 67.8 ft

Embedded length: 56.7 ft

Segment length: 5 ft

Segment weight: 0.113 kips

Segment stiffness: 3,220 kips/in., e = 1.0 except for

TABLE IV-12. - SUMMARY OF INPUT DATA FOR

BELLEVILLE LTP 5 (CONTINUED)

first segment (steel on steel impact), e = 0.6

Soil Properties

Type: 40 ft firm clay, 15 ft partially saturated sand and silt, pile tip in hardpan

RUT: 661 kips

RUP: 483 kips

Load distribution: 0.05 RUT in firm clay layer, 0.22 RUT in fine sand and silt layer, 0.73 RUT at pile tip

Final blow count: 408 blows/ft

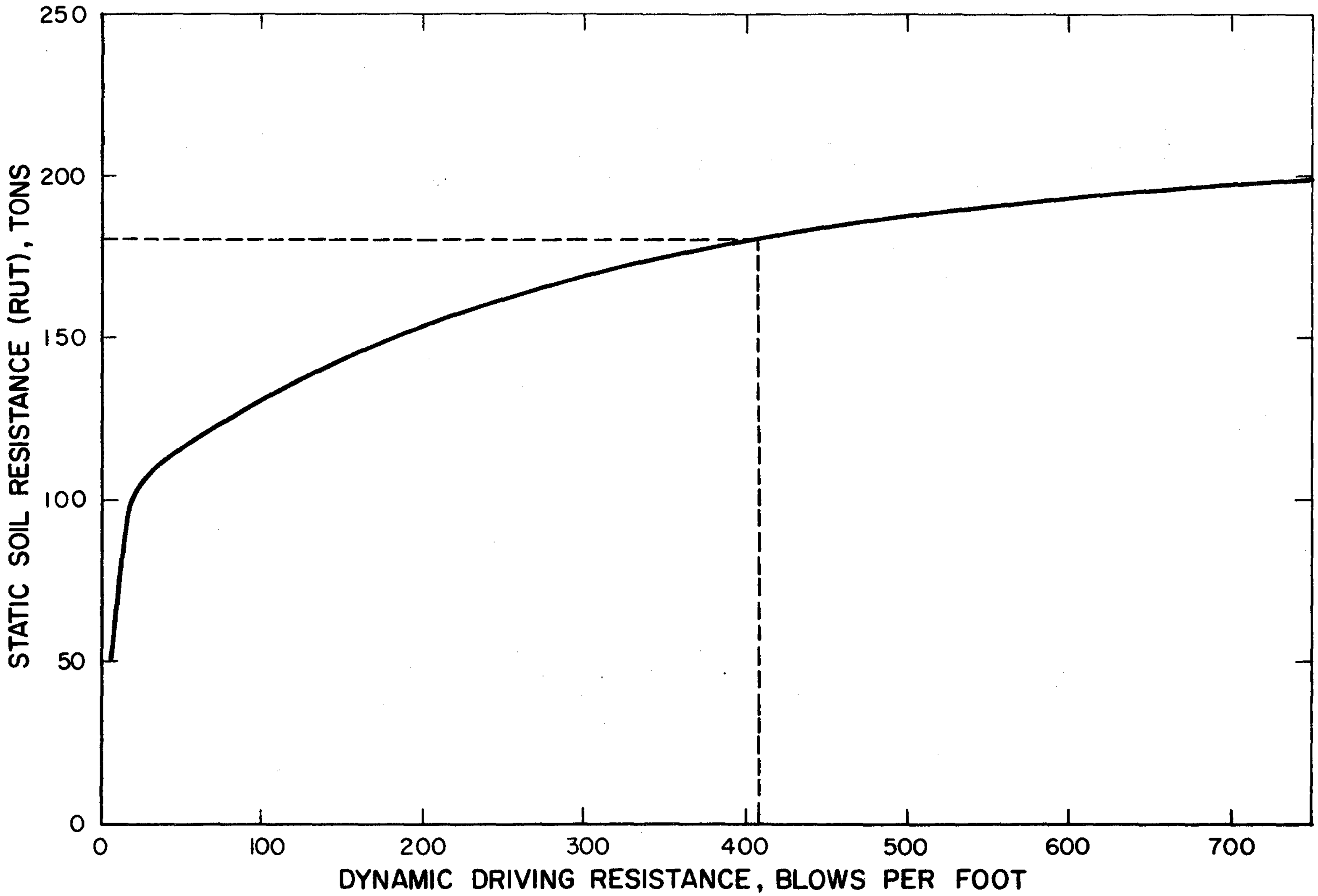

FIG. IV-12.— RUT vs. BLOW COUNT CURVE FOR BELLEVILLE LTP 5

TABLE IV-13. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 6

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. x 12-in. H-pile

Pile length: 59.1 ft

Embedded length: 58 ft

Segment length: 5 ft

Segment weight: 0.265 kips

Segment stiffness: 7,530 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-13. - SUMMARY OF INPUT DATA FOR BELLEVILLE LTP 6 (CONTINUED)

<u>Soil Properties</u>

Type: 50 ft firm clay, 10 ft partially saturated fine sand and silt

RUT: 292 kips

RUP: 18 kips

Load distribution: 0.44 RUT in firm clay layer, 0.50 RUT in fine sand and silt layer, 0.06 RUT at pile tip

Final blow count: 588 blows/ft

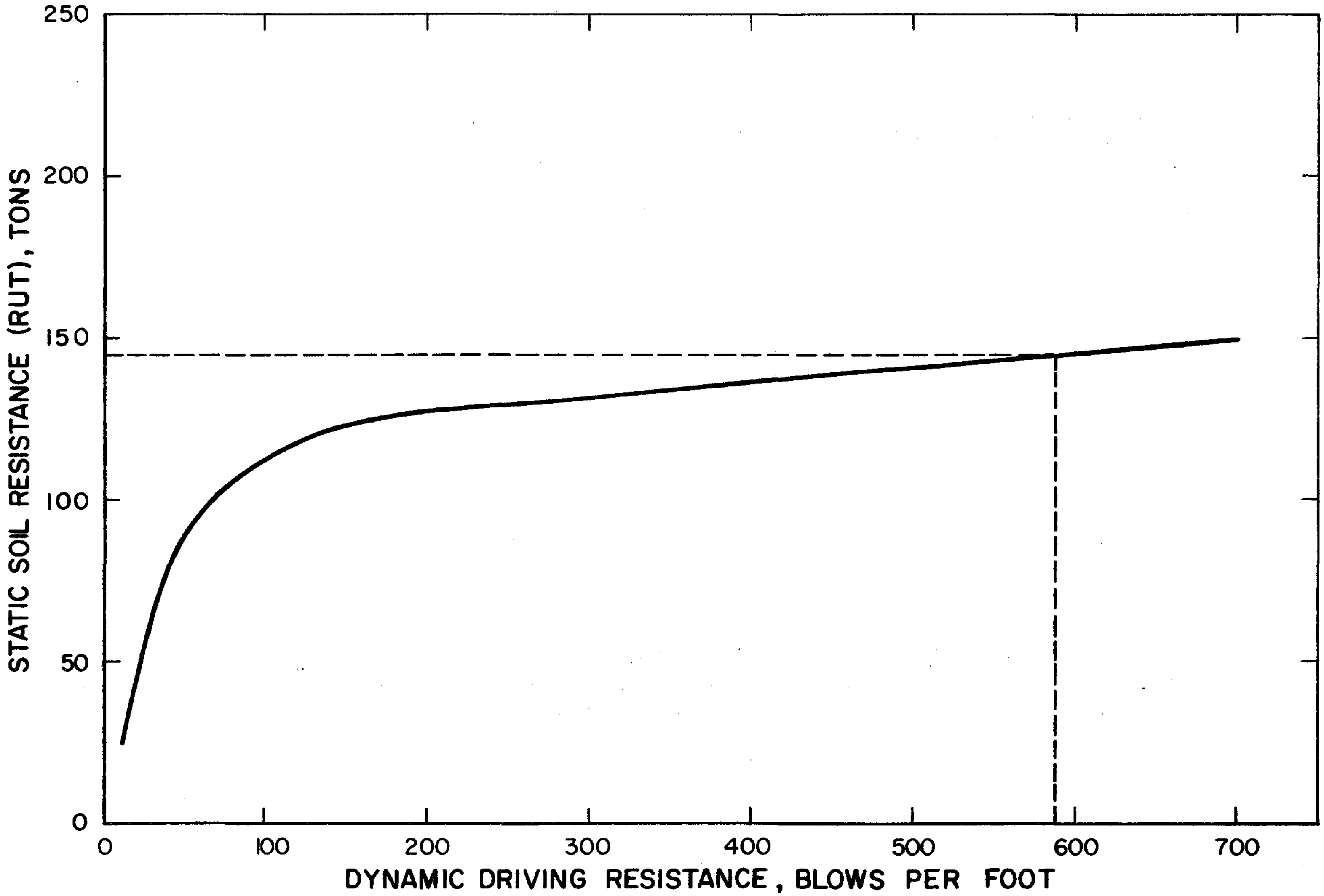

FIG. IV-13.—RUT vs. BLOW COUNT CURVE FOR BELLEVILLE LTP 6

TABLE IV-14. - SUMMARY OF INPUT DATA FOR DETROIT LTP 1

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, #7 gage, open end steel pipe

Pile length: 71.9 ft

Embedded length: 69.5 ft

Segment length: 5 ft

Segment weight: 0.113 kips

Segment stiffness: 3,220 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-14. - SUMMARY OF INPUT DATA FOR

DETROIT LTP 1 (CONTINUED)

Soil Properties

Type: Soft clay

RUT: 40 kips

RUP: 0 kips

Load distribution: 0.64 RUT uniform side load,

0.36 RUT uniform core load (33.7 ft clay core)

Final blow count: 22 blows/ft

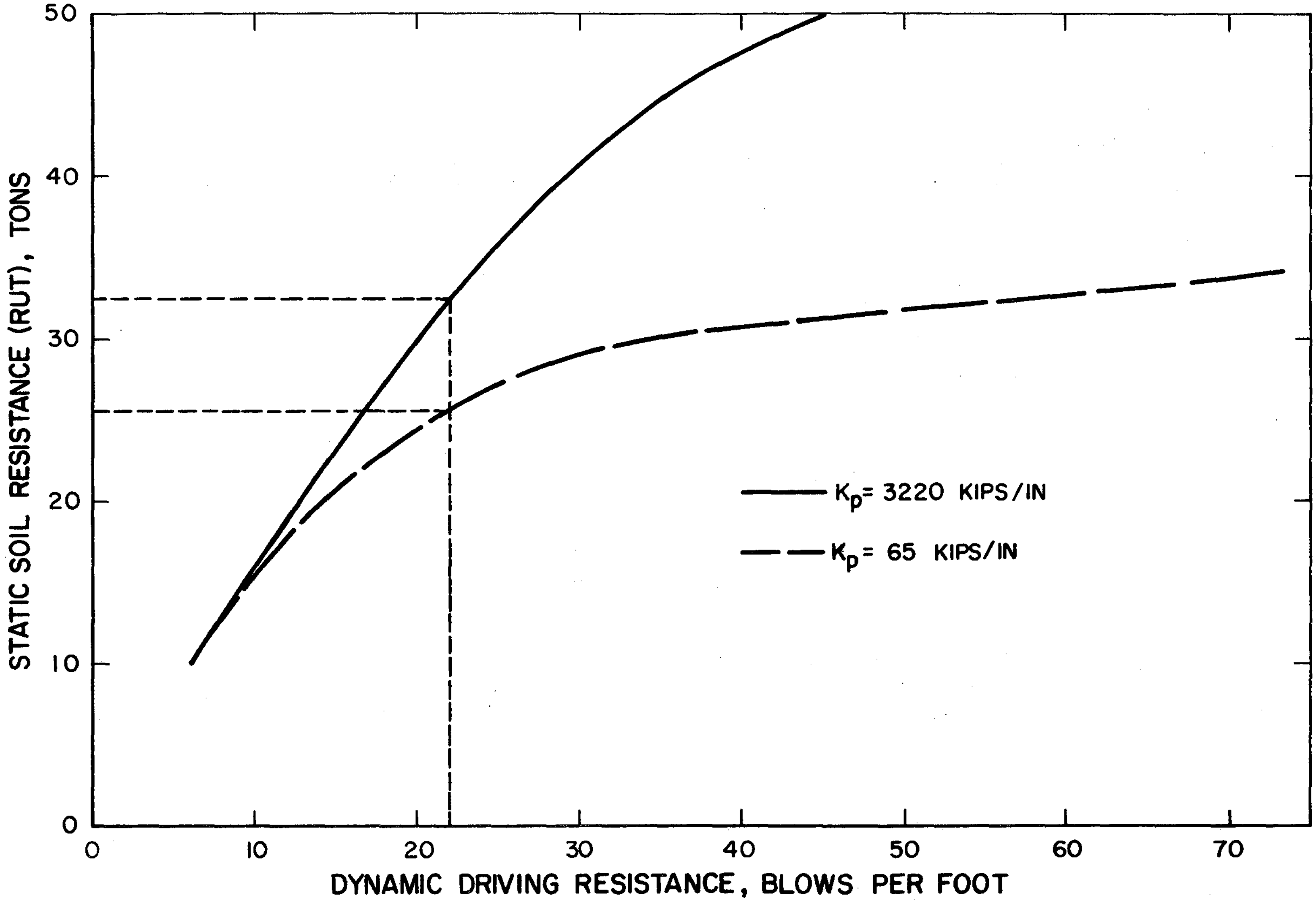

FIG. IV-14.—RUT vs. BLOW COUNT CURVES FOR DETROIT LTP I

TABLE IV-15. - SUMMARY OF INPUT DATA FOR DETROIT LTP 2

<u>Hammer Properties</u>

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

<u>Pile Properties</u>

Type: 12-in. OD, #7 gage, closed end steel pipe

Pile length: 80.7 ft

Embedded length: 78.6 ft

Segment length: 5 ft

Segment weight: 0.113 kips

Segment stiffness: 3,220 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-15. - SUMMARY OF INPUT DATA FOR

DETROIT LTP 2 (CONTINUED)

Soil Properties

Type: Soft clay, pile tip in hardpan

RUT: 277 kips

RUP: 255 kips

Load distribution: 0.08 RUT uniform side load,

0.92 RUT at pile tip

Final blow count: 432 blows/ft

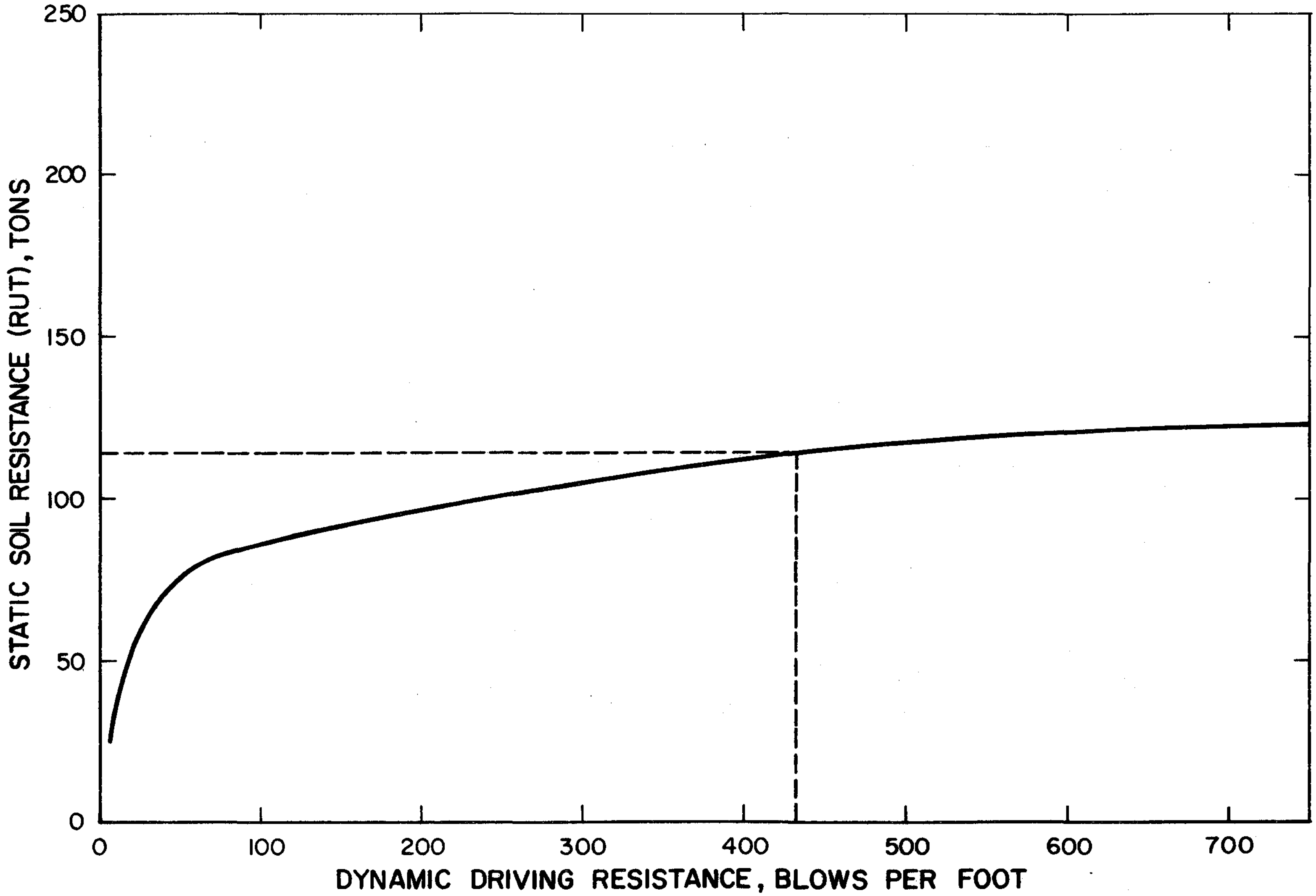

FIG. IV-15.—RUT vs. BLOW COUNT CURVE FOR DETROIT LTP 2

TABLE IV-16. - SUMMARY OF INPUT DATA FOR DETROIT LTP 7

Hammer Properties

Type: MKT DE-30

Rated energy: 22,400 ft-lb

Efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 17.94 fps

Ram weight: 2.8 kips

Ram stiffness: 38,700 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 0.774 kips

Helmet weight: 1.4 kips

Capblock: Oak block (grain vertical), 18 1/2-in. diameter by 2 1/4-in. thick. K_c = 83,600 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, #7 gage, fluted tapered, closed end steel pipe

Pile length: 83.2 ft

Embedded length: 81.1 ft

Segment length: 5 ft

Segment weight: 0.113 kips

TABLE IV-16. - SUMMARY OF INPUT DATA FOR
DETROIT LTP 7 (CONTINUED)

Segment stiffness: 3,220 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

Soil Properties

Type: 75 ft soft clay, 5 ft hardpan

RUT: 272 kips

RUP: 215 kips

Load distribution: 0.16 RUT in soft clay layer, 0.05 RUT in hardpan layer, 0.79 RUT at pile tip

Final blow count: 360 blows/ft

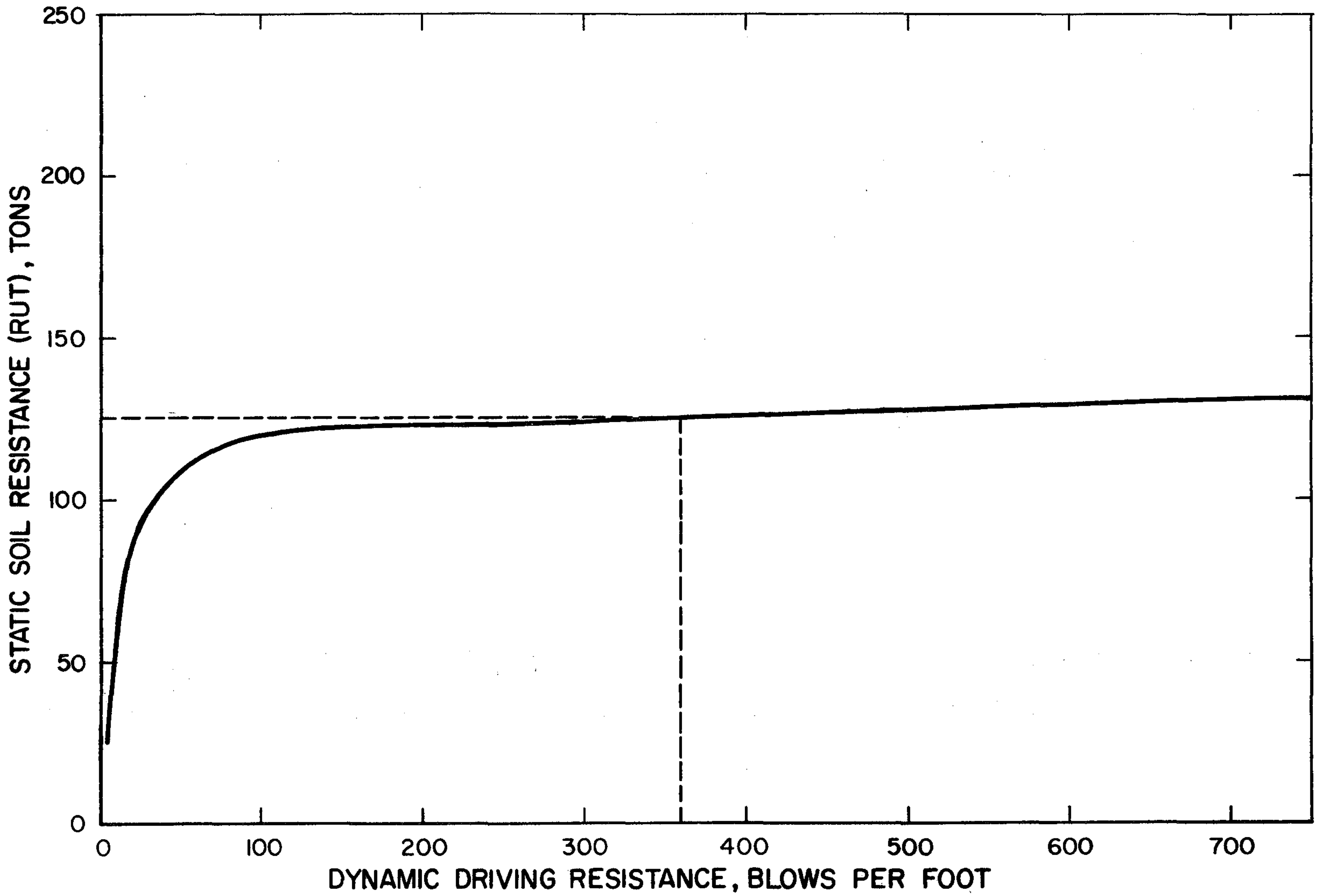

FIG. IV-16.— RUT vs. BLOW COUNT CURVE FOR DETROIT LTP 7

TABLE IV-17. - SUMMARY OF INPUT DATA FOR DETROIT LTP 8

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. x 12-in. H-pile

Pile length: 83.3 ft

Embedded length: 81.1 ft

Segment length: 5 ft

Segment weight: 0.265 kips

Segment stiffness: 7,530 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-17. - SUMMARY OF INPUT DATA FOR

DETROIT LTP 8 (CONTINUED)

Soil Properties

Type: 75 ft soft clay, 5 ft hardpan

RUT: 211 kips

RUP: 80 kips

Load distribution: 0.47 RUT in soft clay layer,

0.15 RUT in hardpan layer,

0.38 RUT at pile tip

Final blow count: 444 blows/ft

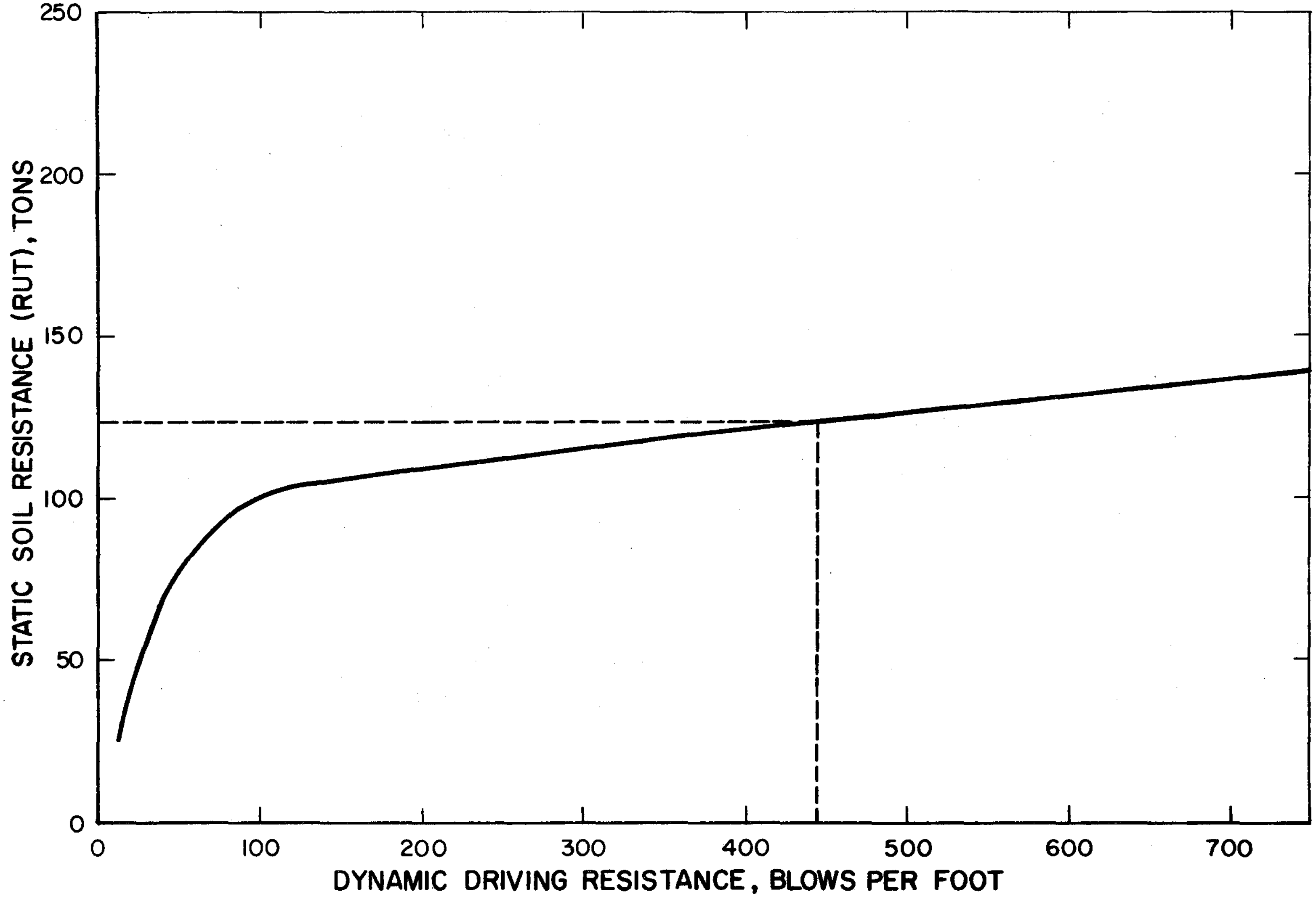

FIG.IV-17.—RUT vs. BLOW COUNT CURVE FOR DETROIT LTP 8

TABLE IV-18. - SUMMARY OF INPUT DATA FOR DETROIT LTP 10

Hammer Properties

Type: Link Belt 312

Rated energy: 18,000 ft-lb

Efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 15.84 fps

Ram weight: 3.855 kips

Ram stiffness: 142,500 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.188 kips

Helmet weight: 1.381 kips

Capblock: Alternating layers of 5 micarta fiber plates, 11-in. diameter by 1/2-in. thick. K_c = 17,100 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 12-in. OD, 0.23-in. wall, closed end steel pipe

Pile length: 83.1 ft

Embedded length: 81 ft

Segment length: 5 ft

Segment weight: 0.145 kips

Segment stiffness: 4,110 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-18. - SUMMARY OF INPUT DATA FOR DETROIT LTP 10 (CONTINUED)

Soil Properties

Type: 75 ft soft clay, 5 ft hardpan

RUT: 366 kips

RUP: 326 kips

Load distribution: 0.08 RUT in soft clay layer,

0.03 RUT in hardpan layer,

0.89 RUT at pile tip

Final blow count: 240 blows/ft

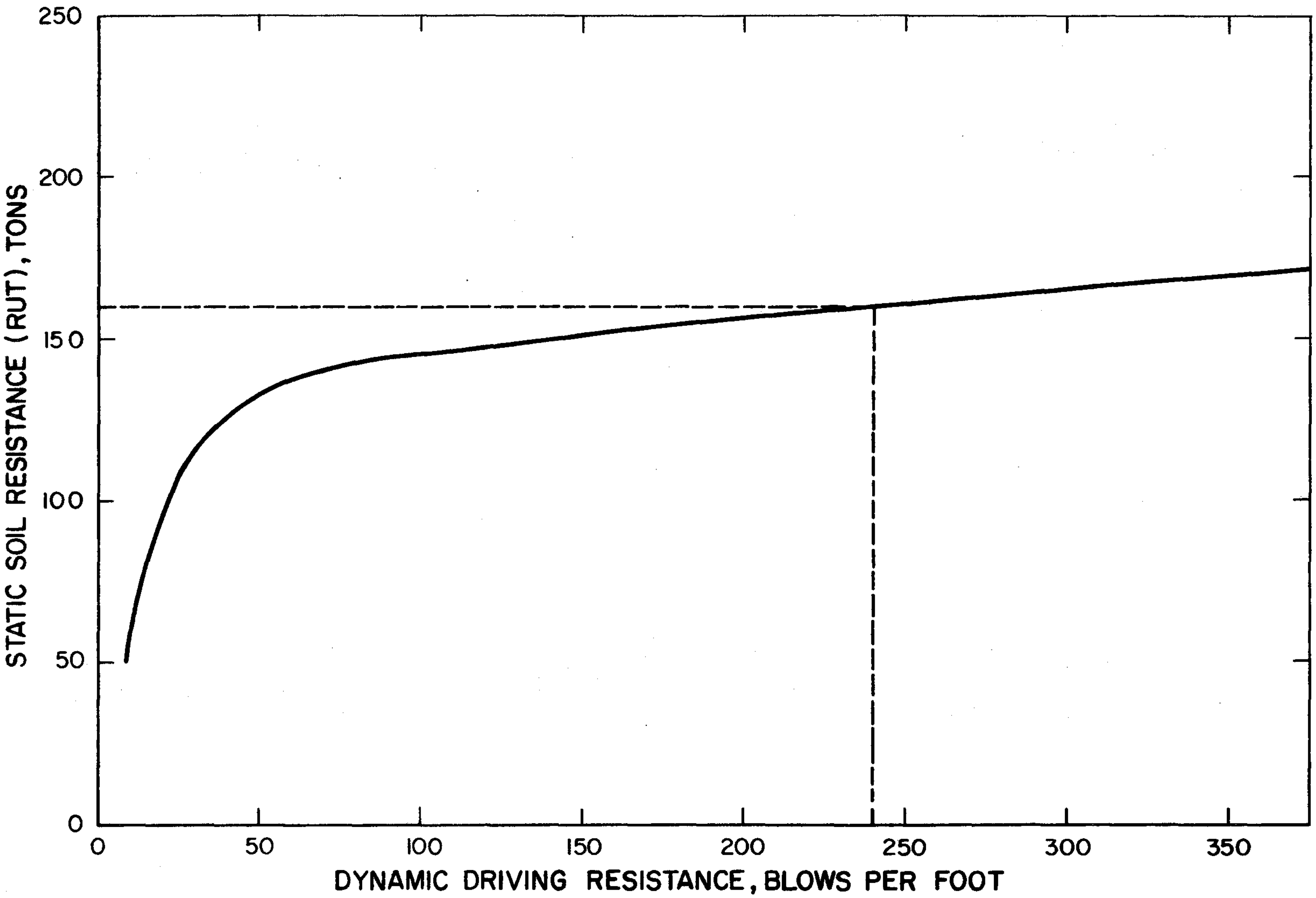

FIG. IV-18.—RUT vs. BLOW COUNT CURVE FOR DETROIT LTP 10

TABLE IV-19. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 2

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, 0.23-in. wall, closed end steel pipe

Pile length: 60 ft

Embedded length: 58 ft

Segment length: 5 ft

Segment weight: 0.145 kips

Segment stiffness: 4,110 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-19. - SUMMARY OF INPUT DATA FOR

MUSKEGON LTP 2 (CONTINUED)

Soil Properties

Type: Partially saturated loose sand

RUT: 200 kips

RUP: 30 kips

Load distribution: 0.85 RUT uniform side load,

0.15 RUT at pile tip

Final blow count: 96 blows/ft

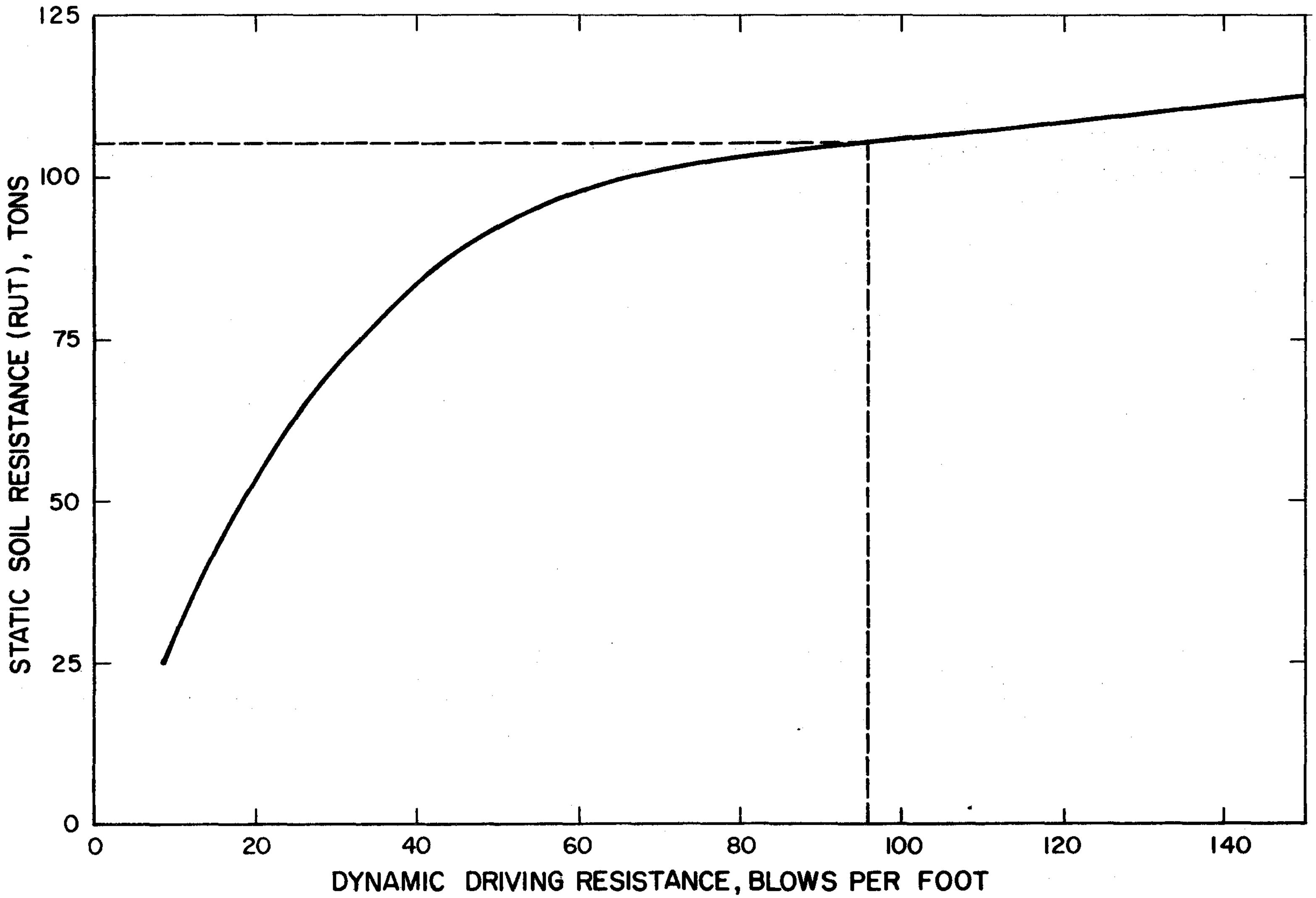

FIG. IV-19.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 2

TABLE IV-20. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 3

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, #7 gage, fluted tapered, closed end steel pipe

Pile length: 60.2 ft

Embedded length: 57.8 ft

Segment length: 5 ft

Segment weight: 0.113 kips

Segment stiffness: 3,220 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-20. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 3 (CONTINUED)

<u>Soil Properties</u>

Type: Partially saturated loose sand

RUT: 120 kips

RUP: 9.6 kips

Load distribution: 0.92 RUT uniform side load,

0.08 RUT at pile tip

Final blow count: 48 blows/ft

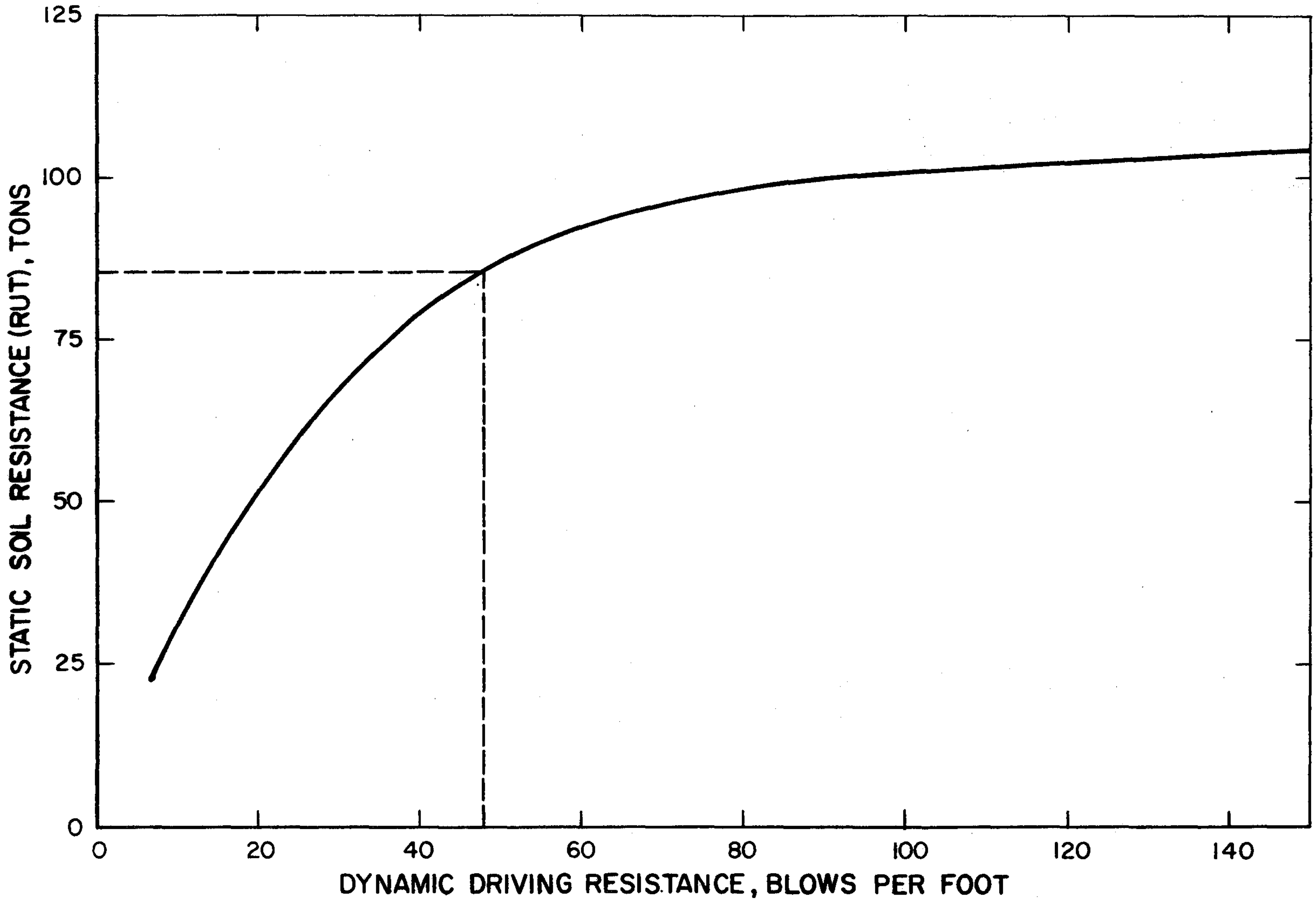

FIG. IV-20.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 3

TABLE IV-21. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 4

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Oak block (grain vertical), 11 1/4-in. diameter by 6 1/4-in. thick on top of two steel plates each 11 1/4-in. diameter by 3/4-in. thick. K_c = 11,125 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, 0.23-in. wall, open end steel pipe (internally jetted)

Pile length: 60 ft

Embedded length: 58 ft

Segment length: 5 ft

Segment weight: 0.145 kips

Segment stiffness: 4,110 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-21. - SUMMARY OF INPUT DATA FOR

MUSKEGON LTP 4 (CONTINUED)

Soil Properties

Type: Partially saturated loose sand

RUT: 94 kips

RUP: 1.9 kips

Load distribution: 0.98 RUT uniform side load,

0.02 RUT at pile tip

Final blow count: 26 blows/ft

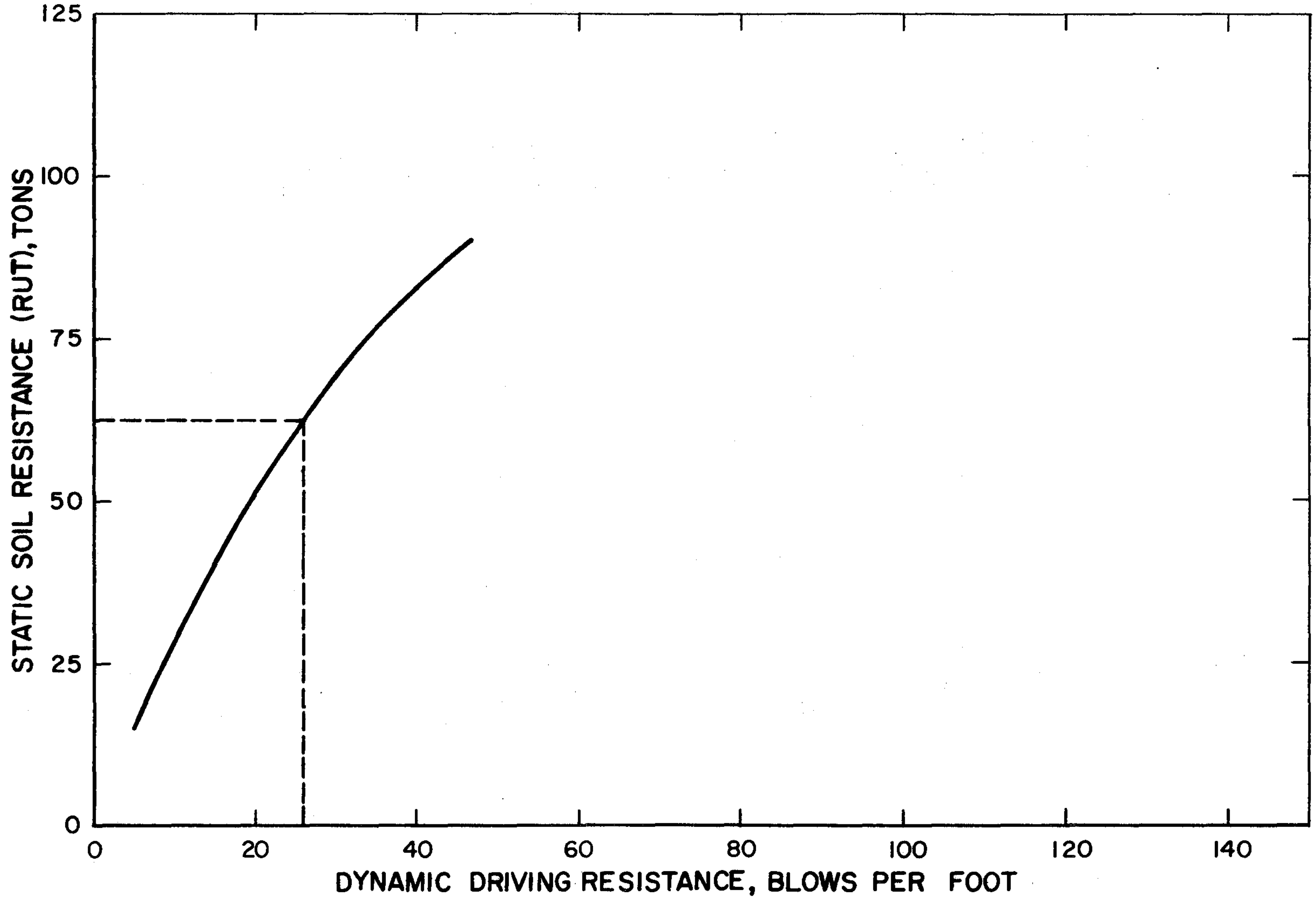

FIG. IV-21.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 4

TABLE IV-22. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 6

Hammer Properties

Type: Delmag D-22

Rated energy: 39,700 ft-lb

Efficiency (%): 100

Explosive force: 158.7 kips

Ram velocity: 17.26 fps

Ram weight: 4.85 kips

Ram stiffness: 49,700 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.147 kips

Helmet weight: 1.463 kips

Capblock: German Oak block (grain vertical), 15-in. x 15-in. x 5-in. thick under steel block, 15-in. x 15-in. x 3-in. thick. K_c = 31,500 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, closed end steel pipe

Pile length: 130 ft

Embedded length: 128 ft

Segment length: 5 ft

Segment weight: 0.157 kips

TABLE IV-22. - SUMMARY OF INPUT DATA FOR

MUSKEGON LTP 6 (CONTINUED)

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

Soil Properties

Type: 60 ft partially saturated loose sand, 50 ft soft peat (no load), 20 ft partially saturated compact sand

RUT: 480 kips

RUP: 120 kips

Load distribution: 0.32 RUT uniform side load in loose sand layer, 0.43 RUT uniform side load in compact sand, 0.25 RUT at pile tip

Final blow count: 36 blows/ft

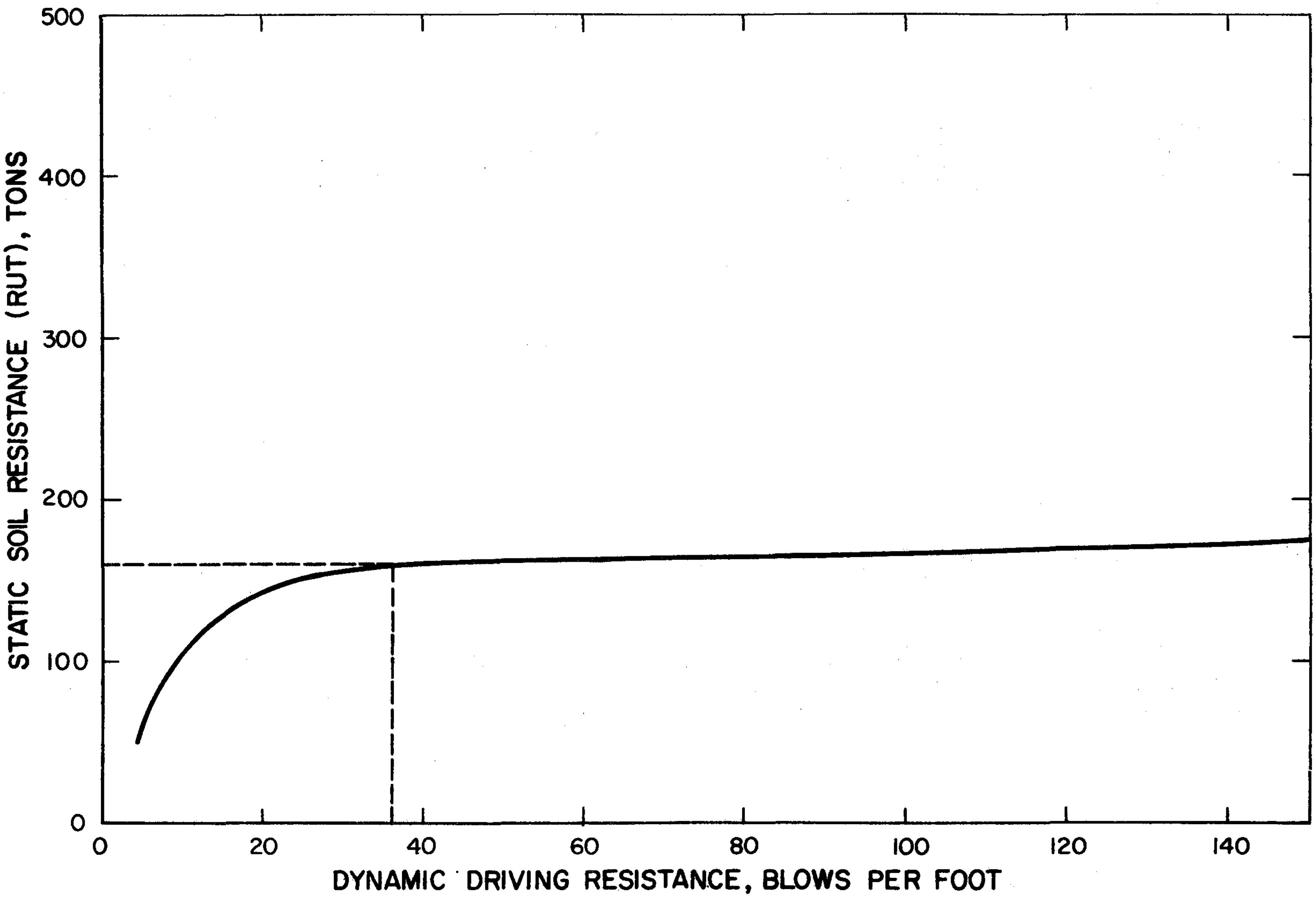

FIG. IV-22.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 6

TABLE IV-23. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 7

Hammer Properties

Type: Vulcan 80C

Rated energy: 24,450 ft lb

Efficiency (%): 84

Ram velocity: 12.8 fps

Ram weight: 8 kips

Helmet weight: 2.14 kips

Capblock: Two micarta fiber blocks each 14-in. diameter by 5-in. thick. K_c = 6,930 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, closed end steel pipe

Pile length: 180.4 ft

Embedded length: 178.4 ft

Segment length: 5 ft

Segment weight: 0.157 kips

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-23. - SUMMARY OF INPUT DATA FOR

MUSKEGON LTP 7 (CONTINUED)

Soil Properties

Type: 60 ft partially saturated loose sand, 50 ft soft peat (no load), 25 ft partially saturated compact sand, 45 ft firm clay, pile tip in very compact sand

RUT: 536 kips

RUP: 246 kips

Load distribution: 0.17 RUT uniform side load in loose sand layer, 0.29 RUT uniform load in compact sand layer, 0.08 RUT uniform side load in clay layer, 0.46 RUT at pile tip

Final blow count: 511 blows/ft

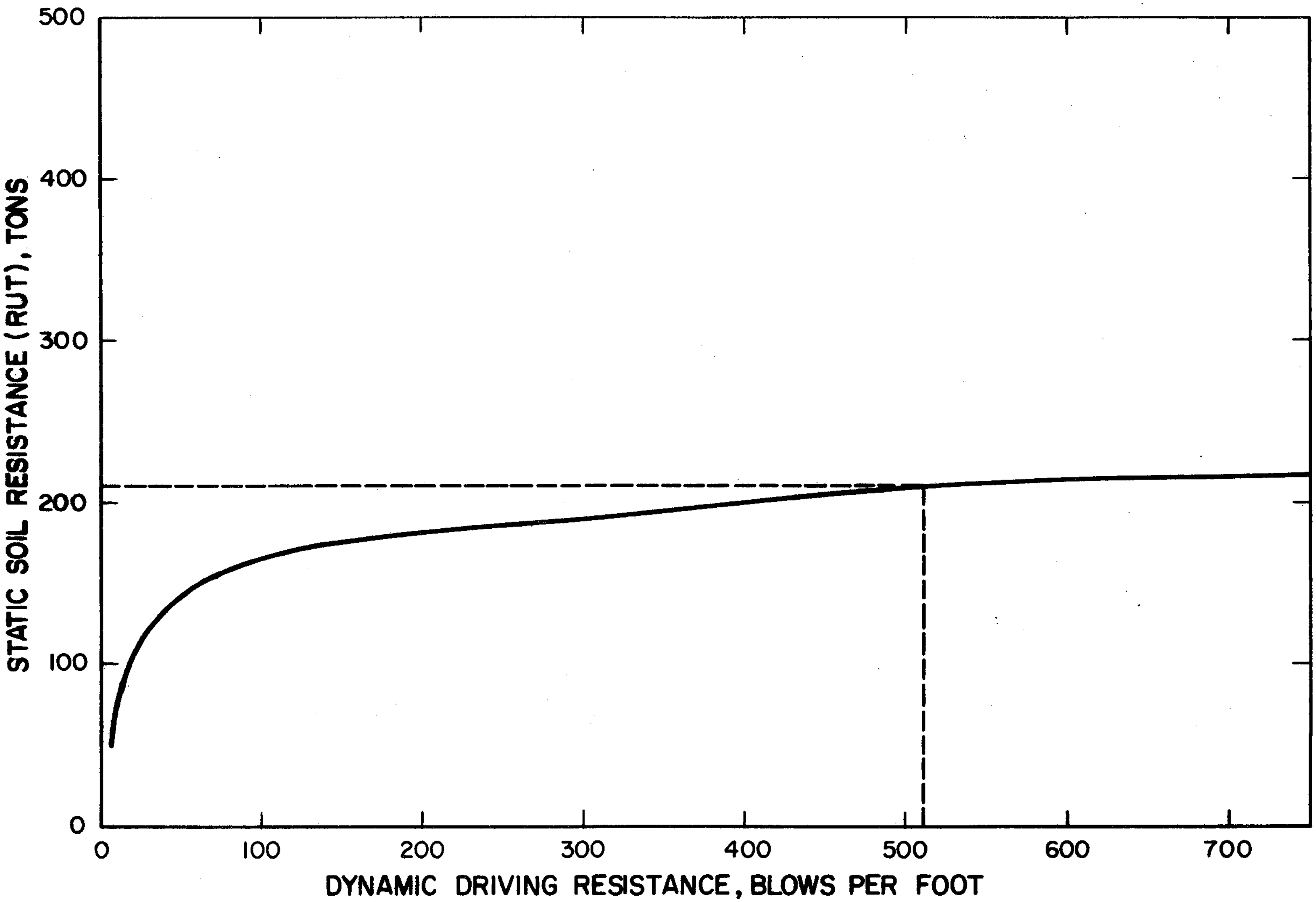

FIG. IV-23.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 7

TABLE IV-24. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 8

Hammer Properties

Type: Delmag D-22

Rated energy: 39,700 ft-lb

Efficiency (%): 100

Explosive force: 158.7 kips

Ram velocity: 17.26 fps

Ram weight: 4.85 kips

Ram stiffness: 49,700 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.147 kips

Helmet weight: 1.463 kips

Capblock: German Oak block (grain vertical), 15-in. x 15-in. x 5-in. thick under steel block, 15-in. x 15-in. x 3-in. thick. K_c = 31,500 kips/in., e = 0.5

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, closed end steel pipe

Pile length: 180.1 ft

Embedded length: 178.2 ft

Segment length: 5 ft

Segment weight: 0.157 kips

TABLE IV-24. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 8 (CONTINUED)

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

Soil Properties

Type: 60 ft partially saturated loose sand, 50 ft soft peat (no load), 25 ft partially saturated compact sand, 45 ft firm clay, pile tip in very compact sand

RUT: 502 kips

RUP: 231 kips

Load distribution: 0.17 RUT uniform side load in loose sand layer, 0.29 RUT uniform side load in compact sand layer, 0.08 RUT uniform side load in clay layer, 0.46 RUT at pile tip

Final blow count: 768 blows/ft

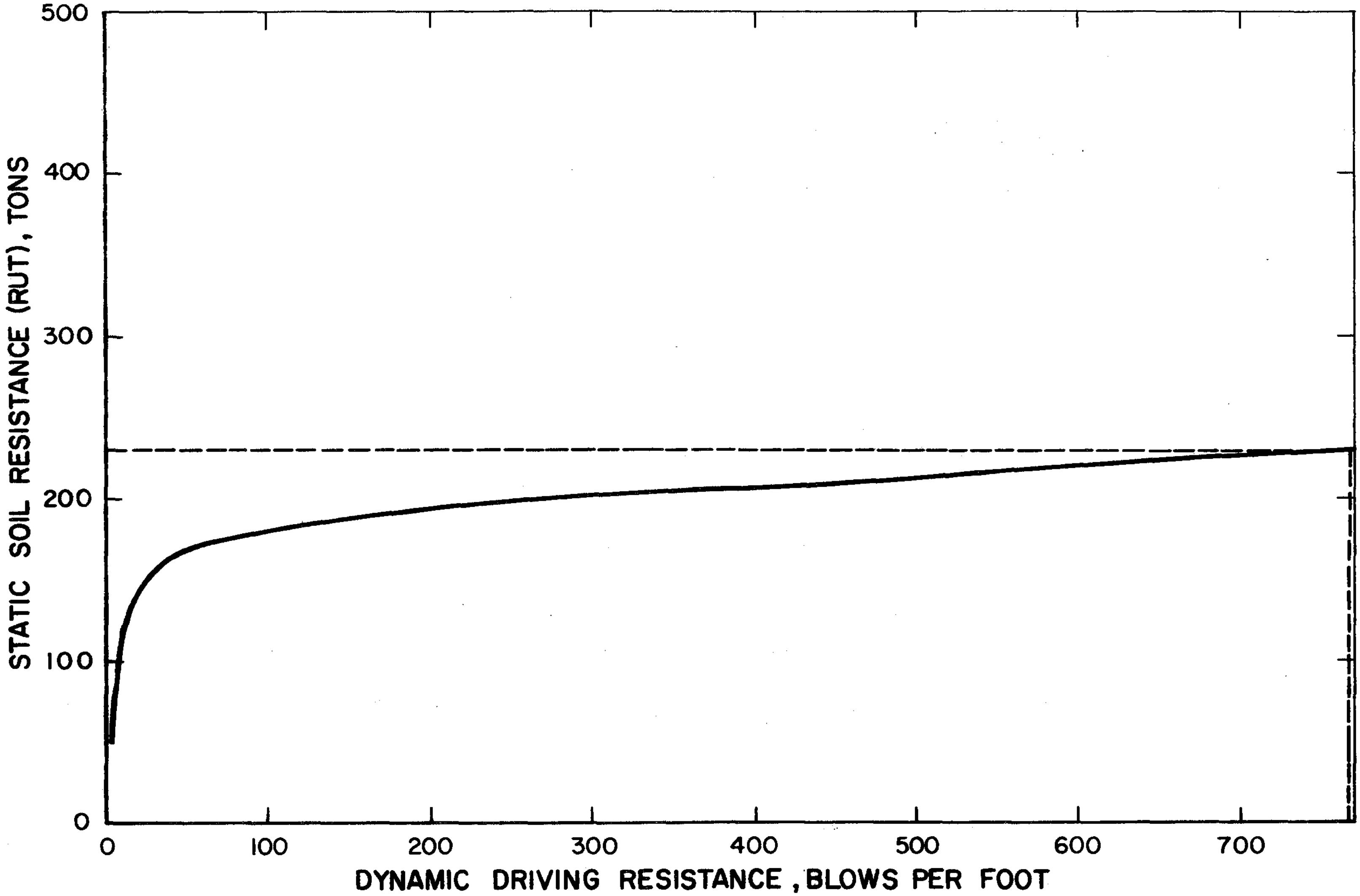

FIG. IV-24.—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 8

TABLE IV-25. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 9

Hammer Properties

Type: Vulcan 80C

Rated energy: 24,450 ft lb

Efficiency (%): 84

Ram velocity: 12.8 fps

Ram weight: 8 kips

Helmet weight: 2.14 kips

Capblock: Two micarta fiber blocks each 14-in. diameter by 5-in. thick. K_c = 6,930 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 12-in. OD, 0.25-in. wall, closed end steel pipe

Pile length: 130.2 ft

Embedded length: 128.2 ft

Segment length: 5 ft

Segment weight: 0.157 kips

Segment stiffness: 4,470 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-25. - SUMMARY OF INPUT DATA FOR MUSKEGON LTP 9 (CONTINUED)

Soil Properties

Type: 60 ft partially saturated loose sand, 50 ft soft peat (no load), 20 ft partially saturated compact sand

RUT: 484 kips

RUP: 121 kips

Load distribution: 0.32 RUT uniform side load in loose sand layer, 0.43 RUT uniform side load in compact sand, 0.25 RUT at pile tip

Final blow count: 66 blows/ft

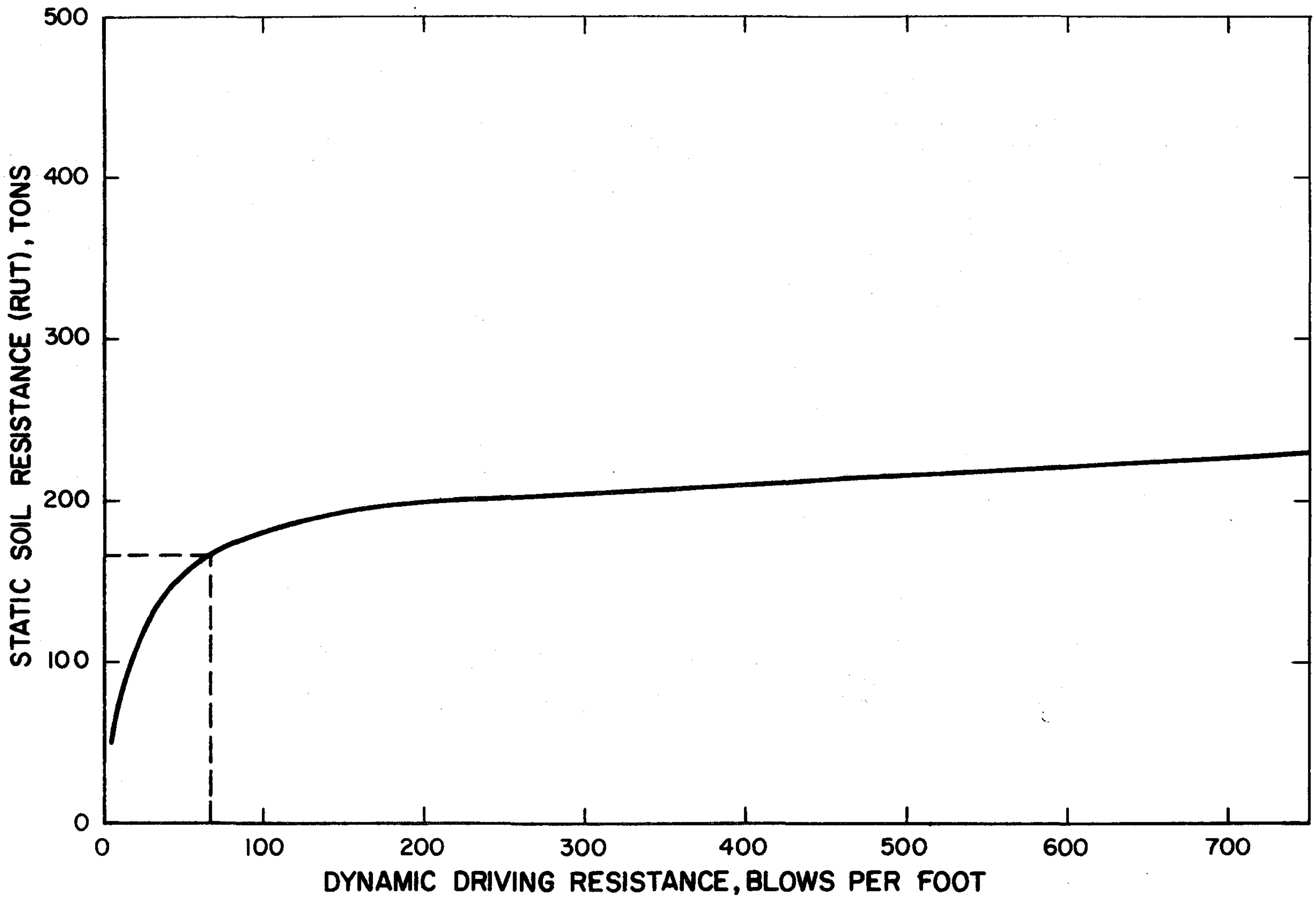

FIG. IV-25,—RUT vs. BLOW COUNT CURVE FOR MUSKEGON LTP 9

TABLE IV-26. - SUMMARY OF INPUT DATA FOR VICTORIA LTP 35

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Garlock abestos disk, 11 1/4-in. diameter by 3-in. thick with 2 steel plates 11 1/4-in. diameter by 3/4-in. thick. K_c = 1,490 kips/in., e = 0.5

Cushion: 6-in. plywood fir, K_c = 1,490 kips/in., K_{c+p} = 1,430 kips/in., e = 0.5

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 35 ft

Embedded length: 26.6 ft

Segment length: 5 ft

Segment weight: 1.353 kips

Segment stiffness: 33,300 kips/in., e = 1.0

TABLE IV-26. - SUMMARY OF INPUT DATA FOR VICTORIA LTP 35 (CONTINUED)

<u>Soil Properties</u>

Type: 10 ft clay, 15 ft partially saturated sand

RUT: 223 kips

RUP: 154 kips

Load distribution: 0.15 RUT uniform load in clay layer, 0.16 RUT uniform load in sand layer, 0.69 RUT at pile tip

Final blow count: 62 blows/ft

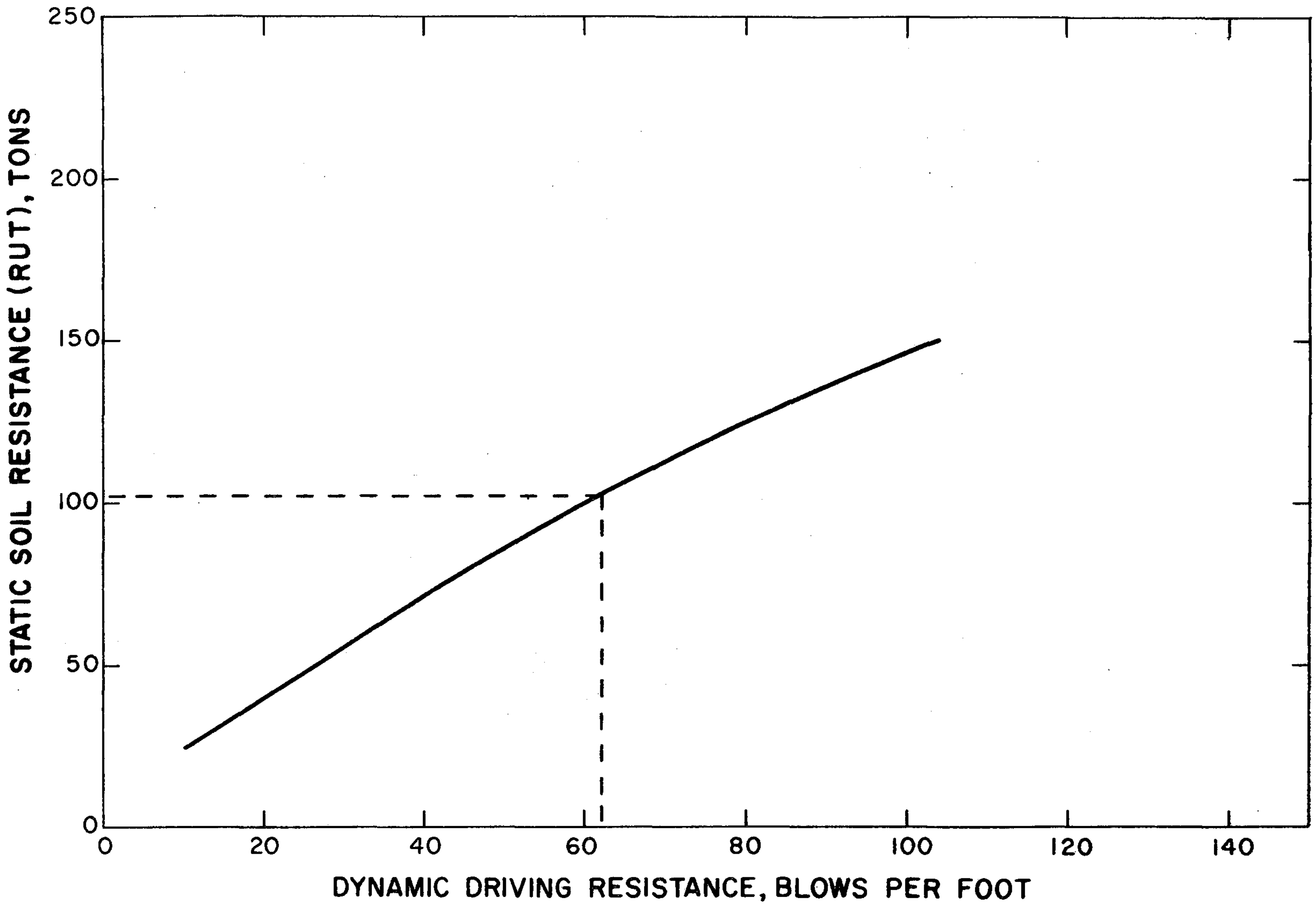

FIG. IV-26.-RUT vs. BLOW COUNT CURVE FOR VICTORIA LTP 35

TABLE IV-27. - SUMMARY OF INPUT DATA FOR VICTORIA LTP 40

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Garlock abestos disk, 11 1/4-in. diameter by 3-in. thick with 2 steel plates 11 1/4-in. diameter by 3/4-in. thick. K_c = 1,490 kips/in. e = 0.5

Cushion: 6-in. plywood fir, K_c = 1,490 kips/in., K_{c+p} = 1,430 kips/in., e = 0.5

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 40 ft

Embedded length: 33.2 ft

Segment length: 5 ft

Segment weight: 1.353 kips

Segment stiffness: 33,300 kips/in., e = 1.0

TABLE IV-27. - SUMMARY OF INPUT DATA FOR

VICTORIA LTP 40 (CONTINUED)

Soil Properties

Type: 15 ft clay, 20 ft partially saturated sand

RUT: 162 kips

RUP: 86 kips

Load distribution: 0.17 RUT uniform load in clay layer, 0.30 uniform load in sand layer, 0.53 RUT at pile tip

Final blow count: 52 blows/ft

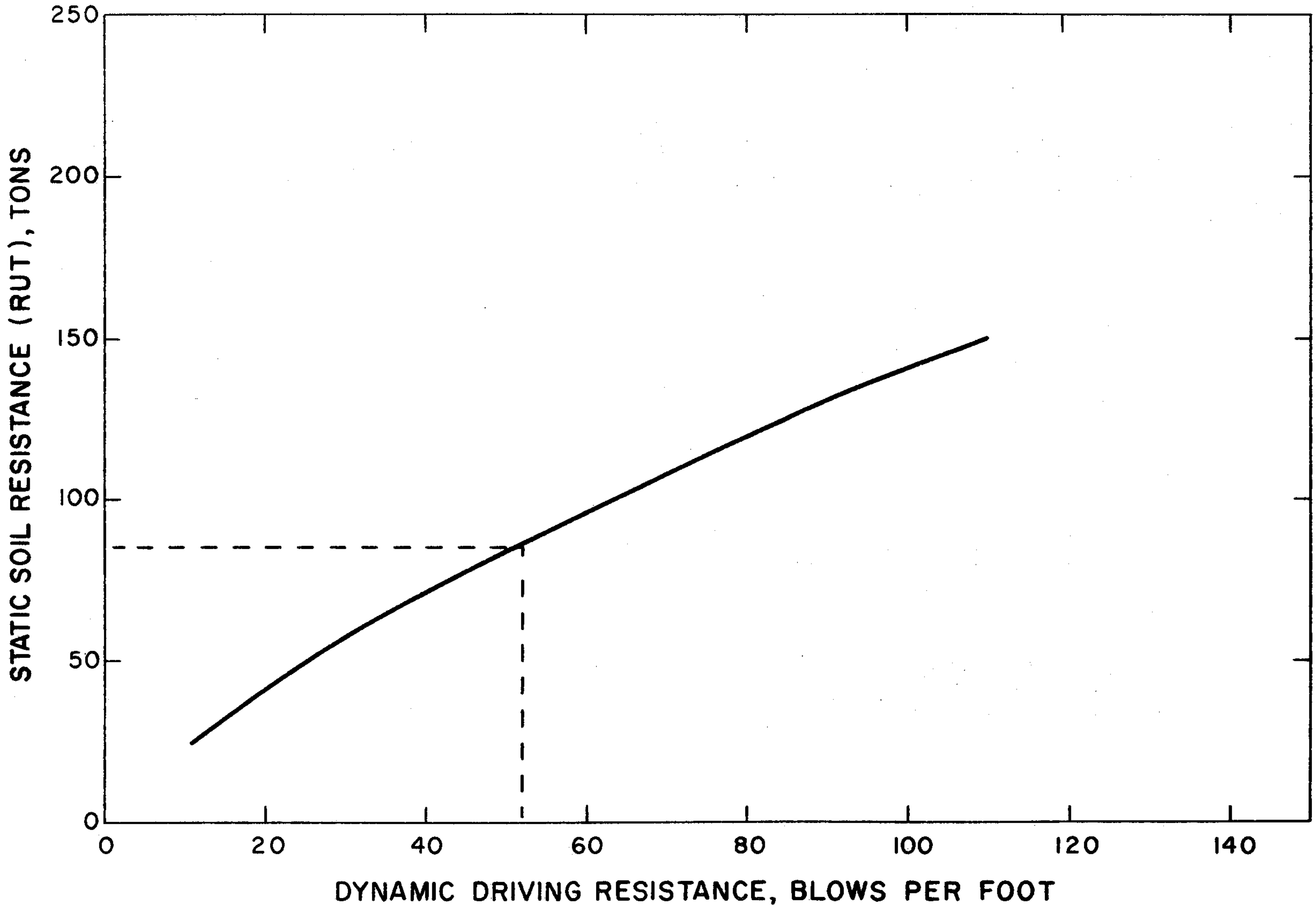

FIG. IV-27.-RUT vs. BLOW COUNT CURVE FOR VICTORIA LTP 40

TABLE IV-28. - SUMMARY OF INPUT DATA FOR VICTORIA LTP 45

Hammer Properties

Type: Vulcan #1

Rated energy: 15,000 ft-lb

Efficiency (%): 60

Ram velocity: 10.8 fps

Ram weight: 5 kips

Helmet weight: 1 kip

Capblock: Garlock abestos disk, 11 1/4-in. diameter by 3-in. thick with 2 steel plates 11 1/4-in. diameter by 3/4-in. thick. K_c = 1,490 kips/in., e = 0.5

Cushion: 6-in. plywood fir, K_c = 1,490 kips/in., K_{c+p} = 1,430 kips/in., e = 0.5

Pile Properties

Type: 16-in. square prestressed concrete

Pile length: 45 ft

Embedded length: 29.5 ft

Segment length: 5 ft

Segment weight: 1.353 kips

Segment stiffness: 33,300 kips/in., e = 1.0

TABLE IV-28. - SUMMARY OF INPUT DATA FOR
VICTORIA LTP 45 (CONTINUED)

Soil Properties

Type: 25 ft clay, 5 ft partially saturated sand and silt

RUT: 340 kips

RUP: 258 kips

Load distribution: 0.18 RUT uniform load in clay layer, 0.06 RUT uniform load in sand and silt layer, 0.76 RUT at pile tip

Final blow count: 395 blows/ft

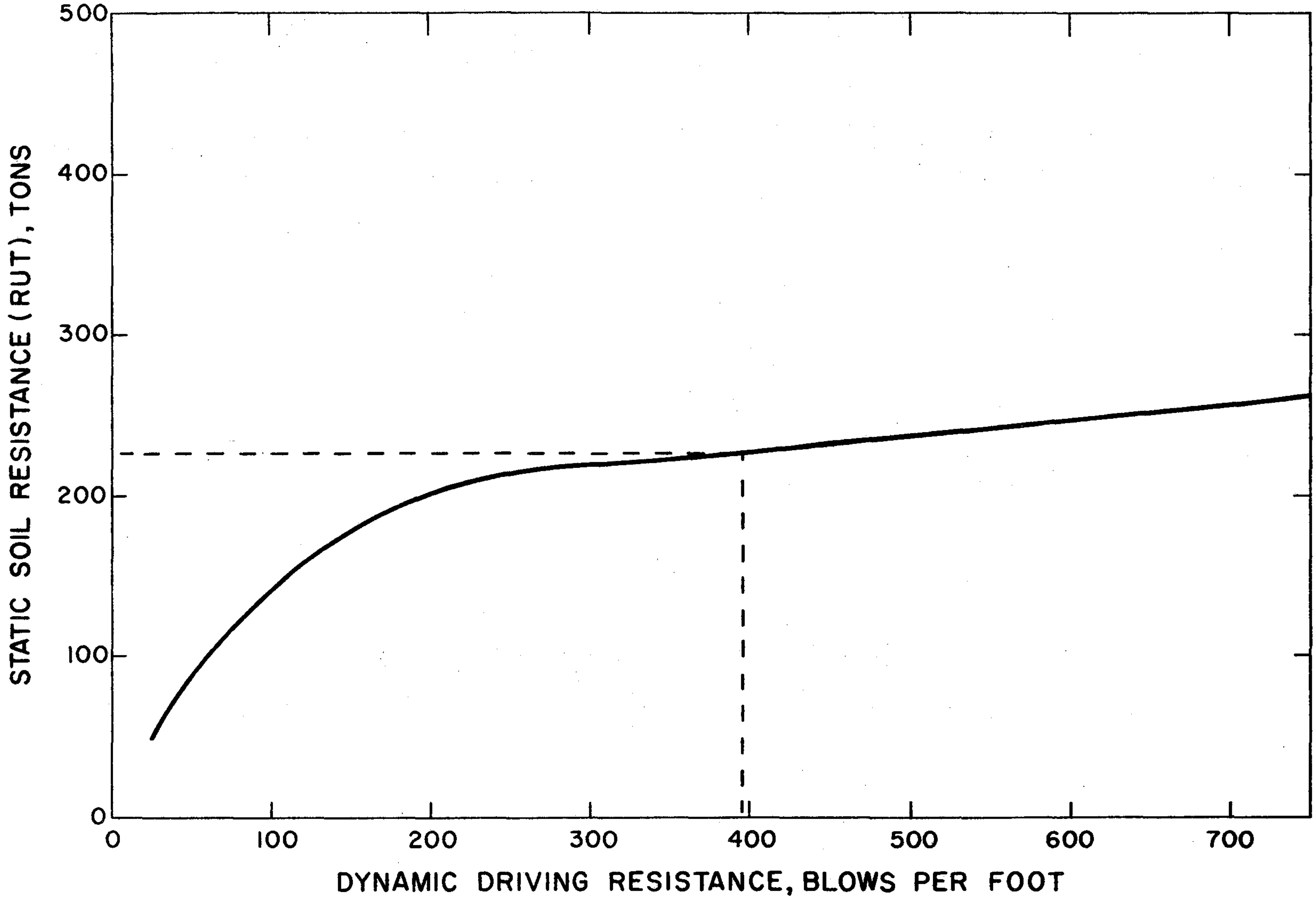

FIG. IV-28.-RUT vs. BLOW COUNT CURVE FOR VICTORIA LTP 45

TABLE IV-29. - SUMMARY OF INPUT DATA FOR CHOCOLATE BAYOU LTP 40

<u>Hammer Properties</u>

Type: Link Belt 520

Rated energy: 30,000 ft-lb

Efficiency (%): 100

Explosive force: 98 kips

Ram velocity: 16.3 fps

Ram weight: 5.07 kips

Ram stiffness: 108,500 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.179 kips

Helmet weight: 1.3 kips

Capblock: Alternating layers of 4 phenal fiber plates, 11-in. diameter by 1/2-in. thick with 4 aluminum plates 11-in. diameter by 1/8-in. thick. K_c = 21,400 kips/in., e = 0.8

Cushion: 6-in. plywood fir, K_c = 1,490 kips/in., K_{c+p} = 1,430 kips/in., e = 0.5

<u>Pile Properties</u>

Type: 16-in. square prestressed concrete

Pile length: 40 ft

Embedded length: 36 ft

Segment length: 5 ft

Segment weight: 1.378 kips

TABLE IV-29. - SUMMARY OF INPUT DATA FOR CHOCOLATE BAYOU LTP 40 (CONTINUED)

Segment stiffness: 33,100 kips/in., e = 1.0

Soil Properties

Type: 25 ft clay, 5 ft partially saturated sand and silt, 5 ft clay, pile tip in sand

RUT: 152 kips

RUP: 40 kips

Load distribution: 0.74 RUT uniform side load, 0.26 RUT at pile tip

Final blow count: 24 blows/ft

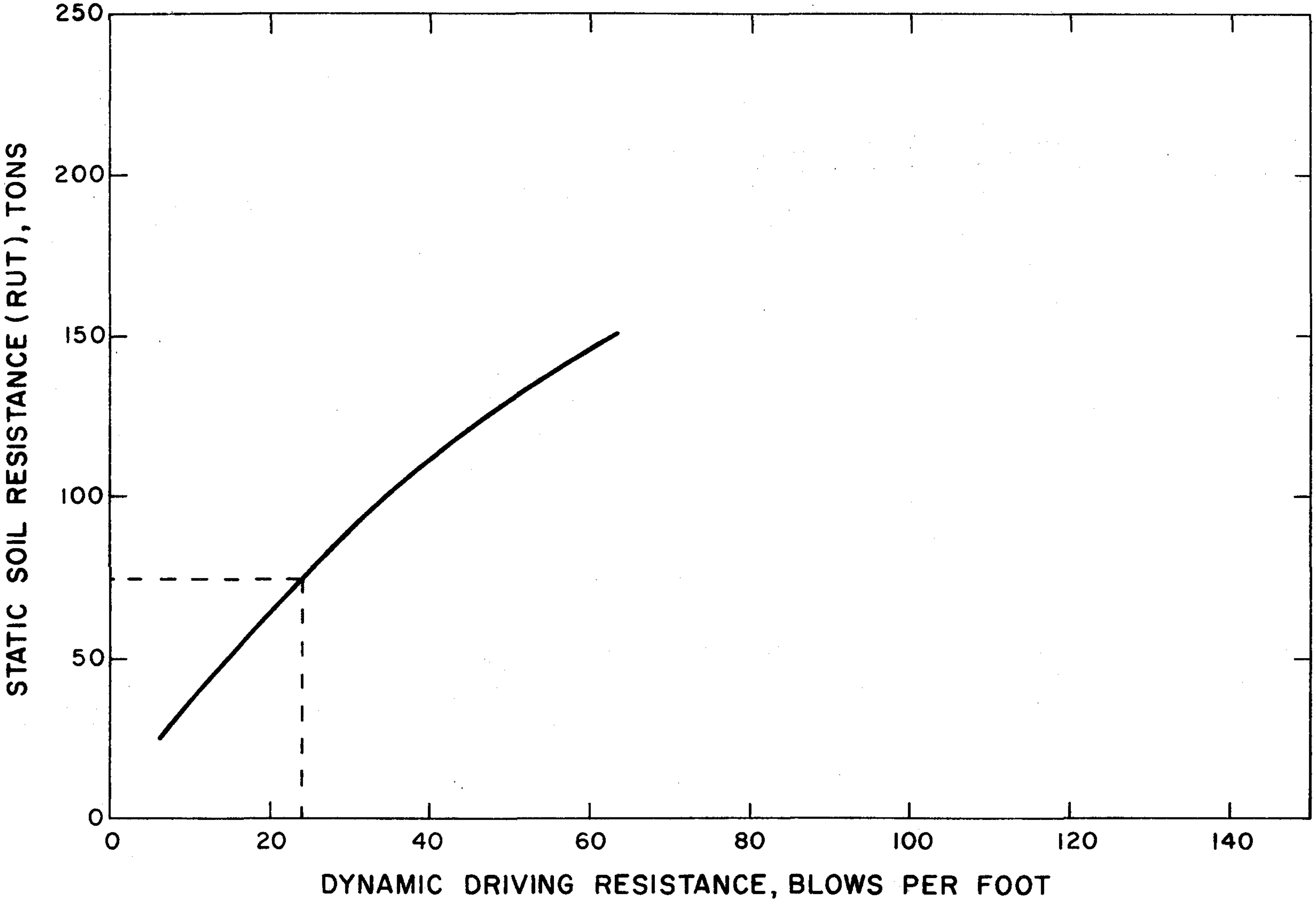

FIG. IV-29.-RUT vs. BLOW COUNT CURVE FOR CHOCOLATE BAYOU LTP 40

TABLE IV-30. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 1

Hammer Properties

Type: Delmag D-30

Rated energy: 54,500 ft-lb

Efficiency (%): 100

Explosive force: 242 kips

Ram velocity: 20 fps

Ram weight: 6.6 kips

Ram stiffness: 39,900 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.61 kips

Helmet weight: 1.3 kips

Capblock: Two 3/4-in. plywood fir, 15-in. x 15-in.

K_c = 5,250 kips/in., e = 0.4

Cushion: 6-in. rough scrap lumber (grain horizontal)

K_c = 977 kips/in., K_{c+p} = 930 kips/in., e = 0.5

Pile Properties

Type: 14-in. square prestressed concrete

Pile length: 24.5 ft

Embedded length: 22 ft

Segment length: 5 ft

Segment weight: 1.02 kips

Segment stiffness: 19,600 kips/in., e = 1.0

TABLE IV-30. - SUMMARY OF INPUT DATA FOR
HOUSTON LTP 1 (CONTINUED)

Soil Properties

Type: 10 ft clay, 10 ft partially saturated sand

RUT: 270 kips

RUP: 8 kips

Load distribution: 0.18 RUT uniform side load in clay layer, 0.79 RUT uniform side load in sand layer, 0.03 RUT at pile tip

Final blow count: 24 blows/ft

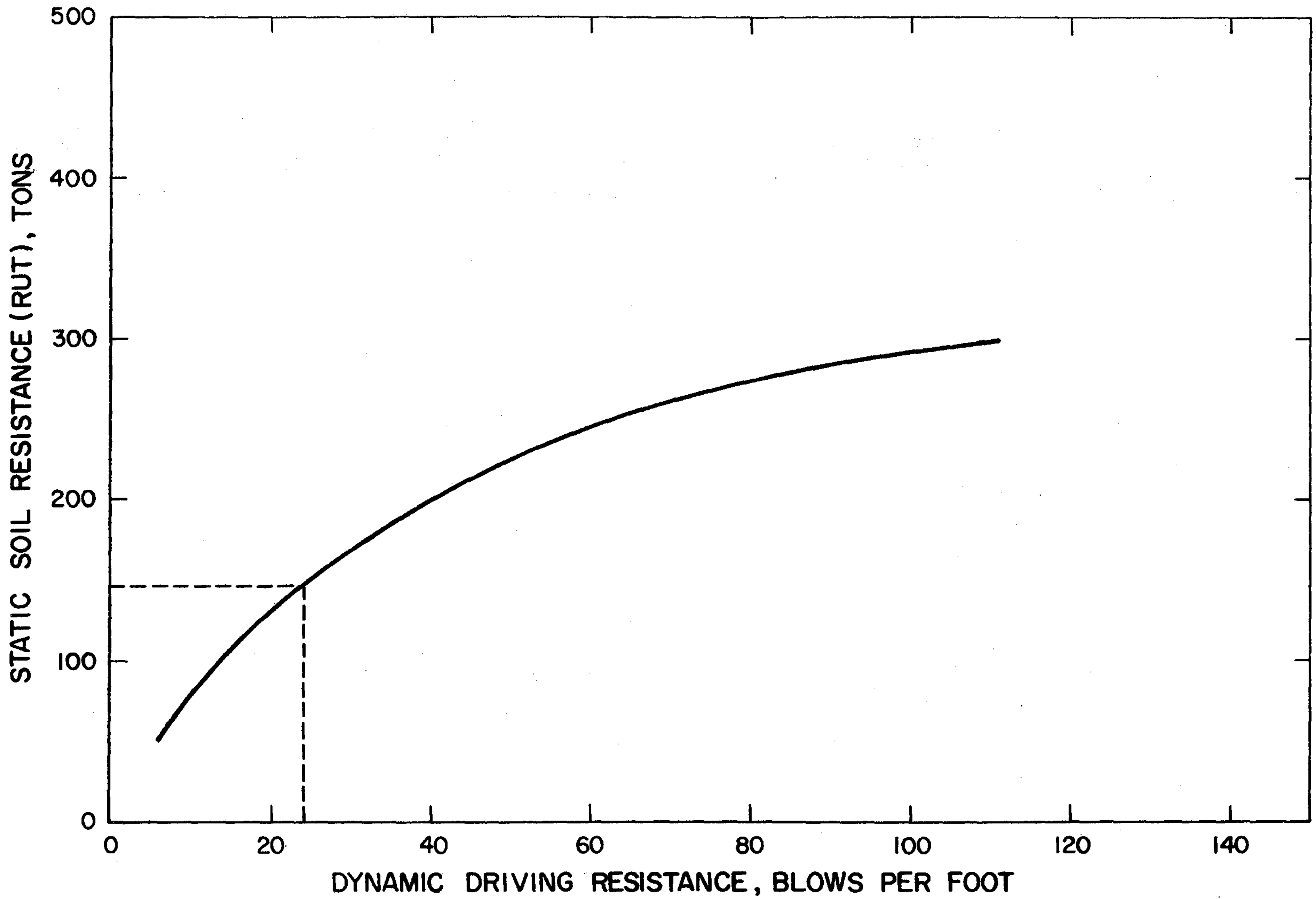

FIG. IV - 30. — RUT vs. BLOW COUNT CURVE FOR HOUSTON LTP I

TABLE IV-31. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 2

Hammer Properties

Type: Delmag D-30

Rated energy: 54,500 ft-lb

Efficiency (%): 100

Explosive force: 242 kips

Ram velocity: 20 fps

Ram weight: 6.6 kips

Ram stiffness: 39,900 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.61 kips

Helmet weight: 1.3 kips

Capblock: Two 3/4-in. plywood fir, 15-in. x 15-in.

K_c = 5,250 kips/in., e = 0.4

Cushion: 6-in. rough scrap lumber (grain horizontal)

K_c = 977 kips/in., K_{c+p} = 930 kips/in., e = 0.5

Pile Properties

Type: 14-in. square prestressed concrete

Pile length: 34.5 ft

Embedded length: 32 ft

Segment length: 5 ft

Segment weight: 1.02 kips

Segment stiffness: 19,600 kips/in., e = 1.0

TABLE IV-31. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 2 (CONTINUED)

Soil Properties

Type: 25 ft clay, 5 ft partially saturated sand

RUT: 161 kips

RUP: 14.5 kips

Load distribution: 0.50 RUT uniform side load in clay layer, 0.41 RUT uniform side load in sand layer, 0.09 RUT at pile tip

Final blow count: 10 blows/ft

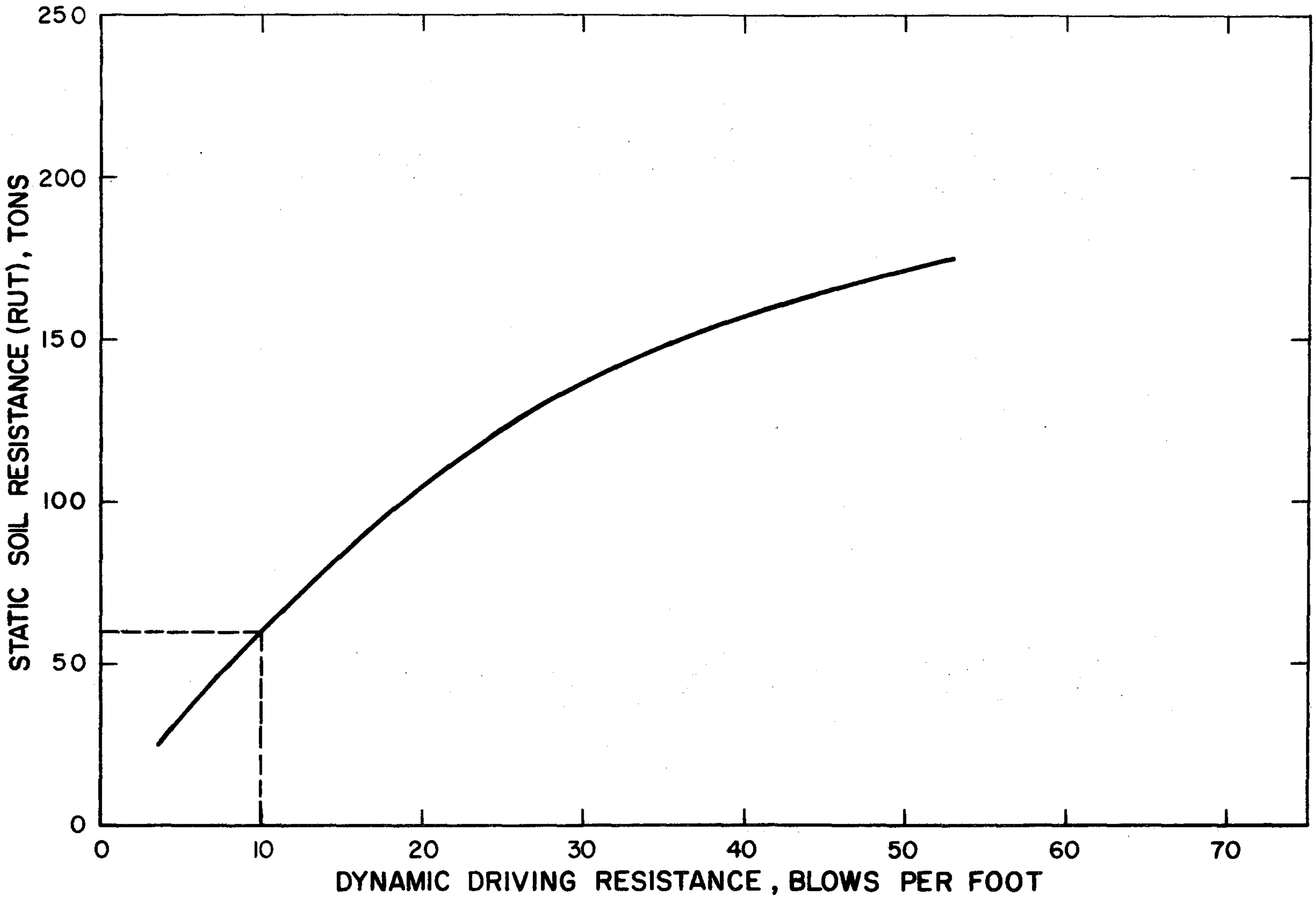

FIG. IV-31.—RUT vs. BLOW COUNT CURVE FOR HOUSTON LTP 2

TABLE IV-32. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 3

Hammer Properties

Type: Delmag D-30

Rated energy: 54,500 ft-lb

Efficiency (%): 100

Explosive force: 242 kips

Ram velocity: 20 fps

Ram weight: 6.6 kips

Ram stiffness: 39,900 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.61 kips

Helmet weight: 1.3 kips

Capblock: Two 3/4-in. plywood fir, 15-in. x 15-in.

K_c = 5,250 kips/in., e = 0.4

Cushion: 6-in. rough scrap lumber (grain horizontal)

K_c = 977 kips/in., K_{c+p} = 930 kips/in., e = 0.5

Pile Properties

Type: 14-in. square prestressed concrete

Pile length: 28.5 ft

Embedded length: 26 ft

Segment length: 5 ft

Segment weight: 1.02 kips

Segment stiffness: 19,600 kips/in., e = 1.0

TABLE IV-32. - SUMMARY OF INPUT DATA FOR

HOUSTON LTP 3 (CONTINUED)

Soil Properties

Type: 5 ft clay, 20 ft partially saturated sand

RUT: 366 kips

RUP: 22 kips

Load distribution: 0.06 RUT uniform side load in clay layer, 0.88 RUT uniform side load in sand layer, 0.06 RUT at pile tip

Final blow count: 38 blows/ft

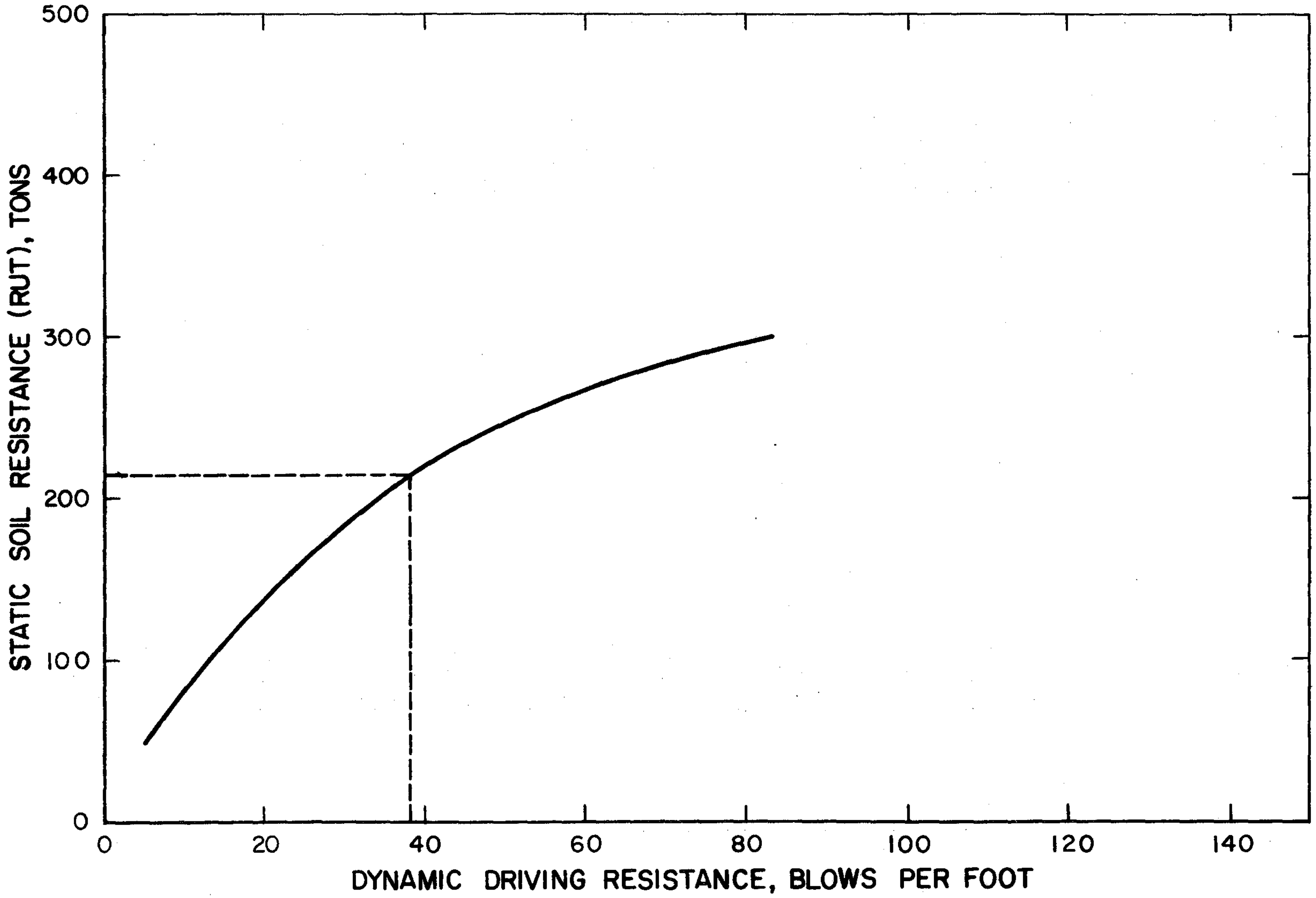

FIG. IV-32.—RUT vs. BLOW COUNT CURVE FOR HOUSTON LTP 3

TABLE IV-33. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 4

Hammer Properties

Type: Delmag D-30

Rated energy: 54,500 ft-lb

Efficiency (%): 100

Explosive force: 242 kips

Ram velocity: 20 fps

Ram weight: 6.6 kips

Ram stiffness: 39,900 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.61 kips

Helmet weight: 1.3 kips

Capblock: Two 3/4-in. plywood fir, 15-in. x 15-in.

K_c = 5,250 kips/in., e = 0.4

Cushion: 6-in. rough scrap lumber (grain horizontal)

K_c = 977 kips/in., K_{c+p} = 930 kips/in., e = 0.5

Pile Properties

Type: 14-in. square prestressed concrete

Pile length: 32.5 ft

Embedded length: 30 ft

Segment length: 5 ft

Segment weight: 1.02 kips

Segment stiffness: 19,600 kips/in., e = 1.0

TABLE IV-33. - SUMMARY OF INPUT DATA FOR

HOUSTON LTP 4 (CONTINUED)

Soil Properties

Type: 25 ft clay, 5 ft partially saturated sand

RUT: 170 kips

RUP: 18.7 kips

Load distribution: 0.68 RUT uniform side load in clay layer, 0.21 RUT uniform side load in sand layer, 0.11 RUT at pile tip

Final blow count: 30 blows/ft

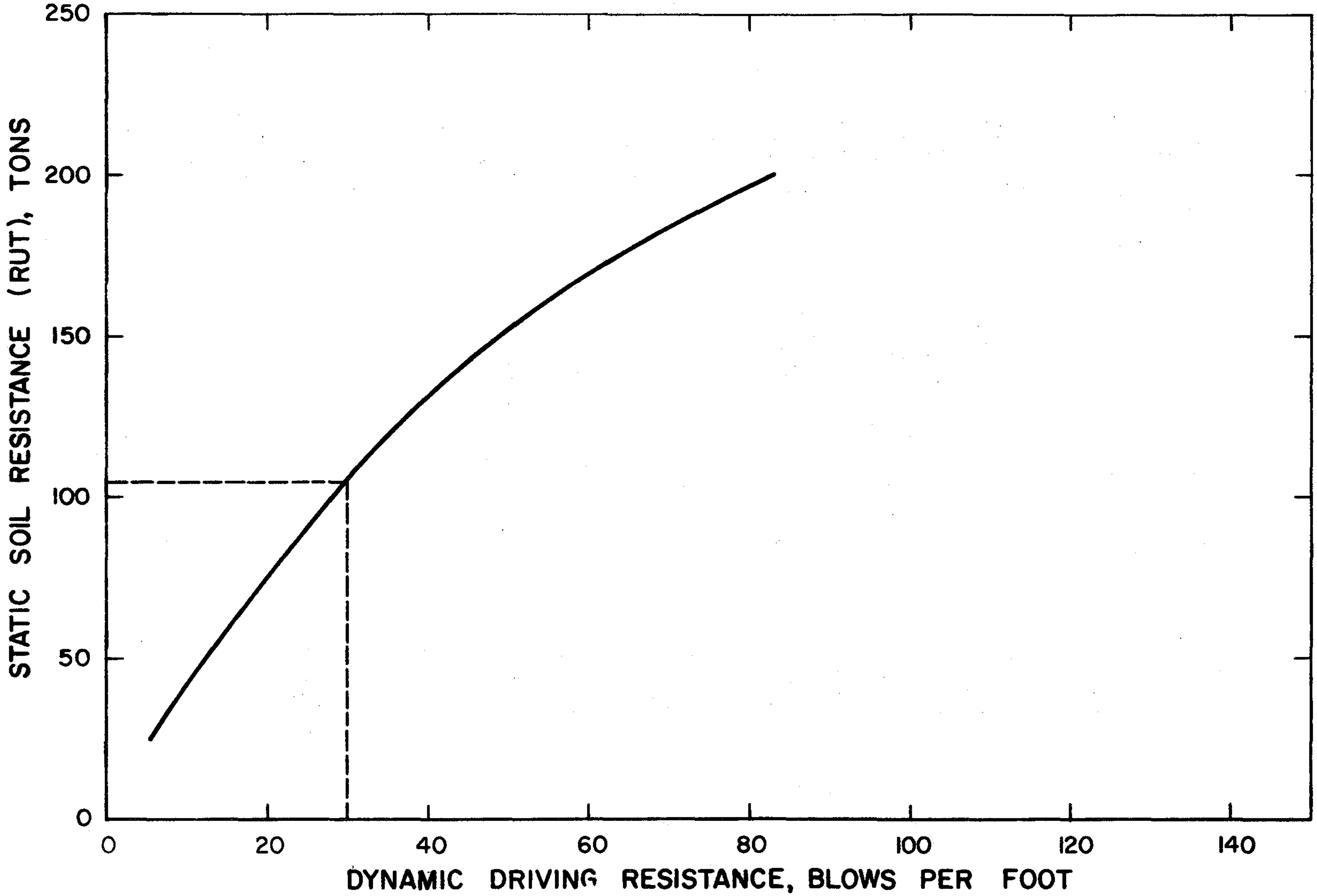

FIG. IV-33.- RUT vs. BLOW COUNT CURVE FOR HOUSTON LTP 4

TABLE IV-34. - SUMMARY OF INPUT DATA FOR HOUSTON LTP 30

Hammer Properties

Type: Delmag D-22

Rated energy: 39,700 ft-lb

Efficiency (%): 100

Explosive force: 158.7 kips

Ram velocity: 19.9 fps

Ram weight: 4.85 kips

Ram stiffness: 49,700 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 1.147 kips

Helmet weight: 1.46 kips

Capblock: Plywood fir, 19.75-in. diameter by 3/4-in. thick. K_c = 14,270 kips/in., e = 0.5

Cushion: 6-in. plywood fir, K_c = 1,142 kips/in., K_{c+p} = 1,090 kips/in., e = 0.5

Pile Properties

Type: 14-in. square prestressed concrete, tapered from 14-in. to 8-in. at tip of pile

Pile length: 30 ft

Embedded length: 26.5 ft

Segment length: 5 ft

Segment weight: Variable - 1.035 kips, 1.035 kips,

TABLE IV-34. - SUMMARY OF INPUT DATA FOR

HOUSTON LTP 30 (CONTINUED)

0.73 kips, 0.555 kips, 0.407 kips

Segment stiffness: Variable - 23,900 kips/in., 21,420 kips/in., 16,800 kips/in., 12,800 kips/in., 9,380 kips/in., e = 1.0

Soil Properties

Type: 15 ft sandy clay, 10 ft silty clay

RUT: 324 kips

RUP: 224 kips

Load distribution: 0.01 RUT first 5 ft sandy clay, 0.14 RUT second 5 ft sandy clay, .01 RUT next 10 ft, .15 RUT next 5 ft, 0.69 RUT at pile tip

Final blow count: 38 blows/ft

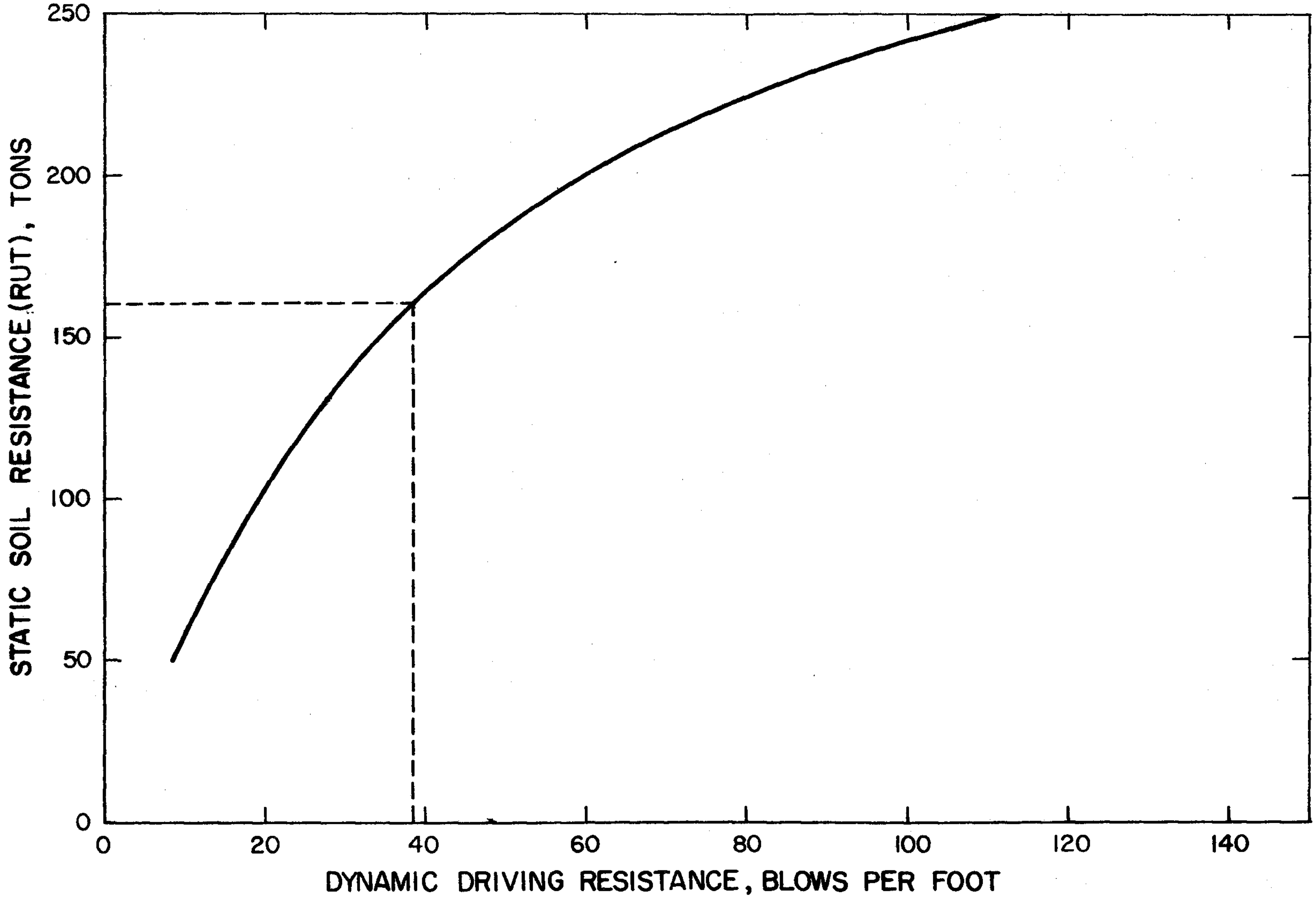

FIG. IV - 34,–RUT vs. BLOW COUNT CURVE FOR HOUSTON LTP 30

TABLE IV-35. - SUMMARY OF INPUT DATA FOR BEAUMONT LTP 53

Hammer Properties

Type: Delmag D-12

Rated energy: 22,600 ft-lb

Efficiency (%): 100

Explosive force: 93.7 kips

Ram velocity: 21 fps

Ram weight: 2.75 kips

Ram stiffness: 31,500 kips/in., e = 0.6 (steel on steel impact)

Anvil weight: 0.816 kips

Helmet weight: 0.597 kips

Capblock: K_c = 18,600 kips/in., e = 0.8

Cushion: None

Pile Properties

Type: 16-in. OD, 0.375-in. wall, closed end steel pipe

Pile length: 53 ft

Embedded length: 50 ft

Segment length: 5 ft

Segment weight: 0.29 kips

Segment stiffness: 8,780 kips/in., e = 1.0 except for first segment (steel on steel impact), e = 0.6

TABLE IV-35. - SUMMARY OF INPUT DATA FOR

BEAUMONT LTP 53 (CONTINUED)

Soil Properties

Type: Clay

RUT: 138 kips

RUP: 36 kips

Load distribution: 0.74 RUT uniform side load,

0.26 RUT at pile tip

Final blow count: 28 blows/ft

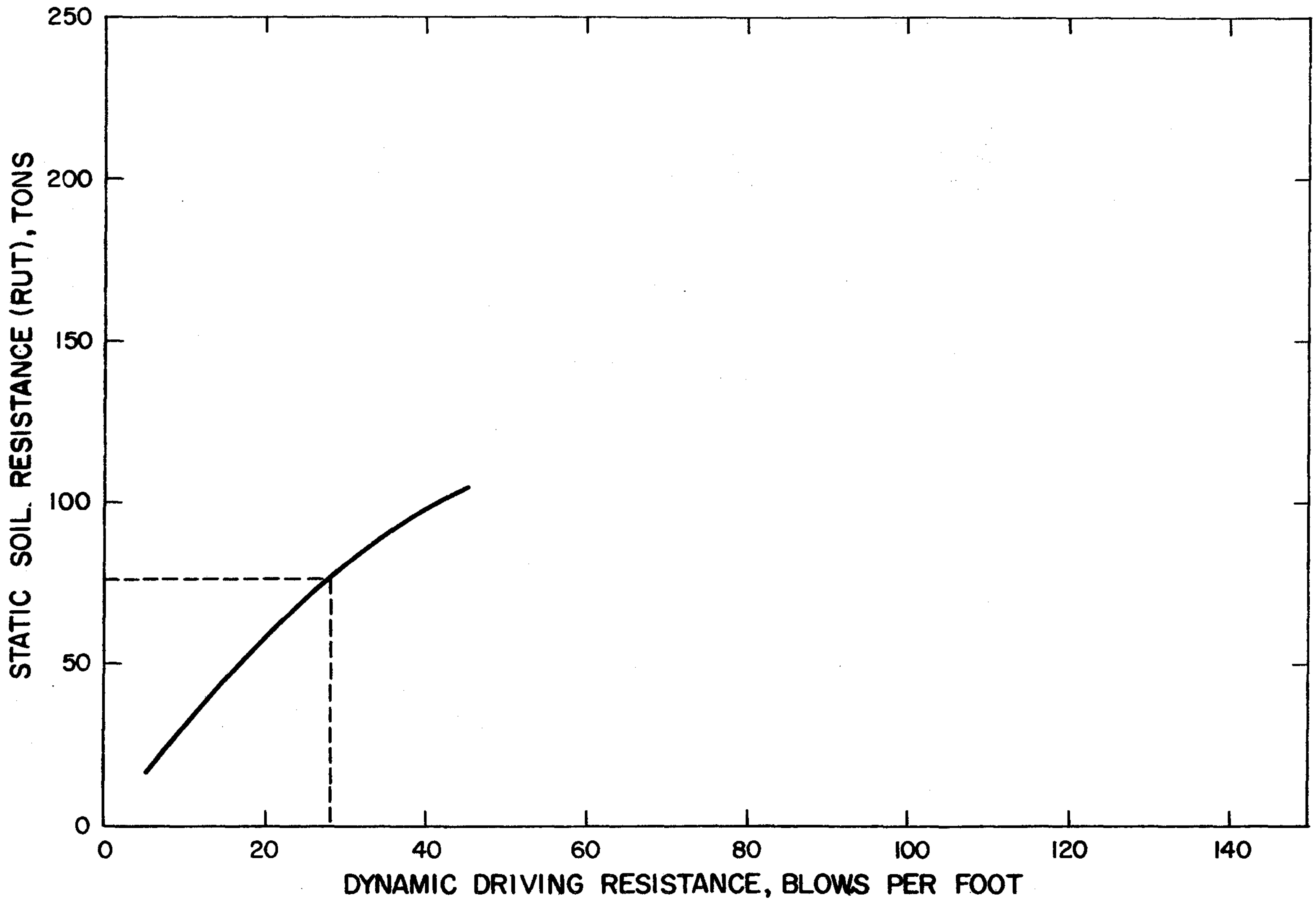

FIG. IV - 35.– RUT vs. BLOW COUNT CURVE FOR BEAUMONT LTP 53

TABLE IV-36. - SUMMARY OF INPUT DATA FOR COPANO BAY LTP 103

Hammer Properties

Type: Vulcan 014

Rated energy: 42,000 ft-lb

Efficiency (%): 90

Ram velocity: 13.2 fps

Ram weight: 14 kips

Helmet weight: 3 kips

Capblock: Gum, 14-in. diameter by 3/4-in. thick

K_c = 7,830 kips/in., e = 0.50

Cushion: 6-in. gum, K_c = 1,620 kips/in.,

K_{c+p} = 1,565 kips/in., e = 0.5

Pile Properties

Type: 18-in. square prestressed concrete

Pile length: 103 ft

Embedded length: 83.5 ft

Segment length: 5 ft

Segment weight: 1.68 kips

Segment stiffness: 41,700 kips/in., e = 1.0

Soil Properties

Type: 75 ft muck (no load), 10 ft saturated silty sand

RUT: 356 kips

TABLE IV-36. - SUMMARY OF INPUT DATA FOR COPANO BAY LTP 103 (CONTINUED)

RUP: 214 kips

Load distribution: 0.40 RUT uniform side load in 10 ft saturated silty sand, 0.60 RUT at pile tip

Final blow count: 29 blows/ft

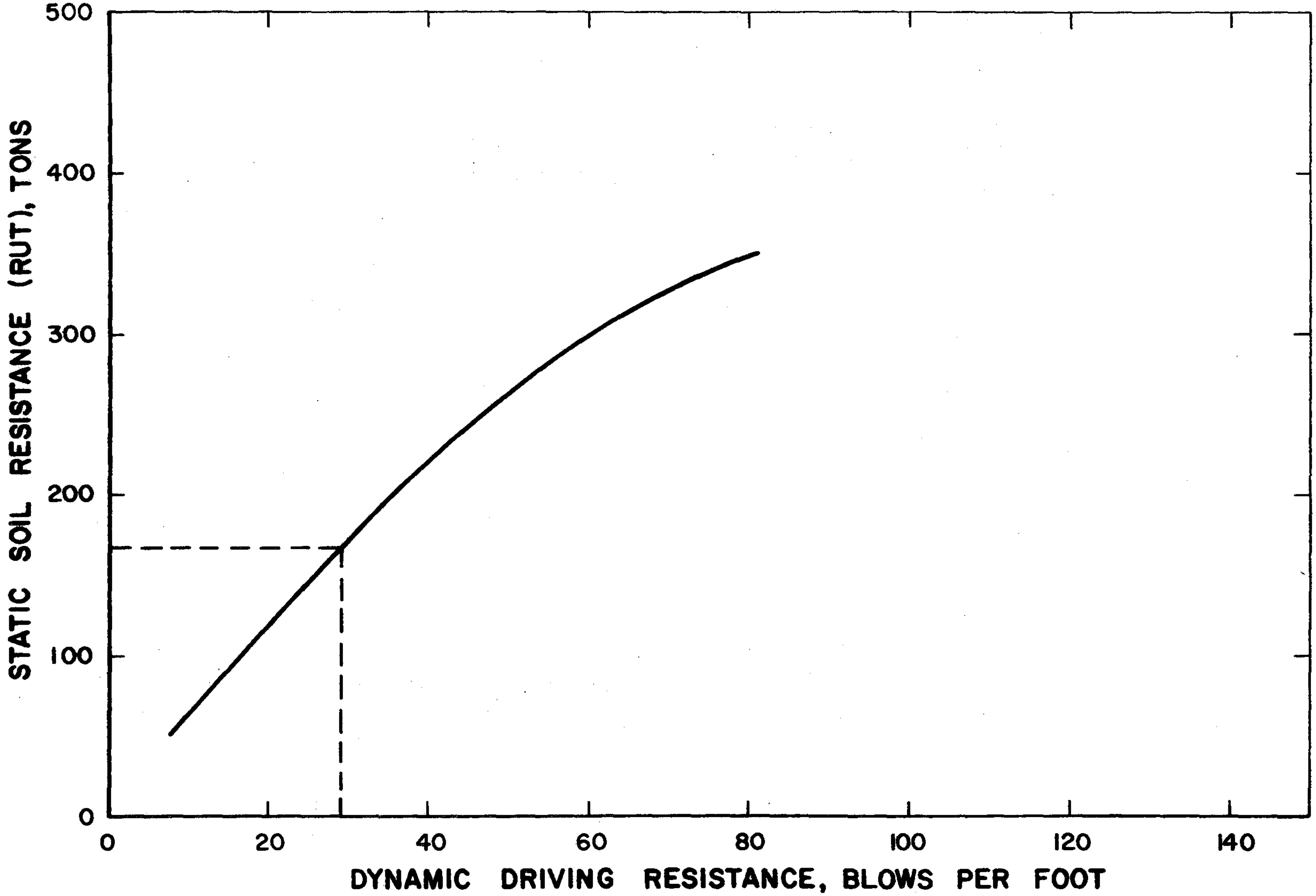

FIG. IV-36.-RUT vs. BLOW COUNT CURVE FOR COPANO BAY LTP 103

TABLE IV-37. - SUMMARY OF INPUT DATA FOR PADRE ISLAND LTP 22

Hammer Properties

Type: Vulcan 014

Rated energy: 42,000 ft-lb

Efficiency (%): 80

Ram velocity: 12.4 fps

Ram weight: 14 kips

Helmet weight: 3.5 kips

Capblock: Alternating 9 aluminum and 9 micarta disks, 17 1/4-in. diameter by 1/4-in. thick each. K_c = 41,700 kips/in., e = 0.8

Cushion: 6-in. green oak (grain horizontal) K_c = 2,290 kips/in., K_{c+p} = 2,080 kips/in., e = 0.5

Pile Properties

Type: 20-in. square prestressed concrete with 11-in. diameter hole

Pile length: 83 ft

Embedded length: 70 ft

Segment length: 5 ft

Segment weight: 1.58 kips

Segment stiffness: 35,300 kips/in., e = 1.0

TABLE IV-37. - SUMMARY OF INPUT DATA FOR PADRE ISLAND LTP 22 (CONTINUED)

Soil Properties

Type: 30 ft sandy clay, 20 ft partially saturated sand, 20 ft soft clay

RUT: 382 kips

RUP: 11.5 kips

Load distribution: 0.17 RUT load in sandy clay layer, 0.46 RUT load in partially saturated sand layer, 0.34 RUT in soft clay layer, 0.03 RUT at pile tip

Final blow count: 35 blows/ft

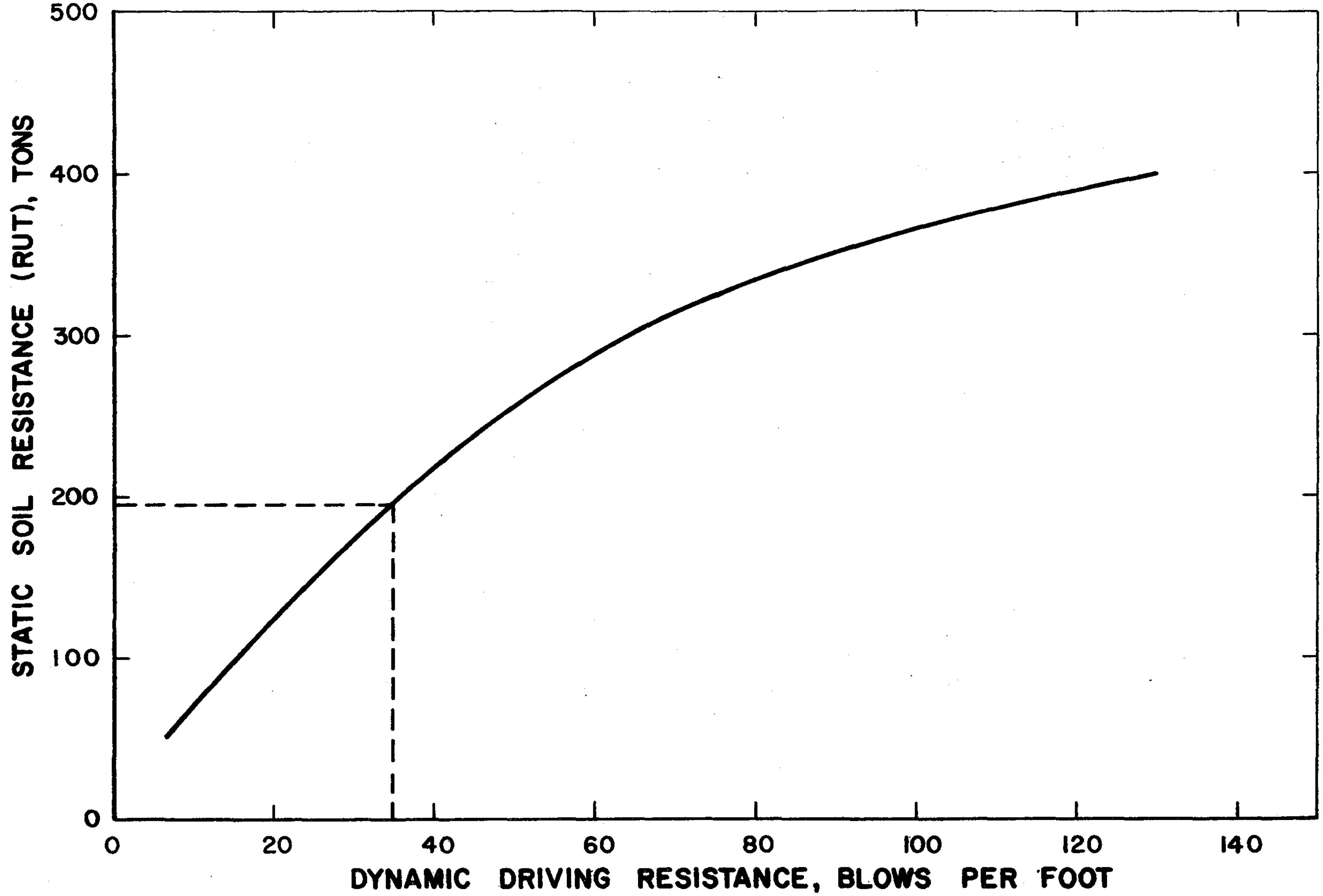

FIG. IV-37.- RUT vs. BLOW COUNT CURVE FOR PADRE ISLAND LTP 22

APPENDIX V. - NOTATION

The following symbols are used in this paper:

e = coefficient of restitution;

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

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

K_c = capblock or cushion stiffness, in kips per inch;

K_p = pile segment stiffness, in kips per inch;

K_{c+p} = combined stiffness of cushion and first pile segment, in kips per inch;

N = a power to which the velocity, V, must be raised for J or J' to be a constant;

NR = number of ram divisions;

OD = outside diameter, in inches;

Q = elastic deformation of the soil, in inches;

$R_{u_{dynamic}}$ = dynamic soil resistance, in pounds;

$R_{u_{static}}$ = static soil resistance, in pounds;

RUP = static point resistance, in kips or tons;

RUT = total static soil resistance, in kips or tons;

RUT_{LT} = total static soil resistance determined by field load test, in kips or tons;

RUT_{WE} = total static soil resistance determined by wave equation analysis, in kips or tons;

RUT_{DR} = total static soil resistance at time of driving, in kips or tons; and

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