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TAMU-SG-71-218 C.O.E. Report No. 144

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Partially supported by the Sea Grant Program,
National Oceanic and Atmospheric Administration,
U.S. Department of Commerce
Institutional Grants GH-59 and GH-101 to
Texas A&M University

Sea Grant Publication No. TAMU-SG-71-218
Coastal and Ocean Engineering Division
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ABSTRACT

Methods to obtain the soil parameters needed to simulate the dynamic response of a laterally loaded pile were developed in this research. Instrumented model piles of various diameters and embedded lengths were driven into stiff clay and tested laterally under free vibration conditions. Field data of bending moments and accelerations versus time were obtained.

In this research the dynamic response of the model piles was predicted using an analytical solution. The nonlinear soil load-displacement characteristics were modeled with a modified Kelvin-Voight rheological model. The field data and the predicted response of the piles were compared and correlated. Using the correlation and laboratory triaxial tests on the soil, the soil parameters required to achieve satisfactory agreement between the field and predicted response of the pile were evaluated.

Results of this study indicate that the two soil parameters, the soil spring and soil quake, which represent the nonlinear characteristics of the soil are functions of the pile diameter. Together these two soil parameters greatly influence the magnitude and distribution of the bending moments with depth. The amount of soil damping is a function of the pile velocity or frequency of vibration and is significant for the velocities and frequencies encountered in this study.

PREFACE

In September, 1968, a research study was initiated to investigate the dynamic response of a laterally loaded pile. During the first year (1968-69) of this study, a numerical method of analysis, adapted for computer usage, was successfully formulated. This work was partially funded by institutional grant GH-26, and research report TAMU-SG-70-224 which covered the first year's work was published.

As a part of a continuing study, Texas A&M University received support in the Sea Grant Program for 1969-70 through institutional grant GH-59. This support was used to investigate the dynamic response of laterally-loaded model piles in clay. Soil parameters were evaluated from field loading tests conducted on instrumented model piles in clay. This report presents the results of this investigation. The field tests were conducted during the summer of 1970 but the analysis of data and the writing of this report was completed during the 1970-71 year under institutional grant GH-101.

This report was written by the senior author in partial fulfillment of the requirements for the Doctor of Philosophy degree. The junior author was the major advisor and principal investigator on the entire project.

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INTRODUCTION

Nature of the Problem. - Many types of offshore structures have recently evolved for many different purposes. The oil companies are perhaps the largest single industry intimately involved with offshore structures and their design. Drilling platforms are being located in deeper water as demands for oil are ever increasing. Storage structures for oil are being built below the ocean surface. Defense installations and weather recording devices are also constructed offshore. As structures are built farther offshore, the design complexities multiply as a result of the more severe sea conditions encountered.

Most offshore structures are supported by piles driven deep into the ocean bottom. The oil companies already have some drilling platforms in water up to 400 feet deep and it will be only a matter of time before they go even deeper or perhaps use some sort of submerged drilling platform. In the design of these structures, frequently the lateral loads imposed by wind, waves, and ice are the critical design factors. These forces are repetitive and are time-dependent. In some instances, such as a breaking wave, these forces could be considered impulsive.

Structures built on pile foundations in seismic regions are also subjected to severe dynamic lateral loads. In the past, these dynamic effects have been given little consideration because of their complexities and the design engineer's inability to cope with them. With the development of the high speed computer, it is now feasible to consider dynamic effects in the design of structures. Knowledge of the soil-pile interaction and water-pile interaction in the case of offshore structures is imperative if the dynamic response of the pile is to be accurately predicted. A literature review showed that much work has been done concerning the soil-pile interaction or water-pile interaction. Data from either full-scale tests or model tests are almost nonexistent and are badly needed if the dynamic response of a laterally loaded pile is ever to be understood.

<u>Present Status of the Question</u>. - Davisson (3) has presented a very complete survey of research on laterally loaded piles through 1960. Much more work on the problem from an analytical viewpoint has been completed in the last ten years due to the increased versatility and use of the electronic computer. Treating a laterally loaded pile as a beam on an elastic foundation, Palmer and Thompson (13), Palmer and Brown (12), and Gleser (6) have each developed a numerical computer solution using finite difference techniques.

Analytical description of the nonlinear soil behavior has always been troublesome to the engineer. Even though some attempts are being made to develop constitutive equations for soils, a generally accepted method is to simulate the soil behavior with some type of rheological model. McClelland and Focht (11) were the first to make a significant attempt to treat the nonlinearity of the soil resistance to lateral deflection. By making empirical correlations between laboratory tests and field tests, they developed nonlinear load-displacement curves. Using the load-displacement curve approach, Matlock and Reese (10) developed an analytical solution for the laterally loaded pile problem that dealt with nonlinear soil-pile interaction using finite difference techniques. These analytical solutions, however, are valid only under static loads and cannot be applied for dynamic loads.

In the early 1960's, Smith (22) and Samson, Hirsch, and Lowery (20) introduced a modified form of the Kelvin-Voight model in pile driving analysis. Tucker (25) in 1964 made the first attempt to handle analytically the dynamic laterally loaded pile problem. Using a finite element representation of the pile, he treated the soil as an elastic medium and used a finite difference technique for solution. In 1970, Ross (19) developed an analytical solution for the dynamic response of an offshore pile when subjected to lateral loads. He utilized Airy or linear wave theory for the water-pile interaction and used a finite element representation of the pile. Ross considered a nonlinear soil load-displacement relationship by using the modified Kelvin-Voight model of Smith (22). Rheological models have been used for years by engineers to define the resistance of a material undergoing dynamic loading. A schematic diagram of the model used by Ross and its load-displacement characteristics are shown in Fig. 1. The dynamic resistance of the model is expressed in equation form as:

$$P_{Dynamic} = P_{Static} \, (1 + JV^N) \tag{1}$$

where: $P_{Dynamic}$ is the dynamic load

P_{Static} is the static load

J is the soil damping factor

V is the velocity of the pile

N is the power to which the velocity must be raised for J to be constant.

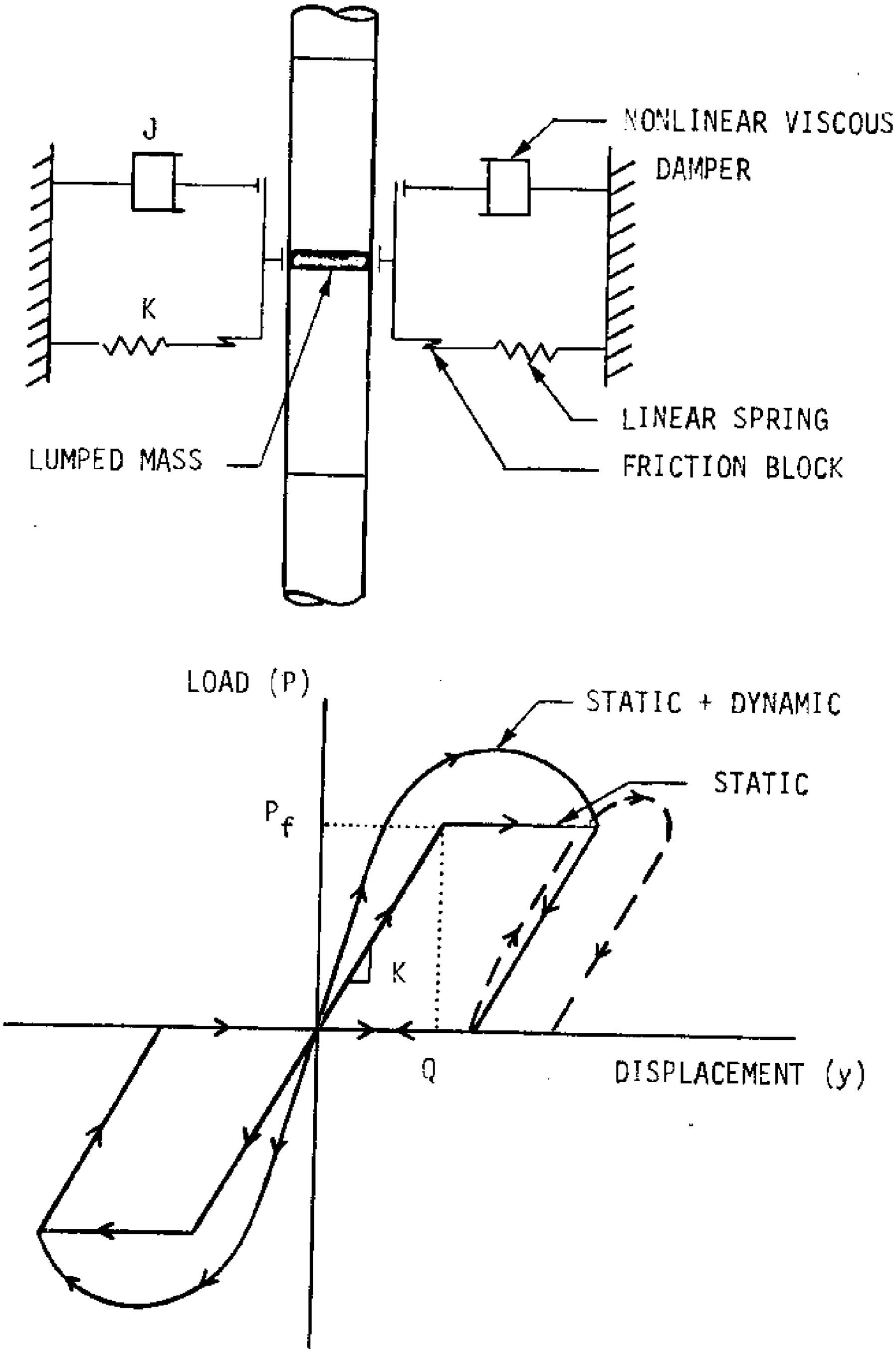
The static load, P_{Static}, is the product of a linear soil spring, K, and a displacement, y, and is expressed in equation form as:

$$P_{Static} = Ky \tag{2}$$

It should be noted that when the displacements are greater than the maximum elastic displacement or quake of the soil, Q, equation 2 becomes:

$$P_{Static} = KQ; \; y \geq Q \tag{3}$$

The linear soil spring, K, can be evaluated in several different ways. Perhaps the best and most reliable method is by conducting horizontal load tests on full-scale piles embedded in the soil and instrumented with strain gages. How-



MODIFIED SMITH EQUATION

$$P_{DYNAMIC} = P_{STATIC}\ (1 + JV^{N}) \quad 0.0<N<1.0$$

FIG. 1.- SOIL RHEOLOGICAL MODEL AND LOAD-DISPLACEMENT CHARACTERISTICS [After Ross (19)]

ever, because of the expense of these field tests, it is desirable to be able to determine K from either laboratory tests or from plate load tests. The value of K for a laterally loaded pile can be determined by multiplying the coefficient of horizontal subgrade reaction, k_H, by the loaded area of the pile segment. Terzaghi (24) suggests determining a coefficient of subgrade reaction, k_s, from vertical plate loads using one foot square rigid plates. The value of k_H and therefore K can then be determined for a pile by dividing k_s by 1.5 times the pile diameter according to Terzaghi (24).

Vesic (26) developed a method of calculating k_H from the soil and pile properties that can be determined in laboratory tests. He expressed k_H as a function of: (1) pile width and stiffness, and (2) Poisson's ratio of the soil and the secant modulus of the soil measured at one half the ultimate load in an undrained triaxial test. Vesic's equation can be simplified and expressed in terms of the pile width, the vertical coefficient of subgrade reaction, and a factor, α. The factor, α, is a function of the pile properties and soil strength and has been found by Broms (1) to vary between 0.32 and 0.40 for steel piles driven into cohesive soils.

The variation of k_H with depth is very important. Several different distributions have been assumed for clays by various investigators. Palmer and Brown (12) expressed k_H as an exponential function of depth. Davisson and Gill (4) have assumed it to be constant or a step function with depth. Reese and Matlock (17) have assumed k_H to be directly proportional to depth. Gill and Demars (5) have indicated that such simplified assumed relationships are deficient because of the nonhomogeneity of naturally occurring soils as well as the nonlinear load-displacement characteristics of a soil. The introduction of load-displacement curves by McClelland and Focht (11) and the computer solution by Matlock and Reese (10) allow for a more rigorous approach. Their methods allow the use of any variation of k_H with depth for static analysis. The true distribution of k_H depends on the type of soil, the soil properties, and the stress history of the soil.

The magnitude of the soil quake, Q, is closely related to the value of the soil spring, K. By definition, Q is the maximum elastic ground deformation and can be determined from a triaxial stress-strain curve. Q can be calculated if the failure load, P_f, (See Fig. 1) and the soil spring, K, are known. McClelland and Focht (11), Gill and Demars (5), Broms (1), Reese (16), and Matlock (9) have all presented methods which can be used to calculate the lateral soil resistance in terms of load per length of pile. When used in conjunction with the equation for displacement of McClelland and Focht (11), Skempton (21), Parker and Cox (14), or Matlock (9), a soil resistance-displacement curve can be generated from which Q can be determined.

Concerning the evaluation of J, Smith (22) arbitrarily set J equal to 0.15. Coyle and Gibson (2) modified Smith's equation by raising the velocity, V, to some power, N. For the highly plastic clays tested, they found N = 0.18 was necessary in order to give J a relatively constant value. They then determined J values empirically from laboratory impact tests. Tucker (25) included a damping factor in his solution but gave no value or method to obtain it. Penzein (15) suggested conducting free vibration tests on clay samples to determine J. Taylor and Hughes (23) concluded that energy dissipation in soil approximates viscous damping at low amplitudes and elasto-plastic characteristics at high amplitudes. They also noted that since the elastic soil modulus is strongly dependent on the strain amplitude, different values will be obtained for different testing techniques.

To date, available field data for laterally loaded piles have been limited to static loading conditions. Data from dynamic tests on laterally loaded piles are not available with which to compare the predicted pile response obtained from the analytical solution developed by Ross. Therefore the test program conducted in this study was considered necessary.

Objectives. - The objectives of this study are:

1. To conduct a series of free-vibration dynamic tests in the field using model, laterally-loaded, instrumented piles driven into stiff clay.
2. To compare the measured dynamic response of the model

test piles with the response predicted by the analytical solution developed by Ross (19).

3. To determine the soil parameters necessary to achieve agreement between the measured and predicted dynamic response of the model test pile.

4. To develop theoretical and/or laboratory methods for use in predicting the soil parameters.

FIELD TESTS

Test Site. - In 1969, a research project on the bearing capacity of axially loaded piles was conducted at Texas A&M University by Rehmet and Coyle (18). A test site consisting of a relatively uniform stiff clay extending to a depth of ten feet was used at the south end of the third runway at the Texas A&M Research Annex. A soil boring was made by Rehmet and Coyle (18), and the boring log developed from this boring is shown in Fig. 2. The clay was considered to be highly plastic as its plasticity index varied from 29 to 44. Water content determinations were made at the time of testing and unconsolidated-undrained triaxial tests were performed on samples procured from the test site. The triaxial tests were made on 1.5 inch diameter undisturbed samples and the resulting stress-strain curves are presented in Appendix III. The unconsolidated-undrained test data were used because the loading rate due to the pile vibration was fast enough to cause an undrained loading condition in the clay.

Test Program. - Table 1 presents the test program which was used in this study. The program consisted of a total of 25 tests made on piles of three different diameters. The data for all 25 tests are tabulated in Appendix IV. An identification number was assigned each test to simplify and shorten the

DEPTH (FT.)	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIT WEIGHT LBS./CU. FT.	PLASTIC LIMIT	LIQUID LIMIT	MOISTURE CONTENT, %	UNDRAINED SHEAR STR. (PSI)
2			DARK CLAY FIRM TO STIFF	124.4	20	49	25.4	16.4
4			DARK CLAY FIRM TO STIFF		19	48	27.0	
6			DARK GRAYISH CLAY, FAIRLY STIFF	124.7	21	56	23.6	28.9
8			RED SANDY CLAY, WITH SOME CALCAREOUS NODULES		19	56	24.0	
10			RED SANDY CLAY, WITH SOME CALCAREOUS NODULES	125.3	21	65	21.0	58.3

LEGEND

TYPE OF SAMPLE	SOIL SYMBOL
PUSH BARREL	DARK CLAY
AUGER	RED SANDY CLAY

FIG. 2. - BORING LOG OF TEST SITE [After Rehmet and Coyle (18)]

Table 1. - Test Program

Test Identification Number	Nominal Pile Diameter in Inches	Embedded Length in Feet	Length Above Groundline in Feet	Number of Tests	Comments
3-8-1 thru 3-8-3	3	8	7	3	Tested 1 day after driving
3-8-4 thru 3-8-6	3	8	7	3	Tested 18 days after driving
2-6-1 and 2-6-2	2	6	7	2	Tested 1 day after driving
2-8-1 and 2-8-2	2	8	7	2	Tested 1 day after driving
2-8-3 thru 2-8-5	2	8	7	3	Tested at higher frequencies
2-8-6 and 2-8-7	2	8	7	2	Tested in water environment
2-10-1 and 2-10-3	2	10	7	3	Tested 1 day after driving
1-8-1 thru 1-8-3	1.25	8	7	3	Tested 1 day after driving
1-8-4 and 1-8-5	1.25	8	7	2	Tested 14 days after driving
1-8-6 and 1-8-7	1.25	8	7	2	Tested 30 days after driving

presentation of the test results. The first number of this identification number is the pile diameter, the second number is the depth of pile embedment, and the last number is the number of the test made at the specified depth. Other details concerning the tests are listed in the comments section of Table 1. For example, Test 3-8-1 denotes the first test on the 3 inch diameter pile at an embedded depth of eight feet. It was tested one day after it was driven.

Piles of three different diameters were tested at an embedded depth of eight feet to check the effects of different diameters on the response of the pile. The 2 inch pile was tested at embedded depths of six, eight, and ten feet to check the effects of varying the embedded lengths. The locations of the strain gages below the groundline were kept the same in all tests as was the length of the pile above the groundline.

In all tests except Tests 2-8-3 thru 2-8-5 and Tests 1-8-1 thru 1-8-7, a 50 pound weight was mounted at the top of the pile in order to reduce the frequency of vibration. The 13 pound attachment used to hold the 50 pound weight increased the total weight to 63 pounds. For Test 2-8-3, the 50 pound weight was removed and for Test 2-8-4, the 13 pound weight attachment was also removed. For Test 2-8-5, a 20 inch section of the pile was removed from the top. These three tests which resulted in higher frequencies were made to study any effect that changing the frequency may have on the pile response.

Tests 2-8-6 and 2-8-7 were conducted while the pile was surrounded by water approximately three feet deep. These two tests were conducted in an attempt to more closely simulate the response of an offshore pile by accounting for the water-pile interaction.

Test Pile Properties and Instrumention. - The model test piles were constructed of standard steel pipe. Since standard steel pipe is rolled and has varying wall thicknesses, measurements of the inside and outside pile diameters were averaged and used to calculate the average cross-sectional area, A, and the average moment of inertia, I, of each pile. These pile properties are presented in Table 2. In all succeeding discussions, pile diameters will be referred to according to the nominal diameter of the pipe used.

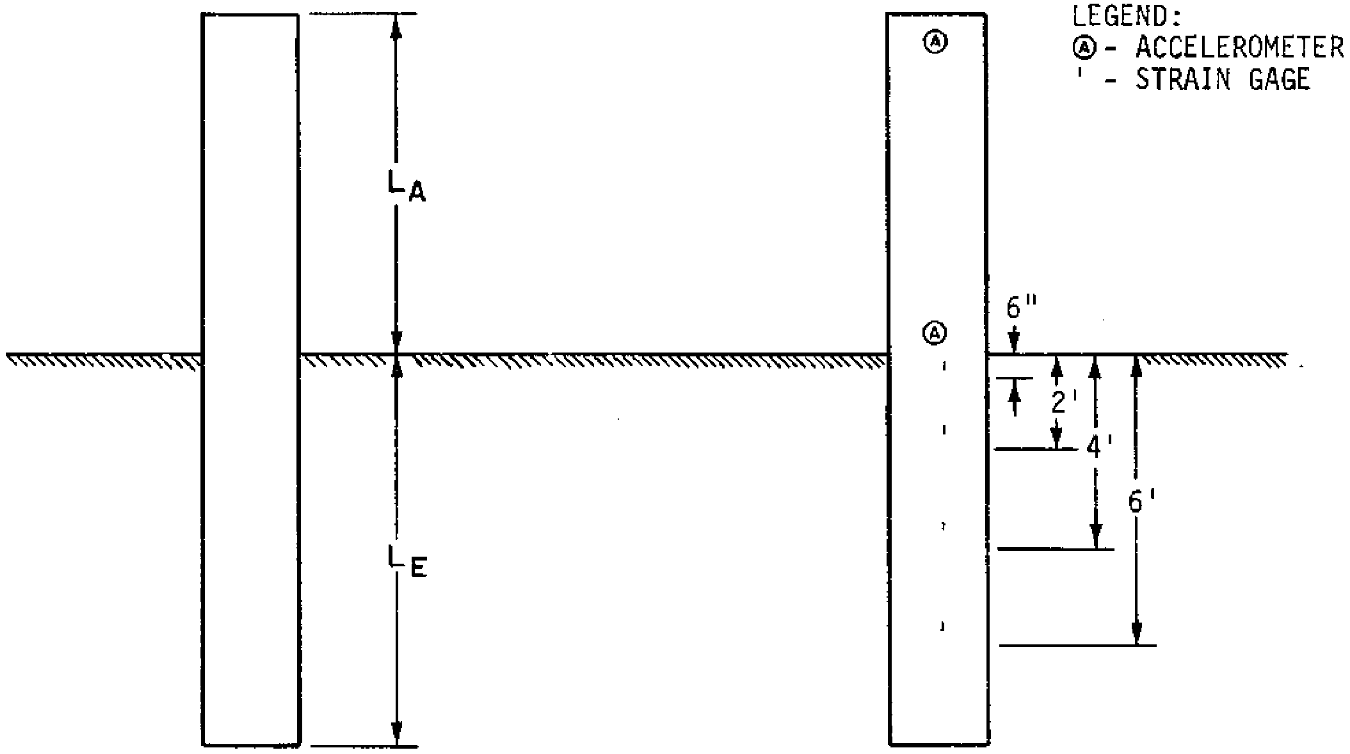
Table 2. - Pile Properties

Nominal Diameter in Inches	Average Outside Diameter in Inches	Average Inside Diameter in Inches	Average Cross-Sectional Area in Inches Squared	Average Moment of Inertia in Inches to the Fourth Power
3	3.50	3.09	2.145	2.846
2	2.383	2.091	1.026	0.6445
1.25	1.667	1.392	0.661	0.195

A test was made on the 3 inch pile to check its actual stiffness with the calculated stiffness. For a given load, the measured deflections and those calculated, assuming an average wall thickness, agreed within ten percent. However, the method used to measure the deflections was not exact. The maximum deviation in deflection was 0.02 inches and the deflection measurements were accurate to ±0.01 inches.

Each test pile was instrumented with four full bridges of strain gages wired to obtain bending moments along the longitudinal axis of the pile. Each bridge consisted of four strain gages with two gages being on each side of the pile where flexure stresses were a maximum. The strain gage bridges were placed so that after the pile was driven to its required depth, the gages would be located at depths below the groundline of six inches, two feet, four feet, and six feet as shown in Fig. 3. In order to install the strain gages on the inside of the pile, it was cut into longitudinal segments. After the gage installation, the pile segments were carefully aligned so that the strain gages would measure the moments about a common axis of bending, and were then welded back together. The welding added to the nonuniformity of the pile but was necessary for the gage installation.

In addition to the strain gages, two accelerometers were attached to the pile, one at the groundline, and one at the pile top as shown in Fig. 3. Strains and accelerations were



A. ORIENTATION

B. INSTRUMENTATION LOCATION

FIG. 3. - PILE ORIENTATION AND INSTRUMENTATION LOCATION

recorded with respect to time with a Honeywell Visicorder on light sensitive paper. Also available on several of the tests was an optical tracker which allowed the displacements at the point of the applied force to be recorded during the time of vibration.

Test Procedure. - To more closely simulate the actual soil conditions in the field, each pipe pile was driven into the stiff clay with a drop hammer. The driving of the pile disturbed and remolded the soil into some unknown condition. A length of pipe with a 63 pound concentrated weight at the top was added to the pile above the ground. The pile was loaded horizontally by means of a cable. A quick release mechanism consisting of an electrically operated hoist was used to release the pile. After release, the pile was allowed to vibrate freely until its motion was damped out.

After analysis of the test data, several shortcomings of the test procedure were discovered. The piles were pulled horizontally by a cable at a distance approximately two feet above the groundline. This caused the pile to be bent into a shape other than the shape of its fundamental mode of vibration. Higher frequencies were introduced into the system because the pile was not deformed into its fundamental shape. These higher frequencies quickly disappeared, usually in the first cycle. If the pile were pulled at the top, the pile would be deformed in its fundamental mode and these higher frequencies would not exist. Each pile was pulled over a different distance and hence a different force was required. It would have been much easier to detect any deviations of the response of the pile due to different diameters or lengths of embedment if all the piles had been pulled with the same force.

An instantaneous release is required for tests of the type made in this study. The electrically operated hoist released the pile too slowly causing the true horizontal load on the pile at the time of release to be lower than the load recorded. As a result, the moments obtained just before release are larger than the actual moments that existed at time zero. Therefore, in correlating the predicted moments with the field moments, the moments at the time of release or time zero were not used in this study. Instead the correlation begins one half cycle later at the first dynamic moment peak.

SOIL PARAMETERS FOR THE MODIFIED KELVIN-VOIGHT MODEL

As mentioned previously, the modified Kelvin-Voight model used by Ross (19) to describe the behavior of the soil load-displacement characteristics for a laterally loaded pile involves using several soil parameters. These parameters which must be determined if the model is to be useful are (1) a linear soil spring, K, (2) a soil quake, Q, and (3) a soil damping factor, J.

Soil Spring, K. - The determination of a correct value of K and its distribution with depth can be determined from instrumented pile tests. It is desirable to be able to determine K from a simple laboratory test or a field test. The soil spring, K, can be calculated from the coefficient of horizontal subgrade reaction, k_H. For brevity, k_H will be referred to as the soil modulus, and will be designated with a lower case k with subscripts. The soil spring will be designated with a non-subscripted capital K. The soil spring, K, and soil modulus, k_H, are related in equation form as follows:

$$K = k_H \, LD \tag{4}$$

where: L is the length of the pile segment

D is the pile diameter

Therefore if k_H is known, K can be calculated by multiplying k_H

by the horizontal projected area of the loaded pile segment.

As a first attempt to determine a value of k_H, several different theoretical methods published in the literature were investigated. Terzaghi (24) proposed a range of soil modulus values for a one foot square plate in a stiff clay based on the knowledge of the deformation characteristics of stiff clay. Using the average value of soil modulus proposed by Terzaghi (24), a k_H = 198 lb. per in.^{3} was determined for the 3 inch pile. Vesic (26) developed a theoretical method by which k_H could be calculated based on the theory of a beam on an elastic foundation. Using Vesic's equation and the soil properties determined from the triaxial tests presented in Appendix III, a k_H = 193 lb. per in.^{3} was calculated. Broms (1) simplified Vesic's equation and using Broms method, a value of k_H = 261 lb. per in.^{3} was obtained for a 3 inch pile. As shown in Table 3, each of these theoretical methods yield results of the same order of magnitude.

Table 3. - Values of k_H for a 3 Inch Pile
In a Stiff Clay

Theoretical Method	k_H in Pounds Per Cubic Inch
Terzaghi	198
Vesic	193
Broms	261

McClelland and Focht (11) have presented a method of relating empirically the results of full-scale static pile tests to triaxial laboratory test data. They suggest that the stress on a pile in the field is 5.5 times the deviator stress obtained from a consolidated-undrained triaxial test for both at the same percent strain. They also suggest that the strain in the field, ε_F, is equal to the pile displacement divided by the pile radius. Using these two suggestions to obtain a field stress versus field displacement curve, a field modulus of horizontal subgrade reaction, k_F, can be obtained from the tangent slope of this curve. Parker and Cox (14) also related strain and displacement, suggesting the field strain be determined by dividing the pile displacement by the pile diameter. Based on footing tests, Skempton (21) has suggested a third possible method to determine the field strain. He suggests dividing the displacement by twice the pile diameter. Calculating the field stresses using McClelland and Focht's suggestion and using the three suggested methods to determine field strains, the values of k_F for the model piles are presented in Table 4. In each case k_F was the slope of the tangent on the field stress versus field pile displacement curve. A typical curve used to determine k_F at a four foot depth for the 3 inch pile is shown in Fig. 4. These curves are generated by first calculating the field strain for an assumed pile displacement. The laboratory deviator stress, σ_Δ, is then determined from the laboratory

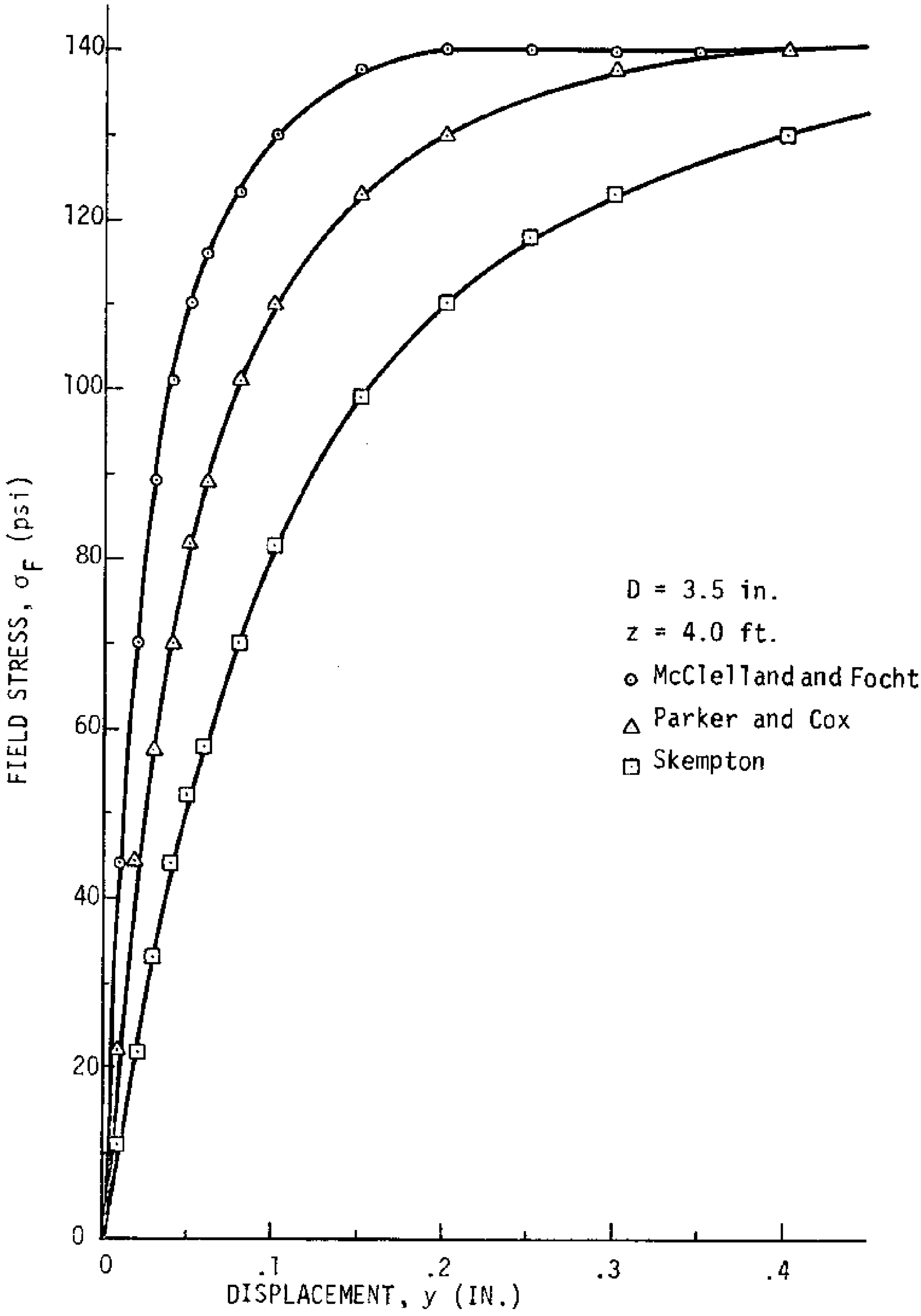


FIG. 4. - FIELD STRESS VERSUS DISPLACEMENT FOR 3 INCH PILE AT 4 FOOT DEPTH

stress-strain curve for the corresponding field strain. The field stress for that strain is then calculated as 5.5 σ_Δ as suggested by McClelland and Focht (11).

Table 4. - Values of k_F at z = 4 Feet

Method	ε_F in Percent	k_F in Pounds Per Cubic Inch		
		3 Inch Pile	2 Inch Pile	1.25 Inch Pile
McClelland and Focht	2y/D	3300	5160	6600
Parker and Cox	y/D	1650	2580	3300
Skempton	y/2D	825	1290	1650

All soil modulus values are extremely large as compared to the values in Table 3. Since the value of 5.5 suggested by McClelland and Focht (11) was based on empirical data from a 24 inch pile tested in soft clay, the validity of applying their method to the model piles in this study is questionable. It was found that by using the strain relationship of Parker and Cox (14)

$$\varepsilon_F = \frac{y}{D} \tag{5}$$

and by assuming the field stress was equal to the triaxial deviator stress,

$$\sigma_F = \sigma_\Delta \tag{6}$$

a soil modulus value of the same order of magnitude as the values in Table 3 could be obtained.

Instead of using the field strain-displacement relationships which are functions of the pile diameter, it was felt that the soil modulus should be a soil parameter which is constant for any given soil. Therefore, a curve of the triaxial deviator stress versus the laboratory soil deformation, ΔL, was used. This laboratory soil modulus is designated as k_L since it is determined as the tangent slope of a laboratory unconsolidated-undrained triaxial test.

The family of curves in Fig. 5 was generated by testing undisturbed soil samples at various depths. Using k_L for each depth as the slope of the tangent of the corresponding σ_Δ versus ΔL curve, a k_L distribution with depth was obtained. This distribution was used in the prediction of the pile response in this study and is shown in Fig. 6. For this study, a soil spring was placed at the location of each strain gage as well as at the groundline and pile tip. Also shown in Fig. 6 with the k_L distribution with depth is the complete idealized pile used in this study.

Since the lengths of the soil specimens in the triaxial tests were greater or equal to the pile diameter, the determination of the soil modulus from a σ_Δ - ΔL plot is considered reasonable. The value of k_L is independent of the pile diameter and is strictly a soil property. The values of K are dependent on the pile diameter as indicated by Equation 4 and

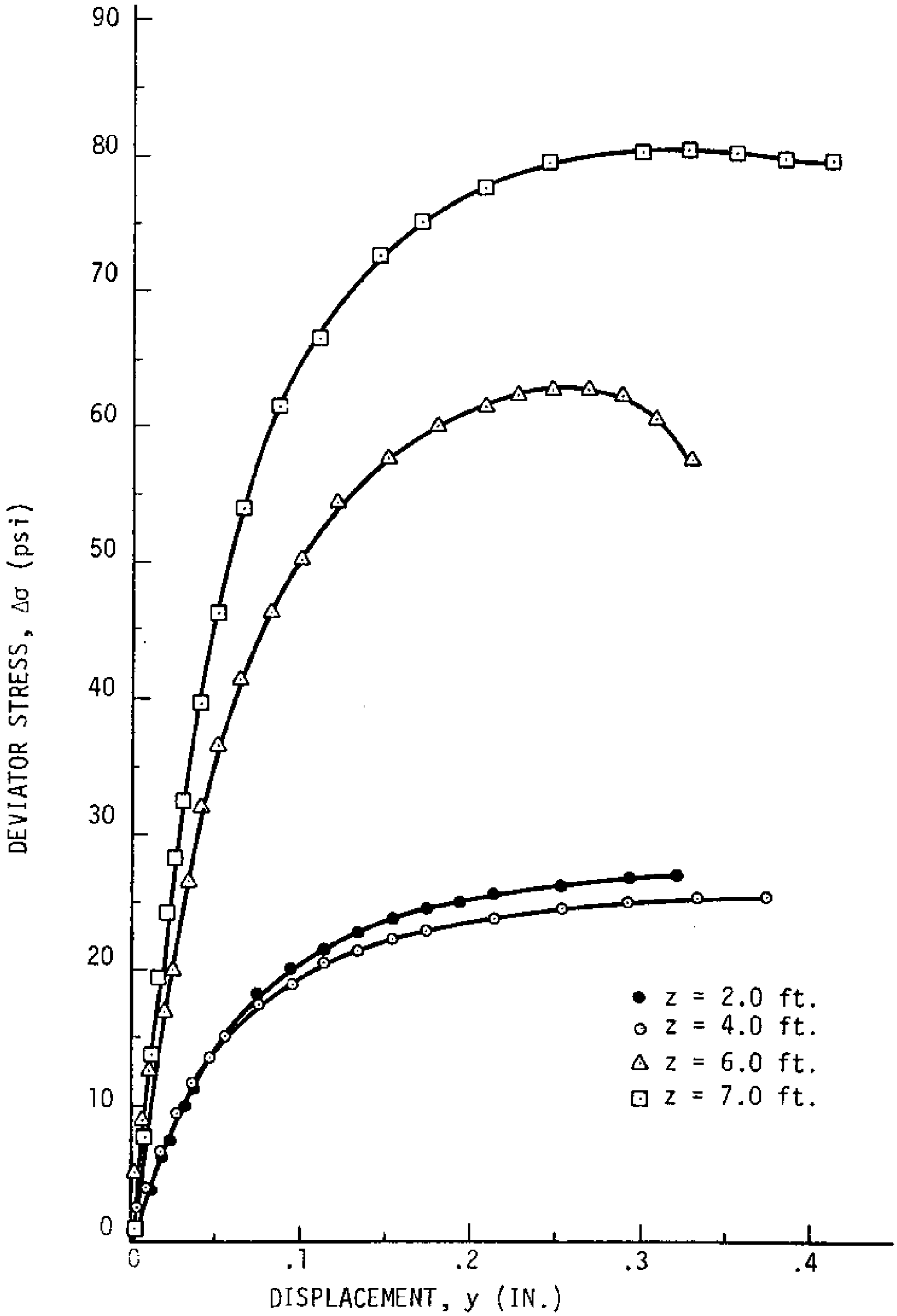


FIG. 5. - DEVIATOR STRESS VERSUS DISPLACEMENT FOR VARIOUS DEPTHS, z.

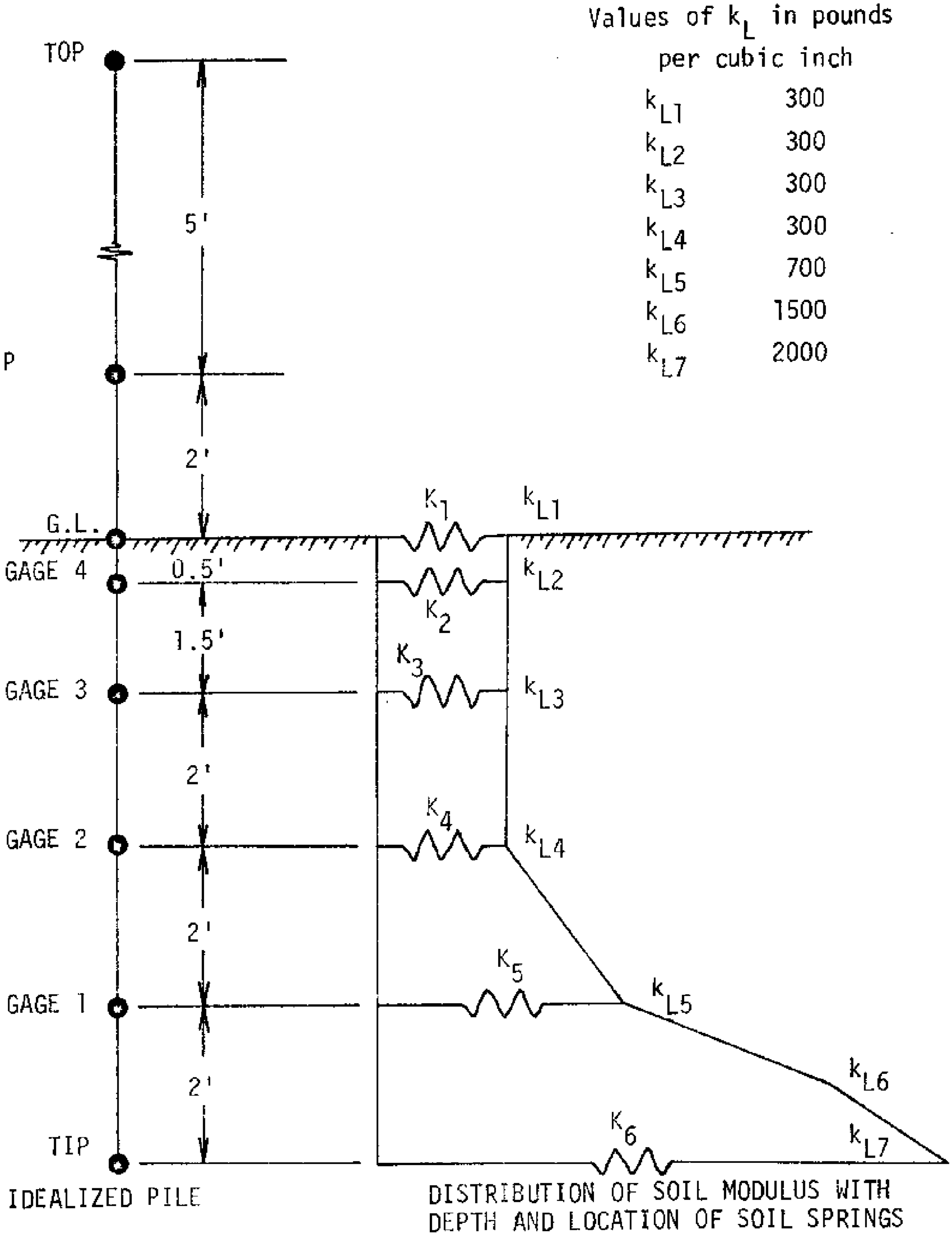


FIG. 6. - IDEALIZED PILE, DISTRIBUTION OF SOIL MODULUS WITH DEPTH AND LOCATION OF SOIL SPRINGS

increase with corresponding increases in pile diameter.

Soil Quake, Q. - The magnitude of the soil quake, Q, is closely related to the soil spring values. By definition, the soil quake is the maximum elastic soil deformation. It seems reasonable that Q is a function of the pile diameter and should be reduced as the pile diameter decreases. From work done by Lowery, et.al. (8) on axially loaded piles, a Q = 0.10 in. for a full-scale field pile has proved to be satisfactory. Therefore a value of Q = 0.10 in. was assumed to be valid for a full-scale laterally loaded pile. It was noted that a Q = 0.10 in. is approximately one percent of a one foot diameter full-scale pile. Therefore for the model piles used in this study, a Q was calculated for each pile using

$$Q = .01D \tag{7}$$

The value of Q for the 3 inch model pile was calculated to be 0.035 in. and a Q = 0.024 in. and Q = 0.017 in. were calculated for the 2 inch and 1.25 inch piles respectively. These values were used in this study, and in most cases, produced reasonable agreement between the predicted and measured pile response.

Fig. 7 shows how reducing Q affects the ultimate soil resistance as well as the effect that the pile diameter in Equation 4 has on the soil spring, K, at a given depth. Equation 4 introduces the effect of the pile diameter on K. Each pile has a different K depending on its diameter with the smaller diameter piles having a smaller contact area between the pile and soil

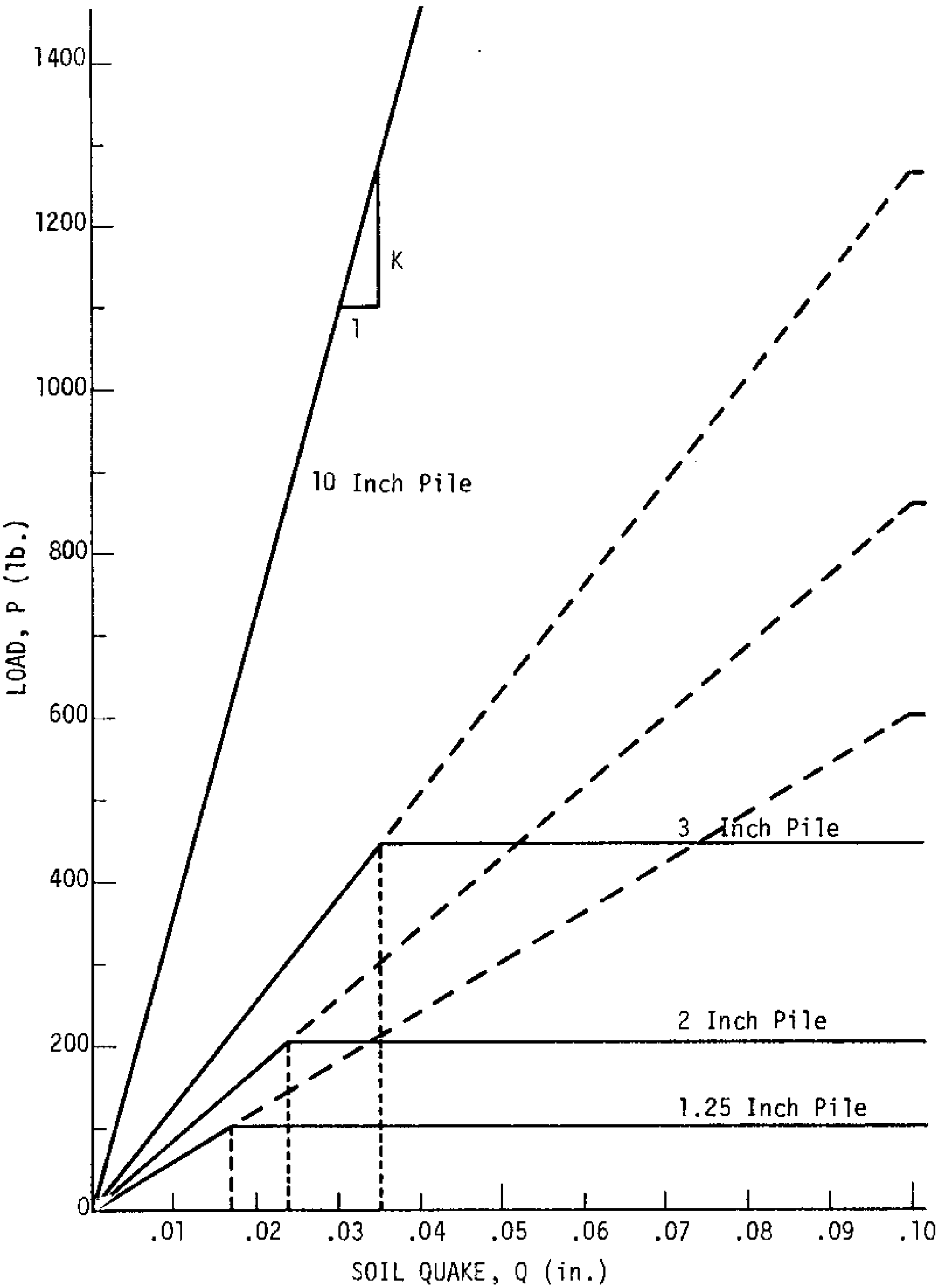


FIG. 7. - EFFECT OF REDUCING SOIL QUAKE, Q, ON THE LOAD, P, FOR A 12 INCH PILE SEGMENT AT z = 2.0 FEET.

and hence a lower K. Consider now the effect of reducing Q. First, consider a constant Q = 0.10 in. for all piles. This case is indicated by the dashed lines in Fig. 7. Because of the different K values resulting from the use of Equation 4, the smaller piles have lower ultimate load values at a common Q value. By reducing Q as the pile diameter decreases, as indicated in Equation 7, the allowable ultimate load is reduced even further. This effect is shown in Fig. 7 by the solid lines. For example, by reducing the Q for the 3 inch piles from 0.10 in. to .035 in. by applying Equation 7, the ultimate soil load is reduced from 1260 lb. to 445 lb. which is a reduction of 65 percent. The 1.25 inch pile failure load is reduced 83 percent from 600 lb. to 101 lb. by applying Equation 7. Consequently, the Q value is a very important soil parameter and greatly effects the prediction of the pile response as it controls the load at which soil failure occurs for a given pile segment.

Soil Damping Factor, J. - The value of the viscous damping factor, J, was first suggested to be 0.15 by Smith (22). To obtain a value of J for a stiff clay, Coyle and Gibson (2) conducted a series of laboratory impact tests. Since the value of J varied with the velocity of loading as indicated in Equation 1, they found that by raising the velocity to the power N = 0.18, J would remain relatively constant for all velocities. Therefore, in this study a value of N = 0.18 was used. From the tests by Coyle and Gibson (2), an initial average value of J = 0.575 was used and found to be low for the tests in this study. Using the soil spring values determined from Fig. 6, a Q = 0.035 and a N = 0.18 for the 3 inch pile, various J values were assumed until agreement was achieved between the predicted and measured amount of damping. A value of J = 2.0 was found to give the best correlation between the measured and predicted responses for the 3 inch pile and was used for all piles.

COMPARISON OF FIELD AND PREDICTED PILE RESPONSE

As mentioned previously, bending moments with respect to time were determined at four points below the groundline from measured strains and accelerations were measured at the groundline and at the pile top as shown in Fig. 3. All data taken in this study are in Appendix IV. In all tests the moments at the six foot depth, Gage 1, were very small and are not presented or discussed in this study. Gages 2 refers to the strain measurements made at a four foot depth; Gage 3 refers to the strain measurements made at a two foot depth; and Gage 4 refers to the strain measurements made at a six inch depth. Accelerometer data will be presented only for the 3 inch diameter pile tests as these data were not used to determine the parameters needed for the soil model. Acceleration data were recorded on all 25 tests and are tabulated in Appendix IV.

In addition to the soil parameter values of K, Q, J, and N needed for use in the Ross solution, a value for structural damping is needed. A preliminary test was made to determine this value. It was found that the structural damping was only 0.3% of critical damping and for these model piles could be neglected. The deflected shape of the pile at the time of release is also needed as an initial condition and is very critical. A finite element computer program was used to determine this deflected shape. A known force was applied to the idealized pile above the groundline. Values of the soil springs and the soil quake were assumed and an initial deflected shape of the pile determined for those assumed values. This deflected shape was then used as the initial condition in the Ross solution to predict the dynamic response of the laterally loaded piles. On several of the tests, the displacement at the point of force application was known and was used to verify the initial deflected shape.

3 Inch Pile Tests. - A total of six tests were made on the 3 inch pile, all at an eight foot depth. The data recorded from these tests were considered the most reliable as it was the largest of the three piles tested and any effects of gaps between the pile wall and the soil due to the driving were minimized. The first three tests were conducted one day after the pile was driven. An optical tracker was available for these three tests allowing displacements with respect to time to be recorded. The tracker target was located at the pull line, approximately two feet above the groundline, allowing the displacements to be measured at this point. Figs. 8 thru 11 present the data that were recorded from Test 3-8-2. Also in these same figures is the predicted response of the computer program using a $Q = 0.035$, a $J = 2.0$, a $N = 0.18$ and K values determined from Fig. 6. These parameters were determined by the methods discussed in the previous section. Only the peak values of moment and

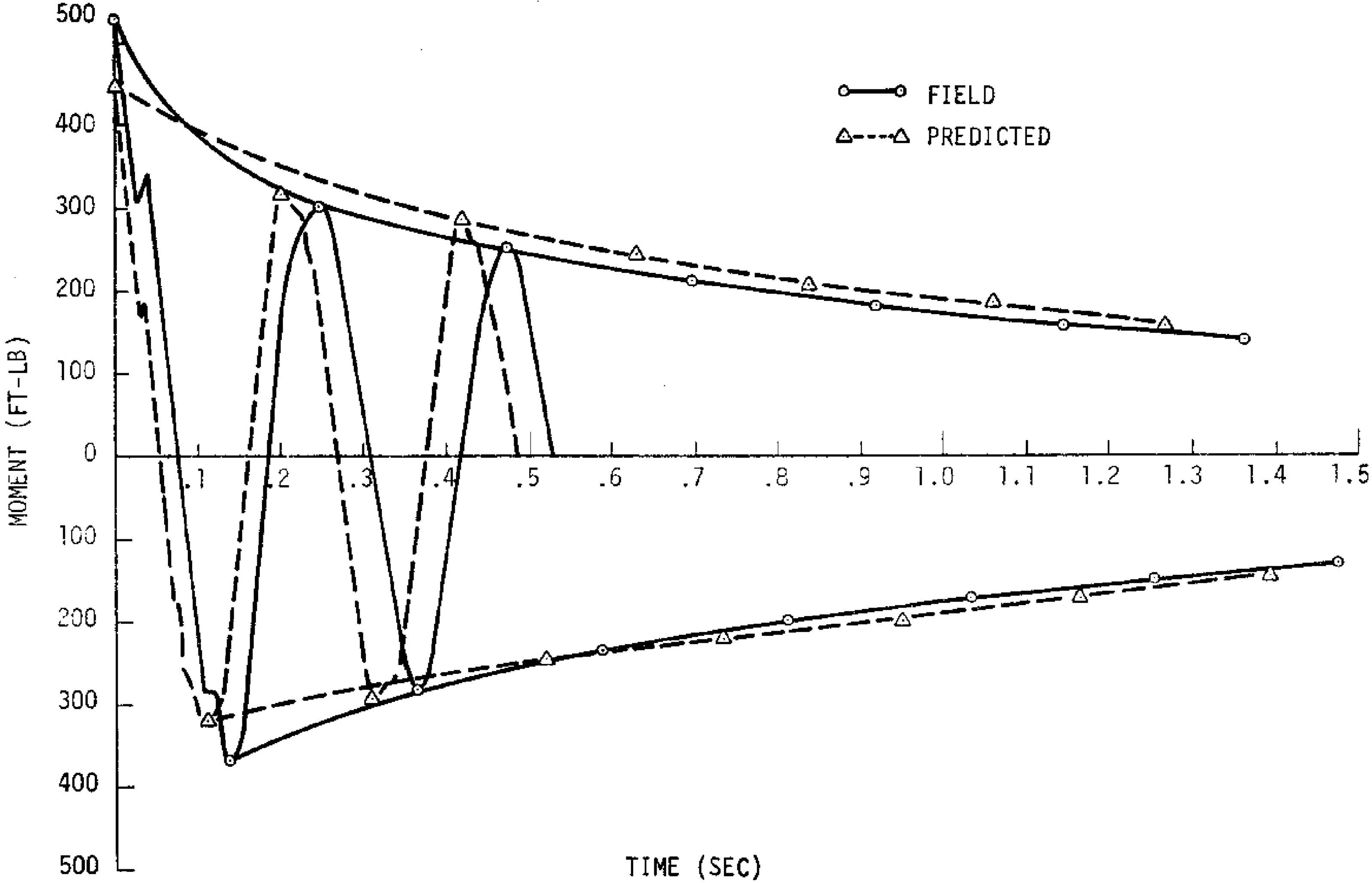


FIG. 8. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-2, z = 0.5 FEET.

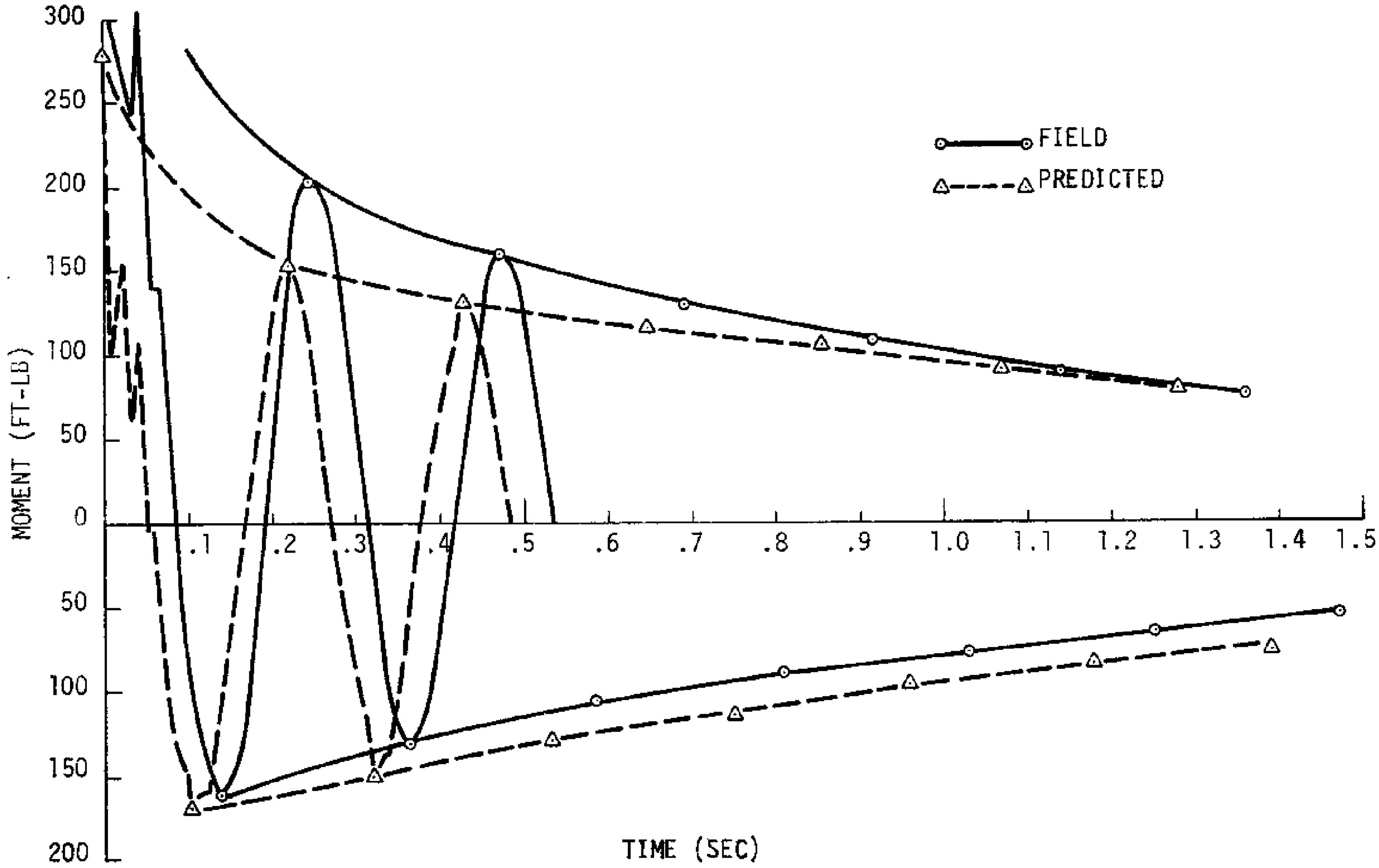


FIG. 9. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-2, z = 2.0 FEET.

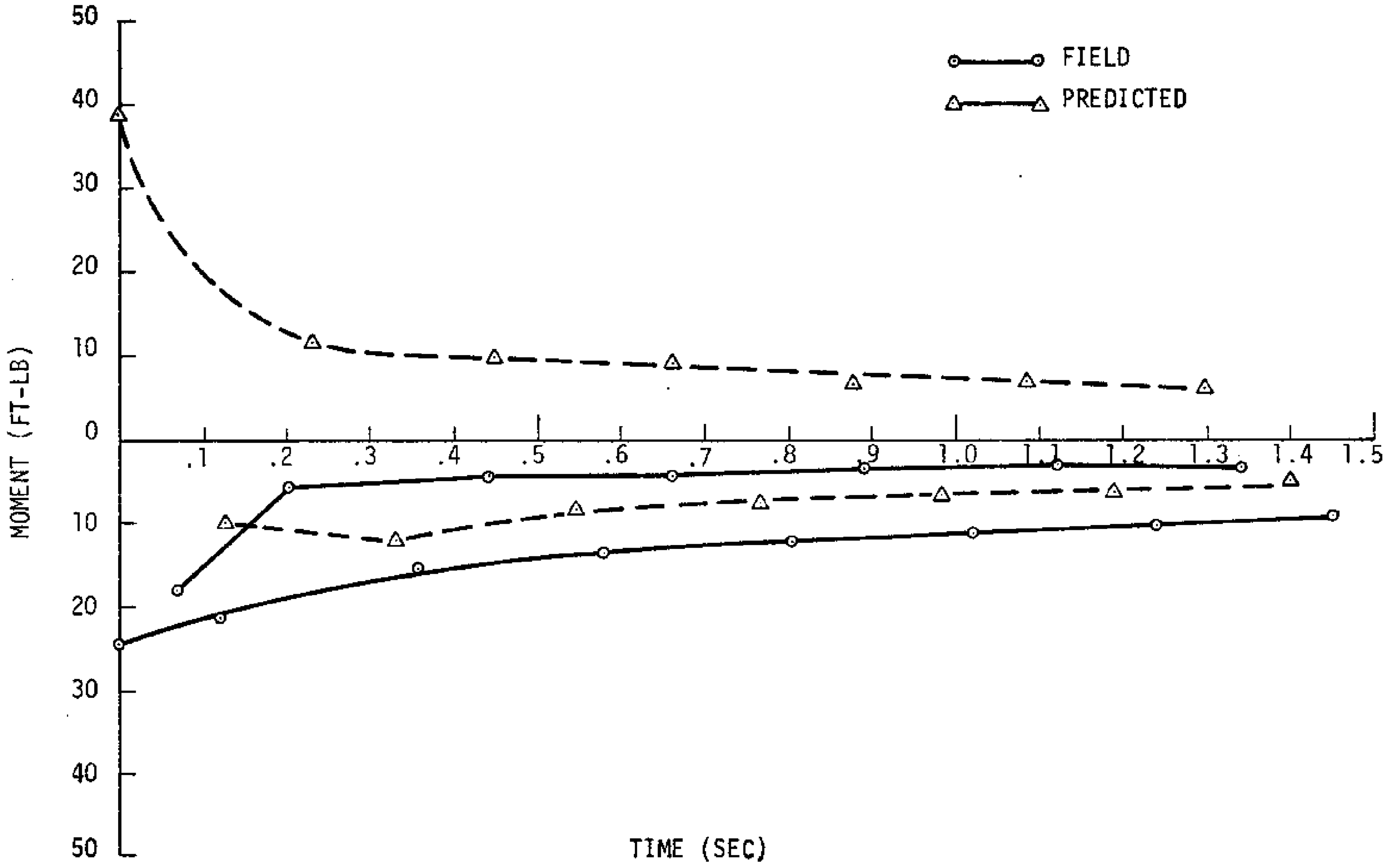


FIG. 10. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-2, z = 4.0 FEET.

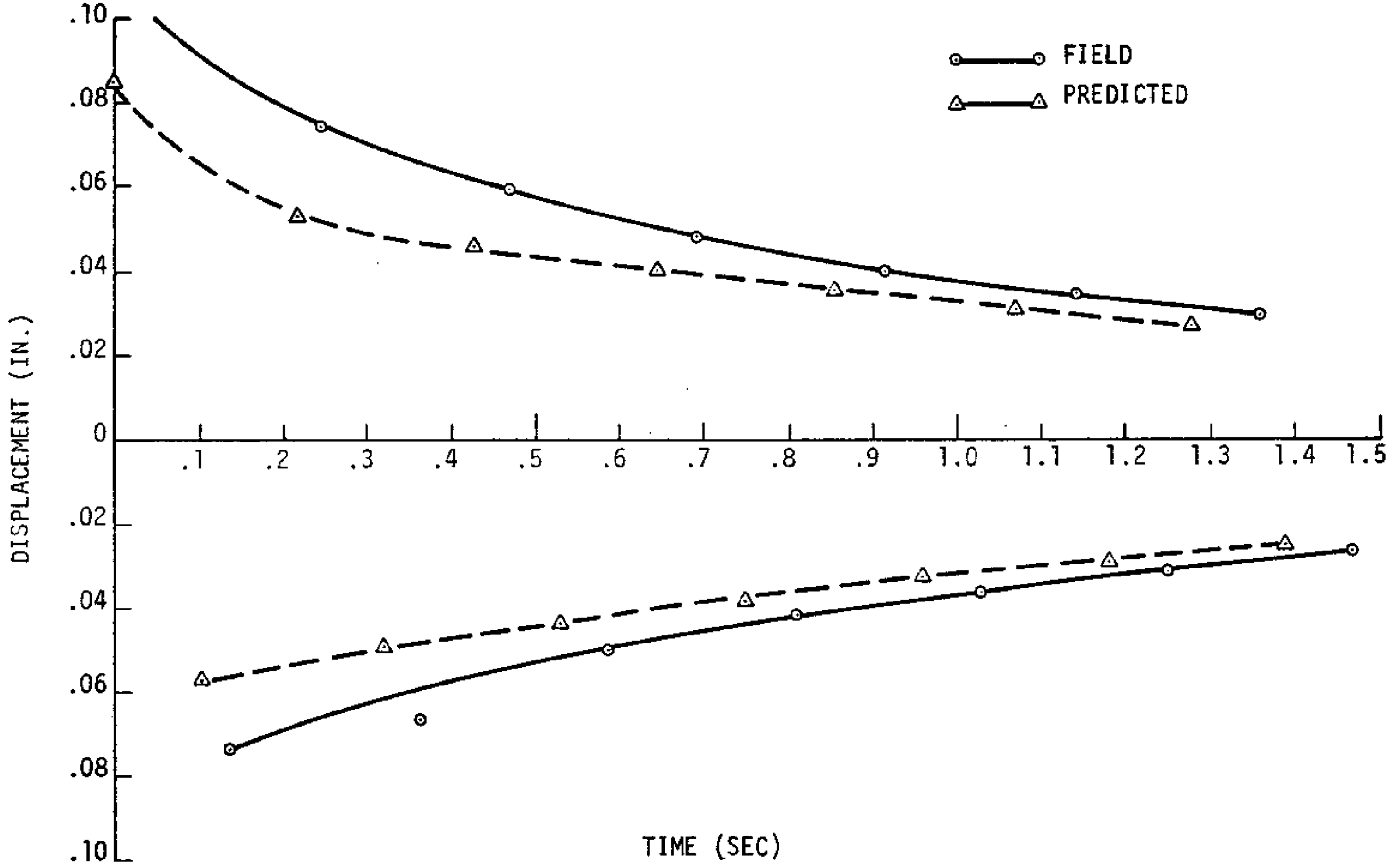


FIG. 11. - DISPLACEMENT VERSUS TIME FOR TEST 3-8-2, AT POINT OF LOAD.

displacement were plotted even though the entire sinusoidal shaped curve was recorded. By using only the peaks, the frequency of vibration can be compared as well as the magnitude of the moments and the rate of damping. The frequency measured was 4.5 cycles per second and the frequency predicted was 5.05 cycles per second, an error on the order of ten percent. The frequencies measured at all gage locations were the same for these three tests on the 3 inch pile.

Fig. 8 shows that the moment peaks at Gage 4 (6 inch depth) were within 25 foot-pounds of the field moments, an error of less than ten percent. Since the maximum moments occur in its vicinity, it is the most critical depth. The moments at Gage 3 (Fig. 9) showed a shift of the axis of oscillation which should be the zero moment line. As seen in Fig. 10, this shift occurred to an even greater extent in the measurements made at the four foot depth by Gage 2. These shifts occurred in all six tests on the 3 inch pile as well as on the tests of the other two diameter piles. Gage 4 always oscillated symmetrically about the zero moment line but Gage 3 always showed a slight shift in the direction of the applied initial force. Gage 2 usually underwent a large enough shift so that it oscillated completely above or below the zero moment line. Since the values of the moments at Gage 2 were small as compared to the moments at Gages 3 and 4, the deviations from the predicted moments were considered unimportant as they were of the same relative magnitude.

Initially it was thought that the shift might be due to faulty instrumentation since the computer predicted no such shift. However, after carefully checking, this possibility was eliminated and it was decided that this shift actually occurred. Since the shift always occurred in the direction of the pull-over, it is believed that during the pullover, the stresses in the soil around the pile were relieved on the side opposite from the direction of pull. This allowed the clay to rebound some amount thus changing the original zero moment position of the pile. At Gage 3 the forces due to the pile vibrating were large enough so that the pile came close to returning to its original position. However, at Gage 2 where the movements were very small, there was not enough force to fail the rebounded soil and allow the pile to return to its original position. It should be noted that the displacements at Gage 2 were less than 0.001 inches.

If this is the correct explanation of this shift, then the soil properties must be changing with time as the pile vibrates and would be different on each side of the pile. As presently written, the computer program can handle only a soil that has the same properties on both sides of the pile. The program could be easily modified to handle the situation of different soil properties. If the pile had been pulled each direction the same amount before testing, these shifts would probably not have occurred as the soil would have been in the same state of stress on both sides.

The displacements recorded by the optical tracker for Test 3-8-2 are shown in Fig. 11. The measured displacements were higher than the predicted displacements which may be due to a small hole existing around the pile because of the pile driving.

Figs. 12 and 13 show the values of accelerations measured and predicted for Test 3-8-2. All of the values of the predicted accelerations at the pile top are too large but are within 20 percent of the measured accelerations. The accelerations at the groundline predicted by the computer are difficult to distinguish because of the many high frequencies the computer introduces. The high frequencies are probably caused by the pile hitting the soil when vibrating or possibly because the pile was not displaced into its fundamental node shape. Some higher frequencies were recorded but tended to damp out quickly whereas they did not disappear in the predicted accelerations from the computer. The predicted and measured values are in the same order of magnitude, however.

After a period of 18 days, the 3 inch pile was tested three more times. The purpose of these additional tests was to see if any soil set up occurred and if so, what its effect was on the pile response. The optical tracker was not used for these tests. The bending moment versus time graphs for Test 3-8-5 are shown in Figs. 14 thru 16. Relative magnitudes

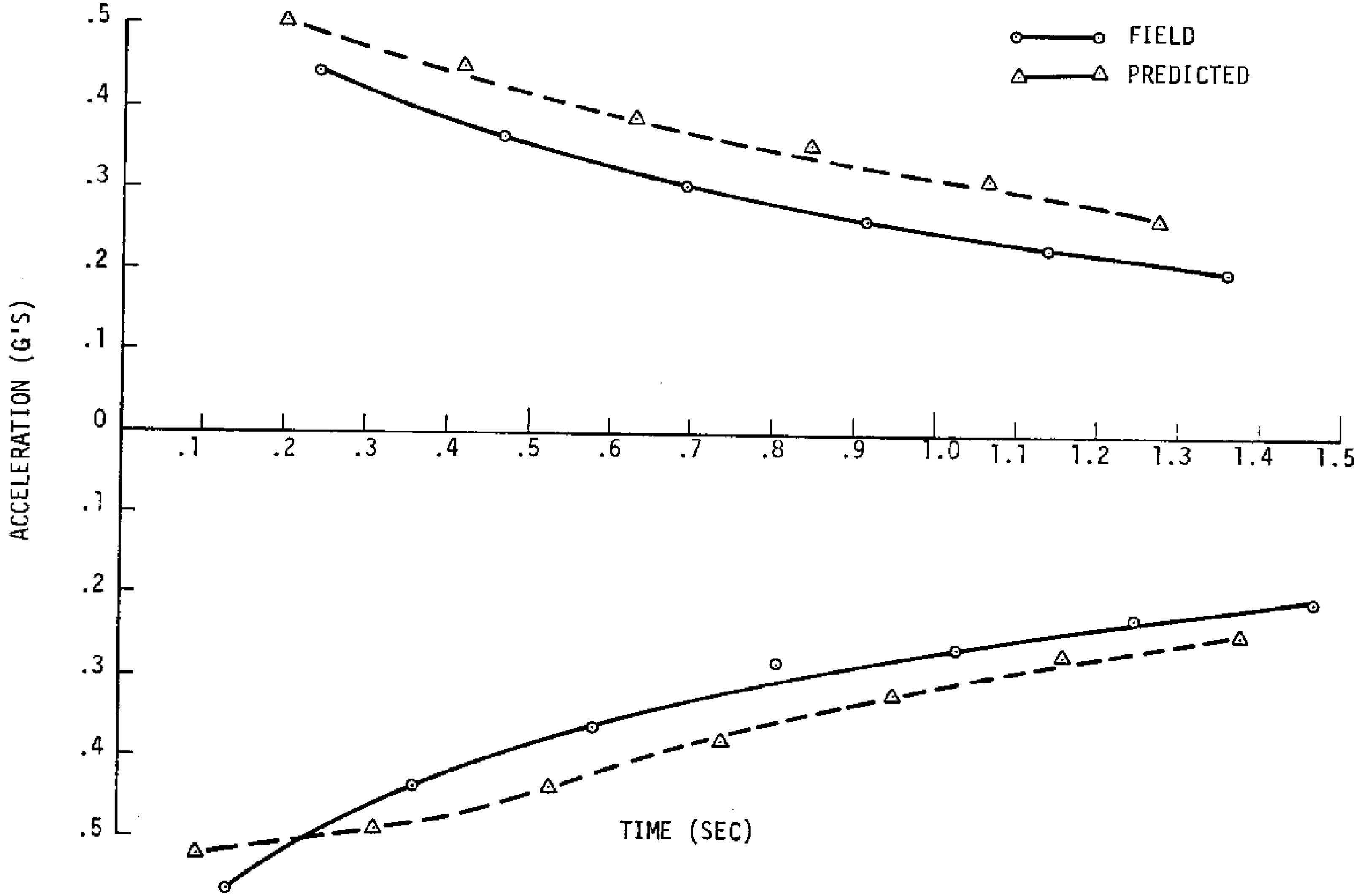


FIG. 12. - ACCELERATION VERSUS TIME AT PILE TOP FOR TEST 3-8-2.

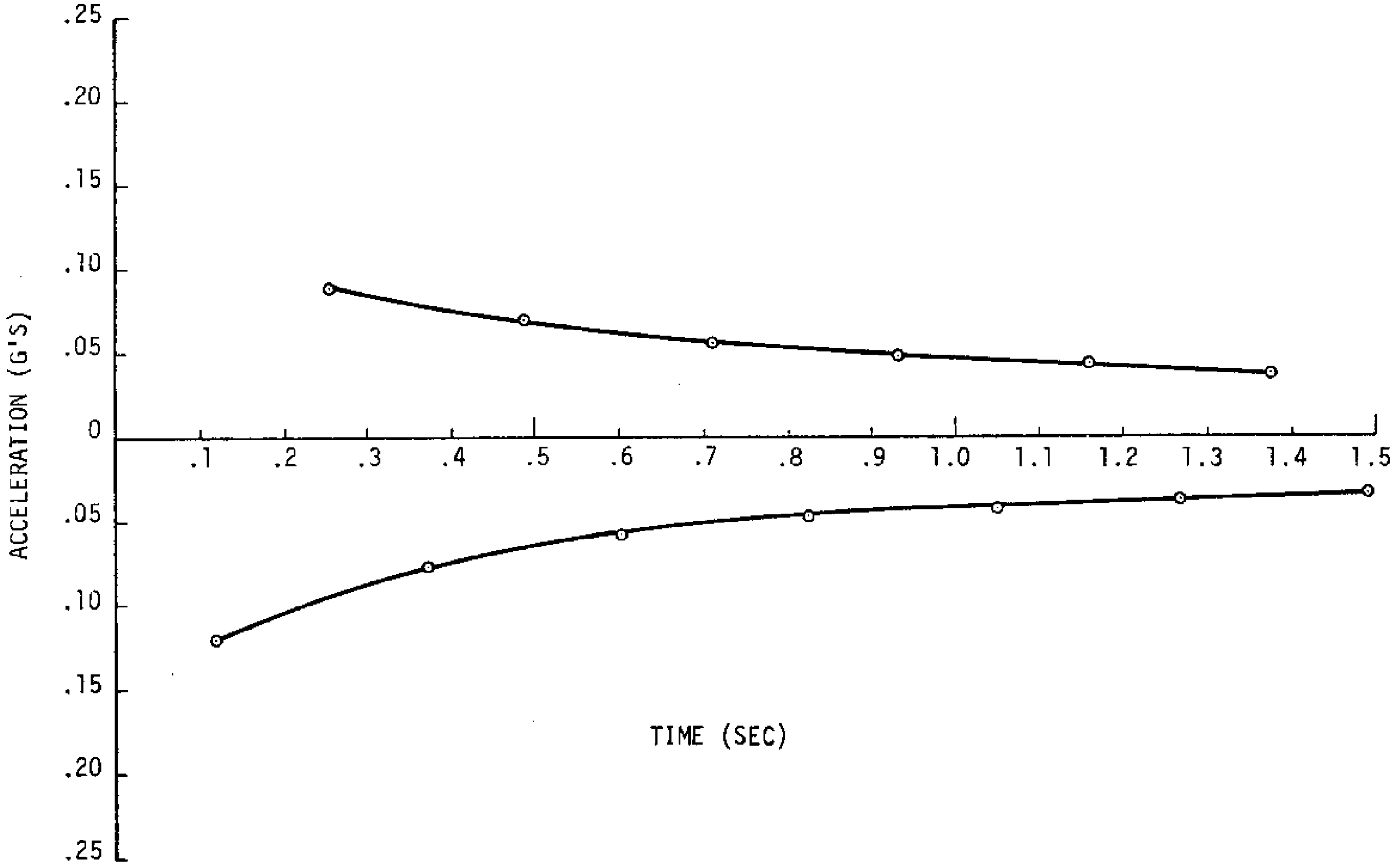


FIG. 13. - ACCELERATION VERSUS TIME AT GROUNDLINE FOR TEST 3-8-2.

of the predicted moments and moments at Gage 4 and Gage 3 agree to within 11 percent. Both Gages 3 and 2 again failed to oscillate symmetrically about a line of zero moment. The predicted and measured amount of damping was very close. The measured frequency of vibration increased to 4.7 cycles per second from the 4.5 cycles per second frequency measured on Test 3-8-2. The pile was pulled over farther in this test than in Test 3-8-2 and thus reduced the predicted frequency of vibration from the 5.05 cycles per second predicted in Test 3-8-2 to 4.7 cycles per second. Fig. 17 presents the measured and predicted accelerations at the pile top. The accelerations agree within 20 percent at all times.

Another method of presenting the same data is shown in Fig. 18. The bending moment curve versus depth is plotted for each positive and negative wave of propagation. The solid lines are the predicted curves with the symbols representing the actual field moments. This method of presentation stresses the importance of relative magnitude of the moments for all gages and shows that in design, Gages 4 and 3 are much more critical than the very small values of Gage 2. This method will not be used as it is felt that the method of bending moment versus time curves presents the data more clearly, especially the frequency of vibration and amount of damping.

1.25 Inch Pile Tests. - A total of seven tests were conducted on the 1.25 inch pile. This was the smallest diameter

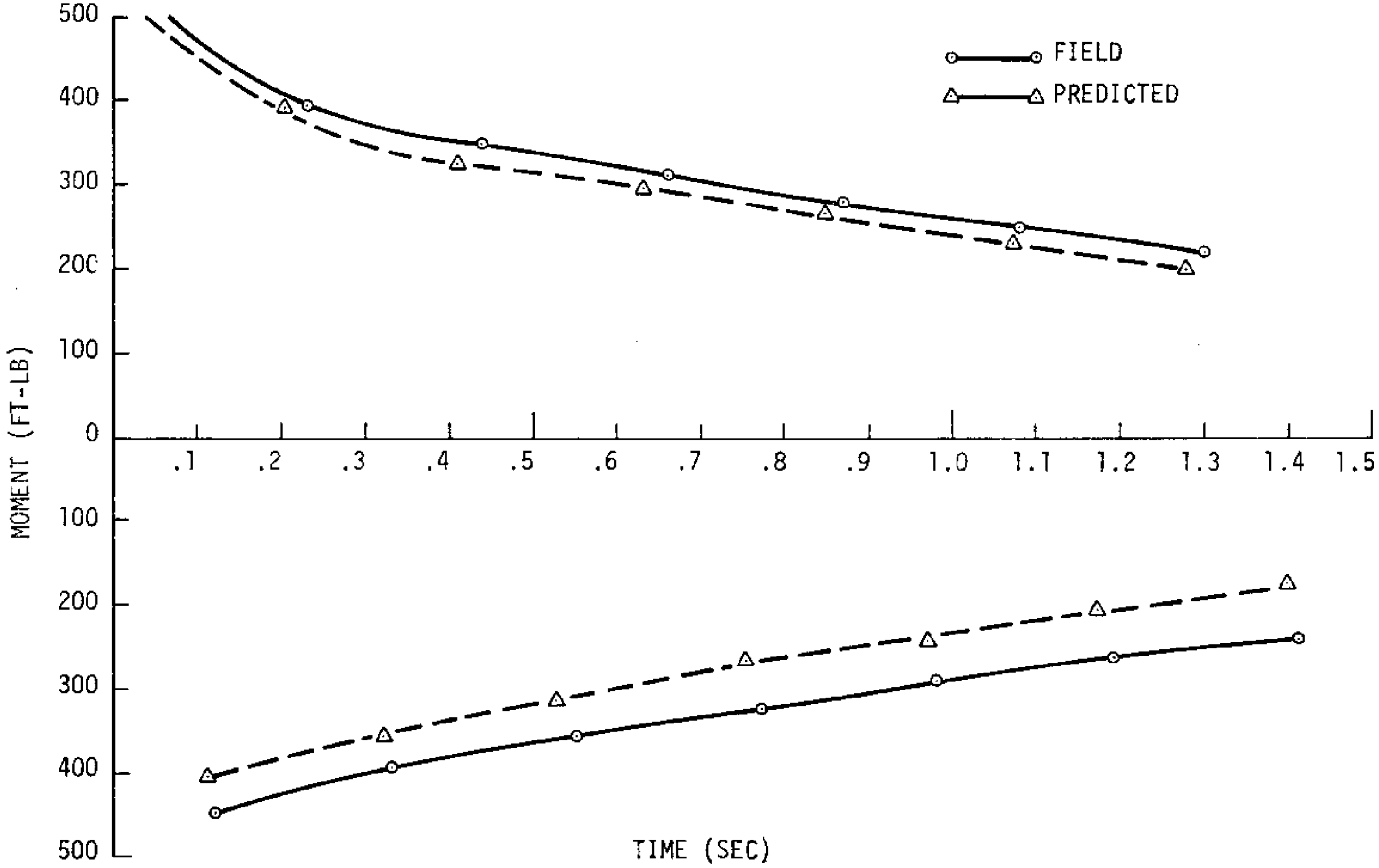


FIG. 14. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-5, z = 0.5 FEET.

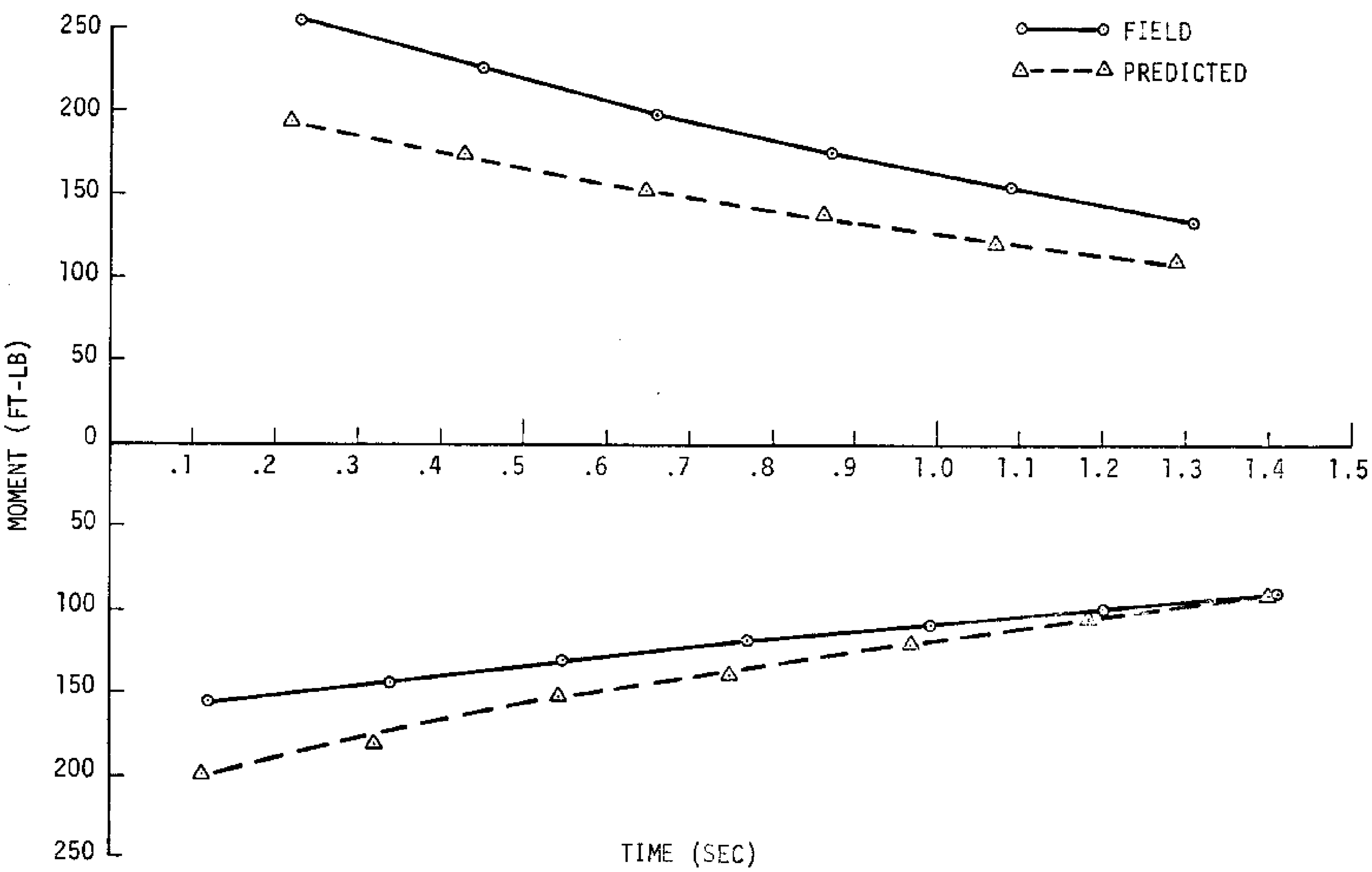


FIG. 15. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-5, z = 2.0 FEET.

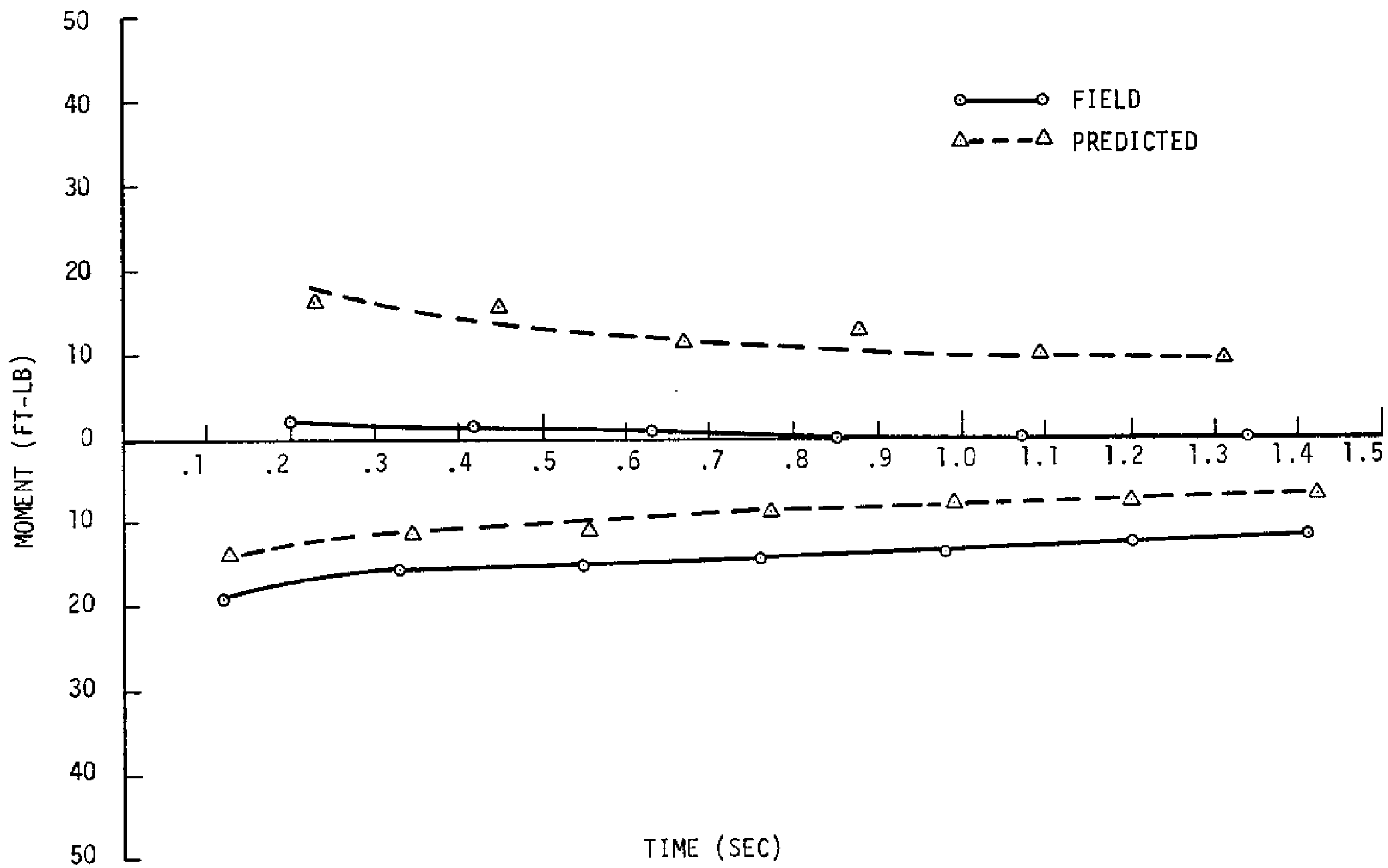


FIG. 16. - BENDING MOMENT VERSUS TIME FOR TEST 3-8-5, z = 4.0 FEET.

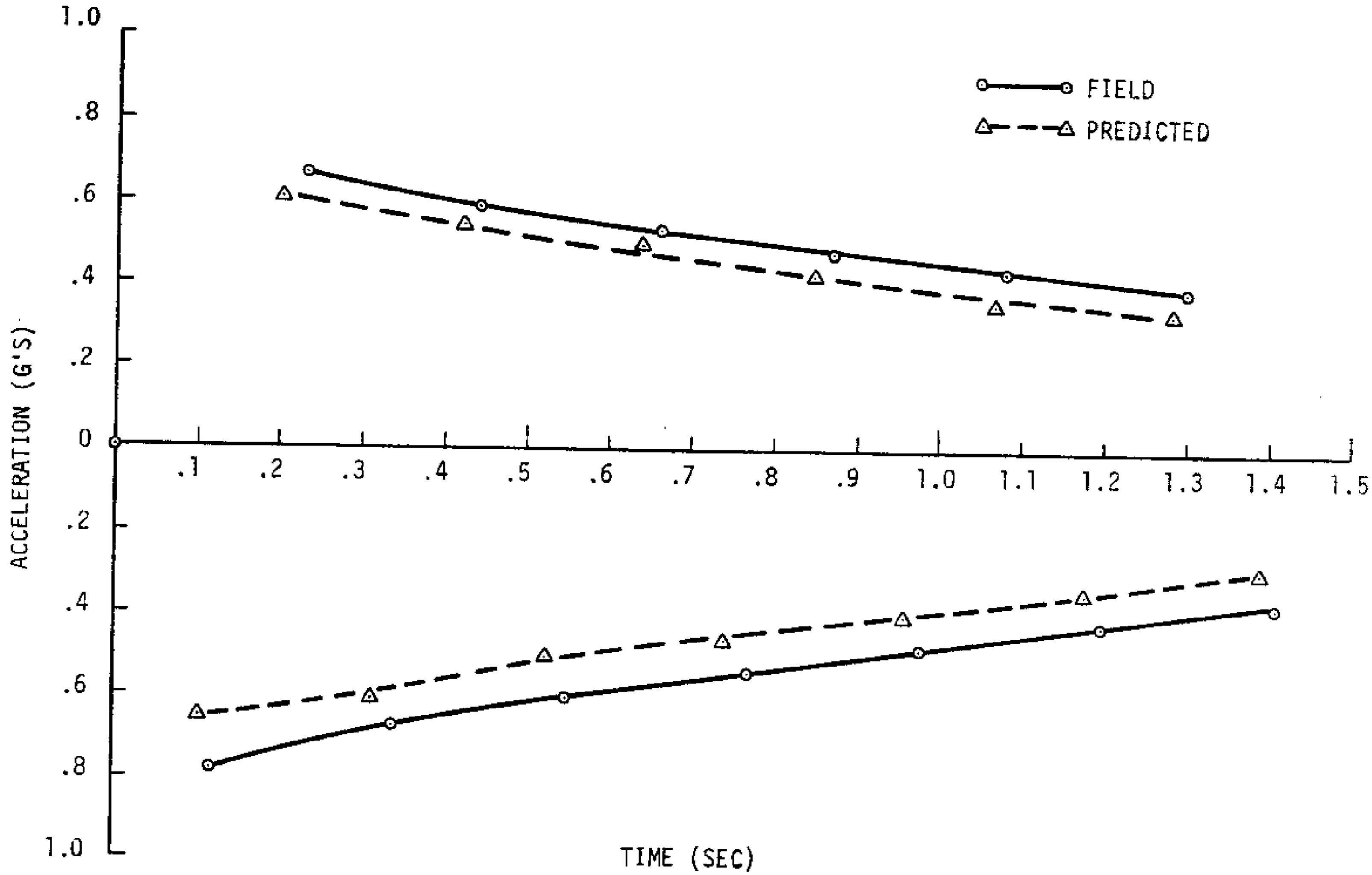


FIG. 17. - ACCELERATION VERSUS TIME AT PILE TOP FOR TEST 3-8-5.

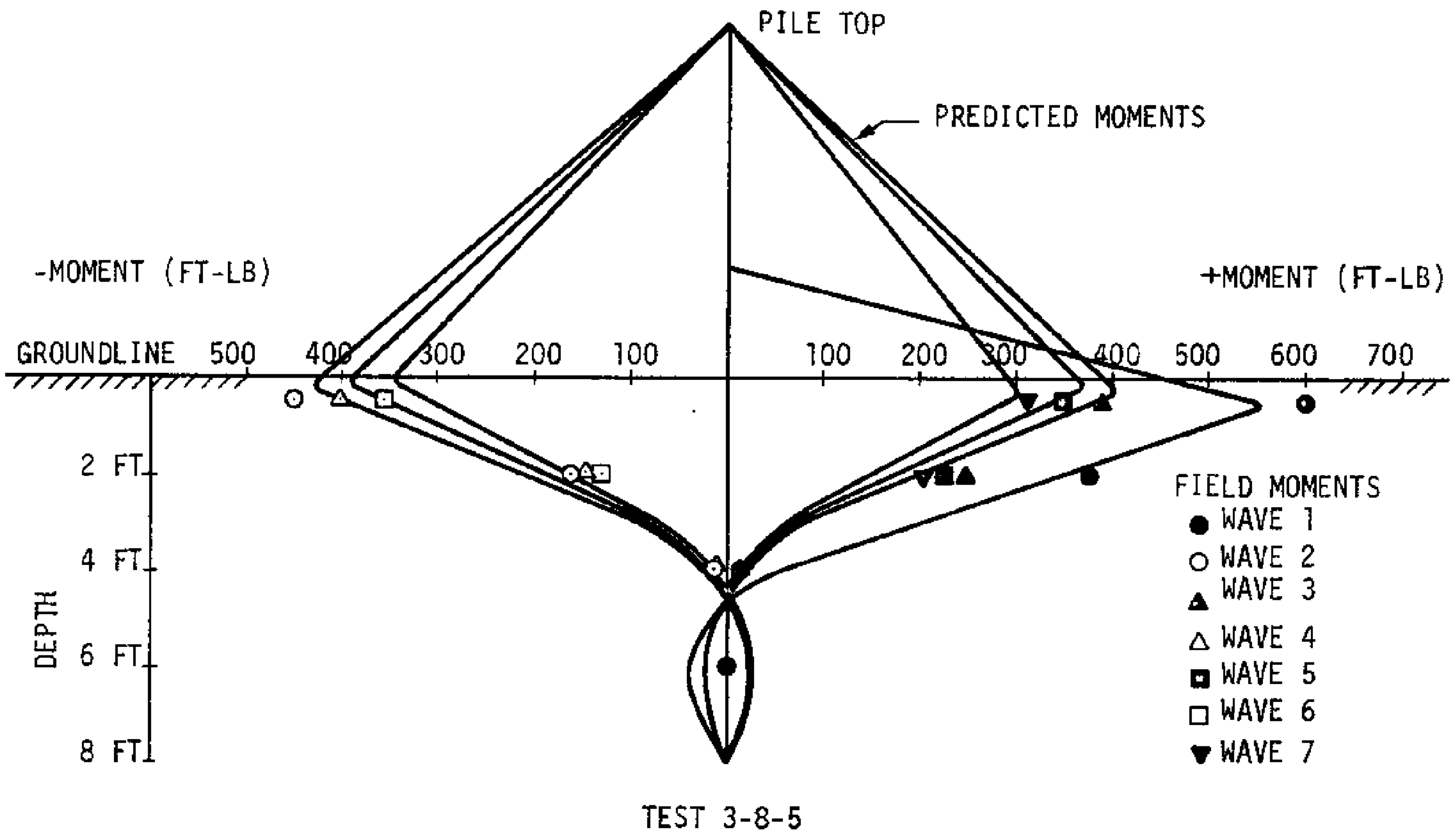


FIG. 18. - FIELD AND PREDICTED MOMENTS VERSUS DEPTH FOR TEST 3-8-5.

pile used and was tested only at an eight foot embedded depth. The first three tests were conducted one day after driving as with the 3 inch pile. The same test procedures were used. The accelerations which were recorded at the groundline and pile top are not presented here but can be found in Appendix IV. Figs. 19 thru 21 show the bending moments with respect to time for both the measured field data and predicted data of Test 1-8-2. Both Gage 3 and Gage 2 failed to oscillate around the original static zero moment line as discussed previously. A gap was developed around the pile while the pile was being driven. Therefore, it was assumed that the soil had zero resistance at the groundline and was accounted for accordingly in the predicted solution. In an attempt to match the shift at Gage 3 from the static zero moment line, it was assumed that the soil represented by the spring at the six inch depth on the side of the direction of pull was permanently deformed due to the static loading whereas the opposite side was not. Consequently, the computer solution also gave a similar type shift. Also, there would be no way to get the shift to be completely on one side of the original static zero moment line as shown in Fig. 21 unless the soil moved by rebounding. Therefore, it is felt that the hypothesis of the soil rebounding on one side when the pile was first loaded is a feasible explanation of the moment shift. The values of the moments at Gage 2 were very small and the deviations from the predicted moments, though

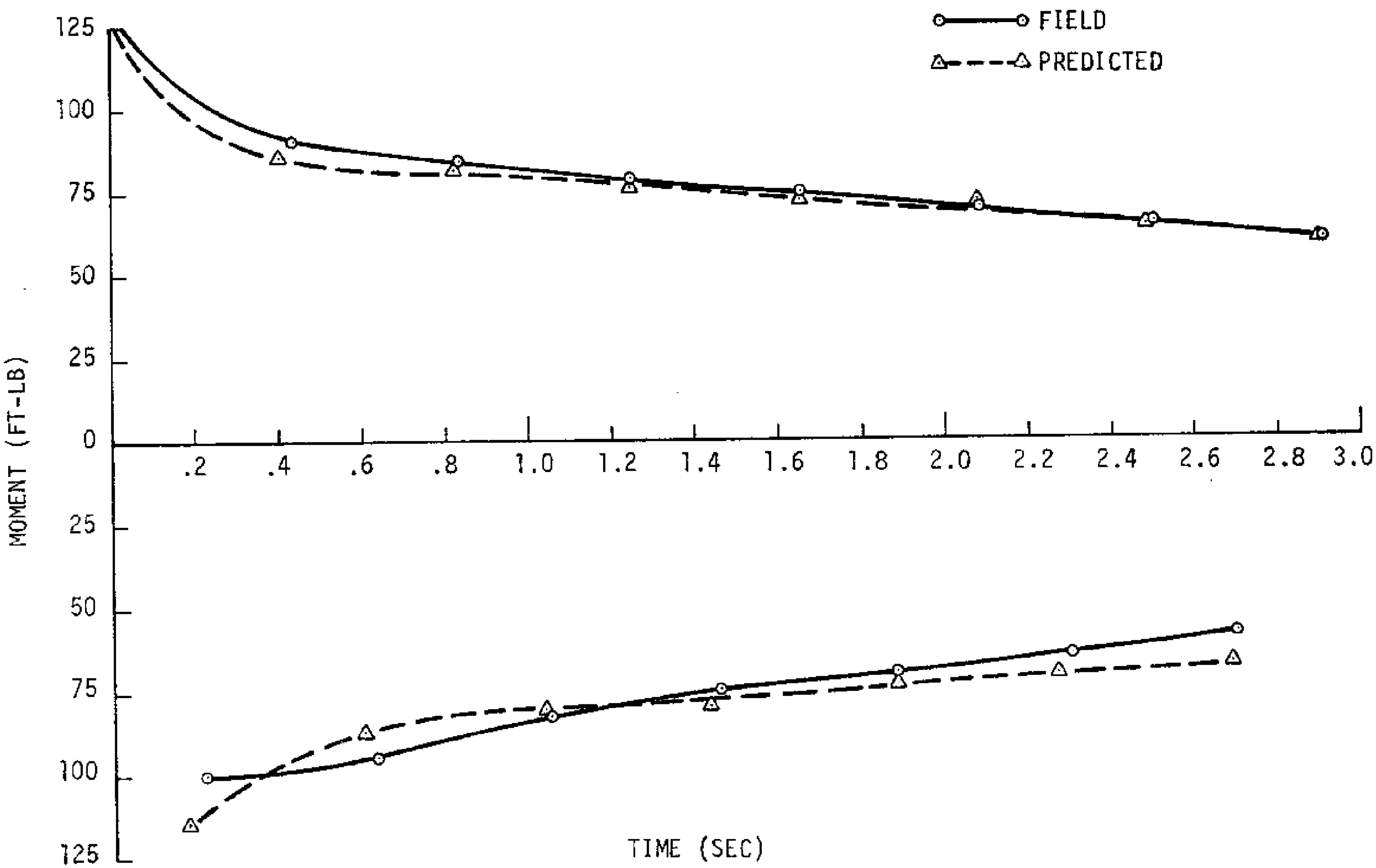


FIG. 19. - BENDING MOMENT VERSUS TIME FOR TEST 1-8-2, z = 0.5 FEET.

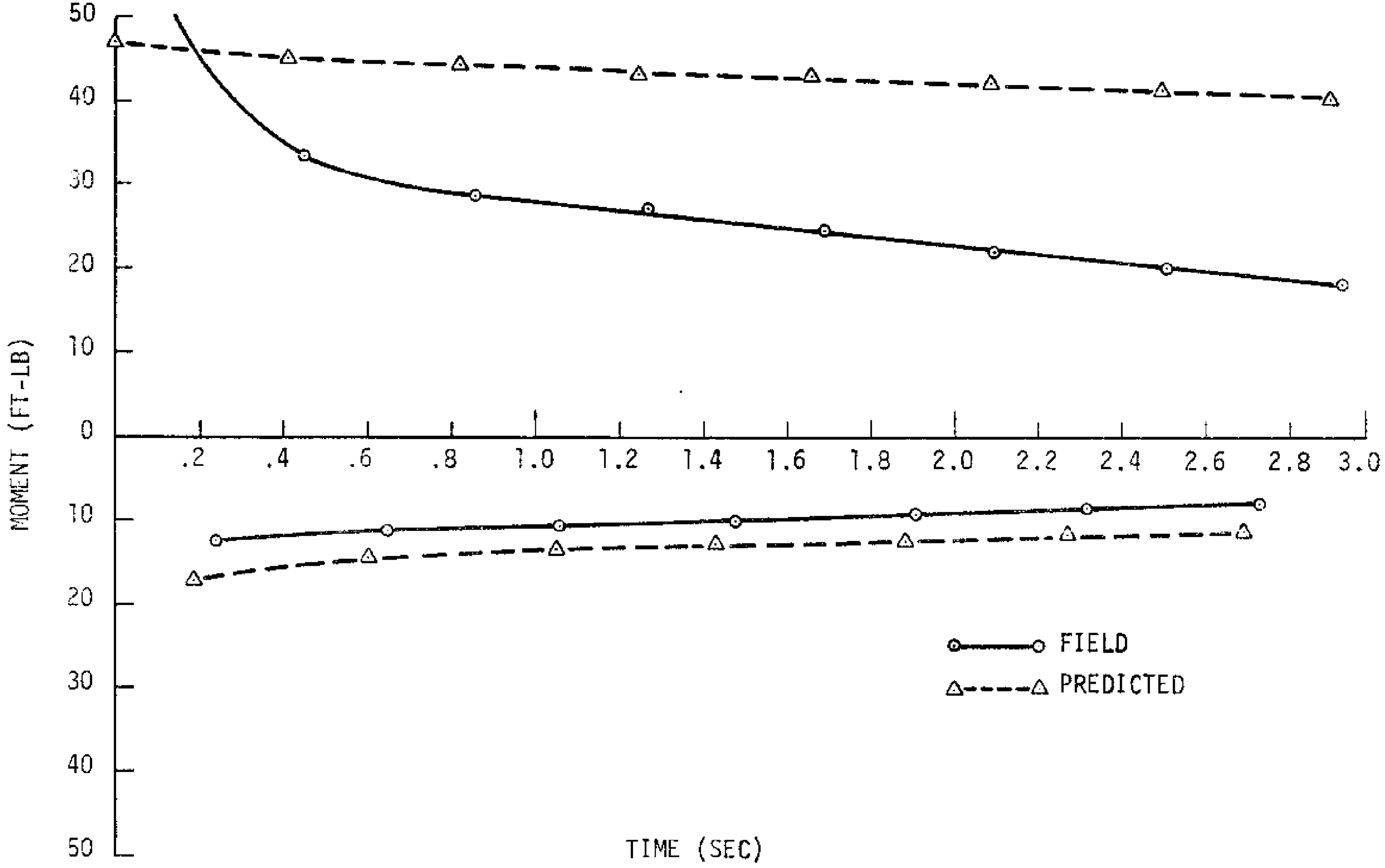


FIG. 20. - BENDING MOMENTS VERSUS TIME FOR TEST 1-8-2, z = 2.0 FEET.

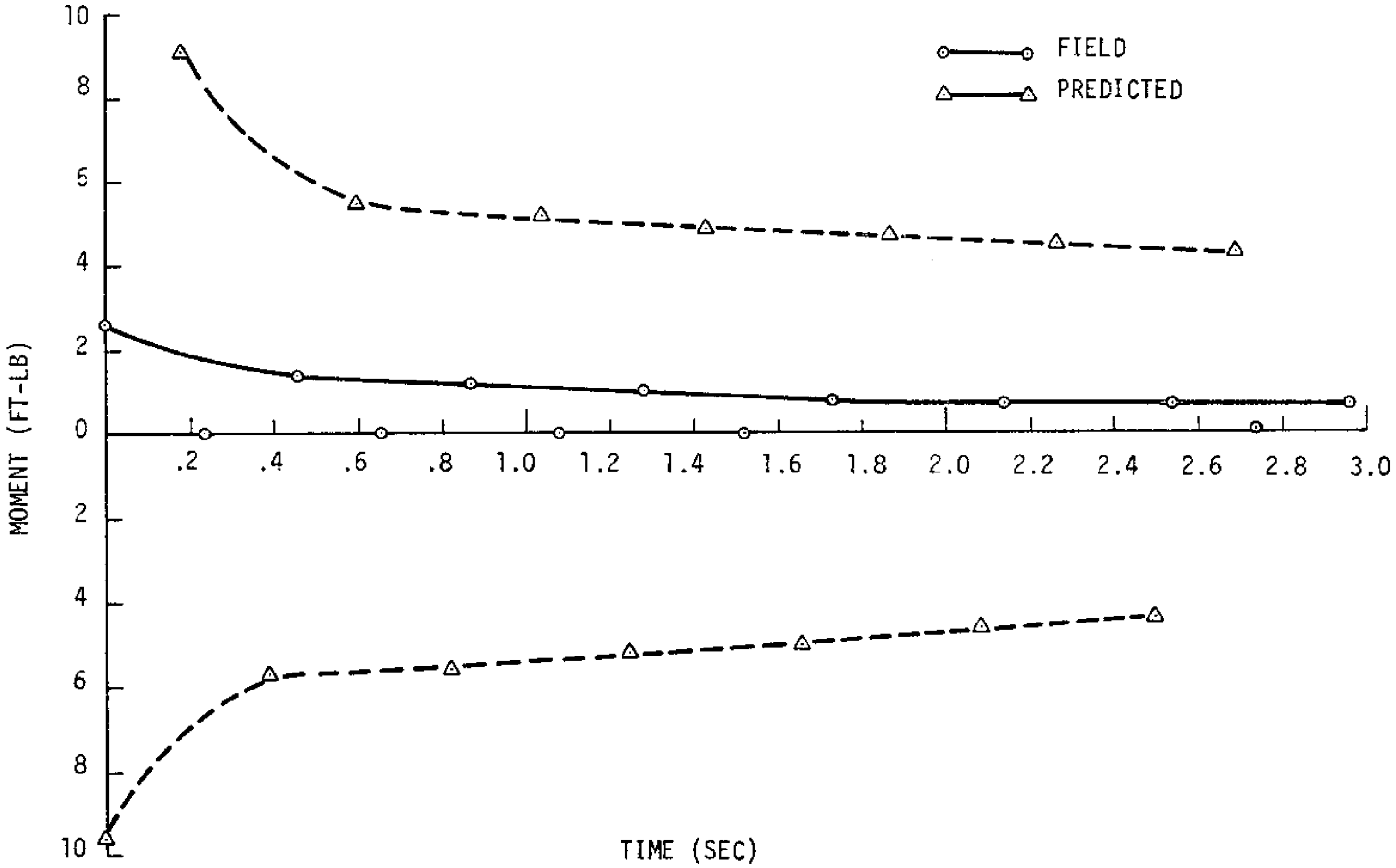


FIG. 21. - BENDING MOMENTS VERSUS TIME FOR TEST 1-8-2, z = 4.0 FEET.

large in percent error, were considered unimportant with respect to magnitude.

The value of J = 2.0 was used for the soil damping constant for the 1.25 inch pile tests and seemed to work adequately. The measured frequency was 2.42 cycles per second whereas the predicted frequency was 2.5 cycles per second, a difference of less than five percent. In terms of percent, agreement at Gage 3 was poor but in terms of magnitude, agreement at Gage 3 was acceptable. At Gage 3, J = 2.0 provided good agreement on one side but was too small for the other side. This substantiates the belief that the soil properties are indeed different on each side of the pile.

After a 14 day period allowed for set up, the 1.25 inch pile was tested two more times. No rain had occurred in these 14 days so any changes would be due to soil set up. No appreciable changes were detected. After a period of 16 more days during which it rained, the pile was tested its final two times. Figs. 22 and 23 show the results of Test 1-8-6. Even though the pile was pulled with a slightly larger force than was used in Test 1-8-2, the moments in Gage 2 were too small to be measured. Also a large shift occurred in Gage 3 but Gage 4 remained about the same as in the previous tests. The frequency increased almost 17 percent to 2.82 cycles per second. The soil evidently expanded upon wetting and gripped the pile tighter. Consequently the frequency increased and the

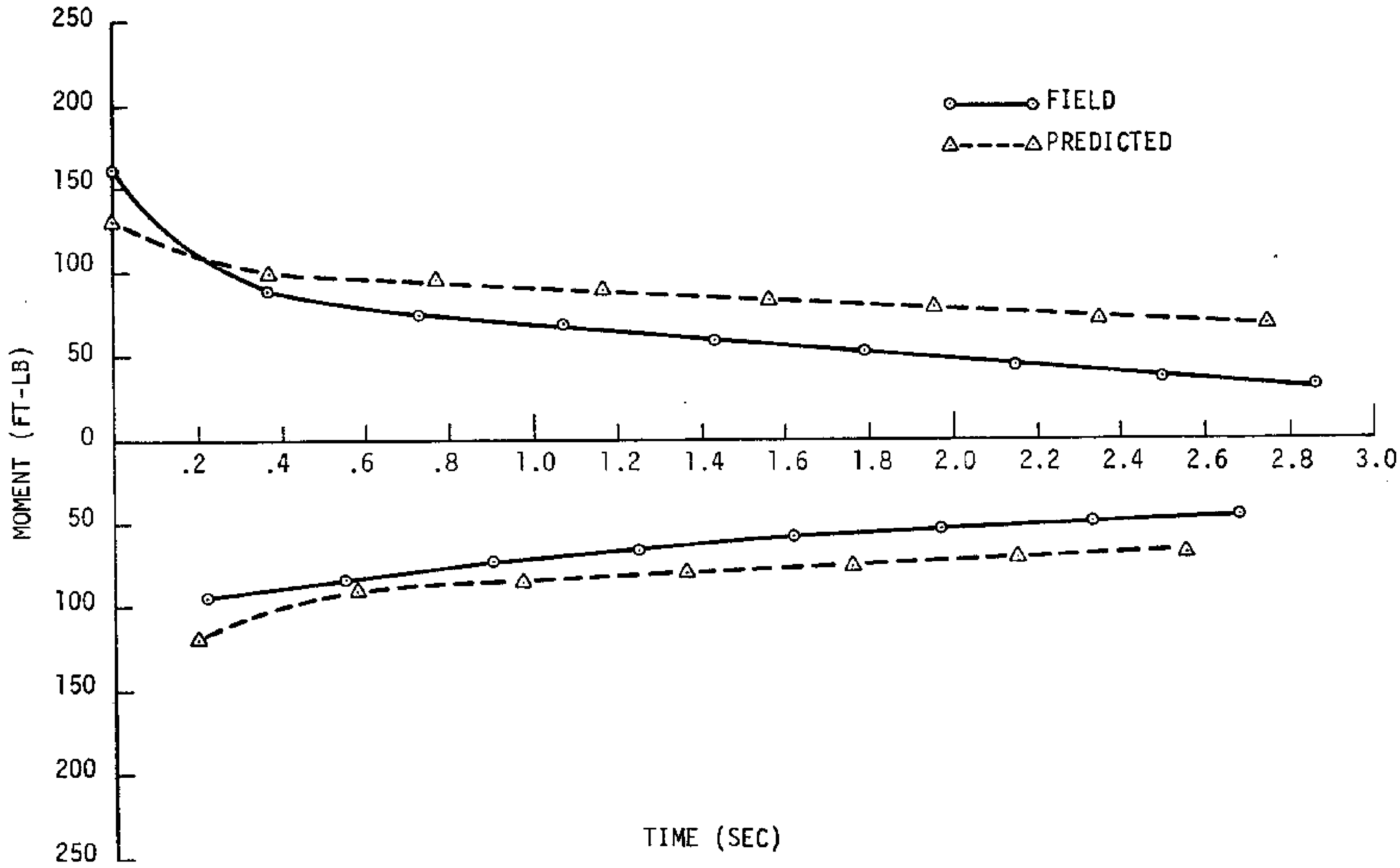


FIG. 22. - BENDING MOMENTS VERSUS TIME FOR TEST 1-8-6, z = 0.5 FEET.

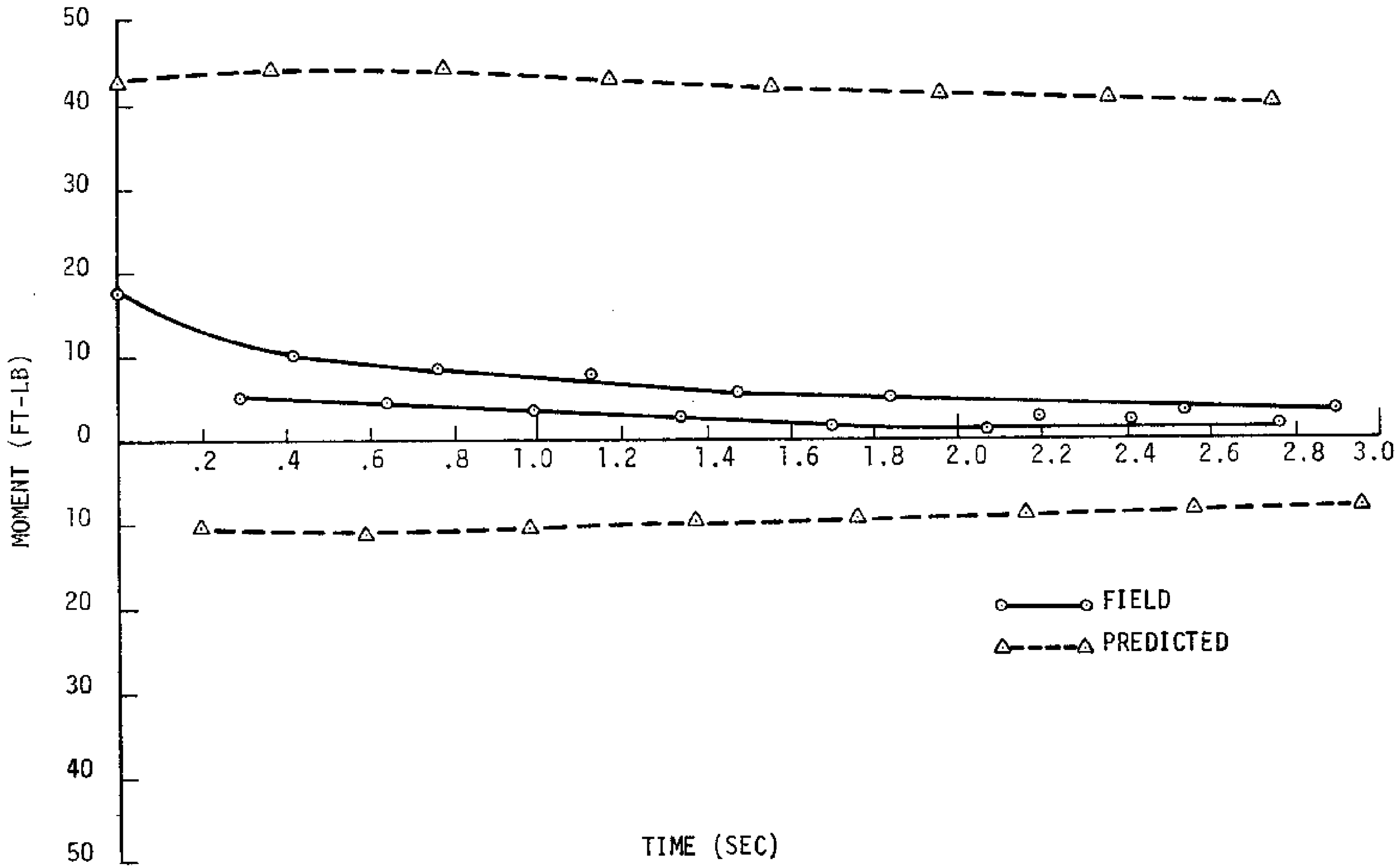


FIG. 23. - BENDING MOMENTS VERSUS TIME FOR TEST 1-8-6, z = 2.0 FEET.

moments at Gage 3 decreased as might be expected in a highly plastic stiff clay which had expanded. The effects due to the swelling clay were less pronounced on the 3 inch pile than on the 1.25 inch pile mainly because the 3 inch pile was much stiffer. Its frequency did show a 4.5 percent increase but the predicted and measured moments showed little variance.

2 Inch Pile Tests. - A third series of tests were made on the 2 inch diameter pile. The pile was tested twice at a six foot depth, twice at an eight foot depth, and three times at a ten foot depth to determine if different embedded lengths affected the response of the pile. Displacements were recorded with the optical tracker on the two six foot depth tests only. Accelerations were recorded but are reported only in Appendix IV. All tests were made within one day of driving, and the same pile was used for all tests.

As occurred in the 1.25 inch pile, a hole was developed at the surface while driving the pile to a six foot depth. Therefore, no soil resistance on either side of the pile at the groundline was assumed in predicting the bending moments. A small gap was also assumed to occur at the six inch depth due to the driving. Results of Test 1 at the six foot depth, Test 2-6-1, are in Figs. 24 thru 27. The moments at Gage 4 agree closely with the predicted moments. Once again Gage 3 did not oscillate symmetrically about the zero moment line. Again the moments at Gage 2 are extremely small and are considered

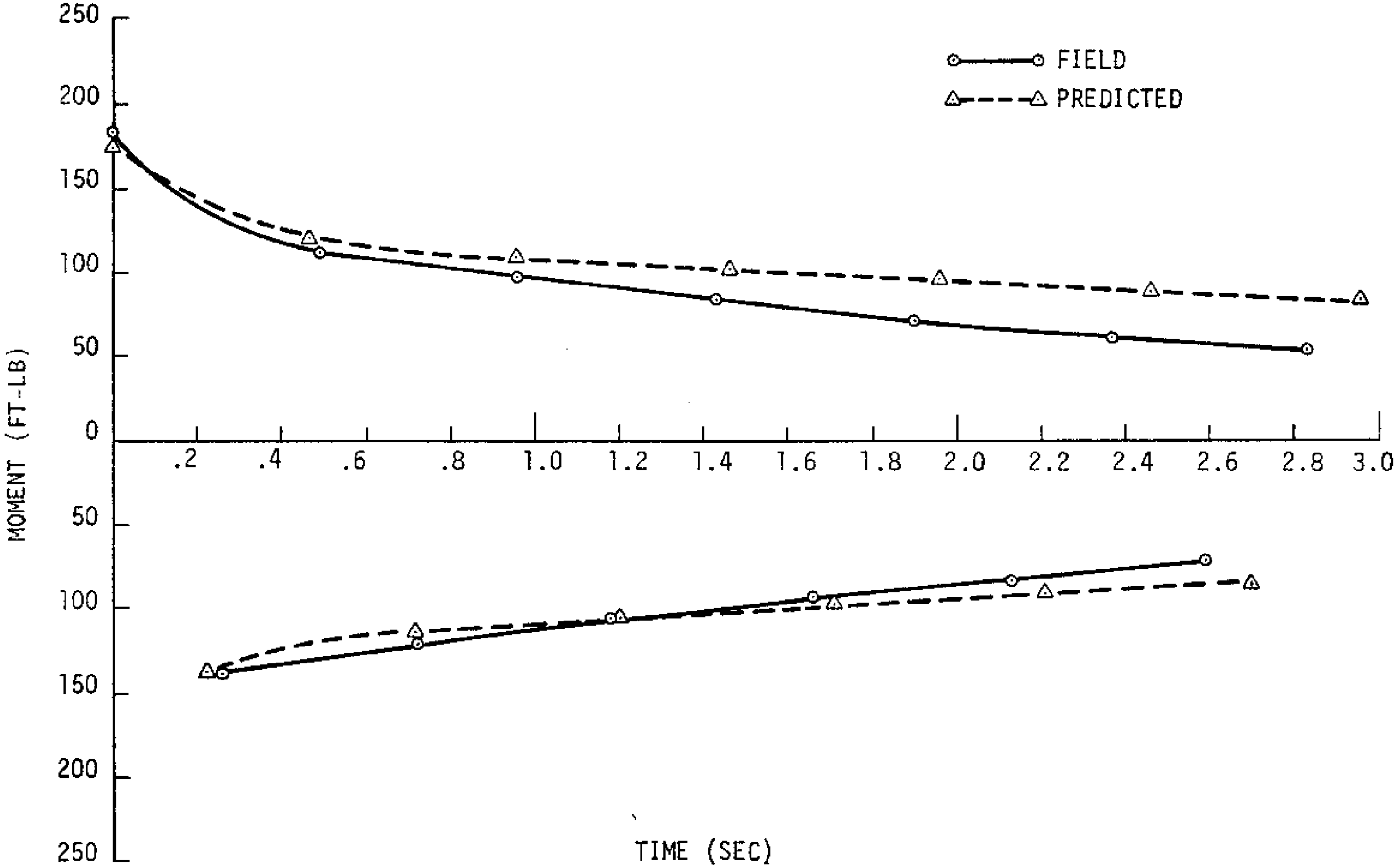


FIG. 24. - BENDING MOMENTS VERSUS TIME FOR TEST 2-6-1, z = 0.5 FEET.

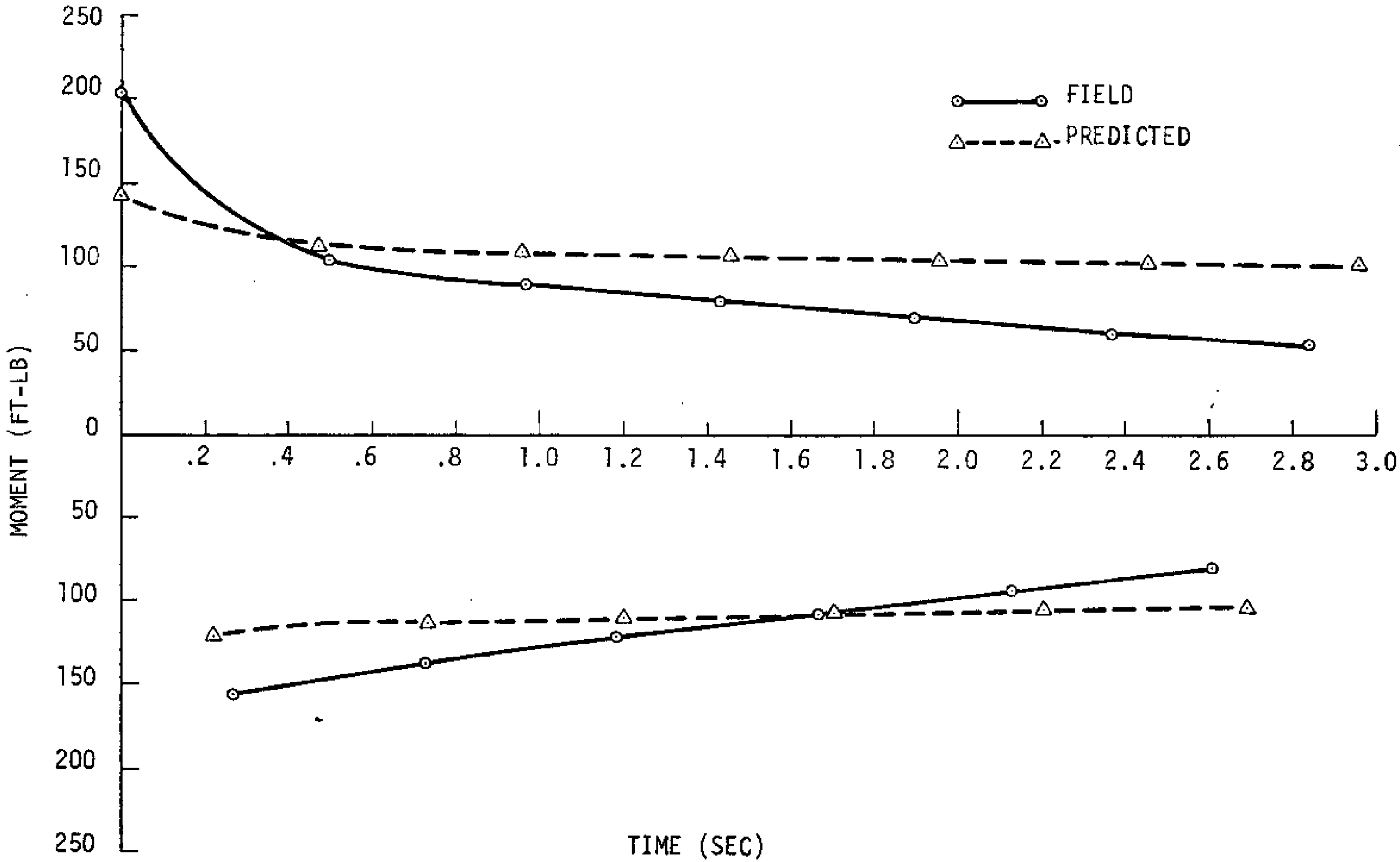


FIG. 25. - BENDING MOMENTS VERSUS TIME FOR TEST 2-6-1, z = 2.0 FEET.

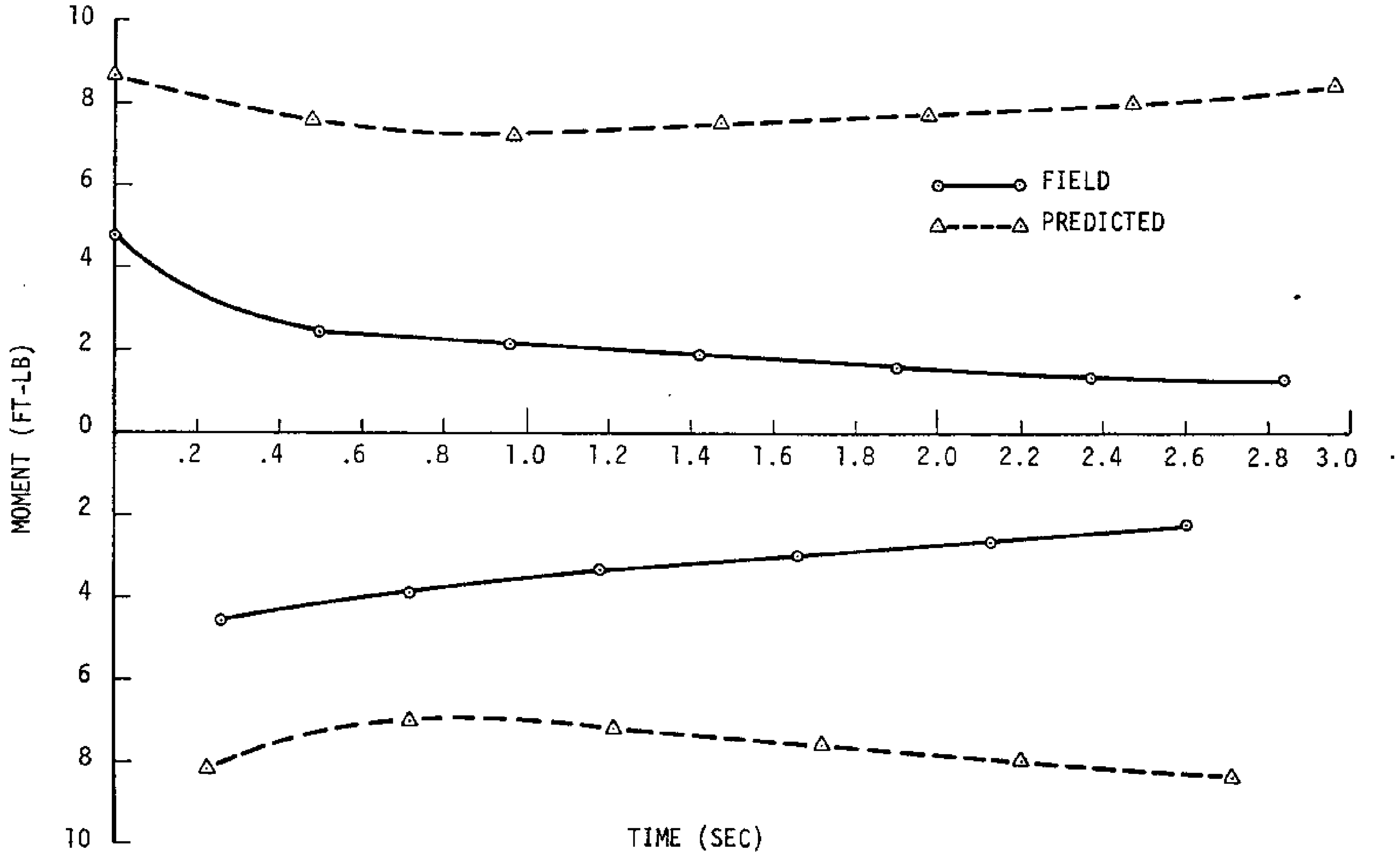


FIG. 26. - BENDING MOMENTS VERSUS TIME FOR TEST 2-6-1, z = 4.0 FEET.

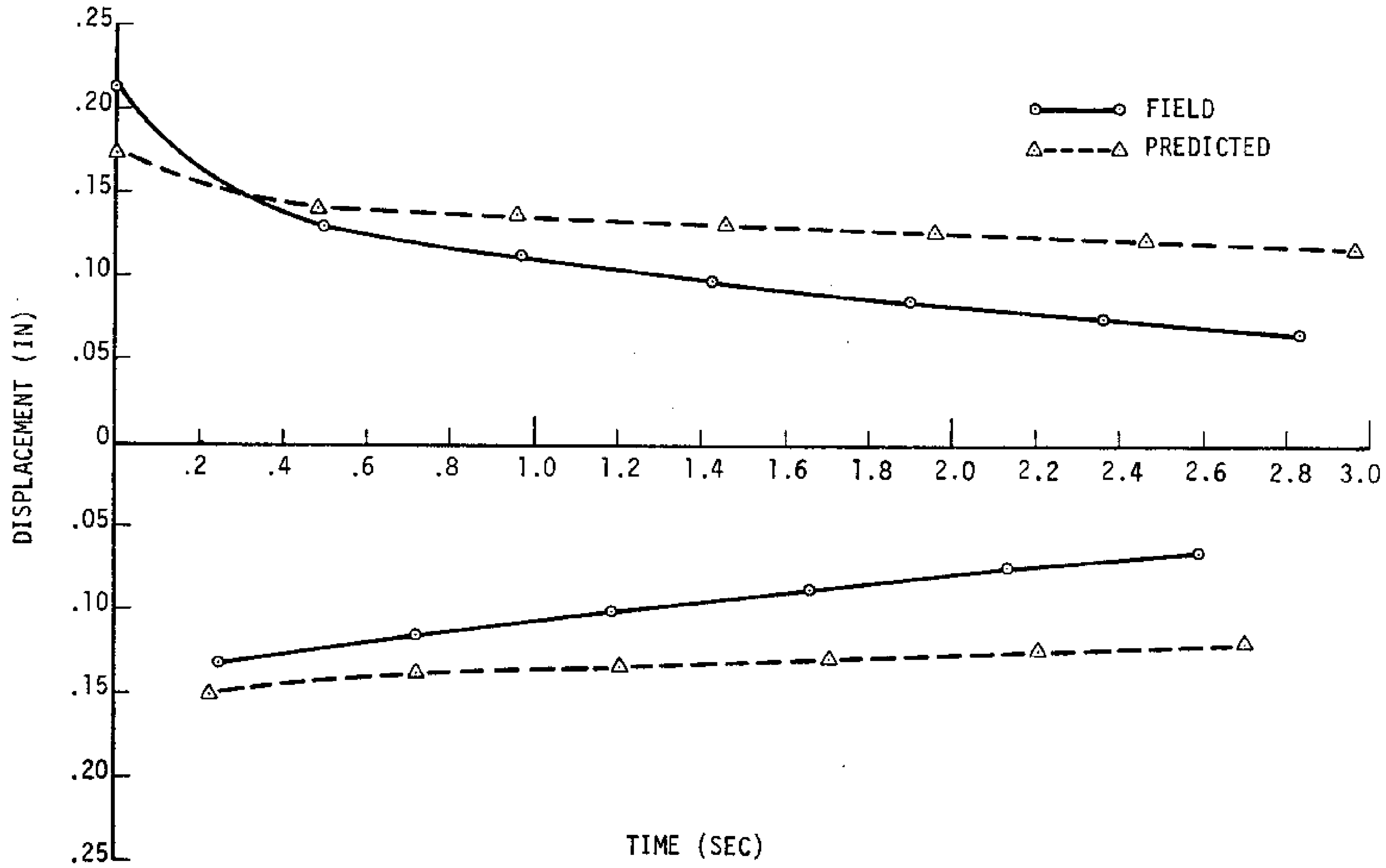


FIG. 27. - DISPLACEMENT VERSUS TIME FOR TEST 2-6-1 AT POINT OF LOAD.

unimportant when compared to the moments at Gages 3 and 4.

Initially the displacements agree within 13 percent but deviate with time. The same soil damping constant, J = 2.0, was used as in the two previous test series. However, it appears that for this pile, which is embedded only six feet, a larger value of damping is needed. The frequency of vibration was measured to be 2.15 cycles per second whereas it was predicted to be 2.0 cycles per second - a deviation of seven percent.

After being tested at a six foot depth, the pile was pulled out. Two feet of pipe were welded to the bottom, and it was then redriven to a depth of eight feet. Two tests were made at this depth. The moments at Gage 4 and Gage 3 of Test 2-8-2 are shown in Figs. 28 and 29. No displacements were recorded and the accelerometer data can be found in Appendix IV. Using the same techniques to get the predicted pile response as was used on the other tests, the moments at Gage 4 agree within 17 percent but the predicted moments at Gage 3 are extremely high. The reason for this is not known. Only by increasing the soil spring constant or soil quake could the predicted moment at Gage 3 be reduced. Gage 2 moments are extremely small and not considered to be critical. A J = 2.0 was again used and is apparently too small, making the predicted moments deviate more and more from those in the field tests as time elapsed. The frequency recorded was 1.98 cycles per second

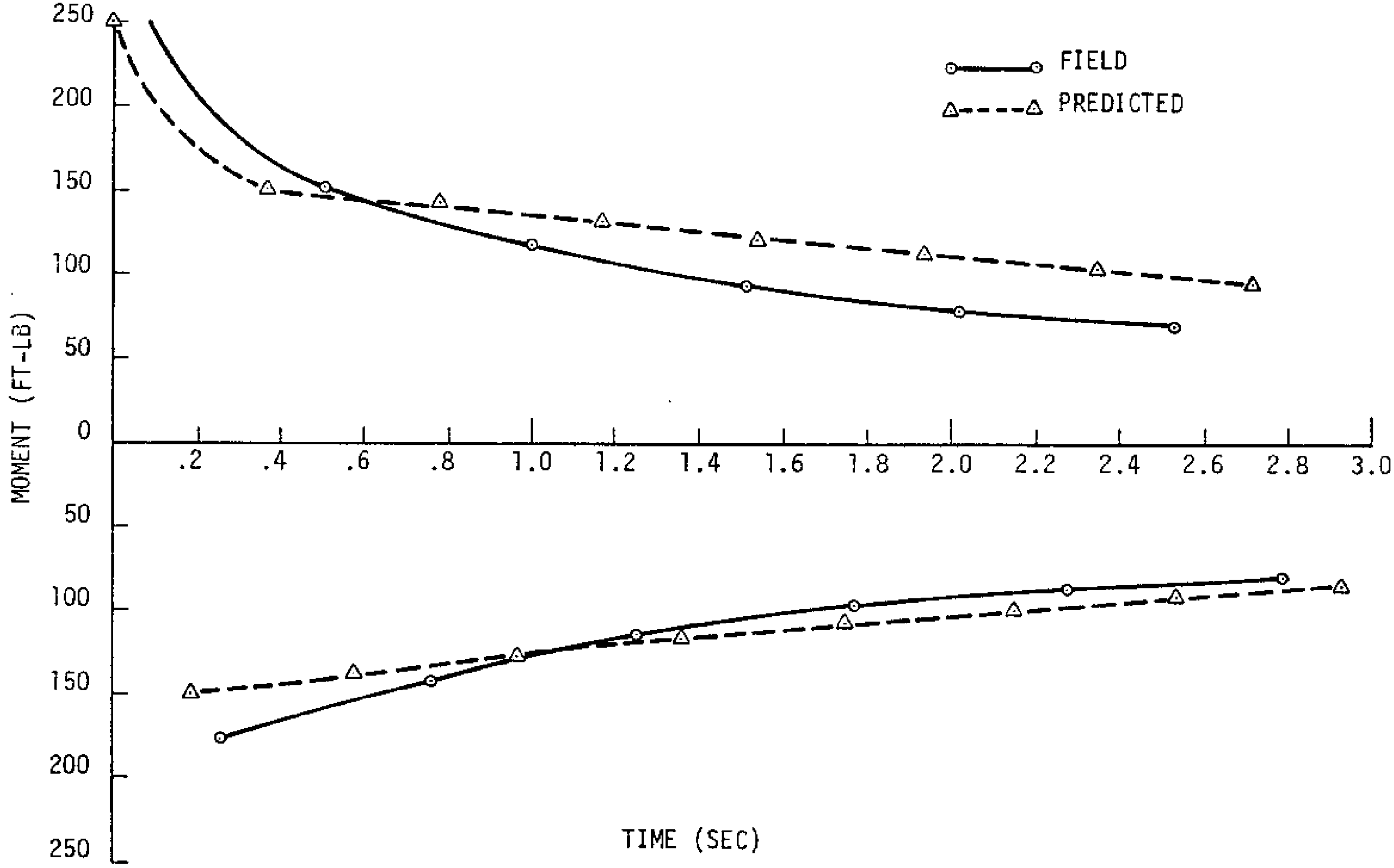


FIG. 28. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-2, z = 0.5 FEET.

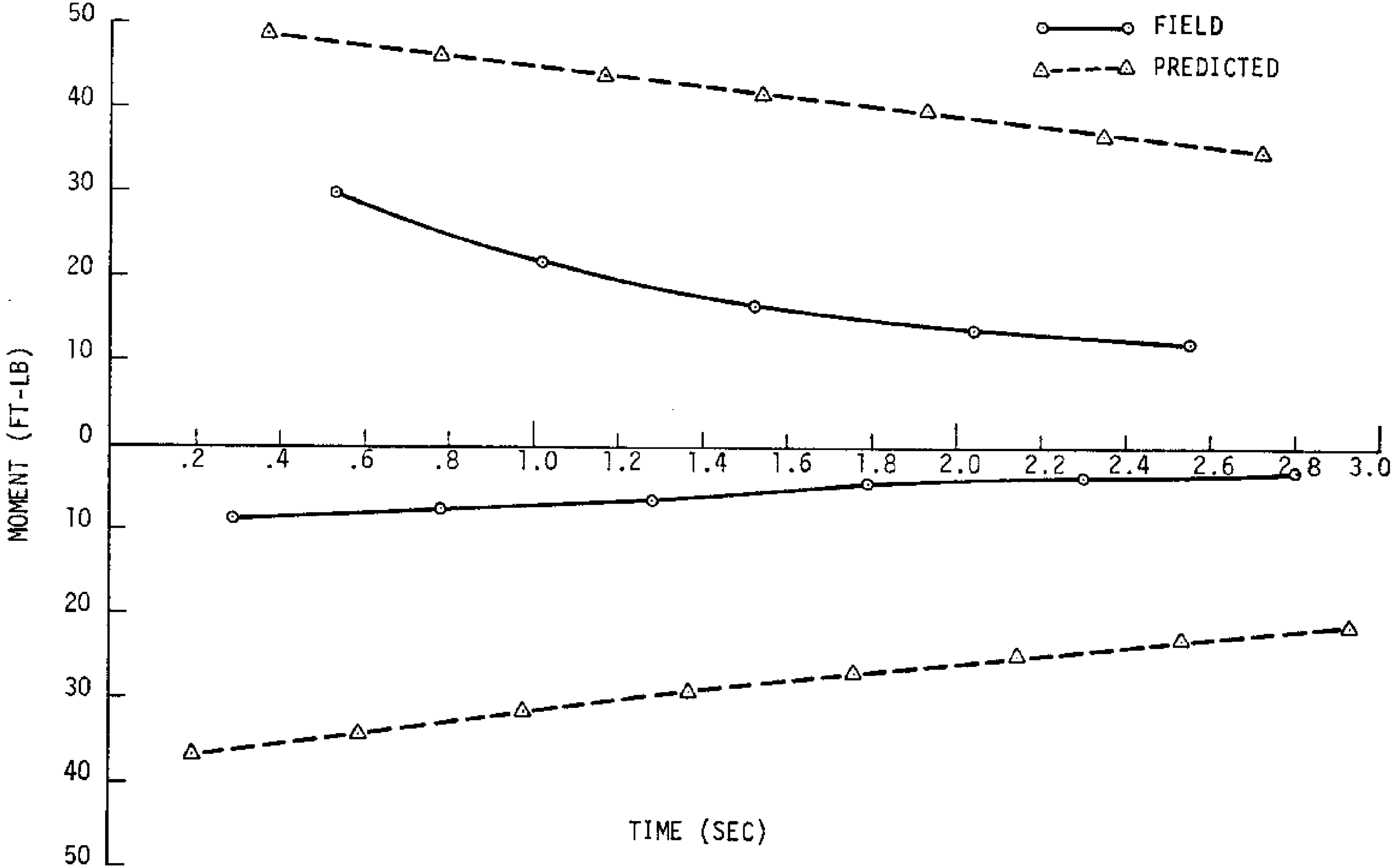


FIG. 29. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-2, z = 2.0 FEET.

whereas the predicted frequency was 2.56 cycles per second, a difference of 22 percent.

A third group of tests in this test series on the 2 inch pile were conducted at an embedded depth of ten feet and consisted of three tests. Again a two foot section was added to the bottom of the pile so that upon redriving to a depth of ten feet, the strain gages would remain at the same depths as in the other tests. The resulting moments at Gages 4 and 3 of Test 2-10-2 are presented in Figs. 30 and 31. Gage 2 data along with the accelerometer data are presented in Appendix IV. The predicted moment values at Gage 4 are within ten percent near the beginning of the test, but the percent error increases with time because of J being too small. The predicted Gage 3 moment values are 18 percent too high and remained so on one side. On the side of the direction of pull, however, much more damping occurred than predicted and the error was much greater. The frequencies deviated by 8.0 percent with the predicted frequency being too high. A gap due to the driving of the pile was assumed to exist around the pile at the groundline.

Five additional tests were made on the 2 inch pile at the eight foot depth. In an attempt to study the effects of frequency, the mass at the pile top was varied three times as discussed previously. For each test, less mass was used than the test before and consequently the frequency of vibration increased. Figs. 32 and 33 present the data from the first of

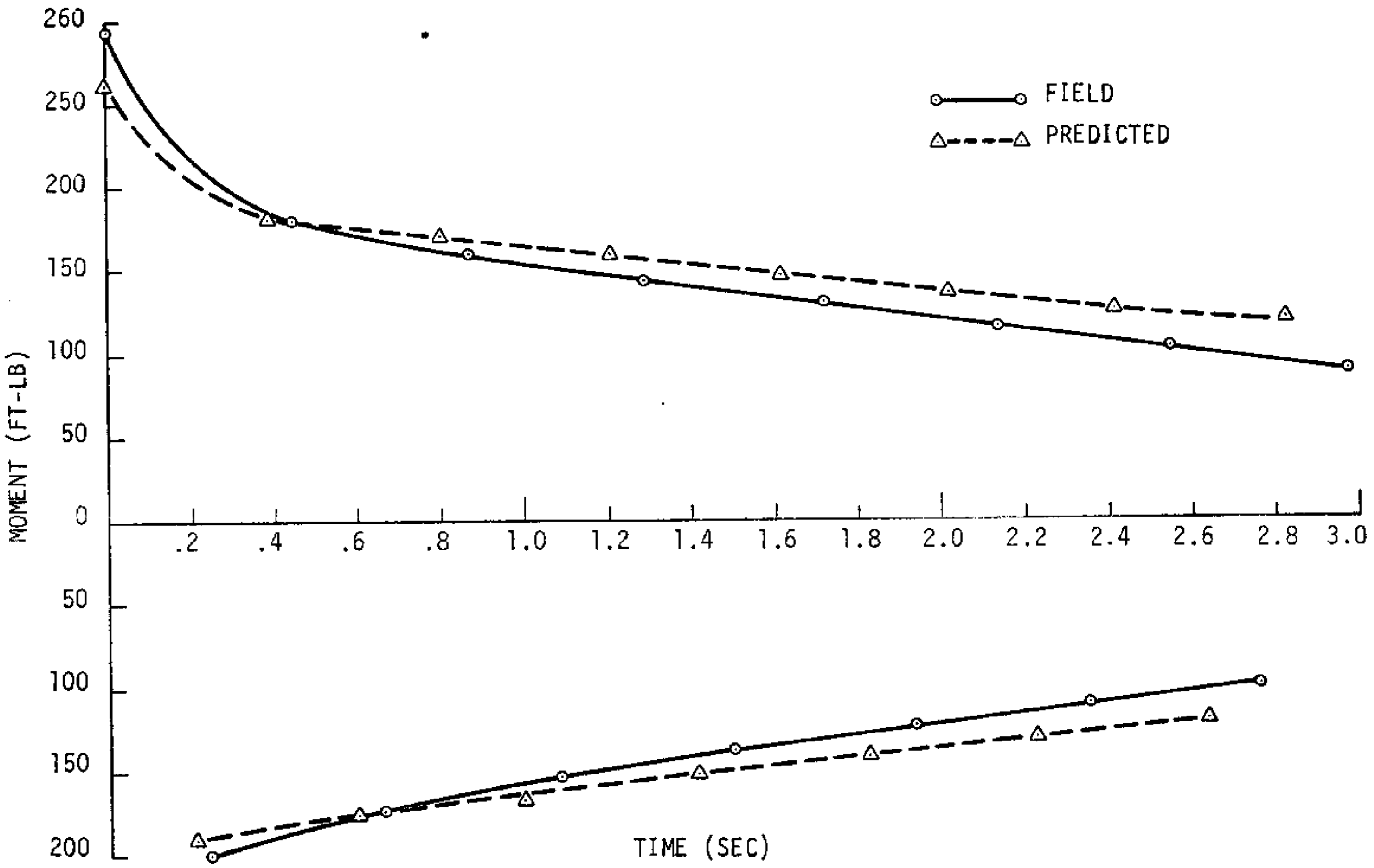


FIG. 30. - BENDING MOMENTS VERSUS TIME FOR TEST 2-10-2, z = 0.5 FEET.

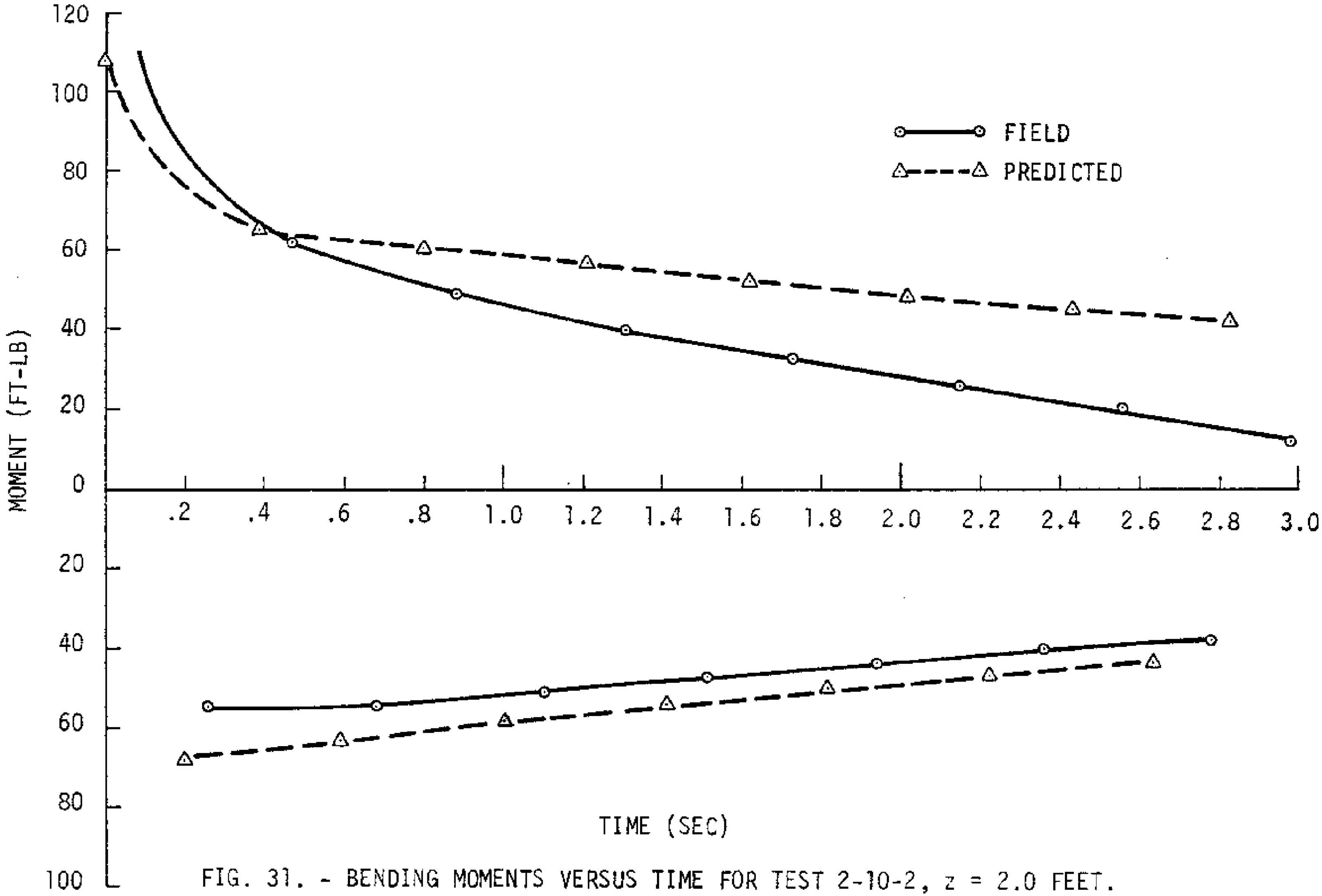


FIG. 31. - BENDING MOMENTS VERSUS TIME FOR TEST 2-10-2, z = 2.0 FEET.

the three higher frequency tests (Test 2-8-3). As can be seen from these two figures, the measured amount of damping increased greatly for this higher frequency. The value of J = 2.0 did not provide sufficient damping in the predicted response so J = 4.0 was tried. This higher J value had little effect on the frequency and made little difference in the predicted amount of damping at the six inch depth. At the two foot depth, the larger J value did increase the predicted amount of damping for two cycles on the side opposite of the direction of initial loading. As was the case in Test 2-8-1, the predicted moments are much too high at Gage 3. The predicted frequency is 4.11 cycles per second as compared to the measured frequency of 3.85 cycles per second.

The final two tests which were conducted on the 2 inch pile were made while it was surrounded by approximately three feet of water. The purpose of these tests was to more closely simulate the field conditions of a pile in the ocean. The measured response and predicted response of Test 2-8-7 are shown in Figs. 34 and 35. Once again the moments at Gage 3 are lower than those predicted which is consistent for all tests made on the 2 inch pile at the eight foot depth. The frequency predicted was 12.7 percent too high being 2.56 cycles per second as compared to the recorded frequency of 2.27 cycles per second.

Due to the water environment, a drag coefficient and a

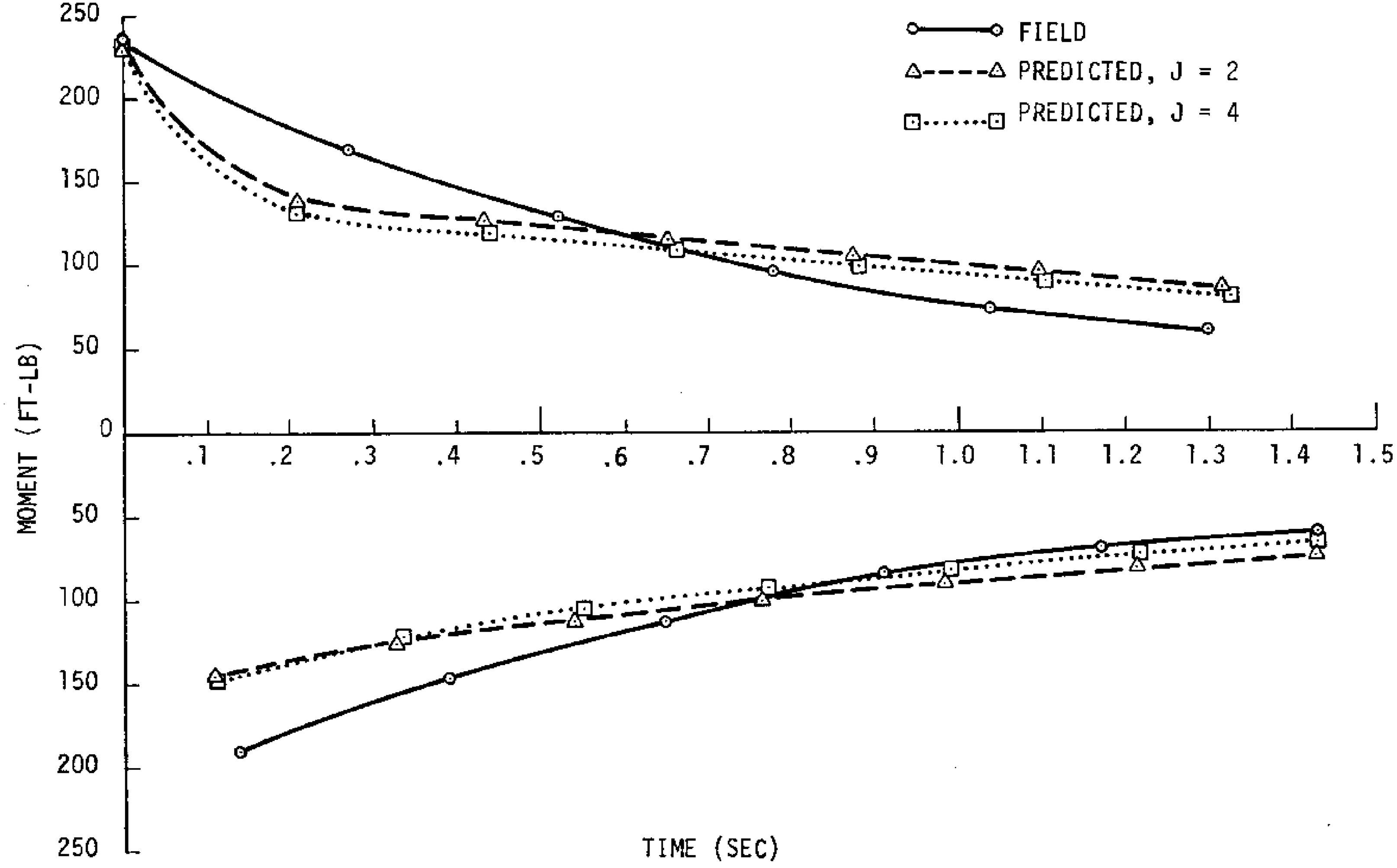


FIG. 32. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-3, z = 0.5 FEET.

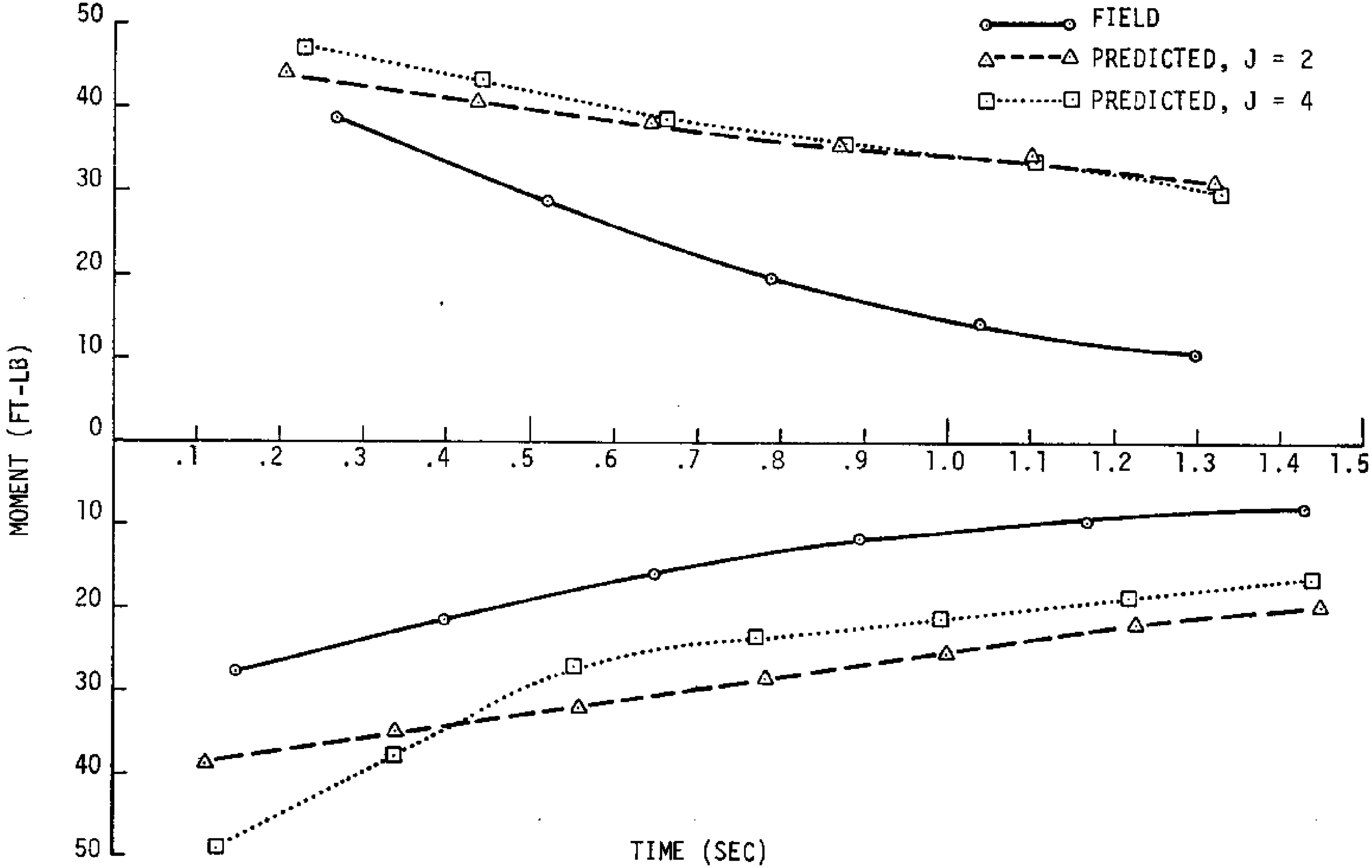


FIG. 33. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-3, z = 2.0 FEET.

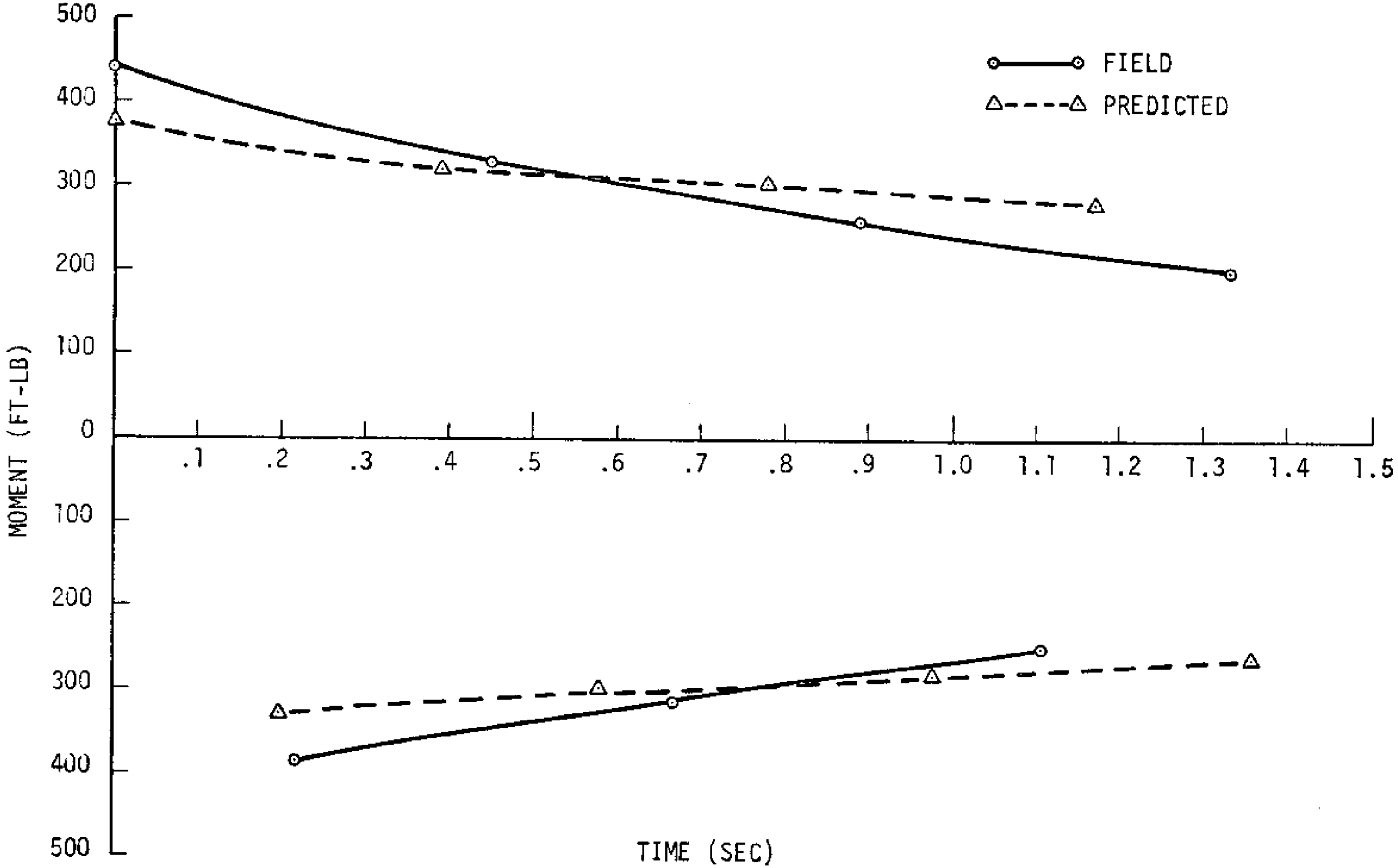


FIG. 34. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-7, z = 0.5 FEET.

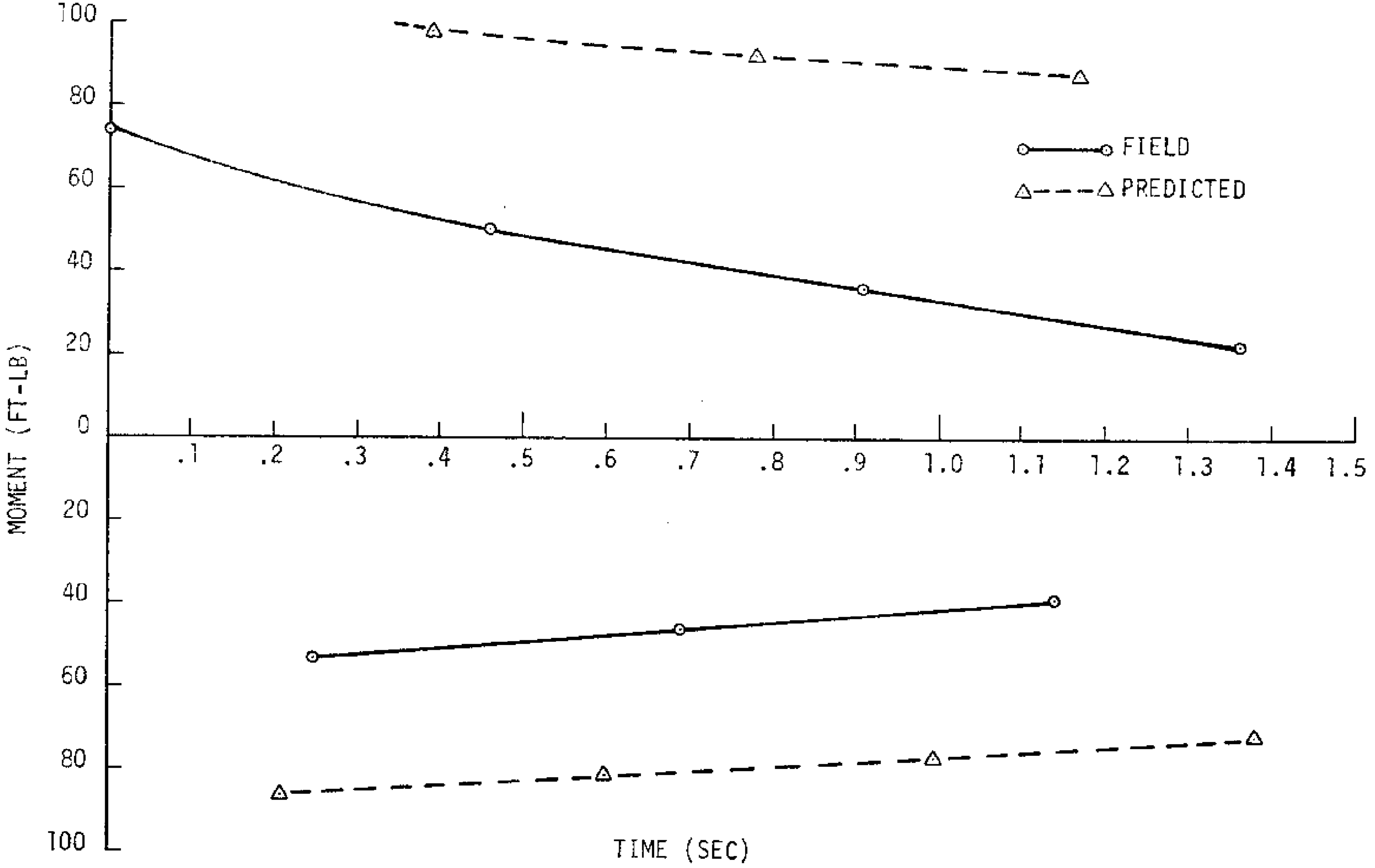


FIG. 35. - BENDING MOMENTS VERSUS TIME FOR TEST 2-8-7, z = 2.0 FEET.

mass coefficient were needed to describe the water-pile interaction. These values are dimensionless and the values used were the average values suggested by Ippen (7). The drag coefficient used was 1.05 and the mass coefficient was 1.4. The water environment increased the amount of damping as expected. However, as in all the tests made on the 2 inch diameter pile, the amount of damping was too low. Therefore, the results of these two tests are considered inconclusive.

SUMMARY OF TEST RESULTS

The problem of predicting the dynamic response of a laterally loaded pile is a very complex one. The complexity of the problem is further increased when the response of a small-scale test pile is being predicted. As is always the case in testing models in soils, scaling is a problem. When full-scale test data are not available, model test data are the next best thing. In this testing program it was found that the effect of a gap between the pile wall and the soil due to the driving of the pile was significant - especially on the two smaller diameter piles. It is felt that the effect of such a gap would not effect the response of a full-scale pile as significantly. For this reason, the field results of the 3 inch pile tests were used in developing the soil parameters for predicting the pile response. These parameters were then applied to the two smaller diameter piles.

Effect of K. - The value of the soil spring, K, and its distribution with depth appears to be the main parameter that controls the magnitude and distribution with depth of the bending moments in laterally loaded piles. Hence, it is probably the most important of the soil parameters to be evaluated. The distribution of k_L with depth and therefore K that is shown in Fig. 6 was used for the predicted response of all the tests.

However, the exact distribution of k_L below a four foot depth is not critical. For example, a uniform k_L was used in the analysis of Test 3-8-2. As can be seen in Figs. 36 and 37, there was little or no change in the predicted response when compared to the predicted response using the k_L distribution in Fig. 6. This also verified that the various lengths of embedment of the 2 inch pile had little or no effect since all were embedded more than six feet. For these tests, especially for the two smaller diameter piles, the spring located at the six inch depth proved to be the most critical and was very sensitive to change.

The values of K also influence the frequency. A less stiff soil is modeled by lowering the K values which then causes the first inflection point of the pile to be located deeper in the ground. Therefore, the point of maximum moment is shifted deeper in the ground and the frequency is reduced because of having a greater length to vibrate.

As seen in Fig. 1, upon unloading, the soil was assumed to rebound at the same slope K, as when it was loaded. It is felt that the solution by Ross should be modified to handle the amount of rebound as a separate parameter. As a separate parameter, the rebound could be varied easily; and its effects on the pile response studied.

As presently accounted for in the Ross solution, the values of K remain constant with respect to time for a given depth.

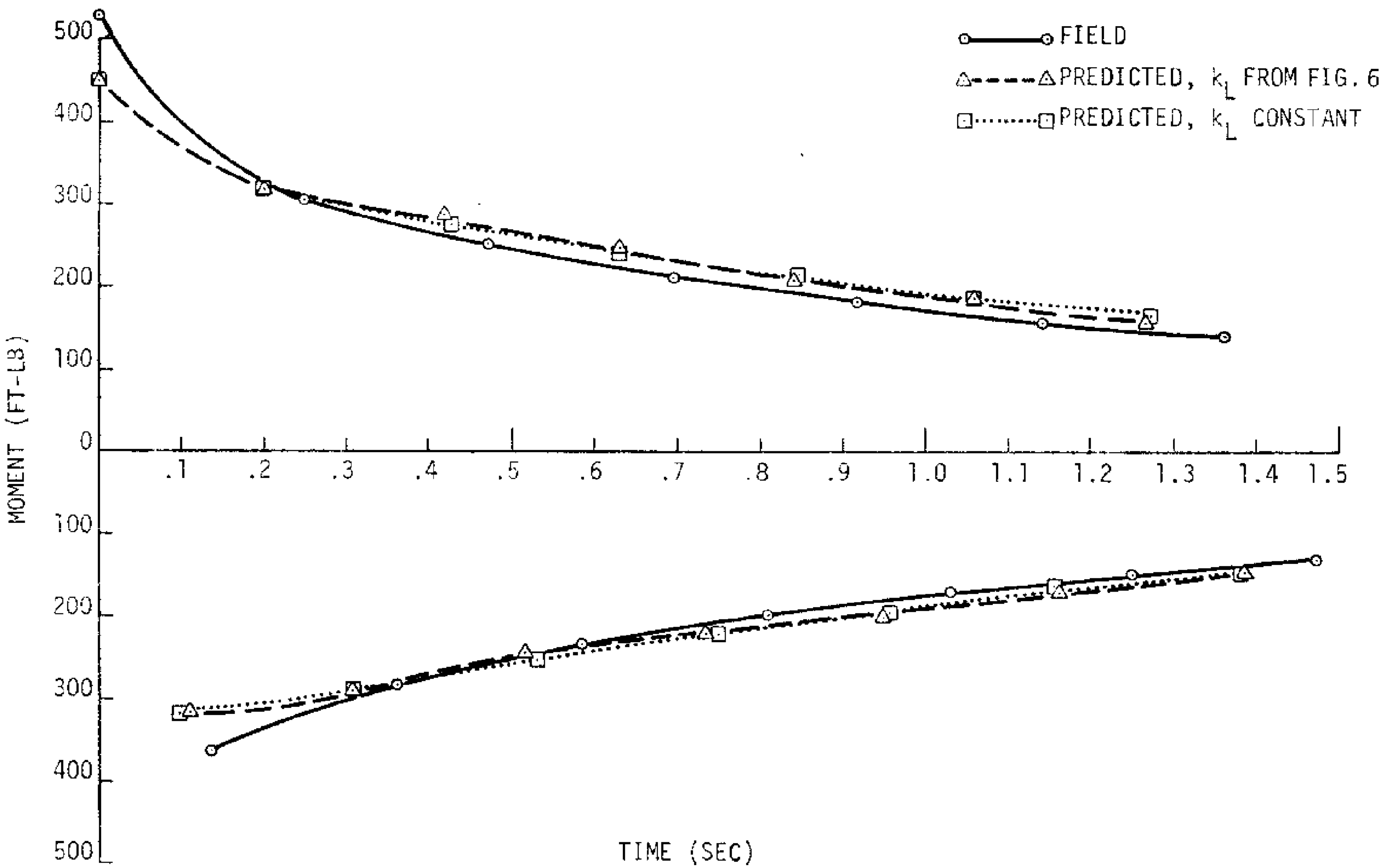


FIG. 36. - BENDING MOMENTS VERSUS TIME FOR TEST 3-8-2, z = 0.5 FEET.

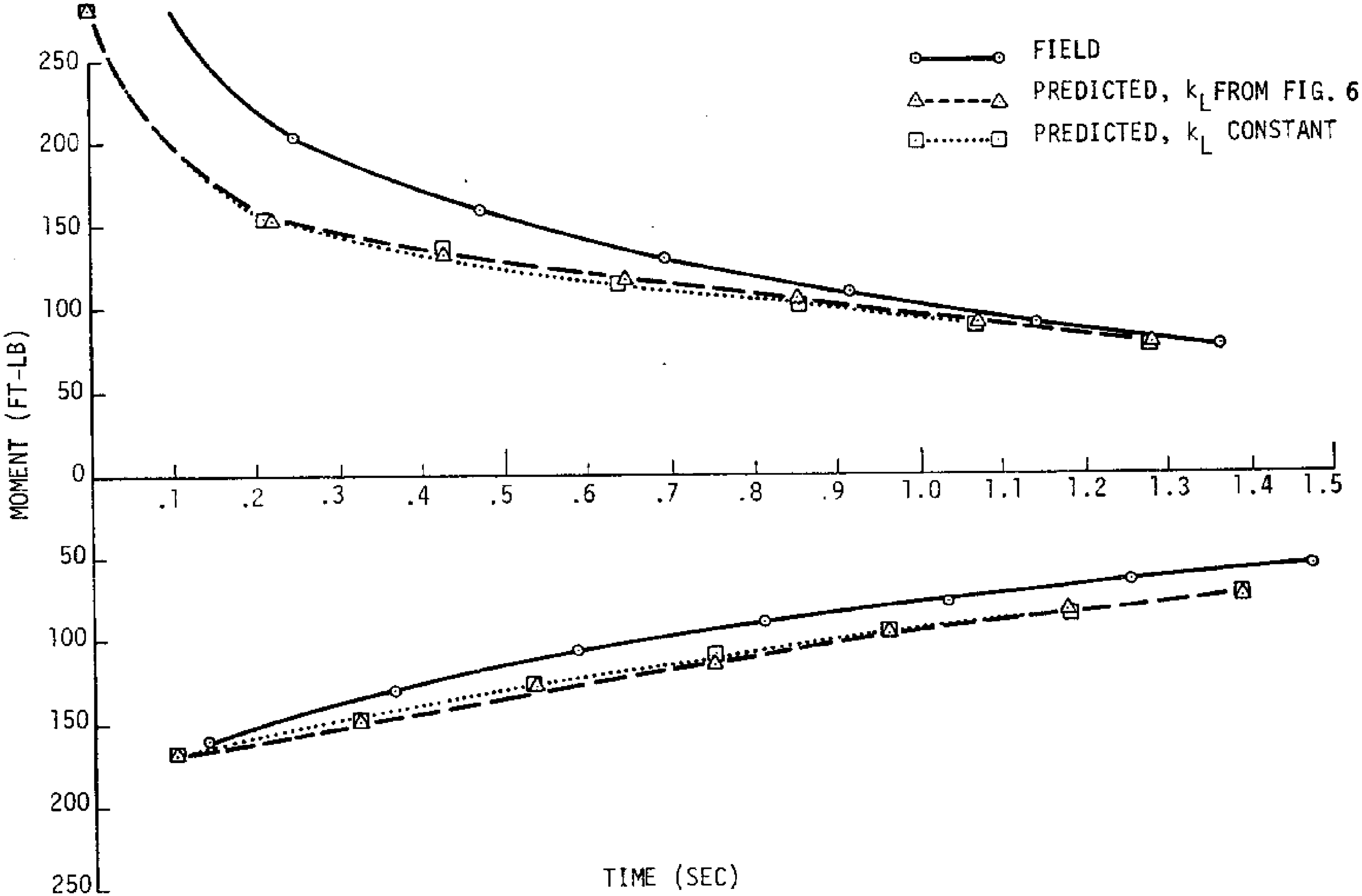


FIG. 37. - BENDING MOMENTS VERSUS TIME FOR TEST 3-8-2, z = 2.0 FEET.

However, due to cyclic loading, it is felt that the soil is remolded and that K should become smaller with time or with the number of loading cycles. Presently there is no way to account for such a change in K in the Ross solution and any such modification would probably be quite complex.

Effect of Q. - The soil quake, Q, as well as the soil spring, K, influences the magnitude of the moments in the piles. Q is related to the pile diameter as indicated in Equation 7 and decreases as the pile diameter decreases. The smaller Q values in turn cause smaller ultimate loads for the clay as indicated in Fig. 7. Therefore it is Q and K together that determine the ultimate load and displacment the soil can undergo before failing. For example, a K = 100 lb. per in. and a Q = 0.10 in. would allow an ultimate load of 10 lb. By reducing K to 50 lb. per in. and increasing Q to 0.2 in., the ultimate soil load remains 10 lb. However, in this second case the soil is less stiff as indicated by the lower K and greater displacements occur with a reduced frequency of vibration.

Effect of J and N. - From all indications, the values of J and N are closely related and are a complex function of velocity, pile diameter, pile length, frequency of vibration, as well as other variables based on the soil and pile properties. It is beyond the scope of this paper to study this problem in the detail necessary but additional work in this area, perhaps in the form of a parameter study, could be quite revealing.

Perhaps the modification mentioned previously of allowing K to be reduced due to load cycling would aid in the study of finding J. By reducing K with time, the rate of damping would also be increased. As predicted in this study, the rate of damping after the first cycle always seems to be linear whereas the measured field results are nonlinear. By increasing N so that the velocity of the pile has a greater influence on the rate of damping, such a nonlinear rate of damping could possibly be predicted.

From these tests, several interesting observations were made regarding the damping of the soil. Ross (19) indicates that the effect of the soil damping is very small for a typical offshore pile. This may be true in the case of wave loadings on offshore piles where frequencies are very slow. However, in this study with higher frequencies, a value of J = 2.0 was necessary for the predicted pile response to match the measured response on the 3 inch pile tests. For the 1.25 inch pile tests a value of J = 2.0 generally produced good agreement between the predicted and measured pile response. However for the 2 inch pile, the damping predicted with J = 2.0 was in all cases too low. In the tests on both the 1.25 inch piles and 2 inch piles the amount of damping was different on each side of the pile. The piles damped faster on the side in the direction of pullover.

From the test results of Test 2-8-3 which was tested at a higher frequency, it is evident that the rate of damping is dependent on the frequency of vibration. Since a value of J = 2.0 was used in the prediction of the response and was found to be much too low, a value of J = 4.0 was tried. The effect of raising J to 4.0 compared to the prediction using J = 2.0 is shown in Figs. 32 and 33. Such a large increase in J gives a relatively small change in the rate of damping. Also, such a strong damping value produces high accelerations and velocities in the computer solution and causes permanent deformation to occur at the pile tip. Therefore, there seems to be a definite upper limit of J that can be used in the Ross solution if N = 0.18. Based on this finding and the fact that the rate of damping is greatly increased as the frequency is increased, the effect that N has on J should be investigated in more detail.

Effect of Pile Diameter and Embedded Length. - Tests were made on piles of three different diameters to see what effects changing the diameter would have on the pile response. However, after testing, it was felt that the effect of the pile diameter alone could not be determined as the piles were of different stiffnesses as well as diameters. It would have been much more desirable to have three piles having different diameters but the same moment of inertia and stiffness. If this had been done and all other variables held constant, the only difference in the pile response would have been due to the different diameters. It was also felt that for these tests, the piles were much too stiff for their corresponding diameters. The frequency of vibration decreased as the pile stiffness and pile diameter decreased.

Tests were made on the 2 inch pile at three different embedded depths to investigate the effect of the embedded length of the piles. No appreciable differences could be determined. All piles vibrated near the same frequency indicating that the soil below the six foot depth had little influence on the response of the pile. The magnitude of the bending moments was unchanged when the embedded length was increased indicating that the extra length had no effect on the pile response. The moments in Gage 3 of Test 2-6-1 were higher than those in Tests 2-8-2 and 2-10-2. This was due to a larger gap being formed while driving the six foot depth pile than occurred at the other depths and not due to the different embedded lengths.

CONCLUSIONS

Conclusions based on the correlation of the dynamic response measured from the model pile tests and the response predicted using the solution of Ross (19) are as follows:

1. The modified Kelvin-Voight rheological model used by Ross (19) to describe the nonlinear soil load-displacement relationship may be used to predict the dynamic response of a laterally loaded pile.
2. Laboratory triaxial tests can be used to determine the soil modulus. The distribution of the soil modulus with depth can be determined by testing soil samples from different depths. For the model piles in the relatively homogeneous stiff clay used in this study, a uniform distribution with depth of k_L is a satisfactory approximation. The k_L values can then be used to calculate the soil spring, K, values.
3. The soil quake, Q, is a function of the pile diameter. Based on the results from these model tests, Q can be approximated as one percent of the pile diameter.
4. The K and Q parameters govern the magnitude and distribution of the bending moments with depth. From this study it was found that the shallow depths were

very critical in predicting the pile response. For these model piles, the displacements were very small below a depth of two feet indicating that at some depth between two and four feet the soil had little influence on the response of the pile. Therefore the values of K and Q below this critical depth are unimportant in predicting the dynamic pile response of a laterally loaded pile.

5. For the tests on the 3 inch pile in this study, a value of J = 2.0 with N = 0.18 gives satisfactory agreement between the predicted and measured amount of damping.
6. The amount of damping increases as the frequency increases and is significant at the frequencies encountered in these tests. The frequency of vibration of the pile may be increased by increasing the pile stiffness, by increasing K, by increasing J, or by decreasing the amount of mass at the pile top.

In summary, the work accomplished by this research provides possible methods by which the necessary parameters for use in the Ross solution for the dynamic response of a laterally loaded pile can be determined. These methods are based on model tests, however, and may need to be modified before being applied to full-scale tests. The next logical step would be to conduct a similar study using full-scale pile test data.

RECOMMENDATIONS

The following areas are recommended for further research:

1. Tests similar to those conducted in this research need to be made on full-scale piles. The conclusions from this study apply only to these model tests in a stiff clay and should be applied to full-scale piles only after more study on the problem has been completed.
2. If additional model tests are conducted, it is recommended that more strain gage bridges be used and closely spaced in the critical shallow depths of the embedded length. Piles of different diameters but the same moment of inertia should be tested in order to more clearly define the effects of the pile diameter on the dynamic response of the pile. All piles should be loaded with the same force at the pile top and should be tested in steady state motion as well as in free vibration. For an instantaneous release in the free vibration tests, the pile should be pulled by a single strand wire and cut.
3. An extensive parameter study needs to be made using the Ross analytical solution to more fully understand the relationship between the soil damping factor J and the exponent N and their dependence upon

the frequency of vibration and velocity of the pile. A parameter study concerning K and Q and a better understanding of their relationship could also prove to be useful. Determining exactly how these parameters are interrelated should aid significantly in determining methods by which they could be evaluated.

4. The analytical solution of Ross should be modified so that the rebound and permanent set due to failure of the soil can be varied. It is also desirable to be able to use different soil properties on each side of the pile and to change the soil properties with time or cycles of loading.

5. Dynamic model field tests should be conducted in sand to see if the modified Kelvin-Voight is a valid model for cohesionless soils. Studies similar to the ones made in this research should be made on the sand parameters to more fully understand the soil load-displacement relationship under dynamic loading conditions.

APPENDIX I - REFERENCES

1. Broms, B. B., "Lateral Resistance of Piles in Cohesive Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, SM2, March, 1964, pp 27-63.

2. Coyle, H. M., and Gibson, G. C., "Empirical Damping Constants for Sands and Clays," Journal of the Soil Mechanics and and Foundations Division, ASCE, Vol. 96, SM3, May, 1970, pp 946-965.

3. Davisson, M. T., "Behavior of Flexible Vertical Piles Subjected to Moment, Shear and Axial Load," PhD Dissertation, University of Illinois, Urbana, Illinois, 1960.

4. Davisson, M. T., and Gill, H. L., "Laterally Loaded Piles in a Layered Soil System," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, SM3, May, 1963, pp 63-94.

5. Gill, H. L. and Demars, K. R., "Displacement of Laterally Loaded Structures in Nonlinearly Responsive Soil," Technical Report R-670, Naval Civil Engineering Laboratory, Port Hueneme, California, April 1970.

6. Gleser, S. M., "Lateral Load Tests on Vertical Fixed-Head and Free-Head Piles," Symposium on Lateral Load Tests on Piles, Special Technical Publication No. 154, ASTM, Philadelphia, Pa., 195[illegible], pp [illegible]-93.

7. Ippen, A. T., (editor), Estuary and Coastline Hydrodynamics, McGraw-Hill Book Company, Incorporated, 1966, pp 357-363.

8. Lowery, L. L., Jr., Hirsch, T. J., Edwards, T. C., Coyle, H. M., and Samson, C. H., Jr., "Pile-Driving Analysis - State of the Art," Research Report No. 33-13, Texas Transportation Institute, Texas A&M University, January, 1969.

9. Matlock, H., "Correlations for Design of Laterally Loaded Piles in Soft Clay," Preprints of the 1970 Offshore Technology Conference, Houston, Texas, Vol. 1, No. OTC 1204, April, 1970, pp 577-588.

10. Matlock, H., and Reese, L. C., "Generalized Solutions for Laterally Loaded Piles," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 86, SM5, October, 1960, pp 63-91.

11. McClelland, B., and Focht, J. A., Jr., "Soil Modulus for Laterally Loaded Piles," Transactions, ASCE, Vol. 123, 1958, pp 1049-1086.

12. Palmer, L. A., and Brown, P. P., "Analysis of Pressure Deflection, Moment, and Shear by the Method of Difference Equations," Symposium on Lateral Load Tests on Piles, Supplement, Special Technical Publication No. 154-A, ASTM, Philadelphia, Pa., 1954, pp 22-32.

13. Palmer, L. A., and Thompson, J. B., "The Earth Pressure and Deflection Along the Embedded Lengths of Piles Subjected to Lateral Thrust," Proceedings, Second International Conference on Soil Mechanics and Foundations Engineering, Rotterdam, Holland, Vol. V, 1948, pp 156-161.

14. Parker, F., Jr., and Cox, W. R., "A Method for the Analysis of Pile Supported Foundations Considering Nonlinear Soil Behavior," Research Report No. 117-1, Center for Highway Research, University of Texas, June, 1969.

15. Penzien, J., "Soil-Pile Foundation Interaction," Earthquake Engineering, Wiegel, R. L., Coordinating Editor, Prentice-Hall, Incorporated, Englewood Cliffs, New Jersey, 1970, pp 349-381.

16. Reese, L. C., Discussion of "Soil Modulus for Laterally Loaded Piles," Transactions, ASCE, Vol. 123, 1958, pp 1071-1074.

17. Reese, L. C., and Matlock, H., "Non-dimensional Solutions for Laterally Loaded Piles with Soil Modulus Assumed Proportional to Depth," Proceedings, Eighth Texas Conference on Soil Mechanics and Foundation Engineering, Bureau of Engineering Research, University of Texas, September, 1956.

18. Rehmet, J. D., and Coyle, H. M., "Preliminary Study of In-Situ Measurements of Friction and Bearing in Clay," Research Report 125-3, Texas Transportation Institute, Texas A&M University, September, 1969.

19. Ross, H. E., Jr., "Dynamic Response of Offshore Piling," PhD Dissertation, Texas A&M University, August, 1970.

20. Samson, C. H., Jr., Hirsch, T. J., and Lowery, L. L., Jr., "Computer Study of Dynamic Behavior of Piling," Journal of the Structural Division, ASCE, Paper No. 3608, ST4, August, 1963.

21. Skempton, A. W., "The Bearing Capacity of Clays," Building Research Congress, Division 1, Part 3, London, 1951, pp 180-189.

22. Smith, E. A. L., "Pile-Driving Analysis by the Wave Equation," Journal of the Soil Mechanics and Foundations Division, ASCE, Paper No. 2574, SM4, August, 1960.

23. Taylor, P. W., and Hughes, J. M. O., "Dynamic Properties of Foundation Subsoils as Determined from Laboratory Tests," Proceedings, Third World Conference on Earthquake Engineering, Vol. 1, pp I-196 to I-212.

24. Terzaghi, K., "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, London, Vol. V, No. 4, 1955, pp 297-326.

25. Tucker, R. L., "Lateral Analysis of Piles with Dynamic Behavior," Proceedings, Conference on Deep Foundations, Mexico City, December, 1964.

26. Vesic, A. B., "Bending of Beams Resting on Isotropic Elastic Solid," Journal of Engineering Mechanics, ASCE, Vol. 87, EM2, April, 1961, pp 35-51.

APPENDIX II. - NOTATION

The following symbols are used in this paper:

A = cross-sectional area in square inches

D = pile diameter in inches

E = modulus of elasticity in pounds per square inch

G = gravitational acceleration

I = moment of inertia in inches to the fourth power

J = soil damping factor in seconds to the N power per foot to the N power

K = soil spring constant in pounds per inch

k_F = field modulus of horizontal subgrade reaction in pounds per cubic inch

k_H = soil modulus of horizontal subgrade reaction in pounds per cubic inch

k_L = laboratory modulus of horizontal subgrade reaction in pounds per cubic inch

k_S = soil modulus of subgrade reaction in pounds per cubic inch

L_A = length of pile above the groundline in feet

L_E = embedded length of pile in feet

l = length of pile segment in idealized model in inches

N = power which to raise the velocity, V

P = horizontal load in pounds

P_f = horizontal failure load in pounds

P_{static} = horizontal static load in pounds

$P_{dynamic}$ = horizontal dynamic load in pounds

psi = pounds per square inch

Q = maximum elastic deformation of the soil or quake in inches

V = velocity in inches per second

y = displacement in inches

z = depth in feet

α = constant based on pile properties

ε_F = field strain in inches per inch

ε_L = laboratory strain in inches per inch

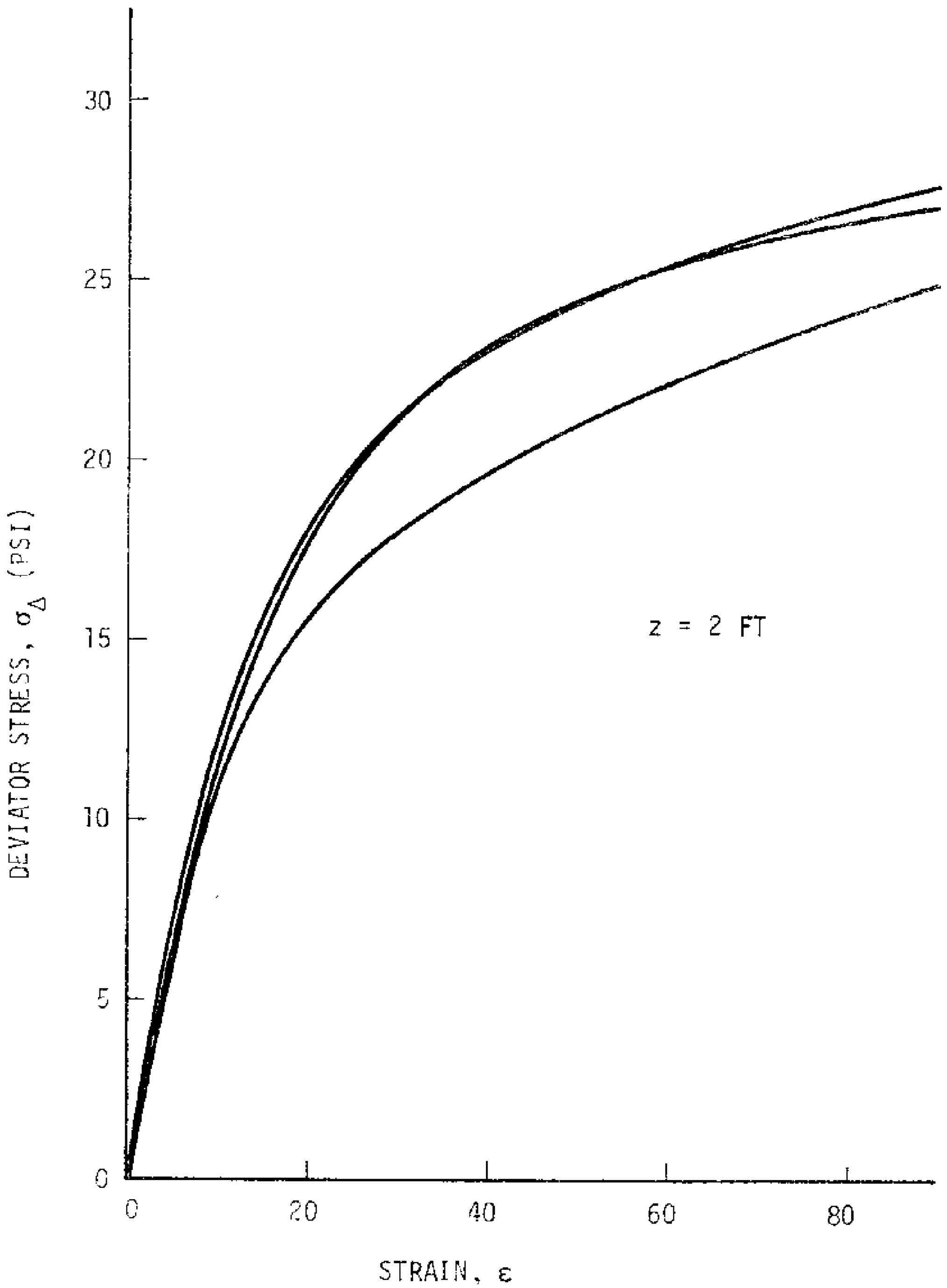
ΔL = deformation of triaxial specimen in inches

σ_Δ = laboratory deviator stress in pounds per square inch

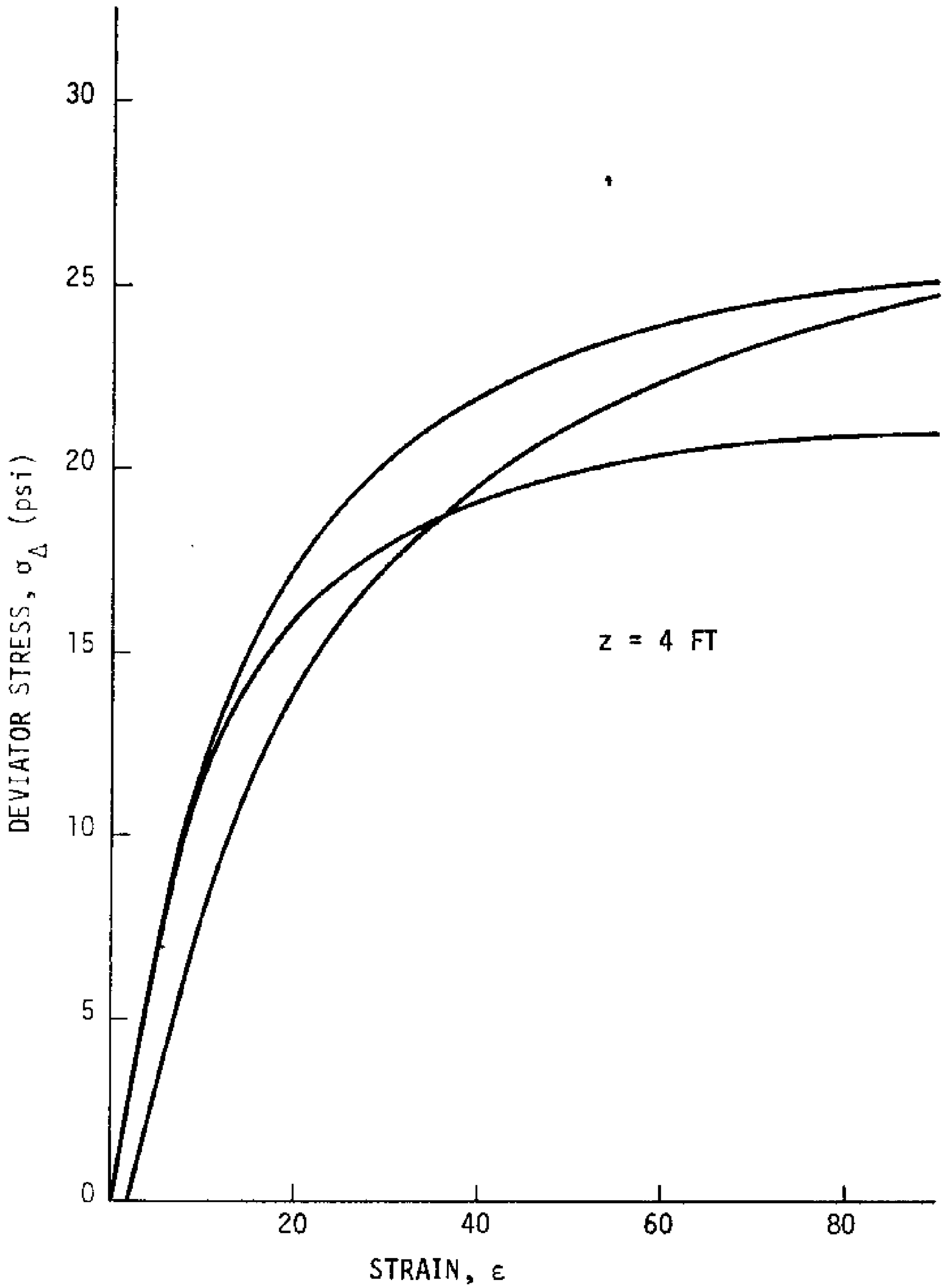
σ_F = field stress in pounds per square inch

APPENDIX III. - LABORATORY SOIL TEST DATA

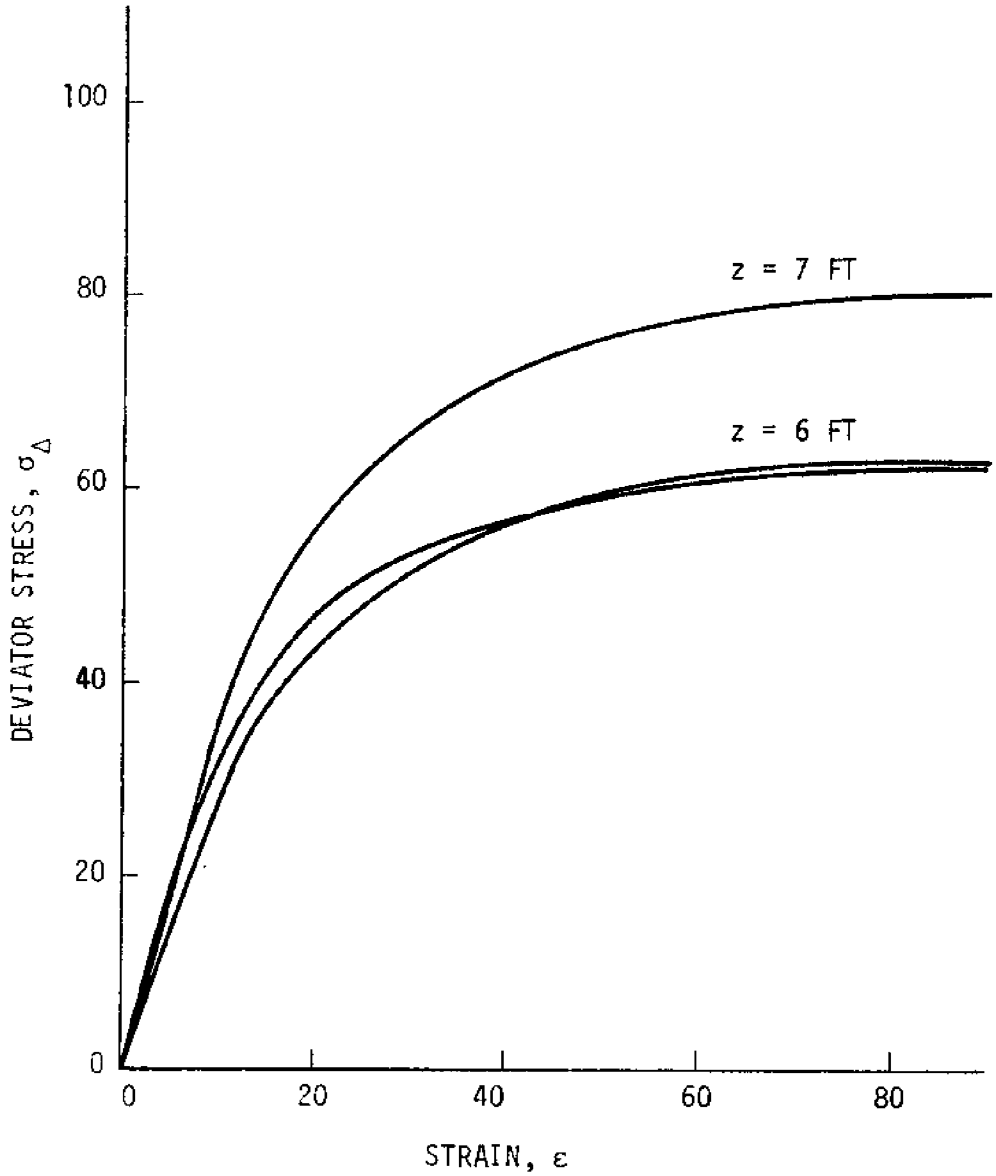
On the following pages are the deviator stress versus strain curves for the stiff clay in which the model tests for this study were made. The curves are from unconsolidated-undrained triaxial tests conducted on 1.5 in. diameter samples.



DEVIATOR STRESS VERSUS STRAIN, z = 2 FT



DEVIATOR STRESS VERSUS STRAIN, z = 4 FT



DEVIATOR STRESS VERSUS STRAIN, z = 6 AND 7 FT

APPENDIX IV. - FIELD DATA

On the following pages are the field data for all 25 tests conducted in this study. The following notation is used:

Gage 1 - Bending moments determined at a depth of six feet

Gage 2 - Bending moments determined at a depth of four feet

Gage 3 - Bending moments determined at a depth of two feet

Gage 4 - Bending moments determined at a depth of 0.5 feet

Gage 5 - Accelerations measured at the groundline

Gage 6 - Accelerations measured at the pile top

Gage 7 - Displacements measured at the point of force application

Displacements were not recorded on all 25 tests. Bending strains were measured by Gage 1 on the 3 inch pile tests only. Gage 1 measured no strains at the six foot depth in the 2 inch and 1.25 inch pile tests.

Test 3-8-1

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lb	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lb	Time Sec	Moment Ft-Lb	Time Sec	Acceleration G	Time Sec	Acceleration G	Time Sec	Displacement In.
.00	-8.84	.00	-23.55	.00	362	.00	390	.00	-.00	.00	.00	.00	.106
.15	.63	.08	-17.27	.13	-113	.13	-287	.03	.101	.03	.44	.14	-.053
.25	-4.10	.12	-19.73	.24	159	.24	245	.14	-.091	.13	-.43	.25	.058
.37	.32	.23	- 6.90	.36	- 96	.36	-239	.26	.071	.24	.38	.35	-.048
.48	-3.47	.35	-16.30	.47	133	.47	211	.37	-.065	.36	-.36	.47	.052
.62	.32	.39	- 6.27	.58	- 78	.58	-204	.49	.06	.47	.32	.59	-.038
.71	-2.94	.58	-15.1	.69	115	.69	183	.59	-.052	.58	-.31	.69	.043
.81	.32	.67	- 5.95	.80	- 68	.80	-179	.71	.048	.69	.28	.80	-.034
.93	-2.64	.80	-13.8	.91	99	.91	161	.81	-.045	.80	-.27	.80	.038
1.03	.00	.90	- 5.95	1.02	- 58	1.02	-157	.93	.043	.91	.24	1.02	-.029
1.14	-2.84	1.02	-13.5	1.13	85	1.13	142	1.04	-.039	1.02	-.23	1.13	.033
1.25	- .32	1.13	- 5.95	1.24	- 50	1.24	-138	1.15	.038	1.13	.22	1.24	-.025
1.35	-2.84	1.24	-12.9	1.35	75	1.35	128	1.26	-.036	1.24	-.21	1.35	.029
1.47	- .32	1.37	- 5.64	1.46	- 43	1.46	-124	1.37	.033	1.35	.19	1.46	-.022
1.57	-2.53	1.46	-11.9	1.57	68	1.57	114	1.48	-.031	1.46	-.18	1.57	.026
1.69	- .32	1.56	- 5.95	1.68	- 38	1.68	-112	1.60	.029	1.57	.17	1.68	-.020
1.79	-2.21	1.67	-11.3	1.73	60	1.73	104	1.70	-.028	1.68	-.17	1.79	.023
1.90	- .63	1.78	- 5.95	1.90	- 32	1.90	- 99	1.80	.027	1.79	.16	1.90	-.018
2.02	-2.21	1.90	-11.3	2.00	56	2.00	94	1.91	-.026	1.90	-.15	2.01	.021
2.13	- .32	1.99	- 6.27	2.11	- 29	2.11	- 91	2.02	.024	2.00	.14	2.12	-.016
2.23	-2.21	2.11	-10.9	2.22	51	2.22	84	2.13	-.023	2.12	-.14	2.23	.019

Test 3-8-2

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time Sec	Accel-eration G	Time Sec	Displace-ment In.
.00	-11.4	.00	-37.0	.00	510	.00	525	.00	.00	.00	.00	.00	.00
.14	1.26	.07	-17.9	.14	-162	.14	-358	.05	.191	.03	.705	.14	.074
.24	- 4.74	.12	-21.6	.24	203	.24	301	.15	-.120	.14	-.562	.24	-.074
.36	1.26	.21	- 5.92	.36	-130	.36	-286	.26	.089	.24	.445	.36	.060
.47	- 4.11	.36	-15.4	.47	160	.47	250	.38	-.077	.36	-.435	.47	-.067
.58	.95	.44	- 4.66	.58	-106	.58	-237	.49	.072	.47	.366	.58	.048
.69	- 3.16	.58	-13.8	.69	130	.69	210	.60	-.056	.58	-.361	.69	-.051
.81	.95	.67	- 4.35	.81	- 89	.81	-200	.71	.056	.69	.305	.81	.040
.92	- 2.84	.81	-12.2	.92	109	.92	180	.82	-.049	.81	-.282	.92	-.043
1.03	.63	.89	- 3.74	1.03	- 77	1.03	-172	.93	.048	.92	.261	1.03	.035
1.14	2.21	1.02	-11.3	1.14	91	1.14	158	1.05	-.042	1.03	-.263	1.14	-.036
1.25	.32	1.12	- 3.42	1.25	- 65	1.25	-150	1.16	.043	1.14	.228	1.25	.030
1.36	- 1.90	1.24	-10.4	1.36	77	1.36	139	1.27	-.038	1.25	-.228	1.36	-.031
1.47	.32	1.34	- 3.74	1.47	- 55	1.47	-132	1.38	.037	1.36	.200	1.47	.027
1.57	- 1.90	1.46	- 9.65	1.57	67	1.57	123	1.49	-.033	1.47	-.203	1.57	-.027
1.69	.00	1.57	- 3.74	1.69	- 48	1.69	-116	1.59	.033	1.57	.175	1.69	.022
1.80	- 1.58	1.68	- 9.34	1.80	60	1.80	108	1.71	-.03	1.69	-.180	1.80	-.025
1.91	.00	1.78	- 4.05	1.91	- 41	1.91	-104	1.81	.029	1.80	.156	1.91	.020
2.02	- 1.26	1.90	- 9.03	2.02	51	2.02	97	1.93	-.027	1.91	-.159	2.02	-.022
2.12	- .32	2.00	- 4.35	2.12	- 38	2.12	- 92	2.03	.025	2.02	.138	2.12	.018
2.23	- .95	2.12	- 8.40	2.23	40	2.23	88	2.14	-.024	2.12	-.142	2.23	-.020
								2.25	.023	2.23	.124		

Test 3-8-3

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time Sec	Accel-eration G	Time Sec	Displace-ment In.
.00	-9.16	.00	-34.79	.00	506	.00	523	.00	.00	.00	-.00	.00	.130
.14	3.47	.09	-17.86	.14	-133	.14	-354	.12	-.110	.14	-.548	.14	-.087
.26	-2.84	.13	-19.43	.26	211	.25	296	.27	.088	.25	.436	.26	.058
.37	2.84	.21	- 5.32	.37	-121	.36	-285	.39	-.069	.37	-.440	.38	-.075
.49	-1.89	.36	-15.04	.48	165	.48	245	.50	.070	.48	.354	.48	.044
.60	2.21	.44	- 4.38	.59	-102	.59	-238	.61	-.055	.59	-.363	.59	-.063
.71	-1.58	.59	-13.16	.71	131	.70	204	.72	.057	.70	.294	.70	.034
.83	2.21	.67	- 4.70	.82	- 85	.82	-198	.84	-.046	.82	-.317	.82	-.056
.94	-1.26	.82	-12.22	.93	110	.93	177	.95	.049	.93	.245	.93	.026
1.05	1.58	.90	- 4.38	1.04	- 75	1.04	-173	1.06	-.040	1.04	-.275	1.04	-.049
1.16	- .94	1.04	-11.28	1.15	90	1.15	151	1.17	.042	1.15	.207	1.15	.020
1.27	1.26	1.13	- 4.38	1.26	- 61	1.26	-151	1.27	-.036	1.26	-.242	1.26	-.045
1.37	- .94	1.25	-10.97	1.37	80	1.37	130	1.39	.036	1.37	.175	1.37	.016
1.49	.63	1.36	- 4.70	1.48	- 56	1.48	-133	1.50	-.032	1.49	-.217	1.48	-.040
1.60	- .94	1.48	-10.34	1.59	68	1.59	114	1.61	.031	1.59	.161	1.59	.011
1.71	.63	1.57	- 4.70	1.71	- 47	1.70	-117	1.72	-.029	1.70	-.191	1.70	-.038
1.81	-1.26	1.70	- 9.09	1.82	57	1.81	102	1.83	.027	1.81	.140	1.81	.009
1.92	.63	1.80	- 4.38	1.92	- 40	1.92	-106	1.94	-.026	1.92	-.175	1.92	-.036
2.03	-1.26	1.92	- 9.09	2.03	49	2.03	89	2.05	.023	2.03	.121	2.03	.006
2.14	.00	2.02	- 4.70	2.14	- 39	2.14	- 96	2.15	-.025	2.14	-.158	2.14	-.033
2.25	-1.26	2.14	- 9.09	2.25	42			2.26	.019	2.25	.103	2.25	.004
2.36	- .31	2.24	- 5.01	2.36	- 34					2.36	-.147	2.36	-.030
		2.36	- 9.09										

Test 3-8-4

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-14.00	.00	-11.15	.00	431	.00	579	.00	.000	.00	.000
.03	-11.28	.02	-15.74	.02	406	.01	545	.02	.311	.02	1.072
.04	-12.83	.03	-11.15	.03	249	.03	312	.04	-.233	.04	.112
.05	- 9.72	.05	-17.71	.04	300	.03	375	.07	.115	.05	.727
.06	-10.50	.06	-15.08	.06	166	.05	167	.11	-.098	.10	-.466
.13	- 5.05	.12	-26.24	.07	19	.12	-409	.13	-.098	.13	-.708
.24	-10.50	.21	- 7.87	.13	-137	.23	346	.23	.087	.23	.620
.34	- 5.05	.34	-24.27	.23	223	.34	-369	.35	-.073	.34	-.592
.45	-10.11	.43	- 7.87	.34	-127	.44	301	.46	.082	.44	.555
.56	- 5.44	.55	-22.96	.45	198	.55	-338	.56	-.062	.55	-.522
.66	- 9.72	.65	- 7.87	.56	-121	.66	261	.67	.075	.65	.503
.77	- 5.83	.77	-22.30	.66	169	.76	-306	.77	-.053	.76	-.452
.89	- 9.72	.86	- 8.52	.77	-111	.88	232	.90	.067	.88	.461
1.00	- 6.22	.98	-22.30	.88	143	.98	-284	1.00	-.047	.99	-.405
1.11	- 9.72	1.08	- 9.84	.98	-108	1.09	204	1.11	.062	1.09	.424
1.21	- 6.22	1.20	-20.99	1.09	124	1.19	-255	1.21	-.040	1.20	-.359
1.32	- 9.72	1.30	- 9.84	1.20	- 95	1.30	178	1.32	.060	1.31	.387
1.42	- 7.39	1.41	-20.99	1.31	102	1.41	-235	1.41	-.031	1.41	-.312
1.53	- 9.72	1.51	-10.49	1.42	- 95	1.51	159	1.53	.055	1.52	.354
1.62	- 7.39	1.62	-20.33	1.63	- 89	1.62	-218	1.62	-.027	1.62	-.275
1.73	- 9.33	1.72	-11.15	1.73	73	1.72	139	1.74	.053	1.73	.331
1.84	- 7.39	1.83	-20.33	1.83	- 83	1.83	-201	1.83	-.020	1.83	-.242
1.95	- 9.33	1.93	-11.15	1.94	60	1.93	122	1.95	.049	1.94	.303
2.05	- 7.39	2.04	-20.33	2.05	- 76	2.04	-187	2.04	-.016	2.04	-.210
2.15	- 9.33	2.14	-11.80	2.15	51	2.14	105	2.16	.047	2.15	.280
2.26	- 8.16	2.25	-19.68	2.26	- 73	2.25	-176	2.26	-.011	2.26	-.186

Test 3-8-5

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time	Accel-eration G
.00	-7.00	.00	- 5.24	.00	483	.00	631	.00	.00	.00	.000
.01	-4.66	.01	- 5.90	.02	272	.02	347	.02	.362	.02	1.212
.02	-6.22	.01	- 9.84	.03	333	.03	412	.03	-.255	.04	.205
.04	-2.33	.03	- 2.62	.05	182	.04	224	.04	.186	.07	- .070
.04	-3.11	.04	-10.49	.07	0	.05	190	.07	-.069	.08	- .550
.11	2.33	.05	- 7.87	.12	-157	.12	-449	.12	-.102	.11	- .815
.23	-3.11	.06	-11.15	.23	256	.23	392	.23	.109	.23	.666
.34	2.72	.07	-10.49	.34	-144	.33	-395	.33	-.071	.34	- .671
.44	-2.72	.12	-19.02	.45	227	.44	349	.43	.098	.44	.592
.55	1.94	.20	1.96	.55	-131	.55	-358	.54	-.060	.55	- .601
.66	-1.94	.33	-15.74	.66	198	.66	310	.66	.089	.66	.536
.76	1.55	.42	1.31	.77	-118	.77	-324	.76	-.053	.77	- .536
.88	-2.72	.55	-15.08	.87	176	.87	278	.88	.082	.87	.480
.98	.77	.63	.65	.99	-109	.98	-290	.98	-.047	.98	- .480
1.09	-2.72	.76	-14.43	1.09	153	1.08	247	1.09	.075	1.08	.438
1.20	.77	.85	.00	1.20	- 99	1.19	-261	1.20	-.042	1.20	- .419
1.30	-1.94	.98	-13.77	1.31	134	1.30	219	1.31	.071	1.30	.396
1.41	.77	1.07	.00	1.41	- 90	1.41	-241	1.41	-.033	1.41	- .373
1.51	-2.33	1.20	-12.46	1.52	112	1.51	196	1.52	.067	1.52	.368
1.64	.00	1.29	.00	1.64	- 86	1.63	-219	1.64	-.027	1.63	- .336
1.75	-2.72	1.41	-11.80	1.74	99	1.74	173	1.74	.064	1.74	.336
1.86	.00	1.51	.00	1.85	- 77	1.85	-202	1.85	-.018	1.85	- .298
1.96	-2.72	1.63	-11.15	1.96	86	1.95	153	1.96	.060	1.95	.303
2.06	.00	1.73	- 1.31	2.06	- 74	2.06	-182	2.06	-.011	2.06	- .266
2.16	-2.72	1.85	-11.15	2.17	70	2.16	136	2.17	.058	2.16	.284
2.28	- .77	1.95	- 1.31	2.27	- 70	2.27	-170	2.27	-.011	2.27	- .228

Test 3-8-6

Gage 1		Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time Sec	Accel-eration G
.00	-12.83	.00	-17.71	.00	719	.00	915	.00	.00	.00	.00
.13	2.72	.12	-29.52	.12	-246	.12	-636	.12	-.154	.12	-.110
.24	- 6.61	.24	- 4.59	.23	380	.23	537	.24	.131	.23	.093
.35	2.33	.34	-24.27	.34	-221	.34	-554	.35	-.116	.34	-.095
.46	- 5.83	.45	- 3.28	.45	336	.45	474	.44	.116	.45	.080
.57	1.94	.55	-20.99	.56	-198	.56	-486	.56	-.098	.56	-.083
.68	- 4.66	.66	- 1.96	.67	288	.67	412	.66	.105	.66	.070
.79	1.55	.77	-19.02	.78	-176	.77	-432	.78	-.089	.77	-.074
.90	- 4.27	.85	- .65	.89	246	.89	358	.89	.093	.88	.062
1.01	1.16	.99	-17.71	1.00	-157	.99	-394	1.00	-.080	1.00	-.064
1.11	- 3.89	1.07	- 1.31	1.10	214	1.10	315	1.11	.082	1.10	.055
1.22	.77	1.21	-15.74	1.21	-141	1.21	-341	1.21	-.071	1.21	-.056
1.33	- 3.50	1.29	- 1.31	1.32	182	1.32	276	1.32	.071	1.32	.049
1.44	.38	1.43	-14.43	1.43	-131	1.43	-304	1.43	-.062	1.43	-.048
1.55	- 3.50	1.51	- 1.31	1.54	156	1.53	241	1.54	.065	1.53	.044
1.66	.77	1.66	-12.46	1.65	-112	1.65	-273	1.66	-.053	1.65	-.044
1.76	- 2.72	1.74	.00	1.76	137	1.76	222	1.76	.056	1.75	.039
1.86	.77	1.86	-11.15	1.86	-102	1.86	-244	1.87	-.047	1.86	-.039
1.97	- 1.94	1.96	.00	1.97	121	1.97	199	1.98	.049	1.97	.035
2.08	.77	2.08	-11.15	2.08	- 93	2.08	-219	2.08	-.042	2.08	-.034
2.19	- 1.94	2.17	- 1.31	2.19	102	2.19	176	2.20	.047	2.19	.032
2.29	.38	2.29	-11.15	2.29	- 83	2.29	-196	2.30	-.036	2.29	-.031

Test 1-8-1

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time Sec	Accel-eration G	Time Sec	Displace-ment In.
.00	3.91	.00	86.04	.00	150	.00	.013	.00	.023	.00	1.524
.04	2.87	.06	56.13	.06	94	.03	.533	.04	.759	.03	1.518
.07	2.87	.10	26.79	.10	33	.16	.202	.13	.136	.15	- .372
.26	.46	.24	-16.36	.25	-146	.27	-.253	.24	-.646	.22	- .619
.44	2.07	.44	42.31	.43	70	.42	.183	.43	.533	.43	1.390
.66	.23	.65	-15.79	.64	-146	.66	-.117	.63	-.510	.63	- .598
.85	1.84	.84	36.95	.83	56	.86	.102	.83	.442	.83	1.277
1.06	.23	1.05	-15.23	1.04	-144	1.06	-.081	1.04	-.476	1.04	- .580
1.25	1.72	1.24	33.28	1.24	48	1.25	.092	1.24	.397	1.24	1.187
1.45	.34	1.45	-13.82	1.44	-137	1.45	-.077	1.44	-.431	1.44	- .554
1.56	1.61	1.65	30.46	1.64	41	1.65	.079	1.64	.363	1.64	1.098
1.88	.34	1.86	-13.25	1.85	-130	1.87	-.068	1.85	-.419	1.85	- .539
2.06	1.49	2.06	27.08	2.05	36	2.06	.077	2.05	.329	2.05	1.021
2.27	.34	2.27	-12.41	2.25	-123	2.27	-.062	2.25	-.385	2.25	- .515
2.49	1.26	2.46	24.26	2.45	29	2.46	.066	2.45	.306	2.45	.949
2.71	.46	2.67	-11.28	2.65	-117	2.67	-.062	2.66	-.363	2.65	- .497
2.88	1.38	2.87	22.00	2.86	25	2.87	.060	2.85	.283	2.85	.884
3.09	.34	3.08	-11.00	3.06	-113	3.08	-.059	3.06	-.340	3.06	- .476

Test 1-8-2

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G	Time Sec	Displacement In.
.00	2.64	.00	73.91	.00	142.65	.00	.015	.00	.00	.00	.583
.24	.00	.24	-12.41	.22	-100.30	.23	-.077	.21	-.453	.22	-.483
.46	1.38	.45	33.28	.44	90.64	.43	.085	.43	.442	.44	.533
.66	.00	.65	-11.28	.63	- 89.16	.64	-.064	.64	-.397	.63	-.456
.87	1.15	.86	28.77	.84	84.70	.87	.079	.84	.397	.84	.489
1.08	.00	1.06	-10.72	1.05	- 82.47	1.08	-.060	1.05	-.374	1.04	-.430
1.28	1.03	1.27	27.08	1.26	78.01	1.28	.077	1.27	.363	1.27	.450
1.52	.00	1.48	-10.15	1.46	- 75.04	1.49	-.057	1.47	-.363	1.46	-.407
1.73	.80	1.69	24.54	1.66	74.30	1.70	.068	1.68	.329	1.68	.419
1.93	114.88	1.91	- 9.30	1.88	- 69.09	1.92	-.051	1.89	-.329	1.89	-.383
2.14	.69	2.10	22.00	2.09	69.84	2.12	.060	2.09	.306	2.09	.383
2.34	114.88	2.32	- 8.74	2.30	- 64.64	2.32	-.045	2.30	-.306	2.29	-.361
2.54	.69	2.51	20.02	2.51	64.64	2.53	.059	2.50	.283	2.51	.356
2.74	.11	2.73	- 8.18	2.70	- 57.95	2.73	-.045	2.72	-.272	2.70	-.332
2.96	.69	2.93	18.05	2.91	59.44	2.94	.055	2.91	.261	2.92	.326
3.15	.11	3.14	- 7.33	3.12	- 53.49	3.15	-.040	3.12	-.261	3.12	-.309
3.37	.69	3.35	16.64	3.33	55.72	3.35	.051	3.34	.238	3.34	.304
3.58	.11	3.55	- 6.77	3.53	- 49.78	3.56	-.038	3.54	-.238	3.53	-.285
3.77	.57	3.75	14.66	3.73	52.75	3.76	.045	3.73	.227	3.75	.282
3.99	.23	3.97	- 6.48	3.94	- 45.32	3.98	-.036	3.95	-.227	3.94	-.267
4.17	.57	4.17	12.97	4.15	49.03	4.17	.043	4.15	.215	4.16	.259
4.40	.11	4.38	- 5.64	4.37	- 42.35	4.39	-.034	4.37	-.215	4.36	-.244
4.58	.57	4.58	11.56	4.57	44.58	4.59	.038	4.56	.193	4.57	.232
4.78	.23	4.79	- 5.36	4.77	- 38.63	5.80	-.030	4.78	-.193	4.77	-.226
4.99	.46	4.99	9.87	4.97	41.60	5.00	.036	4.98	.181	4.97	.213

Test 1-8-3

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G	Time Sec	Displacement In.
.00	1.61	.00	40.72	.00	306	.00	.00	.00	.00	.00	.216
.28	.80	.27	2.58	.26	168	.23	-.014	.24	-.280	.22	-.078
.47	1.04	.45	17.78	.44	268	.43	.007	.44	.224	.45	.135
.69	.69	.65	2.00	.63	171	.66	-.007	.63	-.213	.65	-.066
.84	.80	.85	15.20	.84	263	.85	.004	.83	.190	.84	.125
1.06	.57	1.06	2.00	1.04	174	1.06	-.005	1.03	-.202	1.03	-.059
1.25	.80	1.25	14.05	1.23	259	1.25	.002	1.23	.168	1.23	.115
1.46	.46	1.46	2.00	1.43	178	1.46	-.005	1.43	-.190	1.43	-.053
1.65	.69	1.65	12.90	1.64	257	1.66	.002	1.64	.157	1.64	.107
1.87	.46	1.86	2.29	1.84	182	1.86	-.004	1.83	-.157	1.83	-.046
2.05	.69	2.05	12.04	2.03	253	2.04	.00	2.03	.157	2.04	.101
2.26	.46	2.25	2.58	2.23	185	2.25	-.004	2.23	-.146	2.24	-.039
2.47	.57	2.45	11.18	2.44	250	2.46	.00	2.44	.134	2.44	.095
2.66	.57	2.66	2.86	2.64	187	2.67	-.004	2.64	-.123	2.64	-.034
2.84	.57	2.85	10.61	2.84	246	2.85	.00	2.83	.134	2.84	.088
3.06	.46	3.06	2.86	3.04	188	3.07	-.004	3.03	-.123	3.04	-.030

Test 1-8-4

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	5.93	.00	36.22	.00	100.44	.00	.003	.00	-.121
.12	5.75	.11	23.11	.10	56.98	.12	.066	.08	.275
.29	4.89	.27	- 9.05	.26	- 48.77	.23	-.051	.25	-.407
.47	5.44	.46	8.32	.47	97.54	.47	.036	.46	.242
.68	4.83	.66	-13.94	.66	- 18.35	.67	-.039	.66	-.363
.87	5.32	.87	14.47	.86	55.05	.87	.029	.86	.220
1.09	4.89	1.07	- 8.01	1.05	- 38.14	1.07	-.034	1.05	-.319
1.27	5.32	1.26	12.70	1.25	49.73	1.27	.028	1.25	.198
1.48	4.77	1.46	- 7.49	1.45	- 34.76	1.46	-.030	1.45	-.275
1.66	5.14	1.66	11.03	1.65	44.91	1.62	.026	1.64	.165
1.86	4.77	1.85	- 6.97	1.84	- 30.42	1.86	-.026	1.84	-.264
2.06	5.14	2.05	9.68	2.04	41.52	2.02	.022	2.03	.154
2.25	4.77	2.24	- 6.45	2.23	- 27.52	2.26	-.022	2.23	-.242
2.44	5.01	2.44	8.32	2.43	38.14	2.42	.021	2.43	.143
2.64	4.65	2.64	- 6.14	2.62	- 24.14	2.65	-.019	2.62	-.220
2.84	4.95	2.83	7.18	2.82	34.76	2.81	.019	2.83	.121
3.04	4.59	3.03	- 5.83	3.01	- 21.73	3.03	-.016	3.02	-.187

Test 1-8-5

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	- .18	.00	36.22	.00	86.92	.00	.00	.00	-.022
.11	- .49	.08	26.02	.11	29.94	.09	-.034	.08	.220
.26	-1.22	.26	- 8.01	.24	-61.32	.26	-.014	.25	-.358
.46	- .79	.44	17.28	.44	47.32	.45	.059	.45	.280
.65	-1.28	.64	- 7.39	.64	-56.01	.65	-.013	.64	-.319
.85	- .91	.84	15.51	.83	42.01	.85	.052	.84	.248
1.05	-1.40	1.03	- 6.97	1.03	-51.67	1.05	-.012	1.03	-.297
1.25	-1.04	1.23	13.63	1.23	37.18	1.20	.050	1.23	.225
1.45	-1.46	1.43	- 6.35	1.43	-49.25	1.44	-.007	1.42	-.270
1.65	-1.10	1.63	12.38	1.62	29.94	1.53	.046	1.62	.204
1.89	-2.44	1.83	- 5.72	1.82	-46.84	1.77	-.002	1.82	-.248
2.03	-2.26	2.02	10.93	2.02	25.59	2.03	.043	2.00	.181
2.23	-2.69	2.21	- 5.30	2.21	-43.46	2.12	.001	2.19	-.231
2.41	-2.57	2.41	9.57	2.41	21.73	2.32	.044	2.39	.160
2.62	-2.99	2.60	- 4.78	2.60	-39.59	2.52	.001	2.59	-.204
2.81	-2.69	2.80	8.53	2.78	19.31	2.79	.043	2.78	.143
3.03	-2.93	3.00	- 4.16	2.98	-37.18	2.99	.002	2.98	-.187

Test 1-8-6

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	.00	.00	17.49	.00	161.73	.00	.00	.00	.011
		.11	15.79	.12	- 19.95	.10	.118	.07	.512
		.29	5.07	.22	- 94.51	.23	-.065	.23	-.500
		.42	10.15	.37	88.21	.36	.041	.37	.512
		.64	4.51	.55	- 82.96	.55	-.020	.54	-.489
		.76	8.46	.73	75.61	.74	.037	.72	.455
		.99	3.38	.90	- 73.51	.91	-.016	.90	-.421
		1.13	7.89	1.07	68.26	1.10	.037	1.07	.398
		1.34	2.82	1.25	- 66.16	1.26	-.008	1.25	-.375
		1.48	5.64	1.43	58.81	1.45	.037	1.43	.353
		1.70	1.69	1.62	- 57.76	1.63	-.004	1.61	-.341
		1.84	5.07	1.79	52.51	1.81	.033	1.79	.307
		2.07	1.12	1.97	- 53.56	1.98	-.00	1.96	-.296
		2.19	2.82	2.15	44.10	2.16	.033	2.14	.273
		2.41	2.25	2.33	- 49.35	2.33	-.00	2.32	-.262
		2.54	3.38	2.50	36.75	2.52	.033	2.50	.250
		2.76	1.69	2.68	- 46.20	2.69	-.00	2.68	-.239
		2.90	3.38	2.86	32.55	2.87	.029	2.85	.227
		3.10	1.69	3.04	- 40.95	3.04	.004	3.03	-.205

Test 1-8-7

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	.00	.00	21.44	.00	172.75	.00	.004	.00	.00
		.27	7.67	.22	- 97.66	.23	-.069	.23	-.557
		.42	13.76	.38	98.19	.36	.053	.37	.557
		.63	6.54	.55	- 85.06	.55	-.037	.55	-.557
		.77	11.51	.73	85.06	.77	.029	.73	.466
		.98	6.32	.91	- 74.56	.93	-.037	.90	-.478
		1.14	10.83	1.08	75.61	1.10	.020	1.08	.409
		1.33	6.32	1.26	- 65.63	1.28	-.037	1.26	-.444
		1.49	9.93	1.44	69.31	1.46	.012	1.43	.353
		1.69	6.32	1.62	- 55.13	1.63	-.033	1.62	-.398
		1.86	8.80	1.80	64.06	1.81	-.00	1.80	.307
		2.06	6.09	1.97	- 48.83	1.98	-.041	1.97	-.353
		2.20	8.80	2.15	57.23	2.17	-.00	2.15	.273
		2.42	6.09	2.33	- 43.58	2.34	-.037	2.33	-.307
		2.56	8.80	2.51	49.88	2.52	-.00	2.50	.227
		2.76	6.54	2.69	- 37.28	2.68	-.037	2.68	-.284
		2.91	8.12	2.86	44.10	2.87	-.004	2.86	.193
		3.12	6.32	3.03	- 35.18	3.03	-.029	3.04	-.239

Test 2-6-1

Gage 2 Time Sec	Gage 2 Moment Ft-Lbs	Gage 3 Time Sec	Gage 3 Moment Ft-Lbs	Gage 4 Time Sec	Gage 4 Moment Ft-Lbs	Gage 5 Time Sec	Gage 5 Acceleration G	Gage 6 Time Sec	Gage 6 Acceleration G	Gage 7 Time Sec	Gage 7 Displacement In.
.00	4.77	.00	203	.00	183	.00	.035	.00	-.058	.00	.212
.26	-4.55	.27	-154	.26	-138	.23	.00	.25	-.290	.25	-.131
.50	2.44	.50	104	.49	111	.51	.049	.49	.151	..50	.129
.72	-3.88	.73	-136	.72	-120	.74	.010	.72	-.255	.72	-.114
.96	2.10	.97	90	.96	97	.98	.049	.96	.116	.97	.112
1.18	-3.33	1.19	-119	1.19	-106	1.22	.015	1.19	-.220	1.19	-.100
1.42	1.88	1.43	79	1.43	82	1.43	.044	1.43	.093	1.43	.097
1.66	-2.99	1.67	-107	1.66	- 93	1.68	.020	1.65	-.209	1.66	-.087
1.90	1.55	1.90	71	1.90	70	1.90	.044	1.90	.081	1.90	.085
2.13	-2.66	2.13	- 93	2.13	- 82	2.16	.015	2.13	-.174	2.13	-.074
2.37	1.33	2.37	61	2.37	61	2.37	.039	2.36	.058	2.36	.074
2.60	-2.22	2.61	- 79	2.59	- 71	2.63	.015	2.58	-.174	2.59	-.065
2.84	1.33	2.84	54	2.83	52	2.83	.035	2.84	.046	2.83	.066
3.06	-1.99	3.07	- 69	3.06	- 52	3.10	.015	3.06	-.139	3.06	-.057
3.29	1.11	3.31	46	3.29	48	3.31	.035	3.28	.046	3.30	.057
3.51	-1.66	3.55	- 61	3.52	- 53	3.56	.020	3.52	-.116	3.52	-.050
3.75	.99	3.77	41	3.79	43	3.77	.030	3.74	.035	3.76	.051
3.98	-1.55	4.01	- 55	3.99	- 52	4.00	.015	3.99	-.104	3.99	-.044
4.22	.99	4.24	37	4.22	43	4.23	.030	4.22	.035	4.22	.044
4.45	-1.44	4.48	48	4.44	- 39	4.49	.010	4.43	-.093	4.45	-.039
4.68	.88	4.70	- 30	4.68	34	4.69	.025	4.68	.035	4.68	.039
4.91	-1.44	4.94	46	4.91	- 35	4.94	.010	4.90	-.081	4.92	-.035
5.14	.77	5.16	- 26	5.14	30	5.15	.020	5.14	.023	5.14	.034
5.37	-1.33	5.39	40	5.37	- 34	5.40	.015	5.37	-.070	5.38	-.031
5.60	.55	5.63	- 23	5.60	26	5.63	.020	5.60	.023	5.60	.030
5.83	- .99	5.85	37	5.83	- 28	5.86	.010	5.87	-.058	5.84	-.028

Test 2-6-2

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6		Gage 7	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Accel-eration G	Time Sec	Accel-eration G	Time Sec	Displace-ment In.
.00	4.66	.00	190	.00	158	.00	.006	.00	.007	.00	.250
.25	-3.44	.26	-129	.25	-106	.27	-.020	.25	-.193	.25	-.141
.49	2.77	.49	99	.48	104	.51	.023	.48	.183	.49	.155
.72	-2.77	.72	-113	.72	- 93	.74	-.015	.72	-.162	.72	-.123
.95	2.44	.95	88	.94	91	.98	.018	.95	.162	.95	.137
1.18	-2.44	1.18	- 99	1.18	- 82	1.21	-.014	1.18	-.142	1.18	-.107
1.42	2.33	1.42	79	1.41	80	1.44	.017	1.41	.144	1.42	.121
1.65	-2.10	1.65	- 87	1.65	- 73	1.68	-.012	1.65	-.125	1.65	.092
1.88	2.10	1.88	70	1.87	71	1.90	.014	1.88	.125	1.88	.107
2.11	-1.88	2.11	- 74	2.11	- 63	2.14	-.010	2.11	-.104	2.11	-.078
2.35	1.77	2.35	62	2.35	62	2.35	.013	2.34	.111	2.35	.095
2.57	-1.55	2.58	- 64	2.57	- 56	2.59	-.009	2.57	-.093	2.57	-.068
2.80	1.77	2.81	55	2.80	54	2.81	.012	2.80	.102	2.81	.083
3.02	-1.22	3.04	- 56	3.03	- 49	3.06	-.009	3.03	-.084	3.03	-.057
3.27	1.77	3.27	50	3.27	50	3.28	.009	3.27	.090	3.28	.078
3.49	- .77	3.50	- 50	3.49	- 44	3.51	-.008	3.50	-.070	3.50	-.049
3.73	1.66	3.73	44	3.73	45	3.73	.009	3.73	.084	3.73	.070
3.96	- .77	3.96	- 43	3.96	- 39	3.97	-.007	3.96	-.063	3.96	-.043
4.19	1.55	4.19	40	4.19	41	4.20	.007	4.19	.077	4.19	.064
4.42	- .66	4.42	- 39	4.42	- 35	4.42	-.007	4.42	-.056	4.42	-.037
4.65	1.33	4.65	36	4.64	37	4.65	.007	4.65	.067	4.65	.058
4.87	- .55	4.88	- 34	4.87	- 31	4.89	-.006	4.87	-.046	4.88	-.034
5.11	1.22	5.11	32	5.10	33	5.11	.005	5.11	.063	5.11	.051
5.33	- .44	5.34	- 29	5.33	- 27	5.35	-.005	5.33	-.039	5.34	-.028
5.56	1.11	5.56	28	5.55	29	5.56	.005	5.56	.056	5.56	.046
5.79	- .22	5.80	- 25	5.79	- 23	5.80	-.004	5.79	-.035	5.79	-.024

Test 2-8-1

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-5.43	.00	103.12	.00	301	.00	.002	.00	.015
.28	.51	.29	- .56	.26	-175	.26	-.108	.27	-.081
.53	-2.25	.53	37.75	.52	156	.52	.080	.51	.051
.78	.30	.78	1.12	.76	-137	.76	-.077	.77	-.026
1.02	-1.84	1.02	30.99	1.01	122	1.00	.059	1.03	.026
1.27	.10	1.28	2.81	1.26	-109	1.26	.059	1.27	-.017
1.53	-1.53	1.53	24.79	1.51	98	1.51	.047	1.54	.026
1.78	.00	1.79	3.94	1.77	- 92	1.77	-.049	1.79	-.009
2.04	-1.43	2.04	21.97	2.03	82	2.03	.040	2.04	.021
2.29	- .20	2.30	4.50	2.28	- 80	2.28	-.043	2.29	-.004
2.54	-1.43	2.55	19.72	2.53	75	2.53	.036	2.55	.026
2.80	- .20	2.81	4.50	2.79	- 70	2.78	-.040	2.80	-.002
3.06	-1.33	3.07	16.90	3.04	64	3.04	.032	3.05	.023
3.32	- .41	3.32	5.63	3.31	- 59	3.30	-.031	3.32	-.00
3.59	-1.33	3.58	16.34	3.56	54	3.56	.026	3.57	.021
3.83	- .51	3.84	6.76	3.82	- 48	3.81	-.026	3.82	.002
4.08	-1.23	4.0	15.21	4.07	44	4.07	.020	4.08	.019
4.34	- .61	4.35	7.88	4.33	- 40	4.32	-.021	4.34	.004
4.62	-1.23	4.61	14.08	4.59	37	4.59	.018	4.61	.021

Test 2-8-2

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-4.51	.00	89.59	.00	294	.00	.00	.00	.009
.02	-4.41	.06	65.36	.06	197	.11	.061	.06	.174
.04	-2.97	.12	44.51	.15	23	.20	-.068	.13	.128
.29	1.12	.29	- 8.45	.26	-175	.26	-.119	.21	-.085
.53	-1.84	.53	29.86	.51	152	.52	.078	.53	.043
.77	.71	.78	- 7.32	.76	-140	.76	-.072	.77	-.026
1.02	-1.53	1.02	21.97	1.00	118	1.00	.063	1.03	.030
1.27	.41	1.28	- 6.19	1.25	-112	1.25	-.055	1.26	-.015
1.52	-1.43	1.52	16.90	1.51	93	1.51	.051	1.53	.026
1.78	.20	1.79	- 3.94	1.77	- 95	1.77	-.045	1.79	-.011
2.04	-1.23	2.04	14.08	2.02	78	2.03	.045	2.04	.021
2.29	.00	2.30	- 3.38	2.28	- 85	2.28	-.040	2.30	-.009
2.99	-1.23	2.55	12.39	2.53	70	2.53	.040	2.55	.021
2.80	.00	2.80	- 2.81	2.79	- 73	2.79	-.037	2.81	-.004
3.07	-1.23	3.06	10.14	3.05	59	3.05	.035	3.07	.021
3.32	- .20	3.33	- 2.81	3.31	- 67	3.31	-.031	3.33	-.002
3.58	-1.23	3.58	7.88	3.56	47	3.56	.029	3.59	.017
3.83	- .51	3.85	- 1.69	3.83	- 55	3.83	-.021	3.85	-.00
4.11	-1.23	4.00	5.63	4.08	37	4.08	.025	4.09	.017
4.35	- .71	4.37	- 1.69	4.35	- 47	4.34	-.018	4.38	.002
4.62	-1.43	4.62	4.50	4.61	30	4.62	.022	4.63	.019

Test 2-8-3

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-3.59	.00	56.57	.00	235	.00	.004	.00	.128
.01	-3.69	.02	55.89	.02	224	.07	.132	.05	.321
.15	1.84	.05	45.98	.03	201	.11	-.026	.15	-.487
.27	-2.56	.15	-27.72	.14	-188	.16	-.162	.24	.316
.40	1.43	.27	38.76	.27	168	.29	.155	.29	.316
.52	-1.94	.40	-21.41	.39	-147	.41	-.121	.39	-.371
.65	1.02	.52	28.85	.52	129	.54	.121	.52	.320
.78	-1.43	.65	-16.00	.65	-111	.67	-.087	.65	-.276
.79	-1.43	.79	19.61	.78	95	.79	.092	.78	.240
.91	.71	.90	-11.72	.91	- 84	.92	-.060	.91	-.202
1.04	-1.12	1.04	13.97	1.04	73	1.06	.072	1.04	.184
1.16	.51	1.17	- 9.46	1.17	- 69	1.19	-.045	1.17	-.166
1.30	- .82	1.30	10.59	1.30	60	1.32	.062	1.30	.154
1.43	.20	1.43	- 7.88	1.43	- 59	1.45	-.034	1.43	-.142
1.58	- .92	1.58	8.11	1.56	49	1.59	.053	1.56	.129
1.70	.10	1.70	- 6.53	1.70	- 49	1.72	-.026	1.70	-.116
1.83	- .71	1.83	5.41	1.83	37	1.85	.043	1.83	.098
1.96	.00	1.96	- 4.73	1.96	- 36	1.98	-.015	1.96	-.085
2.08	- .61	2.09	2.70	2.09	22	2.11	.032	2.09	.066
2.22	- .10	2.23	- 3.15	2.23	- 24	2.25	-.002	2.23	-.050
2.37	- .51	2.36	1.57	2.36	13	2.38	.023	2.36	.043
2.49	- .20	2.49	- 1.80	2.49	- 17	2.53	.002	2.49	-.033
2.60	- .41	2.62	.90	2.62	9	2.64	.021	2.62	.033
2.73	- .30	2.74	- 1.57	2.75	- 13	2.76	.004	2.75	-.026
2.86	- .41	2.86	.45	2.87	6	2.89	.021	2.86	.027
2.97	- .20	2.99	- 1.35	2.99	- 11	3.00	.009	2.99	-.018

Test 2-8-4

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-3.59	.00	58.37	.00	198.97	.00	.00	.00	-.043
.06	-3.59	.12	-12.39	.11	-137.61	.13	-.506	.13	-.521
.12	.82	.17	31.55	.16	119.84	.18	.430	.18	.321
.17	-1.84	.22	- 8.34	.21	- 96.33	.23	-.345	.23	-.515
.22	.61	.27	21.41	.27	82.57	.28	.292	.28	.320
.27	-1.23	.32	- 4.05	.32	- 65.94	.33	-.238	.33	-.502
.32	.41	.37	15.10	.37	56.19	.38	.200	.38	.321
.38	- .71	.43	- 1.35	.42	- 47.01	.44	-.170	.42	-.448
.43	.30	.48	11.49	.47	41.85	.49	.149	.48	.323
.48	- .61	.53	.00	.53	- 38.41	.54	-.136	.53	-.361
.53	.20	.58	9.91	.58	34.40	.59	.117	.58	.322
.59	- .51	.64	.22	.63	- 31.53	.65	-.109	.63	-.294
.63	.10	.68	8.34	.68	27.52	.70	.094	.68	.255
.69	- .41	.73	1.35	.73	- 24.65	.75	-.083	.73	-.225
.73	.00	.78	6.76	.78	20.06	.79	.064	.78	.184
.785	- .20	.83	2.02	.83	- 16.62	.84	-.057	.83	-.157
.79	.00	.88	5.41	.88	12.61	.89	.043	.87	.122
.84	.00	.93	2.70	.93	- 10.89	.94	-.036	.93	-.101
		.98	4.50	.98	8.02	.99	.026	.97	.077
		1.03	3.15	1.03	- 7.45	1.04	-.026	1.02	-.066
		1.08	4.50	1.08	5.16	1.09	.017	1.07	.052

Test 2-8-5

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-4.30	.00	65.36	.00	229.93	.00	-.026	.00	.000
.02	-4.41	.10	- 8.11	.10	-108.37	.11	-.402	.10	-1.049
.04	-3.89	.16	25.24	.15	95.75	.17	.338	.15	.856
.10	.41	.20	- 5.41	.20	- 78.55	.22	-.279	.20	- .765
.15	-1.64	.26	17.35	.25	67.08	.27	.236	.25	.613
.20	.30	.31	- 2.47	.30	- 53.90	.32	-.194	.31	- .514
.26	-1.02	.36	11.72	.36	44.15	.37	.155	.36	.404
.31	.20	.42	- .22	.41	- 36.69	.43	-.130	.41	- .345
.36	- .71	.47	9.01	.46	31.53	.48	.111	.46	.286
.41	.00	.52	.67	.52	- 27.52	.53	-.096	.52	- .267
.47	- .51	.57	7.43	.57	25.23	.58	.085	.57	.238
.52	.00	.63	.90	.62	- 23.50	.64	-.077	.62	- .219
.57	- .51	.67	6.31	.67	20.06	.69	.070	.67	.190
.62	- .10	.73	1.57	.72	- 18.34	.74	-.055	.72	- .169
.68	- .41	.77	5.41	.77	13.18	.78	.051	.77	.139
.72	- .20	.82	2.02	.82	- 12.61	.83	-.038	.82	- .118
.78	- .41	.87	4.50	.86	8.60	.88	.034	.86	.091
.80	- .30	.91	2.25	.91	- 8.60	.93	-.023	.91	- .075
.85	.00	.96	3.83	.96	5.16	.98	.021	.96	.059
.90	.00	1.01	2.93	1.01	- 5.16	1.02	-.015	1.01	- .048
		1.06	3.60	1.06	4.01	1.07	.015	1.06	.040
		1.11	2.70	1.10	- 4.01	1.12	-.009	1.11	- .032
		1.16	3.15	1.15	2.86	1.17	.015	1.16	.032
		1.21	2.70	1.20	- 4.01	1.22	-.009	1.20	- .032
		1.25	3.15	1.25	2.29	1.27	.015	1.25	.035

Test 2-8-6

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-7.80	.00	-115.48	.00	400.29	.00	.00	.00	.027
.27	1.40	.28	- 15.33	.25	-311.47	.26	-.256	.25	-.450
.50	-4.80	.50	61.32	.48	261.73	.48	.179	.47	.423
.72	.40	.72	- 11.24	.70	-260.54	.72	-.169	.69	-.328
.94	-4.00	.94	48.03	1.15	-185.93	.93	.133	.91	.323
1.16	.00	1.16	- 5.11	1.37	143.30	1.16	-.133	1.14	-.223
1.39	-3.60	1.39	35.77	1.61	-136.19	1.38	.092	1.37	.246
1.63	- .80	1.63	.00	1.85	99.48	1.62	-.096	1.60	-.132
1.87	-3.20	1.87	27.59	2.08	- 99.48	1.85	.055	1.83	.173
2.10	-1.40	2.10	3.06	2.32	66.32	2.09	-.064	2.07	-.068
2.34	-3.00	2.34	20.44	2.56	- 79.34	2.34	.032	2.32	.118
2.58	-1.60	2.59	4.08	2.81	49.74	2.58	-.041	2.56	-.032
2.81	-2.80	2.82	17.37	3.04	- 72.24	2.84	.027	2.81	.105
3.07	-2.00	3.06	4.08	3.27	37.89	3.06	-.032	3.05	-.023
3.29	-2.80	3.29	14.30	3.50	- 67.50	3.31	.032	3.28	.114
3.53	-2.00	3.52	3.06	3.74	28.42	3.53	-.023	3.51	-.023
3.76	-2.60	3.75	11.24	3.96	- 61.58	3.77	.037	3.74	.118
3.99	.00	3.99	3.06	4.19	21.31	3.99	-.018	3.96	-.014
4.20	.00	4.20	9.19	4.42	- 53.29	4.21	.032	4.19	.114

Test 2-8-7

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-5.40	.00	76.65	.00	439	.00	.133	.00	.00
.24	4.40	.25	-53.14	.22	-386	.24	-.256	.22	-.573
.46	-3.60	.46	50.07	.45	328	.46	.224	.44	.514
.69	3.40	.69	-45.99	.67	-313	.67	-.197	.66	-.446
.91	-2.40	.91	35.77	.89	260	.90	.192	.88	.410
1.13	2.80	1.14	-38.83	1.11	-251	1.12	-.156	1.10	-.337
1.36	-1.80	1.36	22.48	1.33	197	1.35	.147	1.33	.323
1.58	2.40	1.58	-31.68	1.56	-195	1.57	-.110	1.56	-.246
1.81	-1.00	1.81	12.26	1.79	144	1.80	.114	1.79	.246
2.03	1.80	2.04	-26.57	2.02	-146	2.03	-.078	2.01	-.164
2.27	- .80	2.27	4.08	2.25	98	2.26	.082	2.24	.182
2.49	1.40	2.50	-21.46	2.48	-108	2.48	-.055	2.48	-.100
2.73	- .40	2.76	.00	2.72	67	2.73	.060	2.71	.132
2.96	.80	2.98	-19.41	2.95	- 82	2.96	-.032	2.95	-.059
3.20	- .20	3.21	- 5.11	3.19	48	3.20	.050	3.19	.114
3.45	.60	3.45	-17.37	3.43	- 73	3.44	-.023	3.42	-.045
3.67	- .40	3.69	- 6.13	3.66	41	3.68	.046	3.67	.109
3.90	.20	3.93	-17.37	3.90	- 68	3.90	-.023	3.90	-.041
4.13	- .40	4.14	- 7.15	4.12	35	4.14	.041	4.13	.109
4.35	.40	4.37	-16.35	4.35	- 62	4.36	-.018	4.35	-.027
4.60	- .40	4.59	- 9.19	4.57	28	4.59	.046	4.58	.109

Test 2-10-1

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-27.30	.00	220.23	.00	362	.00	.018	.00	.019
.02	-27.30	.02	216.96	.02	343	.05	-.327	.02	.967
.04	-24.45	.04	147.18	.04	180	.10	.165	.04	.409
.05	-25.27	.05	165.71	.05	262	.15	-.022	.07	.153
.07	-23.84	.07	124.28	.08	176	.20	-.008	.10	.079
.09	-23.84	.08	127.55	.13	4	.25	-.018	.11	.126
.28	-17.11	.29	- 37.06	.27	-228	.27	-.037	.27	-.395
.54	-23.23	.53	86.13	.52	201	.50	.065	.52	.349
.78	-17.73	.79	- 30.52	.77	-188	.75	-.039	.77	-.330
1.03	-22.21	1.02	62.14	1.02	161	1.03	.049	1.02	.288
1.28	-18.34	1.29	- 23.98	1.28	-148	1.28	-.029	1.27	-.265
1.55	-21.80	1.55	44.70	1.54	126	1.51	.043	1.53	.233
1.82	-18.54	1.83	- 17.44	1.81	-113	1.80	-.025	1.79	-.209
2.10	-21.19	2.08	29.43	2.07	95	2.05	.039	2.07	.181
2.35	-19.36	2.36	- 13.08	2.35	- 83	2.35	-.018	2.35	-.149
2.64	-20.78	2.63	19.62	2.62	66	2.62	.027	2.62	.126
2.93	-19.56	2.92	- 7.63	2.90	- 55	2.89	-.012	2.90	-.102
3.20	-20.58	3.18	9.81	3.17	39	3.15	.024	3.18	.079
3.50	-19.97	3.49	- 4.36	3.48	- 34	3.45	-.004	3.46	-.056
3.88	-20.38	3.87	3.27	3.84	21	3.85	.008	3.83	.037

Test 2-10-2

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-8.15	.00	163.53	.00	293	.00	.004	.00	.014
.25	.81	.26	- 54.51	.24	-199	.25	-.049	.24	-.367
.47	-4.48	.47	62.14	.45	180	.47	.035	.45	.349
.68	.81	.68	- 54.51	.66	-173	.67	-.029	.66	-.312
.88	-3.87	.88	49.06	.87	159	.89	.029	.87	.312
1.09	.61	1.10	- 51.24	1.08	-153	1.09	-.025	1.07	-.279
1.31	-3.66	1.31	40.33	1.29	142	1.31	.029	1.28	.274
1.51	.20	1.51	- 46.88	1.50	-137	1.51	-.022	1.49	-.246
1.73	-3.26	1.73	32.70	1.72	129	1.73	.027	1.71	.251
1.94	.00	1.94	- 43.61	1.93	-122	1.94	-.018	1.92	-.219
2.15	-3.26	2.15	26.16	2.14	114	2.14	.024	2.12	.228
2.35	- .20	2.36	- 40.33	2.35	-109	2.35	-.015	2.33	-.186
2.57	-3.26	2.56	20.71	2.55	102	2.56	.024	2.55	.205
2.77	- .61	2.78	- 38.15	2.76	- 99	2.77	-.012	2.75	-.158
2.99	-3.26	2.98	11.99	2.97	87	2.98	.024	2.96	.195
3.14	-1.01	3.16	- 21.80	3.14	- 91	3.15	-.008	3.13	-.135
3.37	-3.26	3.37	25.07	3.35	77	3.37	.024	3.34	.181
3.56	-1.42	3.59	- 20.71	3.56	- 84	3.56	-.00	3.55	-.107
3.78	-3.46	3.79	19.62	3.77	66	3.78	.027	3.75	.172

Test 2-10-3

Gage 2		Gage 3		Gage 4		Gage 5		Gage 6	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Acceleration G	Time Sec	Acceleration G
.00	-2.03	.00	130.83	.00	181	.00	.004	.00	.019
.23	3.87	.25	- 22.89	.23	-126	.24	-.039	.23	-.279
.46	.00	.45	51.24	.44	113	.46	.025	.44	.246
.66	3.46	.65	- 23.98	.65	-112	.66	-.020	.64	-.233
.87	.00	.86	43.61	.85	103	.87	.024	.85	.219
1.07	3.46	1.08	- 23.98	1.06	-100	1.08	-.016	1.06	-.219
1.29	.20	1.27	34.88	1.27	90	1.29	.022	1.27	.191
1.49	2.85	1.48	- 23.98	1.48	- 91	1.49	-.012	1.47	-.195
1.72	.20	1.70	29.43	1.69	83	1.71	.020	1.69	.172
1.91	2.64	1.92	- 23.98	1.90	- 84	1.91	-.010	1.90	-.172
2.13	.20	2.13	22.89	2.11	73	2.11	.020	2.11	.163
2.33	2.24	2.34	- 21.80	2.32	- 76	2.33	-.008	2.31	-.144
2.52	.00	2.51	20.71	2.50	64	2.51	.020	2.52	.158
2.71	2.03	2.71	- 20.71	2.71	- 69	2.70	-.006	2.73	-.126
2.93	.00	2.92	16.35	2.91	56	2.92	.018	2.94	.139
3.12	1.83	3.13	- 18.53	3.11	- 63	3.11	-.002	3.10	-.112
3.33	.00	3.33	13.08	3.32	50	3.33	.018	3.31	.130
3.52	1.42	3.53	- 18.53	3.53	- 59	3.54	-.00	3.52	-.098
3.75	.00	3.75	10.90	3.74	46	3.75	.020	3.73	.121