SOIL PARAMETERS REQUIRED TO SIMULATE THE DYNAMIC LATERAL RESPONSE OF MODEL PILES IN SAND

Prepared by

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ABSTRACT

Three instrumented model piles of varying diameters and embedded lengths were driven into sand and field tested laterally under free vibration conditions. The dynamic response of each model pile was measured in the field. Bending moment and acceleration versus time data were obtained.

An analytical computer solution was used to predict the response of the model piles. A modified Voight-Kelvin rheological model was utilized in the analytical computer solution to model the nonlinear load-displacement characteristics of the soil. The predicted response of the model piles was correlated with the measured field data. Using these correlations and laboratory data obtained from tests on soil samples taken at the test site, the soil parameters required to simulate the dynamic field response of the model piles were evaluated.

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 covered the work accomplished. During the second year (1969-70) of this study, soil parameters required to simulate the dynamic lateral response of model piles in clay were evaluated. This work was partially funded by institutional grants GH-59 and GH-101, and research report TAMU-SG-71-218 covering the tests in clay is being published.

As part of the continuing study, support was received for the third year (1970-71) under institutional grant GH-101. This support was used to investigate the dynamic response of laterally-loaded model piles in sand. This report presents the results of the model pile study conducted in sands.

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

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INTRODUCTION

Nature of the Problem. - In recent years the petroleum industry has searched for hydrocarbons on the continental shelves along the coasts of the United States and other countries. Large exploration and production structures are being built in deeper water to continue to satisfy the demand for oil and its products. As more severe sea conditions are encountered in deeper water, this increase in offshore activity has been accompanied by an increase in design complexities and uncertainties. As a result, there is a lack of confidence in the ability of these structures to resist the dynamic lateral loads frequently imposed by wind, waves, and ice.

Present Status of the Question. - Considerable work has been done on the response of laterally loaded piles with static loading. Davisson (3)* has presented a comprehensive survey of the research done on laterally loaded piles through 1960. As early as 1948, Palmer and Thompson (11) developed a numerical computer solution using finite difference techniques to analyze a laterally loaded pile as a beam on an elastic foundation. In 1958, McClelland and Focht (10) developed nonlinear load vs. displacement relationships

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*Numbers in parentheses refer to the references listed in Appendix I.

for a soil surrounding a laterally loaded pile. Matlock and Reese (8) in 1960 developed a solution for the laterally loaded pile problem using finite difference techniques to account for the non-linear soil characteristics.

In 1964, Tucker (19) studied the dynamic problem and developed an analytical solution which utilized a finite element representation of the pile and an elastic description of the soil. However, due to the uncertainties involved with the soil-pile interaction under dynamic loading conditions and the virtual non-existence of published dynamic field test data, treatment of the dynamic problem in the literature is sparse. To the writer's knowledge, the only published full-scale data is contained in a report by Hayashi (5). Unfortunately, much of the text of this extensive study is currently in Japanese.

In 1970, an analytical solution for the response of an offshore pile subjected to dynamic lateral loads was developed by Ross (15). The nonlinear properties of the soil in the analysis by Ross are represented by a modified Voight-Kelvin rheological model. This rheological model was originally suggested by Smith (17) and has been used successfully by Samson, Hirsch, and Lowery (16) in pile driving analysis. The load vs. deformation characteristics of the soil model used by Ross, as shown schematically in Fig. 1, are governed by the equation

$$P_{dynamic} = P_{static} \, (1 + JV^N) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; (1)$$

where P is the dynamic or static load,

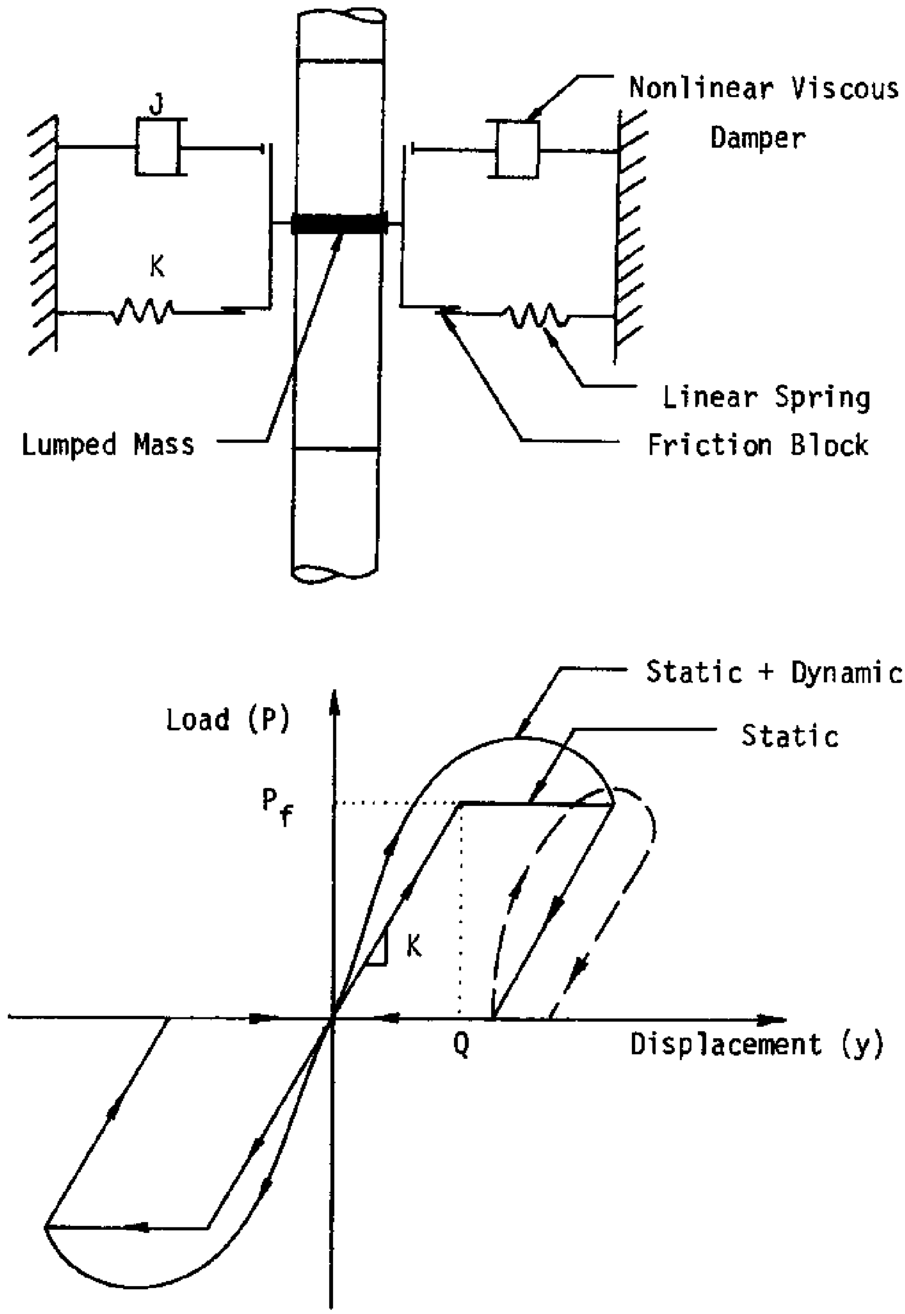


FIGURE 1.- SOIL RHEOLOGICAL MODEL AND LOAD-DISPLACEMENT CHARACTERISTICS USED BY ROSS

J is the soil damping factor,

V is the velocity of the pile,

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

In order to successfully utilize the analytical solution developed by Ross, the following soil parameters must be evaluated for use with the rheological model:

(1) K - the linear soil spring

(2) Q - the maximum elastic soil displacement, or "quake"

(3) J - the soil damping factor.

The linear soil spring, K, for a given pile segment can be obtained by multiplying the coefficient of horizontal subgrade reaction, k_h, by the projected area of the pile segment. Terzaghi (18) recommended lateral load tests on the embedded length of a pile as the best method for determining k_h.

Terzaghi related k_h to pile width D, depth z, and constant of horizontal subgrade reaction n_h, as follows:

$$k_h = n_h \frac{z}{D} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (2)$$

Values of n_h recommended by Terzaghi for a pile 1 ft wide embedded in moist sand for use in the static load case vary from 7 to 56 tons per cu. ft for cohesionless soils of low to high relative density, respectively. Davisson and Salley (4) verified that a triangular variation of subgrade reaction with respect to depth as incorporated in Eq. (2) is reasonable for small model piles in sand.

The soil quake, Q, is the maximum elastic deformation of the soil, and can be approximated from a laboratory triaxial stress vs. strain curve. As indicated in Fig. 1, the value of Q limits the amount of static load that the soil can exert on the pile. The static load, P_{static}, is the product of the linear soil spring, K, and the lateral displacement, y, and is expressed in equation form as:

$$P_{static} = Ky \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

However, when the displacement exceeds the soil quake, Q, Eq. (3) becomes invalid since it does not account for plastic behavior of the soil. A method of considering plastic soil behavior by modifying Eq. (3) is discussed in detail in a later section.

The soil damping factor, J, has been investigated in the laboratory by Coyle and Gibson (2). They developed a relationship between J and ϕ' for sands, where ϕ' is the effective angle of internal shearing resistance.

Objectives. - The objectives of this investigation are:

1. To obtain dynamic field test data utilizing free-vibration tests on laterally loaded, instrumented model piles driven into sand.
2. To compare the measured dynamic response of the model test piles with the response predicted by the analytical solution developed by Ross (15).
3. To determine the soil parameters necessary to achieve agreement between the measured and predicted

dynamic response of the model test piles.

4. To suggest laboratory methods for use in evaluating these soil parameters.

FIELD TESTING PROGRAM

Test Site. - The model piles were tested in a borrow area along State Highway 30 approximately 4.8 miles east of the intersection of State Highway 30 and Farm to Market road 158 in Brazos County.

The soil at this site can visually be classified as a gray, fine sand, uniformly graded, with a trace of silt. It is overlain by a thin layer of tan clayey sand which was removed prior to driving the model piles. Within the 10 ft depth that the model piles were driven, the sand was remarkably uniform.

The results of field and laboratory tests performed on this sand are included in Appendix III. Some of these tests results were obtained as a result of previous work carried out at this test site by Ivey and Dunlap (6).

Model Pile Properties and Instrumentation. - The model piles used in this investigation were constructed and used previously in an investigation by Brown (1). The writer assisted Brown in the instrumentation of the model piles, which were constructed of standard steel pipe. Because standard steel pipe is rolled, it has a varying wall thickness. For this reason, average measurements of the inside and outside diameters were used in calculating the necessary structural properties of the pipe, which are presented in Table 1. In all following discussions referring to a given model pile, the nominal diameter of the pipe will be used for identification.

TABLE 1. - MODEL PILE PROPERTIES

Nominal Diameter (in.)	Average Outside Diameter (in.)	Average Inside Diameter (in.)	Average Cross-Sectional Area ($in.^2$)	Average Moment of Inertia ($in.^4$)
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 preliminary test was made on the 3-in. pile to compare its actual stiffness with the calculated stiffness. For a given load, the measured and calculated deflections agreed within 10 percent.

Each test pile was instrumented with four full bridges of strain gages. At each bridge, four strain gages were used, two on each side of the pipe on the axis of bending. To install the strain gages, the pipe was cut into segments and carefully welded back together, making sure that the strain gages were aligned on a common axis of bending. Although the welding added to the non-uniformity of the pipe, it was the only practical method for installing the strain gage bridges.

The strain gage bridges were placed so that after each model pile was driven to its required depth, the bridges would be located at depths below the ground surface of 6 in., 2 ft, 4 ft, and 6 ft, as depicted in Fig. 2.

Two accelerometers were also mounted on the model piles, one near the ground surface, and the other at the top of the model pile.

A Honeywell 1508 Visicorder recorded the strain and acceleration vs. time data on light-sensitive paper. Strain vs. time data was converted to moment vs. time in the data reduction phase.

Test Series. - A total of ten tests were made on model piles of three different diameters. All of the data for these tests are tabulated in Appendix IV.

The entire test series is outlined in Table 2. To facilitate the referral to a specific test, each test is assigned a three-digit identification number. The first digit indicates the nominal pile diameter, the second is the embedded length of the pile, and the last is the test number at that specific depth.

For all tests, the strain gage bridges were located at constant depths below the groundline as indicated previously in Fig. 2.

To disclose the effects that different embedded lengths have on the model pile response, the 2-in. pile was tested at three different depths. Also at each depth, two different weights were attached at the top to reveal effects due to a change in frequency.

Test Procedure. - To best simulate actual driving conditions, each model pile was driven with a drop-hammer device into the natural deposit of fine sand. The weight used to drive each model

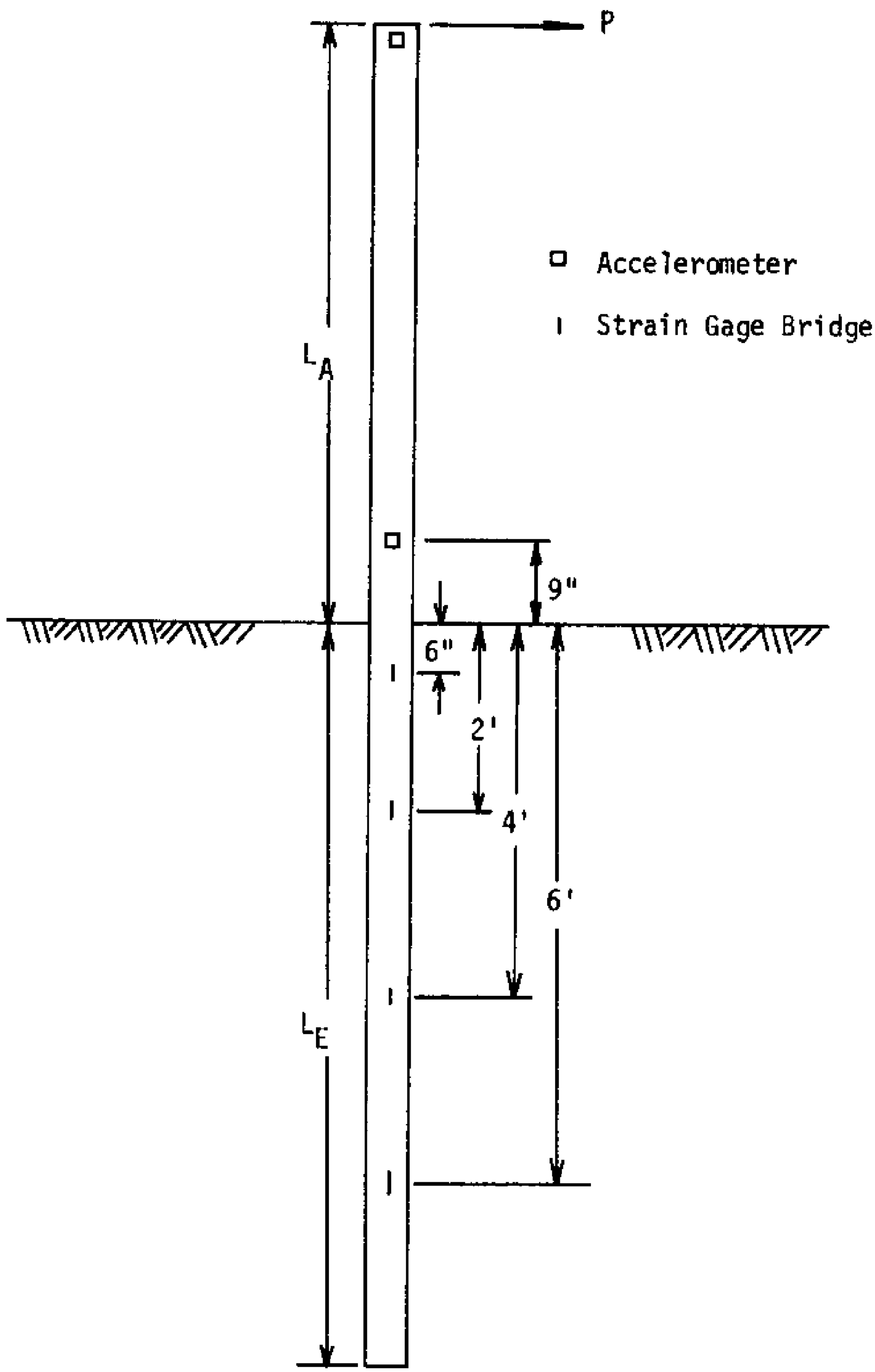


FIGURE 2.- PILE ORIENTATION AND INSTRUMENTATION LOCATION

TABLE 2. - TEST SERIES

Test Identification Number	Nominal Pile Diameter in in.	Embedded Length in ft	Length Above Groundline in ft	Top Weight in lbs	Top Static Lateral Load in lbs
3-8-1	3	8	7.4	67	63
3-8-2	3	8	7.4	67	93
2-10-1	2	10	7.4	14	9.5
2-10-2	2	10	7.4	63	14.5
2-8-1	2	8	7.4	14	19.1
2-8-2	2	8	7.4	63	38.2
2-6-1	2	6	7.4	14	21.4
2-6-2	2	6	7.4	63	28.9
1-8-1	1.25	8	7.4	13	19.9
1-8-2	1.25	8	7.4	13	23.8

pile weighed approximately 500 lbs, with the height of drop ranging from approximately 6 in. to 3 ft, depending on the length of undriven pipe as driving proceeded. Plywood boards were used as cushions during driving to minimize the possibility of buckling the top of the pipe. After the embedded portion of pipe was driven to the proper depth, the top section was added using a threaded connection to form the complete model pile. A weight holder was then mounted at the top of the pile which could contain 50 lbs. The holder for the 1.25-in. and 2-in. pile weighed 13 lbs, while the holder for the 3-in. pile weighed 17 lbs.

Each model pile was loaded horizontally by a wire attached at the top of the pile. This bent the pile into the shape of its fundamental mode of vibration, eliminating higher frequencies that would have been introduced into the system upon release if the pile had been loaded at any other point on the vertical axis. The static load on each pile was measured by a load cell, accurate to within ± 0.1 lb. Just prior to the release of the load, lateral deflections at four points along the vertical axis of the protruding section of each pile were measured by means of a transit and rulers mounted on the pipe axis.

Care was taken when applying load to the 2-in. pile and 1.25-in. pile so that each pile was displaced only the amount required to obtain a measurable deflection and significant moments at the strain bridges. This was done to prevent a soil failure around the model pile due to excessive displacement of the pile. When the 3-in. pile was tested, this precaution was not taken, since it was believed then that the main difficulty would be in obtaining sufficient deflection because of the considerable stiffness and rigidity of the pile-soil system. However, this was not the case, and the results of the 3-in. pile test are discussed in this light in a subsequent section.

The release mechanism was critical to the success of this investigation. If the load on the pile was released too slowly, the recorded initial load would be greater than the actual load operating on the pile during release. During the time span of release, the pile would not respond freely of its own accord, but its deflected shape would be relaxed by the release mechanism. A resulting loss of energy could occur between the initial moment prior to release and the moment of the first peak on the moment vs. time curve. In this investigation, the lateral load was released by snipping the restraining wire with sharp wire cutters. Generally, it is believed that this method was satisfactory. The principal inadequacy of this method was that excessive vibrations were induced into the system at the instant of release, likely due to the separation of the individual strands of wire. These excessive vibrations, which are in the form of higher frequency distortions, occurred mainly in the acceleration vs. time data. This data was not utilized in the evaluation of the soil parameters but is nevertheless presented in Appendix IV.

SOIL PARAMETERS FOR THE MODIFIED VOIGHT-KELVIN MODEL

General. - In order to utilize the analytical solution developed by Ross, soil parameters for use with the rheological soil model must be evaluated. These parameters are: (1) the linear soil spring, K, (2) the maximum elastic soil movement or quake, Q, and (3) the soil damping factor, J. It is very desirable to be able to determine these soil parameters by means other than instrumented pile tests.

Linear Soil Spring, K. - The Winkler analogy of the soil as a medium represented by a series of closely-spaced springs is frequently used in the analysis of piles subjected to static lateral loading. Poulos (13) recently suggested, however, that this assumption is unsatisfactory, since the continuity of the soil mass is not taken into account. Reese and Matlock (14) suggested further that the soil reaction may be a function of the pile properties, the stress vs. strain characteristics of the soil, unit weight of the soil, depth of overburden, pile deflection, rate of loading, and number of cycles of loading, among other things. McClelland and Focht (9) state that there is no unique value of soil modulus for a particular soil, since it varies with depth and pile deflection. Nevertheless, since the spring analogy is mathematically convenient and practical when an electronic computer is available to solve the resulting complicated problem of pile-soil interaction, it remains desirable to utilize the soil modulus concept.

As stated previously, the linear soil spring, K, for a given pile segment can be related to the coefficient of horizontal subgrade reaction, k_h, as follows:

$$K = (k_h)\ L\ D \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (4)$$

where L is the length of the pile segment,

D is the diameter of the pile.

In this investigation, k_h was evaluated using laboratory triaxial test data. Each value of k_h was taken as the tangent slope of a deviator stress vs. sample deformation curve. As noted previously, these curves and other soil test information are given in Appendix III. During testing, each sample was confined by a pressure equal to the calculated overburden pressure at the depth the sample was taken, using the average unit weight of the sand.

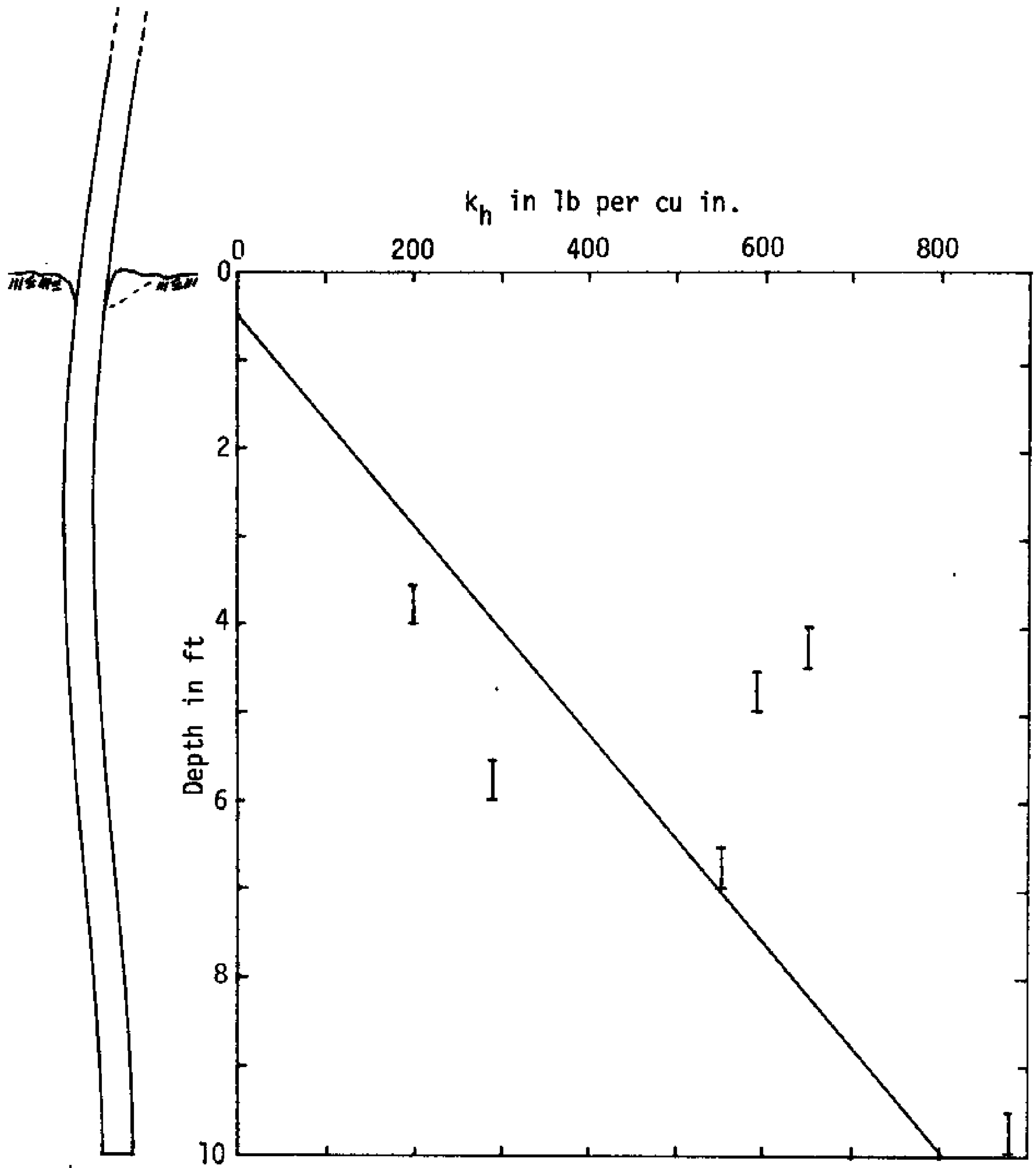
By testing samples from various depths, a distribution of k_h with depth was obtained. It should be noted that each k_h value, as evaluated in the above manner, is a soil property independent of the pile diameter but dependent on depth. This of course does not account for any coincidental relationship between the pile diameter and the sample size. The sample size could very well have a significant effect, as Terzaghi (18) emphasized that the value of k_h depends to some extent on the size of the loaded area.

Hopefully, the effect of the size of the loaded area is accounted for when the distribution of K, the linear soil spring, is established from the k_h distribution by using Eq. 4. In summary, K depends on the elastic properties of the soil, depth, and the loaded area of the pile segment.

The distribution with depth of the coefficient of horizontal subgrade reaction, k_h, as shown in Fig. 3, was used throughout this study for predicting the response of the model piles, except where specifically indicated otherwise in the next section. Instead of increasing from the groundline, the distribution of k_h in Fig. 3 increases linearly from a point 6 in. below the groundline. This neglects any effect that the top 6 in. of soil have on pile response. This assumption is reasonable because as Peck, Davisson, and Hanson (12) suggested, it appears that in the soil near the surface around the pile, the value of soil resistance must decrease markedly with increasing deflection. It is possible that the value of soil resistance could even become zero near the surface it the soil was pushed permanently away from the pile by repeated loading. Furthermore, it was noted that during early driving operations the pile wallowed in the hole until it was driven to a state of slight firmness, causing a distinct gap to be visible between the pile and the surrounding soil. From field observations, this gap is not believed to have extended below about 6 in.

Table 3 relates depths, node location, k_h, and K, and should be helpful in clarifying pile configuration.

Thus far, it has been shown that K, the linear soil spring, accounts for effects on the model pile response due to size of the



a. Assumed Soil Failure

b. k_h vs. Depth

FIGURE 3. - DISTRIBUTION WITH DEPTH OF THE COEFFICIENT OF HORIZONTAL SUBGRADE REACTION

TABLE 3. - DISTRIBUTION OF LINEAR SOIL SPRING WITH DEPTH AND NODE LOCATION

Pile	L_E	Node Location	Depth Below Groundline in ft	k_h lb/in.^{3}	K lb/in.
3 in. Pile	L_E = 8 ft	Tip	8	635	26,700
		Bridge 1	6	465	39,000
		Bridge 2	4	295	24,800
		Bridge 3	2	125	10,500
		Bridge 4	1/2	0	0
		G. L.	0	0	0
2 in. Pile	L_E = 6 ft	Tip Bridge 1	6	469	13,300
		Bridge 2	4	295	16,900
		Bridge 3	2	125	7,150
		Bridge 4	1/2	0	0
		G. L.	0	0	0
	L_E = 8 ft	Tip	8	635	18,100
		Bridge 1	6	465	26,600
		Bridge 2	4	295	10,900
		Bridge 3	2	125	7,150
		Bridge 4	1/2	0	0
		G. L.	0	0	0
	L_E = 10 ft	Tip	10	800	68,500
		Bridge 1	6	465	26,600
		Bridge 2	4	295	16,900
		Bridge 3	2	125	7,150
		Bridge 4	1/2	0	0
		G. L.	0	0	0
1.25 in. Pile	L_E = 8 ft	Tip	8	635	12,700
		Bridge 1	6	465	18,600
		Bridge 2	4	295	11,800
		Bridge 3	2	125	5,000
		Bridge 4	1/2	0	0
		G. L.	0	0	0

loaded area of the pile segment, depth, and the elastic properties of the soil. No statement has been made that K accounts for pile deflection. However, it should be noted that the soil modulus concept is a means used to obtain a certain load distribution with depth that the soil exerts on the pile for a given pile deflection curve. The force the soil can exert is a function of pile displacement as given previously by Eq. 3. Therefore, the effect due to deflection on pile response is considered.

The simple relation given in Eq. 3, however, has some limitations, since it does not account for plastic behavior of the soil. As stated by Davisson (3), a plastic zone of soil resistance occurs in the soil near the surface around laterally loaded piles. At some depth below the ground surface, there is a transition from plastic to elastic soil behavior. To account for this combination of elastic and plastic soil behavior, a value of Q, the soil "quake", must be determined.

Soil Quake, Q. - By definition, the soil quake is the maximum elastic soil deformation that occurs in the ground surrounding the pile. By utilizing a Q-value, the nonlinear elasto-plastic behavior of the soil can be simulated (see Fig. 1) by a linearly-increasing load vs. deformation curve, until the deformation reaches the value of Q. When this happens, the curve breaks to a zero slope, and any further increase in deformation results in no further increase of resistance being exerted by the soil.

Although Q may depend somewhat upon the properties of the soil, it seems reasonable that Q is primarily a function of the pile diameter. Consider that for a given embedded length, a large pile will influence a larger volume of surrounding soil than will a smaller pile. If the values of the respective strains corresponding with failure of the soil are identical in each case, then the values of the deformations at failure will not be equal. The deformation at failure in the soil surrounding the large pile will have a greater value because of the larger volume of soil involved. Thus it seems logical that the value of Q should increase with increasing pile diameter.

Brown (1) has shown that a Q of approximately one percent of the pile diameter is acceptable for piles in clay. This relationship was adopted for this investigation, since it generally produced good agreement between the predicted and measured model pile response. However, since the effects of the top 6 in. of soil were, for the most part, neglected in this study, values of Q were found not to be as critical as Brown (1) found them to be in his investigation in clay. Values of Q used herein were 0.035 in., 0.025 in., and 0.020 in. for the 3-in., 2-in., and 1.25-in. pile, respectively.

It will be appropriate here to reintroduce Eq. 3, which ties together several previously discussed relationships:

$$P_{static} = Ky \quad (\text{for } y < Q) \quad . \; . \; . \; . \; . \; . \; (3a)$$

$$P_{static} = KQ \quad (\text{for } y \geq Q) \quad . \; . \; . \; . \; . \; . \; (3b)$$

The consequences of Eq. 3 are displayed in Fig. 4. It should be

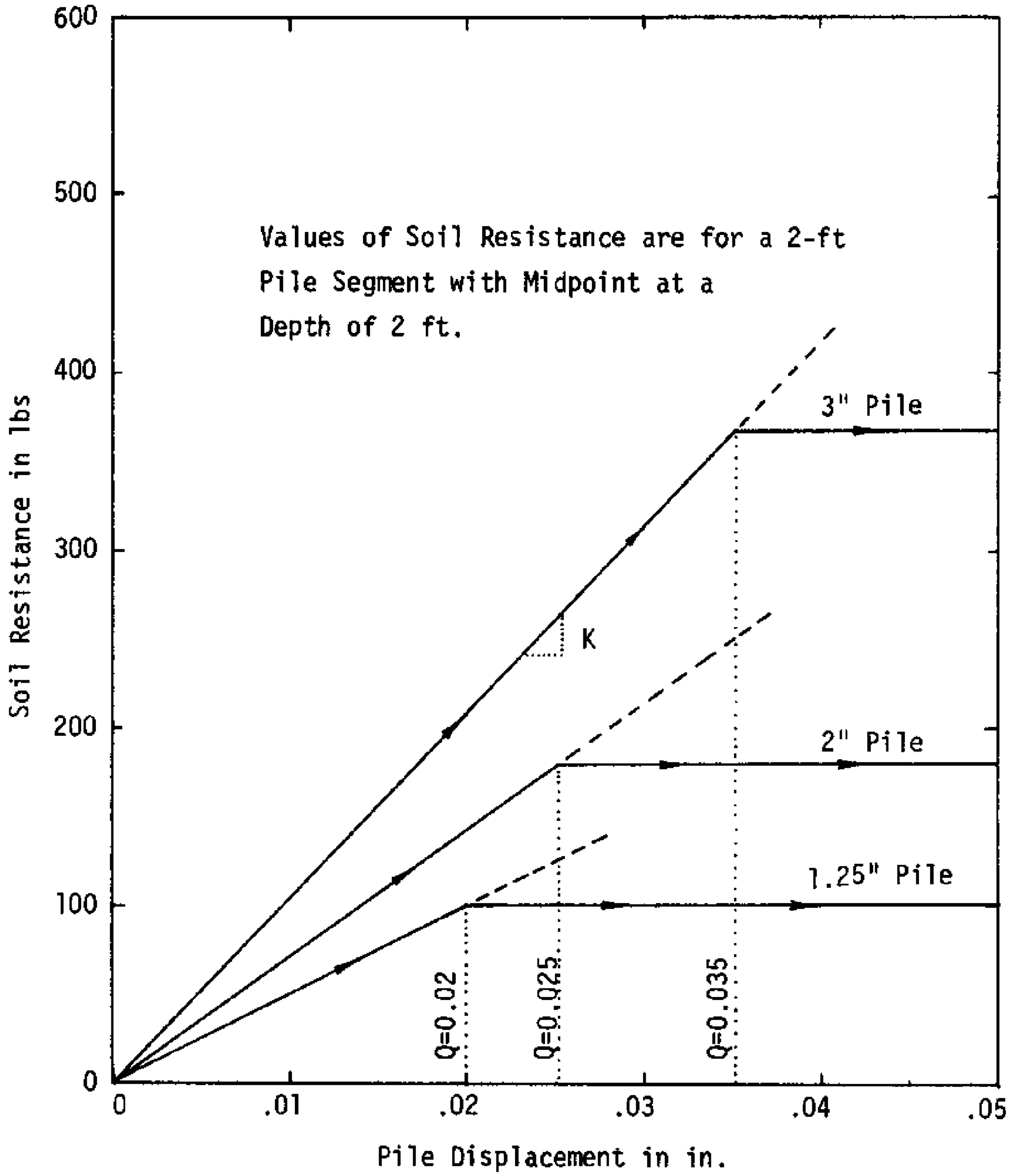


FIGURE 4.- SOIL RESISTANCE VERSUS PILE DISPLACEMENT CHARACTERISTICS

noted that the magnitude of Q controls the load (and deflection) at which soil failure occurs.

Soil Damping Factor, J. - The soil damping factor, J, has been investigated by Coyle and Gibson (2). They related it to the effective angle of shearing resistance, ϕ', in sands. Since the value of J varied with the velocity of loading as shown in Eq. 1, they found that by raising the velocity to the power N = 0.20 (for sands), J remained relatively constant.

In this study, values of N = 0.20 and J = 0.6 were used to simulate the field response of the model piles. The J-value was obtained from the relationship suggested by Coyle and Gibson by using the average value of ϕ' for the sand at the test site.

COMPARISON OF FIELD AND PREDICTED PILE RESPONSE

General. - The computer program developed by Ross (15) for the dynamic response of a laterally loaded offshore pile was used to predict model pile response in this investigation. The program was run on the IBM 360/65 facilities of the Data Processing Center at Texas A&M University.

In addition to the input parameters, K, Q, J, and N, a value of structural damping is required to utilize the analytical solution developed by Ross. A preliminary test disclosed that structural damping was very small compared to the viscous damping caused by the soil, and, for the model piles tested, could be neglected.

To begin execution of the Ross dynamic program, the deflected shape of the pile at the time of release (static deflection curve) must be read in as the initial condition. A finite element static computer program was used to determine this deflected shape by applying a known force (see Table 2) to the top of the simulated model pile. Values of the soil springs (see Table 3) and soil quake (see Fig. 4) were also used in obtaining the initial deflected shape.

Herein, the word "field" refers to the actual or observed data recorded during testing operations, while "predicted" refers to values calculated by the dynamic program.

As discussed previously, bending moments with respect to time were determined from strain vs. time data at four points below the groundline. However, in all tests the moments at bridge 1, depth 6 ft, were very small and are given only in Appendix IV. Bridge 2, 3, and 4 refer to measurements made at depths of 4 ft, 2 ft, and 6 in., respectively.

<u>3-in. Pile Tests</u>. - The 3-in. pile was tested twice at an embedded depth of 8 ft. Both tests were essentially the same, except that in the second test a higher initial static load was applied to induce the pile response, causing correspondingly larger deflections and moments. Thus, since both tests are essentially the same and support the same conclusions, only Test 3-8-1 is presented in Figs. 5 thru 7, although field data for both tests is given in Appendix IV.

Even though the entire sinusoidal-shaped curve was recorded (and predicted), only the moment peaks were plotted. This allows the frequency of vibration, magnitude of the moments, and the amount of damping to be easily compared.

The measured frequency of vibration in Test 3-8-1 was 2.86 cps as compared to the predicted frequency of 3.16 cps. Thus, a longer effective length of vibration (i.e., a deeper point of fixity) existed in the field test than was accounted for in the predicted response. The deeper point of fixity was probably due to excessive deflections at significant depths, which tended to displace the moment distribution pattern downward, as evidenced by the high

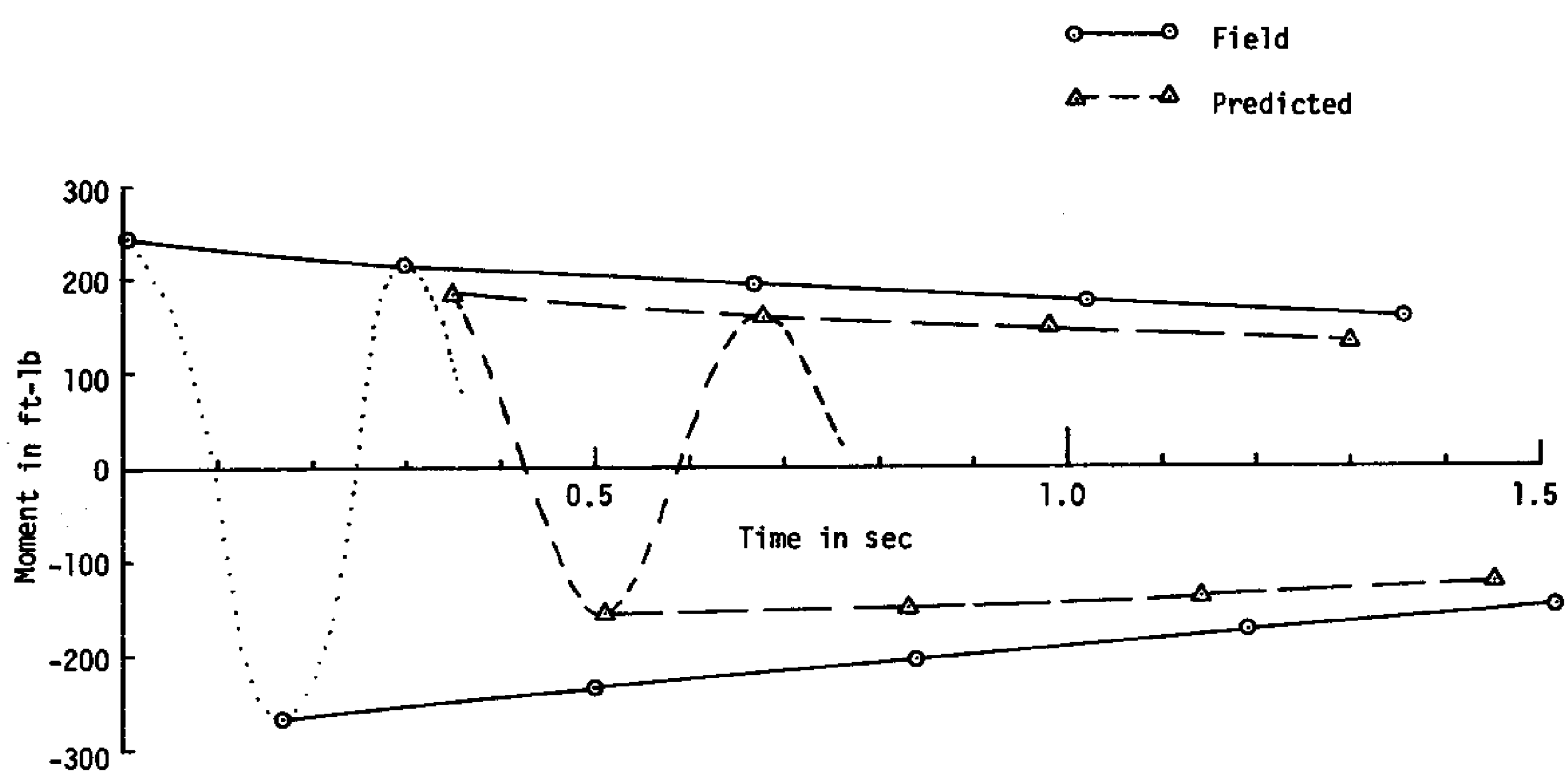


FIGURE 5.- BENDING MOMENT VERSUS TIME FOR TEST 3-8-1 AT BRIDGE 2

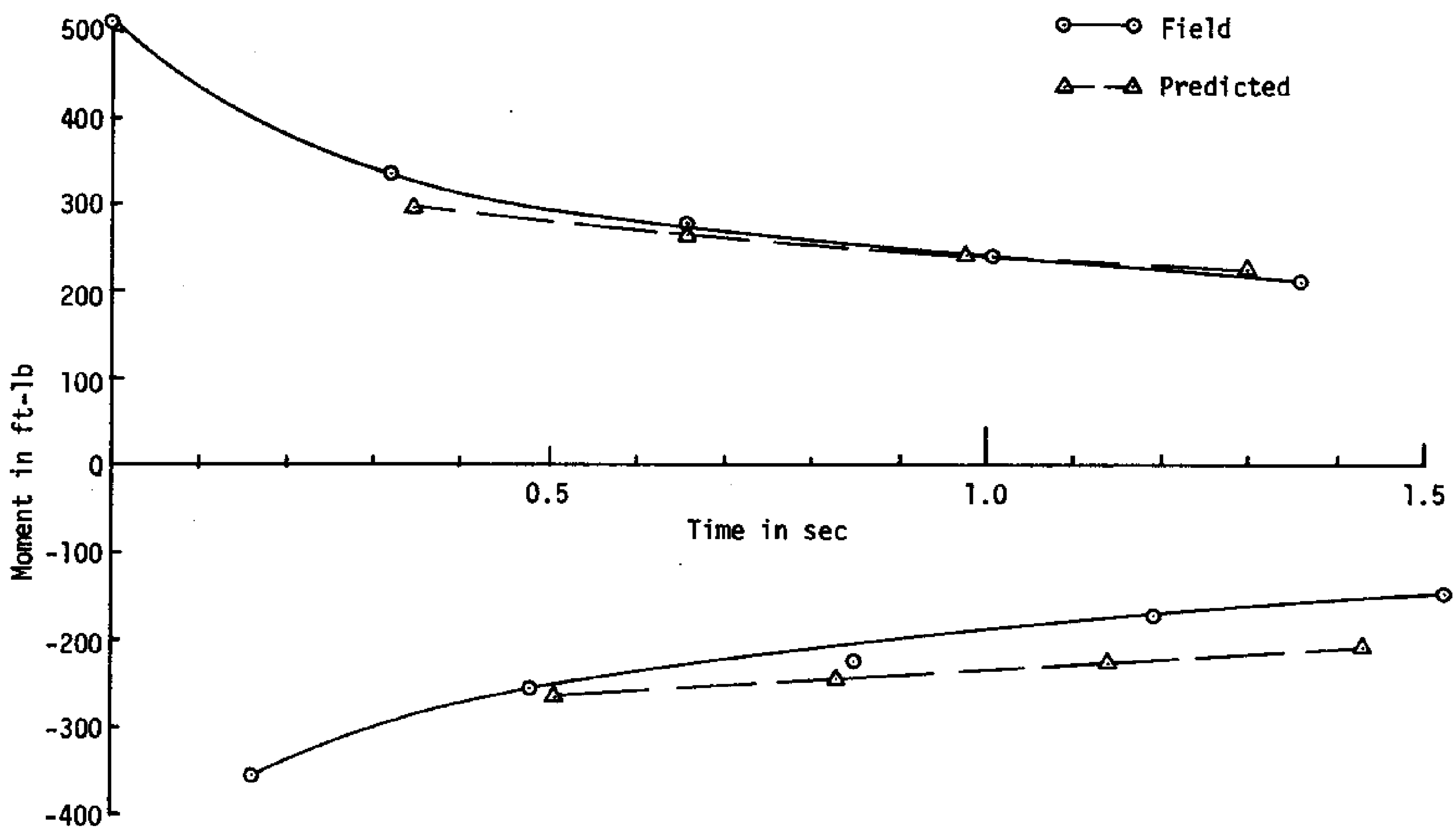


FIGURE 6.- BENDING MOMENT VERSUS TIME FOR TEST 3-8-1 AT BRIDGE 3

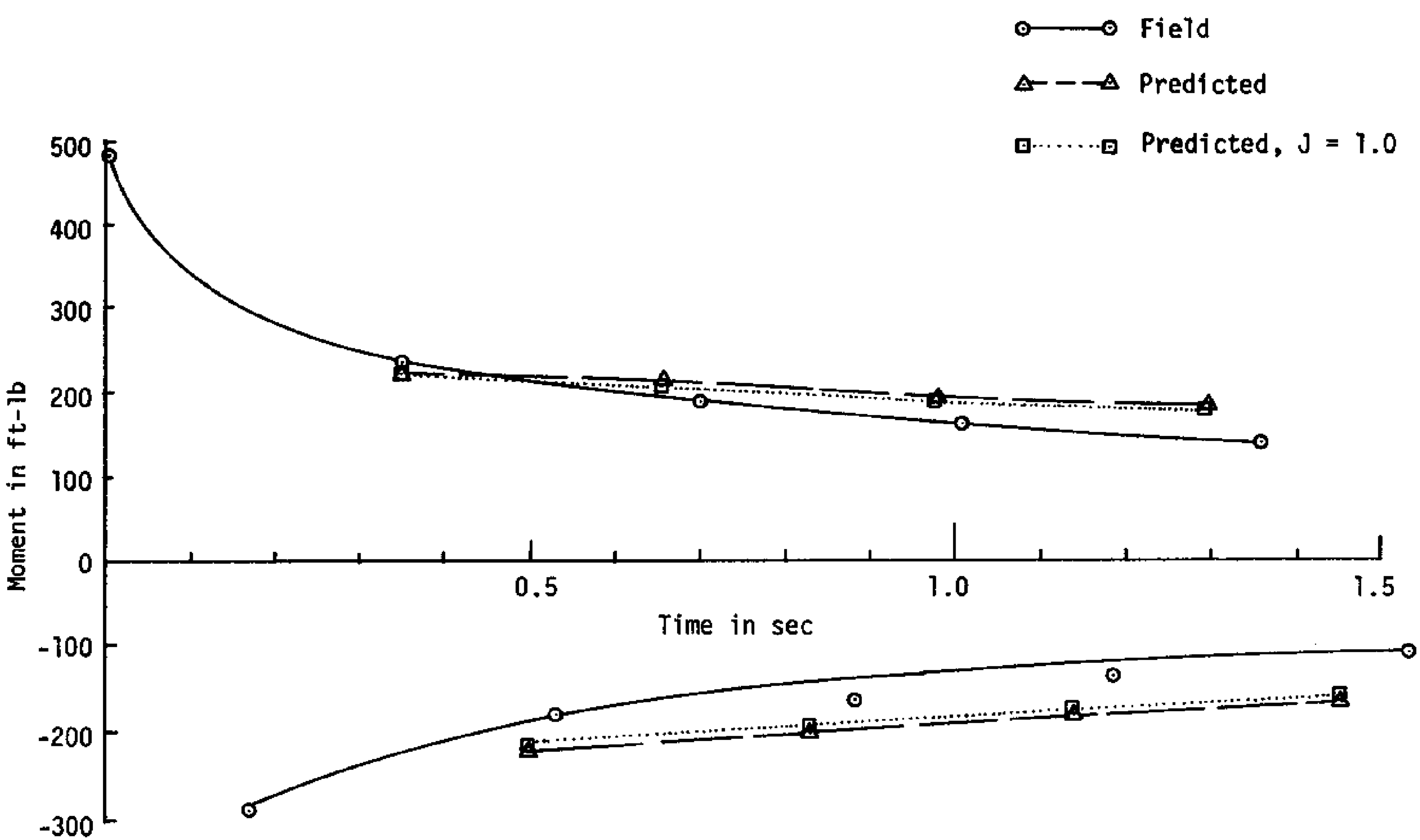


FIGURE 7.- BENDING MOMENT VERSUS TIME FOR TEST 3-8-1 AT BRIDGE 4

moments at bridge 2. (See Fig. 5)

The greatest problem encountered in predicting the response of the 3-in. pile was in simulating the high rate of moment decrease between the time of release at time t = 0 and time t = 0.35 sec. Rapid damping was especially evident at bridges 3 and 4, nearer to the groundline than bridge 2. At first, the suspected cause of this excessive energy loss from the system was the release mechanism, which, if responsible, would have absorbed energy by externally relaxing the deflected shape of the pile. This would have occurred completely within the time span of release, between time t = 0 and before the next moment peak. However, since a relatively high rate of energy loss continued to occur between the second and third peaks, as shown in Figs. 6 and 7, the release mechanism was partially vindicated. The data for Test 3-8-2 indicates a similar, more extreme occurence and, since the chances for a poor release occurring twice in a row are slight, the release mechanism appears to be less likely at fault. Finally, since the moment at bridge 2 does not appear to have a high initial energy loss, it is concluded that the problem was not caused by the release mechanism.

To account for the excessive energy loss, it is believed that the 3-in. pile acted like a short, stiff pile. Thus, relatively large deflections occurred down to a significant depth below the groundline, as evidenced by the relatively high moments at bridge 2. As a result, sliding friction between the pile wall and adjacent soil may have occurred, causing a greater response attenuation. This idea is supported by the continuing excessive damping rate with time that the 3-in. pile undergoes. Furthermore, it appears that the amount of damping increases with frequency. Both Brown (1) and Ross (15) have suggested that the error produced by excessive soil damping is more pronounced at larger pile sizes.

Because of the above circumstances, the predicted response was initiated at time t = 0.35 sec by assuming a reduced initial load on the pile. All attempts to predict the first portion of the moment vs. time curve failed in this case. However, in full-scale offshore installations, this problem which is associated with a short, stiff pile may not be encountered.

2-in. Pile Tests. - A total of six tests were made on the 2-in. pile; two tests at each of three different depths, or embedded lengths. Three differing embedded lengths were utilized in an attempt to study the effect of embedded length on pile response. For each embedded length, two tests were made, each at a different frequency of vibration. This was done by changing the weight attached at the top of the model pile.

The same model pile was used throughout the 2-in. pile test series. After the 10-ft tests (Tests 2-10-1 and 2-10-2), the pile was removed from the ground and a 2-ft section cut from the embedded end. The pile was then redriven and tested at 8-ft (Tests 2-8-1 and 2-8-2). Similarly, this procedure was repeated for testing at 6 ft (Tests 2-6-1 and 2-6-2). Thus, for all tests the depth of each strain bridge below the groundline was constant.

The results of the 2-in. pile tests are depicted in Figs. 8 through 23. To illustrate the typical nature of the acceleration data obtained in this investigation, the complete curves obtained in Test 2-10-1 are plotted in Figs. 11 and 12. The higher-frequency distortions, which were probably caused by the instantaneous release and cutting of the stranded wire, are especially evident in Fig. 11.

In some cases, different amounts of field damping occurred in each direction of pile movement, with the more rapid damping generally occurring in the direction of initial loading. (For example, see Fig. 10.) This may indicate that the soil on the side of the pile in the direction of pull was permanently deformed when the initial lateral load was applied. At present, the Ross analytical solution has no provision to account for differing soil characteristics on the two sides of the pile. While slightly better predicted results might have been obtained for a few tests in this study if such a provision existed, there is probably no real need in a full-scale situation for such a provision.

The almost linear damping characteristics exhibited by the 6- and 10-ft embedded tests indicate that in each case the pile was not initially deflected enough to cause a significant soil failure. This is shown to be an ideal situation by the overall good predictability of field response that was obtained.

The effect that a small soil spring value near the surface has

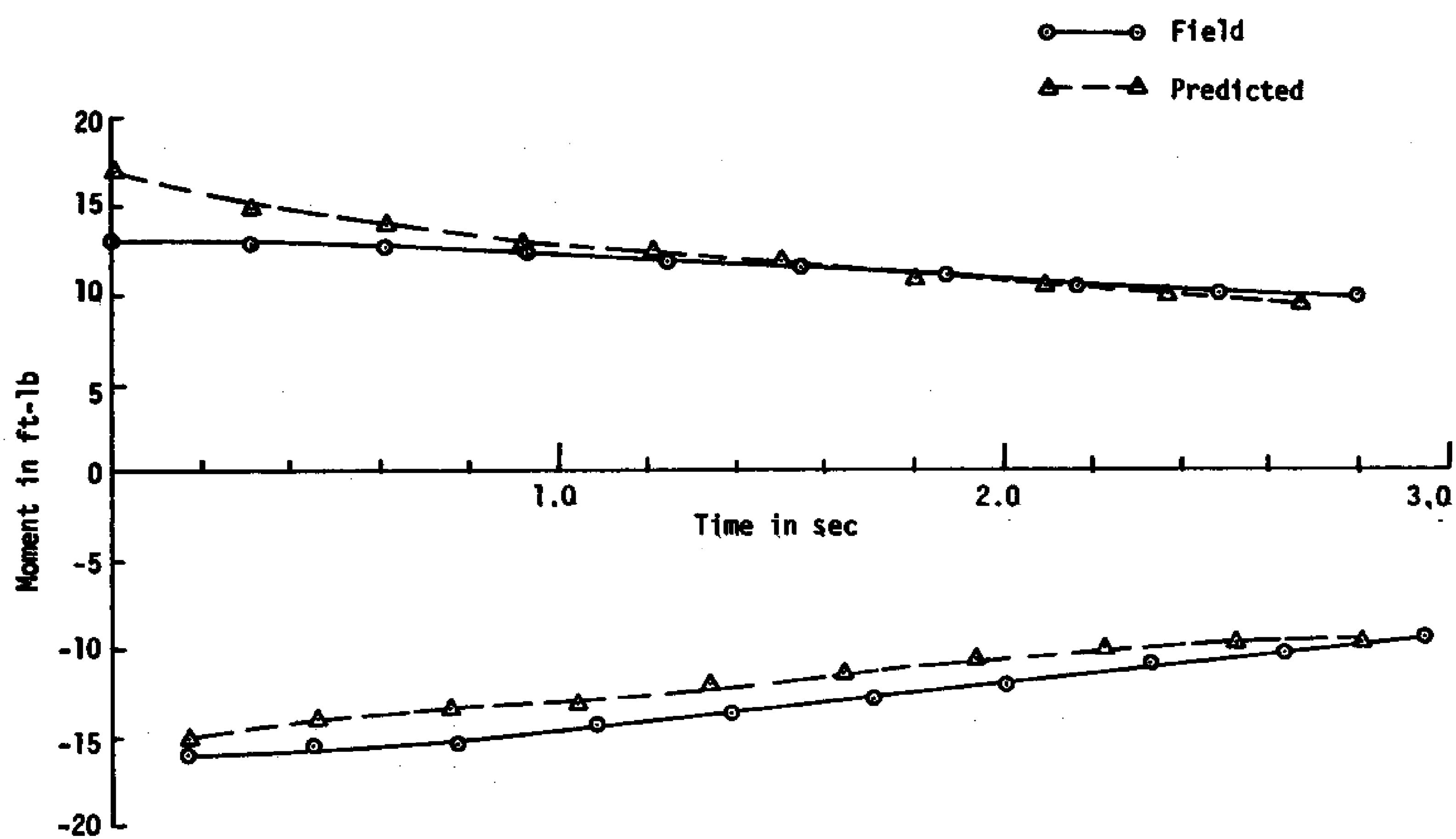


FIGURE 8 .- BENDING MOMENT VERSUS TIME FOR TEST 2-10-1 AT BRIDGE 2

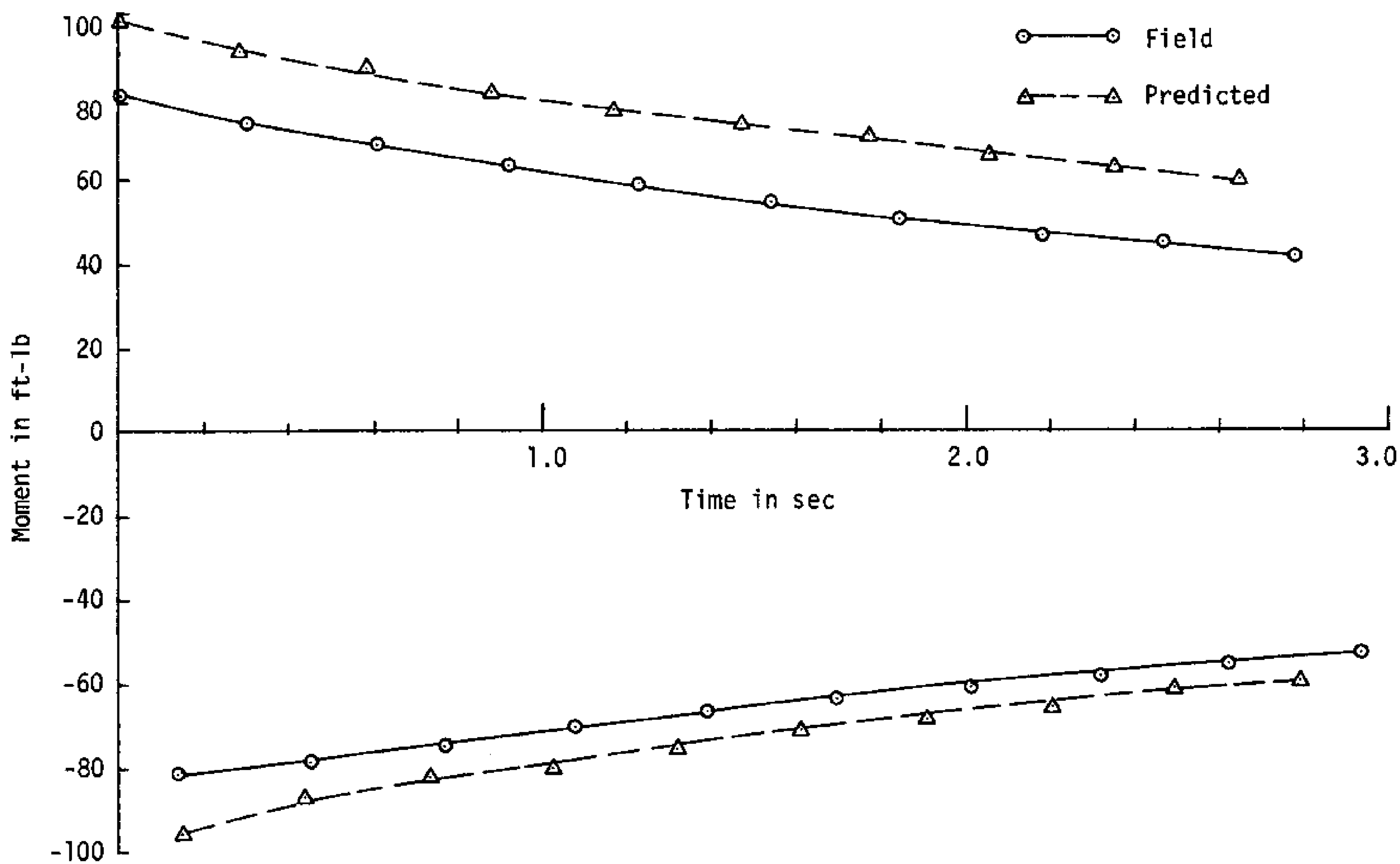


FIGURE 9 .- BENDING MOMENT VERSUS TIME FOR TEST 2-10-1 AT BRIDGE 3

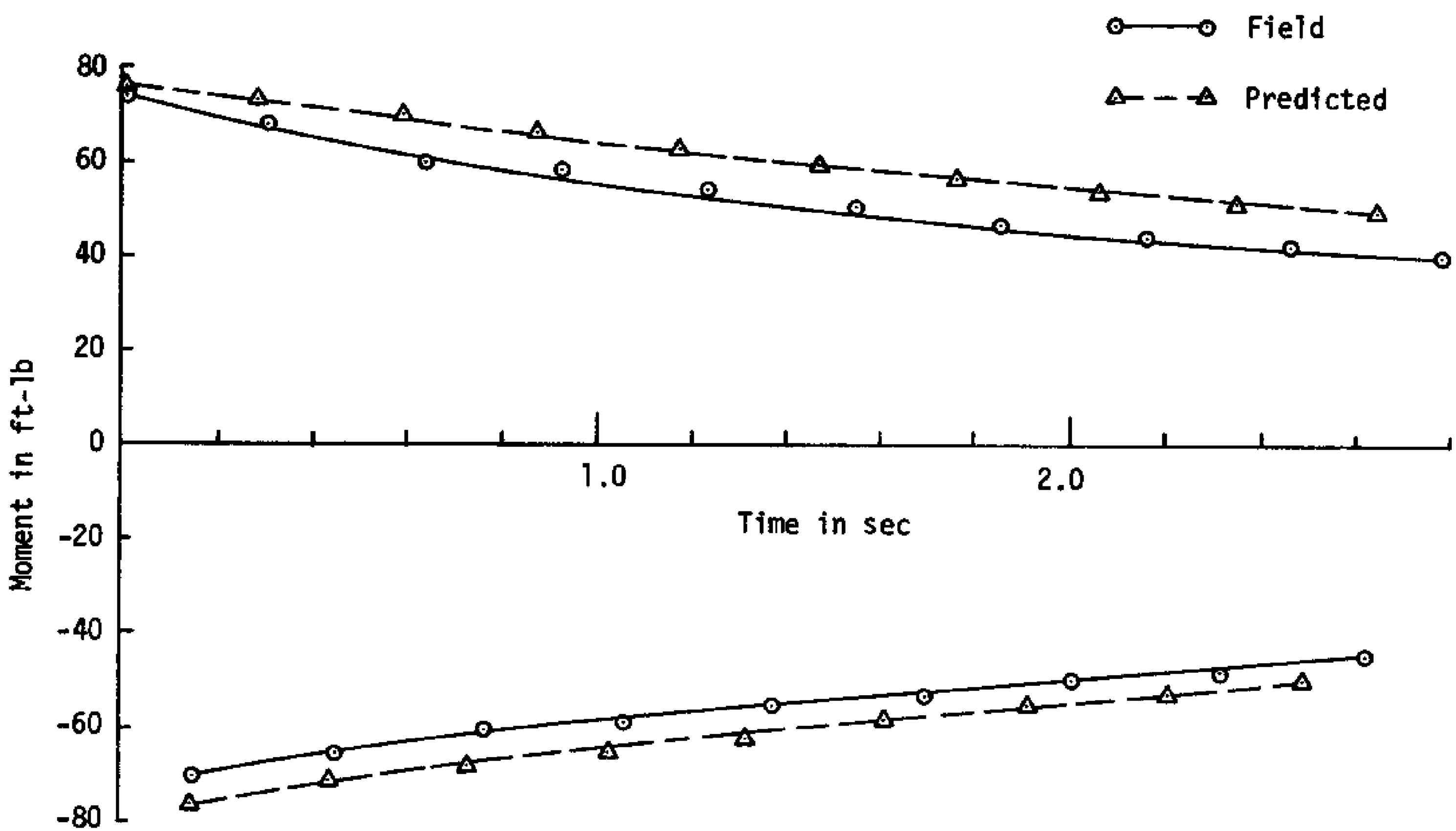


FIGURE 10 .- BENDING MOMENT VERSUS TIME FOR TEST 2-10-1 AT BRIDGE 4

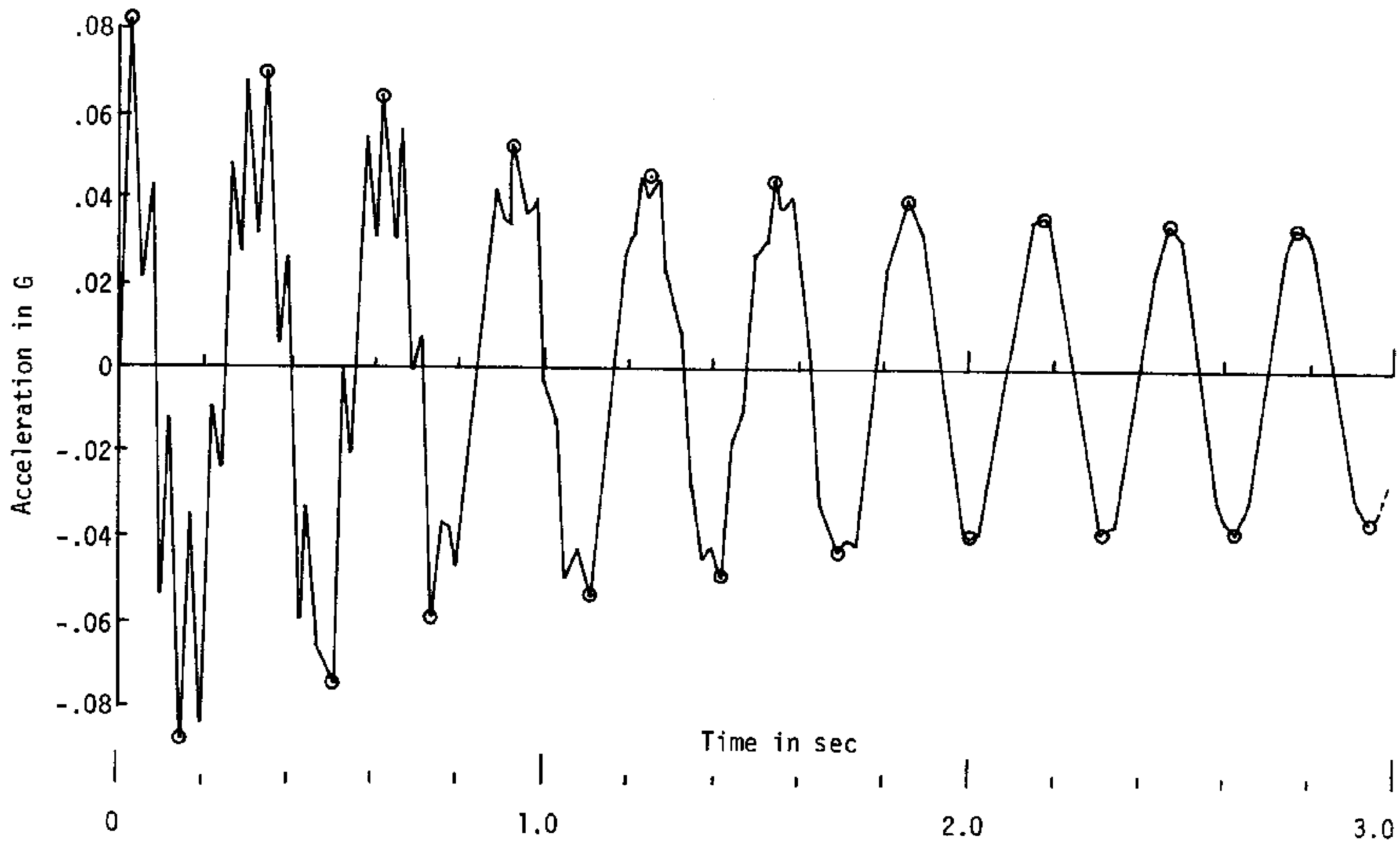


FIGURE 11 .- ACCELERATION VERSUS TIME FOR TEST 2-10-1 AT BOTTOM ACCELEROMETER

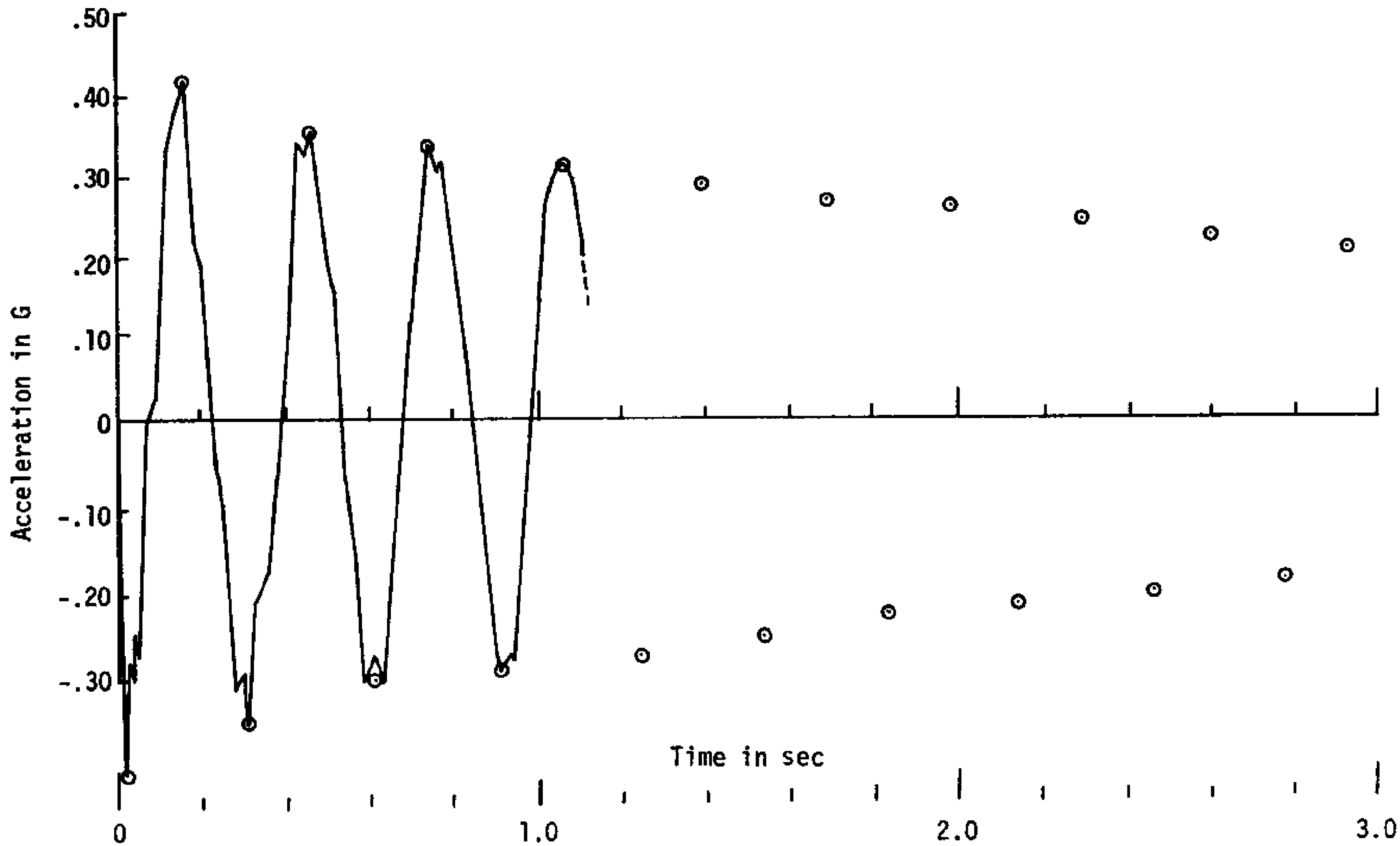


FIGURE 12.- ACCELERATION VERSUS TIME FOR TEST 2-10-1 AT TOP ACCELEROMETER

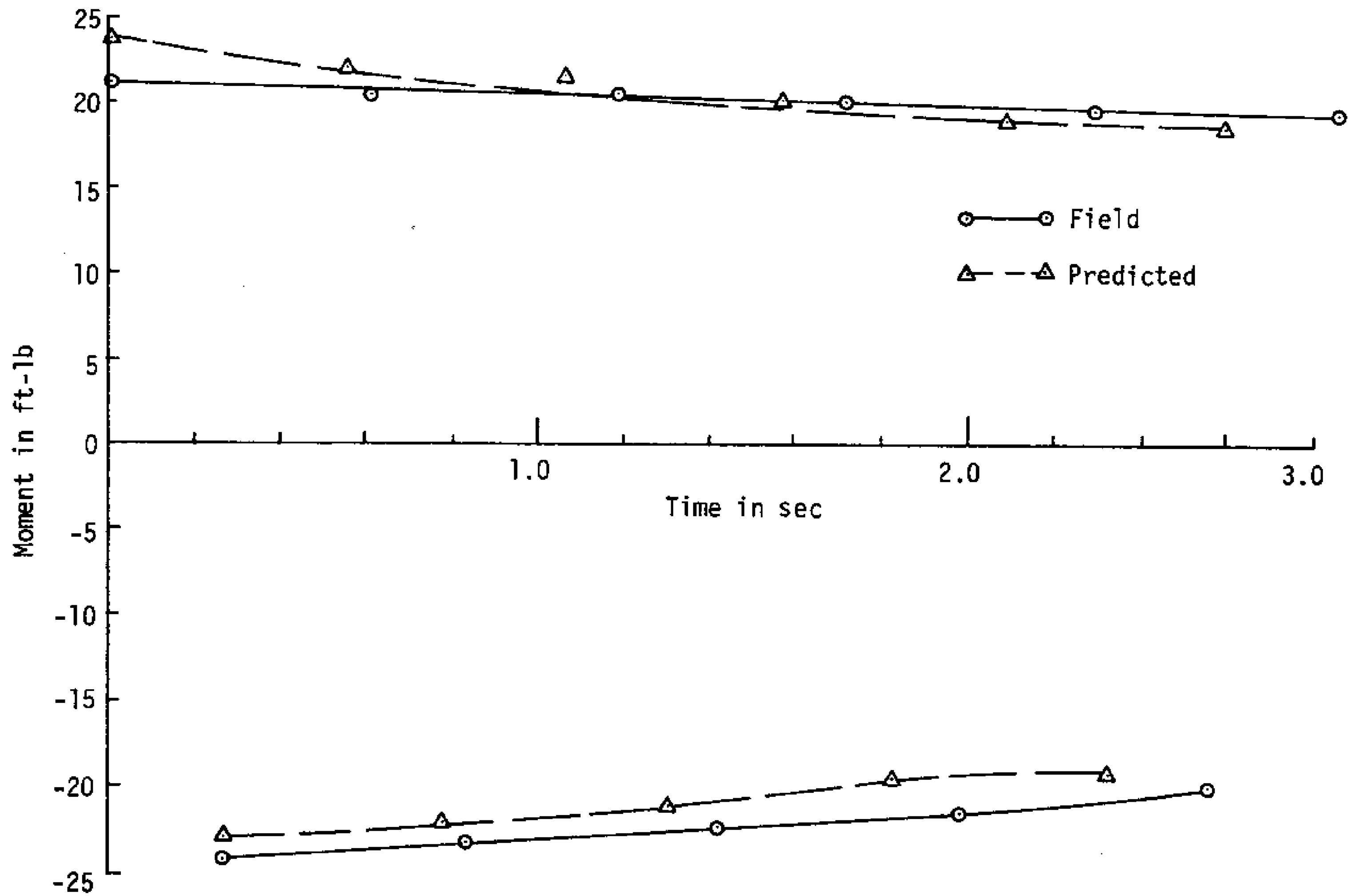


FIGURE 13.- BENDING MOMENT VERSUS TIME FOR TEST 2-10-2 AT BRIDGE 2

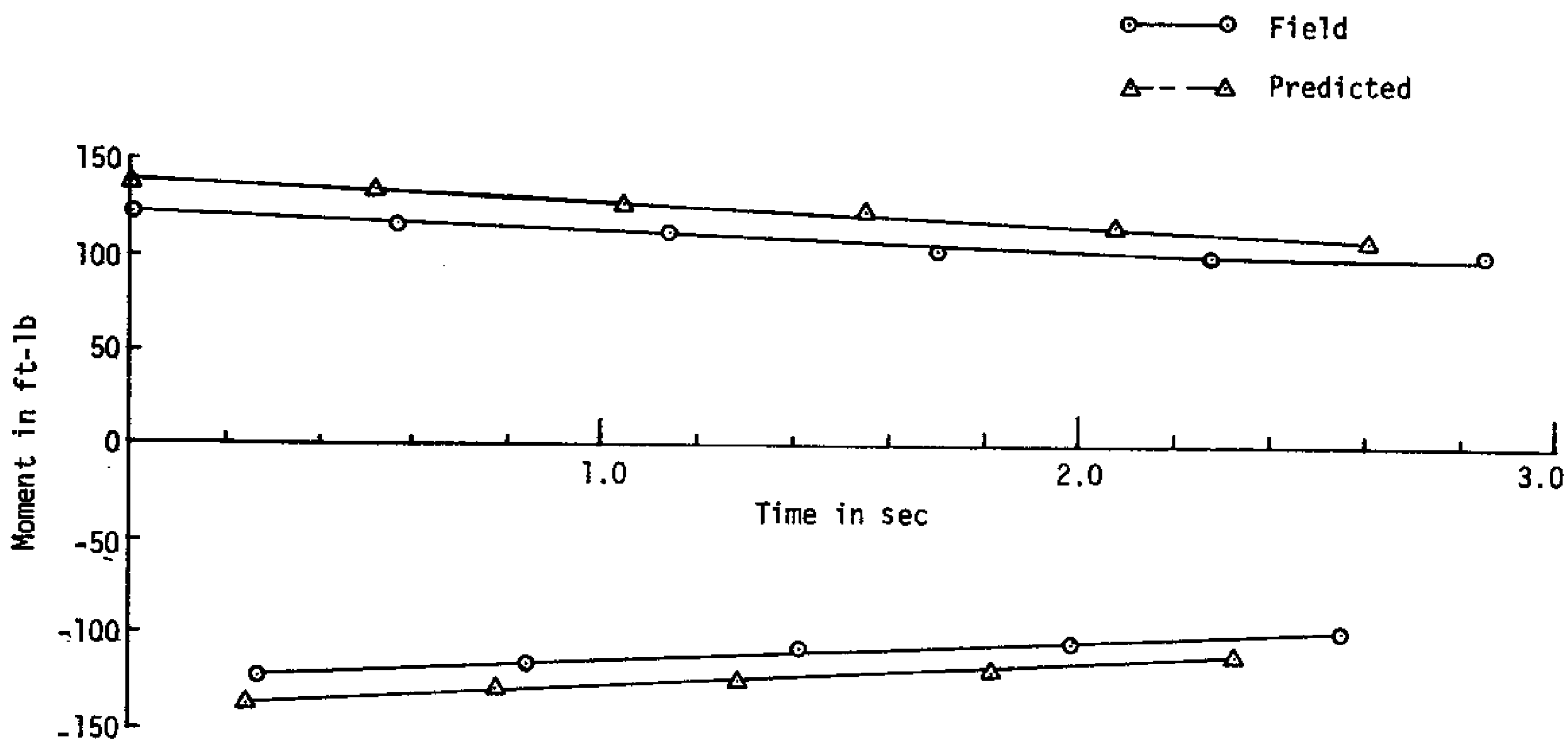


FIGURE 14.- BENDING MOMENT VERSUS TIME FOR TEST 2-10-2 AT BRIDGE 3

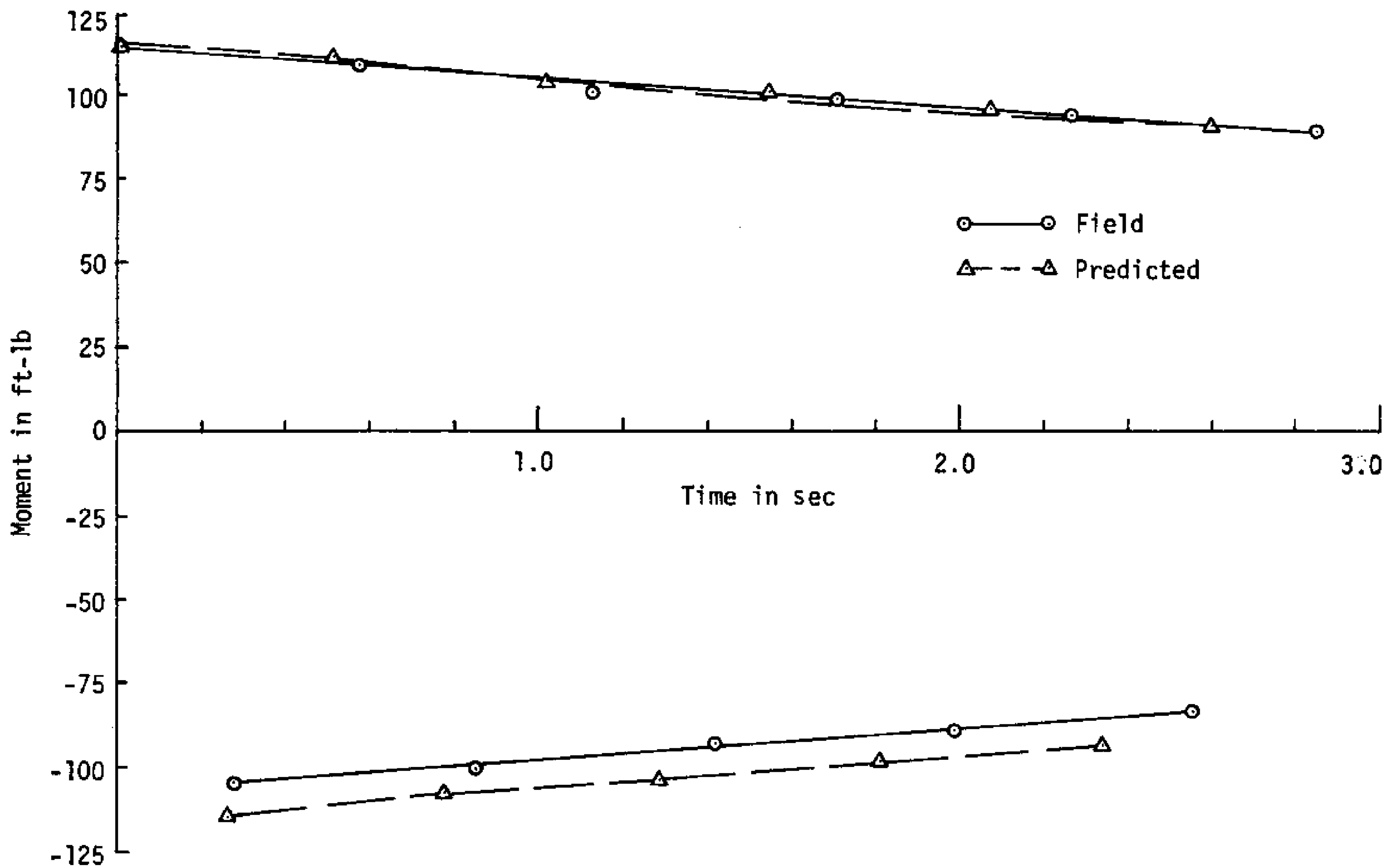


FIGURE 15.- BENDING MOMENT VERSUS TIME FOR TEST 2-10-2 AT BRIDGE 4

on the pile response at a 4-ft depth is shown in Fig. 16. In this case, the top 6 in. of soil contributed slight resistance, and the assumption that K = 0 at bridge 4 is erroneous. The dependency of pile response on the soil resistance near the surface will be more fully discussed in a later section.

The results obtained from Tests 2-8-1 and 2-8-2 are not predictable, the reason for this being two-fold. First, significant bending moments were induced into the pile during driving. Although the pile remained vertical for the first few feet of driving, it was leaning at a slight but noticeable slant after driving was completed. Apparently a significant curvature was forced upon the embedded portion of the pile. This is supported by the fact that the recorded moments are reasonably symmetrical near the surface, as shown in Fig. 23, but become unbalanced with depth, as shown in Figs. 21 and 22. A locked-in curvature causes soil pressures that cannot be predicted on the basis of present methods of analysis. Secondly, during the free vibration phase after release, the axis of vibration deviated from the axis on which the strain bridges were mounted, resulting in what appears to be erratic damping characteristics. Since the strain bridges were sensitive to bending stresses only on this given axis, the measured field moments obtained from strain data became unreliable as time increased. It should be pointed out that the instrumentation was not faulty because, after the 8-ft tests were completed, the pile was removed and used again for tests at the 6-ft depth

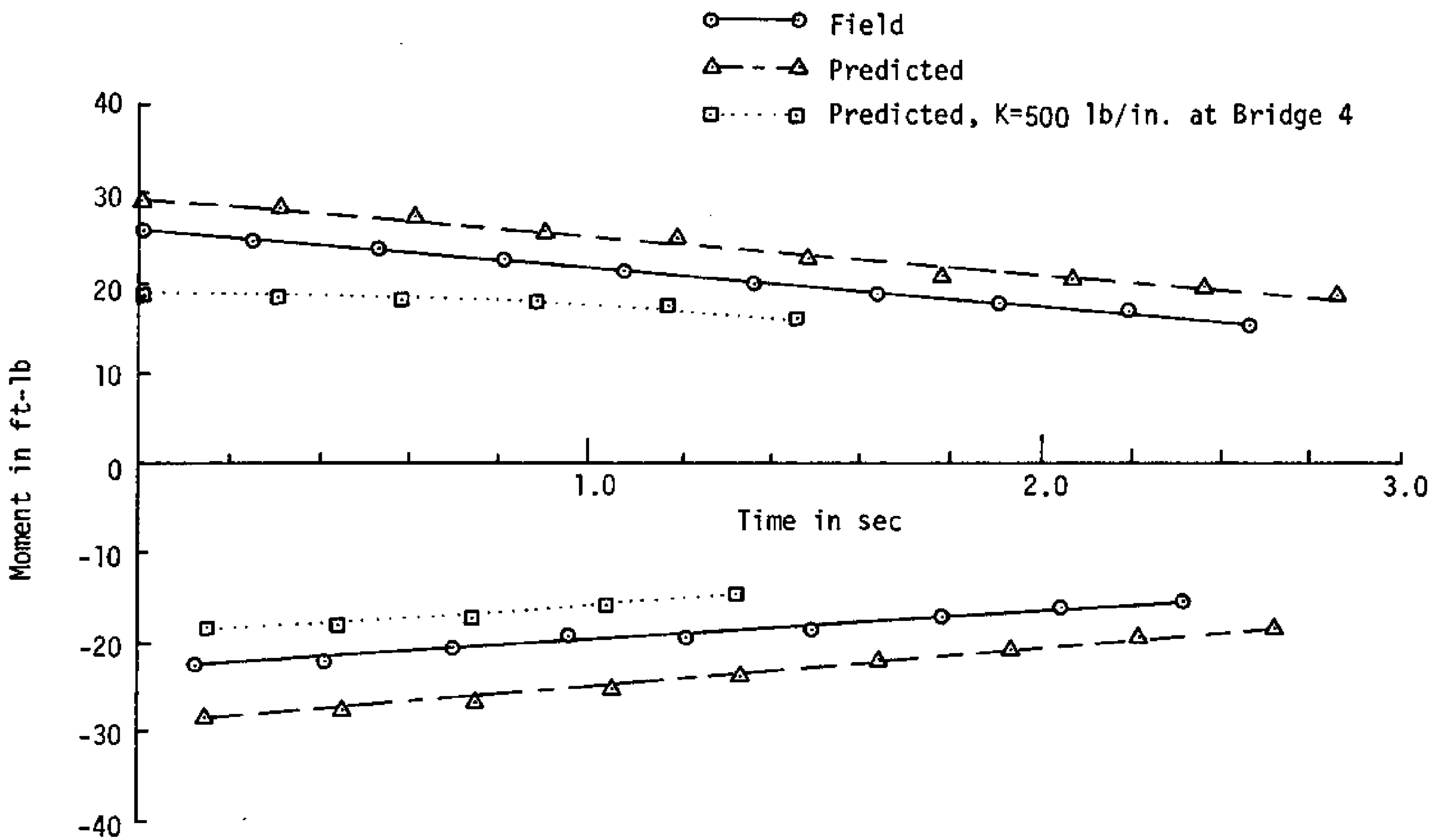


FIGURE 16 .- BENDING MOMENT VERSUS TIME FOR TEST 2-6-1 AT BRIDGE 2

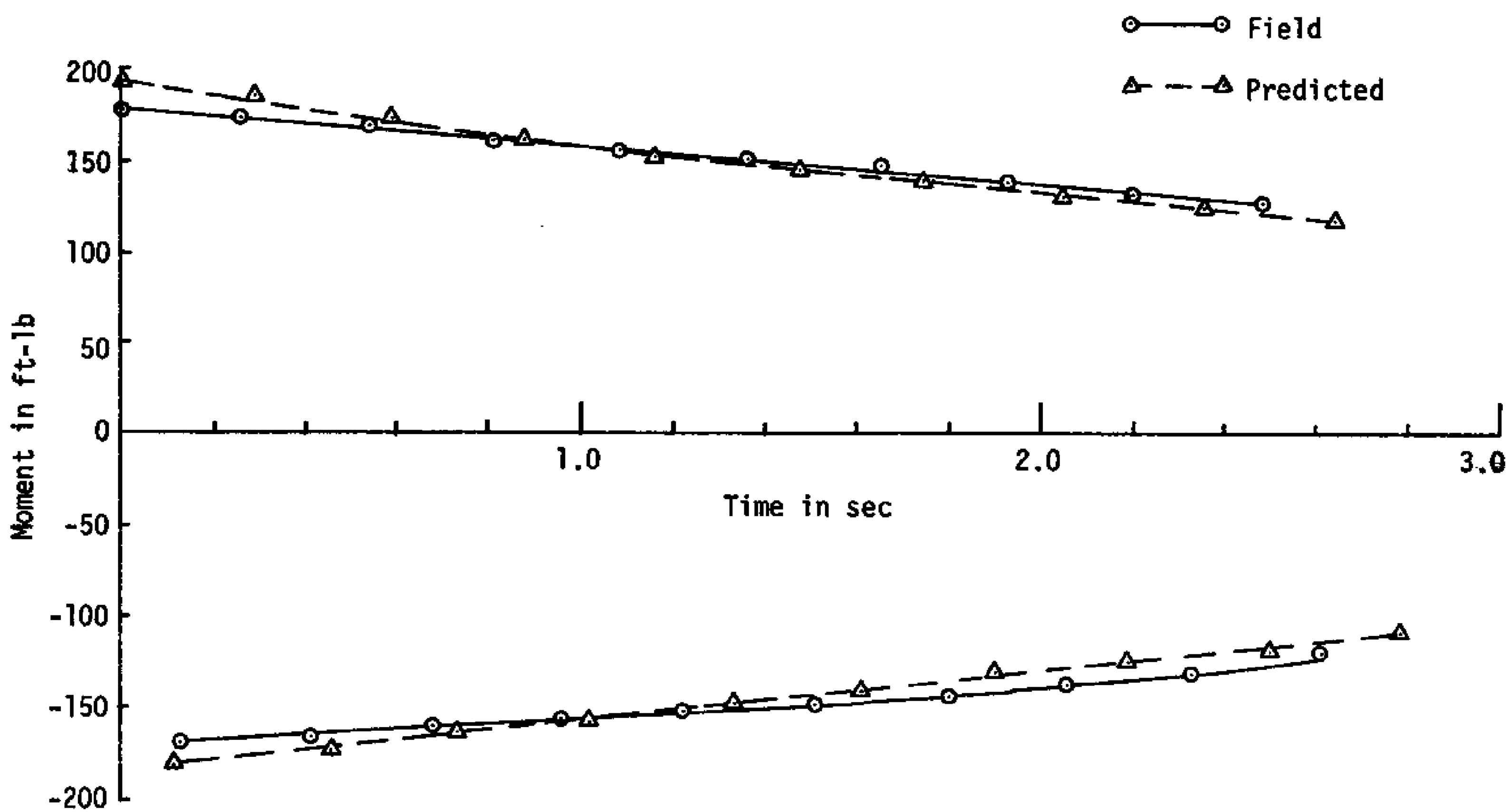


FIGURE 17.- BENDING MOMENT VERSUS TIME FOR TEST 2-6-1 AT BRIDGE 3

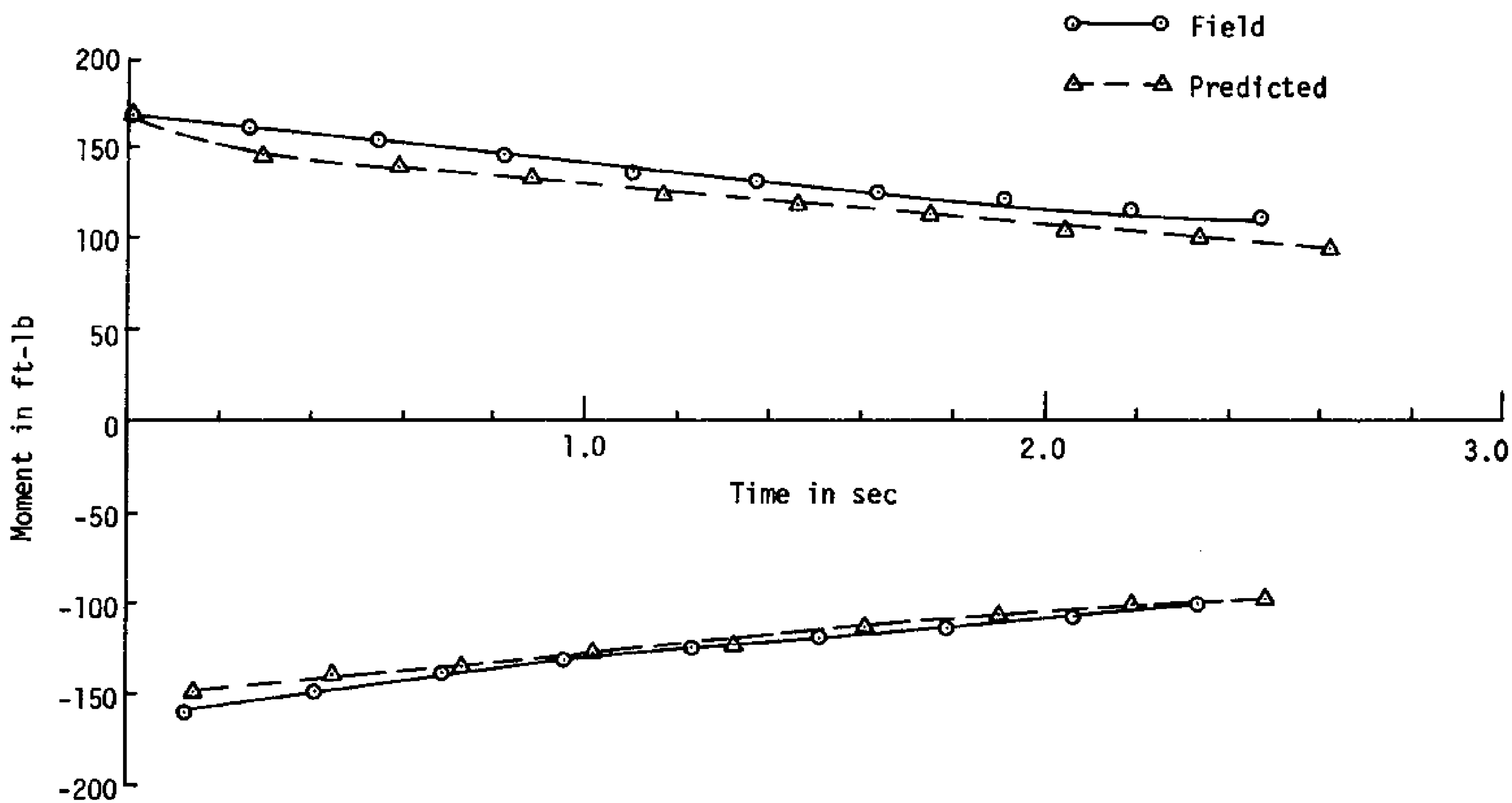


FIGURE 18.- BENDING MOMENT VERSUS TIME FOR TEST 2-6-1 AT BRIDGE 4

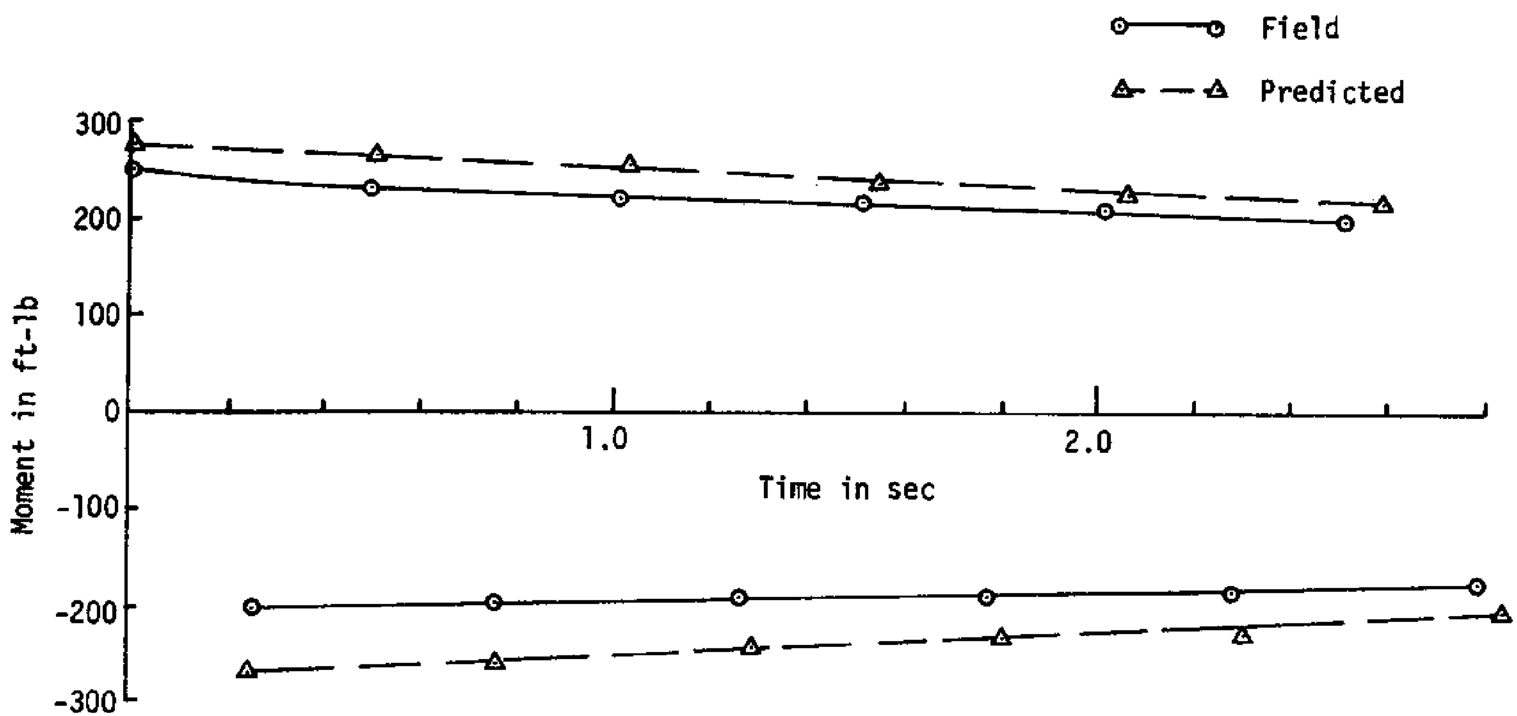


FIGURE 19.- BENDING MOMENT VERSUS TIME FOR TEST 2-6-2 AT BRIDGE 3

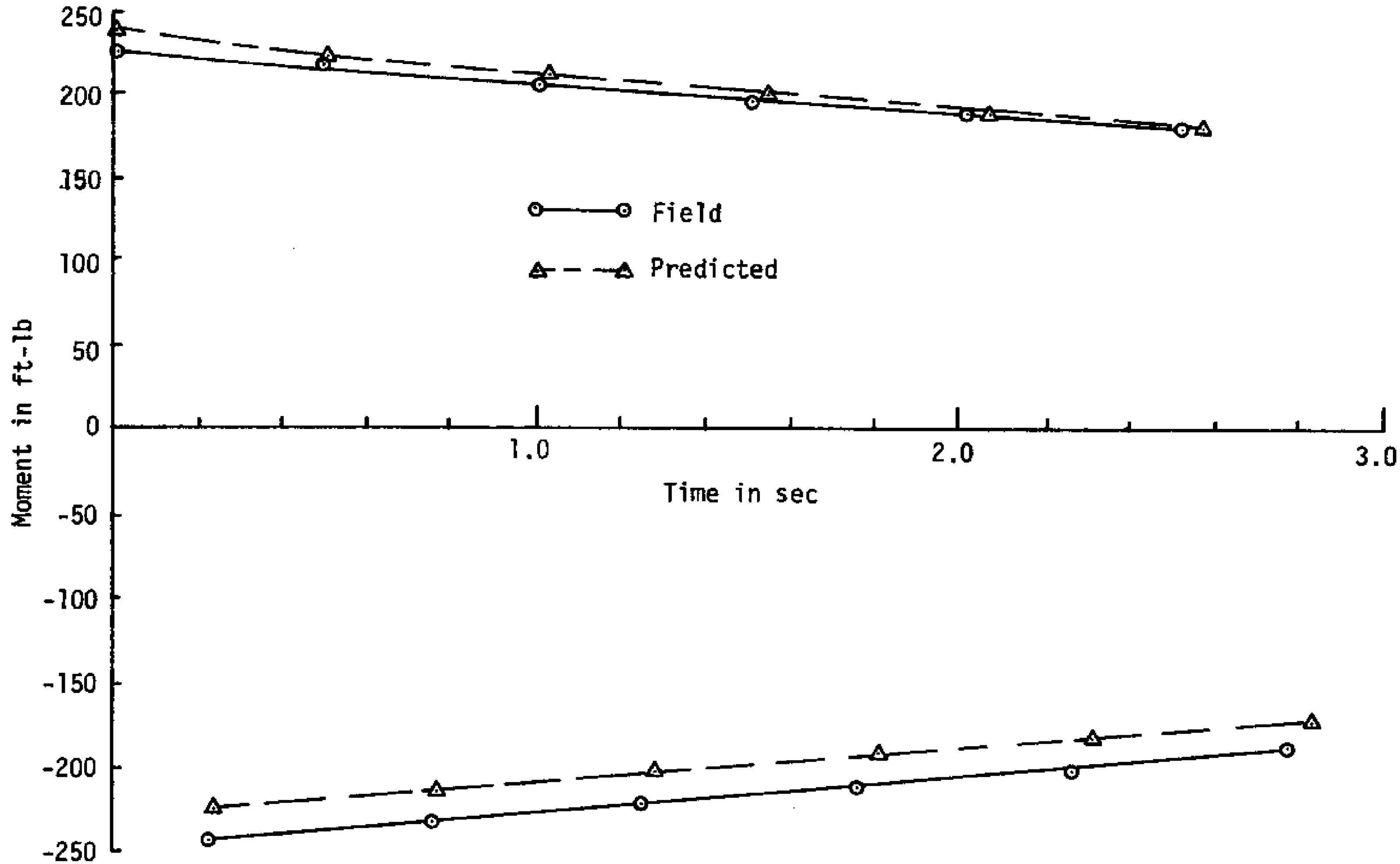


FIGURE 20.- BENDING MOMENT VERSUS TIME FOR TEST 2-6-2 AT BRIDGE 4

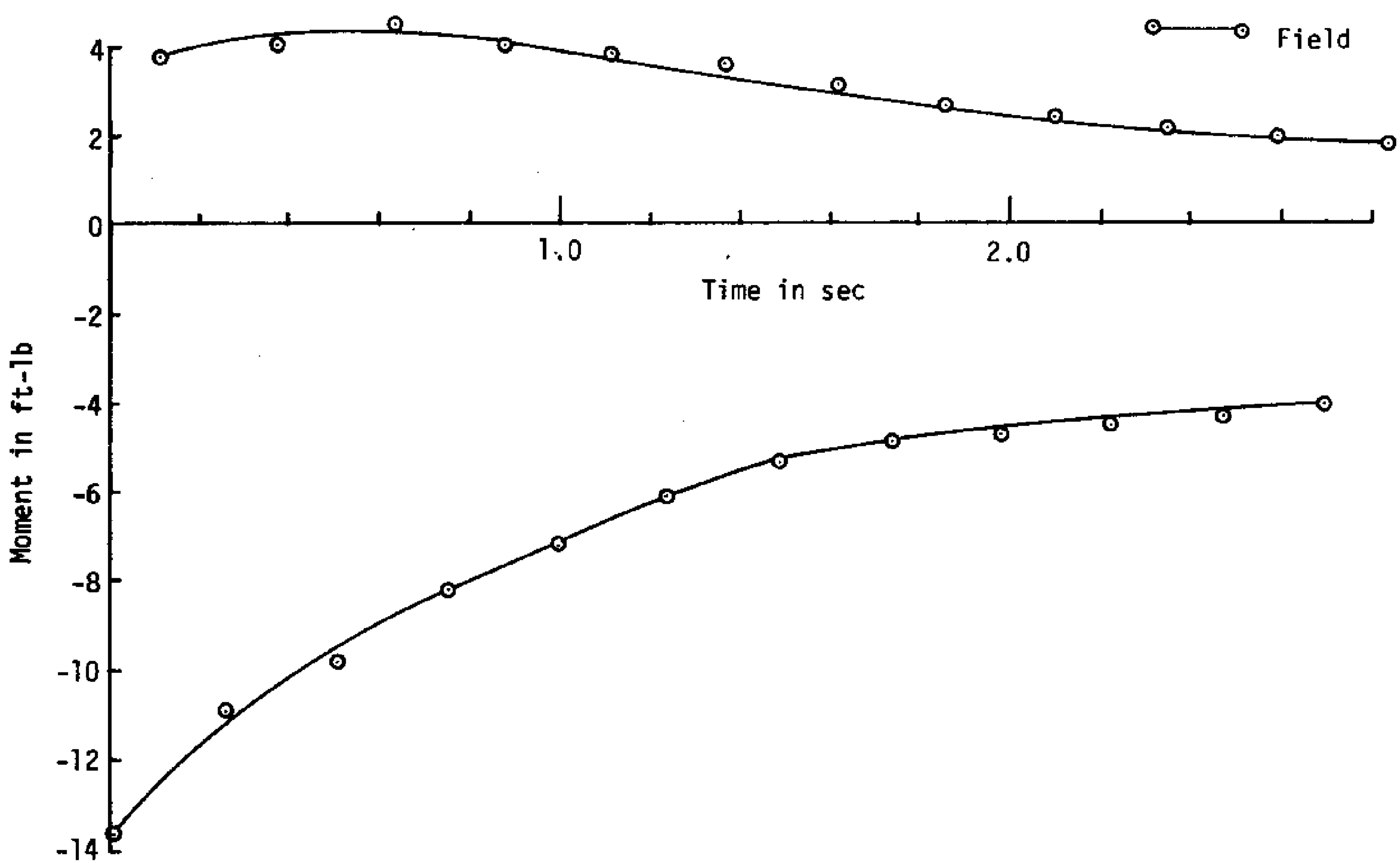


FIGURE 21.- BENDING MOMENT VERSUS TIME FOR TEST 2-8-1 AT BRIDGE 2

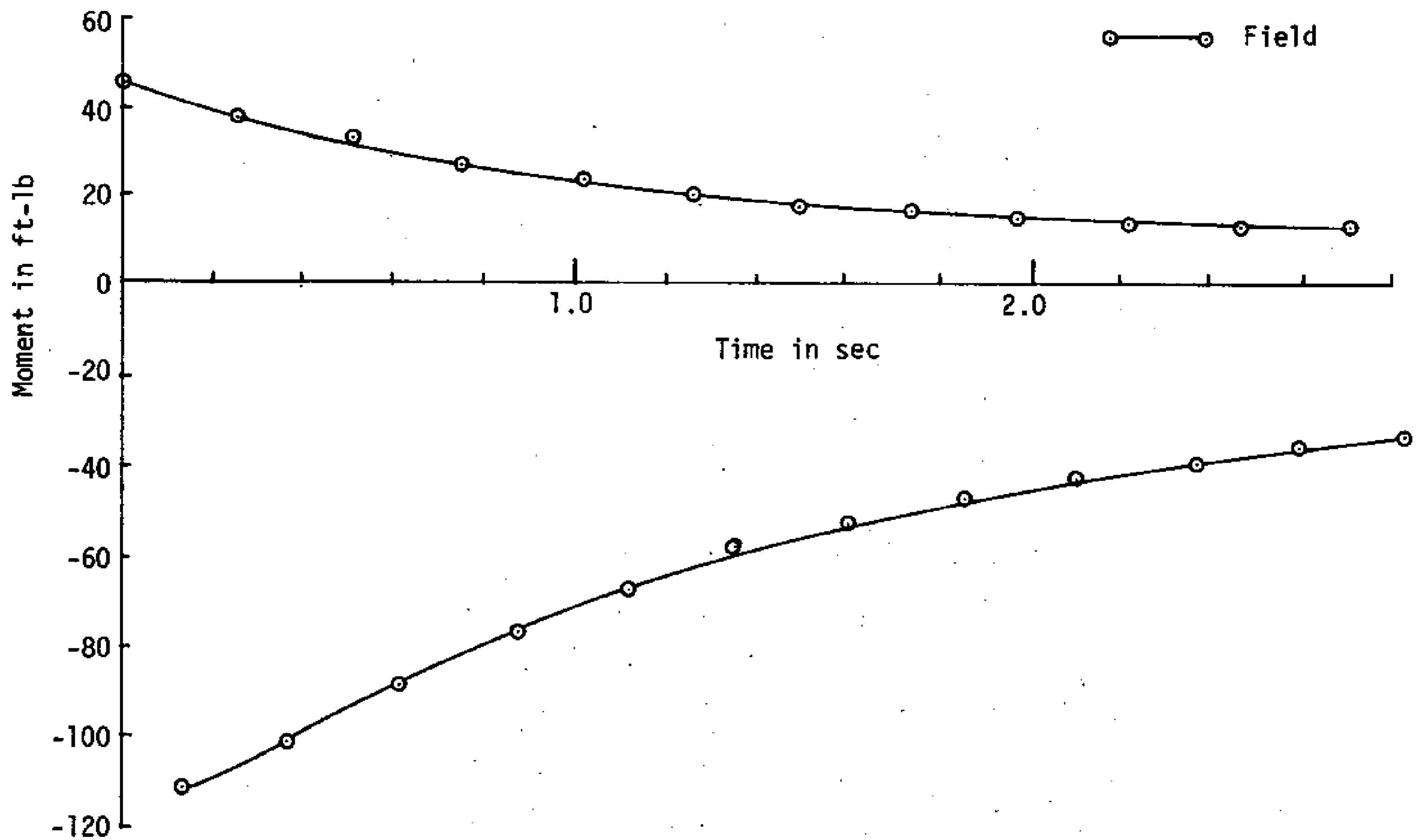


FIGURE 22.- BENDING MOMENT VERSUS TIME FOR TEST 2-8-1 AT BRIDGE 3

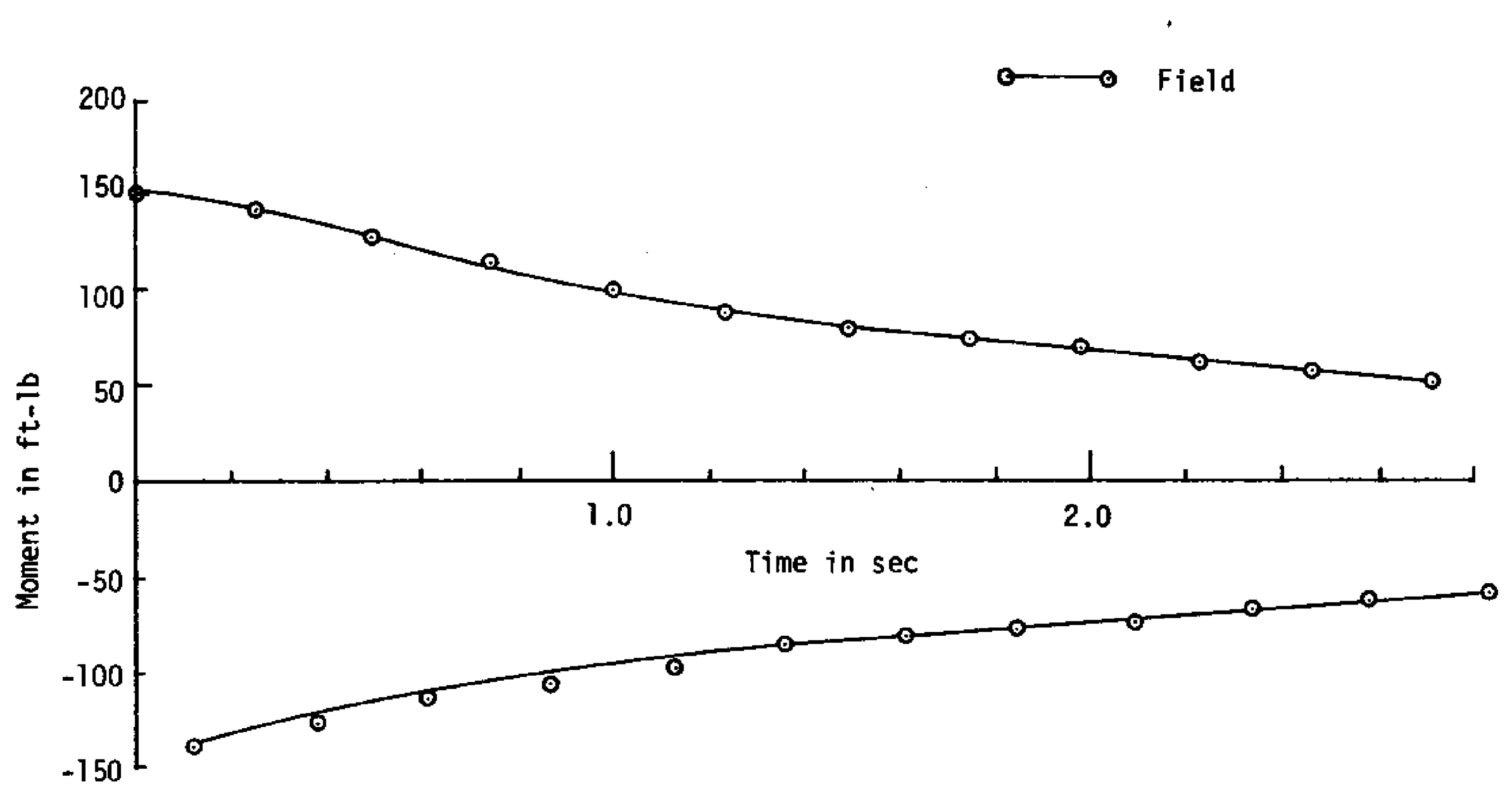


FIGURE 23.- BENDING MOMENT VERSUS TIME FOR TEST 2-8-1 AT BRIDGE 4

(Tests 2-6-1 and 2-6-2), which are predictable. Except for redriving, no modification or correction was made for the latter tests except to shorten the pile by 2 ft.

Test 2-8-1, shown in Figs. 21 through 23, is representative of Test 2-8-2. Complete data for both tests can be found in Appendix IV.

As an interesting consequence apparently due to the induced moments caused by driving, note that the initial moment at bridge 2 (Fig. 21) is negative. This is the only evidence supporting the occurrence of an inflection point in the 2-in. pile.

In spite of the unreliable 8-ft pile test results, any effects due to the embedded length should be evident since the 6-ft and 10-ft depths represent the extremes of the three test depths. The effects due to embedded length are discussed in a later section.

1.25-in. Pile Tests. - Two tests were made on the 1.25-in. pile. Results of both tests were identical except for the higher moments in the second test caused by a slightly higher initial top lateral load. As expected, the moments induced at the 4-ft depth (bridge 2) in both tests were negligible because of the relatively small stiffness of the 1.25-in. pile as compared to the stiffnesses of the two larger piles. Thus, the point of fixity of the 1.25-in. pile is located nearer to the groundline than it was for the larger piles.

The predicted and field response for Test 1-8-1 at bridges 3 and 4 is shown in Figs. 24 and 25. The characteristics of the

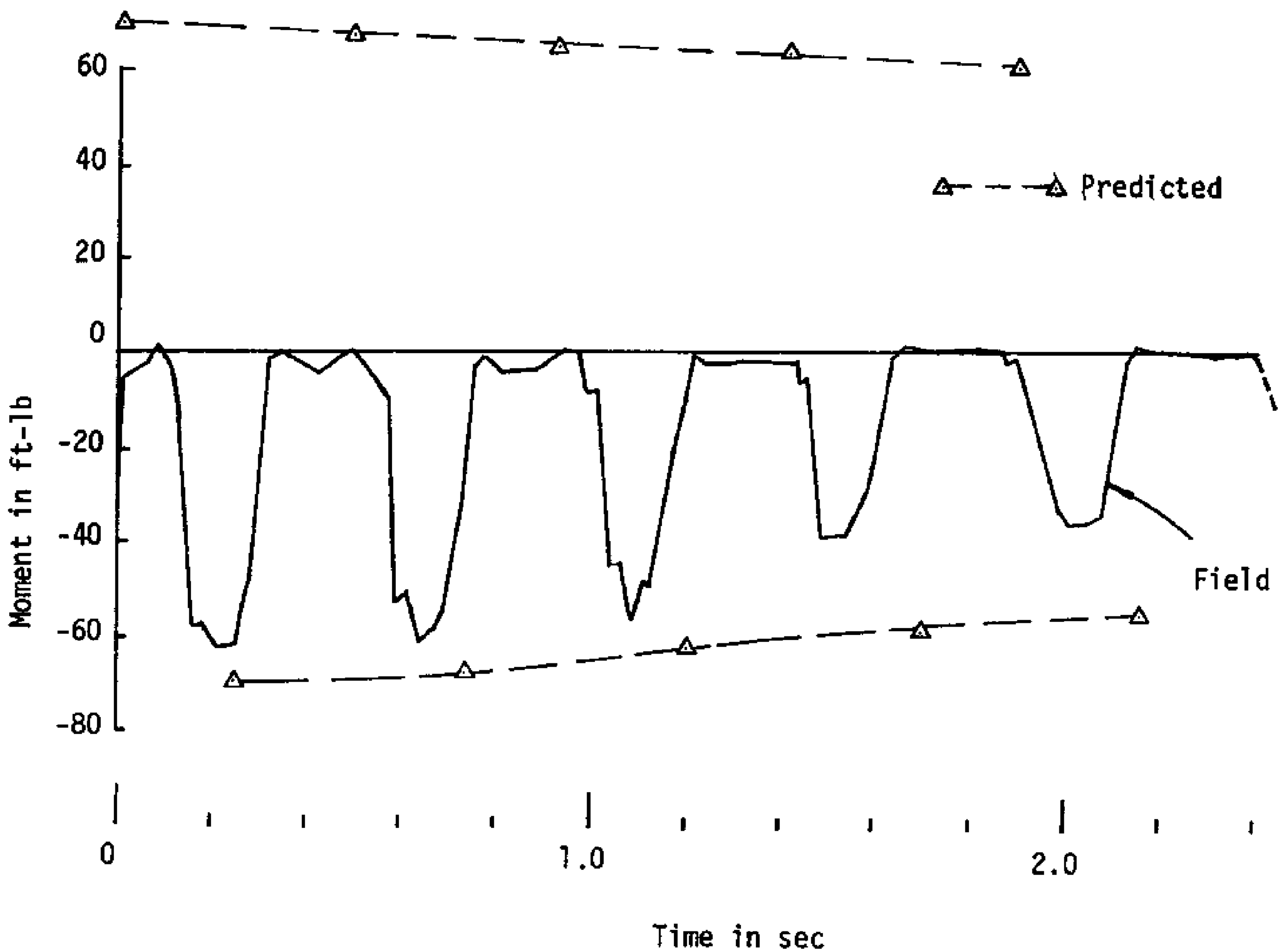


FIGURE 24.- BENDING MOMENT VERSUS TIME FOR TEST 1-8-1 AT BRIDGE 3

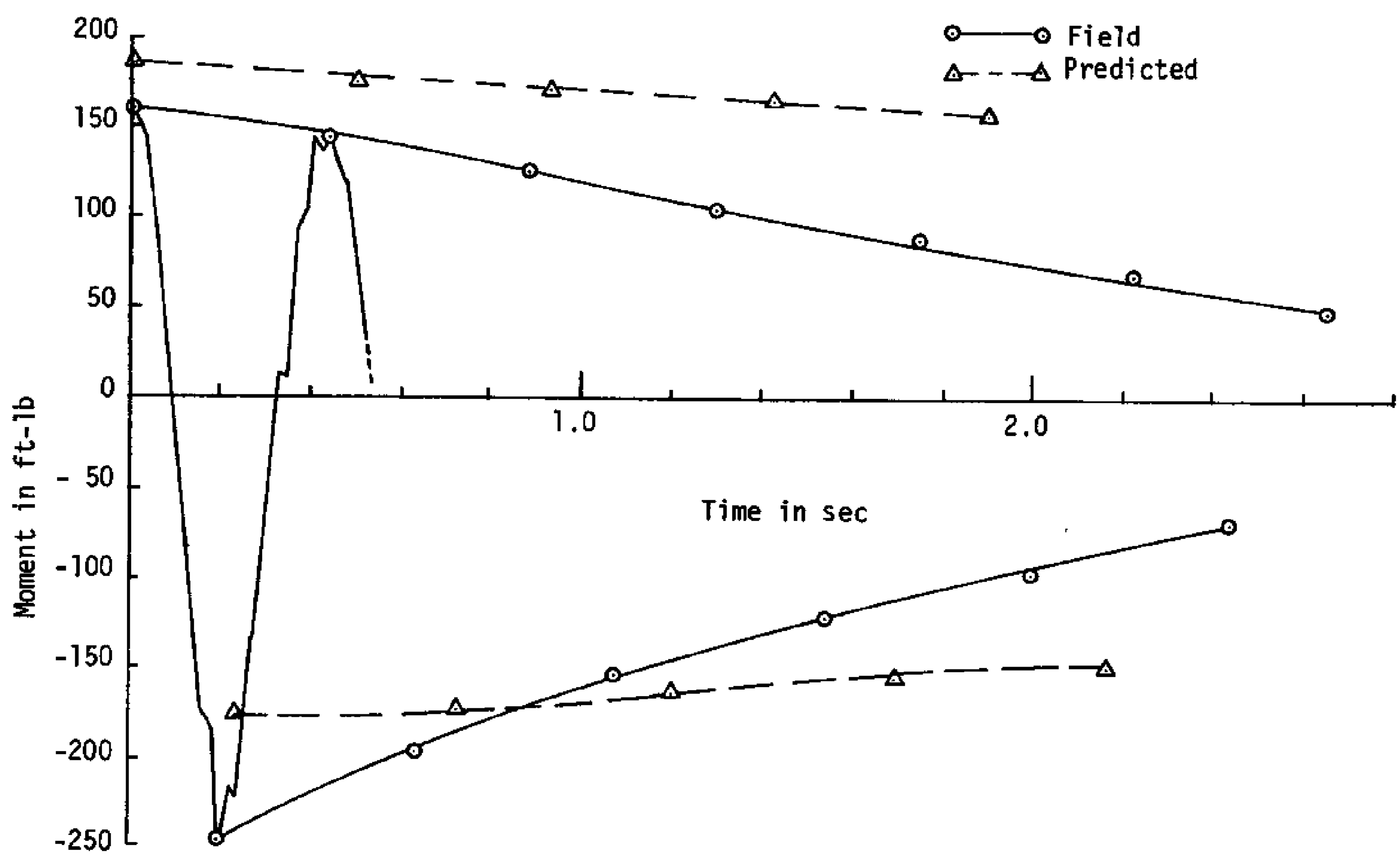


FIGURE 25.- BENDING MOMENT VERSUS TIME FOR TEST 1-8-1 AT BRIDGE 4

moment vs. time curve at bridge 3 are so peculiar that the entire curve is plotted in Fig. 24. Note that each positive moment peak is "clipped" at or near the zero axis. It is not known whether this is due to faulty instrumentation, nonuniformity of the pile, locked-in curvature due to driving, or a combination of these. Due to the relatively small stiffness of this pile, a locked-in curvature could have easily been induced when the pile was driven into the dense sand.

No definite explanation can be given for the non-symmetry and high damping rate of the moments at bridge 4, in Fig. 25. There was no noticeable tilt, or slant of the pile after it was driven, and the axis of vibration did not appear to deviate from the axis of sensitivity of the strain bridges. Evidently, the small size of the 1.25-in. pile caused its response to be more dependent on conditions which were amplified by scaling. This aspect will be more fully discussed in the next section.

SUMMARY OF TEST RESULTS

General. - Predicting the dynamic response of a laterally loaded pile is unduly complicated when any attempt is made to reduce the problem to that of a small scale model study. Scaling of the pile-soil system is detrimental in that any reduction of the pile size is not accompanied by a compensating reduction in soil grain size together with a change in other soil properties. As the size of the pile is reduced, the response becomes increasingly dependent on conditions which can be assessed only qualitatively, if at all. Examples of such conditions which were apparent in this study are the nonuniformity of the model pile itself, the hole around the driven pile, induced moments present in the pile after driving due to locked-in curvature, and soil disturbance adjacent to the pile wall. The nonuniformity of the model pile was due to the method of manufacturing the pipe, welding during instrumentation, and the threaded connection used to attach the protruding section. The hole around the pile, soil disturbance, and the induced moments were due primarily to the driving operation.

With the exception of induced moments due to driving, it is not believed that the above conditions will adversely affect the predictability of full-scale pile response.

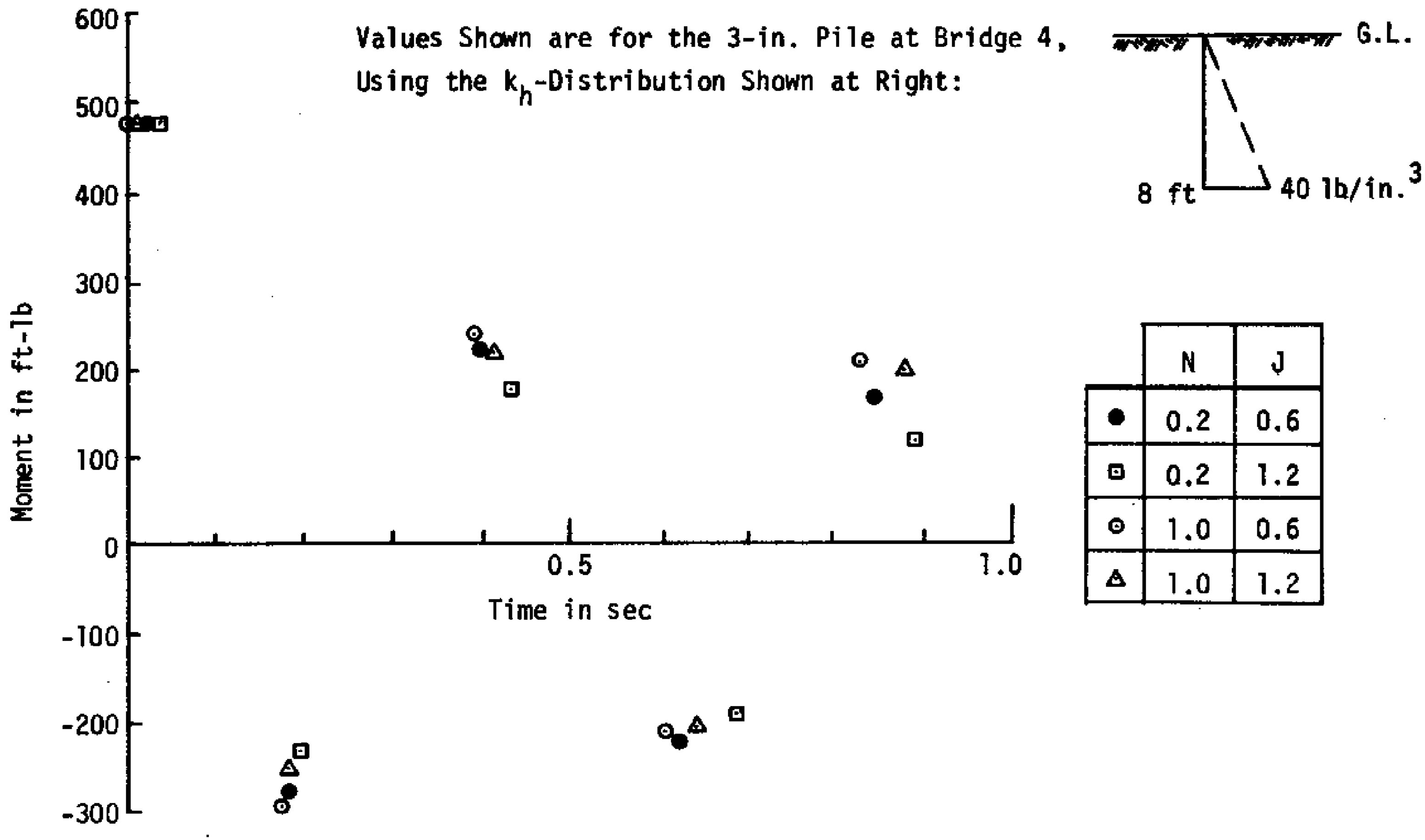
Aspects of the Soil Spring, K. - The magnitude and distribution with depth of the soil spring K (in conjunction with Q) controls to a large extent the magnitude and distribution with depth of the bending moments. Frequency of vibration is also largely controlled by K. A stiffer, more rigid soil is simulated by increasing the K-values, which causes the point of fixity of the pile to become closer to the groundline. Thus, the frequency of vibration is increased due to a shorter effective length of oscillation.

It is widely recognized that the response of a laterally loaded pile is largely controlled by the soil region near the surface, down to some depth that depends on the diameter and structural stiffness of the pile. This is supported by the results of this investigation, as both the 3.5-in. and 2-in. piles apparently were fixed (for all practical purposes) at some point above the 6-ft depth, while the 1.25-in. pile was fixed at some point above the 4-ft depth. It was also found that the predicted moments at the 2- and 4-ft depths were fairly responsive to changes in the soil spring located at the 6-in. depth. However, the value of K at this shallow depth was not as critical (for sand) as it was found to be in Brown's (1) investigation in stiff clay because the sand did not possess the great stiffness (large K) in the region near the groundline that was characteristic of the clay.

Effects of the Soil Quake, Q. - The soil quake, Q, and the spring constant, K, together determine the distribution with depth of the moments in each pile, and the ultimate load the soil can sustain before failing. The value of Q alone is the ultimate displacement that will result in a soil failure. Therefore, Q accounts for the smaller soil resistance due to plastic soil behavior which occurs in the region adjacent to the groundline. The relationship of Q to predicted soil-pile load-displacement charactertistics is illustrated in Fig. 4.

Effects of J and N. - In this investigation a J-value of 0.6 was generally satisfactory when used in conjunction with an N-value of 0.20. As shown in Fig. 7, the predicted response near the groundline in this study is not very responsive to a significant change in J. However, the values of J and N are closely related, and the influence that a change in either has on predicted pile response is a function of velocity, pile properties and size, frequency of vibration, and, as found in this study, soil properties. Referring to Figs. 26 and 7, it appears that the influence of J and N on predicted pile response increases as the soil stiffness decreases. The negligible change in predicted response (due to a change in J) which is shown in Fig. 7 is based on a stiff soil, modeled by the k_h-distribution illustrated in Fig. 3. The predicted pile response shown in Fig. 26, however, is based on a soil of considerable less stiffness, indicated by the k_h-distribution shown in Fig. 26. The small values of k_h used in predicting the response shown in Fig. 26 are hypothetical, but nevertheless demonstrate that effects of J and N can be significant.

Effects of a Gap Around the Pile. - Because the effect on pile response due to the top 6 in. of soil was neglected in this investigation, any hole or gap between the pile wall and surrounding



	N	J
●	0.2	0.6
⊡	0.2	1.2
⊙	1.0	0.6
△	1.0	1.2

FIGURE 26 .- EFFECTS OF N AND J ON PREDICTED PILE RESPONSE

soil will not affect the predicted response unless it is introduced at some node (strain bridge location) deeper than the 6-in. depth. Since indications are that no significant gap existed below approximately 6 in., there is no justification for assuming otherwise. It should be noted, however, that if considerable soil stiffness had existed at or very near to the surface as characteristic of a stiff clay, a hole around the pile would have considerable effect.

Effects of Pile Diameter and Embedded Length. - From Eq. 4, the most obvious effect of pile diameter is its influence on the value of K. Any additional effects due to the pile diameter were not discernible in this investigation, since the piles tested were of three different stiffnesses as well as diameters. However, other effects may exist, as the results of any loading test in sand depend on the size of the loaded area.

The 2-in. pile was tested at three differing embedded depths to investigate the effect that depth has on model pile response. Since the 2-in. pile was essentially fixed at some depth less than 6-ft, no significant effects were apparent. However, a slight (but definite) influence on frequency of vibration was established from the data and confirmed by the predicted response, as indicated in Table 4, shown on the following page. Table 4 shows that for both field and predicted response, the frequency tends to increase slightly as the embedded length decreases. It is emphasized that this effect is slight, and may not occur once some sufficient depth is reached.

TABLE 4. - EFFECT OF EMBEDDED LENGTH ON FREQUENCY

OF VIBRATION OF THE 2-IN. PILE

Test No.	Field Frequency (cps)	Predicted Frequency (cps)
2-10-1	3.23	3.35
2-10-2	1.74	1.94
2-6-1	3.60	3.44
2-6-2	1.96	1.94

CONCLUSIONS AND RECOMMENDATIONS

Conclusions. - The broad objective of this study was to verify that the dynamic field response of a laterally loaded model pile in sand could be reasonably predicted by using soil parameters obtained from standard field and laboratory tests. As significant corollaries of this objective, the analytical solution developed by Ross (15) was justified for use with cohesionless soils, and a variety of field test data was collected. No conclusions can be made, nor should any be inferred from the outcome of this study concerning full-scale piles. The results herein are limited in scope because they were obtained from small-scale model tests in one specific type of soil. However, the following specific conclusions can be drawn for the specific piles and soil concerned:

1. The soil spring, K, is the most important soil parameter to be used in the static analysis of a laterally loaded pile. A triangular distribution of k_h with depth yielded generally favorable results. In this investigation, laboratory triaxial tests were used successfully to obtain k_h-values. The k_h values were used with Eq. 4 to obtain K-values.
2. The soil quake, Q, must be used to account for plastic soil behavior where pile deflections are excessive, usually near the groundline. Q is a

function of pile diameter and may be related to the soil properties as well. In this investigation, Q was approximated as one percent of the pile diameter.

3. Differing embedded lengths beyond the depth of fixity were found to influence frequency slightly.
4. For a value of N = 0.20, effects on pile response due to a change in J were insignificant. However, J and N are closely related, and are probably a complicated function of the testing configuration and soil-pile characteristics and properties. As found in this study, the influence of J and N on predicted pile response increased as the soil stiffness decreased.
5. The amount of damping increased as the frequency increased and was significant at the frequencies encountered in these tests. However, such frequencies are not likely to be caused by forces due to offshore wave action.

Recommendations. - The following recommendations are made concerning any future research in this area:

1. A parameter study should be made, using the Ross analytical solution, on a real offshore soil-pile-platform system. The properties of the soil and configuration of the pile and platform should represent a typical offshore installation. This parameter study would determine the sensitivity

of the solution to each of the required input parameters, and to certain pile-platform characteristics. Thus, additional research would not be attempted regarding any parameter(s) which exerted an insignificant influence on the predicted response. As an immediate and practical outcome of such a study, the practicing coastal engineer could be provided with information relating the manner in which each input parameter influences the solution, and therefore, his design.

2. If additional model studies are conducted, more strain bridges should be placed within the critical shallow depths of the embedded length. All piles should be loaded with an identical force at the top, with the requirement of maintaining small deflections possibly dictating an exception to this. Some form of an electromagnet should be used for a smooth, instantaneous release. From the standpoint of determining effects due to differing diameters, it would be desirable to select piles of different diameter but the same moment of inertia and stiffness. Thus, with all other variables held constant, any differences in pile response should be due to diameter effects alone.

3. A similar study should be conducted by instrumenting

full-scale piles which will be subjected to lateral loads.

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APPENDIX II. - NOTATION

The following symbols and abbreviations are used in this thesis:

cps = cycles per second

cu in. = cubic inches

D = outside diameter of pile, in inches

ft = feet

ft-lb = foot-pounds of moment

G = gravitational acceleration

in. = inches

J = soil damping factor, in seconds per foot

K = soil spring value, in pounds per inch

k_h = coefficient of horizontal subgrade reaction, in pounds per cubic inch

L = length of segment in idealized model, in feet

L_A = length of pile above the groundline, in feet

L_E = length of pile embedded below the groundline, in feet

lb = pounds

n_h = constant of horizontal subgrade reaction for D = 1 ft, in tons per cubic foot

N = dimensionless exponent of the velocity, V

No. = number

P = horizontal static load applied at top of pile, in pounds

$P_{dynamic}$ = horizontal dynamic load on idealized pile segment

P_f = horizontal failure load of soil around idealized pile segment

P_{static} = horizontal static load on idealized pile segment

p = pressure, in pounds per square inch

% = percent

ϕ' = effective angle of internal shearing resistance, in degrees

psi = pounds per square inch

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

sec = seconds

σ_3 = confining pressure, in pounds per square inch

sq. = square

V = lateral velocity of an idealized pile segment

y = lateral displacement, in inches

z = depth below groundline, in feet

APPENDIX III. - FIELD AND LABORATORY SOIL TEST RESULTS

Field Description of the Soil. - Visually, the soil at the test site may be classified as a gray fine sand, uniformly graded, with a trace of silt. The small amount of apparent cohesion that the sand possesses can be attributed to its partly-saturated state. An overlying thin layer of tan clayey sand was removed from the immediate test area prior to driving the model test piles.

Groundwater Conditions. - The groundwater table was below the embedded test pile depth at all times during the course of the field testing program.

Sieve Analysis. - The results of a sieve analysis performed on a representative sample of the material from the test site is contained in Table A1.

TABLE A1. - SIEVE ANALYSIS

Sieve No.	% Retained
10	0.0
20	0.1
40	0.6
80	65.0
100	80.5
200	99.2
-200 Amount	0.8%

Triaxial Testing. - All laboratory triaxial tests were performed under drained conditions on undisturbed samples 1-5/8 in. in diameter and approximately 3 in. in length. The results of tests made to determine k_h values are shown in Fig. A1. Other triaxial tests were performed to determine the apparent cohesion and the effective angle of internal shearing resistance of the sand. These test results are tabulated in Table A2.

TABLE A2. - TRIAXIAL TEST RESULTS

Depth (ft)	Apparent Cohesion (lb/in.^{3})	ϕ' (degrees)
1.25-1.5	0.8	31
3.25-3.5	0.0	40
5.25-5.5	1.5	36
7.25-7.5	2.0	35

Standard Penetration Test. - The results obtained from a Standard Penetration Test and from triaxial tests are compared in Fig. A2.

Other Information. - Dry unit weight, moisture content, and relative density values are tabulated in Table A3.

TABLE A3. - UNIT WEIGHT, MOISTURE CONTENT, AND RELATIVE DENSITY

Depth (ft)	Dry Unit Weight (lb/ft^3)	Moisture Content (%)	Relative Density (%)
1-2	105	15.5	----
3-4	94	6.0	78.3
5-6	92	5.6	77.2
7-8	91	14.6	46.5
8-9	95	15.2	----

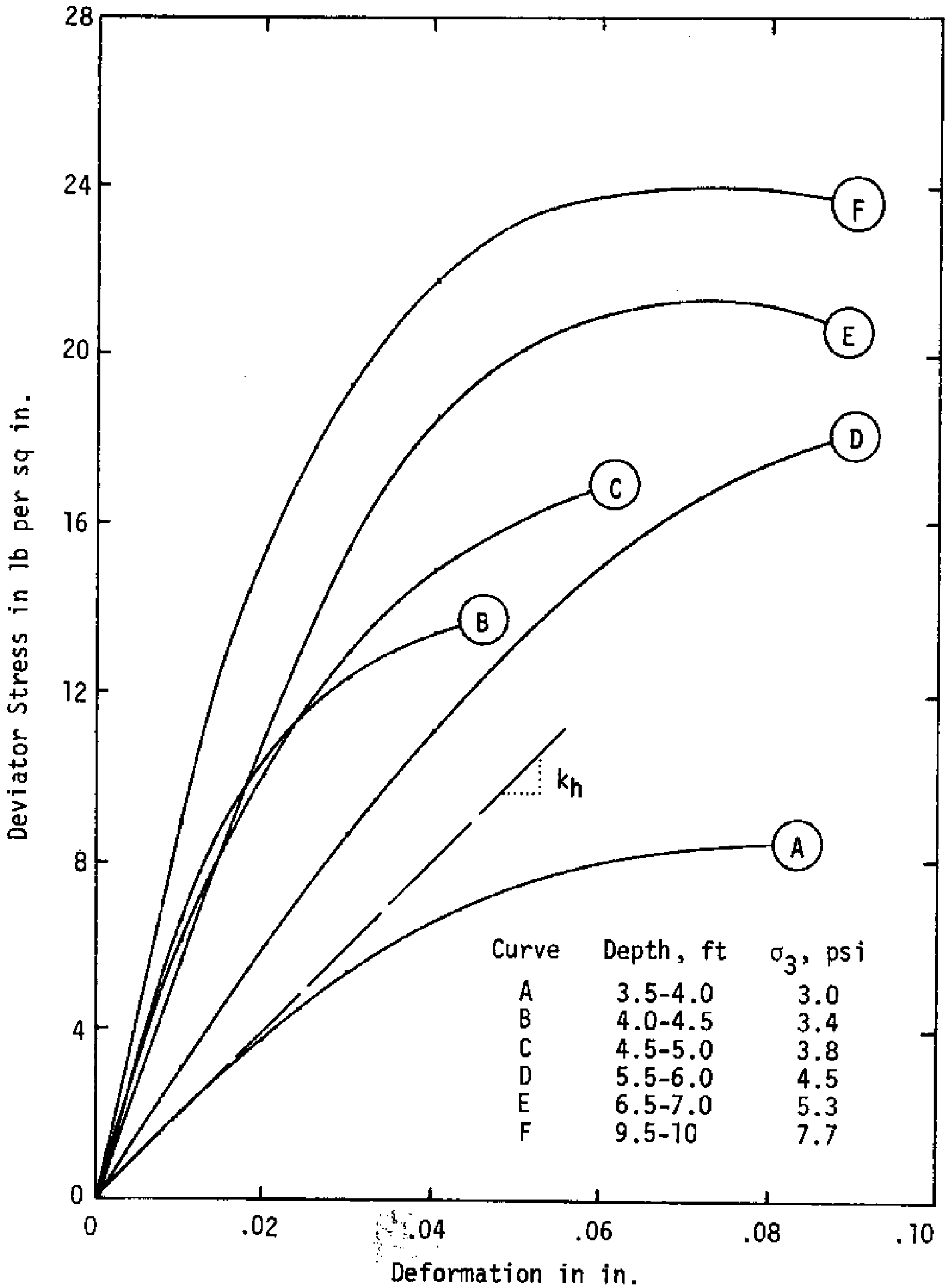

FIGURE A1.- DEVIATOR STRESS VERSUS DEFORMATION CURVES

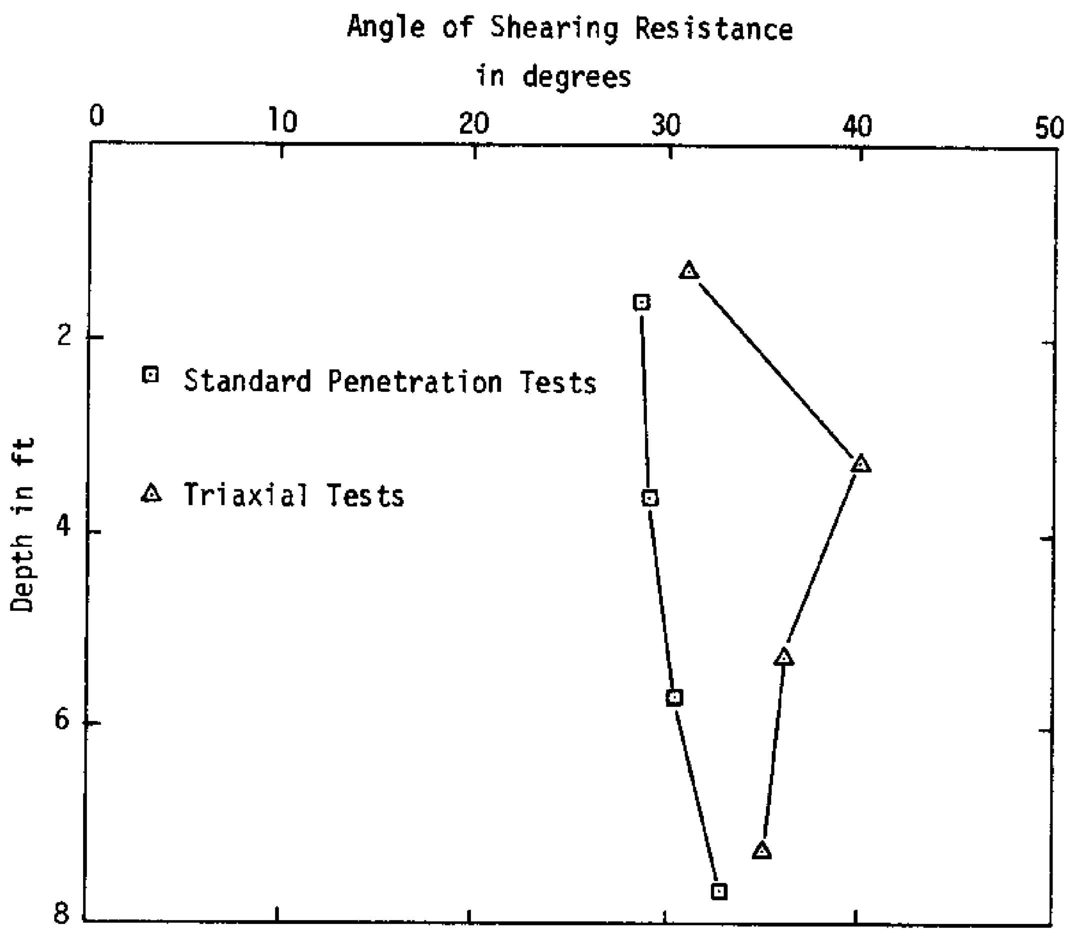

FIGURE A2.- COMPARISON OF THE ANGLES OF SHEARING RESISTANCE AS OBTAINED BY STANDARD PENETRATION TESTS AND BY TRIAXIAL TESTS [AFTER IVEY AND DUNLAP (6)]

APPENDIX IV. - FIELD DATA

On the following 15 pages are the field test data for all tests conducted in this investigation. The following notation is used:

Bridge 1 - Bending moments measured at a depth of 6 ft.

Bridge 2 - Bending moments measured at a depth of 4 ft.

Bridge 3 - Bending moments measured at a depth of 2 ft.

Bridge 4 - Bending moments measured at a depth of 0.5 ft.

Top Accel. - Accelerations measured at the top of the pile.

Bottom Accel. - Accelerations measured 9 in. above the groundline.

Test 3-8-1

Bridge 1*

Bridge 2

Time Sec	Moment Ft-Lbs
.00	243
.17	-278
.30	215
.50	-236
.67	195
.84	-206
1.02	175
1.19	-175
1.36	156
1.52	-150
1.68	130
1.85	-120
2.02	107
2.20	- 95
2.37	85
2.55	- 70
2.73	65
2.91	- 50

Bridge 3

Time Sec	Moment Ft-Lbs
.00	510
.16	-355
.32	332
.48	-256
.66	278
.85	-223
1.01	238
1.19	-175
1.36	208
1.53	-150
1.70	175
1.86	-120
2.02	140
2.20	- 80
2.36	100
2.55	- 60
2.72	80
2.91	- 35

Bridge 4

Time Sec	Moment Ft-Lbs
.00	480
.17	-295
.35	235
.53	-185
.70	188
.88	-165
1.01	160
1.19	-140
1.36	135
1.53	-113
1.69	112
1.85	- 93
2.03	94
2.20	- 80
2.38	65
2.55	- 60
2.73	45
2.91	- 40

Top Accel.

Time Sec	Accel. G
.01	-.85
.02	-.41
.03	-.55
.04	-.35
.05	-.19
.06	-.14
.07	-.02
.08	-.04
.11	.23
.14	.33
.17	.37
.20	.32
.22	.19
.24	.14
.25	.05
.32	-.23
.36	-.29
.40	-.20
.47	.13
.53	.23
.57	.19
.66	-.15
.71	-.23
.75	-.15
.83	.14
.88	.21
.92	.14
1.01	-.12
1.05	-.20
1.09	-.13
1.18	.12
1.23	.18
1.27	.13
1.36	-.11
1.40	-.16
1.45	-.10
1.54	.12
1.58	.16
1.62	.12
1.82	-.09
1.75	-.12
1.80	-.08
1.90	.11
1.93	.13
1.97	.10
2.10	-.09
2.28	.11
2.45	-.07
2.63	.09
2.81	-.05
2.98	.07

Bottom Accel.

Time Sec	Accel. G
.01	.12
.02	.18
.03	.25
.04	.42
.06	.72
.08	.00
.09	.02
.11	-.83
.15	-.03
.17	-.37
.19	-.23
.22	-.58
.25	-.11
.27	-.15
.29	.31
.30	.37
.31	.29
.33	.00
.35	.26
.37	.15
.39	.30
.41	.15
.42	.22
.45	-.09
.47	-.08
.50	-.34
.52	-.29
.53	-.30
.55	-.27
.57	-.35
.59	-.19

*Moments ranged from 0 to -3 ft-lbs.

.61	-.17	.90	-.26	1.36	.11	1.92	-.18
.64	.07	.92	-.27	1.40	.15	1.97	-.14
.66	.11	.96	-.10	1.44	.14	2.10	.09
.68	.20	.99	.07	1.48	.03	2.27	-.15
.70	.19	1.03	.16	1.53	-.14	2.45	.07
.72	.22	1.06	.17	1.58	-.22	2.58	-.11
.74	.19	1.08	.18	1.63	-.16	2.64	-.11
.76	.17	1.12	.10	1.71	.08	2.80	.03
.83	-.22	1.18	-.19	1.75	.12	2.94	-.10
.86	-.25	1.23	-.25	1.79	.09		
.88	-.28	1.28	-.19	1.88	-.14		

Test 3-8-2

Bridge 1*

Bridge 2

Time Sec	Moment Ft-Lbs
.00	270
.03	284
.05	263
.09	-166
.11	-272
.13	-280
.14	-304
.16	-379
.18	-381
.20	-385
.21	-371
.22	-324
.28	48
.29	146
.30	164
.31	152
.32	144
.35	265
.37	249
.38	274
.41	243
.42	217
.44	115
.46	58
.48	- 95
.50	-188
.53	-247
.56	-265
.59	-243
.62	-129
.69	121
.71	180
.74	203
.78	164
.82	46
.86	- 89
.89	-172
.92	-207
.96	-170
1.05	107
1.10	164
1.14	133
1.22	- 85
1.27	-162
1.33	-103
1.40	75
1.46	127
1.50	95
1.53	38
1.63	-125
1.82	93
1.99	- 87
2.17	58
2.35	- 54
2.53	32
2.72	- 32
2.92	18

Bridge 3

Time Sec	Moment Ft-Lbs
.00	750
.03	667
.06	123
.11	-301
.13	-363
.15	-486
.17	-513
.21	-431
.26	- 68
.30	205
.35	438
.36	417
.38	479
.40	383
.44	123
.49	-154
.55	-280
.61	-164
.74	280
.92	-219
1.10	208
1.27	-164
1.45	157
1.63	-127
1.81	102
1.99	- 82
2.16	61
2.34	- 52
2.52	34
2.72	- 20

Bridge 4

Time Sec	Moment Ft-Lbs
.00	725
.03	479
.07	13
.11	-219
.15	-406
.19	-405
.28	86

*Moments ranged from 0 to -6 ft-lbs.

.30	126
.31	146
.35	279
.38	306
.41	213
.47	- 33
.50	-159
.56	-206
.61	-133
.70	133
.74	193
.77	159
.86	- 73
.92	-166
.97	- 99
1.05	100
1.10	146
1.15	93
1.23	- 86
1.27	-126
1.33	- 73
1.40	59
1.45	100
1.52	20
1.62	-100
1.80	73
1.97	- 66
2.15	46
2.33	- 46
2.51	20
2.71	- 26

Top Accel.

Time Sec	Accel. G
.01	-1.05
.02	- .50
.03	- .47
.04	- .50
.05	- .32
.07	- .02
.10	.24
.11	.22
.14	.46
.17	.54
.19	.52
.22	.30
.24	.23
.27	.01
.30	- .12
.33	- .27
.34	- .24
.36	- .40
.38	- .33
.39	- .32
.40	- .35
.42	- .18
.43	- .19
.44	- .12
.46	.03
.48	.09
.50	.21
.56	.27
.60	.20
.68	- .09
.74	- .22
.78	- .15
.87	.13
.92	.21
.97	.15
1.05	- .09
1.10	- .16
1.15	- .09
1.22	.10
1.27	.16
1.32	.11
1.40	- .05
1.46	- .12
1.50	- .08
1.53	- .03
1.63	.13
1.81	- .08
1.99	.09
2.17	- .04
2.35	.07
2.54	- .02
2.72	.05
2.92	- .01

Bottom Accel.

Time Sec	Accel. G
.01	.24
.01	.17
.02	.32
.02	.23
.03	.41
.03	.38
.04	.57
.06	1.11
.07	.54
.09	- .23
.11	- .80
.13	- .33
.15	.23
.17	- .45
.19	- .23
.20	- .24
.23	- .79
.26	.05
.27	- .02
.31	.94
.33	- .85
.36	.90
.37	- .13
.39	.66
.41	.06
.43	.67
.45	.00
.47	.35
.50	- .41
.52	- .23
.57	- .14
.60	- .31
.63	- .10
.64	- .18
.67	.19
.69	.10
.72	.26
.74	.21
.76	.27
.80	.25
.88	- .15
.91	- .15
.94	- .19
.96	- .16
.97	- .18
1.00	- .04
1.02	.00
1.05	.15
1.07	.14
1.10	.23
1.12	.21
1.14	.23
1.18	.11
1.22	- .06
1.26	- .15
1.28	- .12
1.30	- .15
1.33	- .10
1.35	- .09
1.42	.14
1.44	.15
1.47	.19
1.52	.14
1.64	- .10
1.82	.15
1.99	- .06
2.16	.10
2.34	- .02
2.50	.07
2.70	- .00
2.88	.04

Test 2-10-1

Bridge 1*

Bridge 2

Time Sec	Moment Ft-Lbs
.00	-13.2
.02	-13.6
.06	- 9.4
.09	2.2
.14	15.9
.18	15.9
.23	2.4
.26	- 8.2
.29	-13.2
.34	-13.2
.38	- 2.2
.42	10.0
.45	15.4
.50	14.5
.53	2.9
.57	- 8.0
.61	-12.7
.65	-11.4
.70	.2
.73	11.6
.77	15.4
.81	12.5
.88	- 7.1
.91	-11.2
.93	-12.3
.96	-11.2
.98	- 8.9
1.01	.2
1.04	11.2
1.07	13.8
1.09	14.5
1.11	12.3
1.15	3.1
1.18	- 5.3
1.21	- 9.8
1.24	-11.6
1.28	- 9.8
1.32	- .4
1.35	8.7
1.39	13.6
1.43	10.0
1.49	- 6.2
1.54	-11.2
1.55	-11.6
1.71	12.7
1.87	-11.2
2.01	12.0
2.17	-10.5
2.33	11.2
2.49	-10.5
2.64	10.3
2.79	- 9.8
2.95	9.4

Bridge 3

Time Sec	Moment Ft-Lbs
.00	-78.6
.01	-77.4
.10	43.5
.14	81.0
.17	75.0
.30	-73.8
.34	-59.2
.41	48.4
.45	78.6
.49	64.1
.57	-43.5
.61	-68.9
.65	-47.1
.77	75.0
.92	-62.9
1.08	70.1
1.23	-58.0
1.39	66.5
1.54	-54.4
1.70	64.1
1.85	-49.6
2.01	61.7
2.18	-48.4
2.32	58.0
2.47	-44.7
2.63	55.6
2.79	-42.3
2.94	52.0

Bridge 4

Time Sec	Moment Ft-Lbs
.00	-74.8
.15	70.7
.30	-67.6
.45	65.5
.61	-61.3
.76	61.3
.92	-58.2
1.07	58.2
1.23	-54.0
1.38	55.1
1.54	-50.9
1.70	53.0
1.85	-46.8
2.01	49.9
2.16	-43.6
2.32	47.8
2.47	-41.6
2.63	44.7
2.78	-39.5
2.94	41.6

Top Accel.

Time Sec	Accel. G
.02	-.28
.03	-.31
.04	-.24
.04	-.27
.07	.00
.07	.00
.08	.06
.09	.06
.12	.33
.14	.33
.16	.41
.18	.23
.20	.21
.23	-.03
.25	-.08
.28	-.31
.30	-.28
.31	-.35
.33	-.21
.36	-.16
.38	.06
.41	.13
.43	.34
.45	.32
.47	.36
.50	.20
.52	.16
.54	-.06
.57	-.15
.59	-.30
.61	-.27
.63	-.31
.65	-.18
.67	-.10
.70	.12
.72	.18
.75	.33
.77	.29
.78	.31
.83	.09
.91	-.28
.94	-.26
1.02	.15

*Moments ranged from -1 to 1 ft-lbs.

1.07	.31
1.13	.15
1.20	-.20
1.24	-.27
1.28	-.11
1.36	.24
1.39	.28
1.44	.15
1.51	-.21
1.53	-.23
1.54	-.24
1.70	.27
1.84	-.22
2.00	.26
2.15	-.21
2.31	.24
2.47	-.19
2.62	.22
2.78	-.18
2.93	.21

Bottom Accel.

Time Sec	Accel. G
.00	.00
.03	.08
.06	.02
.08	.04
.10	-.05
.12	-.01
.15	-.08
.17	-.03
.19	-.08
.22	-.01
.24	-.02
.27	.04
.29	.02
.31	.06
.33	.03
.35	.06
.38	.00
.40	.02
.43	-.05
.44	-.03
.47	-.06
.48	-.03
.51	-.07
.53	.00
.55	-.01
.58	.05
.60	.02
.62	.06
.64	.03
.66	.05
.69	.00
.71	.00
.74	-.05
.76	-.03
.78	-.05
.80	-.04
.81	-.05
.85	-.00
.86	-.00
.89	.04
.91	.03
.93	.05
.96	.03
.98	.04
1.01	.11
1.03	-.01
1.05	-.05
1.08	-.04
1.11	-.05
1.14	-.02
1.16	-.01
1.19	.02
1.21	.03
1.23	.04
1.25	.03
1.27	.04
1.29	.02
1.32	.01
1.35	-.02
1.38	-.04
1.40	-.04
1.42	-.05
1.45	-.01
1.47	-.00
1.50	.02
1.52	.02
1.53	.03
1.54	.04
1.58	.04
1.62	.00
1.65	-.03
1.69	-.04
1.71	-.04
1.73	-.04
1.76	-.01
1.81	.02
1.85	.04
1.89	.03
1.99	-.03
2.03	-.03
2.14	.03
2.19	.03
2.31	-.03
2.35	-.03
2.43	.02
2.47	.03
2.50	.03
2.63	-.03
2.79	.03
2.94	-.03

Test 2-10-2

Bridge 1*

Bridge 2

Time Sec	Moment Ft-Lbs
.00	21.0
.05	21.0
.09	18.0
.16	- 8.8
.22	-17.1
.28	-24.5
.33	-23.5
.38	-15.5
.46	8.1
.52	19.3
.62	20.5
.66	17.2
.71	8.1
.79	-17.4
.85	-23.0
.89	-23.0
.94	-15.9
1.06	15.1
1.12	20.3
1.18	19.8
1.23	16.7
1.27	9.1
1.35	-13.8
1.41	-22.0
1.46	-22.0
1.52	-11.8
1.59	7.0
1.63	15.0
1.71	19.2
1.79	17.3
1.85	6.0
1.93	-15.3
1.98	-20.9
2.02	-20.9
2.08	-14.2
2.18	9.0
2.22	16.0
2.29	19.1
2.37	16.7
2.50	-14.6
2.57	-20.4
2.64	-15.5
2.75	9.5
2.80	16.1
2.86	19.0
2.94	16.0
3.00	5.1

Bridge 3

Time Sec	Moment Ft-Lbs
.00	123.4
.04	117.3
.08	72.6
.17	- 37.5
.22	- 99.2
.28	-122.2
.34	-101.6
.39	- 53.2
.45	19.3
.49	71.3
.52	101.6
.57	116.1
.62	99.2
.66	56.8
.79	- 90.7
.85	-116.1
.90	- 99.2
.99	- 12.1
1.09	94.3
1.14	110.1
1.20	88.3
1.28	1.2
1.37	- 90.7
1.42	-110.1
1.48	- 91.9
1.52	- 54.4
1.70	107.6
1.99	-105.2
2.28	101.6
2.56	- 99.2
2.85	99.2

Bridge 4

Time Sec	Moment Ft-Lbs
.00	115.4
.05	93.6
.11	26.0
.20	- 66.5
.25	- 97.7
.28	-106.0
.32	- 93.6
.38	- 50.9
.57	108.1
.85	-100.8
1.13	101.9
1.42	- 94.6
1.52	- 43.6
1.71	98.8
1.99	- 89.4
2.28	93.6
2.56	- 85.2
2.89	88.4

Top Accel.

Time Sec	Accel. G
.07	-.08
.18	.08
.24	.15
.28	.18
.35	.13
.42	.02
.51	-.11
.57	-.14
.62	-.11
.77	.11
.84	.17
.93	.12
1.07	-.08
1.13	-.13
1.21	-.08
1.37	.13
1.42	.16
1.49	.12
1.53	.08
1.64	-.07
1.71	-.13
1.78	-.08
2.00	.15
2.29	-.12
2.57	.14
2.86	-.11

Bottom Accel.

Time Sec	Accel. G
.00	.00
.02	.011
.03	.011
.03	.007
.05	.038
.08	.010
.11	.028
.13	-.002
.16	.010
.19	-.024
.21	-.008
.24	-.039
.27	-.019
.29	-.037

*Moments ranged from -1 to 1 ft-lbs.

.32	-.020	.50	.009	.67	.022	1.52	-.023
.34	-.037	.53	.029	.70	.002	1.72	.023
.37	-.017	.55	.022	.73	.007	2.01	-.026
.39	-.024	.57	.022	.76	-.018	2.29	.021
.42	.00	.59	.015	1.89	-.028	2.59	-.024
.45	-.007	.62	.032	1.16	.023	2.86	.019
.48	.020	.65	.014	1.45	-.027		

Test 2-8-1

Bridge 1*

Bridge 2

Time Sec	Moment Ft-Lbs
.00	-13.6
.02	-13.6
.04	- 8.5
.10	3.8
.12	3.8
.14	2.4
.16	3.3
.18	.4
.23	- 8.9
.26	-10.9
.28	- 8.2
.33	.8
.36	4.0
.38	3.5
.40	4.0
.43	1.1
.48	- 7.1
.51	- 9.8
.54	- 7.1
.58	1.5
.61	3.8
.63	4.4
.66	3.3
.75	- 8.2
.88	4.0
1.00	- 7.1
1.12	3.8
1.24	- 6.0
1.37	3.5
1.49	- 5.3
1.52	- 4.4
1.62	2.9
1.74	- 4.9
1.86	2.6
1.98	- 4.7
2.10	2.2
2.23	- 4.4
2.35	2.0
2.47	- 4.2
2.59	2.0
2.71	- 3.8
2.84	1.7
2.95	- 3.8

Bridge 3

Time Sec	Moment Ft-Lbs
.00	45.5
.02	45.5
.04	29.5
.08	- 46.6
.10	- 93.3
.13	-111.5
.15	-100.1
.18	- 37.5
.20	3.4
.22	23.8
.26	37.5
.29	20.4
.32	- 13.6
.36	- 81.9
.38	-101.2
.41	- 77.3
.45	- 3.4
.47	15.9
.51	33.0
.55	11.3
.59	- 59.1
.62	- 88.7
.66	- 64.8
.70	2.2
.73	20.4
.75	27.3
.79	19.3
.81	- 1.1
.85	- 63.7
.88	- 77.3
.90	- 63.7
.96	7.9
1.00	22.7
1.04	7.9
1.09	- 51.2
1.12	- 67.1
1.15	- 47.7
1.21	10.2
1.25	19.3
1.29	7.9
1.34	- 45.5
1.36	- 58.0
1.39	- 45.5
1.43	- 2.2
1.47	14.7
1.49	18.2
1.52	14.7
1.61	- 53.4
1.74	15.9
1.86	- 47.7
1.98	14.7
2.10	- 43.2
2.22	13.6
2.34	- 39.8
2.47	13.6
2.59	- 36.4
2.71	13.6
2.83	- 34.1
2.95	12.5

Bridge 4

Time Sec	Moment Ft-Lbs
.00	151.8
.01	153.9
.06	- 20.8
.10	-116.4
.12	-137.2
.16	-110.2
.19	- 20.8
.23	116.4

*Moments ranged from 0 to -3 ft-lbs.

.25	143.5
.27	119.6
.31	- 21.8
.35	-104.0
.38	-126.8
.40	-111.2
.48	97.7
.50	128.9
.52	101.9
.59	- 83.2
.62	-116.4
.65	- 93.6
.75	113.3
.87	-106.0
1.00	100.8
1.12	- 97.7
1.24	89.4
1.36	- 88.4
1.49	80.0
1.52	52.0
1.61	- 83.2
1.74	72.8
1.85	- 78.0
1.88	66.5
2.10	- 72.8
2.22	62.4
2.34	- 68.6
2.46	57.2
2.59	- 65.5
2.71	53.0
2.83	- 62.4
2.95	48.8

Top Accel.

Time Sec	Accel. G
.02	-.63
.03	.80
.05	.16
.06	.12
.08	-.40
.09	-.46
.12	-.70
.14	-.59
.16	-.56
.19	-.13
.20	-.06
.22	.47
.23	.55
.25	.86
.26	.77
.27	.79
.35	-.58
.38	-.56
.40	-.60
.43	-.22
.49	.69
.51	.75
.55	.10
.59	-.38
.61	-.52
.63	-.56
.65	-.49
.70	.01
.75	.66
.88	-.49
1.00	-.38
1.12	-.45
1.15	-.33
1.23	.45
1.25	.52
1.26	.44
1.36	-.39
1.49	.45
1.52	.32
1.62	-.36
1.74	.41
1.86	-.33
1.98	.38
2.10	-.31
2.22	.35
2.34	-.28
2.47	.33
2.59	-.27
2.71	.30
2.83	-.26
2.95	.28

Bottom Accel.

Time Sec	Accel. G
.00	.00
.01	-.010
.03	.049
.04	.018
.05	.058
.07	.014
.09	.037
.11	-.059
.12	-.043
.15	-.115
.18	-.030
.19	-.039
.21	.030
.22	.017
.24	.043
.25	.030
.27	.046
.28	.034
.30	.049
.31	.028
.32	.039
.35	-.020
.36	-.020
.39	-.089
.42	-.055
.43	-.048
.47	.017
.48	.030
.49	.025
.51	.039
.52	.033
.54	.039
.55	.033
.56	.039
.58	.009
.60	-.009
.63	-.055
.65	-.065
.67	-.046
.71	.013
.75	.028
.78	.031
.81	.028
.84	.005
.88	-.046
.90	-.060
.92	-.041
.97	.018
1.02	.027
1.05	.030
1.07	.016
1.11	-.037
1.14	-.046
1.17	-.033
1.21	.010
1.29	.025
1.32	.009
1.36	-.035
1.39	-.045
1.41	-.034
1.46	.010
1.52	.021
1.54	.024
1.56	.013
1.61	-.027
1.63	-.037
1.66	-.030
1.71	.013
1.78	.023
1.81	.010
1.88	-.034
1.97	.016
2.03	.020
2.12	-.030
2.20	.013
2.27	.018
2.37	-.028
2.51	.016
2.57	.017
2.61	-.025
2.73	.016
2.89	-.025
2.97	.016

Test 2-8-2

Bridge 1*

Bridge 2+

Bridge 3

Time Sec	Moment Ft-Lbs
.00	101.2
.08	48.9
.13	- 43.2
.18	-158.1
.24	-213.9
.32	-145.6
.37	- 13.6
.50	92.1
.59	29.5
.66	-116.0
.74	-198.0
.82	-124.0
.88	7.9
.95	69.4
.99	85.3
1.05	64.8
1.12	- 36.4
1.17	-129.7
1.23	-178.6
1.30	-124.0
1.38	15.9
1.43	60.3
1.48	78.5
1.51	76.2
1.73	-162.7
1.98	70.5
2.21	-143.3
2.47	62.5
2.69	-128.5
2.95	55.7

Bridge 4

Time Sec	Moment Ft-Lbs
.00	310.9
.03	267.2
.12	- 53.0
.18	-194.4
.24	-239.2
.30	-195.5
.38	.0
.45	216.3
.49	282.8
.54	215.2
.60	7.2
.66	-158.0
.73	-222.5
.82	-147.6
.87	- 11.4
.95	215.2
.98	261.0
1.02	-215.2
1.09	- 7.2
1.15	148.7
1.23	206.9
1.30	147.6
1.37	- 7.2
1.42	-163.2
1.47	-234.0
1.50	-215.2
1.71	192.4
1.78	147.6
1.84	12.4
1.91	-163.2
1.96	-210.0
2.01	-163.2
2.20	177.8
2.45	-186.1
2.68	163.2
2.93	-167.4

Top Accel.

Time Sec	Accel. G
.06	.33
.12	-.02
.16	-.22
.24	-.33
.33	-.20
.43	.30
.49	.47
.56	.29
.64	-.16
.74	-.31
.82	-.20
.94	.35
.98	.43
1.03	.33
1.15	-.19
1.23	-.28
1.33	-.13
1.42	.28
1.48	.39
1.50	.36
1.53	.26
1.71	-.26
1.96	.35
2.21	-.23
2.44	.32
2.69	-.21
2.93	.28

Bottom Accel.

Time Sec	Accel. G
.00	.00
.05	.021
.16	.028
.22	-.070
.24	-.054
.27	-.062
.30	-.089
.36	.020
.67	.017
.71	-.065
.74	-.049
.78	-.073
.82	-.044
.85	.018
.87	.00
.89	.029
.91	.005
.93	.019
.94	.007
.96	.023
.97	.013
.99	.023
1.07	.016
1.09	.023
1.11	.006
1.13	.018
1.15	.007
1.17	.015
1.20	-.053
1.23	-.046
1.26	-.070
1.29	-.050
1.31	-.045
1.33	.012
1.35	-.008
1.37	.026
1.39	.00
1.41	.019
1.43	.005
1.44	.016
1.46	.012
1.50	.018
1.61	.013

*Moments ranged from 0 to -3 ft-lbs.

+Moments not obtained.

1.63	-.005	1.83	-.014	2.16	-.019	2.47	.018
1.65	.013	1.85	.022	2.21	-.051	2.58	.004
1.69	-.039	1.87	-.002	2.26	-.047	2.69	-.041
1.71	-.039	1.98	.020	2.29	-.004	2.74	-.040
1.74	-.066	2.07	.005	2.31	-.016	2.80	-.004
1.77	-.046	2.09	.014	2.33	.013	2.93	.012
1.79	-.045	2.12	-.005	2.35	-.001		
1.81	.002	2.14	.007	2.37	.015		

Test 2-6-1

Bridge 1

Time Sec	Moment Ft-Lbs
.00	.0

Bridge 2

Time Sec	Moment Ft-Lbs
.00	26.0
.13	-22.3
.25	24.8
.41	-22.4
.53	24.0
.67	-20.5
.81	22.5
.95	-19.5
1.08	21.1
1.21	-19.1
1.36	20.0
1.50	-18.8
1.65	19.0
1.79	-17.4
1.91	17.2
2.05	-16.3
2.19	16.4
2.33	-15.2
2.47	15.0
2.60	-13.0
2.75	13.9

Bridge 3

Time Sec	Moment Ft-Lbs
.00	177.8
.04	110.1
.09	- 84.7
.11	-156.0
.13	-169.4
.16	-150.0
.20	.0
.23	130.6
.26	171.8
.29	166.9
.34	21.7
.39	-160.9
.42	-166.9
.47	.0
.51	131.8
.54	169.4
.58	119.7
.62	.0
.67	-159.7
.69	-159.7
.75	- 1.2
.78	119.7
.82	162.1
.85	130.6
.89	10.8
.93	-134.3
.96	-156.0
.99	-121.0
1.02	.0
1.09	154.8
1.23	-151.2
1.37	148.8
1.51	-147.6
1.65	143.9
1.79	-142.7
1.92	135.5
2.06	-136.7
2.20	130.6
2.34	-129.4
2.48	124.6
2.61	-122.2
2.75	117.3
2.89	-116.1
3.03	112.5

Bridge 4

Time Sec	Moment Ft-Lbs
.00	169.5
.07	- 7.2
.10	-121.6
.13	-158.0
.16	-119.6
.19	- 16.6
.23	119.6
.26	160.1
.30	118.5
.34	- 7.2
.39	-137.2
.41	-148.7
.47	14.5
.51	116.4
.54	152.8
.57	116.4
.62	- 17.6
.67	-136.2
.69	-137.2
.75	14.5
.78	116.4
.82	145.6
.85	116.4
.89	- 7.2
.93	-110.2
.95	-133.1
.98	-110.2
1.09	137.2
1.23	-124.8
1.37	132.0
1.51	-118.5
1.64	124.8
1.78	-113.3
1.92	121.6
2.06	-107.1
2.19	115.4

2.33	-100.8
2.47	111.2
2.61	- 96.7
2.75	106.0
2.88	- 91.5
3.02	100.8

<u>Top Accel.</u>

Time Sec	Accel. G
.01	.72
.02	.84
.03	.64
.03	.68
.05	.23
.06	.29
.08	.29
.10	- .29
.10	- .61
.11	- .63
.12	- .73
.13	- .76
.15	- .74
.17	- .79
.21	- .41
.25	.27
.26	.79
.27	.76
.30	.99
.31	.66
.34	.00
.35	.02
.36	- .50
.37	- .63
.39	- .56
.40	-1.00
.42	- .68
.43	- .75
.45	- .31
.46	- .15
.49	.39
.50	.51
.53	.82
.54	.76
.56	.87
.57	.58
.58	.56
.63	-.04
.66	-.62
.67	-.62
.68	-.90
.70	-.64
.71	-.52
.78	.55
.82	.79
.92	.62
.96	-.45
.99	-.48
1.02	.05
1.06	.55
1.08	.67
1.10	.77
1.14	.53
1.16	.11
1.20	-.41
1.23	-.71
1.27	-.40
1.30	.00
1.33	.54
1.35	.64
1.37	.75
1.39	.64
1.41	.56
1.44	.11
1.47	-.41
1.51	-.66
1.55	-.41
1.59	.10
1.61	.48
1.65	.70
1.69	.46
1.75	-.41
1.79	-.63
1.82	-.41
1.93	.65
2.07	-.59
2.20	.63
2.34	-.55
2.48	.60
2.62	-.52
2.76	.57
2.89	-.48

<u>Bottom Accel.</u>

Time Sec	Accel. G
.03	.151
.03	.135
.04	.146
.05	.104
.06	.111
.07	.047
.09	-.021
.11	-.028
.13	-.085
.14	-.060
.17	-.101
.20	-.034
.21	-.043
.23	.059
.25	.073
.27	.149
.28	.088
.30	.184
.33	.059
.34	.091
.37	-.068
.39	-.056
.40	-.108
.42	-.077
.43	-.099
.44	-.065
.46	-.096
.48	-.013
.49	-.021
.52	.095
.53	.082
.55	.139
.57	.109
.58	.151
.61	.057
.62	.071
.65	-.031
.66	-.015
.67	-.088
.69	-.043
.71	-.091
.73	-.059
.74	-.066
.76	.002
.78	.025
.80	.102
.85	.133
.89	.078
.92	-.005
.95	-.070
.97	-.054
.99	-.081
1.01	-.046
1.03	-.046
1.05	.027
1.06	.042
1.09	.118
1.11	.111
1.12	.134
1.15	.083
1.17	.067
1.19	-.004
1.20	-.016
1.23	-.065
1.25	-.053
1.27	-.078
1.28	-.046
1.30	-.043
1.32	.017
1.34	.040
1.37	.110
1.38	.109
1.40	.125
1.42	.093
1.44	.072
1.50	-.057
1.53	-.056
1.55	-.069
1.65	.117
1.80	-.065
1.95	.112
2.09	-.060

2.23	.105	2.50	.100	2.78	.096	3.06	.091
2.37	-.055	2.64	-.049	2.92	-.046		

Test 2-6-2

Bridge 1

Time Sec	Moment Ft-Lbs
.00	.0

Bridge 2*

Bridge 3

Time Sec	Moment Ft-Lbs
.00	244.4
.04	229.9
.12	35.0
.16	-102.8
.20	-174.2
.23	-200.8
.26	-204.4
.29	-196.0
.31	-175.4
.37	10.8
.43	162.1
.47	221.4
.50	232.3
.54	216.5
.57	164.5
.63	9.6
.69	-160.9
.73	-196.0
.76	-200.8
.80	-194.8
.82	-165.7
.88	20.5
.93	147.6
.98	216.5
1.02	225.0
1.05	199.6
1.09	131.8
1.15	- 24.2
1.19	-145.2
1.23	-186.3
1.27	-197.2
1.31	-183.9
1.33	-150.0
1.39	10.8
1.46	166.9
1.48	205.7
1.52	219.0
1.55	203.2
1.57	171.8
1.78	-193.6
2.02	210.5
2.28	-188.7
2.53	200.8
2.79	-183.9
3.04	192.3

Bridge 4

Time Sec	Moment Ft-Lbs
.00	235.0
.03	202.8
.09	89.4
.15	- 74.8
.19	-191.3
.24	-244.4
.28	-213.2
.34	- 73.8
.40	80.0
.44	164.3
.50	217.3
.56	168.4
.63	7.2
.70	-160.1
.72	-220.4
.76	-234.0
.79	-202.8
.86	- 22.8
.93	134.1
.97	185.1
1.01	204.8
1.04	188.2
1.08	142.4
1.15	- 21.8
1.23	-201.7
1.26	-222.5
1.29	-197.6
1.37	- 21.8
1.43	113.3
1.47	174.7
1.51	193.4
1.76	-212.1
2.02	186.1
2.27	-201.7
2.53	176.8
2.78	-188.2
3.04	170.5

Top Accel.

Time Sec	Accel. G
.00	.40
.09	.21
.15	-.05
.19	-.29
.24	-.39
.27	-.36
.36	-.01
.41	.18
.46	.31
.50	.36
.55	.30
.61	.13
.67	-.14
.70	-.27
.76	-.36
.80	-.26
.87	.01
.91	.17
.96	.29
1.01	.34
1.07	.26
1.14	.01
1.20	-.21
1.26	-.34
1.32	-.22
1.38	.01
1.43	.18
1.47	.28
1.52	.32
1.77	-.32
2.03	.31
2.28	-.31
2.54	.30
2.79	-.29
3.04	.29

Bottom Accel.

Time	Accel.

*Moments not obtained

Sec	G	.32	-.062	.67	-.037	1.06	.061
		.35	-.029	.68	-.018	1.09	.028
.00	.00	.37	-.030	.70	-.061	1.10	.038
.01	.027	.39	-.004	.72	-.026	1.13	.00
.02	.015	.41	-.004	.74	-.046	1.15	.007
.05	.085	.44	.025	.75	-.027	1.17	-.025
.06	.079	.46	.028	.77	-.037	1.18	-.014
.08	.020	.48	.075	.79	-.025	1.21	-.049
.10	.046	.50	.030	.83	-.063	1.22	-.030
.13	-.017	.52	.044	.86	-.020	1.24	-.044
.17	-.061	.53	.034	.87	-.038	1.29	-.026
.19	-.022	.56	.083	.89	.001	1.33	-.064
.21	-.042	.58	.014	.91	-.013	1.36	-.020
.22	-.028	.60	.049	.94	.034	1.38	-.040
.24	-.046	.62	-.015	.99	.067	1.40	.00
.29	-.031	.64	.016	1.01	.037		

Test 1-8-1

Bridge 1

Time Sec	Moment Ft-Lbs
.00	.0

Bridge 2

Time Sec	Moment Ft-Lbs
.00	-.3

Bridge 3

Time Sec	Moment Ft-Lbs
.00	- 5.6
.05	- 2.0
.08	1.7
.11	- 4.4
.13	-14.8
.16	-57.4
.18	-57.4
.21	-62.7
.25	-62.7
.28	-47.0
.32	- .2
.34	.0
.43	- 4.1
.50	.5
.52	1.1
.55	- 7.1
.56	- 6.5
.57	- 9.4
.59	-52.0
.62	-50.6
.64	-61.2
.67	-58.3
.68	-59.4
.69	-54.7
.73	-30.7
.75	- 3.2
.77	- .5
.88	- 3.2
.94	.0
.97	.5
.99	- 6.5
1.01	- 6.5
1.04	-44.9
1.06	-42.6
1.09	-56.5
1.11	-47.9
1.13	-49.1
1.17	-21.6
1.19	-11.5
1.21	- .2
1.32	- 1.4
1.41	.5
1.43	- 1.1
1.44	- 5.9
1.45	- 5.3
1.46	- 5.9
1.49	-37.5
1.53	-34.6
1.54	-38.1
1.87	1.1
2.00	-35.2
2.30	.8

Bridge 4

Time Sec	Moment Ft-Lbs
.00	160.2
.03	142.3
.06	92.1
.10	- 33.1
.13	- 84.1
.16	-172.7
.18	-187.0
.20	-247.9
.22	-216.5
.23	-221.0
.27	-137.8
.33	11.6
.34	11.6
.37	90.3
.39	102.0
.41	140.5
.43	136.0
.44	144.9
.46	128.8

.48	117.2
.52	30.4
.54	- 40.2
.56	- 60.8
.60	-150.3
.61	-151.2
.64	-197.7
.66	-172.7
.68	-170.9
.74	- 66.2
.77	16.1
.78	8.9
.81	78.7
.82	85.0
.83	81.4
.85	121.7
.86	115.4
.87	112.7
.88	124.4
.90	105.6
.91	97.5
.94	41.1
.96	20.5
.98	- 34.9
1.00	- 47.4
1.04	-124.4
1.06	-125.3
1.08	-156.6
1.11	-135.1
1.13	-127.9
1.18	- 60.8
1.19	- 35.8
1.22	19.6
1.23	11.6
1.26	75.1
1.27	71.6
1.29	103.8
1.31	93.0
1.33	102.9
1.35	75.1
1.36	76.9
1.39	25.0
1.41	7.1
1.43	- 37.5
1.45	- 46.5
1.49	-102.9
1.51	-102.9
1.53	-114.5
1.55	-124.4
1.75	87.7
2.00	- 97.5
2.22	66.2
2.44	- 72.4
2.66	48.3
2.89	- 58.1

Top Accel.

Time Sec	Accel. G
.06	-.22
.07	-.15
.10	.04
.11	.02
.14	.23
.16	.19
.17	.23
.18	.38
.20	.32
.22	.43
.24	.29
.26	.31
.29	.14
.32	.07
.35	-.14
.37	-.11
.40	-.32
.41	-.27
.43	-.41
.45	-.32
.46	-.36
.49	-.19
.49	-.20
.52	.00
.55	.03
.56	.18
.58	.21
.59	.15
.61	.20
.62	.33
.64	.26
.66	.35
.69	.22
.71	.23
.74	.09
.75	.06
.76	.07
.79	-.13
.80	-.09
.82	-.12
.84	-.28
.85	-.22
.87	-.32
.89	-.23
.90	-.28
.93	-.13
.94	-.15
.97	.03
.99	.03
1.01	.17
1.03	.18
1.03	.12
1.05	.16
1.06	.27
1.07	.27
1.09	.21
1.10	.28
1.12	.26
1.15	.17
1.16	.18
1.20	.02
1.21	.05
1.24	-.11
1.26	-.08
1.28	-.23
1.30	-.17
1.31	-.27
1.33	-.17
1.35	-.22
1.37	-.09
1.39	-.10
1.42	.04
1.44	.03
1.46	.15
1.47	.16
1.49	.11
1.50	.16
1.53	.23
1.77	-.21
1.98	.18
2.23	-.13
2.34	.14
2.67	-.07
2.89	.10

Bottom Accel.

Time Sec	Accel. G
.00	.00
.03	.052
.05	.018
.06	.052
.08	.008
.10	.030
.11	-.014
.12	-.004
.13	-.004
.14	.098
.17	-.113
.19	.022
.21	-.089
.23	.008
.26	-.089
.28	-.004
.30	-.078
.34	.115
.36	-.042
.37	.055
.38	.043
.39	.061
.40	-.022
.42	.075
.43	-.004
.44	.030
.45	.065
.47	.00
.49	.052
.50	.057
.51	-.002

.53	.054	.80	-.057	1.02	.102	1.30	.059
.55	-.061	.81	.047	1.05	-.104	1.32	-.015
.58	.120	.83	.063	1.08	.044	1.32	.061
.60	-.123	.84	-.020	1.10	-.086	1.36	-.013
.63	.044	.86	.065	1.12	.016	1.38	.057
.65	-.094	.88	-.011	1.15	-.078	1.40	-.019
.67	.014	.90	.061	1.18	.029	1.42	.042
.70	-.088	.91	-.004	1.20	-.037	1.44	-.051
.73	.015	.93	.053	1.23	.103	1.47	.087
.75	-.058	.95	-.007	1.25	-.061	1.50	-.088
.78	.120	.97	.049	1.27	.065		
.79	-.014	.99	-.057	1.28	-.019		

Test 1-8-2

Bridge 1		Bridge 2		Bridge 3		Bridge 4	
Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs	Time Sec	Moment Ft-Lbs
.00	.0	.00	-.7	.00	-8.5	.00	178.4