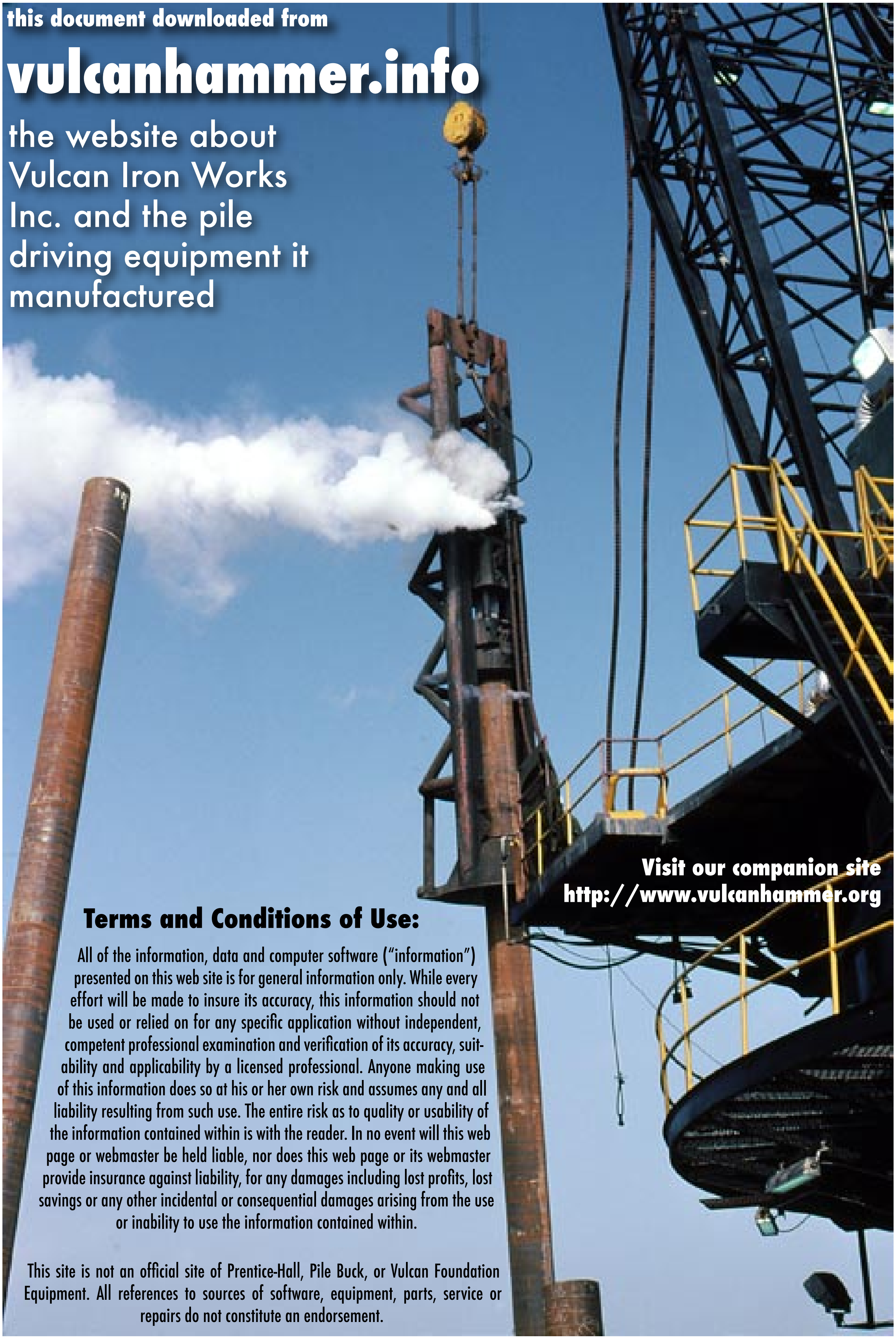

CENTRIFUGAL MODELING OF THE DYNAMIC RESPONSE OF PILES

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ABSTRACT

The dynamic response of laterally loaded single piles and pile groups (each consisting of four evenly-spaced piles, and spaced at different distances in each group), embedded in loose, dense, dry and saturated sands, is studied using the centrifugal modeling technique. The response of single piles and pile groups to forced vibrations was found to depend strongly on the magnitude and frequency of loading as well as the density of the soils. The results indicate that as the level of force increased: (1) nonlinear softening behavior was evidenced by a decrease in the resonant frequency of the soil-pile system, (2) there was an increase in internal soil-pile damping, and (3) the maximum bending moment moved progressively deeper below the soil surface and increased substantially in magnitude. Also, significant interaction effects were observed with close pile spacing. Finally, the experimental stiffness and damping results were compared with theoretical values as predicted by Novak's work.

INTRODUCTION

The prediction of pile response to horizontal, time dependent loadings such as caused by earthquakes, machine vibrations, wind or wave actions, is one of the most challenging problems in foundation engineering. Reliable predictions are made difficult by the complex interaction between the elastic pile and the inelastic soil. A substantial number of theoretical studies have been conducted in recent years to determine the dynamic response of piles subjected to horizontal or vertical loads. Most of these studies are limited, however, to the consideration of an elastic soil and perfect bonding between the pile and the surrounding medium, hypotheses which may be valid for small levels of excitation. When dealing with large amplitude forces and displacements, nonlinear soil behavior and nonlinear effects such as separation and gapping must be taken into account. Despite the very few nonlinear studies (see, e.g., Ref. 1), it seems that experimental studies will be the only sure way to explore these nonlinear effects. Traditionally, designers have thus had to rely upon full-scale tests as the principal source of design information. These tests require much time and capital for successful completion. Further, with respect to their theoretical utility, full-scale tests often yield data of insufficient generality due to the inherent inhomogeneity of in-situ soil strata. In this paper, the dynamic response of laterally loaded single piles and pile groups is studied using the centrifugal modeling technique and the experimental results are compared with theoretical results anticipated by well established theories [3,4].

A centrifuge can simulate gravity-induced stresses in soil deposits at a reduced geometrical scale through centrifugal modeling. Conceptually, the technique consists of increasing the confining stress in the model soil so that it is identical to the confining stress in the prototype soil at homologous points. Hence, the stress-strain behavior of any horizontal soil layer in the model is the same as that of the homologous layer in the prototype. The technique allows soil-structure interaction tests to be performed at a conveniently reduced scale, and provides data applicable to full-scale problems. Further, the tests can be performed on any particular soil type and/or deposit, and for any structure configuration. The modeling technique leads to a set of scaling relationships, or scaling laws, that affect time, physical dimensions, and the many derivatives of these combinations such as velocity, acceleration, force, etc. These scaling relations have been discussed in the literature (see e.g., [12] and [13]), and the relations between the quantities of interest here have been listed in [6]. The technique promises to be an invaluable aid for studying a variety of complex geotechnical problems (see, e.g., [10] and [13]) and in particular, for studying dynamic soil-structure interaction problems (see, e.g., [2], [6], [7], [9], [11], [12]).

EQUIPMENT AND INSTRUMENTATION

A. Centrifuge

The centrifuge used is a Model 1230-1 Genesco "G-accelerator" hydraulically driven; for detailed description see Ref. 6.

B. Piles and Deformation Sensors

The pile models used for the experiments were made from 0.2188 inches

O.D., 0.0156 inches wall thickness 3003 H-14 aluminum alloy tubing (yield strength = 21 x 10^3 psi; modulus of elasticity = 10 x 10^6 psi). Eight model piles with the same diameter but different lengths to achieve various embedment depths were used for the experiments as summarized in Table 1. Two piles of type No. 1 were gauged with 10 micro-measurement strain gauges located at 0.0799, 0.572, 1.06, 1.56 and 2.35 inches below the soil surface. To increase the strain reading, two gauges were glued at each depth on opposite sides of the pile and wired into adjacent positions in a common bridge. The gauged piles were coated with micro-measurement "M-bound" protective coating. The piles had a flexural rigidity, EI, of approximately 410 lbs-in^2 (based on an average of the values measured experimentally from dynamic and static calibration tests).

The single piles were mounted with a cap which contained a coil and an accelerometer, and weighed 0.0397 lbs. Four piles of type No. 1 were rigidly mounted on a 3/16 inch thick aluminum mat to simulate a pile group. The mat, a 5 x 5 inches square, was designed to conveniently accommodate three separate pile groups of different pile spacings as shown in Fig. 1. The three pile spacings selected for these experiments were 1.0, 2.5 and 4.0 inches to achieve S/D ratios (pile spacing/diameter) of 4.56, 11.4 and 18.3, respectively.

Displacements of the pile caps were obtained by integrating twice the reading of the Kistler 811 picotron miniature accelerometer attached to the pile cap. Occasionally, the accelerometer output was amplified before recording by a Sensotec SA-B transducer amplifier calibrated to 140 gain.

C. Signal Conditioning and Data Acquisition

All strain gauges, coil and accelerometer signals were recorded on a 4-channel Norland 3001 Digital Oscilloscope with accompanying 3106 Monitor which acquires waveform data, stores the data in its digital memory and can display the waveforms on a CRT screen. The waveforms were stored on a Norland 2701-R flexible disk permanent storage system. For each test, the time scale and voltage range were specially selected for each waveform in order to optimize the resolution of the signals and to assure a minimum of five cycles in the display. After processing, the results were plotted on a Hewlett-Packard 7045A X-Y plotter.

D. Forced Vibration Device

An electromagnetic "shaker" [6] was used to vibrate the piles, in the lateral direction, over a broad range of frequencies. The device consists of two stationary electromagnets mounted on an aluminum frame and a smaller dumbell-shaped coil coaxially located between the two larger electromagnets and mounted to the pile cap. Details about the design are reported in [2,7,9]. The system is not mechanically operated and is independent of the high g's created in the centrifuge. Further, the system has high frequency capabilities (over 4 kHz), allowing investigation of high vibration mode responses.

E. Shaker Calibration

An important part of the experimental investigation was to develop an adequate procedure for calibrating the coil and establishing an accurate relationship between the output voltage of the coil and the resulting applied force to the pile cap. The calibration was achieved by vibrating (at 1 g) a cantilevered pile clamped at the bottom end and with its upper end (mounted with the pile cap) held between the poles of the stationary magnets. A constant current of 250 mA was passed through the two electromagnets and the coil-magnet spacing was maintained at 0.1 inch. The input current to the coil was then increased by 50 mA increments for each frequency of interest. The resulting force amplitude vs coil output voltage was obtained by simple analysis of the vibrating system.

F. Soil Tested

A fine uniform silica sand (Ottawa sand) with a mean grain diameter of 0.008 inches was used throughout the experiments. Various density and water content of the deposit (about 9 inches deep) were used:

1. Loose, dry sand: prepared by pouring the sand into the bucket and smoothing the surface.
2. Dense, dry sand: the top five inches of the deposit were placed in 0.25 to 0.5 inch layers which were tamped with a 2 lb weight.
3. Medium dense, saturated sand: the sand was prepared as in (2) above, except that the water table was brought to the current soil surface after each new layer had been tamped.

Finally, each soil deposit was allowed to consolidate at 100 g's for about one hour before the experiments were started.

FORCED VIBRATION TEST RESULTS

A. Response Curves: Single Piles

Typical response curves of acceleration amplitudes (absolute values; peak to peak) versus exciting frequencies for piles 1 at three load levels (load level 3 is the largest) are shown in Fig. 2. First and second mode resonances are easily discernible. The high sharp peaks of first mode response indicate that there is little damping associated with that mode. Accelerations, frequencies and damping values (calculated using the one-half-power points method), are presented in Table 2 for the first, second, and third resonance modes. The data show that:

1. The natural frequencies decrease with increasing loads, which is a typical behavior of a softening dynamical system. It is an evidence of soil nonlinearity and is caused by a decrease in pile-soil stiffness with an increase in dynamic stress and strain.
2. The damping at resonance increases with increasing load (at both the first and second modes), another evidence of soil nonlinearity, and decreases with increasing frequency. Further, an important qualitative observation about the damping mechanism can be made from Fig. 2. As clearly displayed in Fig. 2, the positive slope of the response curve around the first mode is far steeper than the negative slope. This implies that damping is higher just above the first natural frequency than below. This result is consistent with a mechanism proposed by Novak [4].

The strain data measured along the length of the pile were converted to moment values and are displayed in Fig. 3 for the first and second resonance modes, respectively. As the load increases, it is observed that the maximum bending moment moves progressively deeper below the soil surface, and increases substantially in magnitude

B. Response Curves: Group Piles

Accelerations, frequencies and damping values are presented in Table 4 for the first and second modes at two load levels for three pile groups consisting of 4 piles each at different spacings. It is of interest to note that:

1. The pile groups with 4.0 and 2.5 inches spacings display stiffness characteristics similar to those observed for the single piles, i.e., the natural frequencies decrease with increasing load amplitude. However, the third pile group with a 1.0 inch spacing displays behavior radically different. As indicated in Table 4, an increase in load level induces a marked increase in natural frequency in that case, i.e., the system stiffens with increasing load. This behavior is in sharp contrast to the behavior observed for the other two groups and is in accordance with the theoretical

results of Poulos [5] which predict significant interaction effects for low S/D ratios.

2. The damping values clearly reflect three trends: an increase in damping with increasing load level, a decrease in damping with decreasing spacing between the piles, and an increase in damping with higher mode responses. All these observations clearly suggest a highly nonlinear pile-soil behavior.

Bending moments at first resonance along the piles for various spacing are shown in Fig. 4. It is observed that the maximum moment decreases sharply with decreasing pile spacing.

C. Parametric Study: Single Piles

In these tests the effects of slenderness ratio L/r_0, soil density, water content, load level and excitation frequency on a single pile's dynamic stiffness and damping characteristics were investigated. For convenience, the tests were performed at excitation frequencies between the first and second resonances (of Fig. 2) of 400, 500, 600, 700, 800, and 900 Hz, and at three load levels (see Ref. 2 for details). Seven piles (Piles 2 - 8) with different slenderness ratios were tested as summarized in Table 1. These tests were performed in loose dry sand deposits. However, Pile #4 was also tested in dry dense and saturated medium dense sand deposits. Typical results are shown in Fig. 5 which shows the hysteresis loops measured at 900 Hz excitation frequency at three load levels for Pile #4 embedded in a dry dense sand deposit. From such loops the equivalent stiffness and damping characteristics of each pile, at each load level, and for each excitation frequency were determined.

The objective of this parametric study was to compare the experimental results with theoretical values as predicted by well established techniques. For that purpose Novak's computer program "PILAY" [3] was used to obtain theoretical values for comparison with the test results. To facilitate the comparisons, all quantities involved were non-dimensionalized as suggested by Novak [4], and the following non-dimensionalized form of frequency, stiffness and damping were adopted:

$$a_0 = \omega r_0/V_s \quad , \tag{1}$$

$$f_{u1} = k \frac{r_0^3}{E_p I_p} \quad , \tag{2}$$

$$f_{u2} = C \frac{r_0^2 V_s}{E_p I_p} \quad , \tag{3}$$

where $E_p I_p$ is the bending stiffness of the pile; V_s is the shear wave velocity in soil; r_0 is the outer radius of the pile, C is the equivalent viscous damping, k is the equivalent stiffness, and ω is the frequency (in rad/sec).

The shear wave velocities were computed from the following formula [8].

$$V_s = [170 - (78.2)e]\sigma_0^{0.25} \quad , \tag{4}$$

in which $\sigma_0 = K_0 \sigma_v$ and $\sigma_v = \gamma_s z$ (5)

where z = depth below surface; γ_s = unit weight of the soil; K_0 = lateral stress coefficient at rest; and e = void ratio.

The values used are shown in Table 6. The shear wave velocity below the tip of the pile was assumed to be 1.5 times the velocity at the tip, and the unit weight of the soil was assumed constant throughout the depth of the deposit.

Typical results are presented in Figs. 6 - 9. Figures 6, 7, and 8 show the variations of the pile dynamic stiffness f_{u1} and damping f_{u2} parameters with frequency in dry loose, dense and saturated medium dense sand deposits for similar slenderness ratios. Also shown in dashed lines in those figures are the corresponding theoretical results obtained by using PILAY [3]. Figure 9 shows the variations of the pile dynamic stiffness f_{u1} and damping f_{u2} parameters with slenderness ratio in a dry loose sand deposit at a given frequency.

The experimental results indicate that the pile stiffness is very much frequency dependent but that the damping is not. The theoretical stiffness predictions do not exhibit the strong frequency dependence observed in the experimental results, but the damping predictions do agree well with the experimental results. Four words of caution are in order at this stage: (1) it should be remembered that the centrifuge tests were carried within the confined environment of the centifuge bucket, and therefore may have been affected by reflecting waves from bottom and sides of the bucket. This problem is being researched extensively at Princeton University. (2) The frequency range of the experiments is rather small and limited to frequencies between the first and second resonance modes of the piles. (3) Novak's work is limited to the consideration of an elastic soil and perfect bonding between the pile and the surrounding medium, hypotheses which may be valid for small levels of excitation. It has been long recognized, however, that nonlinear effects, such as nonlinear soil behavior, slippage, and eventual gapping, play a fundamental role in the response of piles to cyclic loads of moderate to large amplitudes. Obviously, in the tests most of these nonlinear effects were encountered. (4) Additional work is needed, both analytically and experimentally, to refine the approximate expressions for pile stiffness and damping and to define better their range of applicability and accuracy of some of these formulas and compare and evaluate more fully various solutions.

CONCLUSIONS

The results of this experimental investigation demonstrate the feasibility of performing dynamic soil-structure interaction tests at a reduced scale in a centrifuge. Nonlinear effects, such as nonlinear soil behavior, slippage, and eventual gapping, which play a fundamental role in the dynamic response of piles to cyclic loads of moderate to large amplitudes can be explored via the centrifugal modeling. In a relatively limited frequency range (between the first and second resonant modes of single piles), the experimental results indicated that the pile stiffness is strongly frequency-dependent but that the damping is not. The theoretical stiffness predictions do not exhibit the frequency dependence observed experimentally, but the damping predictions do agree well with the test results. Reflection of waves from the walls of the centrifuge bucket could affect the experimental results, and additional work is needed to study this effect. Nonetheless, the results reinforce confidence in the centrifugal modeling technique by demonstrating its versatility and accuracy; many important design parameters can be measured accurately in the centrifuge with great convenience and low cost.

ACKNOWLEDGMENTS

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TABLE 1

MODEL PILES USED IN THE TESTS

Wall Thickness = $3.969\ 10^{-4}$m

PILE No.	TOTAL LENGTH L_1 (m.)	EMBEDMENT L (m.)	OUTER RADIUS r_0 (m.)	SLENDERNESS RATIO L/r_0
1	0.270	0.222	0.003	74.07
2	0.252	0.212	0.003	70.67
3	0.230	0.190	0.003	63.33
4	0.218	0.178	0.003	59.33
4*	0.218	0.158	0.003	52.67
4**	0.218	0.163	0.003	54.33
5	0.202	0.162	0.003	54.00
6	0.181	0.141	0.003	47.00
7	0.157	0.117	0.003	39.00
8	0.123	0.083	0.003	27.67

*Pile tested in dense dry sand.
**Pile tested in saturated medium dense sand.

TABLE 2

ACCELERATIONS, FREQUENCIES AND CORRESPONDING DAMPING RATIOS AT THE RESONANT PEAKS FOR EACH MODE OF THE SINGLE PILE - 3 LOAD LEVELS

Load Level	FIRST MODE			SECOND MODE			THIRD MODE		
	Freq (Hz)	Accel (G)	Damping	Freq (Hz)	Accel (G)	Damping	Freq (Hz)	Accel (G)	Damping
1	132.9	15.36	2.56%	1061	11.67	2.13%	2816	2.26	1.59%
2	127.4	40.61	2.96%	1040	36.56	1.89%	--	--	--
3	125.4	54.20	3.37%	1040	46.86	2.50%	--	--	--

TABLE 3

LOCATION AND MAGNITUDE OF MAXIMUM
BENDING MOMENTS IN THE SINGLE PILE

Load Level	FIRST MODE		SECOND MODE		THIRD MODE	
	Maximum Moment(N-M)	Depth (cm)	Maximum Moment(N-M)	Depth (cm)	Maximum Moment(N-M)	Depth (cm)
1	.073	0	.0016	.21	.000031	0
2	.18	.43	.0043	1.01	--	--
3	.24	.40	.0057	1.25	--	--

TABLE 4

ACCELERATIONS, FREQUENCIES AND CORRESPONDING
DAMPING RATIOS AT EACH RESONANT PEAK:
FOR THREE PILE GROUPS - TWO LOAD LEVELS

Load Level	Pile Spacing (in)	FIRST MODE			SECOND MODE		
		Freq (Hz)	Accel (G)	Damp-ing	Freq (Hz)	Accel (G)	Damp-ing
4	4.0	620.5	21.2	0.64%	3275	1.55	0.68%
5	4.0	619.1	55.5	0.89%	--	--	--
4	2.5	631.9	4.35	0.49%	3246	.89	--
5	2.5	629.7	44.5	0.68%	--	--	--
4	1.0	666.5	1.19	0.34%	3218	1.49	0.77%
5	1.0	671.1	1.79	0.46%	--	--	--

TABLE 5

LOCATION AND MAGNITUDE
OF THE MAXIMUM BENDING MOMENTS
IN THE PILE GROUPS

Pile Spacing (inches)	Load Level	FIRST MODE		SECOND MODE	
		Maximum Moment (N-M)	Depth (cm)	Maximum Moment (N-M)	Depth (cm)
4.0	4	2.57×10^{-3}	1.10	1.93×10^{-3}	0
4.0	5	2.75×10^{-3}	.86	--	--
2.5	5	2.10×10^{-3}	1.10	--	--
1.0	5	$.062 \times 10^{-3}$	.49	--	--

TABLE 6

SHEAR WAVE VELOCITY DETERMINATION

Soil Type	γ_s kg/m^3	K_0	e
Dry loose sand	1437.308	0.6	0.78
Dry dense sand	1743.120	0.4	0.51
Saturated medium dense sand	1987.767	0.4	0.68

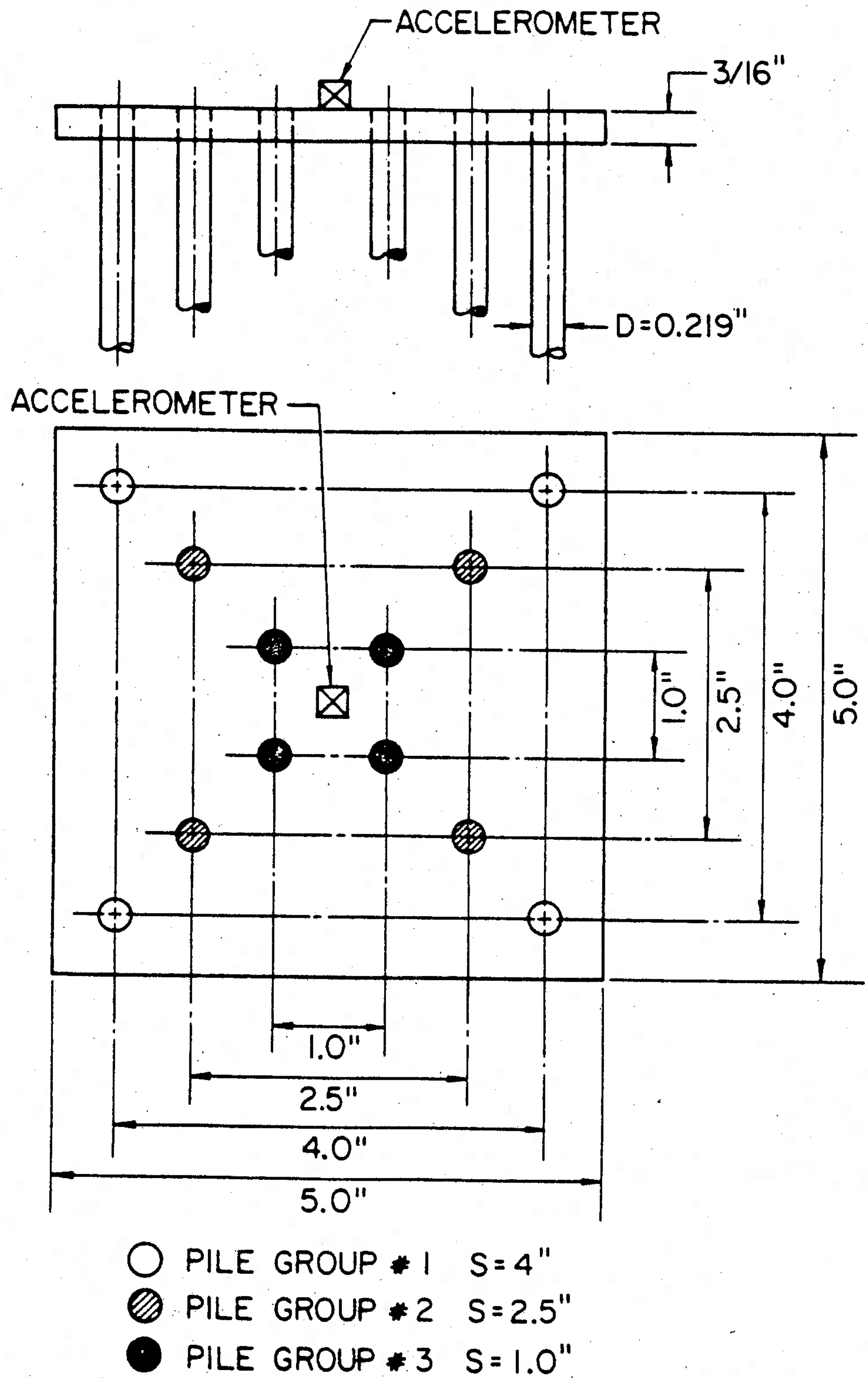

Fig. 1 Layout of the three pile groups used in the tests.

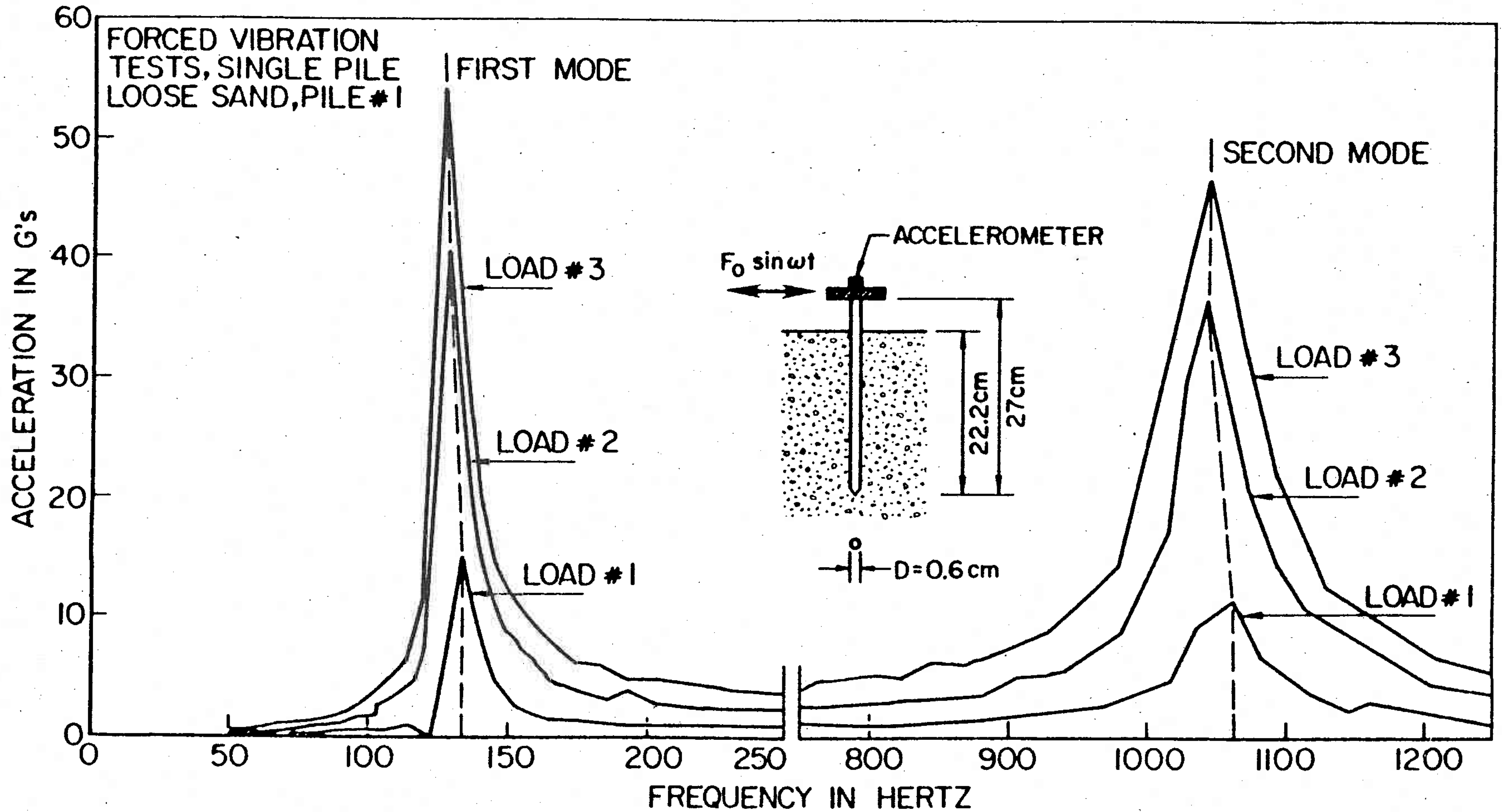

Fig. 2 First and second mode response curves.

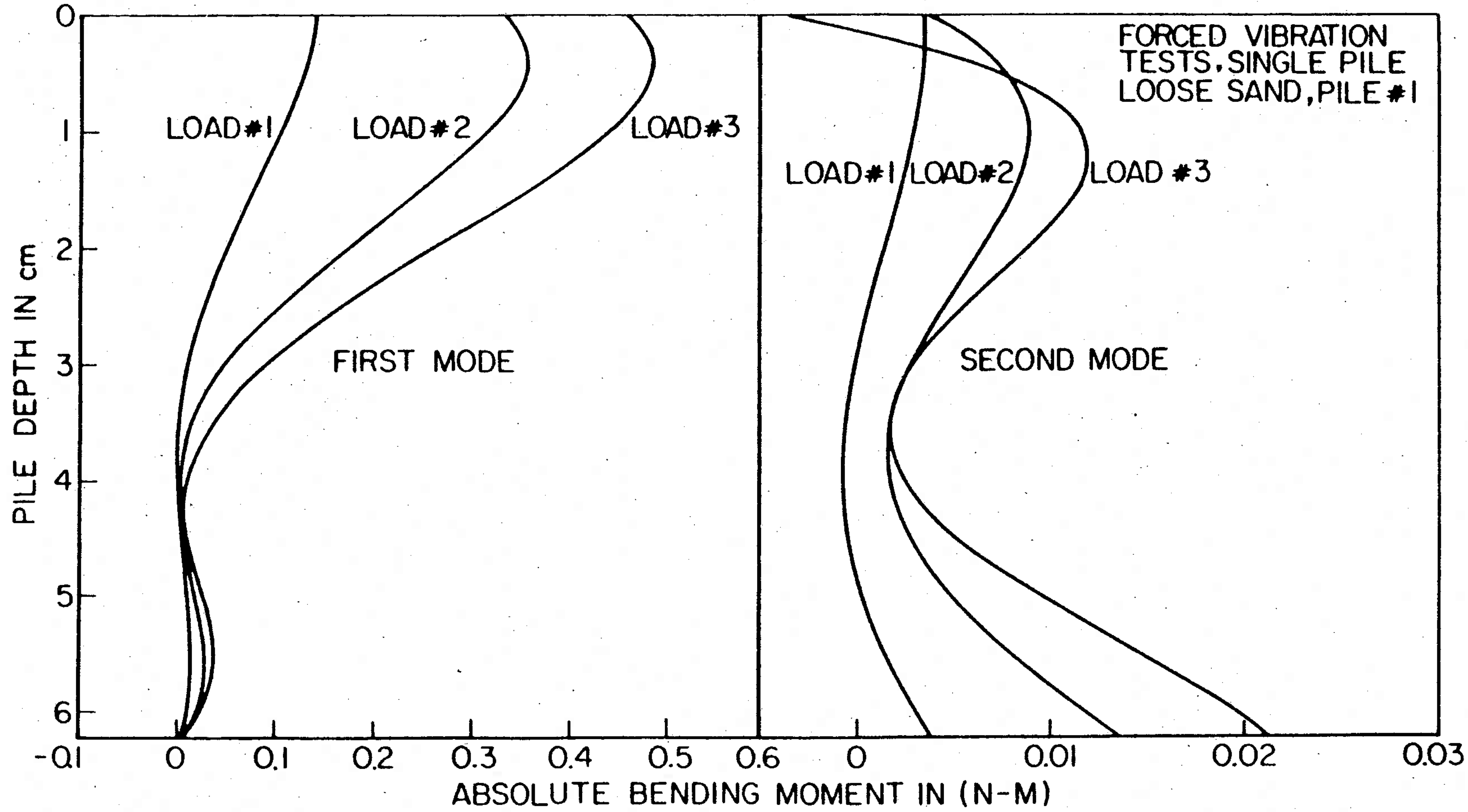

Fig. 3 Bending moments of the first two modes.

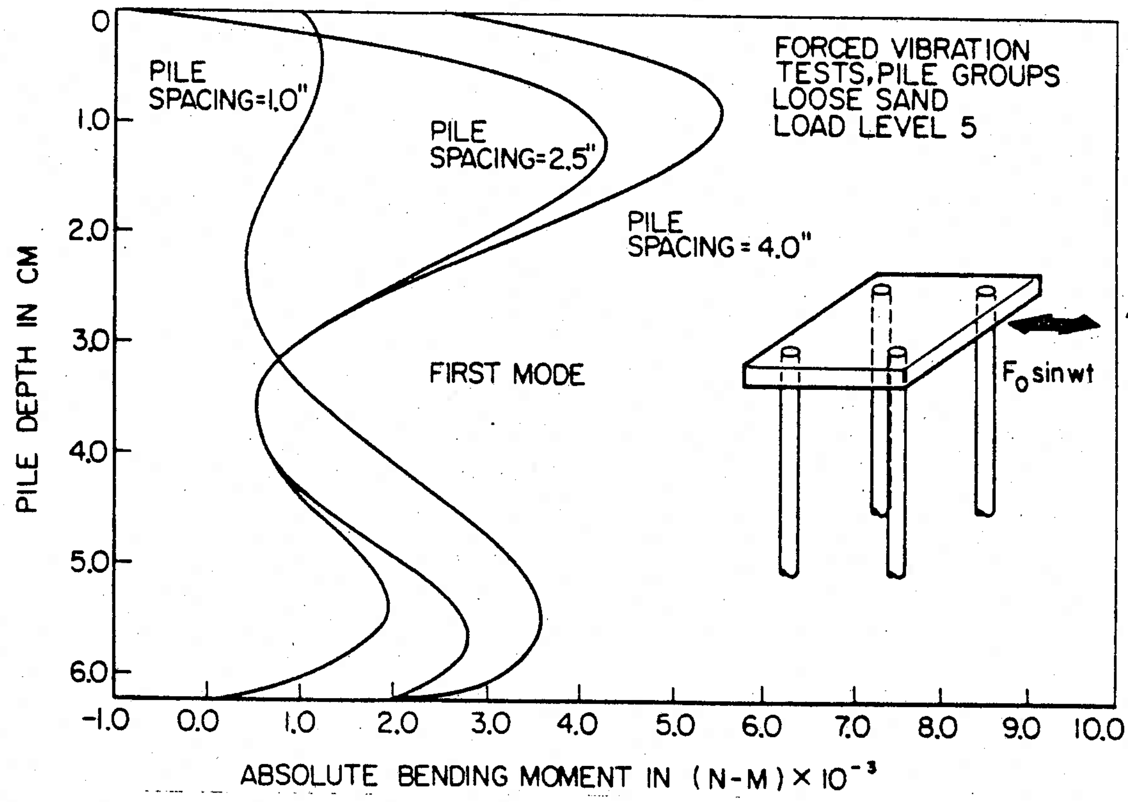

Fig. 4 Bending moment of the pile group.

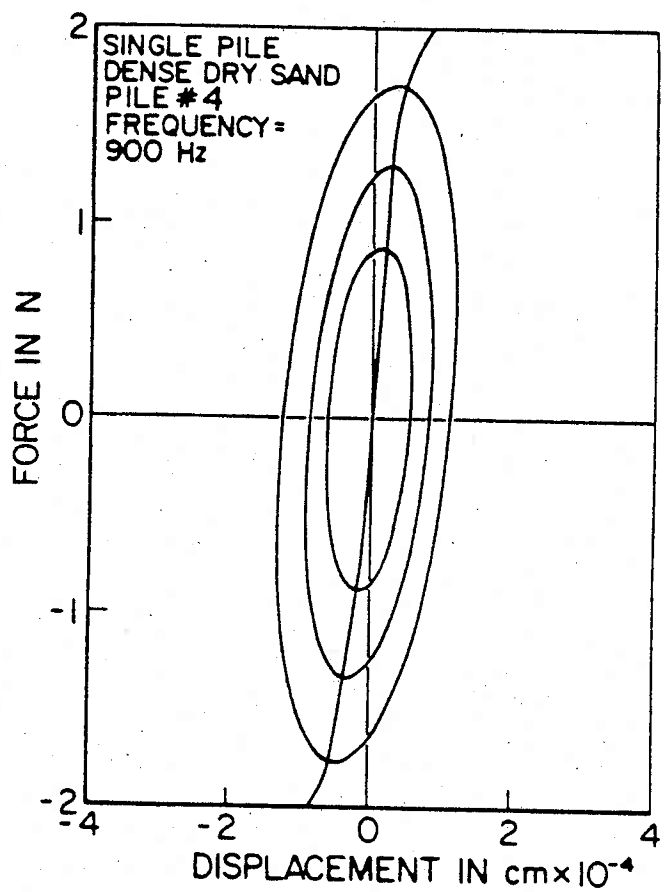

Fig. 5 Hysteretic response of a single pile in dense dry sand.

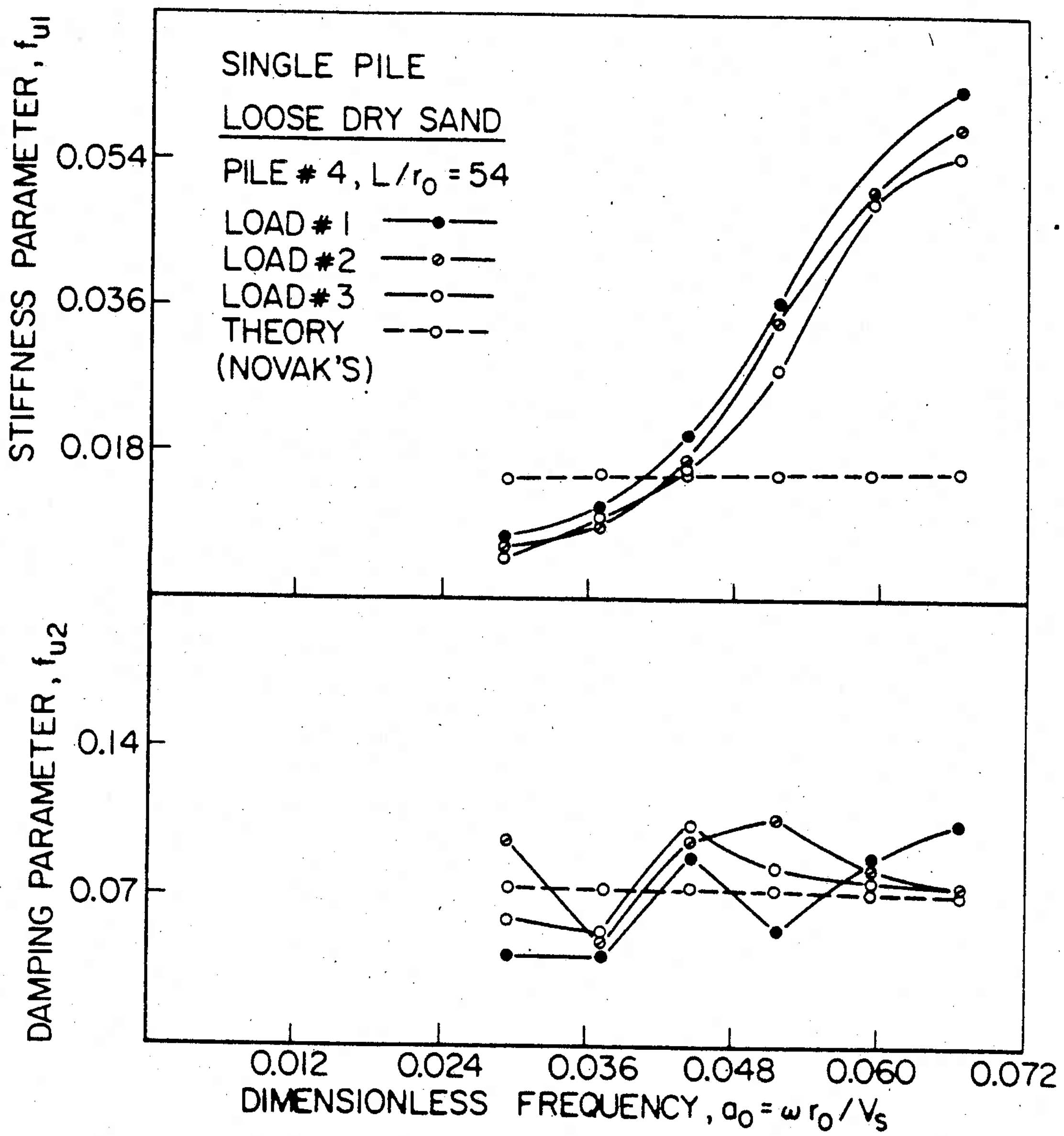

Fig. 6 Variations of stiffness parameter, f_{u1}, and of damping parameter, f_{u2}, with dimensionless frequency, $a_0 = \omega r_0/V_s$.

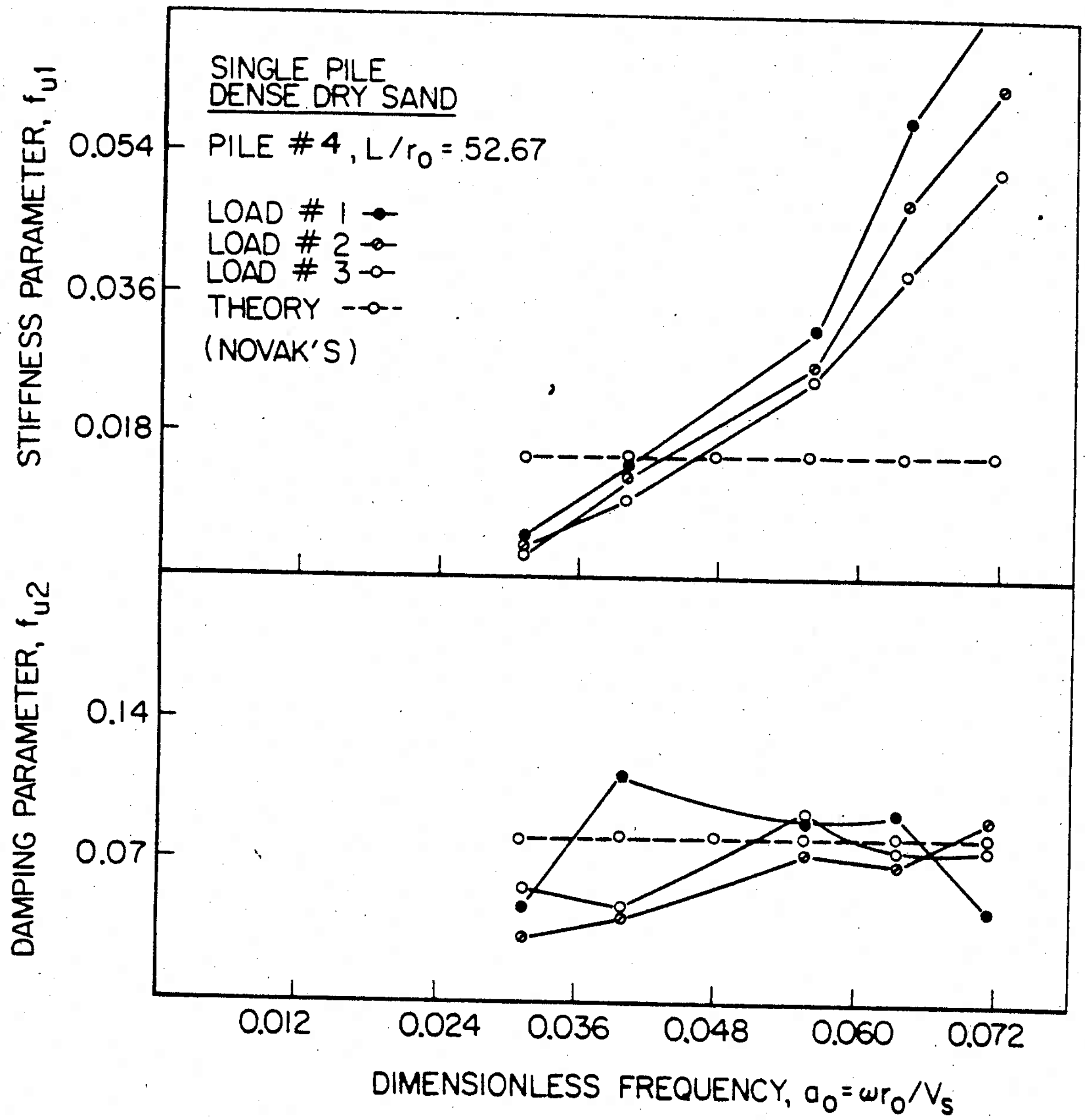

Fig. 7 Variations of Stiffness parameter, f_{u1}, and damping parameter, f_{u2}, with dimensionless frequency, $a_0 = \omega r_0/V_s$.

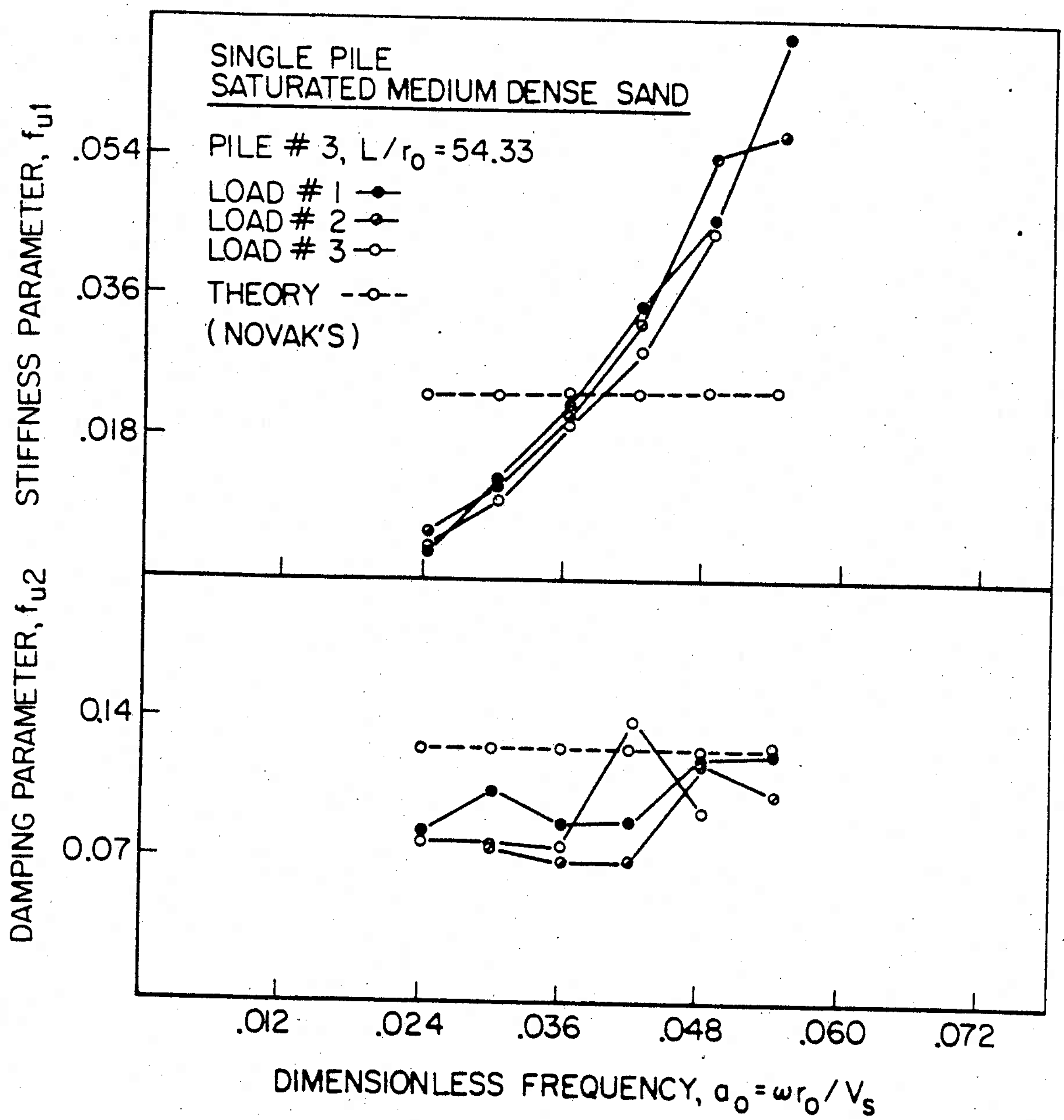

Fig. 8 Variations of stiffness parameter, f_{u1}, and damping parameter, f_{u2}, with dimensionless frequency, $a_0 = \omega r_0/V_s$.

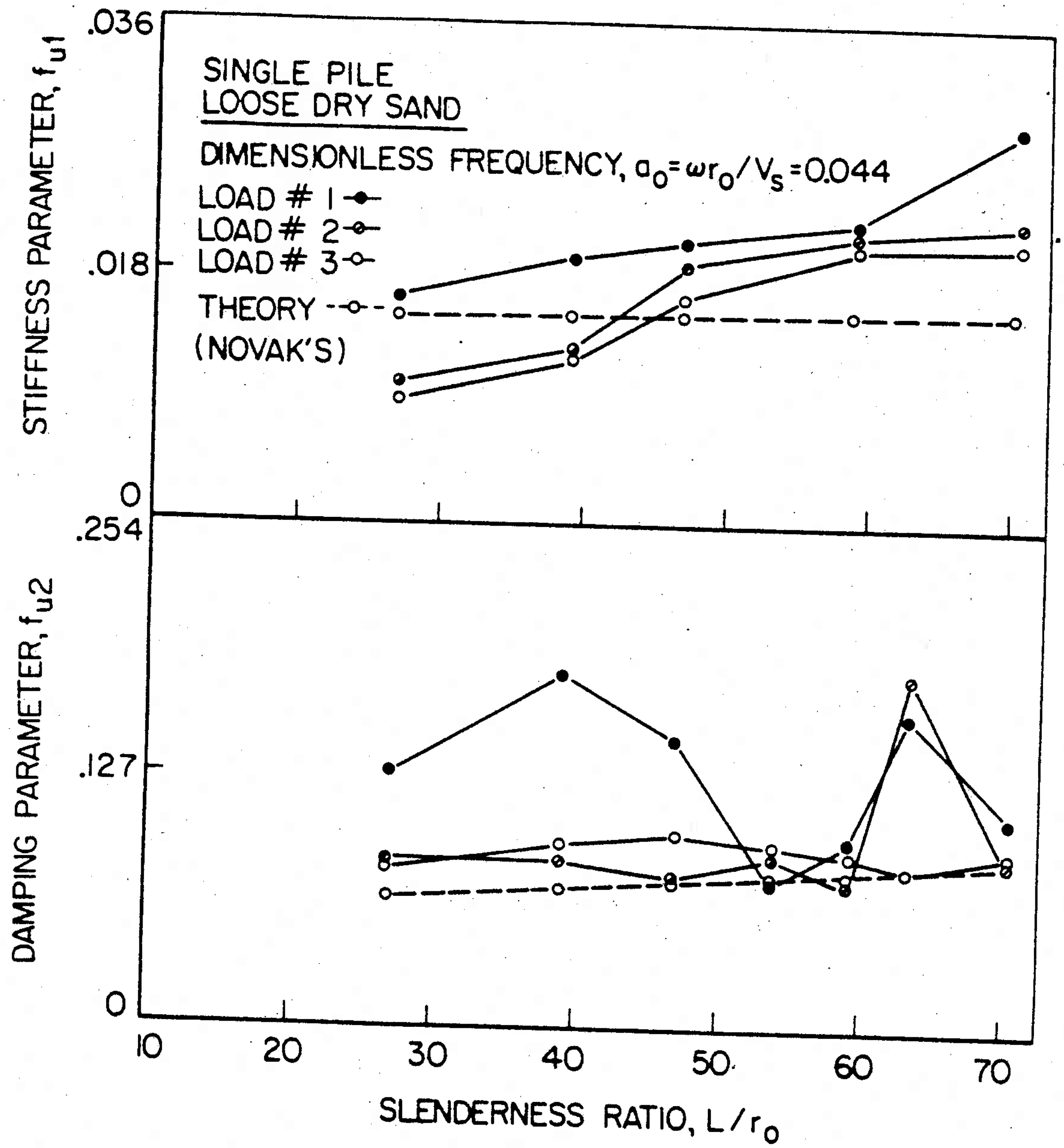

Fig. 9 Variations of stiffness parameter, f_{u1}, and damping parameter, f_{u2}, with slenderness ratio, L/r_0.