Seismic response of single piles in shake table studies

W.B.Gohl & W.D.Liam Finn
Department of Civil Engineering, University of British Columbia, Vancouver, Canada

ABSTRACT: Shaking table tests were conducted on model piles using the large UBC shake table to provide a data base for checking the predictive capability of computational models for analyzing the response of piles to earthquake loading. Tests on model piles embedded in dense dry sands are discussed herein. Pile bending moments, deflections and accelerations, and acceleration response at the soil surface were measured during the testing. The natural frequencies of the pile and the foundation were also determined using a number of different test methods. The shear wave velocity distribution in the sand foundation was measured using piezoceramic bender elements.

1. INTRODUCTION

This paper summarizes data obtained during shaking table studies on model piles carried out at the University of British Columbia (UBC). These studies were conducted to establish a data base against which the performance of computational models used to predict the response of piles to earthquake loading could be checked. Herein data illustrating the behaviour of a single pile embedded in dense dry sand will be presented.

Data from full scale instrumented pile foundations are available only for low levels of earthquake shaking, where free field acceleration levels have not exceeded 0.1 g. Computational procedures appropriate for analyzing such cases are based on either viscoelastic or linear elastic Winkler type foundation models (Nogami and Novak, 1977; Novak et al., 1978; Berger et al., 1977). Several researchers have compared the measured pile foundation response with predictions made using these models (Hamada and Ishida, 1980; Yasuyuki and Yoshida, 1980; Sugimura, 1977). The comparisons suggest that the accuracy of prediction of pile response is dependent on the proper selection of strain compatible elastic soil parameters, which is difficult to accomplish.

Detailed measurements of full scale pile behaviour are not available for stronger earthquake shaking so that it is not possible to validate those analytical procedures which propose to account for soil nonlinearity. Winkler models with nonlinear force-deflection relations are commonly used in such cases. (Reese et al., 1974; Matlock et al., 1978; Kagawa and Kraft, 1981; Nogami, 1985). Therefore, a series of shaking table tests were carried out using the UBC shaking table to obtain data on single pile response during strong shaking.

Stress levels in the foundation soils in model pile tests using a shake table are much lower than corresponding stresses in full scale tests. Therefore, it is necessary to measure soil properties appropriate to these low confining stress levels. Piezoceramic bender elements were used to measure shear wave velocity distributions from which low strain shear moduli, G_{max}, were computed.

This paper is concerned primarily with describing the test equipment, instrumentation and experimental test procedures. Representative data on shear wave velocities, pile bending moments, deflections and accelerations, and acceleration response at the soil surface will be presented. Tests for measuring the natural frequencies of the model pile and foundation soils are also described. Finally, general conclusions are reached about the influence of soil nonlinearity on pile and foundation responses.

2. SHAKING TABLE TEST PROCEDURES

The UBC shaking table measures 3.0m by 3.0m. Table motion is controlled by a hydraulic actuator and feedback control system described by Ramsay (1982). A level dry sand foundation was prepared in a rigid container bolted to the table. Two 25mm thick styrofoam pads were placed at each end of the container to prevent wave reflection from the sides of the box perpendicular to the direction of base motion. The plan area of the soil container after the above inserts were installed was 463mm by 463mm with a total depth of 635mm.

The pile foundations were prepared using C-109 Ottawa sand which is a rounded sand with minimum and maximum void ratios of 0.50 and 0.82, respectively. A gradation curve for the sand is shown in Figure 1. Pile tests were conducted in both loose and dense sand.

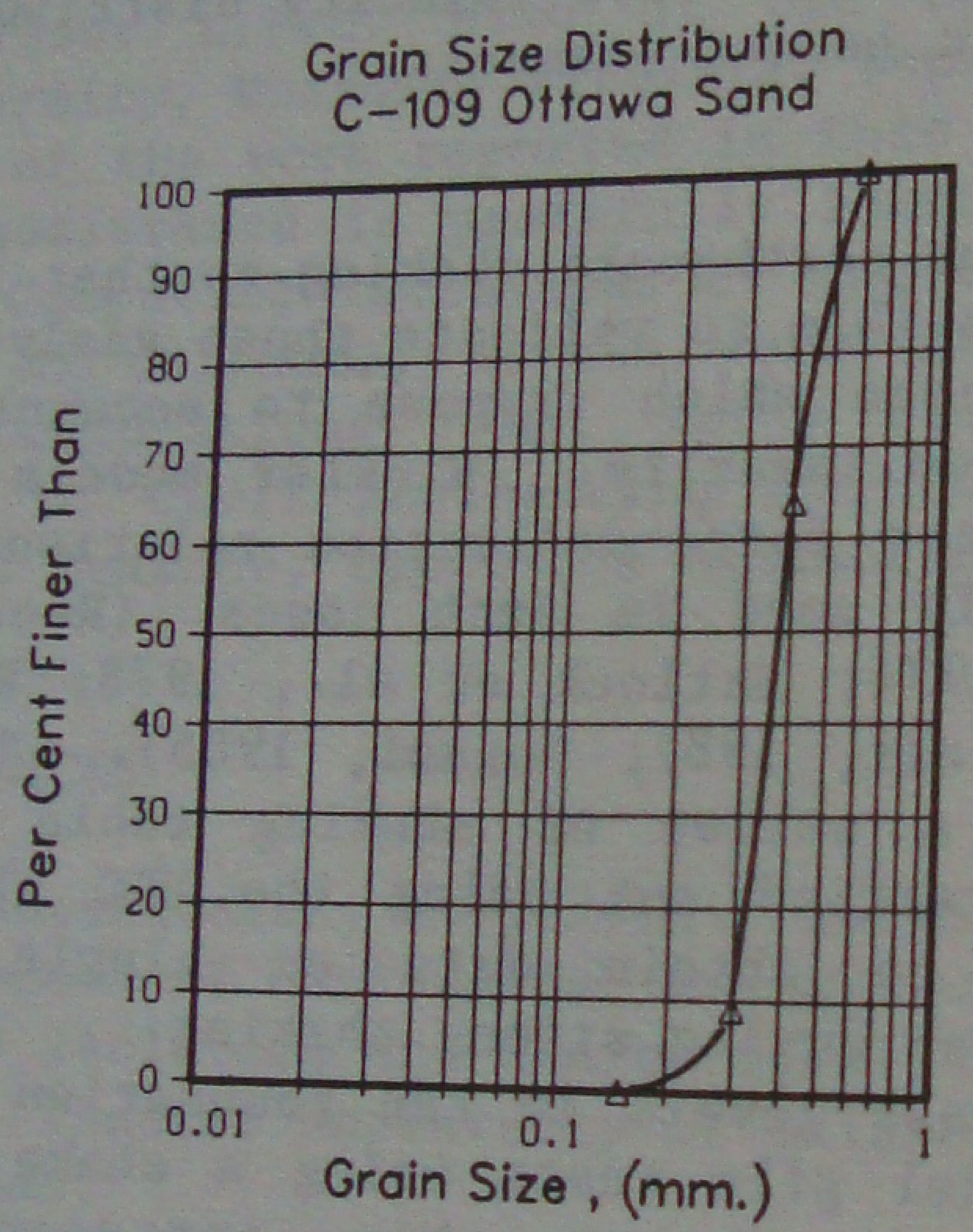

Fig. 1. Grain size distribution for C-109 Ottawa sand.

Only data from the dense sand tests will be presented to illustrate typical aspects of pile response.

A loose sand with an initial average void ratio of 0.71 was prepared by spreading constant weights of dry sand uniformly over a perforated screen placed on the base of the container or on top of the previous sand layer. The newly placed sand was levelled by hand and then bulked by pulling the screen up through the sand. During sand placement a vertical array of piezoceramic bender elements was installed to measure shear wave velocities, as described later. Instrumented piles were then pushed into the loose sand foundation by hand. The distance between the centre of the pile and the boundary of the soil container was 36 pile diameters. Following a series of tests with the pile embedded in loose sand, a dense sand foundation was achieved by densifying the loose sand using high frequency vibration. The loose sand foundation was shaken for several minutes using sinusoidal input with frequencies of 25 to 30 Hz and peak acceleration amplitudes of 0.5 g or greater. The large amplitude shaking was necessary to achieve significant sand densification. The model pile was restrained from movement in the sand during the vibration process by fixing the head of the pile using piano wire. An average void ratio of 0.57 or less was achieved, corresponding to relative densities of 78 percent or greater.

Following foundation placement, an accelerometer was placed in the surface of the sand a distance of 180 mm (28 pile diameters) from the centre of the model pile along a line parallel to the direction of shaking. The accelerometer measured horizontal surface accelerations in the direction of shaking. Another accelerometer was placed on the base of the soil container to measure input accelerations.

The model pile used in the study was constructed of hollow aluminum tubing having an outside diameter of 6.35 mm. The flexural rigidity (EI) of the pile was measured using transverse dead loading of the test pile while it was clamped at one end and free at the other. From measurements of pile deflection versus applied transverse load, the pile EI value was computed to be 4.65×10^6 N-mm^2 (1621 lb.in^2) using static beam bending formulae. A rigid pile head mass with a weight of 7.78 N was clamped to the head of the pile to simulate the effect of a superstructure (Figure 2). An accelerometer and two displacement transducers were mounted on the pile head mass. Seven pairs of calibrated strain gauges were placed at various points along the outside of the pile to measure peak bending strains. The electrical leads from the gauges were brought up the inside of the pile. Including the contribution of strain gauges and electrical lead wires, the pile has a weight per unit length of 0.0005 N/mm (0.0028 lb/in).

After pile insertion and connection of all associated instrumentation, the stick-up of the pile head mass above the soil surface was measured. The stick-up varied from 46.0 to 48.4 mm for the dense sand tests. Four lightweight settlement

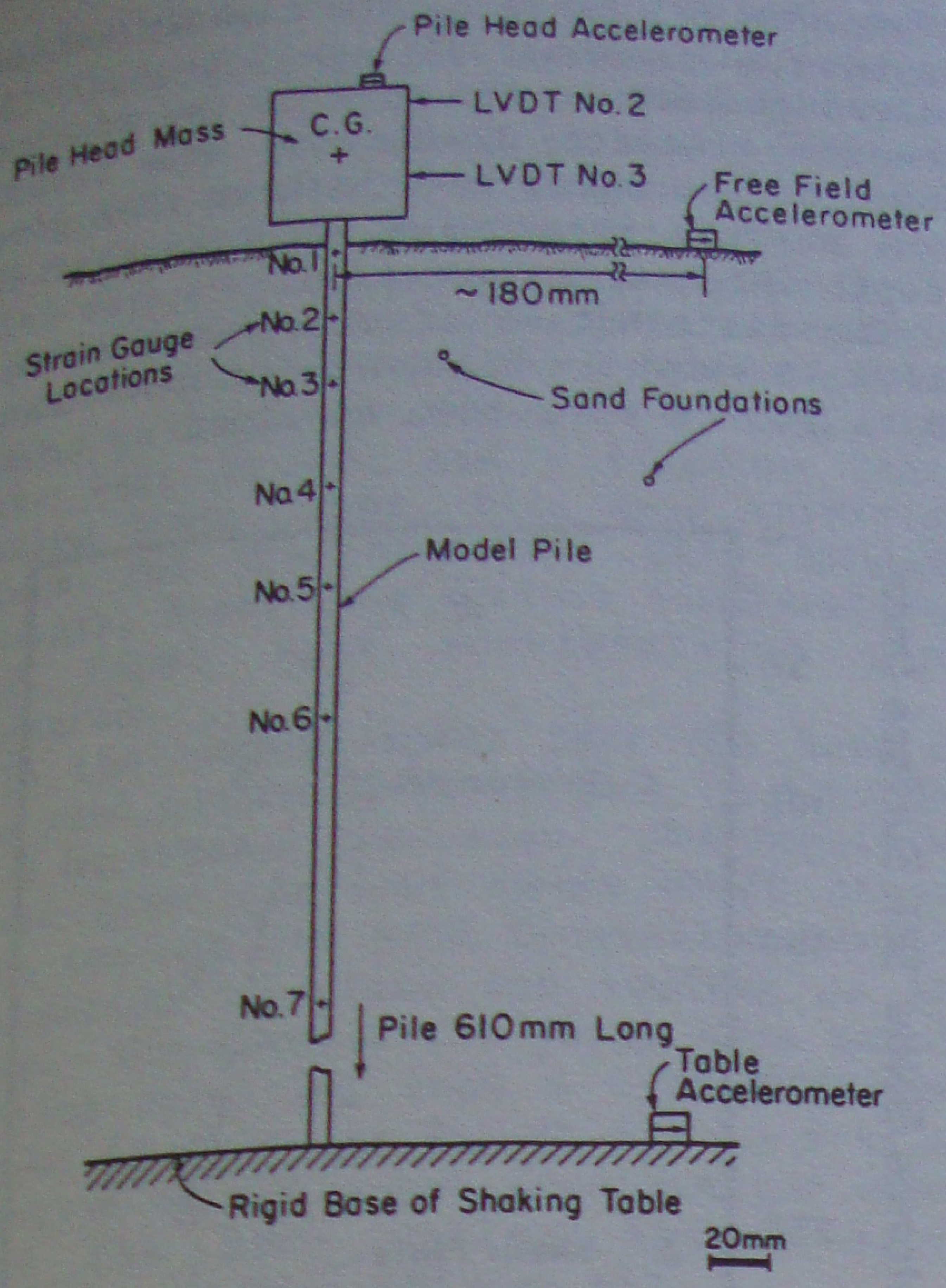

Fig. 2. Single pile showing instrumentation layout.

plates were placed a minimum of 20 pile diameters from the centre of the pile to measure surface settlement. The settlement is due to the cyclic shear strains generated by the base motion. From the settlement plate measurements, average void ratio changes in the dense sand foundations were found to be negligible during base excitation.

Prior to testing, shear wave velocity measurements were made in the sand using the procedure described below.

3. SHEAR WAVE VELOCITY MEASUREMENTS

Shirley and Hampton (1978) developed a procedure for measuring shear wave velocities in soil specimens using piezoceramic bender elements. The Norwegian Geotechnical Institute (1984) measured shear wave velocities in triaxial specimens using both bender elements and the more conventional resonant column technique. Within the limits of experimental error both methods gave similar results.

Finn and Gohl (1987) extended the use of bender elements to measure shear wave velocities in centrifuged soil models. The same technique has been used to measure shear wave velocities in the soil models used in the shake table tests. The bender element consists of a sandwich of two piezoceramic plates rigidly bonded together. The polarization of the ceramic material in each plate and the electrical connections are such that when a driving voltage is applied to the element one plate elongates and the other contracts so that the element bends as in Figure 3a. One edge of the element is attached to a bearing plate which allows the element to be oriented as desired in the sand. An element may be used as a wave source or as a receiver. In the shake table tests the source and receiver bender elements in the array were installed as shown in Figure 3b.

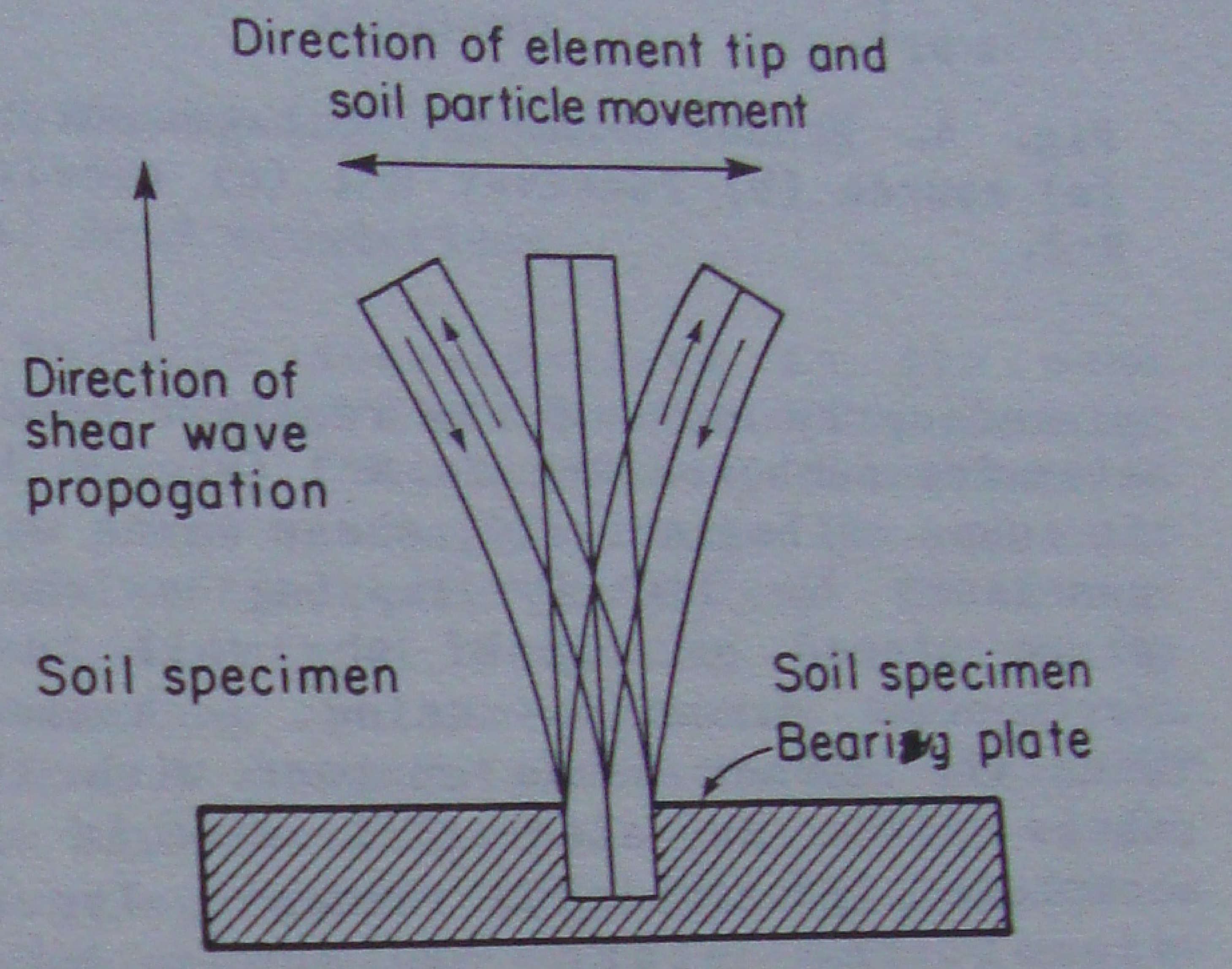

Fig. 3a. Piezoceramic bender element (after NGI, 1984).

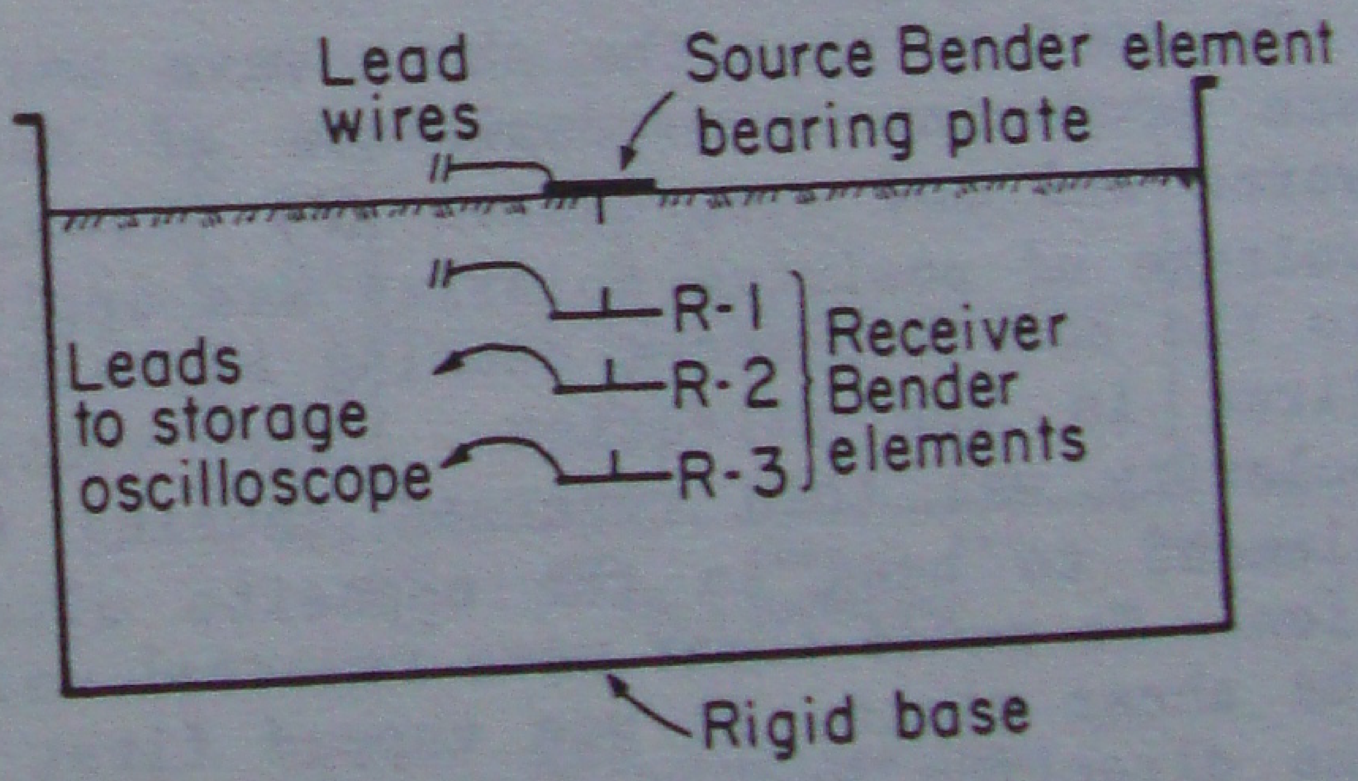

Fig. 3b. Schematic drawing showing bender element layout.

Shear waves were generated using two different methods in the present study. In the first, the source element was excited with a square wave voltage pulse (Figure 4a). The small amplitude bending of the source element under the input generates a shear wave which propagates downward into the foundation soil and is

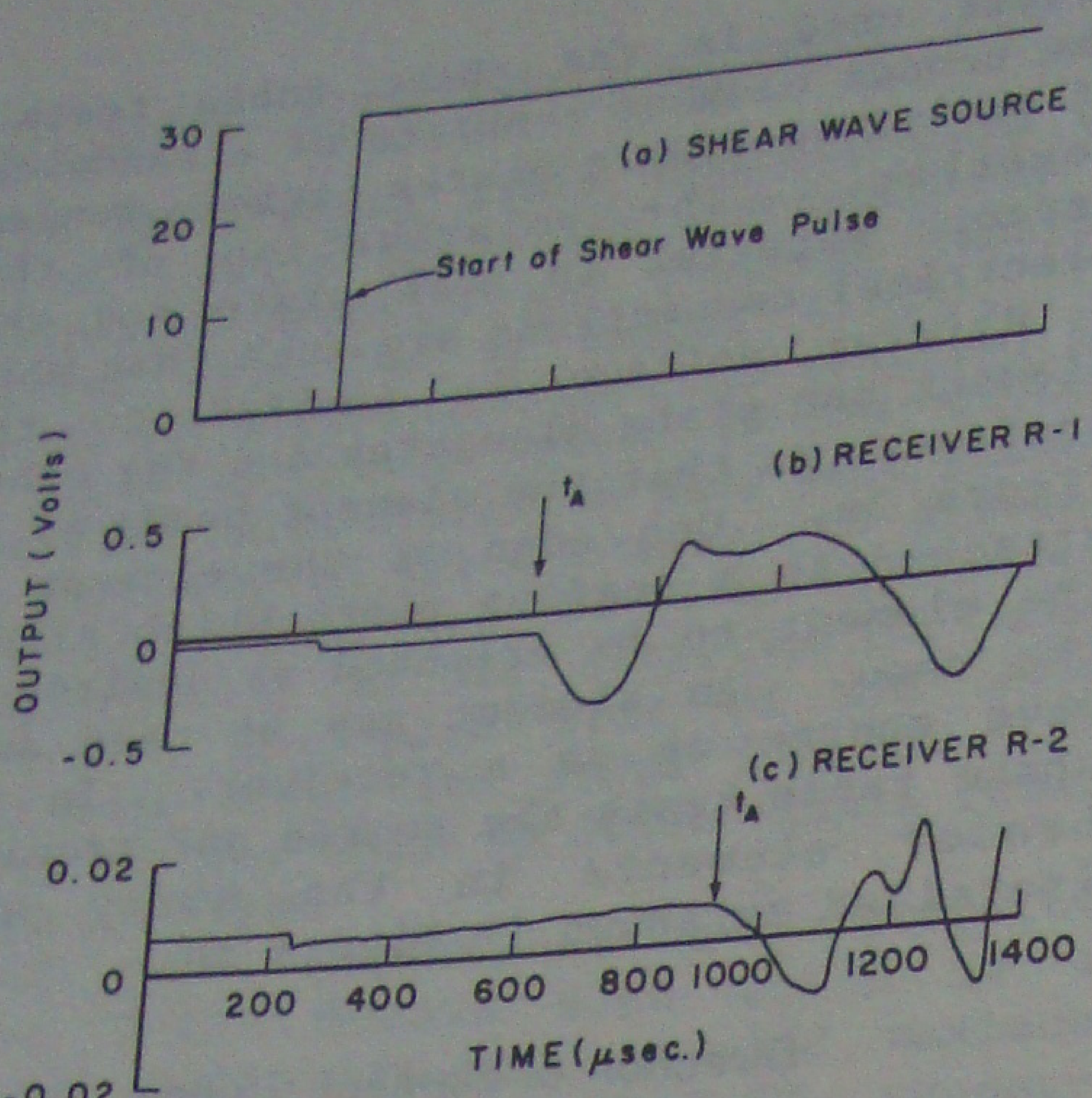

Fig. 4. Bender element voltage outputs (a) source (b) receiver R-1 (c) receiver R-2.

picked up by each of the receiver bender elements numbered R-1 to R-3 (Figure 3b) in turn. Alternatively, shear waves were generated by lightly tapping a shear plate placed on top of the soil in a horizontal direction using a hammer. When the hammer makes contact with the shear plate an electrical circuit is completed, generating a voltage pulse to signal the start of shear wave propagation. In each case, the minute soil vibration caused by the shear wave propagation mechanically excites the receivers which then respond with an electrical signal.

To confirm that the shear wave source was producing a predominance of shear waves and not compression (P) waves, a voltage of positive polarity was applied to the source element causing it to bend first in one direction. The voltage polarity was then reversed causing the element to bend in the opposite direction. To cause shear wave reversal using the shear plate, it was tapped first in one direction and then in the opposite direction. The receivers responded with a positive voltage change followed by a negative voltage change in response to the positive and negative shear wave pulses, respectively. This reversal confirmed that shear waves were being generated.

Voltage outputs from the shear wave source and receivers were stored using a digital storage oscilloscope. Amplified signals produced by two adjacent receivers are shown in Figures 4b,c. The arrival times t_A are clearly evident from the recorded voltage signals. The distance between the tips of two adjacent bender elements divided by the transit time of the shear wave between them gives the average shear wave velocity over the depth interval in question.

The distribution of shear wave velocities is shown in Figure 5 for dense sand. Data points have been obtained up to the

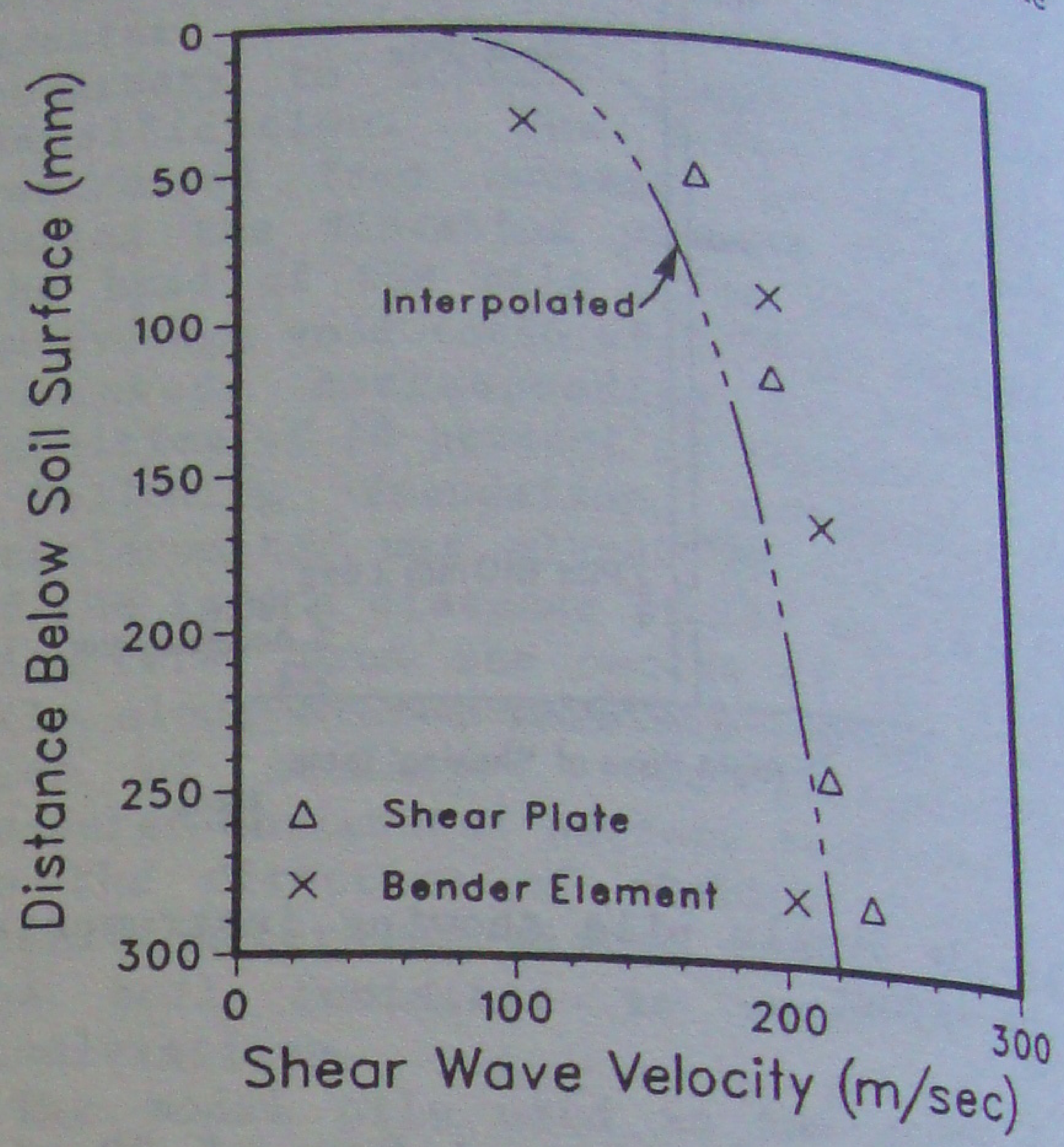

Fig. 5. Shear wave velocities in dense sand foundations on shake table.

300 mm depth and are shown for both methods of wave propagation. The distribution may be described by $V_s = Az^n$ where $n = 0.2$, A = constant = 280.0 m^{1-n}/sec, z is the depth below the soil surface in metres and V_s is the shear wave velocity in m/sec. The measured shear wave velocity data points follow this distribution closely. This agrees with data from Stokoe et al. (1986) showing that V_s varies as the mean normal stress raised to the power of 0.20 to 0.25. The average shear wave velocity over the entire thickness of the sand foundation (= 0.61 m) is computed to be 211 m/sec.

4. NATURAL FREQUENCY TESTS

Tests were carried out to estimate the natural frequencies of the model pile and sand foundation. The frequencies of the pile are functions of the structural properties of the pile, the pile head mass and the lateral stiffness of the soil around the pile. The soil stiffness is controlled by the insitu moduli which are strain dependent. Therefore, the natural frequencies will depend on the amplitudes of the pile vibrations at which they are measured. The natural frequencies of the sand foundation in the free field are also dependent on excitation amplitude.

The natural frequencies were determined by three different tests: a hammer impact test, a frequency sweep test using sine wave base motion, and a ringdown test. During these tests, pile head accelerations and displacements, pile bending moments, foundation surface accelerations and input base accelerations were measured.

In the hammer impact test the base of the sand container was struck by a hammer in a horizontal direction. The impulsive force generates body waves which propagate through the sand foundation causing foundation vibration and loading on the pile. The form of the input base acceleration resulting from a typical hammer impact is shown in Figure 6a. A Fourier spectrum of the input acceleration shows that the base motion has a broad frequency content (Figure 6b).

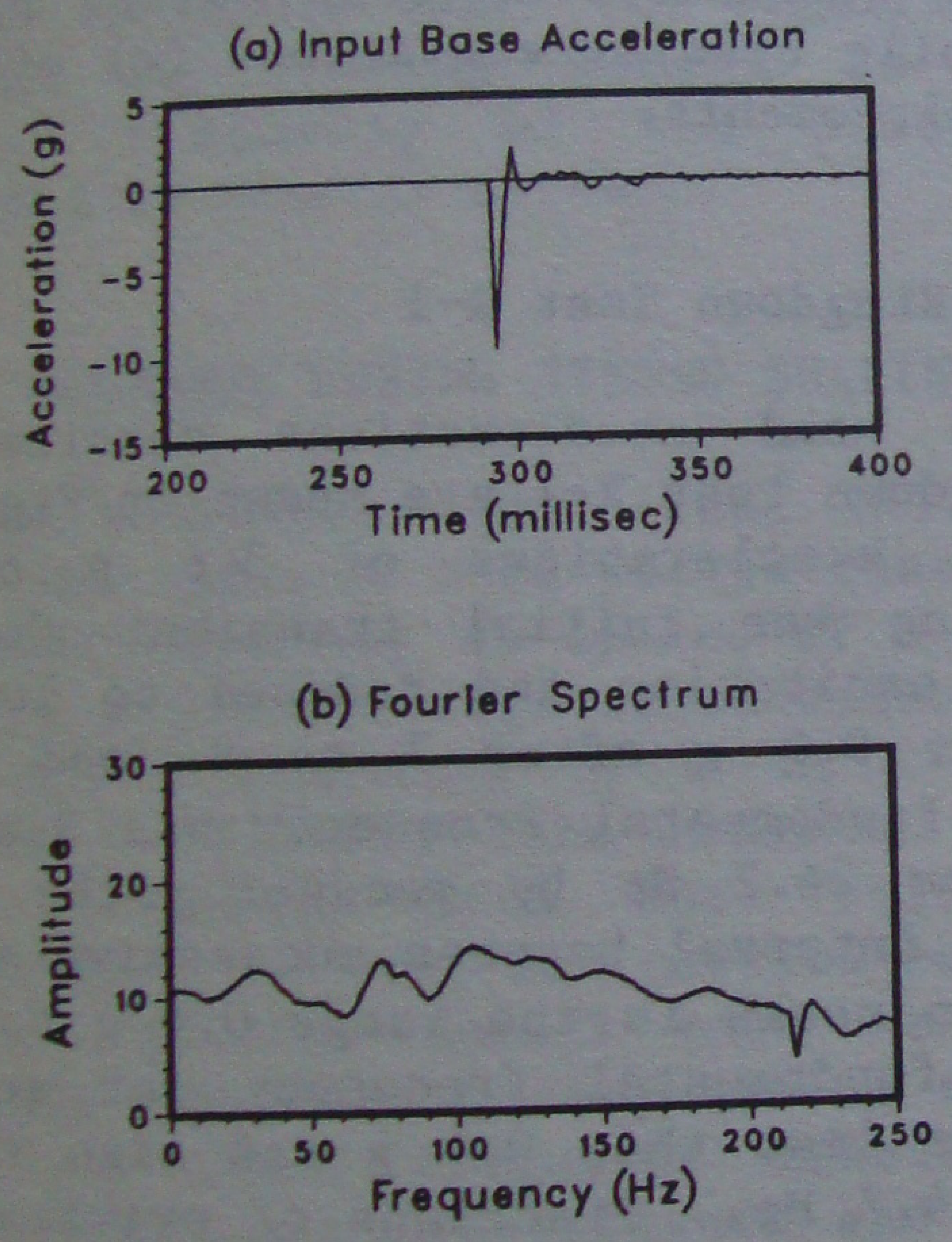

Fig. 6. Typical hammer impact test (a) input base accelerations (b) Fourier spectrum.

Frequency sweeps were conducted using variable frequency sinusoidal base motions with peak average accelerations of 0.20 g. The input frequencies used were varied between 5 and 50 Hz so that soil-pile amplification could be determined over a broad frequency range. For each frequency input, the sinusoidal base excitation was gradually increased to its peak amplitude and was maintained at this level for at least 20 cyles so that steady state system response was obtained.

In a ringdown test, the pile head was displaced a fixed amount and then released so that the pile vibrated freely.

The amplitudes of pile head accelerations and displacements, and foundation surface accelerations were not the same during the different frequency tests. Therefore, the strain fields which define the moduli were not the same. These facts must be kept in mind when comparing the results of the frequency tests.

4.1 Hammer Impact Test H-4

(a) Sand Foundation:

Surface accelerations in the sand foundation in the free field are shown in Figure 7a. The maximum acceleration was 1.1 g and was rapidly damped to near zero acceleration after 4 cycles. The fundamental natural frequency determined by measuring the time interval between the first 4 acceleration peaks was estimated to be 56 Hz. The Fourier spectrum of the accelerations (Figure 7b) shows a predominant fundamental frequency in the range of 55 to 60 Hz. The presence of higher modes is also indicated by the spectrum.

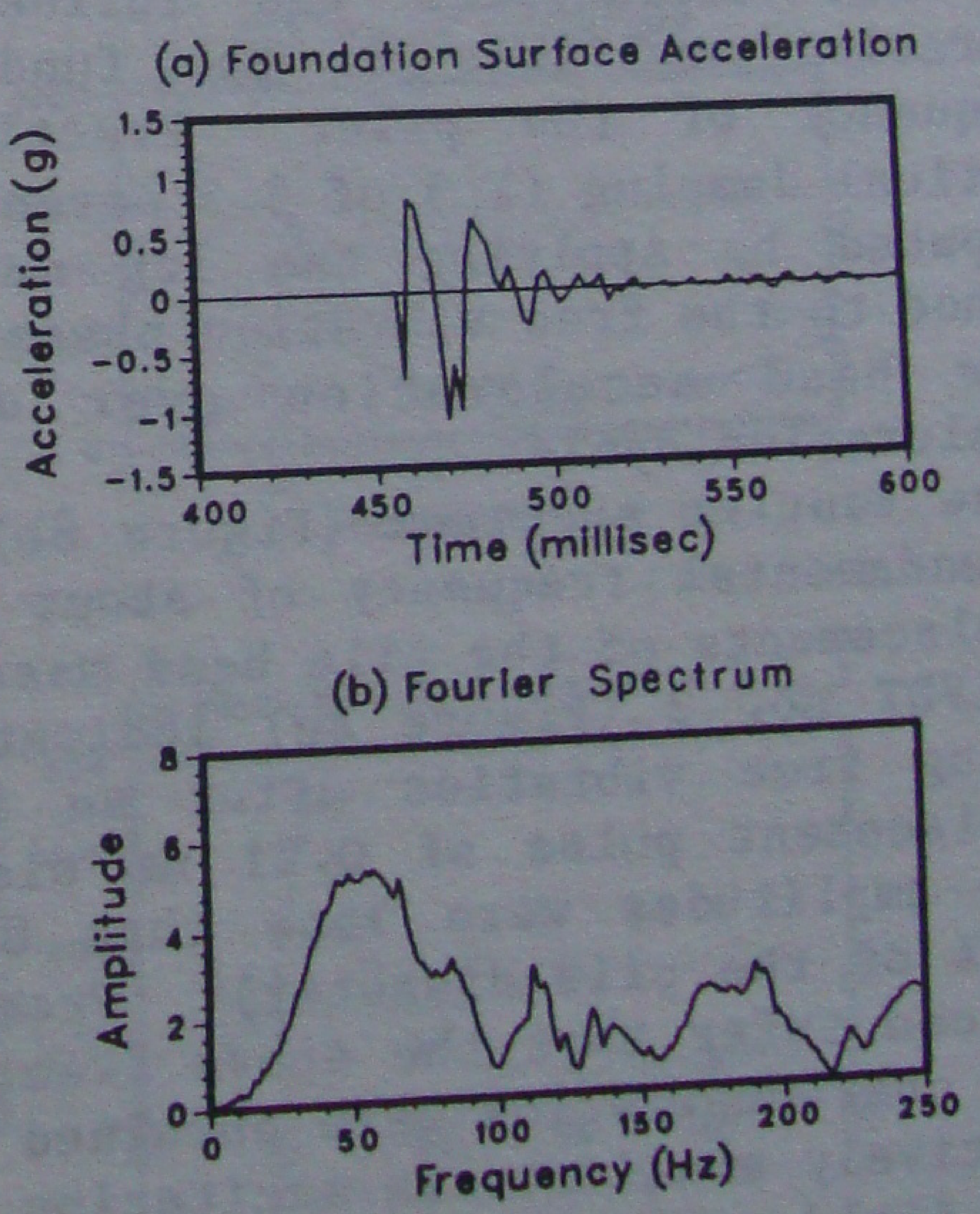

Fig. 7. Hammer impact test H-4. (a) the foundation surface accelerations, (b) the Fourier spectrum.

It is of interest to compare the natural frequency estimated from the measured insitu shear wave velocities, with that measured in the hammer test. The sand foundation is assumed to be uniform with a constant shear wave velocity corresponding to the average velocity of 211 m/sec determined previously. The fundamental frequency, f_1, of such a layer is given by $f_1 = V_s/4H$ where V_s is the shear wave velocity of the layer and H is the thickness. This equation gives a fundamental frequency f_1 = 86 Hz which is considerably higher than the experimental values of 55 to 60 Hz. The difference is due to the strain-softening of the foundation sand and to using an average shear wave velocity in the above calculations. The average shear wave velocity is based on low strains less than 10^{-4}%. The high accelerations generated in the hammer test resulted in larger strains and consequently lower moduli and reduced soil stiffness.

(b) Model Pile

Pile head accelerations are shown in Figure 8a. Peak accelerations of 1.7 g occurred during the initial transient phase of the excitation and reduced to levels of about 0.2 g after 6 to 7 load cycles. The fundamental natural frequency was determined to be 22.9 Hz by measuring the average time interval between successive acceleration peaks in the range 0.2 g to 0.6 g. The natural frequency was found to be 24.0 Hz on the average for accelerations less than 0.2 g. This illustrates the influence of vibration amplitude on the fundamental frequency of the pile. Fractions of critical damping (λ_1) of 5.5 percent were computed by applying the log decrement method to the free vibration phase of the pile head acceleration over several acceleration cycles.

The Fourier spectrum (Figure 8b) shows a fundamental frequency of about 24 Hz. Displacements of the pile head mass given by LVDT no. 2 (Figure 8c) indicate that during free vibration after an initial displacement pulse of 0.21 mm displacement amplitudes were less than 0.03 mm (0.5% of the pile diameter). From this, one can infer that the above free vibration characteristics were obtained during relatively small strain excitation of the near field soil.

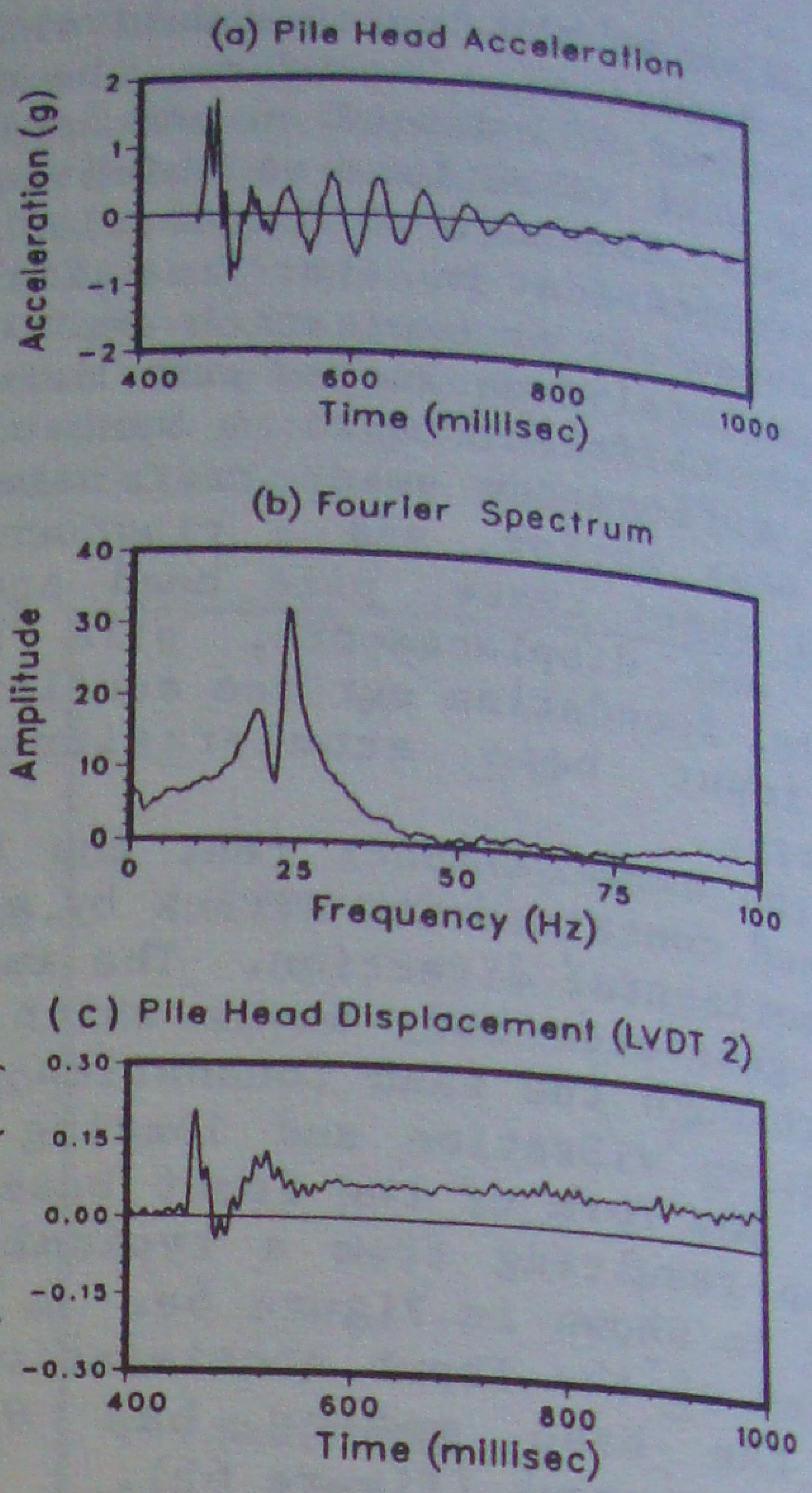

Fig. 8. Hammer impact test H-4. (a) Pile head accelerations (b) Fourier spectrum of pile head accelerations (c) pile head displacements.

4.2 Ringdown Test R-1

Pile head accelerations obtained from ringdown test R-1 are shown in Figure 9a. Peak accelerations of 3.6 g occurred during the initial transient phases of the excitation and reduced to levels of about 0.2 g after 7 to 8 load cycles. The fundamental frequency was determined to be 24.2 Hz by measuring the average time interval between successive acceleration peaks in the range 0.4 g to 2.0 g. The fundamental frequency for accelerations less than 0.4 g was also found to be 24.2 Hz. Fractions of critical damping (λ_1) of 4.4 percent were computed by applying the log decrement method. The Fourier spectrum (Figure 9b) shows a fundamental frequency of about 24 Hz. The above values are similar to those obtained from the hammer impact test indicating that about the same amount of strain softening occurred in the near field soils in both tests. The pile head displacements obtained from LVDT no. 2

(Figure 9c) again show that after an initial displacement pulse of 0.1 mm pile displacements were small (less than 0.04 mm) during free vibration.

4.3 Frequency Sweep Tests

Data are presented describing the pile response to at least 20 cycles of sinusoidal base motion with a peak average acceleration of 0.2 g. The input accelerations were amplified through the sand foundations resulting in accelerations of 0.6 g or greater at the pile head. The Fourier amplitudes of the pile head accelerations are plotted versus input frequencies in Figure 10a which shows the fundamental frequency to be approximately 12 Hz. The fundamental frequency obtained is only one half that measured in the hammer impact and ringdown test. These results reflect the much greater strain softening caused by the high steady state response amplitudes in the frequency sweep tests. Peak bending moments measured by strain gauge no. 3 are plotted versus applied forcing frequencies in Figure 10b. The significant increase in bending moment as the resonant frequency of the pile is approached is clearly evident.

5. PILE RESPONSE DURING STRONG SHAKING

A forced vibration test (test 14) was carried out using a base motion consisting of approximately 28 cycles of a sine wave with a peak average acceleration of 0.60 g and a frequency of 10 Hz. This frequency is close to the resonant frequency determined from the frequency sweep tests. The test was carried out to define in greater detail pile bending moment distributions during strong shaking at an input frequency likely to induce significant pile flexure and nonlinear soil-pile interaction.

The acceleration input at the base of the model is shown in Figure 11a. Accelerations recorded at the surface of the foundation layer and at the pile head are shown in Figures 11b,c. Pile head displacements given by LVDT no. 2 and LVDT no. 3 are shown in Figures 12a,b. Time histories of pile bending moment at various points along the pile are shown in Figures 13a,b,c. Figure 13d shows the peak bending moment distributions along the pile during the first and fifteenth load cycles at a peak base motion amplitude of 0.6 g.

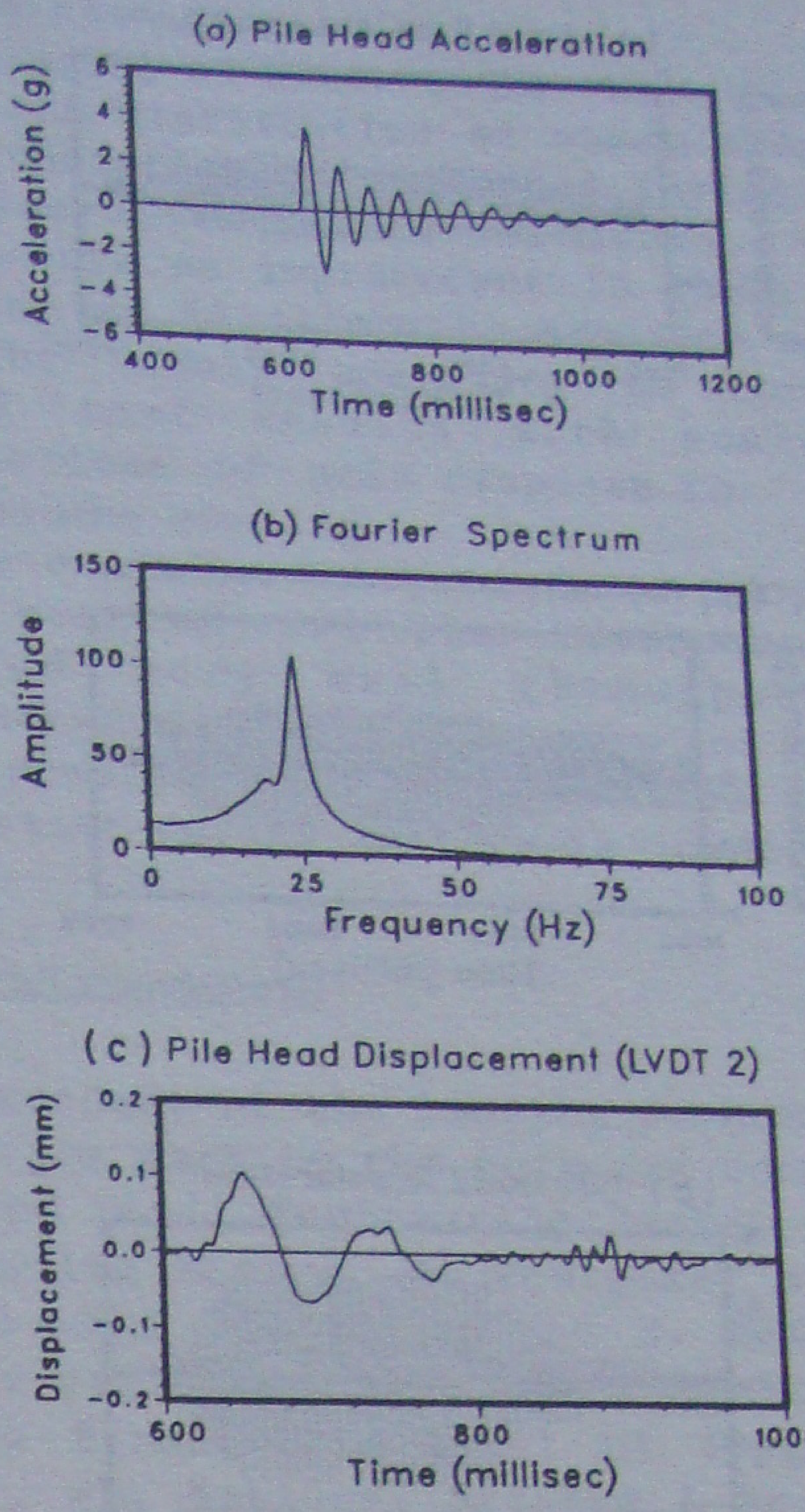

Fig. 9. Ringdown test R-1 (a) pile head acceleration (b) Fourier spectrum of pile head accelerations (c) pile head displacement.

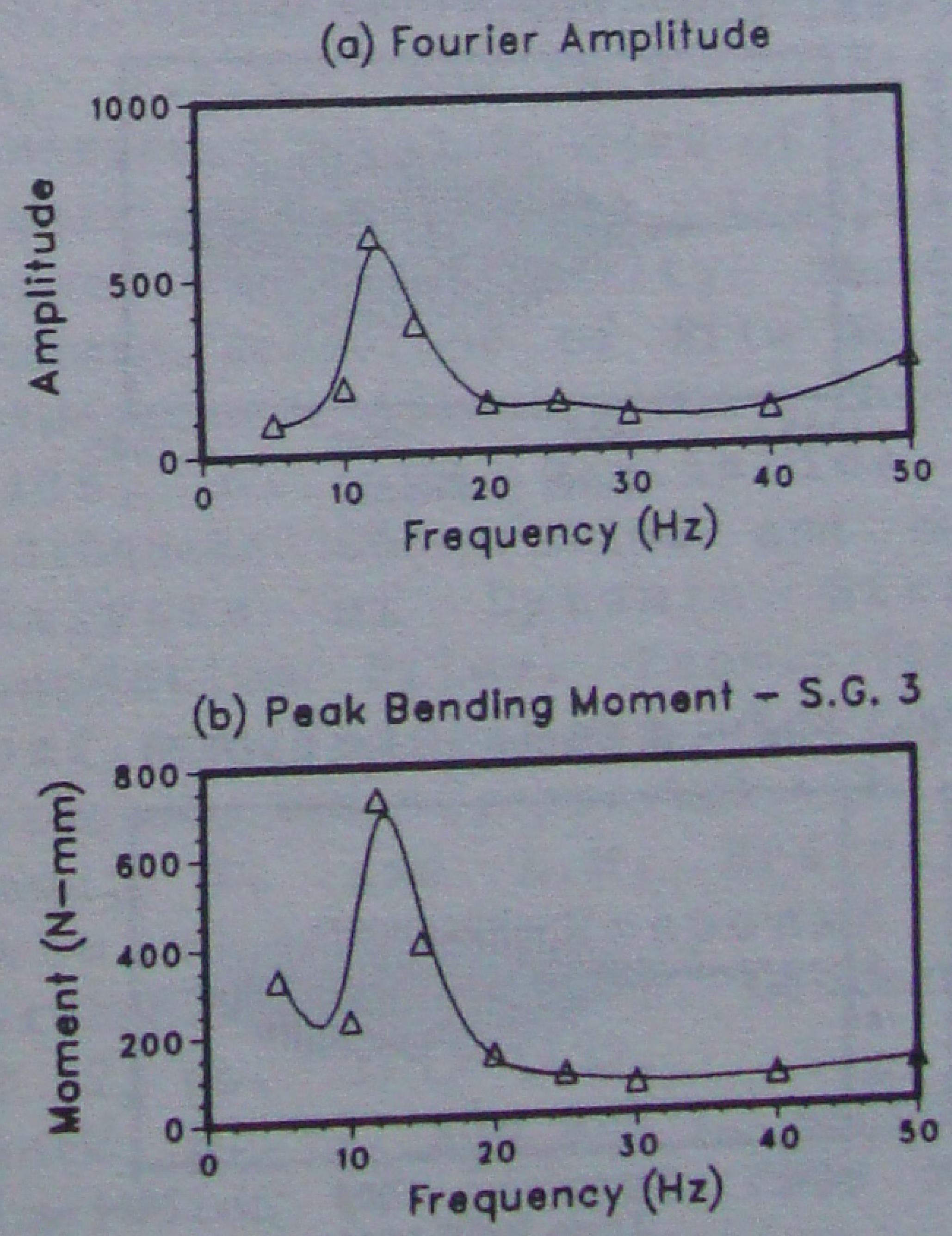

Fig. 10. Frequency sweep tests (a) Fourier amplitude of pile head accelerations, and (b) bending moments versus input frequency.

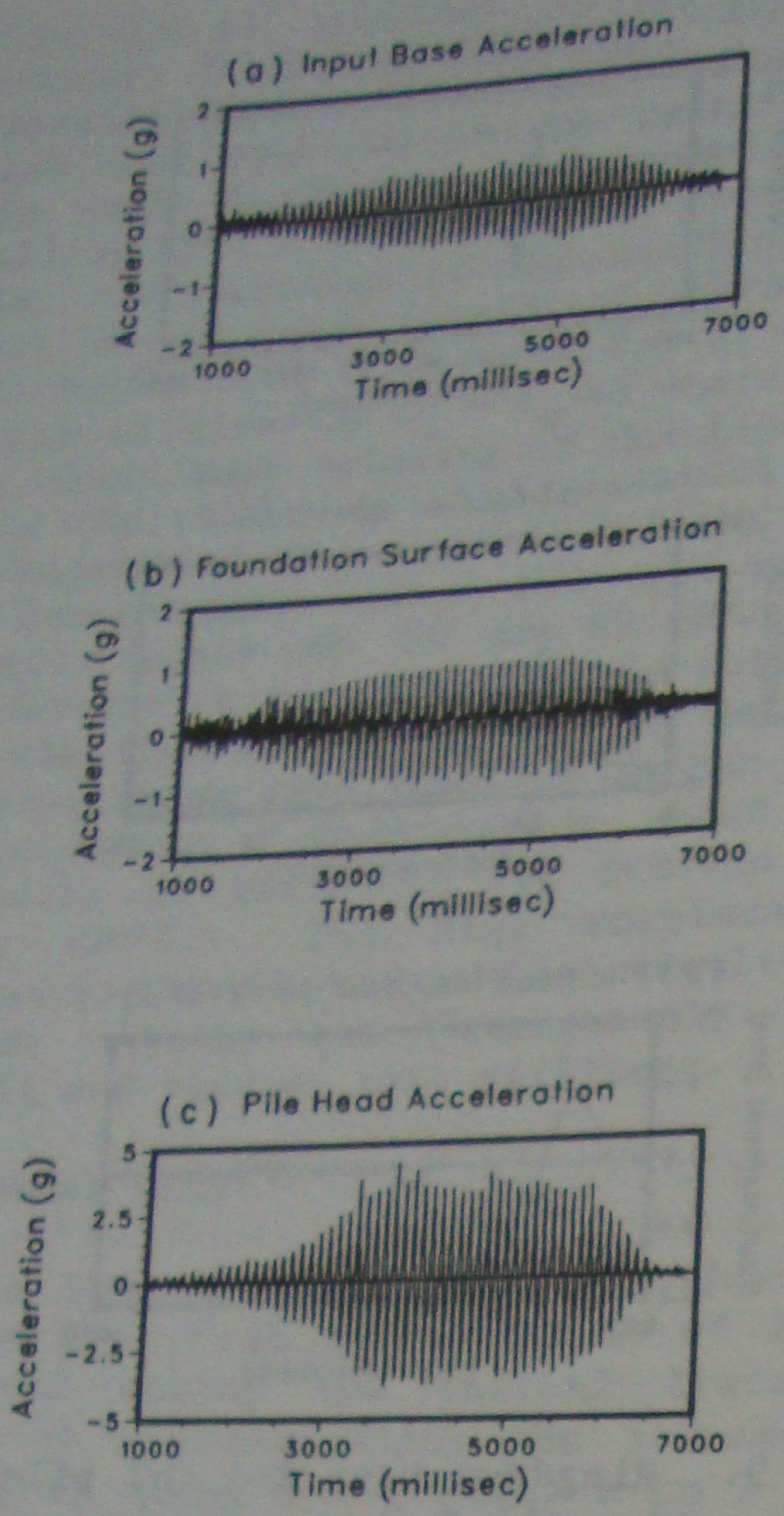

Fig. 11. Shaking table test no. 14 (a) input base accelerations (b) foundation surface accelerations (c) pile head accelerations.

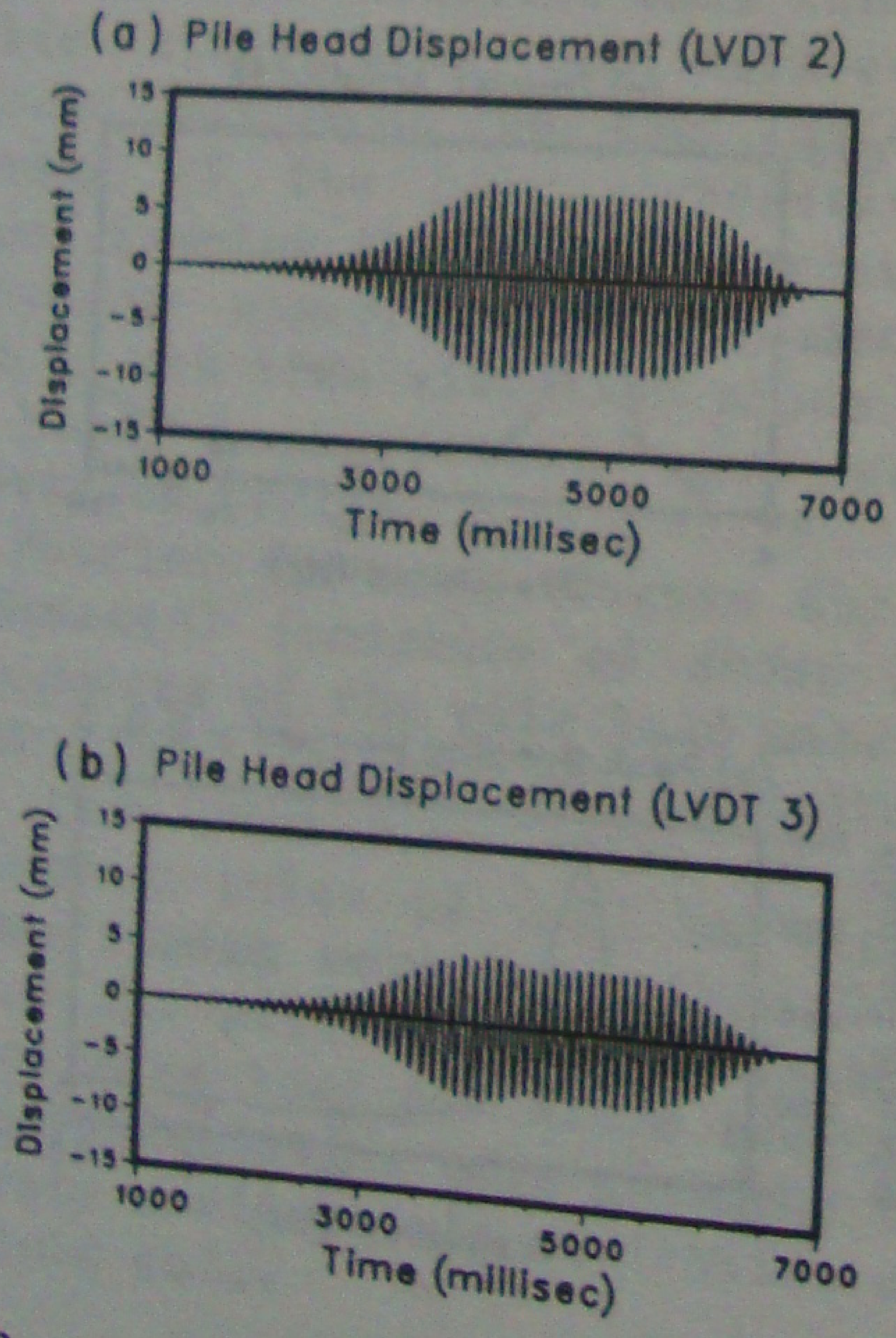

Fig. 12. Shake table test no. 14 – pile head displacements (a) LVDT no. 2 (b) LVDT no. 3.

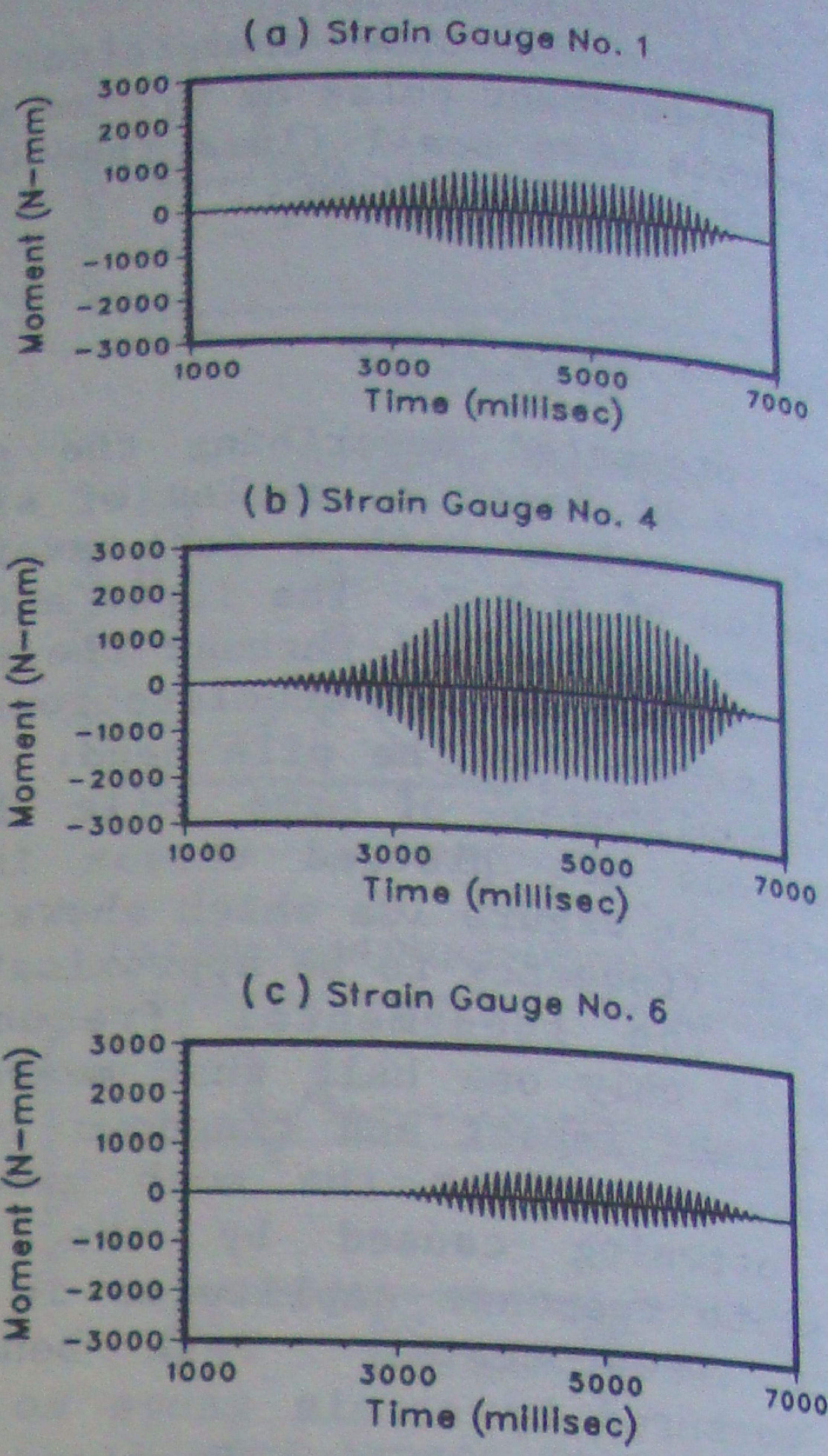

Fig. 13. Shake table test no. 14 - bending moment time histories (a) strain gauge no. 1 (b) strain gauge no. 4 (c) strain gauge no. 6.

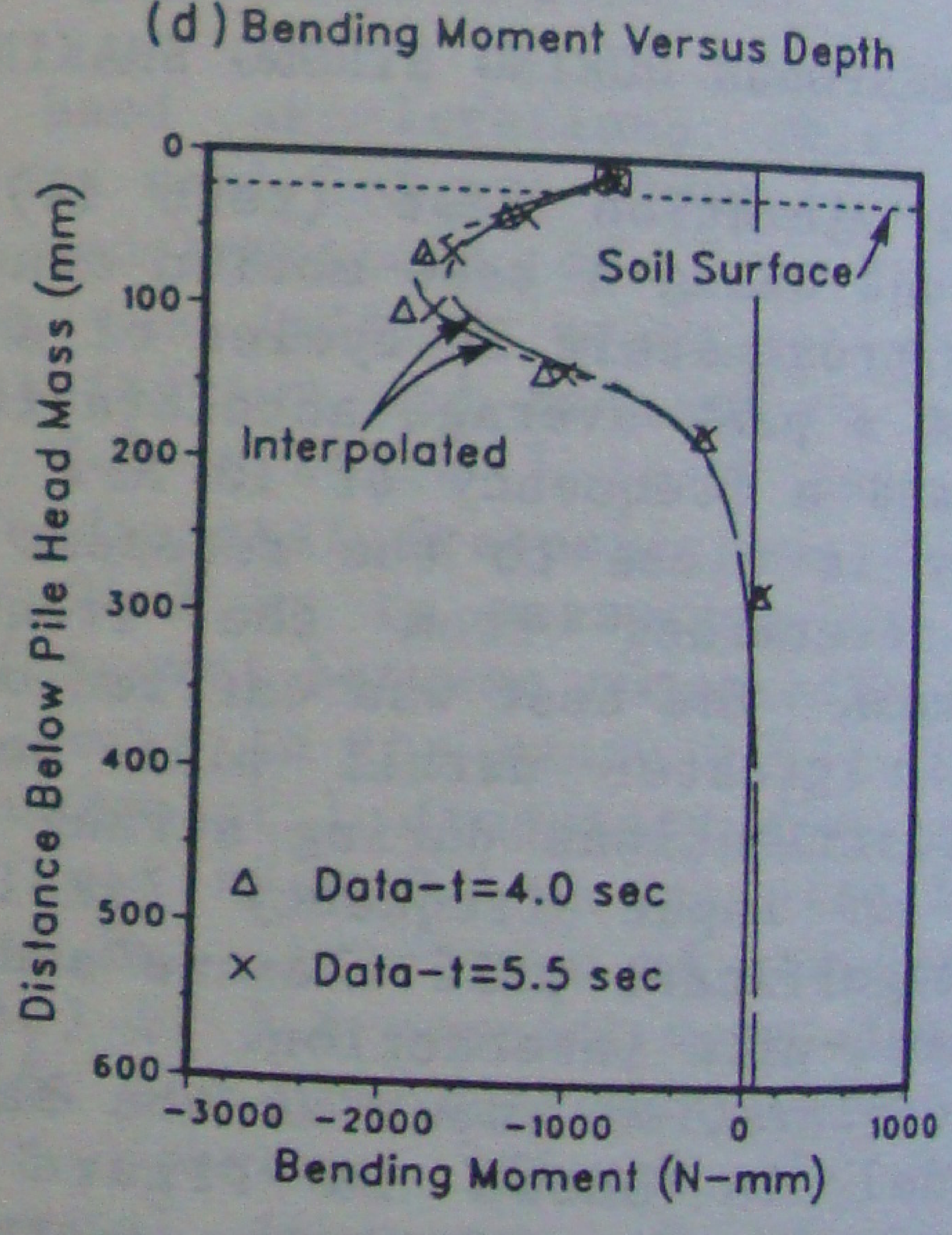

Fig. 13d. Bending moment distribution - shake table test no. 14.

From the data, the following observations may be made:

1) The peak pile head and soil surface accelerations (3.5 g and 0.9 g, respectively) were strongly amplified relative

to the input base acceleration of 0.6 g.

2) Bending moments increase to a maximum near strain gauge 4 (Figure 13d) and then decrease to approximately zero at greater depths. This indicates that the pile may be considered long in the sense that the lower parts of the pile do not influence the pile head response to inertia loads applied at the pile head. The maximum bending moment occurs approximately 13 pile diameters below the soil surface, and equals 40 percent of the initial yield moment of the pile.

3) The spatial variation of bending moments along the pile (Figure 13d) show that all points along the pile experience the same sign of bending moment at any instant in time with the exception of strain gauge no. 7 which shows a small moment of opposite sign. This suggests that the free headed pile is vibrating substantially in its first mode.

4) Peak bending moments occurring in the pile decrease with number of load cycles (Figure 13). This indicates that strain hardening of the soil adjacent to the pile is occurring due to compaction of the sand under the repetitive loading.

5) The maximum pile deflection near the base of the pile head mass (LVDT no. 3) is approximately one pile diameter, suggesting that significant soil non-linearity developed during the lateral pile motion.

6. CONCLUSION

Simulated earthquake loading tests were conducted on single piles embedded in dense dry sand to provide data for checking current methods for the analysis of the seismic response of pile foundations.

The fundamental natural frequency of the pile was determined in tests involving different amplitudes of excitation. Test data show a pronounced reduction in natural frequency ranging from 24 to 12 Hz as the strength of shaking increases. This is attributed to increasing cyclic strain softening of the soils near the pile which leads to lower moduli and lateral soil stiffness.

The lateral pile response to sinusoidal base motion input showed that the free headed pile vibrated mainly in its first mode. Peak bending moments in the pile occurred approximately 13 pile diameters below the soil surface and decreased slightly with increased loading cycles due to sand compaction.

A feature of these shake table tests is that the distribution of shear moduli in the foundation was measured insitu using piezoceramic bender elements. This represents an improvement in shake table testing as it allows an accurate measure of the moduli necessary to correlate model test results with analytical predictions of pile response to earthquake shaking.

The experimental procedures adopted in the program and the instrumentation worked very well throughout and demonstrated that this type of testing can provide useful information on pile behaviour during earthquake loading.

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REFERENCES

Berger, E., S. Mahin, R. Pyke. 1977. Simplified Method for Evaluating Soil-Pile-Structure Interaction Effects. 9th O.T.C. Houston, Texas, pp. 589-594.

Finn, W.D.L. and W.B. Gohl. 1987. Centrifuge Model Studies of Piles Under Simulated Earthquake Loading. ASCE Convention, Specialty Session on Dynamic Behaviour of Pile Foundations (pre-print), Atlantic City, N.J.

Hamada, M. and O. Ishida. 1980. Earthquake Observation and Numerical Analysis of Dynamic Strain of Foundation Piles. Proc. 7th World Conf.on Earthquake Engineering, Istanbul, Vol. 3, pp. 435-442.

Kagawa, T. and L.M. Kraft. 1981a. Lateral Pile Response During Earthquakes. ASCE, J. Geotech. Eng., GT 12, pp. 1713-1732.

Matlock, H., S. Foo and L. Bryant. 1978. Simulation of Lateral Pile Behaviour Under Earthquake Motion. Proc. Earthquake Engineering and Soil Dynamics, ASCE Specialty Conference, Pasadena, California, pp. 601-619.

Nogami, T. 1985. Nonlinear Dynamic Winkler Model for Lateral Cyclic Response Analysis of Single Piles. Soil Dynamics and Earthquake Engineering. Proc. 2nd Int. Conf. on board the liner Q.E. II.

Nogami, T. and M. Novak. 1977. Resistance of Soil to a Horizontally Vibrating Pile. Journal of Earthquake Engineering and Structural Dynamics. Vol. 5, pp. 249-261.

Novak, M., T. Nogami and F. Aboul-Ella. 1978. Dynamic Soil Reactions for Plane Strain Case. ASCE, Journal of the Engineering Mechanics Division, Vol. 104, No. EM4, pp. 953-959.

Norwegian Geotechnical Institute. 1984. The Use of Piezoceramic Bender Elements to Measure G_{max} in the Laboratory. Internal report by R. Dyvik, No. 40014-3.

Ramsay, S.R. 1982. A Study of the Dynamic Performance of a Shaking Table. Dept. of Civil Engineering, U.B.C., M.Sc. Thesis.

Reese, L.C., W.C. Cox and S.E. Koop. 1974. Analysis of Laterally Loaded Piles in Sands. 6th Offshore Technology Conference, Houston, Texas, OTC 2080.

Shirley, D.J. and L.D. Hampton. 1978. Shear Wave Measurements in Laboratory Sediments. J. Acoustical Society of America, Vol. 63, No. 2, pp. 607-613.

Stokoe, K.H., S.H. Lee and D.P. Knox. 1984. Shear Modulii Measurements Under True Triaxial Stresses. Proc. ASCE Convention, Geotechnical Engineering Division, from "Advances in the Art of Testing Soils Under Cyclic Conditions", Detroit, Mich., pp. 166-186.

Sugimura, Y. 1977. Earthquake Observation and Dynamic Analysis of a Building Supported on Long Piles. Proc. 6th World Conf. on Earthquake Engineering, Vol. 2, New Delhi, pp. 1570-1575.

Yasuyuki, E. and Y. Yoshida. 1980. Convenient Aseismic Design of Pile Foundations. Proc. 7th World Conf. on Earthquake Engineering, Istanbul, Vol. 3, pp. 419-426.