FULL-SCALE PILE VIBRATION TESTS

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16. Abstract (Limit: 200 words)

A test program designed to study the potential phenomenon of liquefaction in soil adjacent to pile lead and to monitor the dynamic response characteristics of the pile-soil system during vibration is summarized. A full-scale, instrumented steel pipe pile was driven into saturated silty sand and subjected to dynamic lateral loads by use of two vibration generators. Pile bending moments, displacements, and accelerations, as well as pore pressures and ground velocities in the surrounding soil, were measured. The vibratory loads were of the magnitude of actual force expected during earthquake loading. Partial liquefaction and subsidence developed around the head of the pile during vibration near the first mode resonance frequency. The liquefaction resulted in a loss of soil resistance in the critical upper layers of soil surrounding the pile. Field tests and analytical studies are summarized and results are presented in terms of pile resonance frequencies, model shapes, damping, bending moments, forces, and displacements.

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The tasks involved in this research effort were divided between the two principal parties involved, Ertec Western, Inc. and the California Institute of Technology. Ertec assumed responsibility for site selection and investigation, design, construction, and installation of the pile-platform system, strain gage and deflection instrumentation, and overall management of the project. Caltech supplied the vibration generators, other instrumentation, and all data acquisition equipment, and was primarily responsible for obtaining and processing data during and after the tests. The tests were planned jointly by the two groups, whose personnel participated in all the tests.

The principal investigators for the research were Dr. Ronald F. Scott of Caltech and Dr. Chan-F. Tsai of Ertec. Dr. Tsai also performed the pre-test analyses of pile response as an aid to designing the system. For Ertec, Hudson Matlock served as

a technical advisor for many aspects of the project; design and construction of the pile-platform system and instrumentation were performed by Daniel Steussy, Dewaine Bogard, and Nigel Kay. Barbara Turner assisted on both the pre-test analysis and instrument set-up parts of the study. The piezometers, accelerometers, and special signal processing equipment were manufactured by John Lee, and the vibration generators were installed and operated by Raul Relles, both of Caltech. John Lee also supervised the design and operation of the data acquisition systems. Personnel participating in the tests included Chan-F. Tsai, Ronald Scott, Dan Steussy, John Lee, Raul Relles, John Ting, and Fred Randall, who also were responsible for writing the report. Test analyses were performed by Ting and Scott with the assistance of Randall.

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ABSTRACT

A full-scale, instrumented steel pipe pile was driven into saturated silty sand and subjected to dynamic lateral loads by use of two vibration generators. The principal objectives of the test program were to study the potential phenomenon of liquefaction in the soil adjacent to the pile lead, and to monitor the dynamic response characteristics of the pile-soil system during vibration.

The vibratory loads were of the magnitude of actual force expected during earthquake loading. Pile bending moments, displacements, and accelerations, as well as pore pressures and ground velocities in the surrounding soil were measured. The frequeny and magnitude of vibratory loads were varied during the tests, and "plucking" tests were performed.

Partial liquefaction and subsidence developed around the head of the pile during vibration near the first mode resonance frequency. The liquefaction resulted in a loss of soil resistance in the critical upper layers of soil surrounding the pile.

This report summarizes the field teests and analytical studies, and presents the results in terms of pile resonance frequencies, model shapes, damping, bending moments, forces, and displacements.

1. INTRODUCTION

1.1 Background

The use of piles to support structures is widespread. By their nature, piles are employed in areas where the soil conditions near ground surface are poor, frequently involving saturated materials of low relative density and strength. When the pile-supported structure is in a seismic area, it has the potential of being subjected to cyclic lateral (as well as vertical) loads. The pile system is shaken by the seismic waves propagating through the contiguous soil. Due to the influence of structural stiffness and mass, the tops of the piles undergo motion different from that of the supporting soil mass. This soil-pile interaction under cyclic loading is of importance in seismic design of piles.

Although the behavior of the piles under cyclic vertical loading is of interest and has not been extensively studied, the loads are incremental to the existing vertical force for which the piles have been designed, with a suitable factor of safety. In the normal static state small or zero lateral loads are borne by the piles, so that earthquake-generated forces in the horizontal direction constitute a new, primary rather than additive loading in this direction.

The interaction of the structure with the pile-soil system has a direct effect on the response of the structure to the input vibrations. Some information is available on the static response of piles and pile groups to lateral load and this can be employed in some designs to develop the structural characteristics. However, a number of studies has shown that soils deteriorate in stiffness and strength under cyclic loading conditions. Clay softens under repeated straining, and saturated sands demonstrate excess pore pressure buildup and sometimes liquefaction. When a pile is imbedded in a saturated clay below water, investigations have shown that lateral loads cause cavities to be opened in the soil-pile interface starting at the mudline [4]. The lateral pile response softens, and the cavities propagate down the pile. The lateral pile movement pumps water in and out of the holes and mixes soil and water to extend the development of softening. This process has been studied to a limited extent. On the other hand, the behavior of saturated sands and silts under these pile loading conditions does not appear to have been investigated in great detail, except by means of limited field measurements and centrifuge studies.

A great deal of research effort has been devoted to the response of saturated sands to cyclic loading and the behavior has been well-characterized. Depending on the relative density of the sand, the number of cycles of shear loading required to cause liquefaction, or an arbitrary large cyclic strain (5% or 10%), is related to the ratio of the cyclic shear stress amplitude to the initial overburden effective stress. Other variables such as soil type, previous history of stress, and random variation in the shear stress level all have an effect.

The information on the soil behavior can be combined with the geometry of the soil arrangement and the load configuration in computer studies to indicate how a particular structure (earth dam, offshore gravity platform) will respond to a hypothetical earthquake loading. However, confirmation of such calculations by fullscale tests is lacking except in those cases where structures have been affected by real earthquakes. In the latter circumstance, most of the relevant data obtained in controlled tests by instrumentation is usually missing, and must be inferred. It is thus difficult, if not impossible, to determine load levels, displacements, stresses, and strains generated during the transitory motion. An example of the failure of a pile-supported structure was the bridge over the Showa River in Niigata, Japan. During the earthquake of June 1964, in which soil liquefaction was widespread, a number of spans of the bridge fell off the supporting piers into the river. Displacement of the piles was apparently the direct cause of the collapse, but it is not possible to say whether vibratory motion of the piles contributed to local liquefaction, or the collapse was the result of general liquefaction of the river bed.

The best method of obtaining immediately useful results on the field performance of piles imbedded in saturated fine sand and subjected to cyclic lateral dynamic loads is to test full-scale piles under these conditions. Only a few such tests have been conducted. Centrifuge tests on a pile with almost the same dimensions as the present test pile have been reported [8,9].

Novak and Grigg [7] carried out vibration tests on large model (or small prototype) piles, and normal-size pile dynamic tests have been performed by Richart and Chon [10]. Alpan [1] studied 30 cm square prestressed reinforced concrete piles 4.7 m long in ring-down tests. These tests were not performed at large enough amplitudes in the soils affected to cause substantial change in the soil's properties. In particular the question of soil liquefaction was not addressed. Because of the importance of the dynamic lateral behavior of piles to the integrity of structures in seismic zones, and especially to offshore oil structures, it was therefore proposed to the National Science Foundation to carry out dynamic pile tests under fairly high lateral loads. This report describes the research investigation.

1.2 Scope of Work

Two steel pipe piles were driven into a medium-dense, saturated, fine sand at mean tide level on a beach in the U.S. Navy Weapons Station in Seal Beach, California. The piles, having a 24-inch outside diameter and a 0.5-inch-thick wall, were driven to a depth of 32 ft. The piles were subsequently instrumented with strain gages. The instrumentation and soil conditions are described in later sections of this report. A steel platform was constructed, welded to one of the piles, and loaded with approximately 24 tons of lead weights. On the platform were attached two shaking machines from the Dynamics Laboratory of the California Institue of Technology.

By adding weights to the rotating baskets of these machines and by changing the frequency, the acting dynamic force on the pile was varied from a few hundred to a few thousand pounds. At the higher level, this force was a substantial fraction of the lateral dynamic force which such a pile as part of a structure might encounter in an earthquake. The rotational rate of the shaking machine was precisely set by a speed controller which was employed to sweep the frequency range from 0 to about 8.5 Hertz to seek out resonances. The load at any given frequency was altered by changing the weights in the shaking machine. A photograph of the test setup is shown in Figure 1.1.

With such a system, various tests of interest were performed. At a low load amplitude, natural frequencies of the loaded pile were explored by a frequency sweep without developing large-enough strains in the adjacent soil to cause liquefaction or significant excess pore pressure build up. At maximum load from two machines at resonance, it was possible to develop considerable lateral movements of the piles at ground surface. If liquefaction of the adjacent sand occurs under these conditions, it would have developed in the latter test.

Full utilization of the test results required the mobilization of a suitable compendium of instrumentation. This is described in more detail subsequently, and included load and displacement equipment and strain gages in the piles. The platform displacement and acceleration were recorded during the dynamic tests, and ground movements were observed adjacent to the piles through the use of an array of Ranger seismometers. Visual observation and changes in pile response were not sufficient in themselves to indicate the presence or absence of liquefaction, so that pore-pressure measuring equipment was installed in the soil.

To get a clearer idea of damping in the pile-soil system, "ring-down" tests were also included. The pile was pulled laterally by a cord which was abruptly released, so that the pile vibrated to rest from the initially displaced position. Records of all instrumentation during this test indicated the levels of force and moment achieved.

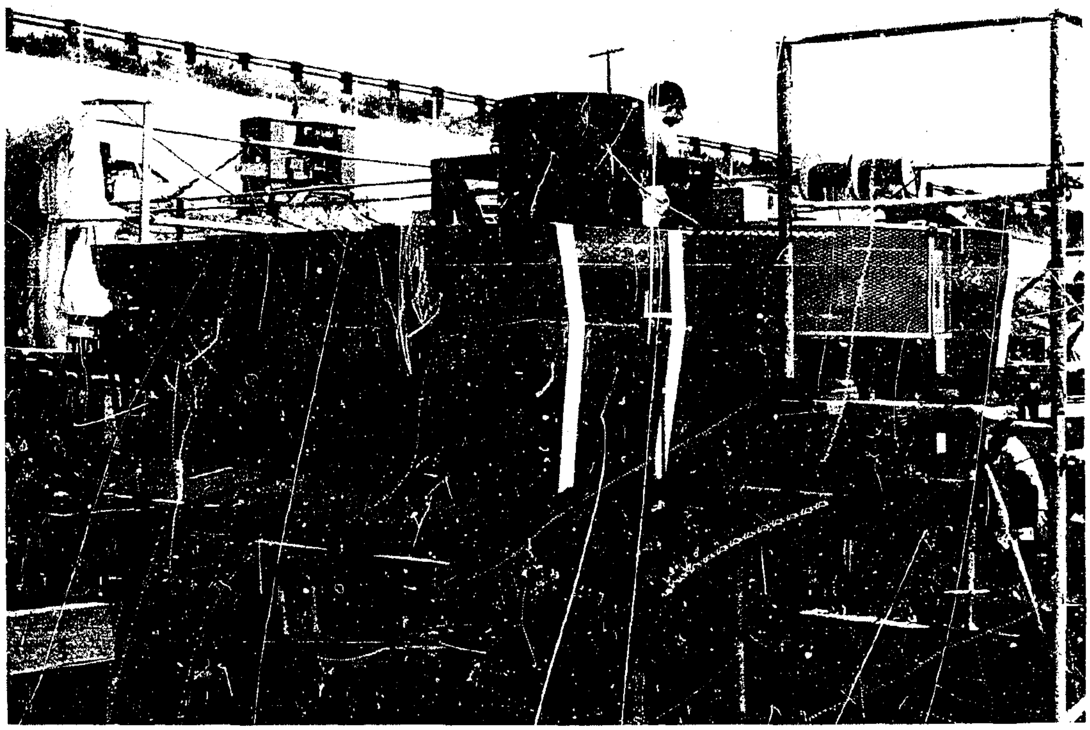
One difficulty with all such dynamic tests is that the soil properties change during the test. If liquefaction occurs in a zone adjacent to a pile, the overall soil behavior will be significantly altered. As a consequence, resonance will not occur in subsequent tests at the same frequency. This was noted during the tests. The changing soil response was monitored by the pile and pore pressure instrumentation.

1.3 Purpose of the Research

As a result of the experiments outlined in the previous section, it was intended to obtain information on the dynamic behavior of a free-end, vertically loaded, full-scale pile embedded in a medium-dense saturated fine sand. The resonance frequencies of the pile were determined at different levels of the forcing function and were ultimately related to the soil properties. The characteristics of the system important to the design of piles under lateral seismic loading were elucidated. Particular emphasis was placed on measuring the ground and pore water pressure response in the region adjacent to the pile. The pile behavior was compared to its response calculated by different current analysis methods.

1.4 Report Outline

This report contains detailed descriptions of the site, soil investigation, test setup, and instrumentation. The analytical studies which preceded the experiments are summarized. Finally, the test program to date is described, and preliminary results and interpretations are presented. Conclusions and recommendations for future work are also included.



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NSF PILE TEST	

VIBRATION TEST OF PILE IN PROGRESS

8-81

FIGURE 1.1

2. DESCRIPTION OF TEST SITE

2.1 General

The purpose of the test program described herein is to study the interaction between a pile and a saturated, cohesionless soil under vibratory lateral loading. One of the major objectives in the study is to examine the effects of potential liquefaction, full or partial, on the soil-pile system during vibration. The first obvious test requirement, therefore, was to find a test site with a soil profile of loose to medium dense sand or silty sand and with a water table very close to the ground surface. Through the cooperation of the Naval Civil Engineering Laboratories at Port Hueneme, California, such a site was located at the North Island Naval Air Station in San Diego, California. Initial plans were then drawn up by Ertec Western and Caltech to perform the pile vibration tests at the North Island site. However, it was found that the water table at the North Island site is located several feet below the ground surface. For optimal test results, therefore, extensive and undesirable excavation would have been necessary. These and other economic considerations later forced the relocation of the test program to a site closer to the Los Angeles - Long Beach area.

A site which conformed to the needs of the research program was found from a search of company records by personnel in the Ertec Foundation Engineering Group. The region was located at the southern city limits of Seal Beach, Orange County, California, five miles south of the Long Beach city limits (Fig. 2.1). The U.S. Navy owns the property, which lies on the principal tidal inlet into Huntington Harbor at the eastern end of Anaheim Bay (Fig. 2.2). Permission from the Navy to use the property was granted in January, 1981.

2.2 Topography and Climate

The test site is a sandy, flat-bottomed small bay or cut into the shoreline adjacent to Anaheim Bay (Fig. 2.3). The cut is approximately 120 ft (37 m) wide and 250 ft (76 m) long, and is flooded twice daily by tides. Elevation differences of low and high tides vary from 3.0 to 8.5 ft (0.91 to 2.59 m). The temperature of the seawater, identical in the bay and in the soil tested, was a constant 68° F (20° C). The average daytime air temperature for the periods of testing was about 78° F (26° C).

2.3 Soil Profile

The soil at the Seal Beach test site consists of 18 to 20 ft (5.5 to 6.1 m) of medium dense uniform silty sand overlying strata of silt, clayey silt, sand, and siltstone. The upper layer of sand is of the most interest to the project, since effects of the pile on the soil below about 10 pile diameters [20 ft (6.1 m) in this case] are negligible except under the very largest of lateral loads. The twice-daily floodings of the area ensure the saturation of the soil, making the location an ideal one for the study of vibration and liquefaction (Fig. 2.4).

The site soil conditions were evaluated by performing two types of in-situ soil tests on the site soils during the preliminary geotechnical investigations: cone penetration tests with the Ertec electric friction cone penetrometer, and continuous standard penetration tests with the standard split spoon sampler. The soil profile is presented in Fig. 2.4, and field logs of both test programs are given in Appendix A. Locations of boreholes and soundings are also shown in Appendix A.

10 A

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SEAL BEACH SITE LOCATION

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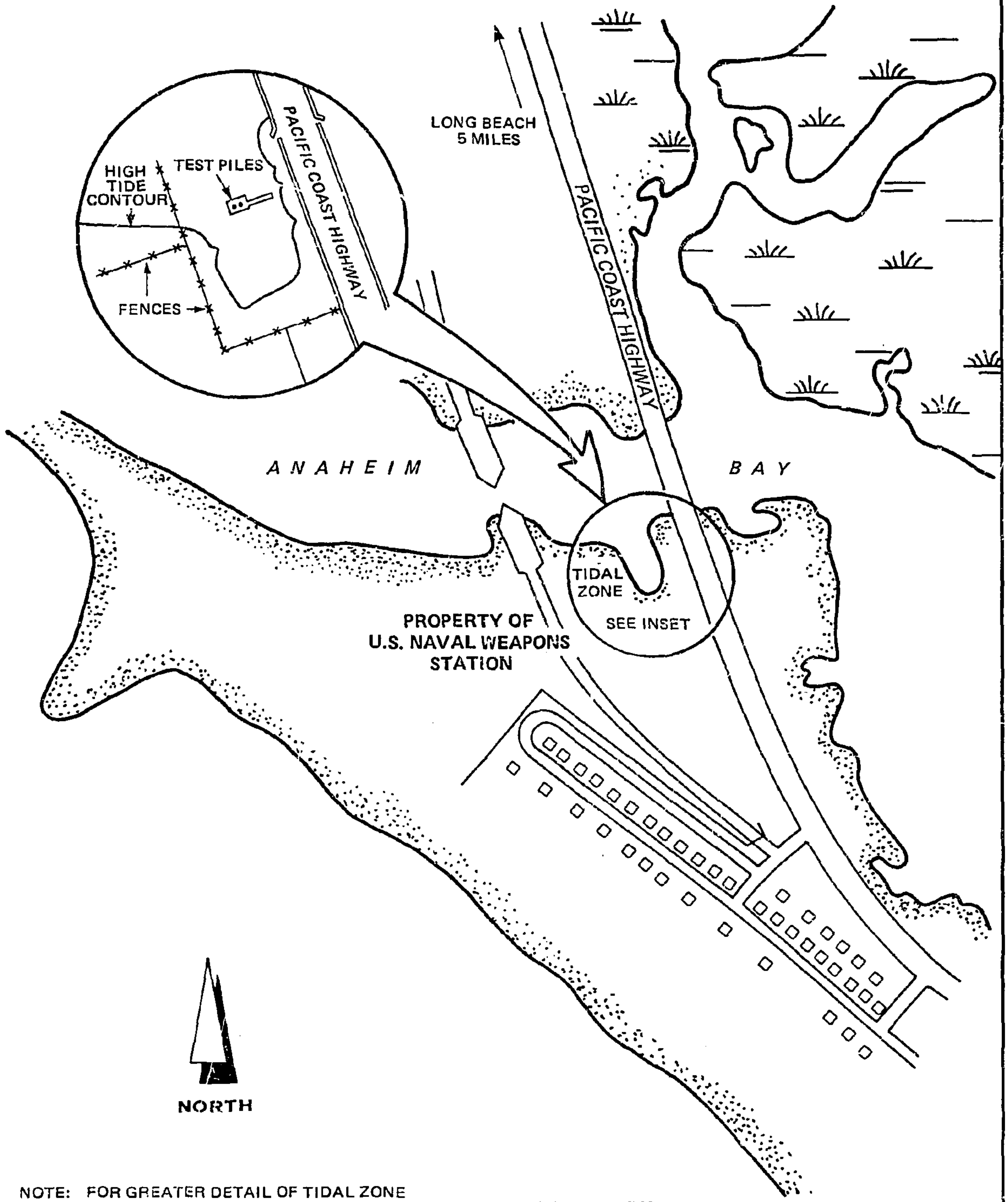
FIGURE 2.1



NOTE: FOR GREATER DETAIL OF TIDAL ZONE AND TEST PILE LOCATION, REFER TO FIG. A. 1

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SEAL BEACH SITE LAYOUT

8-81 FIGURE 2.2

Compiled by ____ Drawn by ____ Checked by ____ Approved by ____

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PHOTOGRAPH OF SITE LOCATION
WITH PILES IN PLACE,
FACING NORTHWEST

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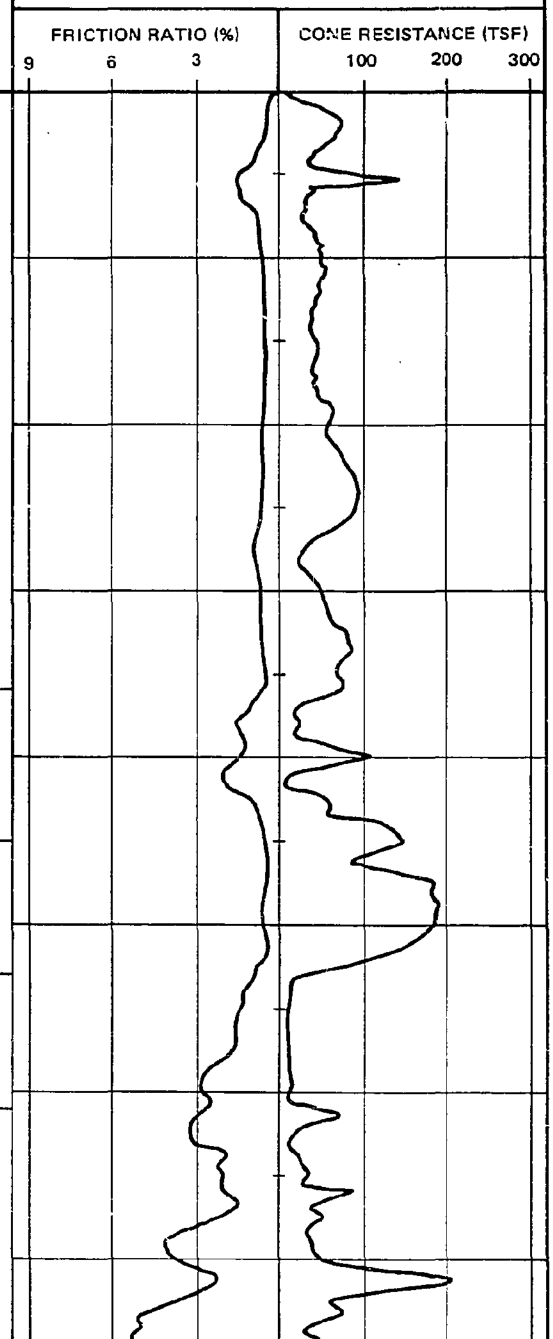
FIGURE 2.3

10 D

DEPTH (FT)	SYMBOL	SAMPLE TYPE	BLOWS/FT*	SOIL DESCRIPTION
		S	13	DARK GRAY, FINE SILTY SAND WITH SCATTERED SILT AND SMALL SHELLS, LOOSE TO MEDIUM DENSE
		S	16	
5				
		NR	12	
		NR	13	
10		S	13	
		S	10	
		S	17	
		S	10	
15		S	14	
		S	9	
		S	11	DARK GRAY SILT WITH SOME SAND AND CLAY, MANY SMALL SHELLS, MEDIUM DENSE
20		S	10	
		S	20	
		S	25	DARK GRAY, MEDIUM TO FINE SAND, WITH SOME SILT, VERY SHELLY, DENSE
25		S	21	
		S	5	
		S	7	DARK GRAY, FIRM CLAYEY SILT
		S	14	
30		S	10	GRAY-GREEN FRAGMENTED SILTSTONE AND STIFF SILT
		S	28	
		S	22	
35		S	31	
		NR	23	

TYPICAL CONE PENETROMETER PROFILE (CPT-3)



ELEVATION: MEAN SEA LEVEL DATE DRILLED: 11-18-80 (SPT)
EQUIPMENT USED: FAILING 750 & CPT TRUCK
WATER LEVEL: 0'

[S] STANDARD SPILT SPOON SAMPLE (ASTM D1586)
NR NO RECOVERY
*AVERAGE VALUE OF BORINGS 1 & 2 (WHEN APPLICABLE)
*STANDARD 140 LB HAMMER DROPPED 30"

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SEAL BEACH SITE SOIL PROFILE

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FIGURE 2.4

Approved by ____ Checked by ____ Drawn by ____ Compiled by ____

3. TEST CONFIGURATION

3.1 General

The field test program involved a considerable amount of planning, coordination, and eventually mobilization of personnel and equipment. Aside from the normal amount of preparation necessary for a large instrumentation project of this type, several other factors contributed to the complexity of the undertaking. The most important consideration was the lack of security at the site, which made impossible the establishment of a semi-permanent facility for storing and operating equipment. The periodic flooding of the site by tides also affected project planning. This chapter describes most of the components brought together for the field tests and the steps taken in the progress of the project.

3.2 Site Preparation

Commencement of the test program was preceded by some modifications to the site, made necessary by several reasons. The topography of the location made access to the site by heavy equipment nearly impossible. Since the test piles were eventually driven at the mean tide mark, normal tidal fluctuations placed some restrictions on testing operations. The location of the site, in a secluded area obscured from view from roadways and other points of surveillance, made vandalism a possibility and sometimes an actual problem.

To facilitate access, some earth moving was done to provide an entrance to the area and a ramp from the higher elevation of the surrounding embankment down to the test location. In order to work around the test piles during periods of high tide, during which the depth of water around the piles sometimes reached four feet, scaffolding was erected around the piles and test platform. The scaffolding was extended to the adjacent embankment to provide a walkway to the piles during high tide. The extension also served as a support for instrumentation cables and power cords during the tests, since the generators and instrumentation truck were positioned on the embankment for most of the vibration tests. Two fences were erected to provide some security to the area, one across the roadway leading to the site and the other surrounding the test piles and scaffolding. Gates were installed and secured with chains and padlocks.

Unfortunately, some vandalism did occur despite the precautions that were taken. Vandals damaged windows in the crane used for pile driving, and the fences were torn down repeatedly so that unauthorized individuals could park their vehicles near the bay. However, no damage occurred that severely affected the actual test program. Intermittent patrols by U.S. Navy police were of assistance. Because of this problem, it was necessary to set up all of the recorders, generators, and associated cables prior to each day's testing and remove them at the end of the day, even if testing were to continue on the following day.

3.3 Test Piles

The two steel pipe piles driven at the test site were 40 ft (12.1 m) long and 24 in. (610 mm) in outside diameter, with a wall thickness of 1/2 in. (12.7 mm). The diameter of the test piles was chosen to be representative of the size of piles used in conventional construction, but also had to satisfy two other criteria: the pile diameter had to be small enough to exhibit measurable response during the expected ranges of lateral loading, and the diameter had to be large enough for a welder to work inside on the steel gage tubes (discussed subsequently). The thickness of the pile wall was originally selected to be 3/8 in. (9.5 mm); but this was later changed to 1/2 in. (12.7 mm) for the increased strength thought to be required for driving through the dense sand layer between the depths of 22 (6.7 m) and 27 ft (8.2 m) (See Fig. 2.4.).

From the static analyses described in Chapter 5, the maximum bending moments in the pile during maximum possible loading were expected to occur within 10 to 12 ft (3.0 to 3.7 m) from the ground surface. Therefore, the strain gage instrumentation used to measure pile wall strains (Chapter 4) was designed to extend from 2 ft (0.6 m) above the ground surface to 16 ft (4.9 m) below. Based on pretest analytical studies, this arrangement was believed to be adequate to define the moment curve with reasonable accuracy. However, as mentioned in Appendix E, deeper measurement of strain would have been more desirable so that the entire moment curves could have been obtained with greater accuracy.

For this program, a method was developed to instrument the test pile without subjecting the strain gages to the damaging shocks of pile-driving. The method consisted of welding square sealed steel tubing down the inside of the pile before pile installation, and then potting prefabricated strain-gaged tubes with epoxy inside the tubing after the pile was driven. Four lengths of steel tubing, 1-1/2 in. (38 mm) square (outside), were thoroughly cleaned on the inside by sandblasting. Steel caps were welded onto the ends of each length, one of which was designed with a fitting so that those and all subsequent welds could be pressure-checked for leaks. The four square tubes were then welded to the inside pile wall, recessed 6 in. (152 mm) from the top end of the pile and on longitudinal axes diametrically opposite. The welds averaged 6 in. (150 mm) long, spaced 12 in. (300 mm) apart, and were positioned on both sides of each square tube where it met the pile wall. Although the tubes were not continuously welded to the pile, it was considered that the welding scheme would result in compatible deformation (or strain) of pile wall and tube during bending. The lengths of tubing were sealed and pressurized, and all welds were given a final inspection. The procedure is detailed further in Chapter 4.

Two piles were fabricated in the above manner and transported to the site on 15 April, 1981. Both piles were driven on the same day to a depth of 32 ft (9.8 m), centered 12 ft (3.7 m) apart. The diesel hammer used was a Delmag D30-02, capable of delivering energies up to 66,100 ft-lb (9140 kg-m). A summary of the pile driving records is presented in Appendix A. No driving difficulties were encountered, and in both piles the soil plugged inside after a driving distance of approximately 20 ft (6.1 m). After the piles were driven, the tops were fitted with lockable steel caps to prevent unauthorized access to the inside of the piles.

3.4 Vibration Platform

For economic reasons, the testing program evolved from the original concept of simultaneous vibration of two connected piles with some head restraint to the vibration of a single, free-head pile. A platform was designed and built to attach to the pile to hold both the vibration generators and the lead ingots used for dead weight (Fig. 3.1). The steel platform, constructed in December, 1980, was attached to the pile on 20 April, 1981. Steel lugs, upon which the platform was placed, were welded to the pile for this purpose. The two lugs were attached on opposite sides of the pile, perpendicular to the axis of loading. After the platform was placed over the pile and aligned on the lugs, it was welded into place. The method of attachment was designed to simulate a free-head condition of loading without restraint on the pile head.

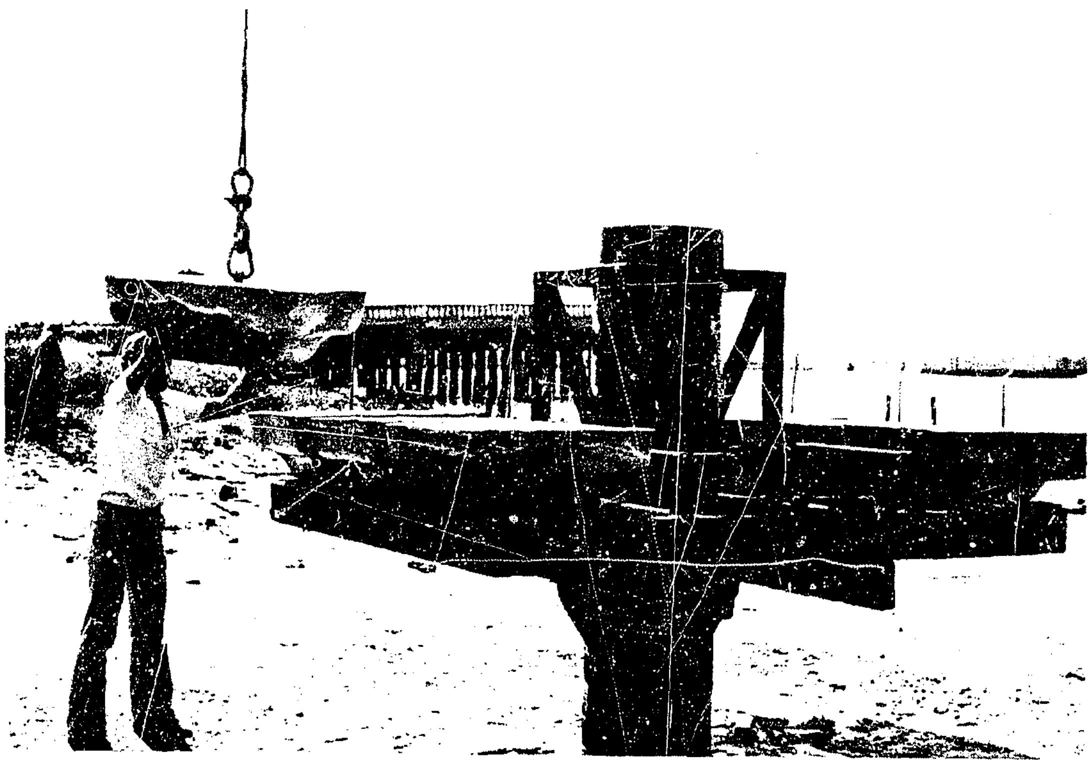
After the platform was secured to the pile, 12 lead ingots were loaded onto the platform, six to a side. Each ingot weighed approximately 2 tons (18 kN), resulting in a total dead load of about 24 tons (213 kN). The amount of the dead weight was selected to be representative of that expected in design applications, or approximately one-third of the expected axial capacity.

3.5 Vibration Generators

Vibration loading of the instrumented pile was achieved by the use of two vibration generators provided by Caltech [3]. Each generator consists of two motor-driven, counter-rotating weight "bucket" assemblies. Lead weights can be added to the buckets to provide dynamic horizontal forces up to 5,000 lb. (22 kN) for each shaker. Each generator is controlled by a separate console, which supplies power to the two motors that rotate the buckets. The two generator/console units may be joined together electrically and operated synchronously through one of the consoles, designated as the "Master" console. The motor speed can be varied from the console to give vibration frequencies between 0 and 8.6 Hz with empty buckets, and 0 to 3.5 Hz with maximum weight in the buckets, depending on the compliance of the system being shaken. Complete specifications and accompanying figures of the vibration generator system are presented in Appendix B.

The electrical power required for the vibration generators is single phase, 30 ampere, 220 volt. Since the vibration generators were operated at a large range of frequencies and applied loads, the fluctuations in current were quite significant during testing. A trailer-mounted, 30 KW, diesel-powered AC generator was used to supply the necessary power.

16A



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PLATFORM USED TO HOLD
LEAD WEIGHTS AND
VIBRATION GENERATORS

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FIGURE 3.1

4. INSTRUMENTATION

4.1 General

Five parameters were measured to ascertain the behavior of the NSF test pile during cyclic and transient loading. For a given magnitude and frequency of vibratory loading, the parameters were:

1. Bending moment (strain) in the pile
2. Porewater pressure in the surrounding soil
3. Acceleration of the pile head
4. Lateral displacement of the pile head
5. Ground velocity at the surface of the surrounding free field

The following sections decribe the instruments used to monitor the above parameters and the equipment used to record the measurements. The methods used to reduce the data are discussed in Chapter 7. The configuration of the test set-up is depicted in Fig. 4.1.

4.2 Strain Gages

One of the major objectives of the test program was to examine bending moments in the pile during vibration and to see how those moments changed when the loads and vibration frequencies were varied. The method most commonly used to obtain bending moments in a structural member is to attach electrical resistance strain gages to some surface of the member to measure mechanical strain at that point. The moment is then determined from the following well-known relation:

$$\varepsilon = \frac{M\,y}{E\,I}$$

where ε is mechanical strain, M is bending moment, y is the distance from the neutral axis to the point of strain measurement, and EI is the flexural stiffness of the member. The damaging effects of pile-driving and a moisture-laden environment, however, make the instrumentation of piles with strain gages an activity prone to failure.

As mentioned in Chapter 3, an approach of instrumenting the test pile after pile driving was employed to avoid the risk of gage damage during driving or as a result of moisture intrusion. The method consisted of epoxying pre-fabricated strain-gaged steel tubes into sealed sections of larger steel tubing, previously welded to the inside of the pile wall. Using this method, bonded electrical resistance strain gages could be mounted onto the smaller sections of steel tubing under laboratory conditions, checked for function and stability, then sealed against moisture. The larger sections of steel tubing were welded to the inside of the pile at a fabrication yard, sealed, and checked for leaks.

The strain gage configuration is shown in Fig. 4.2. Four gages were employed at each depth of measurement in tubes at opposite ends of a diameter and were wired together in a Wheatstone bridge arrangement. In this manner the two gages on one side of the pile could be wired as opposing each other as well. For a given moment in the pile, therefore, the respective pairs of opposing gages would be in tension and compression. Since the strain-gaged tubes were positioned directly opposite each other, the strains were always equal in magnitude and opposite in sign.

The construction and fabrication of the pile and welded sections of tubing were discussed in the previous chapter. Strain-gaging of the inner steel tubes was performed separately in a clean, controlled, laboratory environment (Fig. 4.3). Steel tubes, one inch in outside diameter and 1/16 in. (1.6 mm) in wall thickness, were thoroughly cleaned and precisely marked at the gage locations. The eight pairs of gages were placed at 32 in. (813 mm) intervals, with the exception of the top two at an interval of 24 in. (610 mm). Affixing and sealing of the gages were performed according to manufacturer's specifications except where extra precautions were taken to clean and moisture-proof the tubes and gages.

Before the strain tubes were epoxied into the pile, both of the recipient steel tubes were again checked for moisture and given a final cleaning. Epoxy potting compound was then mixed and poured into the welded tubes, and the strain-gaged tubes were pushed into place, permanently potting them and sealing the gages from any further intrusion of moisture. Initial checking indicated that a solder terminal of one bottom station gage was touching the inside of the welded steel tubing, shorting gage to ground. This bottom gage functioned intermittently throughout the test program. The other gages performed satisfactorily.

4.3 Piezometers

During the cyclic stressing of a sand, porewater pressure increases, causing a reduction of effective stress. To measure the fluctuations and progressive increases of porewater pressure around the pile during vibration, piezometers were placed at varying depths and distances from the pile. Two types of piezometers were used: hollow tubes incorporating electrical resistance strain gage pressure transducers, and conventional stand pipe piezometers.

Each electrical pore pressure unit consisted of a hollow, cylindrical aluminum tube, in which holes were bored, the outside of which were covered by fine screen wire. Small pressure transducers (Sensotec "S" Type) were threaded into the holes from the inside and sealed on the inside against water intrusion. Water pressure, through the screen, acted directly on the transducer diaphragm whose deflection was measured electrically. The transducer wires were brought to the surface through the tube. Drawings of the piezometers are given in Appendix C.

Four piezometer units as described above were used. The units were built into three sections of pipe which were pushed into the ground 11, 23-1/2, and 59 in. (280, 600, and 1500 mm) away from the pile wall along the axis of loading. The piezometer arrangement is shown in Fig. 4.4. A hydraulic frame was used to install the piezometers as depicted in Fig. 4.5. As the piezometers were pushed into the sand, continuous readings of pore pressure were obtained.

Due to a large zero shift of the transducers during the period of testing, electrical output was recordable on the HP recorder only at decreased sensitivity. Consequently, small changes in pore pressure were barely detectable even when the piezometers were operating properly. To get a visual indication of the porewater pressure changes during vibration, two lucite standpipe piezometers were pushed down until the porous tips were 34 in. (864 mm) below the ground surface. The tubes were then filled with a mixture of ink and seawater to the ground surface. The standpipes were positioned next to the two innermost electrical transducer piezometers, 11 and 23-1/2 in. (280 and 600 mm) away from the pile. The standpipes were used only for the last day of testing, and only the one closest to the pile exhibited a change in water head during pile vibration. Inspection of the standpipes at the end of the day revealed that the porous tips of both units had plugged with soil.

4.4 Accelerometer Package

A miniature, three-dimensional accelerometer package was built for attachment to the pile wall. Three Entran ECA-125 miniature accelerometers were mounted on an aluminum plate, which was in turn bolted to the pile 24 in. (610 mm) from the ground surface (Fig. 4.6). The package functioned properly throughout the test program.

4.5 Displacement Transducers

Lateral deflection of the pile head during loading was determined in two ways: double integration of the moment-versus-depth curve obtained from the strain gage readings, and direct measurement at two points on the pile itself. The latter was accomplished through the use of two Direct Current-Linear Variable Differential Transducers (DC-LVDT) located at 12 and 36 in. (305 and 915 mm) above the ground surface. Long stroke (±3.0 in., ±76 mm) DC-LVDT's were used for the measurements, manufactured by Schaevitz (Model 3000 DC-D). Comparison of the computed deflections from double integration of the moment curves and the measured deflections from the DC-LVDT's shows maximum difference of less than 0.1 in. (2.54 mm) (See Figs. 6.4 through 6.7).

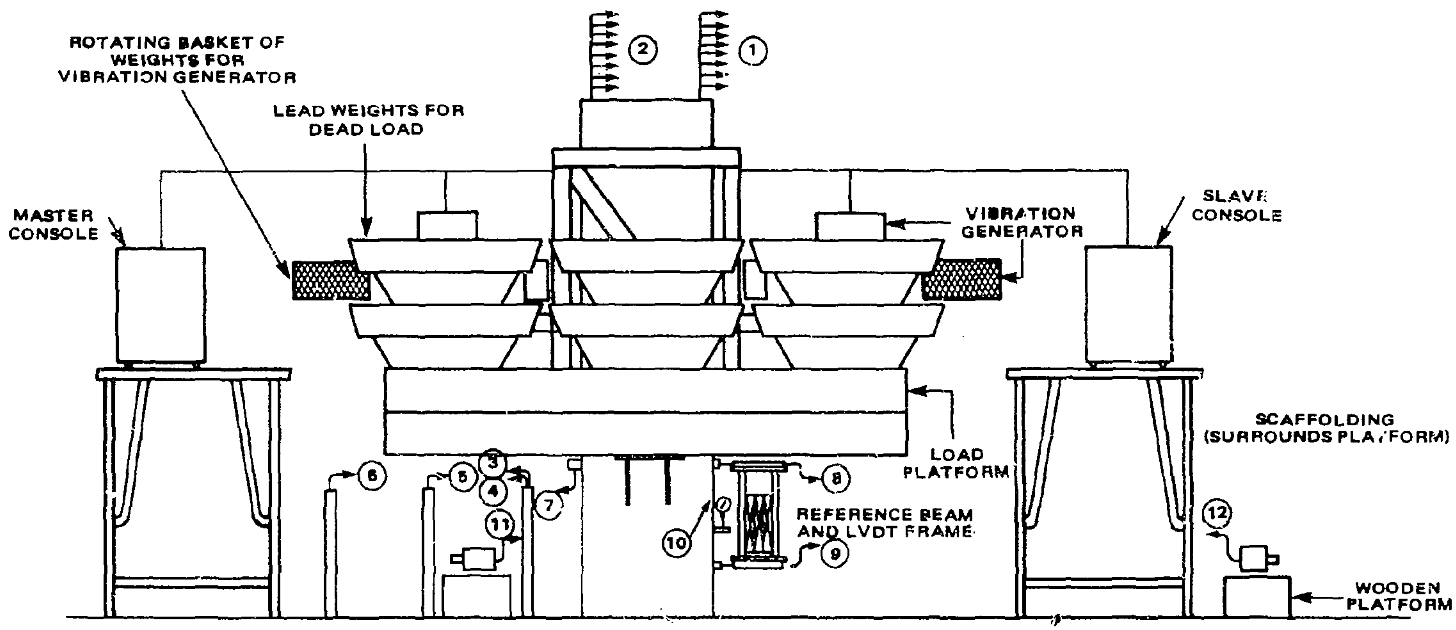
4.6 Seismometers

Two Ranger Model SS-1 Seismometers were used to monitor free-field vibrations around the test pile. For most of the tests, the seismometers were placed on wooden platforms, which were located at fixed positions throughout a given test sequence. For the last test, in which the pile was vibrated at resonance for an extended period of time, one seismometer was moved to various locations within one quadrant of the ground surface. This procedure was performed to map the ground vibrations around the vibrating pile in the radial and tangential directions.

4.7 Data Acquisition Systems

The diagram presented in Fig. 4.7 illustrates the general configuration of the data acquisition systems used for the vibration tests. More comprehensive charts are given in Appendix C.

Checked by ________ / ________ Drawn by ________ / ________ Checked by ________ / ________ Approved by ________ / ________



1in = 25.4 mm
1ft = 0.3048 m

INSTRUMENTATION SUMMARY

1	STRAIN TUBE 1 } CONNECTED TO FORM 8 FULL BRIDGES	7	3 AXIS ACCELEROMETER
2	STRAIN TUBE 2 }	8	LVDT, ±3" STROKE, 36" ABOVE GROUND
3	PIEZOMETER, 8' DEEP, 11" FROM PILE	9	LVDT, ±3" STROKE, 12" ABOVE GROUND
4	PIEZOMETER, 3' DEEP, 11" FROM PILE	10	DIAL GAUGE, VERTICAL MOVEMENT
5	PIEZOMETER, 3' DEEP, 23.5" FROM PILE	11	RANGER SEISMOMETER
6	PIEZOMETER, 3' DEEP, 59" FROM PILE	12	RANGER SEISMOMETER

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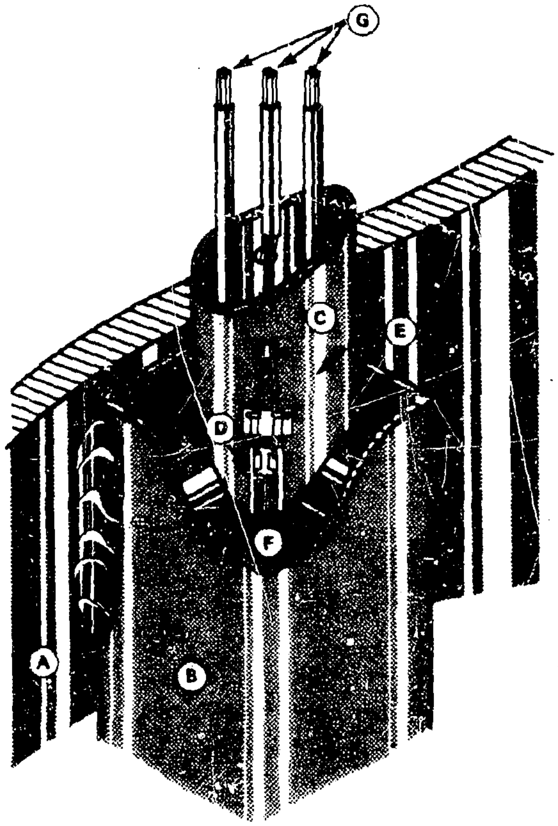
NSF PILE TEST

TEST CONFIGURATION, SIDE VIEW

8-81

FIGURE 4.1

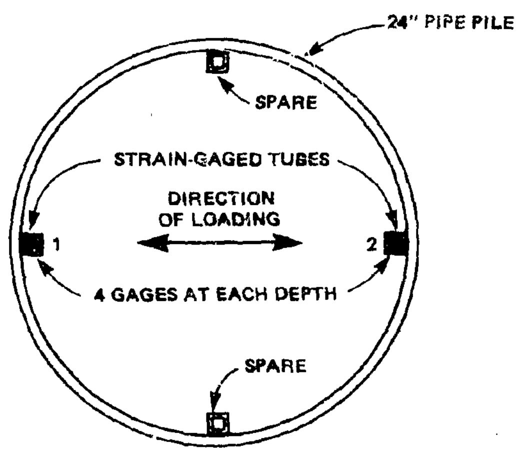
23A



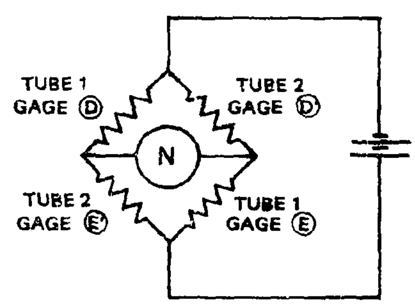
CUTAWAY VIEW OF ONE STRAIN GAGE TUBE

PROCEDURE FOR INSTRUMENTING NSF PILE

1. WELD 1 1/2" SQUARE STEEL TUBES (B) TO INSIDE OF 24" DIAMETER STEEL PILE (A)
2. PREFABRICATE 1" DIAMETER STEEL TUBE (C) BY EPOXYING 8 PAIRS STRAIN GAGES (D) AND (E)
3. POT 2 STRAIN-GAGED TUBES INTO 2 OPPOSITE SQUARE TUBES WITH EPOXY COMPOUND (F)
4. CONNECT EACH OF 8 STRAIN GAGE CABLES (G) (3 SHOWN) IN BRIDGE CONFIGURATION SHOWN BELOW



CROSS SECTION OF INSTRUMENTED PILE



CONFIGURATION OF STRAIN GAGE BRIDGE
(1 OF 8 IN PILE)

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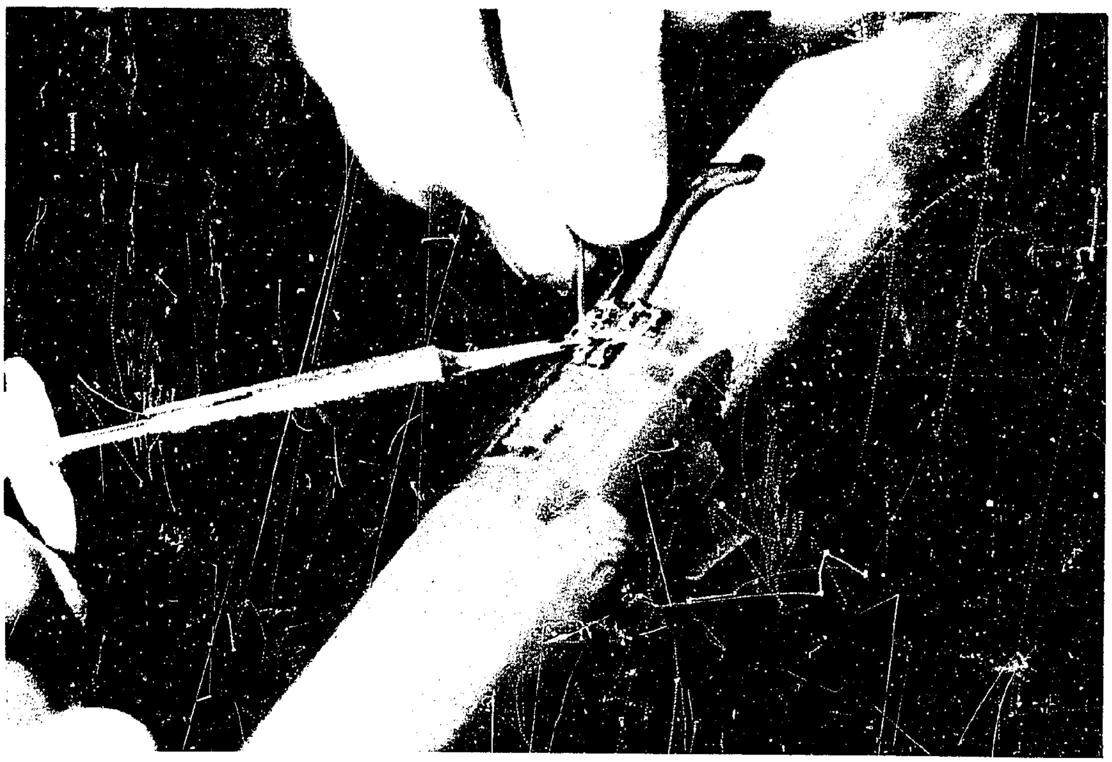
STRAIN GAGE CONFIGURATION

8-81 FIGURE 4.2

Compiled by ____ Drawn by ____ Checked by ____ Approved by ____



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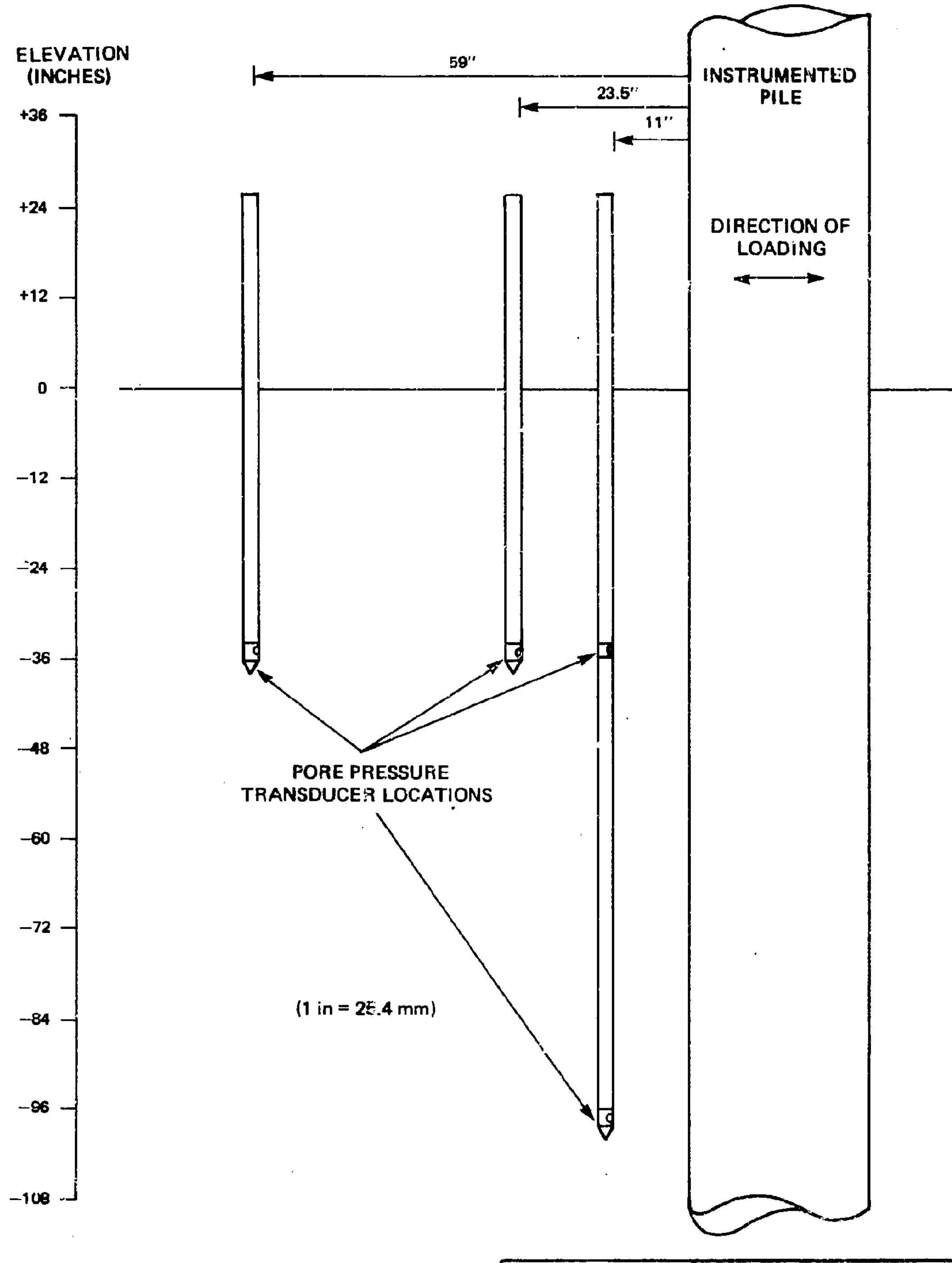
STRAIN GAGE ON
1" DIAMETER STEEL TUBE

8-81

FIGURE 4.3

230



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NSF PILE TEST

PIEZOMETER LOCATIONS

9-81

FIGURE 4.4

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INSTALLATION OF PIEZOMETERS

8-81

FIGURE 4.5

23F

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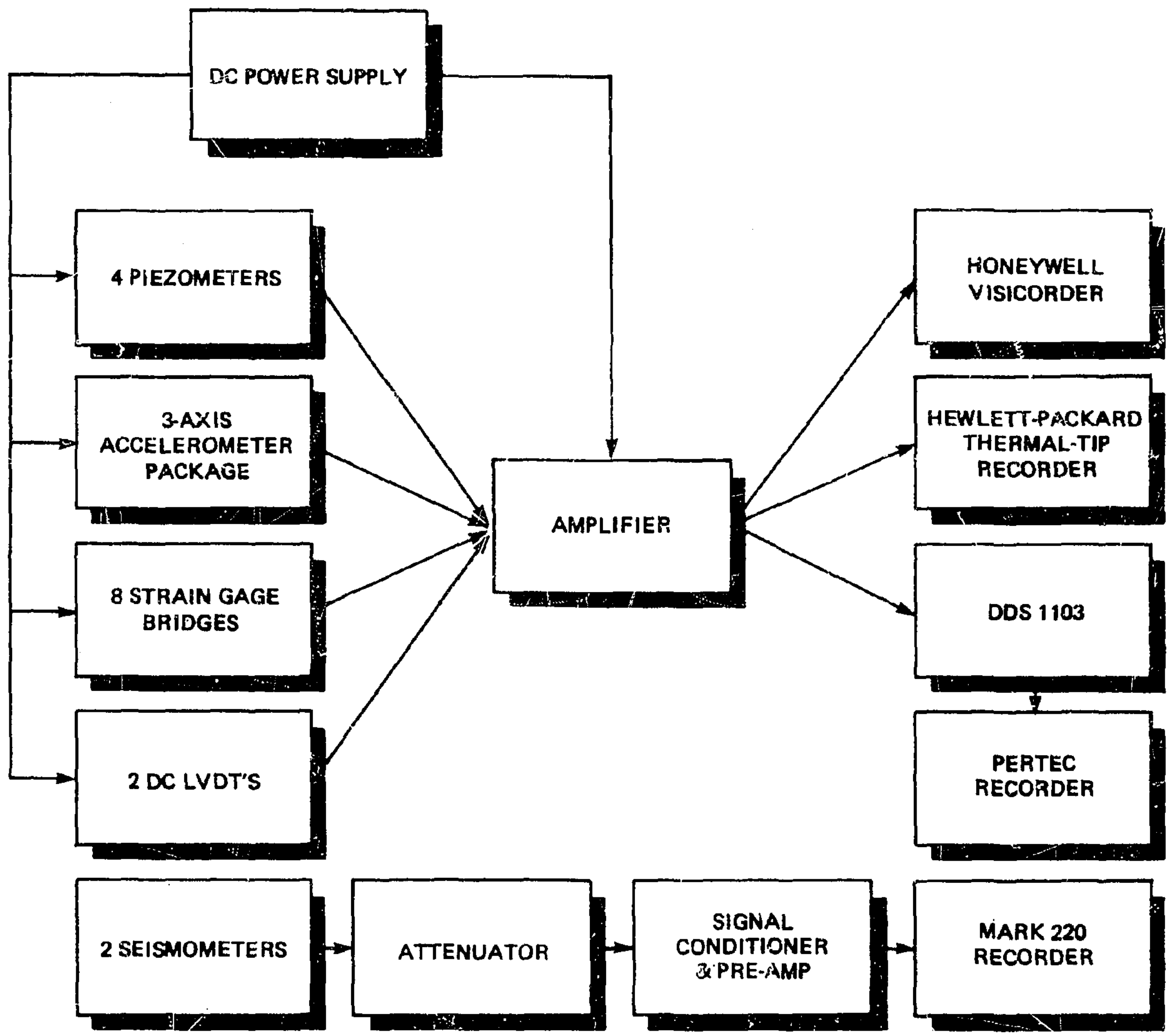
NSF PILE TEST

ACCELEROMETER PACKAGE
LOCATED ON PILE, 2' (0.61 M)
ABOVE GROUND SURFACE

8-81

FIGURE 4.6



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NSF PILE TEST

DATA ACQUISITION SYSTEMS

8-81

FIGURE 4.7

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

5. PRE-TEST ANALYTICAL STUDIES

5.1 Introduction

The primary purposes for the pre-test analytical study were:

1. To calculate the ultimate axial pile capacity which controlled the required dead load on the pile (representative of a typical structural load);

2. To understand the general behavior of the proposed pile under dynamic lateral loads (for example, natural frequencies of the soil-pile system); and

3. To estimate displacement and maximum moment profiles resulting from various levels of loads. This information was used to guide the instrumentation setup, including total number and locations of strain gages required.

Since the pre-test analytical studies were performed to guide the detailed instrumentation design of the test-pile setup, several assumptions and limited parametric variations were made during the analyses.

On the basis of preliminary field exploration program results, the average friction angle of the sandy soils at the site was estimated to be approximately 30°. The test pile used in the analysis was two feet (610 mm) in diameter and had a wall thickness of 1/2-in. (12.7 mm).

5.2 Static Response Evaluation

Both axial and lateral behavior of the test pile were investigated here.

Axial Capacity. The ultimate axial capacity of the test pile was estimated using the procedure recommended by API [2]. By assuming the pile was embedded 30 ft. (9.1 m) below the surface, the effective unit weight of the sand was 60 lb/ft^3 (0.80 Mg/m^3), and that the coefficient of lateral earth pressure was 0.8, the ultimate capacities were calculated to be 72 tons and 98 tons (641 and 872 kN) for friction angles of 28^o and 32^o, respectively. If a factor safety of 3 was assumed, a dead weight of 24 to 33 tons (214 to 294 kN) was required to simulate a typical building load on the pile.

Lateral Behavior of the Pile Under Pseudo-Static Lateral Load. In order to gain information regarding moment and displacement distribution along the test pile under lateral loads, analyses using the computer program BMCOL [5] were carried out with lateral load-resistance (p-y) curves defined using the API recommended procedure (Reese et al., 1974). This program utilizes a mechanical model of the pile-soil system. Soil resistance is represented by nonlinear springs. This is a widely accepted procedure for analyses of laterally loaded piles. For a pile embedded 30 feet into the soil and loaded at a point 5 ft (1.5 m) above ground surface, results of the BMCOL analyses are shown in Figures 5.1 and 5.2, giving displacement and moment distribution along the piles. In order to study the potential influence of the local soil liquefaction on pile response, series of analyses were performed assuming that the uppermost 5 ft (1.5m) of soil had liquefied as a result of vibration. Results of these analyses are shown in Figures 5.3 and 5.4.

A comparison of results between liquefied and non-liquefied studies indicate that the primary effects of liquefaction of the upper soil is to increase the depth of maximum moment, as would be expected. This information was utilized in the instrumentation phase of the study to ensure optimal location of the strain gages.

It should be noted in Figs. 5.1 through 5.4 that the two upper boundary conditions consisted of controlled displacements of 0.5 and 2.0 in. (12.7 and 50.8 mm) respectively. The tip of the pile was assumed fixed. In Figs. 5.1 and 5.2 no liquefaction is assumed to have occurred, and in Figs. 5.3 and 5.4 the soil is taken to have liquefied to a depth of 5 ft (1.5m) below the original ground surface. In consequence, the loads required to produce the 0.5 and 2.0 in. deflections in Fig. 5.4 are smaller than the loads required for the case of Fig. 5.2. The smaller moments indicated in Fig. 5.4 reflect this. These calculations were performed for the purpose of identifying the depths at which the maximum moments occurred so that the strain gages could be located appropriately.

5.3 Dynamic Response Evaluation

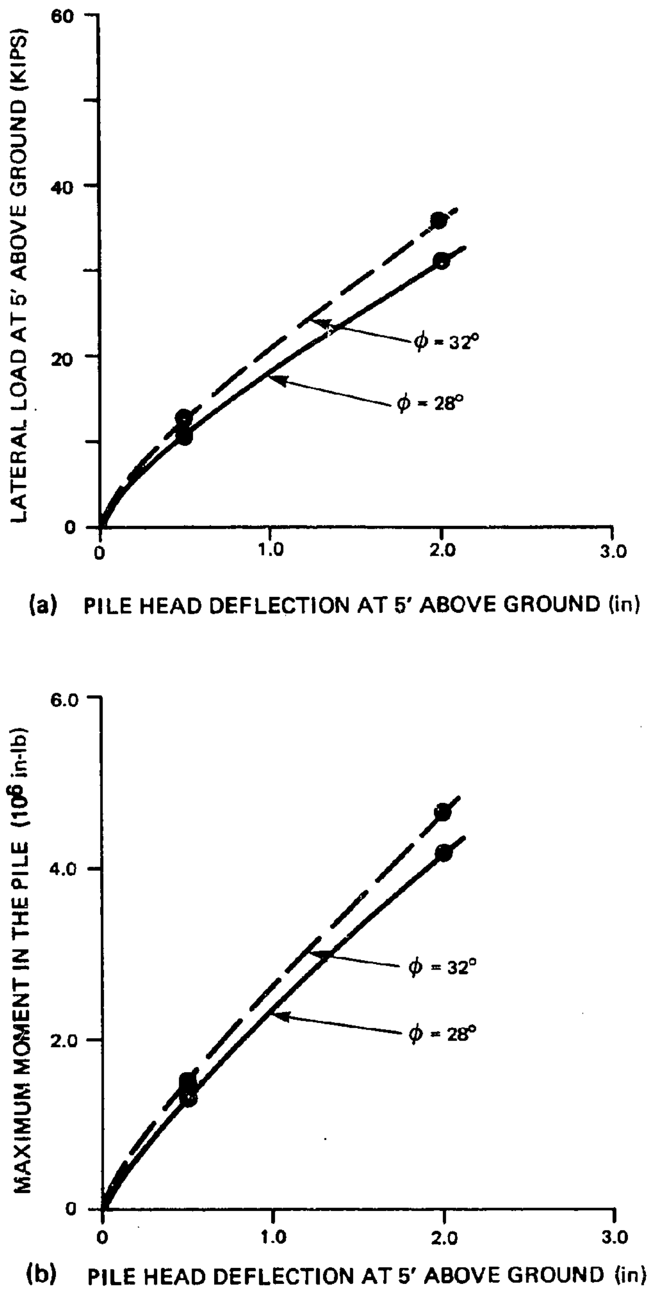
SPASM Analyses. The effects of vibration on the dynamic lateral behavior of the test pile were investigated using the computer program SPASM [6]. This program is a dynamic version of the conventional BMCOL program. By applying various harmonic force and displacement boundary conditions at the top of the pile, the dynamic response of a soil-pile system can be obtained. As shown in Table 5.1, for a soil-pile system with 50 tons (445 kN) of dead weight resting on the pile (5 ft (1.5m) above the ground surface), the maximum vibration-induced displacement is about 0.1 to 0.26 in (2.5 to 6.6 mm). This potential displacement provided necessary information regarding the specification of the displacement measuring devices.

Natural Frequency Estimation. One important aspect in the evaluation of a soil-pile system subjected to dynamic loads is the estimation of the natural frequency of the system. Several procedures with varying degrees of sophistication are available in the literature. In general, the complexity of the required input increases as the degree of sophistication increases. In view of the available information and the approximate nature of the pretest analytical study, only a simplified procedure was selected in addition to the SPASM analyses. By assuming the soil-pile system could be represented by a single-degree-of-freedom system, (i.e. 50 ton mass resting on the proposed pile with different lengths of cantilever beam approximated), the first natural frequency of the system was estimated to be between 1.68 and 2.5 Hz. These values indicated that with the operating range of frequency of the Caltech shaking machines (0 - 8.5 Hz), resonant frequency of the soil-pile system in the lowest mode was obtainable. The actual measured first natural frequency lay between the estimated range.

TABLE 5.1

SUMMARY OF SPASM ANALYSIS

Harmonic Lateral Loading at 5 ft above ground (lb)	Forcing Frequencies (Hz)	Maximum Displacement at Mudline (in)	Remarks
800	1.0	0.012	Dead weight of 50 tons applied at top of the pile (5 ft above ground)
	1.67	0.018	
	2.5	0.07	
	3.125	0.028	
1500	2.5	0.104	
	3.125	0.05	
5000	2.5	0.24	
	3.125	0.13	
5000	2.5	0.263	Dead weight of 36 ton applied
	3.125	0.205	



1ft = 0.305 m
1in = 25.4 mm
1kip = 4.45 kN
10^6 in-lb = 113 kN-m

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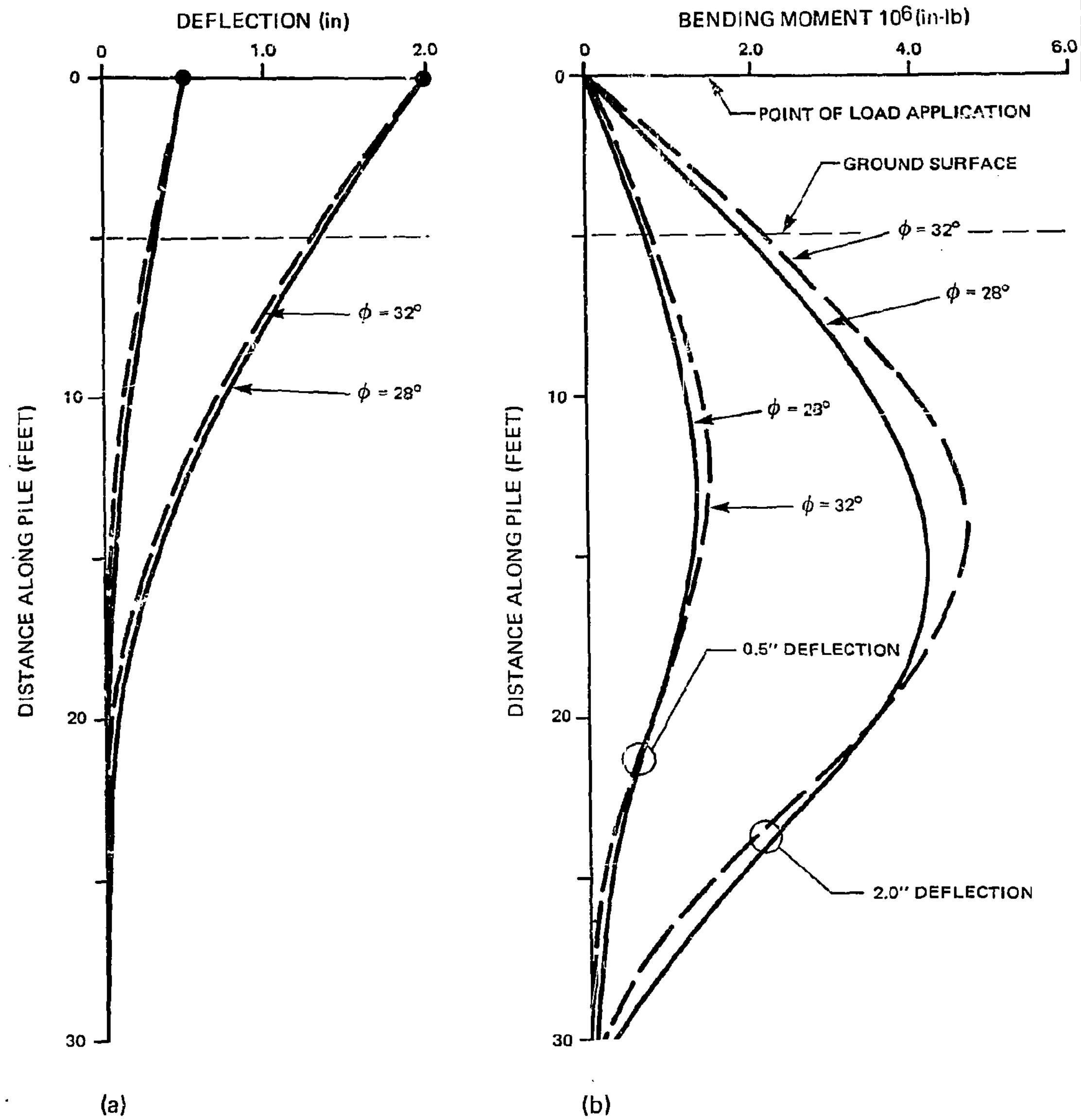
SUMMARY PLOT FOR
FREE-HEAD PILE RESPONSE
NO LIQUEFACTION

8-81

FIGURE 5.1

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

28C



1 ft = 0.305 m
1 in = 25.4 mm
10^6 in-lb = 113 kN-m

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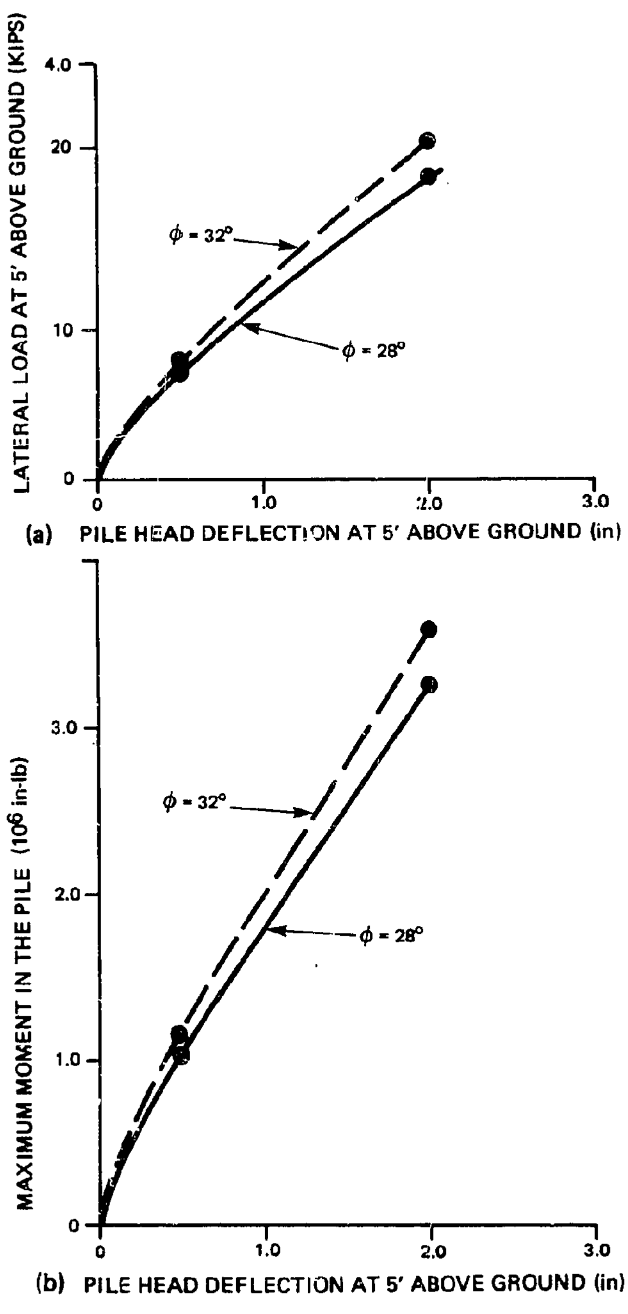
DISPLACEMENT AND MOMENT PROFILES
NO LIQUEFACTION

8-81

FIGURE 5.2

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______



1ft = 0.305 m
1in = 25.4 mm
1 kip = 4.45 kN
10^6 in-lb = 113 kN-m

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NSF PILE TEST

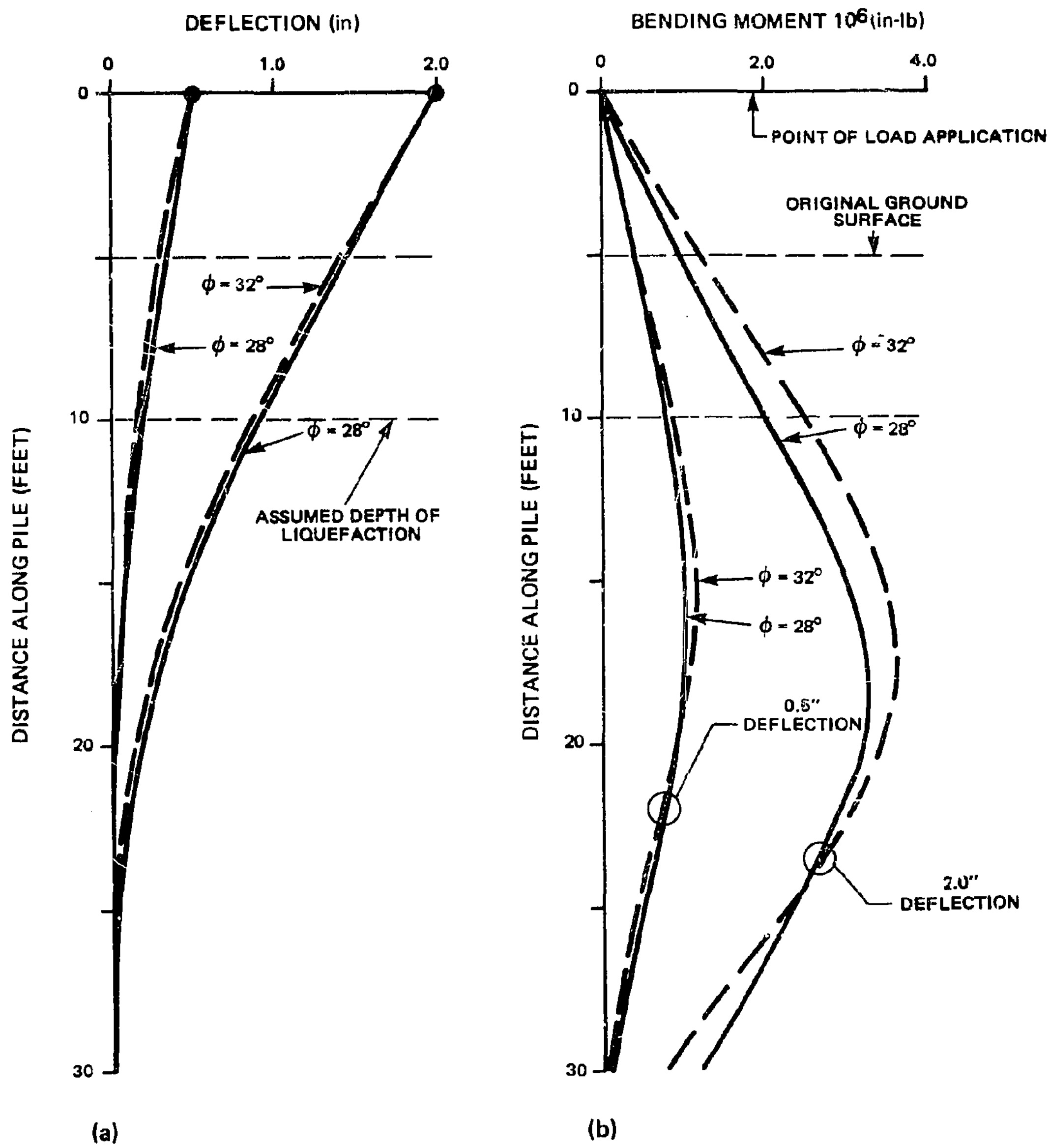
SUMMARY PLOT FOR
FREE-HEAD PILE RESPONSE
5' LIQUEFACTION ASSUMED

8-81

FIGURE 5.3

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

28 E



1 ft = 0.305 m
1 in = 25.4 mm
10^6 in-lb = 113 kN-m

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NSF PILE TEST

DISPLACEMENT AND
MOMENT PROFILES
WITH 5' LIQUEFACTION ASSUMED

8-81

FIGURE 5.4

6. TEST PROGRAM AND RESULTS

6.1 Introduction

The principal information sought in the investigation was the soil-pile response and the changes in the response or liquefaction tendency of the soil to the lateral vibration of the pile. Consequently, different amounts of vibration force, from very low or calibration level to values representing essentially realistic seismic levels, were applied to the pile to examine the effects of vibration disturbance on the soil. Details of the sequences and individual tests are given in the following paragraphs.

6.2 Test Program

As far as possible, each test series began with a frequency sweep at a low level of force. Using one shaker only with empty baskets, the force level obtained was about 300 lb (1.3kN) at first mode resonance. This defined the pile-soil characteristics at small strains. Then more weights were put in the swinging baskets of both shaking machines, and the system brought up to and past, if possible, first-mode resonance. On a few test attempts, too much weight was put in the swinging baskets, and the relatively small drive motors of the shakers could not put out enough power to reach and pass the first resonance frequency. In effect, the system stalled at a stage where the pile and soil were absorbing all the power that the shaking machines could put out. It was necessary to stop the machines, reduce the weights in the baskets and try again, successively, until a full frequency sweep could be attained. As a consequence the peak force levels that could be reached to cover the entire range of frequencies accessible to the machine (zero to about 8.5 Hz) were disappointingly low [1500 lb (6.7 kN) at resonance]. On different days it was found that the resonance hump could be surpassed with differing numbers of weights in the baskets. This is probably due to variations in the stiffness of the soil surrounding the pile caused by vibration and compaction. On reaching the upper limit of frequency of the apparatus in any sweep, the rotation speed of the shakers was gradually reduced once more to zero. For each of the frequency sweeps, records of all the transducer outputs were obtained, except for intermittent malfunctioning of one of the pile strain gages. Two of the pore pressure transducers did not operate in the last test series, but fluctuating pore pressures were observed during the other tests. Each test series except the last was finished with a low force-level sweep again, to examine the change in soil behavior during the day's tests.

In one medium-level force test (Test no. 12), the pile was maintained at its resonance frequency for 15 to 20 minutes while a single Ranger seismometer was used to map out the velocity field in one quadrant around the pile to a distance of about 80 ft (23.4m) from the pile.

The last experiments performed were plucking tests of the pile. A rope was attached to the pile at about the line of action of the shaking machines´ force and was pulled with a four-wheel drive truck. In some of these tests the rope was cut with an axe; in others, the rope was pulled until it parted. In either case, the pile rang down from the initial deflection, giving values of fundamental frequency and damping at low strains. The rope force was about 700 lb (3.1 kN).

A photograph is shown in Fig. 6.1 of the pile during vibration at resonance. A slight conical depression can be noted in the sand adjacent to the pile, indicating that some compaction took place due to an increase in pore pressure and the subsequent consolidation.

6.3 General

Using the various gage factors and accounting for the different amplifiers, input voltages, and recorder sensitivities, the raw transducer outputs were converted to physical units, as described in Appendix D. The pertinent data for each forced vibration test is summarized in Table 6.1, and include the frequency range and force level ranges of shaking, damping ratio, and natural frequency, and the peak displacement, acceleration, and moment in the pile at the resonance frequency. Various low force level frequency sweeps (tests 1, 2, 6, 7, and 10) indicate a natural frequency of between 2.27 and 2.87 Hz, depending on test history and probably stress-strain history of the sand adjacent to the pile immediately prior to each test. Higher force level shaking test (numbers 5, 9, and 12) indicate a natural frequency between 1.72 and 2.87 Hz. Typical peak amplitudes of displacement at resonance at 1.0 ft (305 mm) above ground surface range from 0.025 in. (0.64 mm) in test no. 1 at 364 lbf (1.62 kN) to greater than 0.43 in. (10.9 mm) at 1762 lbf (7.84 kN) peak force in test no. 2. Typical damping ratios ranged from 1.5 to 7.0% of critical based on the half bandwidth method. Computed peak pile accelerations at resonance at 1.0 ft (305 mm) above ground surface range from 0.02 g for test no. 1 to 0.17 g in test no. 12, while peak moments in the pile at resonance ranged up to 5.8 x 10^5 lbf-in. (67 kN-m).

6.4 Frequency Sweeps

Fig. 6.2 shows the typical response curve for a low force level frequency sweep, test no. 6, while Fig. 6.3 shows the response for a fairly high force level sweep, test no. 9. Figs. 6.4 and 6.5 show the typical strain (curvature and moment), slope, and displacement of the pile with depth for test no. 6 at resonance (2.27 Hz), and at 8.62 Hz, respectively. Note the onset of the second natural mode of vibration in Fig. 6.5. However, owing to limitations of the shaking machines in obtaining higher frequencies, the actual second resonant peak was not reached. The mode shape for a higher force level shaking test (no. 9) at resonance is plotted in Fig. 6.6. The maximum moment induced in the pile at resonance for this test is located about 14 ft (4.27 m) from the top of the pile, i.e. 6 ft (1.83 m) below ground surface, indicating that the pile is fairly flexible relative to the soil system.

6.5 Ringdown Tests

Table 6.2 in summarizes the four ringdown tests conducted by pulling the pile laterally with a truck and nylon rope, then cutting the rope to allow the pile to "snap back." Typical peak displacements range up to 0.007 in (0.18 mm) at the first peak after release, well below the amplitudes for the lower force level forced vibration tests. Natural frequencies of vibration during ringdown ranged from 4.1 to 4.3 Hz, with a damping ratio obtained from the logarithmic decrement method of between 1.4 and 2.8% of critical. Fig. 6.7 shows the typical pile shape just before release and for the first peak after release in test no. 16. Peak force in the rope prior to release is estimated to be about 725 lbf (3.22 kN). The peak moment in the pile prior to shaking was 4.41×10^4 lb-in (5.0 kN-m), again well below that induced during a typical low level frequency sweep. The displacements at 1.0 ft (305 mm) above ground surface during ringdown are shown in Fig. 6.8.

6.6 Velocity Profile

The velocity profile of the ground surrounding the pile at resonance was obtained from the Ranger Seismometer during test no. 12. The typical velocity during each cycle of (harmonic) shaking was decidedly not harmonic. At locations fairly close to the pile, a waveform similar to that in Fig. 6.11(a) was seen, with a continuous change in shape with increasing distance, as in 6.11(b). For the fairly close locations, the non-sinusoidal waveform is probably due to the formation of a cavity behind the pile during shaking, and subsequent impact of the pile with this cavity wall during reversal of the pile displacement. At greater distances, a much more complex form is present, as seen in Fig. 6.11(c), possibly due to reflections from deeper strata. Fig. 6.12 shows the velocity as a function of location. It is hoped that such data may eventually be used with some simplifying assumptions to estimate the radiation damping in the soil system around the pile.

6.7 Pore Pressure Fluctuation

The electrical signals from the piezometers were recorded at a low sensitivity due to the large zero shift of the pore pressure transducers, as explained earlier. Consequently, long period pore pressure changes during vibration loading were barely detectable. As shown in the reproduction of actual records in Fig. 6.13, oscillations of pore pressures were noted but no strong tendency for increases in pressure was observed during vibration. However, a rapid increase of 10 in. (254 mm) of water head inside one of the lucite standpipe piezometers was observed during 30 cycles of the last series of medium level vibrations. The piezometer tip was located 36 in. (914 mm) below the ground surface. This 30 percent increase in pore pressure corresponds to a pore pressure ratio of about 0.4 (ratio of excess pore pressure to vertical effective stress). Other phenomena such as small sand boils, "bleeding" of water at the ground surface, and consolidation of the sand around the pile (see Figure 6.1) were observed during the vibration. These observations indicate that a progressive buildup in pore pressure and possibly partial liquefaction of the near surface sand had occurred. A conceptual illustration of the above described phenomena during and immediately after the vibration test is shown in Fig. 6.14.

6.8 Discussion of Results

The observed changes in natural frequency from test to test may be attributed to two factors: level of strain and vibration-induced increases in pore pressure. Although it is difficult to accurately define a level of shear strain in the soil for a given test at resonance, attempts were made to estimate the level of strain in the upper 5 ft (1.5 m) of soil. The natural frequency (f_n) is plotted in Fig. 6.9 against the logarithm of the amplitude of displacement (A_n) at 12 in. (305 mm) above ground surface for all tests. Also included are estimates of corresponding shear strains in the soil. Forced vibration tests where resonance was not actually observed are plotted with arrows indicating the probable location of the actual f_n - A_n data. The overall trend of a decreasing natural frequency with increasing amplitude of vibration is expected. An increasing level of shaking induces a more nonlinear, softening response from the soil and a buildup in excess pore pressure. Both of these resulted in a decreased stiffness and natural frequency. The results shown in Fig. 6.9 indicate that a stiffening of the soil-pile system probably occurred between tests performed on 1 July and 14 July. This stiffening strongly suggests that considerable excess pore pressures were generated during the 1 July tests, the dissipation of which resulted in densification of the soil surrounding the pile.

Similarly, the damping ratios obtained from all shaking and ringdown tests are plotted against the amplitude of displacement in Fig. 6.10, and indicate an increasing damping ratio with increasing amplitude. This is also anticipated, as the higher displacements are indicative of a more nonlinear softening and hence more hysteretic response.

In conclusion, the test results indicate that partial liquefaction did indeed occur around the head of the pile, caused by a buildup of vibration-induced excess pore pressure. Comparison of the results with pre-test analytical studies suggest that predictions of natural frequency and pile response agree reasonably with the field case. Since detailed analyses of the test results are not within the scope of the proposed work, much additional work is recommended. Efforts should be made in the near future to further analyze the test results and to recommend modifications to current procedures of evaluating dynamic soil-pile interaction.

TABLE 6.1

SUMMARY OF FREQUENCY SWEEP PILE SHAKING TESTS

TEST NUMBER	TEST DATE (1981)	RUN NUMBER	FREQUENCY RANGE (HZ)	FORCE RANGE (LBF) [KN]	FREQUENCY FN (HZ)	DAMPING RATIO β (%)	AT RESONANCE: PEAK DISPLACEMENT AN FN AT 1' ABOVE GROUND SURFACE (IN) [MM]	AN/FN2 (IN/HZ2) [MM/HZ2]	COMPUTED PEAK ACCELERATION AT 1' ABOVE GROUND SURFACE	PEAK HORIZONTAL FORCE (LBF) [N]	PEAK MOMENT IN PILE (LBF-IN) [KN-M]	COMMENTS
1	JUNE 26	1	0-6.75	0-2414 [0-10.7]	2.62	3.8	0.025 [0.63]	3.64×10^{-3} [9.18×10^{-2}]	0.018	364 [1.62]	1.79×10^{5} [20.2]	LOW LEVEL FREQUENCY SWEEP; VIRGIN CONDITIONS
2	JUNE 30	1	0-8.18	0-3546 [0-15.8]	2.80	3.0	0.034 [0.84]	4.34×10^{-3} [1.07×10^{-1}]	0.027	416 [1.85]	2.46×10^{5} [27.7]	LOW LEVEL FREQUENCY SWEEP
3	JULY 1	1	0-1.24	0-1762 [0-7.8]	>1.24	NA	> 0.43 [10.9]	NA	0.068	1762 [7.8]	—	HIGH LEVEL SHAKING; RESPONSE NOT OBTAINED
4	JULY 1	2	0-1.50	0-1931 [0-8.6]	>1.50	NA	> 0.37 [9.40]	NA	0.085	1931 [8.6]	—	HIGH LEVEL SHAKING; RESPONSE NOT OBTAINED
5	JULY 1	3	0-3.86	0-4172 [0-18.6]	1.72	6.0	0.22 [5.59]	7.44×10^{-2} [1.89]	0.067	828 [3.7]	—	MEDIUM LEVEL FREQUENCY SWEEP
6	JULY 1	4	0-8.62	0-3938 [0-17.5]	2.27	3.0	0.045 [1.14]	8.73×10^{-3} [2.21×10^{-1}]	0.024	273 [1.2]	4.15×10^{5} [46.9]	LOW LEVEL FREQUENCY SWEEP
7	JULY 1	5	0-8.45	0-3784 [0-6.8]	2.50	7.0	0.045 [1.14]	7.20×10^{-3} [1.82×10^{-1}]	0.029	331 [1.5]	—	LOW LEVEL FREQUENCY SWEEP
8	JULY 1	6	0-1.85	0-1390 [0-6.2]	> 1.85	NA	> 0.283 [6.43]	NA	0.099	1390 [6.2]	—	MEDIUM LEVEL SHAKING; RESPONSE NOT OBTAINED
9	JULY 1	7	0-3.58	0-4709 [0-20.9]	2.01	5.5	0.259 [6.58]	6.41×10^{-2} [1.63]	0.107	1483 [6.6]	5.8×10^{5} [67]	MEDIUM LEVEL FREQUENCY SWEEP
10	JULY 14	1	0-8.43	0-3766 [0-16.8]	2.67	1.5	0.045 [1.15]	6.31×10^{-3} [1.62×10^{-2}]	0.033	377 [1.7]	—	LOW LEVEL FREQUENCY SWEEP
11	JULY 14	2	0-2 57	0-2427 [0-10.8]	> 2.57	NA	> 0.25 [6.32]	NA	0.169	2427 [10.8]	—	MEDIUM LEVEL SHAKING; RESPONSE NOT OBTAINED
12	JULY 14	3	0-2.9	0-2357 [0-10.5]	2.87	NA	0.207 [5.26]	2.51×10^{-2} [6.39×10^{-1}]	0.174	2309 [10.3]	—	MEDIUM LEVEL FREQUENCY SWEEP; VELOCITY PROFILE TAKEN

NA = NOT AVAILABLE

— = NOT COMPUTED

TABLE 6.2

SUMMARY OF RING-DOWN TESTS

TEST NUMBER	TEST DATE (1981)	RUN NUMBER	NATURAL FREQUENCY FN (AVG) [HZ]	DAMPING RATIO β (%)	PEAK DISPLACEMENT A 1' ABOVE GROUND SURFACE PRIOR TO RELEASE (LFB) [KN]	PEAK HORIZONTAL FORCE (LFB) [KN]	PEAK DISPLACEMENT A 1' ABOVE GROUND SURFACE AFTER RELEASE (IN) [MM]	PEAK COMPUTED ACCELERATION 1' ABOVE GROUND SURFACE (G)	PEAK MOMENT IN PILE (LBF-IN) [KN-M]	COMMENTS
13	JULY 14	4	4.06	2.5	0.0098 [0.249]	—	0.0072 [0.183]	0.012	—	LARGE RING-DOWN; ROPE SNAPPED AT TRUCK
14	JULY 14	5	4.3	2.0	—	—	0.0018 [0.047]	0.003	—	SMALL RING-DOWN ; ROPE CUT
15	JULY 14	6	4.3	1.4	—	—	0.0017 [0.043]	0.003	—	SMALL RING-DOWN; ROPE CUT
16	JULY 14	7	4.2	2.8	0.0080 [0.203]	725 [0.203]	0.0045 [0.114]	0.008	4.4 X 10^4 [5.0]	LARGE RING-DOWN; ROPE SNAPPED IN MIDDLE

— = NOT COMPUTED

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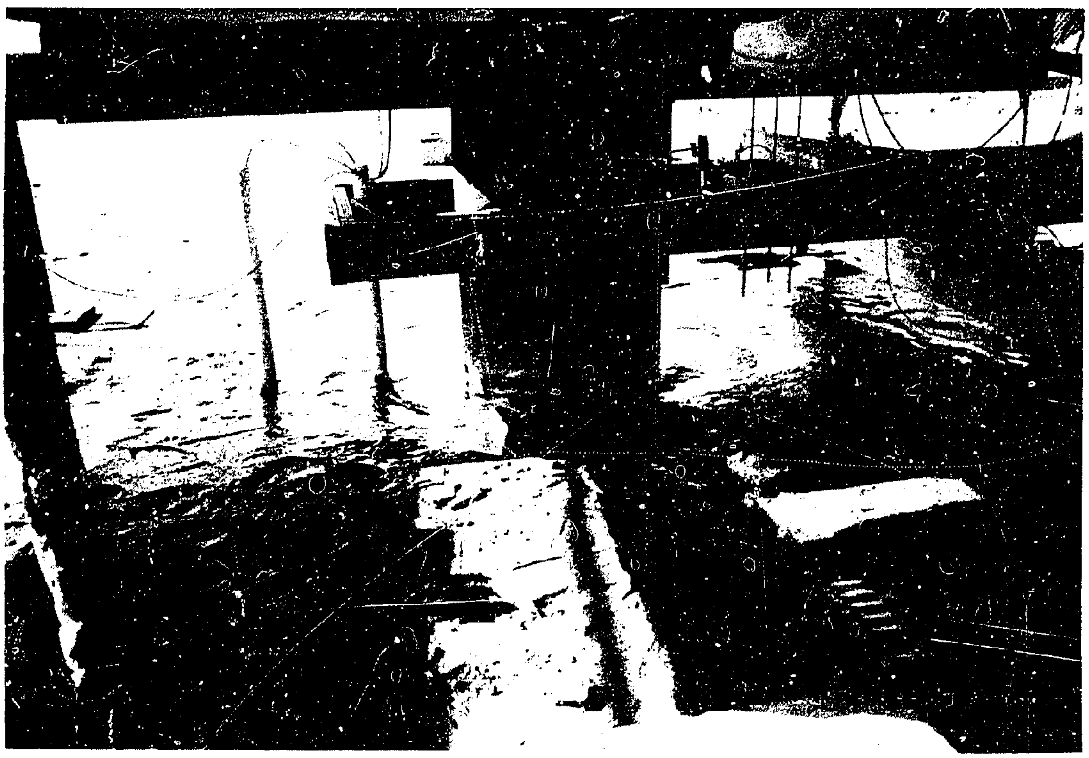
PROJECT NO.: 79-243

NSF PILE TEST

SUMMARY TESTS:
FREQUENCY SWEEPS &
RING-DOWN TESTS



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PROJECT NO.: 79-243

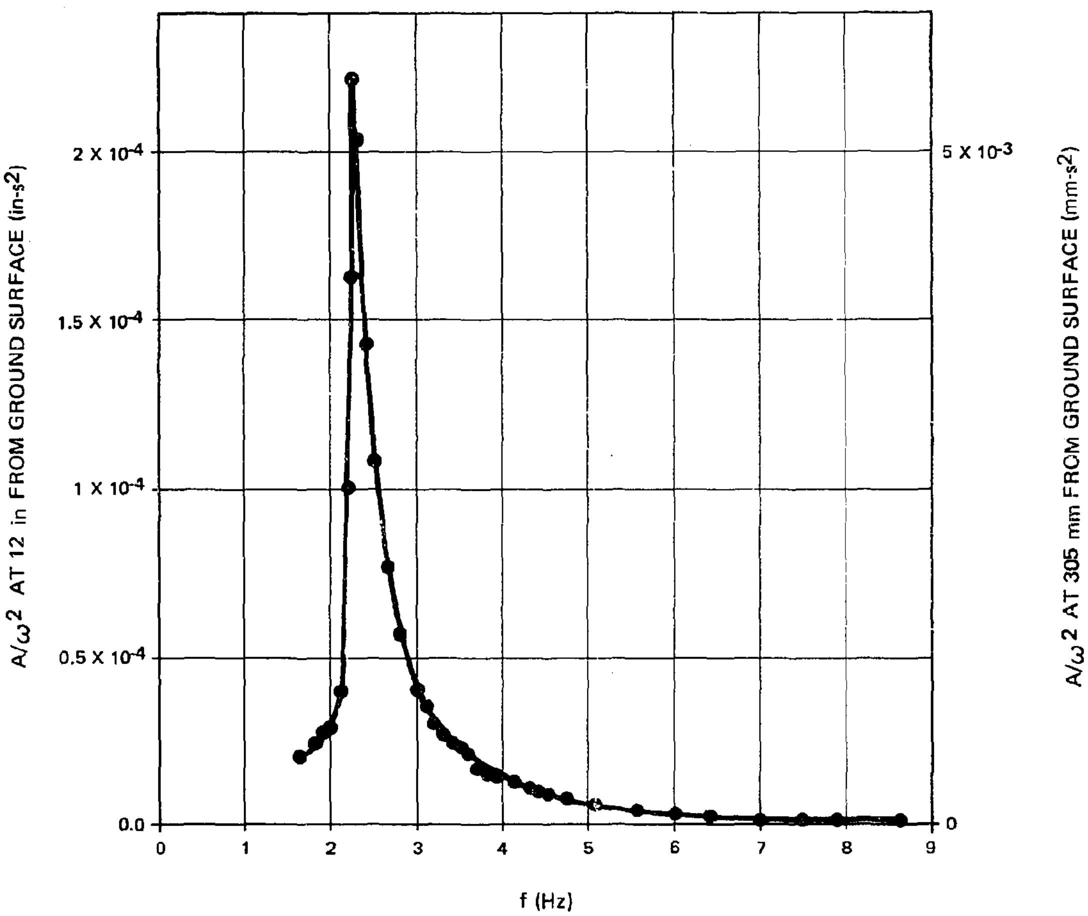
NSF PILE TEST

CONICAL DEPRESSION FORMING
AROUND PILE DURING VIBRATION
AT RESONANCE FREQUENCY

8-81

FIGURE 6.1

36C



f_1 = 2.27 Hz
β = 3%
PEAK LATERAL FORCE AT 1 Hz: 53 LBF (0.24 kN)

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PROJECT NO.: 79-243

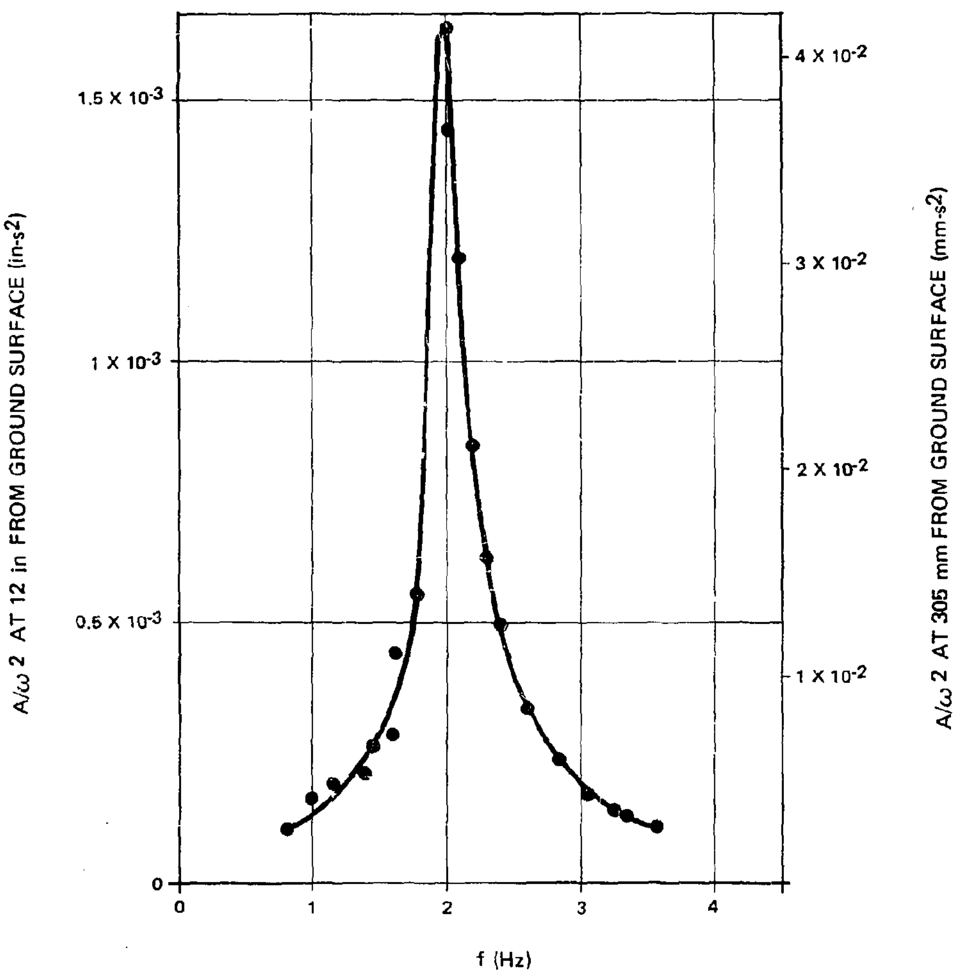
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 6
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81

FIGURE 6.2

Compiled by ______ / Drawn by ______ / Checked by ______ / Approved by ______



f_1 = 2.01 Hz

β = 5.5%

PEAK LATERAL FORCE AT 1 Hz: 367 LBF (1.63 kN)

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PROJECT NO.: 79-243

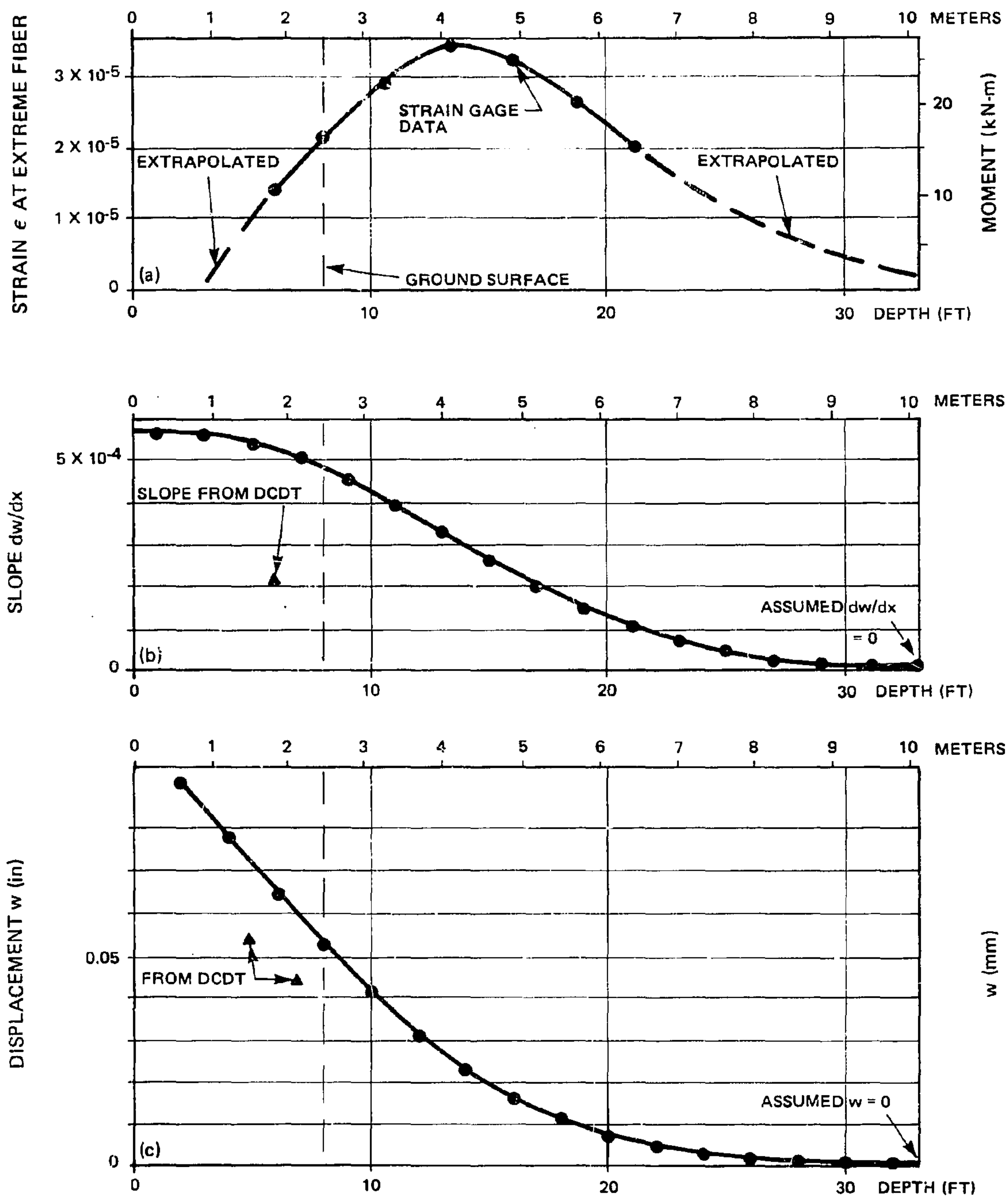
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 9
NORMALIZED TO 367 LBF (1.63 kN)
PEAK LATERAL FORCE

8-81

FIGURE 6.3

Compiled by ____ Drawn by ____ Checked by ____ Approved by ____



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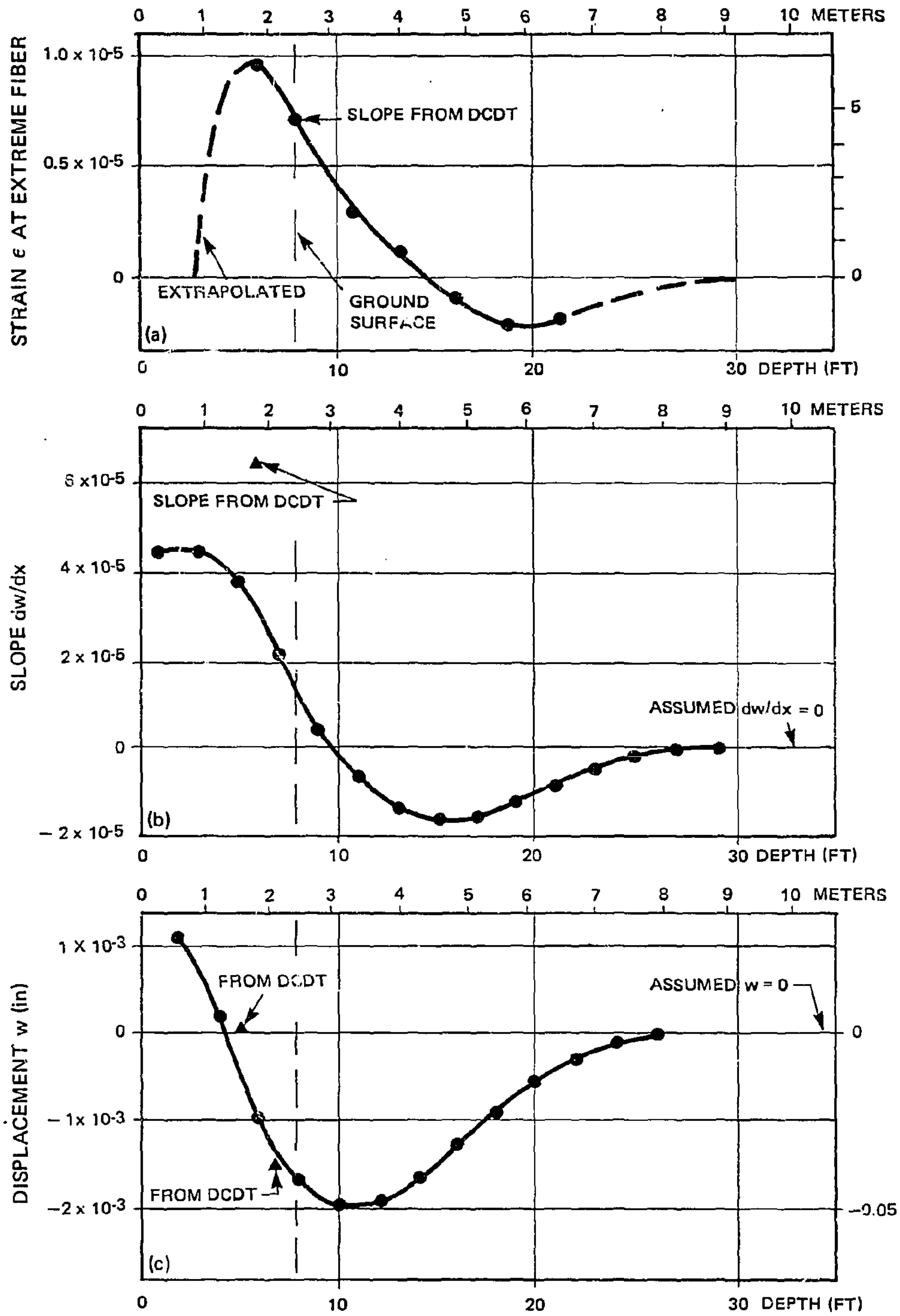
PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT PROFILES OF PILE FOR TEST NO. 6 AT RESONANCE, f_1 = 2.27 Hz

8-81

FIGURE 6.4

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

36 F



Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

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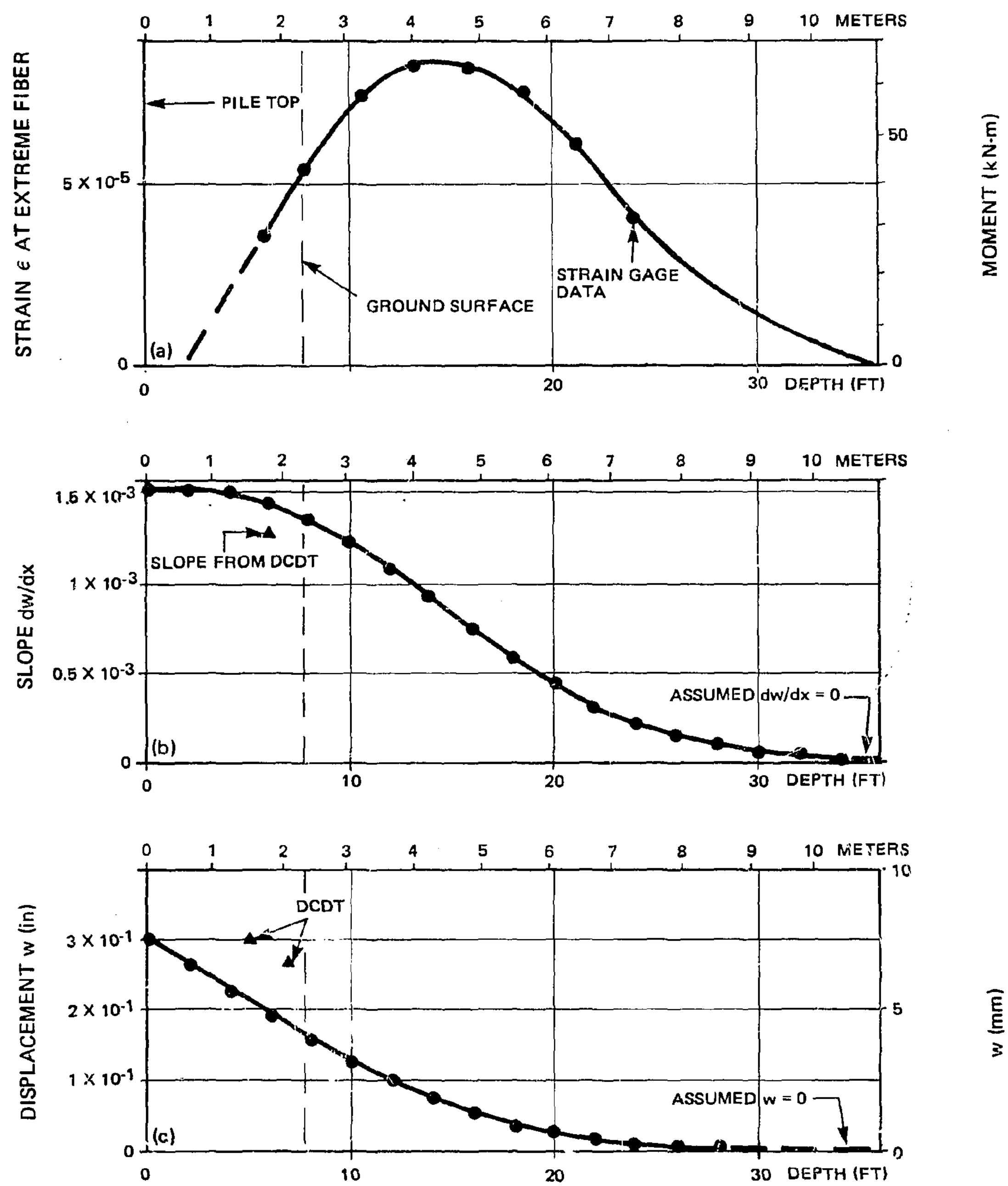
PROJECT NO.: 79-243

NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT PROFILES FOR TEST NO. 6 AT f = 8.62 Hz (NEAR 2ND MODE)

FIGURE 6.5



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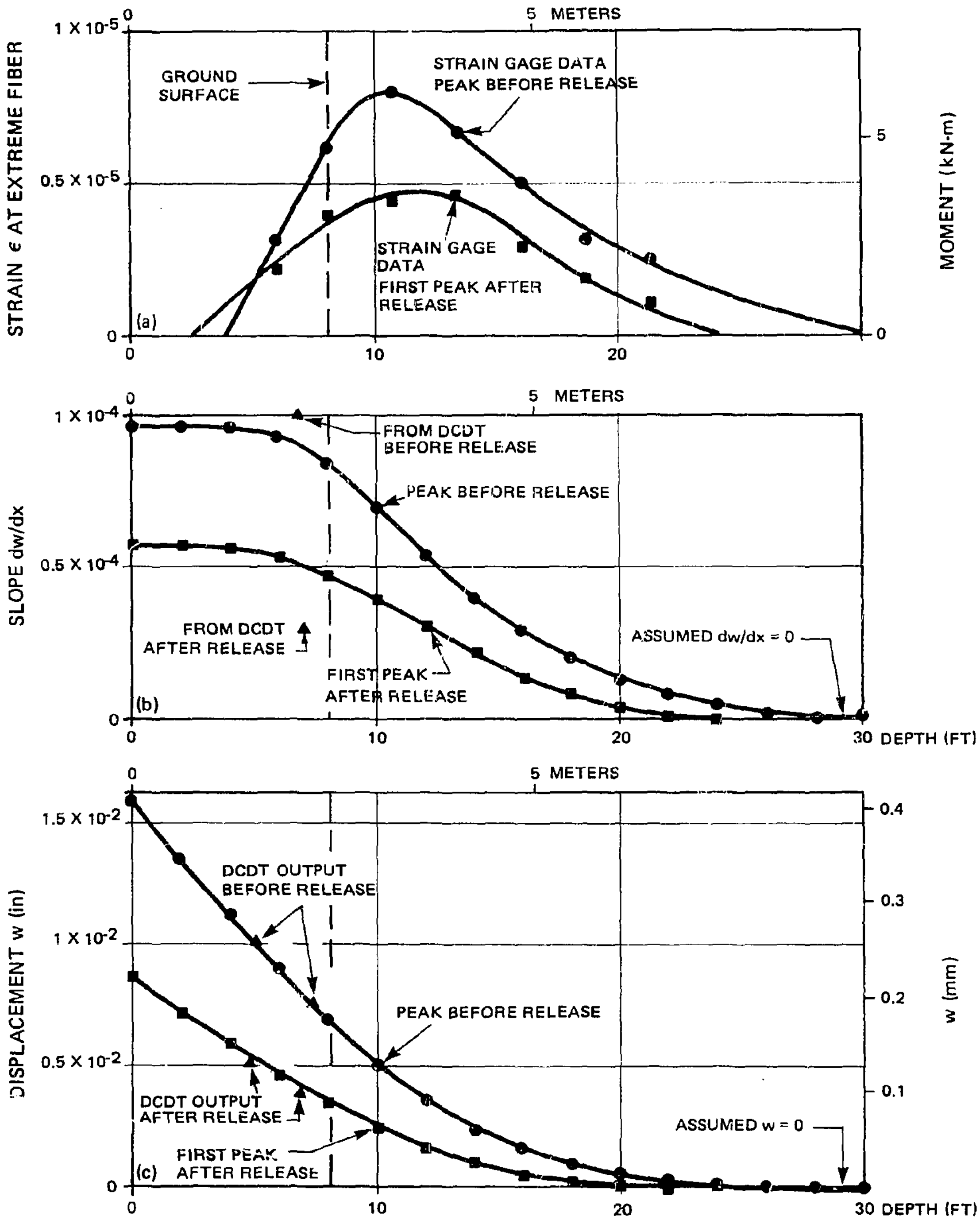
NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH FOR TEST NO. 9 DURING RESONANCE, f_1 = 2.01 Hz

8-81

FIGURE 6.6

Approved by
Checked by
Drawn by



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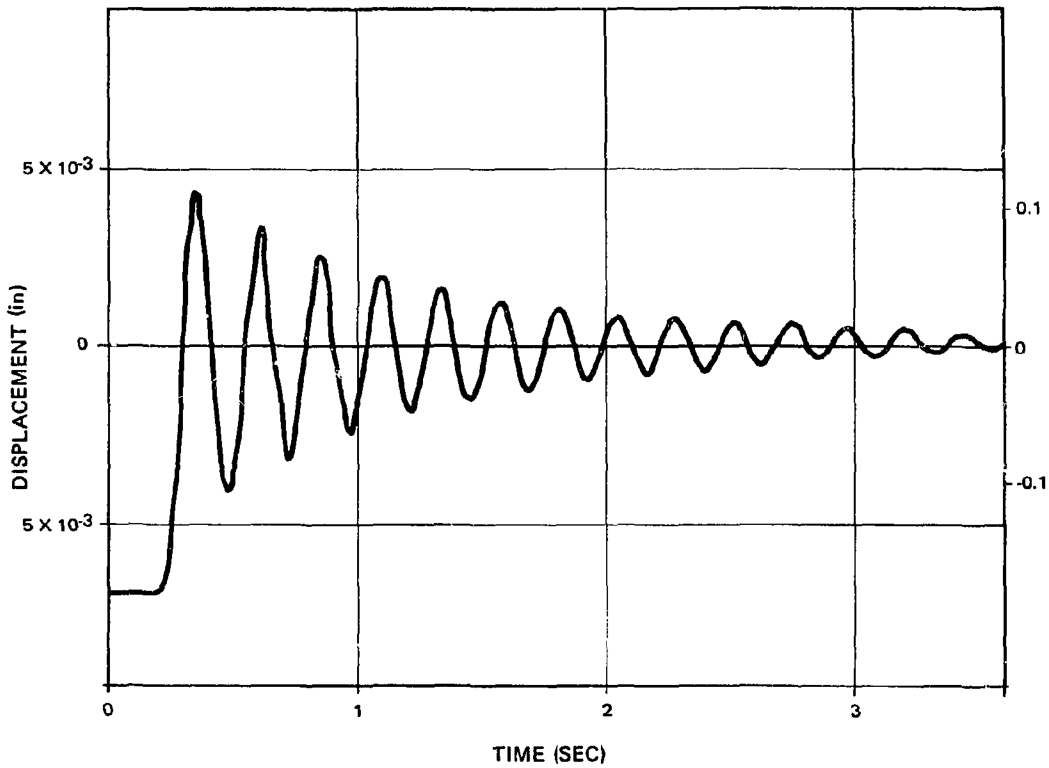
	PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH DURING RING-DOWN TEST NO. 16
8-81	FIGURE 6.7

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______



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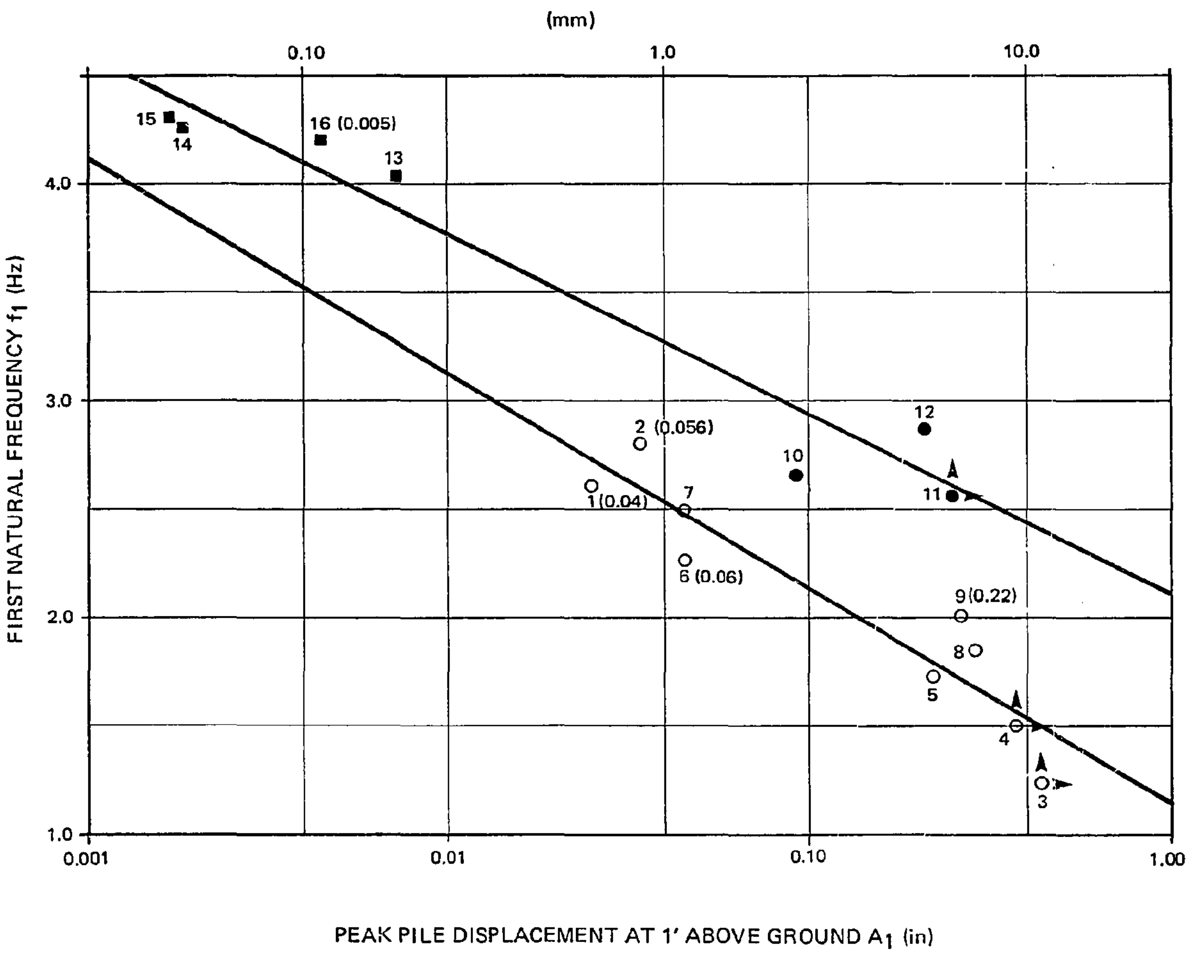
NSF PILE TEST

DISPLACEMENT-TIME RECORD OF PILE
DURING RING-DOWN TEST NO. 16
AT 1 FT (305 mm)
ABOVE GROUND SURFACE

8-81

FIGURE 6.8

Drawn by ______ Checked by ______ Approved by ______



OPEN = ON OR BEFORE JULY 1
SOLID = JULY 14
O = SHAKER TEST RESULT
■ = RING-DOWN TEST RESULT
A_1 = FIRST PEAK AFTER RELEASE
( ) = ESTIMATED MAXIMUM DYNAMIC SHEARING STRAIN (%)

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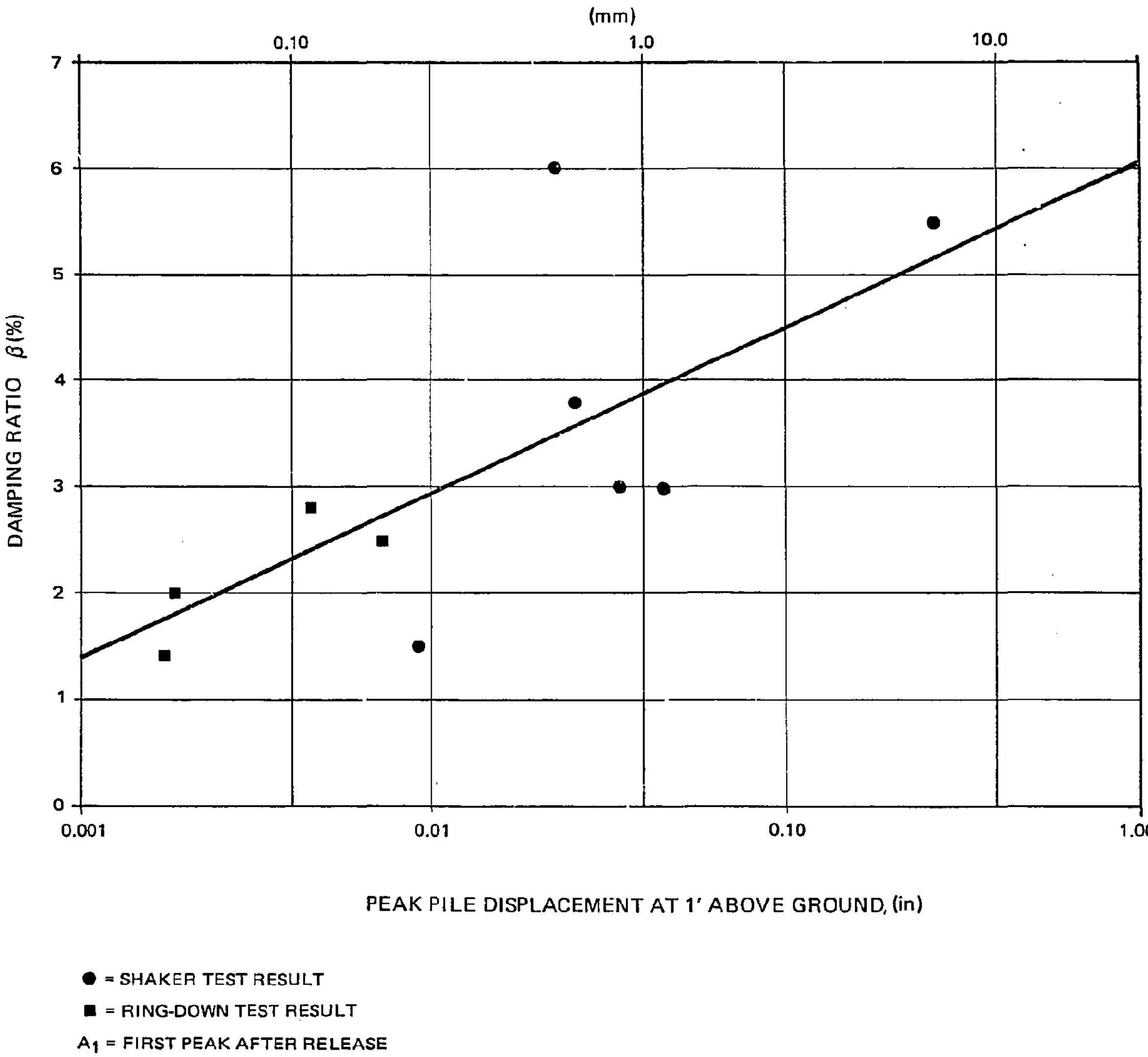
NSF PILE TEST

1ST NATURAL FREQUENCY AS A FUNCTION OF PEAK PILE DISPLACEMENT A_1 AT 1 FT (305 mm) ABOVE GROUND SURFACE AT RESONANCE

8-81

FIGURE 6.9

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______



Compiled by ______ / Drawn by ______ / Checked by ______ / Approved by ______ /

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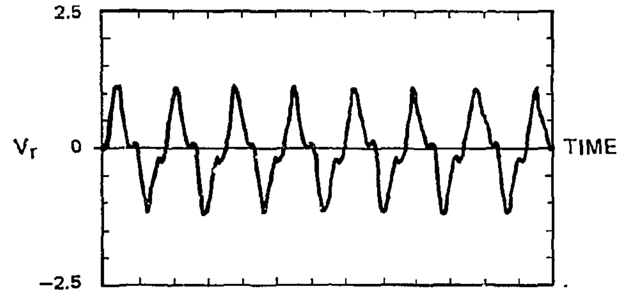
NSF PILE TEST

DAMPING RATIO β AS A FUNCTION OF PEAK PILE DISPLACEMENT A_1 AT 1 FT (305 mm) ABOVE GROUND SURFACE AT RESONANCE

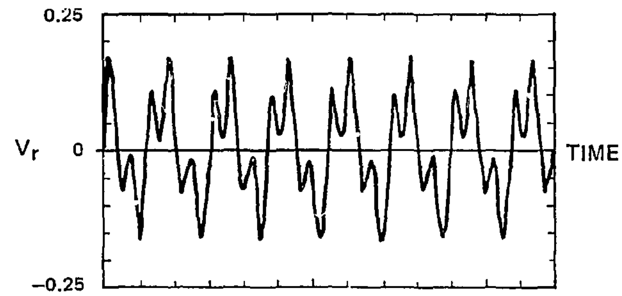
8-81

FIGURE 6.10

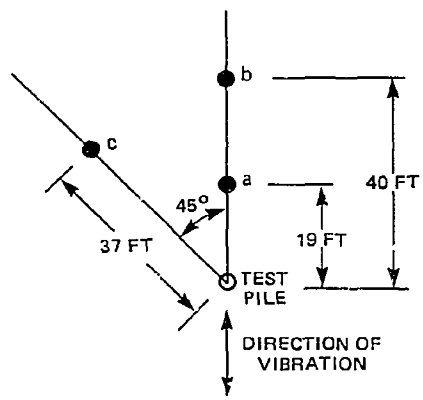
RADIAL GROUND VELOCITY F_r (mm/sec)

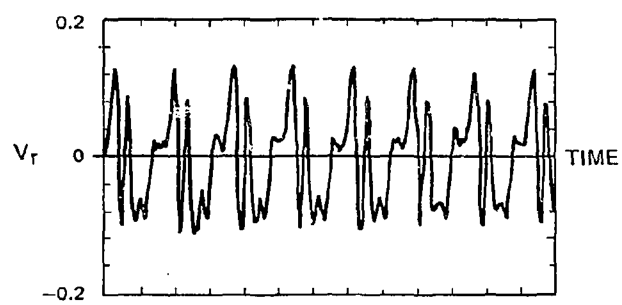


(a) 19 FT (5.79 m), 0° FROM PILE



(b) 40 FT (12.19 m), 0° FROM PILE





(c) 37 FT (11.18 m), 45° FROM PILE

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

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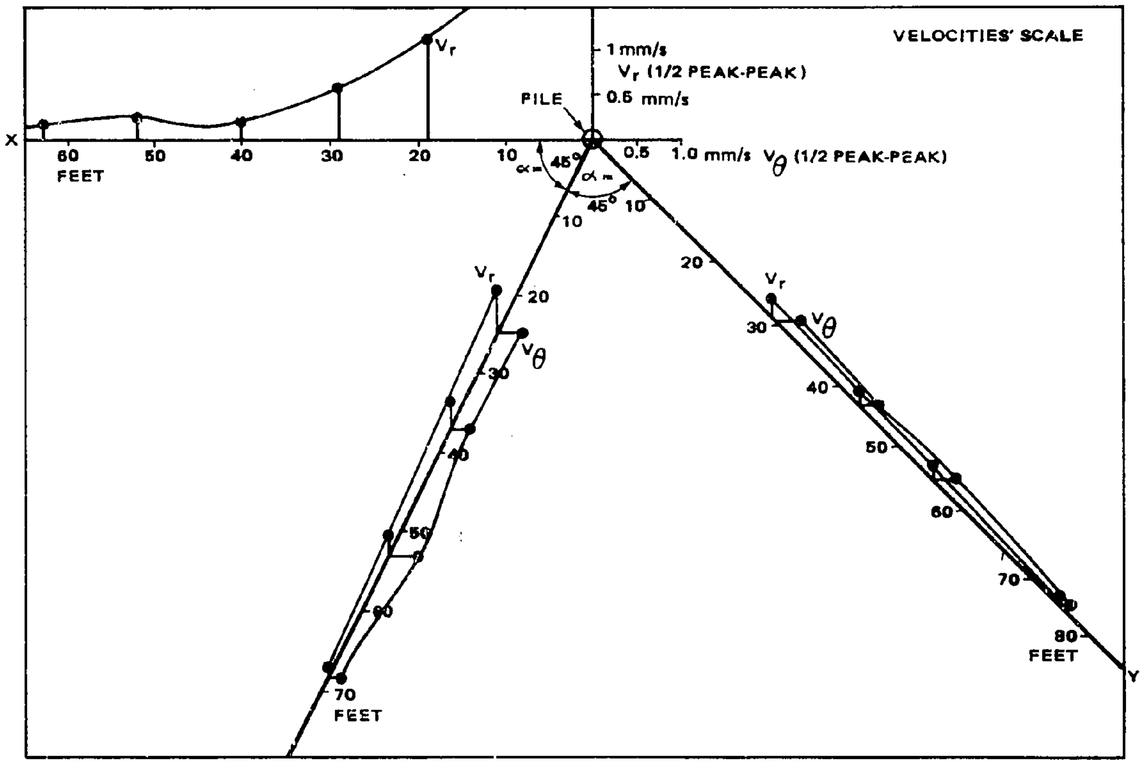
PROJECT NO.:	79-243
NSF PILE TEST	

RADIAL GROUND VELOCITY WAVE FORMS AT DIFFERENT LOCATIONS (TEST NO. 12)

8-81 FIGURE 6.11

36 M



Compiled by ____ Drawn by ____ Checked by ____ Approved by ____

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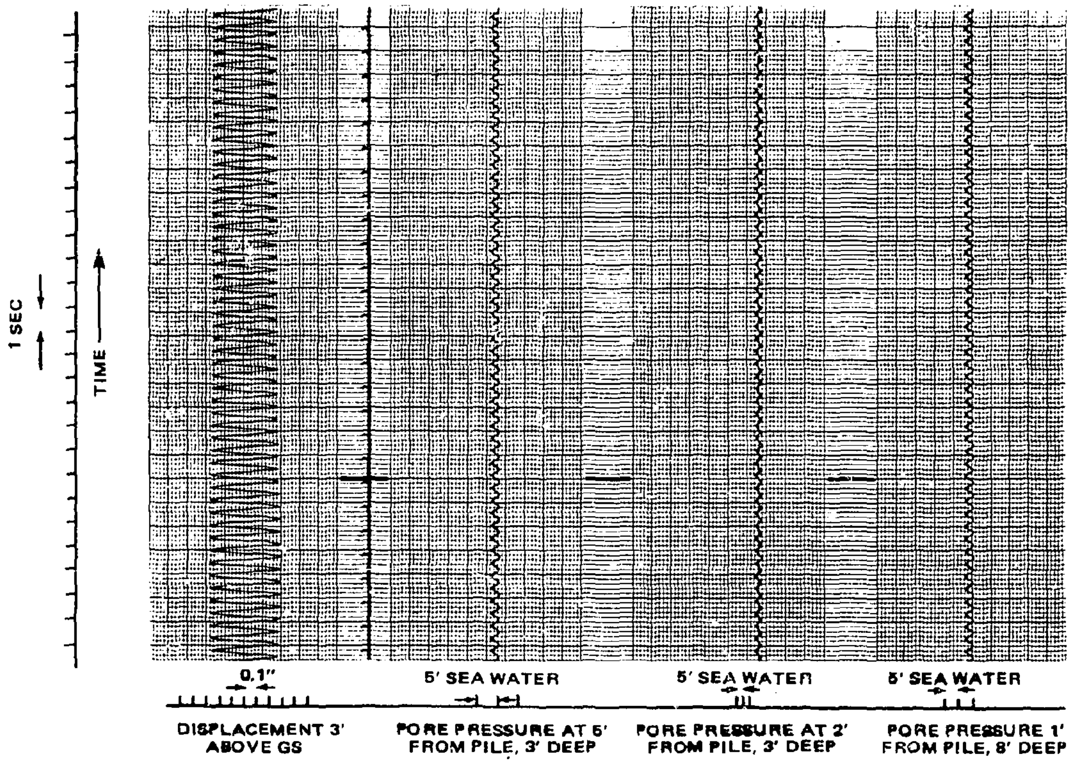
PERSPECTIVE VIEW OF GROUND VELOCITIES FOR TEST NO. 12 AT RESONANCE, f_1 = 2.87 Hz	
8-81	FIGURE 6.12

36.N

Compiled by ________ / Drawn by ________ / Checked by ________ / Approved by ________



(NOTE DIFFERENT SCALES IN PORE PRESSURE RECORDS, CAUSED BY DIFFERENT GAIN SETTINGS FOR EACH CHANNEL)

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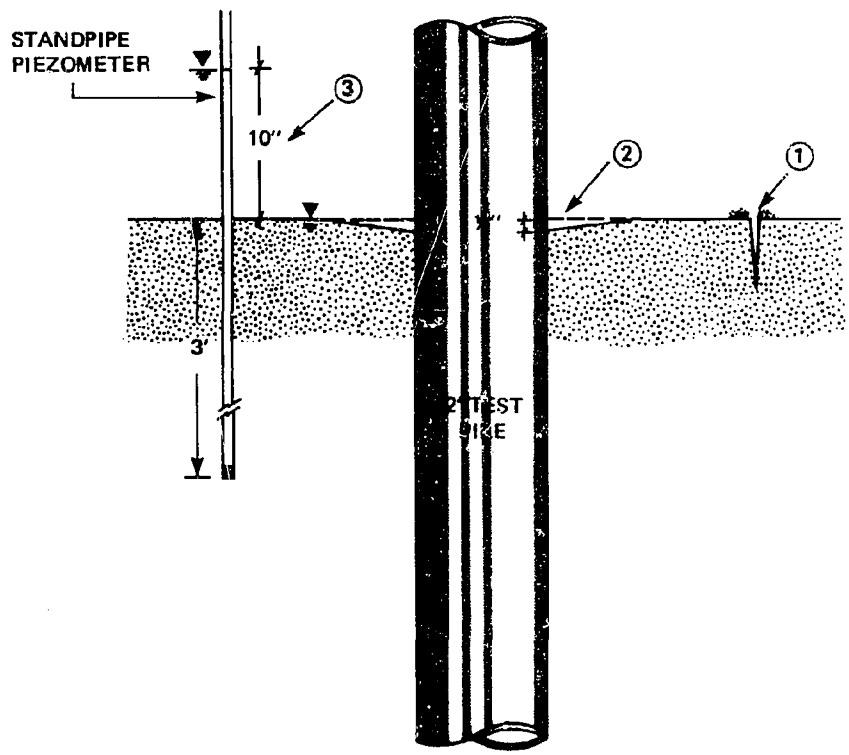
RECORD OF PORE PRESSURE OSCILLATIONS IN THREE PIEZOMETERS DURING TEST NO. 9	
10-81	FIGURE 6.13



① MINIATURE SAND BOIL
② PORE PRESSURE DISSIPATION RELATED CONSOLIDATION
③ EXCESS PORE PRESSURE INCREASE (AFTER ABOUT 30 CYCLES OF MEDIUM LEVEL SHAKING)

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NSF PILE TEST

SCHEMATIC ILLUSTRATION OF VIBRATION-INDUCED PORE PRESSURE BUILDUP RELATED RESPONSE

8-81

FIGURE 6.14

Compiled by ____ / Drawn by ____ / Checked by ____ / Approved by ____ /

7. CONCLUSIONS

The following comments briefly summarize the principal conclusions drawn from the results of the test program.

(1) Both observations and measurements clearly indicated that pore pressures in the soil surrounding the pile reached sufficiently high values during vibration tests that further indicated by the development of a depressed or subsided soil surface area around the pile as the test progressed and as the excess pore pressures dissipated.

(2) The effect of the partial liquefaction was also reflected in the observation that the stiffness of the pile-soil system reduced considerably during a sequence of large amplitude vibrations. The frequency of the first mode of vibration also decreased with increasing amplitude of pile motion, more than normally would be expected from the effects of soil non-linearity alone. Upon dissipation of excess pore pressures after a lapse of time between tests, the soil regained stiffness when low amplitude vibration tests were resumed.

(3) It was also observed that the damping of the pile-soil system was surprisingly low, on the order of 1 to 5% of critical damping. The damping values increased with the amplitude of pile motion.

(4) Large amounts of data were accumulated upon completion of the test program. Due to the fact that detailed analyses of the results were not in the original scope of work, it is strongly recommended that detailed analyses should be made to quantitatively define the effects of pore pressure development on moment distribution of the pile during vibration.

(5) In addition to the results obtained, experiences gained from this test program also indicate that a number of improvements can be effected in the equipment and tests if further work is to be carried out. These could include (a) higher shaking forces, and (c) more sensitive and stable pore pressure transducers.

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7. Novak, M. and R.F. Grigg, "Dynamic Experiments with Small Pile Foundations," Can. Geotech. Jour. 13, No. 4, Nov.-Dec. 1976 (Piles 2.4 and 3.5 inches diameter, 90 inches long).

8. Reese, L. C., W. R. Cox, and F. D. Koop, "Analysis of Laterally Loaded Piles in Sand," Sixth Offshore Technology Conference, Preprints, Paper No. OTC 2080, Houston, Texas, 1974, Vol. II, pp. 473-483.

9. Scott, R.F., Liu, H.P., and J. Ting, "Dynamic Pile Tests by Centrifuge Modeling," Proceedings, 6th World Conference on Earthquake Engineering, New Delhi, India, Paper 4-50, 1978.

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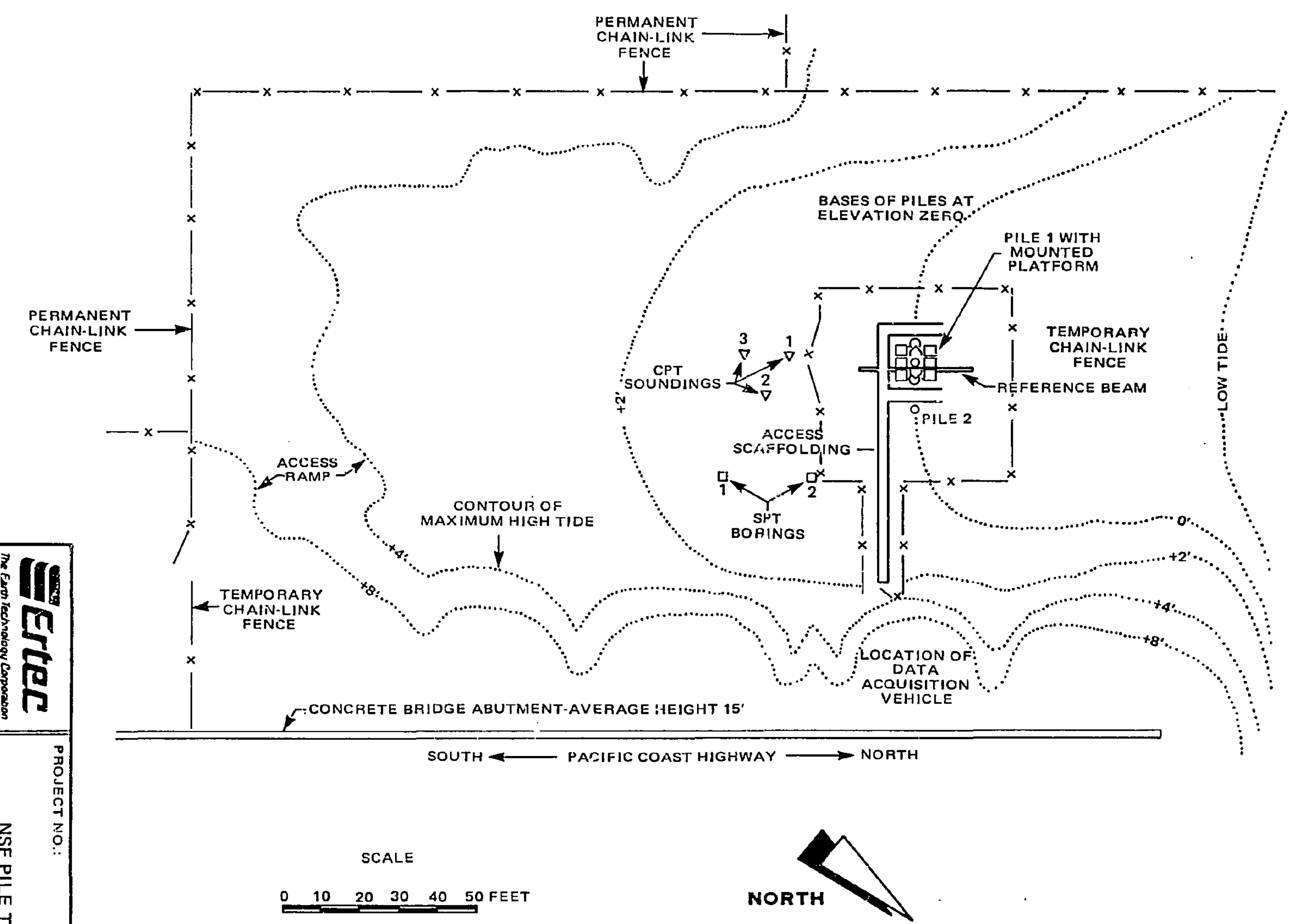
11. Tajimi, H., "Piles Subjected to Dynamic Lateral Loading," State of the Art Report, Specialty Session 10, 9th Intl. Conf. Soil Mech. and Found. Eng. 3, 553-557, 1977.

APPENDIX A

GEOTECHNICAL INVESTIGATIONS

Compiled by ______ / ______ Drawn by ______ / ______ Checked by ______ / ______ Approved by ______ / ______



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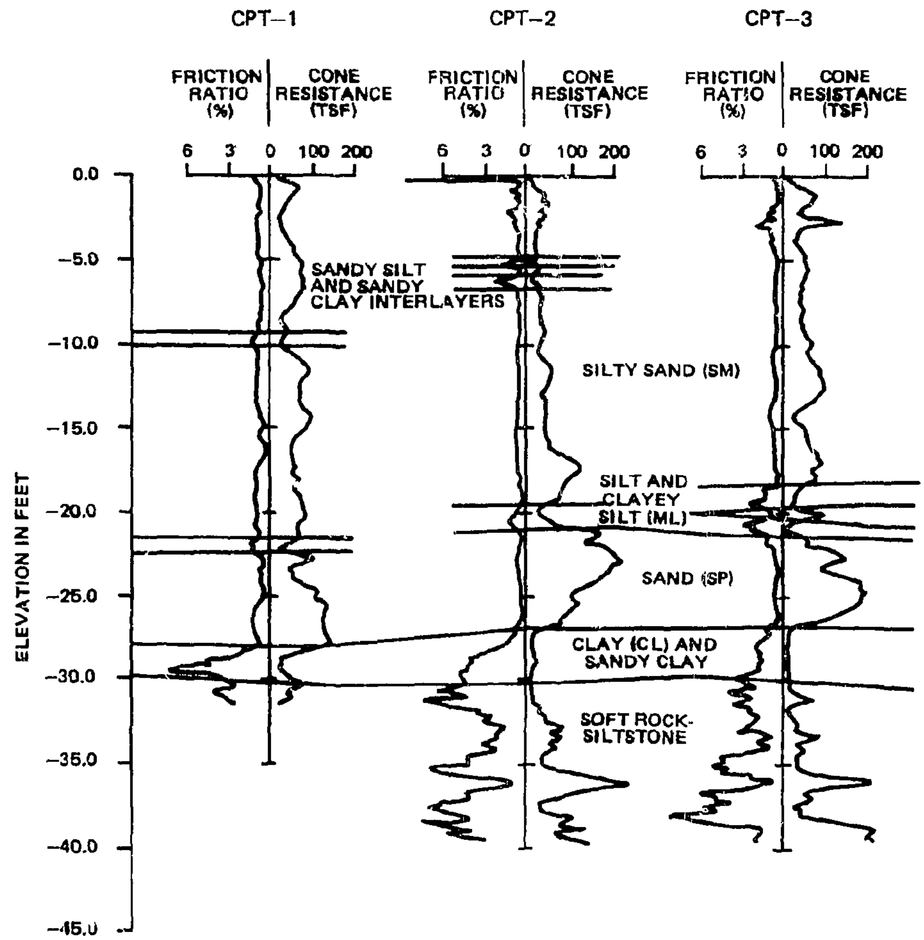
NSF PILE TEST

PLAN VIEW OF SITE, SHOWING LOCATIONS OF PILES, SPT BOREHOLES, AND CPT SOUNDINGS

8-81

FIGURE A.1



LOCATIONS OF SOUNDINGS ON FIGURE A-1

Compiled by / Drawn by / Checked by / Approved by

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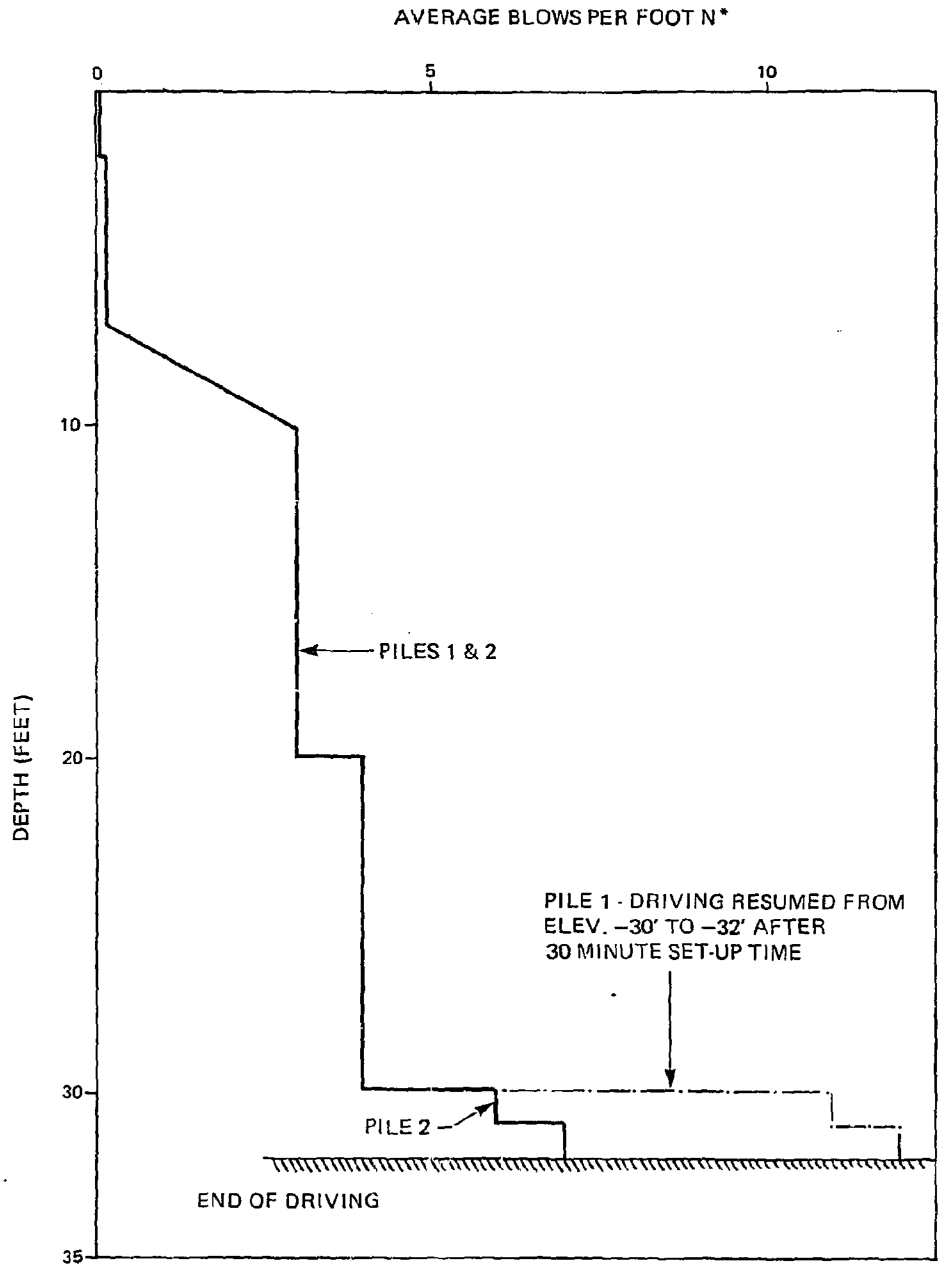
PROJECT NO.: 79-243

NSF PILE TEST

SEAL BEACH SITE
CPT PROFILE

8-81

FIGURE A-2



*AVERAGE BLOW COUNT VALUES PLOTTED BASED ON FIELD ACTIVITY LOG

Compiled by ______ / ______ Drawn by ______ / ______ Checked by ______ / ______ Approved by ______ / ______

Ertec — The Earth Technology Corporation	PROJECT NO.: 79-243 NSF PILE TEST
PILE DRIVING RECORDS	
10-81	FIGURE A.3

SHEET NO. 1 OF 2

FIELD LOG OF BORING NO. 1

PROJECT NO. 79-243-01 DATE (S) 11-18-80 ELEVATION 0' M.S.L.
PROJECT NAME NSF DATUM
DRILLING COMPANY PITCHER LOGGED BY ANPKAY
DRILL SIZE ; METHOD SPT & WASH WATER LEVEL +0.5' AFTER - HOURS
DRIVING WEIGHT 140# AVERAGE DROP 30" TIME 10:15 (SETUP) 10:30 (START) (STOP)

Depth	Bulk	Core	Tip Depth	Recovery	Blowcount	Moisture	Consistency	Color	USCS Symbol	Percent GR/SA/FINES	Description
0	X	1			5,6,7 (13)						SAND, SILTY, FINE, BLACK
2											AUGERED AND CASED TO 4' AFTER SPT
4	X	2			5,7,9 (16)						SAND, SILTY w/ SILT, SL. SANDY, FIRM NEAR END OF RUN
7	X				5,7,6 (13)						SAMPLE LOST
	X				5,6,7 (13)						SAMPLE LOST
9	X	3			4,6,7 (13)						FINE SAND, SILTY, DK. GRAY
1											DITTO
	X	4			4,5,8 (13)						
2	X	5			7,8,9 (17)						SAND, FINE, SILTY, DK. GRAY
4	X	6			3,4,6 (10)						DITTO
	X	7			3,6,6 (12)						DITTO, SL. SILTY, SILTY
7	X	8			2,4,5 (9)						SILT, SANDY OVER SAND, V. SHELLY, SL. SILTY
9	X	9			3,4,7 (11)						SHELLS OVER SILT, SANDY & SAND, V. SILTY
2											

NOTES: BOREHOLE 160' FROM CORNER OF FENCE, 65' FROM BRIDGE ABUTMENT

Prep.By:____ Date:____ Ckd.By:____ Date:____ Corr.By:____ Date:____ ReCkd.By:____ Date:____ Appr.:____ Date:____

SHEET NO. 2 OF 2

FIELD LOG OF BORING NO. 1

PROJECT NO. 79-243-01 DATE (S) 11-18-80 ELEVATION 0' MSL

PROJECT NAME NSF DATUM

DRILLING COMPANY PITCHER LOGGED BY AND KAY

DRILL SIZE; METHOD SPT & WASH WATER LEVEL ______ AFTER ______ HOURS

DRIVING WEIGHT ______ AVERAGE DROP ______ TIME ______ (SETUP) ______ (START) 1:45 (STOP)

Depth	SAMPLES Bulk	Core	Tip Depth	Recovery	Blowcount	Moisture	Consistency	Color	USCS Symbol	Percent GR/SA/Fines	Description
20	X	10			3,4,5 (9)						SILT-SAND (FINE), DK. GRAY, SOME SHELLS (POSSIBLY TOPHOLE)
	X	11			3,8,12 (20)						SILT, CLAY, SOFT, GRAY-GREEN, SHELLY OVER
2	X	12			3,3,11						SAND, SL. SILTY, MED. FINE, DK. GRAY, SOME SHELL
	X	13			(14?)						DITTO - NOT MUCH SILT, A LOT OF SHELL, GENERALLY LESS THAN 1/4" Ø
5	X	14			3,8,9 (17)						DITTO
	X				6,2,3						DITTO
	X	15			(5)						SILT, SL. CLAYEY, MED. GRAY
7	X	16			2,3,4 (7)						DITTO, FIRM
	X	17			1,4,8 (12)						DITTO, FIRM - STIFF
30	X	18			3,3,6 (9)						SILTSTONE, FRAGMENTS, CLAYEY, LT. GRAY, WEATHERED, STIFF, HARD FRAGS. TO 1/2" Ø, HARD SILT, NO CLAY, SHELLS, SOME TOPHOLE CLAYEY SILT.
2	X	19			6,12,16 (28)						SILT, NON-PLASTIC (JUST V. SLIGHTLY GREASY FEEL) COMPACT (FIRM-STIFF?) GRAY-GREEN
4	X	20			5,9,13 (22)						SILT, SL. PLASTIC, STIFF-HARD? LT. GRAY-GREEN BECOMING LT. BROWN
5	X	21			7,10,21 (31)						SILT, SANDY - SL. SANDY, STIFF (-HARD?) V. COMPACT, BROWN
7	X				7,11,12 (23)						SAMPLE LOST, SANDY?
9											
A											

NOTES:

Prep. By ______ Date ______ Ckd. By ______ Date ______ Corr By ______ Date ______ ReCkd. By ______ Date ______ Appr. ______ Date ______

L-16A

SHEET NO. 1 OF 2

FIELD LOG OF BORING NO. 2

PROJECT NO. 79-243-01 DATE (S) 11-18-80 ELEVATION 0' MSL

PROJECT NAME NSF DATUM

DRILLING COMPANY PITCHER LOGGED BY ANP KAY

DRILL SIZE; METHOD SPT & WASH WATER LEVEL AFTER HOURS

DRIVING WEIGHT 140# AVERAGE DROP 30" TIME 1:45 (SETUP) 2:05 (START) (STOP)

Depth	SAMPLES: BULK	CORE	TIP DEPTH	RECOVERY	BLOWCOUNT	MOISTURE	CONSISTENCY	COLOR	USCS SYMBOL	PERCENT GR/SA/FINES	DESCRIPTION
0											
1											
2			WASHED & CASED								
3											
4											
5											
6	1				4,5,5 (10)						SANDY SILT, FIRM ?, DK. GRAY - BLACK, SMELLS SL. ORGANIC
7											
8			WASH								
9											
10	X				3,3,4 (7)						NO SAMPLE
11											
12											
13											
14											
15	2				3,8,8 (16)						SAND (FINE) V. SILTY, & SILT, SANDY DK. GRAY
16											
17											
18											
19											
20											

NOTES: BOREHOLE 137' FROM CORNER OF FENCE, 65' FROM BRIDGE ABUTMENT, 23' UPHILL FROM HOLE 1

Prep.By ____ Date: ____ Ckd. By: ____ Date: ____ Corr By: ____ Date: ____ ReCkd.By: ____ Date: ____ Appr.: ____ Date: ____

L-16A

FIELD LOG OF BORING NO. 2

SHEET NO. 2 OF 2

PROJECT NO. 79-243-01 DATE (S) 11-18-80 ELEVATION 0' MSL

PROJECT NAME NSF DATUM

DRILLING COMPANY PITCHER LOGGED BY ANP KAY

DRILL SIZE; METHOD SPT & WASH WATER LEVEL ____ AFTER ____ HOURS

DRIVING WEIGHT 140# AVERAGE DROP 30" TIME ____ SETUP ____ START 4:00 STOP

SAMPLES: BULK	CORE	TIP DEPTH	RECOVERY	BLOWCOUNT	MOISTURE	CONSISTENCY	COLOR	USCS SYMBOL	PERCENT GR/SA/FINES	DESCRIPTION
X	3			3,5,6 (11)						SAND, SL. SILTY, V. SHELLY, DK GRAY
X	4	(3")		3,15,16 (31)						DITTO
X	5			1,12,13 (25)						DITTO
X	6			2,8,8 (16)						SILT, CLAYEY, FIRM, DK. GRAY
X	7			3,4,6 (10)						DITTO
X	8			7,19,30 (49)						SILT, SL. SANDY (V. SL. CLAYEY ?) DENSE OR COMPACT, BROWN, LT. BROWN.

Depth scale (left margin): 2, 2, 4, 7, 9, 3, 2, 4, 6, 7, 9, 4

NOTES:

Prep.By____ Date____ Ckd.By:____ Date:____ Corr By:____ Date____ ReCkd.By:____ Date:____ Appr.____ Date:____

APPENDIX B

SPECIFICATIONS

VIBRATION GENERATORS

Table B.1 - Maximum frequency (Hz) for each weight combination which gives maximum design force of 5,000 lbs.

		Small Weights				
		0	S1	S2	S3	S4
Large Weights	0	9.7	7.2	6.0	5.2	4.7
	L1	5.0	4.6	4.2	3.9	3.7
	L2	3.8	3.6	3.4	3.3	3.1
	L3	3.2	3.1	3.0	2.8	2.8
	L4	2.8	2.7	2.6	2.6	2.5

Table B.2 - "WR" (Lb-In) for each weight combination which gives maximum design force of 5,000 lbs.

		Small Weights				
		0	S1	S2	S3	S4
Large Weights	0	520	947	1374	1801	2228
	L1	1935	2362	2789	3216	3643
	L2	3350	3777	4204	4631	5058
	L3	4765	5192	5619	6046	6473
	L4	6180	6607	7034	7461	7888

FORCE $= 0.102 \times (WR) \times f^2$

f = frequency (Hz)

(WR) is in "Lb-In"

FORCE is in "Lb"

S = small weight (center section)

L = large weight (side section)

The number (1, 2, 3, or 4) following "S" or "L" indicates the number of weights of that size placed in each section of that size in each weight bucket.

TABLE B.3

SPECIFICATIONS

Single Unit (Master or Slave)

Weight

Controller Console:	100 lbs. (approx.)
Vibration Generator (no lead weights):	600 lbs. (approx.)
Lead Weights:	900 lbs. (approx. including weight racks)

Input Power Requirements

Voltage:	220 VAC, 50 or 60 Hz, single phase
Current:	15 amps (maximum)

Force Output

Maximum Unidirectional Sinusoidal Force in the Horizontal Plane:	100 lbs. at 0.35 Hz 5000 lbs. over 2.5 Hz to 9.7 Hz range
Direction:	Adjustable in 3-1/3° steps over 360° range

Frequency Control

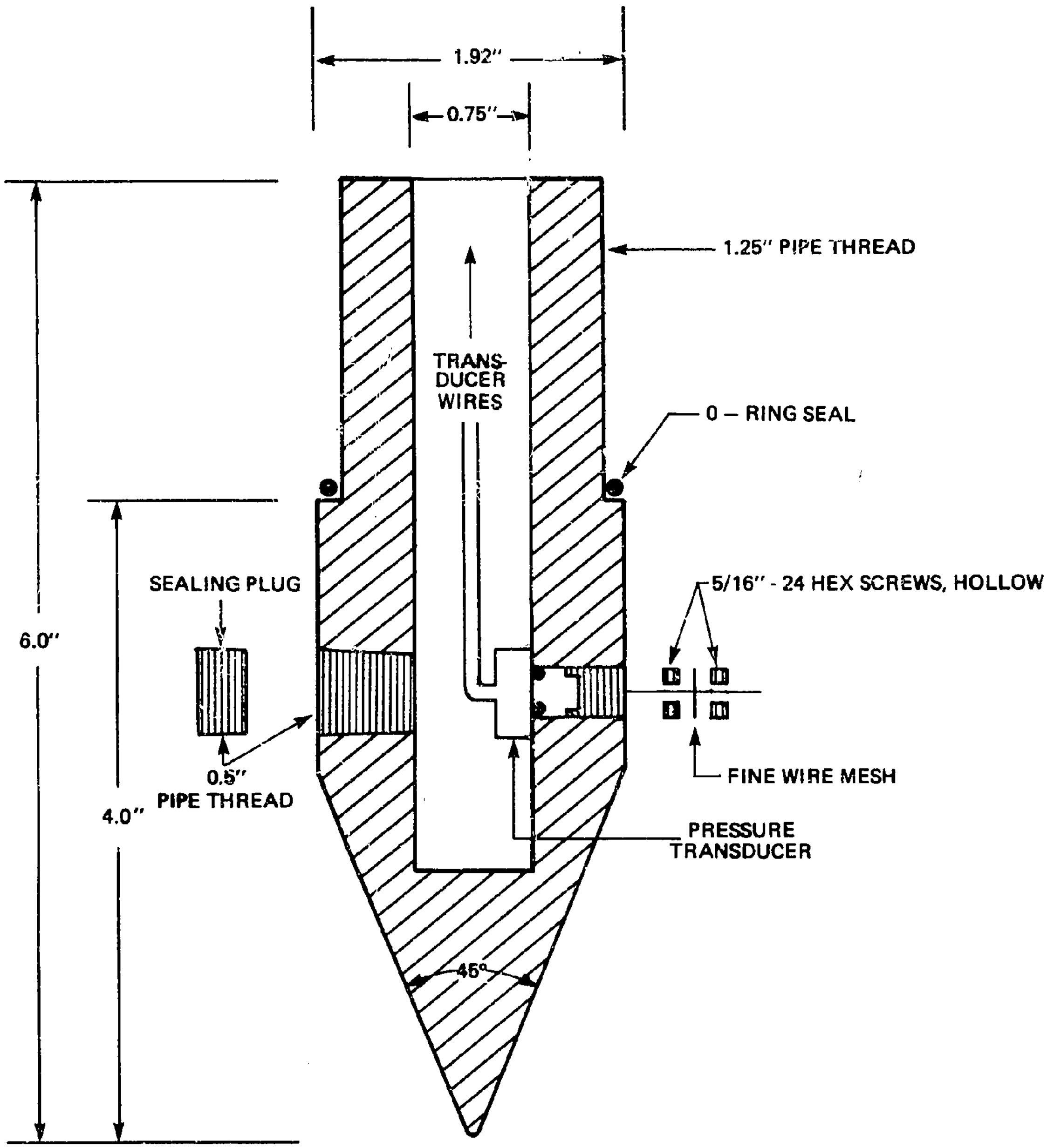
Range:	0 to 9.7 Hz
Adjustment:	Continuous
Stability:	$\pm$0.005 Hz
Measurement:	Encoder with 1000 pulses per revolution feeding a digital counter in console. Counter can read to $\pm$0.001 Hz

Master-Slave Phase Control

Adjustment:	108° switch Continuous dial with 1° divisions
Phase Error between Master and Slave:	Up to 2° nominal

APPENDIX C

PIEZOMETERS AND DATA ACQUISITION SYSTEM



DIMENSIONS ARE IN INCHES
1 in = 25.4 mm

MATERIAL: ALUMINUM
QUANTITY: 3
DESIGNED AND MANUFACTURED BY CALTECH

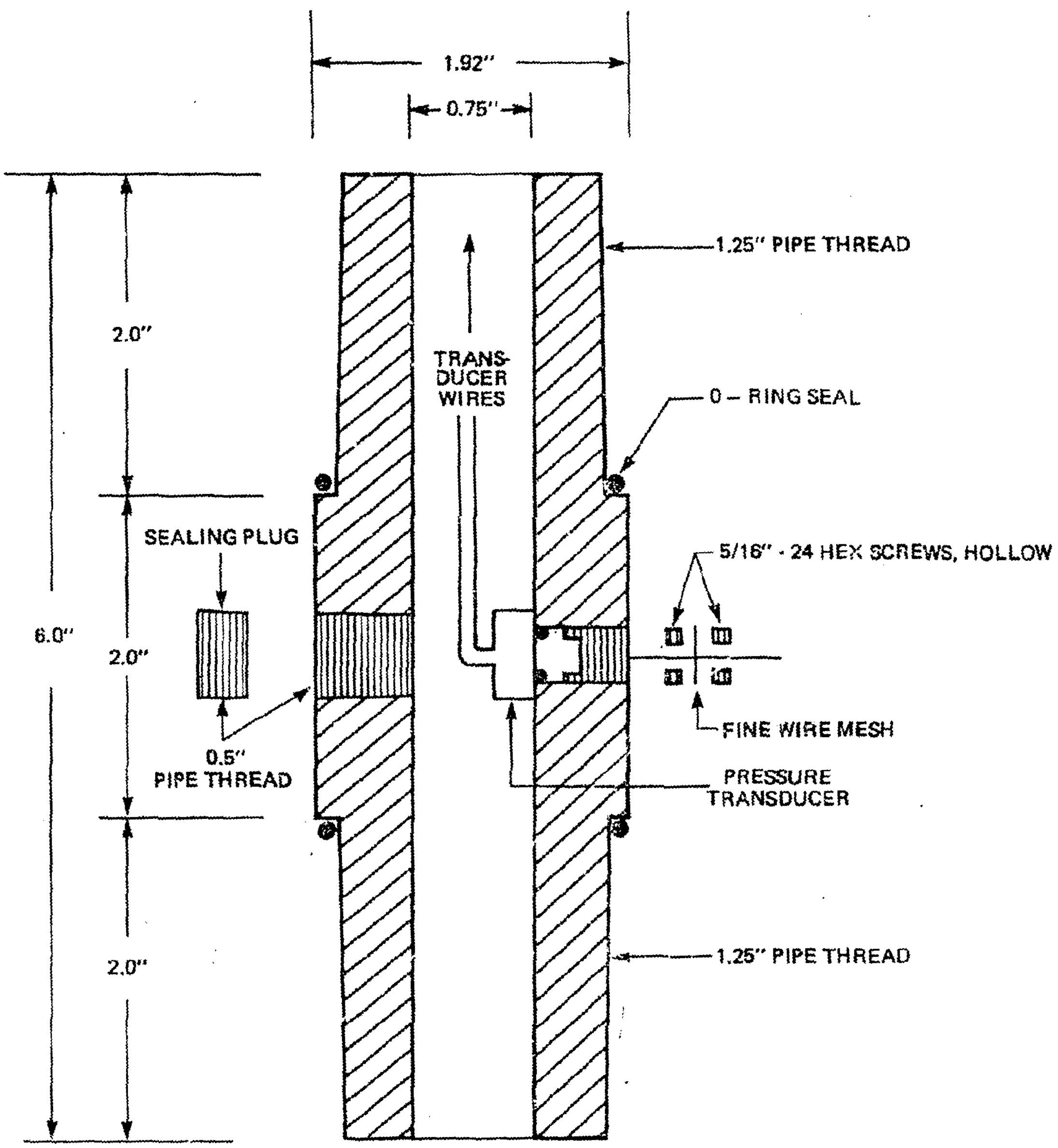
Ertec The Earth Technology Corporation	PROJECT NO.: 79-243 NSF PILE TEST

SECTION DRAWING OF
PIEZOMETER TIP

8-81

FIGURE C.1

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______



DIMENSIONS ARE IN INCHES
1 in = 25.4 mm

MATERIAL: ALUMINUM
QUANTITY: 1
DESIGNED AND MANUFACTURED BY CALTECH

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PROJECT NO.: 79-243

NSF PILE TEST

SECTION DRAWING OF
MIDDLE PIEZOMETER

8-81

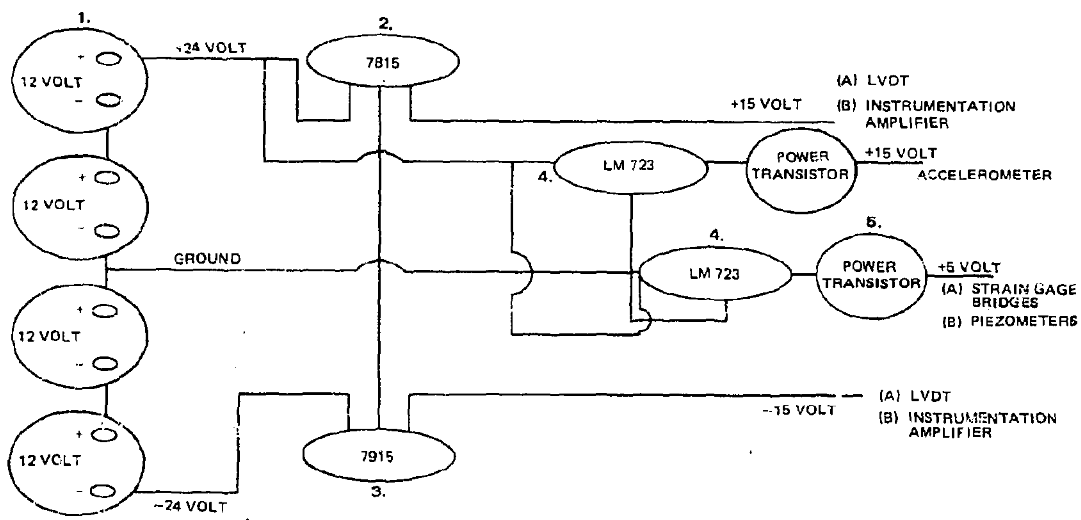
FIGURE C.2

Approved by
Checked by
Drawn by
Compiled by

COMPONENTS

1. 4 GEL/CELL BATTERIES – MODEL GC 1245
2. 7815 INTEGRATED CIRCUIT AMPLIFIER
3. 7915 INTEGRATED CIRCUIT AMPLIFIER
4. LM 723 MICRO ADJUSTABLE VOLTAGE REGULATOR
5. 3 TERMINAL POWER TRANSISTOR 2N4914

SYSTEM



MANUFACTURED BY CALTECH

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PROJECT NO.: 79-243

NSF PILE TEST

INSTRUMENT POWER SUPPLY

8-81

FIGURE C.3

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

AMPLIFIER COMPONENTS

1. LF 352 INSTRUMENTATION AMPLIFIER
   CMRR 80 – 90 OB
   ADJUSTABLE 1-1000 (RUN AT 150 FOR TESTS)

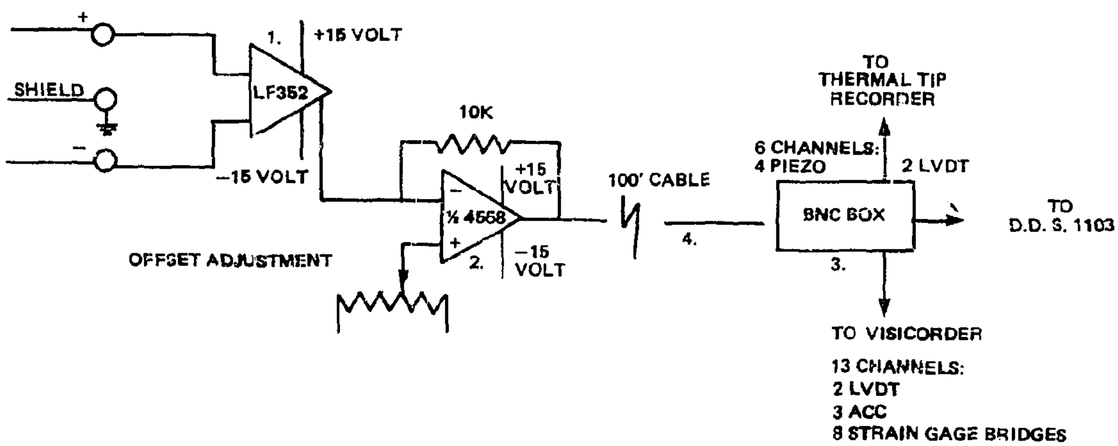
2. 4558 OP-AMP
   DUAL 8 PIN

3. BNC BOX JUNCTION BOX

4. 100′ CABLE 19 CONDUCTOR INDIVIDUALLY SHIELDED
   TWISTED PAIR BY BELDEN

TYPICAL INPUT SHOWN

16 INPUTS SUPPLIED



NOTE: MANUFACTURED BY CALTECH

Ertec The Earth Technology Corporation	PROJECT NO.: 79-243 NSF PILE TEST
INSTRUMENT AMPLIFIER	
8-81	FIGURE C.4

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

APPENDIX D

TEST RESULTS

APPENDIX D

TEST RESULTS

D.1 Reduction of Data

This Appendix describes the procedures used in reducing, analyzing. and plotting the individual analog data from each test, and presents results from all the tests. Although a digitizer and tape encoder were used on the first three test dates (June 26, June 30, July 1, 1981), as well as analog recording, hardware problems prevented subsequent access to any digital data which may have been recorded on these dates. Consequently, the bulk of the data was hand-digitized from the strip chart recordings after completion of the tests, primarily to obtain peak displacements and strains at a particular frequency of vibration.

As already mentioned, the data recorded for each test include: strain gage outputs at eight depth locations using full strain gage bridges on the pile; lateral displacements at two locations using long-stroke DC-LVDT's; up to three components of acceleration on the pile at one location; ground velocity at various locations using a Ranger seismometer; pore pressures at four locations using electrical pore pressure tranducers, which were supplemented by clear plastic standpipes at the two near piezometer locations for testing on July 14.

Raw strain gage bridge outputs, zeroed prior to testing each day, were converted to strain in the pile by using the average manufacturer's stated gage factor for the gages, accounting for input voltage, amplification, and recorder sensitivity. From solid mechanics theory, the pile curvature $1/\rho$ is equal to the second derivative of lateral displacement, w, with respect to depth x, $\frac{d^2w}{dx^2}$, which is just the fiber strain, ε_c, divided by the distance to the neutral axis, c. In the tests, the distance from the strain tube to the neutral axis is 10.75 in. (273 mm). By plotting the discrete data at the eight strain gage locations, the entire $\frac{d^2w}{dx^2}$ curve may be approximated. Assuming the slope, $\frac{dw}{dx}$, is zero at the bottom of the pile the $\frac{d^2w}{dx^2}$ curve may be numerically integrated to obtain the $\frac{dw}{dx}$ curve. Similarly, by assuming the lateral displacement, w, is zero at depth, the entire displacement curve may be obtained by numerically integrating from the slope curve. The slope and displacement curves may be verified using the DCDT outputs above ground surface, and this typically indicates a fairly good correlation (as shown in Fig. D.10 for test no. 6 at resonance). Typical peak displacement at one foot above ground surface ranged from 0.002 in. (0.05 mm) for the pile ringdown tests to greater than 0.4 in. (10.2 mm) for the large-force shaking tests.

Since the moment in the pile, M, is equal to $-EI\frac{d^2w}{dx^2}$, then by assuming $E = 30 \times 10^6$ psi (2.07×10^{11} Pa) for mild steel and $I = 2.93 \times 10^3$ in.^{4} (1.22×10^{-3} m^4) for the open pipe pile, the moment in the pile could be computed. Similarly, the fiber stress $\sigma_c = \frac{Mc}{I}$, where c is the distance from the fiber to the neutral axis of bending, is obtained. These are shown together with the strain and curvature in Figs. D.8 through D.13 for various tests.

Peak horizontal forces during shaking were computed from available reference data on the shaking machines (Tables B.1 and B.2), based on the centrifugal force relationship. For a given number and size of weights placed in the swinging arms, the peak force could be computed as a function of the input frequency squared. Typical peak forces generated in the shaking tests ranged up to 3766 lbf (16.7 kN) in Test no. 10 at 8.43 hz.

The input shaking frequency could be read directly from the shaker controllers, and could be verified from the period of the various observed transducer outputs. Frequency sweeps up to 8.6 Hz were conducted, with resonance occurring between 1.7 and 2.9 Hz, depending on the level of force and stress history of the sand adjacent to the pile prior to shaking.

Horizontal accelerations of the pile at 0° and 90° to the direction of shaking may be computed using the gage calibration factors (App. C-3), input voltage amplification factors, and recorder sensitivity. For the shaking tests, peak horizontal inline accelerations computed from the amplitude of displacement and forcing frequency yielded values typically within $\pm 50\%$ of the measured accelerations at two feet above ground surface. Typical measured peak horizontal accelerations ranged from 0.006 g in the last ringdown test no. 16, to 0.265 g in the large shaking test no. 9.

Pore pressure data were obtained for the piezometers, which were zeroed at the beginning of each testing day. Pressure differences could be computed using the calibration factors, input voltage, amplification factors, and sensitivity of the recorder. Since the water table remained approximately at the ground surface during each test sequence, the actual pore pressure could be computed from the measured pore pressure differences. However, problems with transducer offsets leading to insufficient sensitivity on the HP recorder, coupled with hardware connection difficulties with the Honeywell Visicorder, precluded accurate pore pressure measurements on all but the strongest shaking tests, such as test no. 9. The standpipes installed on July 1 provided a more accurate picture of the pore pressure buildup during cyclic excitation, before themselves clogging up. Visual observation of the near standpipe during test no. 10 indicated a gradual excess pore pressure buildup to about 8 in. (203 mm) of sea water at resonance.

Ground velocity measurements using Ranger SS-1 Seismometers were made at various locations. While the data easily yield relative magnitudes, obtaining absolute values is more difficult. By using internal data for each individual seismometer, it is possible to compute a theoretical calibration factor for the system, accurate to about $\pm$ 25%. This was done for the velocity profile data obtained in test no. 12. Since the components of horizontal ground velocity were not sinusoidal, digitization and vector addition of the components at different times during each loading cyclic are necessary for data reduction.

D.2 Description of Summary Plots

Two basic types of summary plots were made for this preliminary data summary: response curves and mode shapes. For the former, the peak displacement amplitude A (one-half the peak-to-peak displacement) at one foot above the ground surface, corrected for eccentric force by dividing by frequency squared, f^2, $\frac{A}{f^2}$, is plotted against the input frequency. For tests 1 and 2, shown in Figs. D.1 and D.2, DCDT outputs were not available due to difficulties with the digital tape system. Consequently, the peak amplitudes for these tests were obtained by double integration of the strain data. As seen in Fig. D.8, this procedure yields displacements which are in accord with the approximate DCDT output as read on an oscilloscope.

Resonance for a given loading system corresponds to the peak corrected A/f^2 response. The damping ratio ($\beta = \frac{c}{c_{crit}}$) may be obtained from half the bandwidth, Δf , at the peak response A_n/f_n^2, divided by $\sqrt{2}$, i.e. $\beta = \frac{\Delta f}{2\sqrt{2}}$. Typical damping ratios obtained from frequency sweeps range from 1.5 to 7% of critical. For the first several frequency sweeps, response was obtained at discrete frequencies. The later sweeps were carried out at continuously varying frequencies. For these, response was plotted at increasingly finer frequency intervals near resonance. Typically, the more points used near resonance, the smaller the damping ratios computed.

The mode shapes were obtained as already described by double integration of the strain data. The typical shape at resonance is shown in Fig. D.8. At the higher frequencies, say 8.6 Hz in test no. 6, the second natural mode of vibration starts to dominate, as seen in Figure D.11. This frequency (8.6 Hz) is the highest that the shaking machines could produce, and as a result resonance at the second mode could not be obtained.

For ringdown testing, peak pre-release displacement and strains were obtained by measuring from the preloading baseline. After release, values were obtained using the end of test baseline. Frequencies of the ringdowns were obtained by digitization of the peak displacements. Damping of the pile system was obtained from the log decrement, ie. $2\pi n\beta = \ln(\frac{w_i}{w_{i+n}})$ where w_i is the displacement of the ith cycle, and w_{i+n} is the displacement of the i+nth cycle. Typical damping for the ringdown tests ranged from 1.5 to 3%.

Peak force in the rope, F, was estimated using data from the top two strain gages. From $M = -EI \frac{d^2w}{dx^2} = Fe$, where e is the distance from the gage to the line of action of the lateral force, the peak force exerted by the rope was estimated to be about 725 lbf (3.22 kN) in the last ringdown test, no. 16. The peak moment generated in the pile by this force was 0.44 x 10^5 lbf-in (5.0 kN-m), as compared to a value of about 5.09 x 10^5 lbf-in (5.75 kN-m) in the maximum force resonance shaking test (test no. 9). The corresponding peak fiber stresses were 195 psi (1.34 MPa) for ringdown test no. 16 and 2251 psi (15.5 MPa) for shaking test no. 9.

D.3 Results of Testing

From the procedures described in the previous sections of this Appendix, the raw data for the various strain gages, displacement transducers, accelerometers, piezometers, and seismometers are reduced to physical units, and are compiled in standard plots and tables. Table D.1 presents the complete summary of all the forced vibration tests using the shaking machines, and includes the pertinent information about each test, such as frequency range, force range, natural frequency (if obtained), and the damping ratio, peak pile displacement, peak pile acceleration and peak moment in the pile at resonance. Table D.2 presents a similar summary for the ringdown tests no. 13 to 16.

Figs. D.1 through D.7 present the response curves for each frequency sweep where resonance was obtained. Peak strain, slope, and displacement plots at resonance for selected tests are found in Figs. D.8 through D.12, and include a mode shape at 8.62 hz, indicative of the second mode of vibration for the pile system. Fig. D.13 shows the pile shapes for ringdown test no. 16 just prior to release and for the first peak after release of the loading force.

TABLE D.1

SUMMARY OF FREQUENCY SWEEP PILE SHAKING TESTS

TEST NUMBER	TEST DATE (1981)	RUN NUMBER	FREQUENCY RANGE (HZ)	FORCE RANGE (LBF) [KN]	FREQUENCY FN (HZ)	DAMPING RATIO β (%)	AT RESONANCE: PEAK DISPLACEMENT AN FN AT 1' ABOVE GROUND SURFACE (IN) [MM]	AT RESONANCE: AN/FN2 (IN/HZ2) [MM/HZ2]	AT RESONANCE: COMPUTED PEAK ACCELERATION AT 1' ABOVE GROUND SURFACE	AT RESONANCE: PEAK HORIZONTAL FORCE (LBF) [N]	AT RESONANCE: PEAK MOMENT IN PILE (LBF-IN) [KN-M]	COMMENTS
1	JUNE 26	1	0-6.75	0-2414 [0-10.7]	2.62	3.8	0.025 [0.63]	3.64×10^{-3} [9.18×10^{-2}]	0.018	364 [1.62]	1.79×10^{5} [20.2]	LOW LEVEL FREQUENCY SWEEP VIRGIN CONDITIONS
2	JUNE 30	1	0-8.18	0-3546 [0-15.8]	2.80	3.0	0.034 [0.84]	4.34×10^{-3} [1.07×10^{-1}]	0.027	416 [1.85]	2.46×10^{5} [27.7]	LOW LEVEL FREQUENCY SWEEP
3	JULY 1	1	0-1.24	0-1762 [0-7.8]	>1.24	NA	> 0.43 [10.9]	NA	0.068	1762 [7.8]	—	HIGH LEVEL SHAKING; RESPONSE NOT OBTAINED
4	JULY 1	2	0-1.50	0-1931 [0-8.6]	>1.50	NA	> 0.37 [9.40]	NA	0.085	1931 [8.6]	—	HIGH LEVEL SHAKING; RESPONSE NOT OBTAINED
5	JULY 1	3	0-3.86	0-4172 [0-18.6]	1.72	6.0	0.22 [5.59]	7.44×10^{-2} [1.89]	0.067	828 [3.7]	—	MEDIUM LEVEL FREQUENCY SWEEP
6	JULY 1	4	0-8.62	0-3938 [0-17.5]	2.27	3.0	0.045 [1.14]	8.73×10^{-3} [2.21×10^{-1}]	0.024	273 [1.2]	4.15×10^{5} [46.9]	LOW LEVEL FREQUENCY SWEEP
7	JULY 1	5	0-8.45	0-3784 [0-6.8]	2.50	7.0	0.045 [1.14]	7.20×10^{-3} [1.82×10^{-1}]	0.029	331 [1.5]	—	LOW LEVEL FREQUENCY SWEEP
8	JULY 1	6	0-1.85	0-1390 [0-6.2]	>1.85	NA	> 0.283 [6.43]	NA	0.099	1390 [6.2]	—	MEDIUM LEVEL SHAKING; RESPONSE NOT OBTAINED
9	JULY 1	7	0-3.58	0-4709 [0-20.9]	2.01	5.5	0.259 [6.58]	6.41×10^{-2} [1.63]	0.107	1483 [6.6]	5.8×10^{5} [67]	MEDIUM LEVEL FREQUENCY SWEEP
10	JULY 14	1	0-8.43	0-3766 [0-16.8]	2.67	1.5	0.045 [1.15]	6.31×10^{-3} [1.62×10^{-2}]	0.033	377 [1.7]	—	LOW LEVEL FREQUENCY SWEEP
11	JULY 14	2	0-2.57	0-2427 [0-10.8]	> 2.57	NA	> 0.25 [6.32]	NA	0.169	2427 [10.8]	—	MEDIUM LEVEL SHAKING; RESPONSE NOT OBTAINED
12	JULY 14	3	0-2.9	0-2357 [0-10.5]	2.87	NA	0.207 [5.26]	2.51×10^{-2} [6.39×10^{-1}]	0.174	2309 [10.3]	—	MEDIUM LEVEL FREQUENCY SWEEP; VELOCITY PROFILE TAKEN

NA = NOT AVAILABLE

— = NOT COMPUTED

TABLE D.2

SUMMARY OF RING-DOWN TESTS

TEST NUMBER	TEST DATE (1981)	RUN NUMBER	NATURAL FREQUENCY FN (AVG) [HZ]	DAMPING RATIO β (%)	PEAK DISPLACEMENT A 1' ABOVE GROUND SURFACE PRIOR TO RELEASE (LFB) [KN]	PEAK HORIZONTAL FORCE (LFB) [KN]	PEAK DISPLACEMENT A 1' ABOVE GROUND SURFACE AFTER RELEASE (IN) [MM]	PEAK COMPUTED ACCELERATION 1' ABOVE GROUND SURFACE (G)	PEAK MOMENT IN PILE (LBFl-IN) [KN-M]	COMMENTS
13	JULY 14	4	4.06	2.5	0.0098 [0.249]	—	0.0072 [0.183]	0.012	—	LARGE RING-DOWN; ROPE SNAPPED AT TRUCK
14	JULY 14	5	4.3	2.0	—	—	0.0018 [0.047]	0.003	—	SMALL RING-DOWN ; ROPE CUT
15	JULY 14	6	4.3	1.4	—	—	0.0017 [0.043]	0.003	—	SMALL RING-DOWN; ROPE CUT
16	JULY 14	7	4.2	2.8	0.0080 [0.203]	725 [0.203]	0.0045 [0.114]	0.008	4.4×10^4 [5.0]	LARGE RING-DOWN; ROPE SNAPPED IN MIDDLE

— = NOT COMPUTED

TS
CY SWEEP
CY SWEEP
G; INED
G; INED
QUENCY SWEEP
CY SWEEP
CY SWEEP
KING; INED
QUENCY SWEEP
CY SWEEP
KING; INED
QUENCY SWEEP; TAKEN

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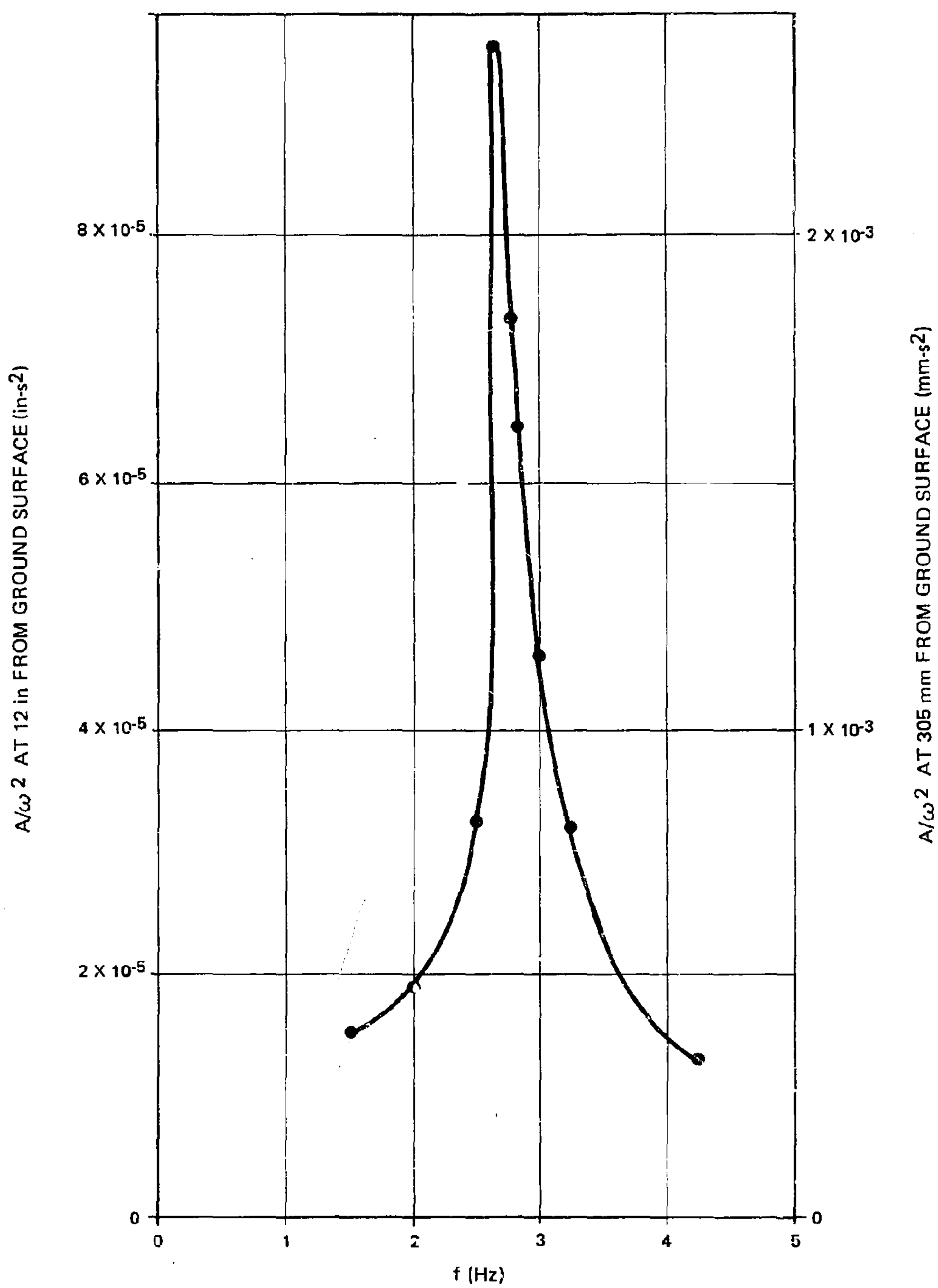
NSF PILE TEST

SUMMARY TESTS: FREQUENCY SWEEPS & RING-DOWN TESTS

8-81



f_1 - 2.62 Hz
β = 3.8%
PEAK LATERAL FORCE AT 1 Hz:
53 LBF (0.24 kN)

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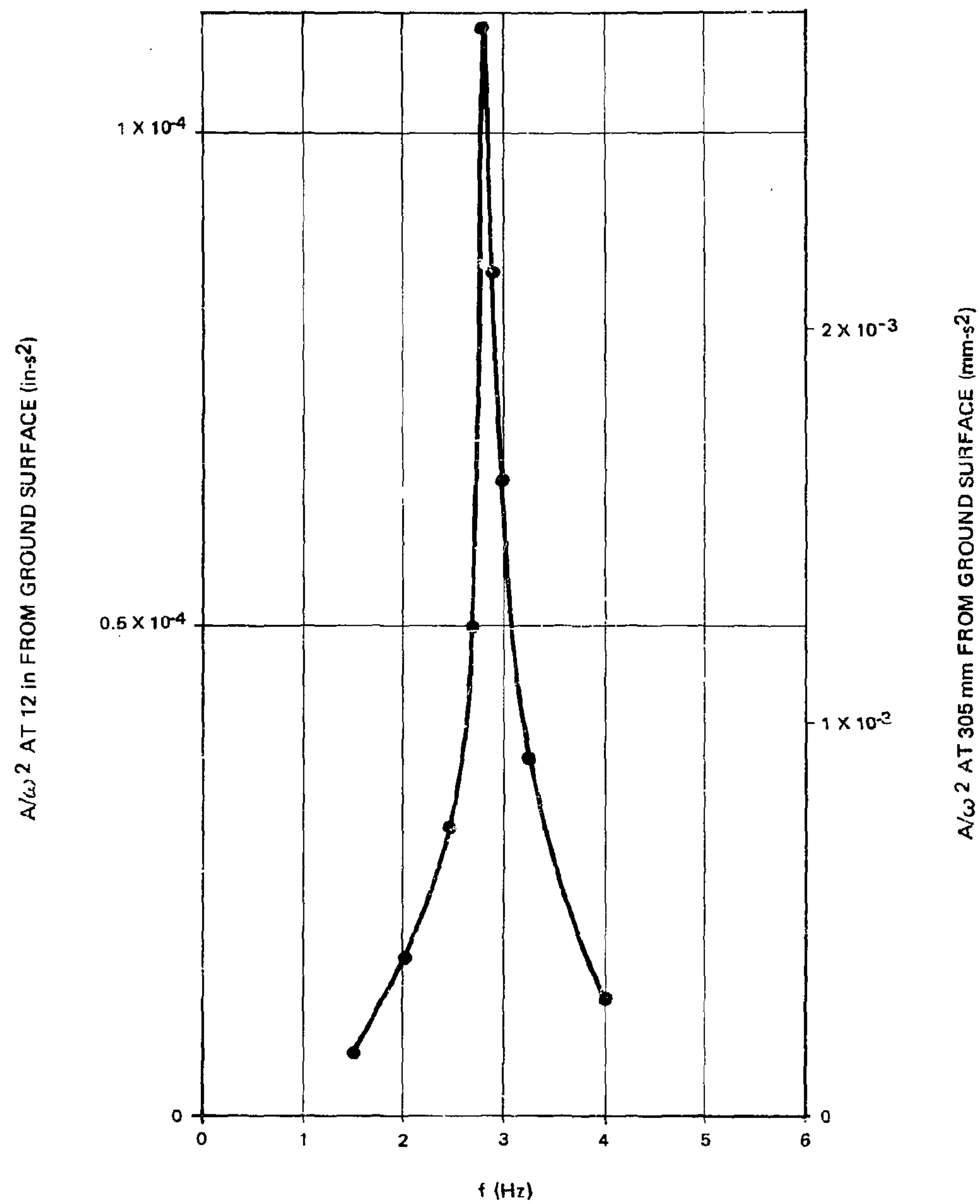
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 1
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81

FIGURE D.1

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______



f_1 = 2.80 Hz

β = 3.0%

PEAK LATERAL FORCE AT 1 Hz:
53 LBF (0.24 kN)

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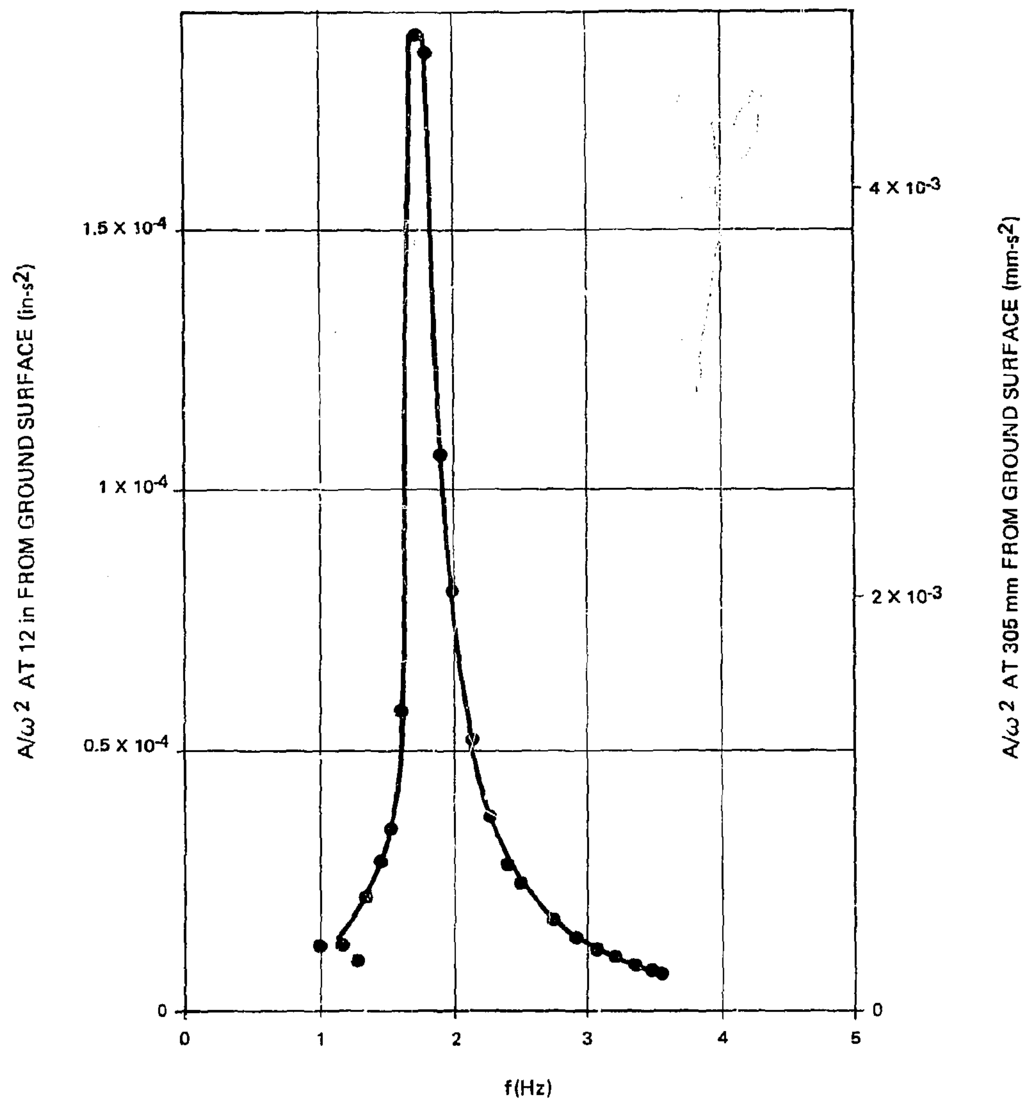
DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 2
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81 FIGURE D.2

Drawn by ____ / ____ Checked by ____ / ____ Approved by ____ /



f_1 = 1.72 Hz
β = 6.0%
PEAK LATERAL FORCE AT 1 Hz: 280 LBF (1.24 kN)

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PROJECT NO.: 79-243

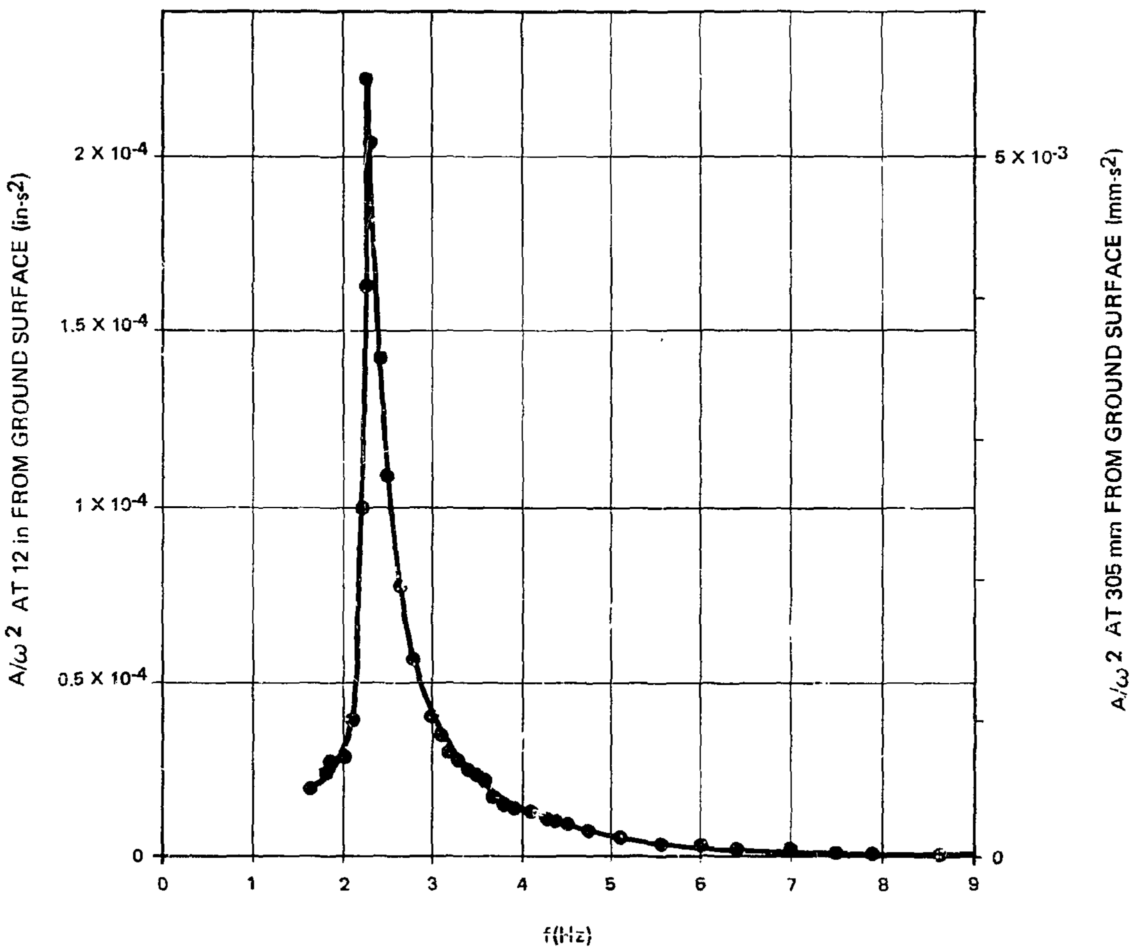
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 5
NORMALIZED TO 280 LBF (1.24 kN)
PEAK LATERAL FORCE

8-81

FIGURE D.3

Compiled by
Drawn by
Checked by
Approved by



$f_1 = 2.27$ Hz

$\beta = 3\%$

PEAK LATERAL FORCE AT 1 Hz: 53 LBF (0.24 kN)

Ertec
The Earth Technology Corporation

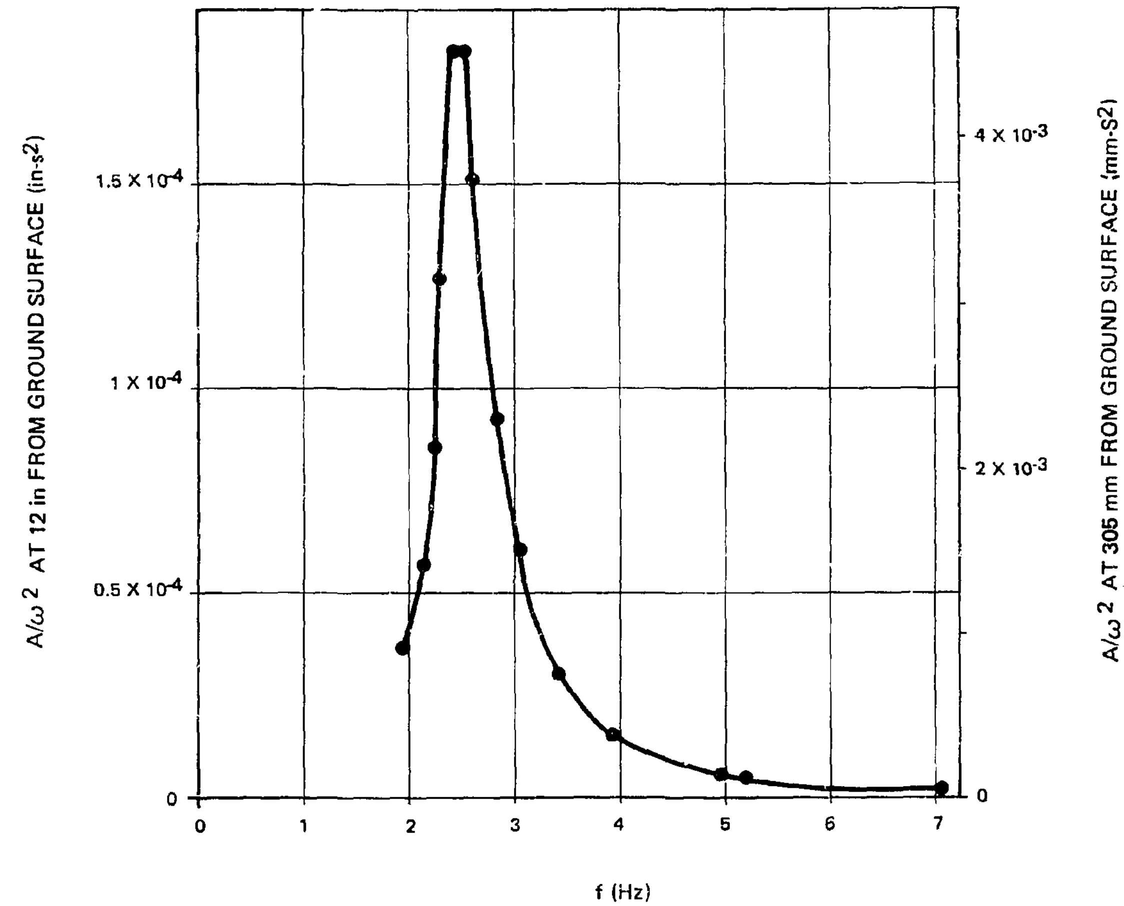
PROJECT NO.: 79-243

NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 6
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81 FIGURE D.4

Compiled by Drawn by Checked by Approved by



$f_1 = 2.50$ Hz
$\beta = 7\%$
PEAK LATERAL FORCE AT 1 Hz: 53 LBF (0.24 kN)

Ertec
The Earth Technology Corporation

PROJECT NO.: 79-243

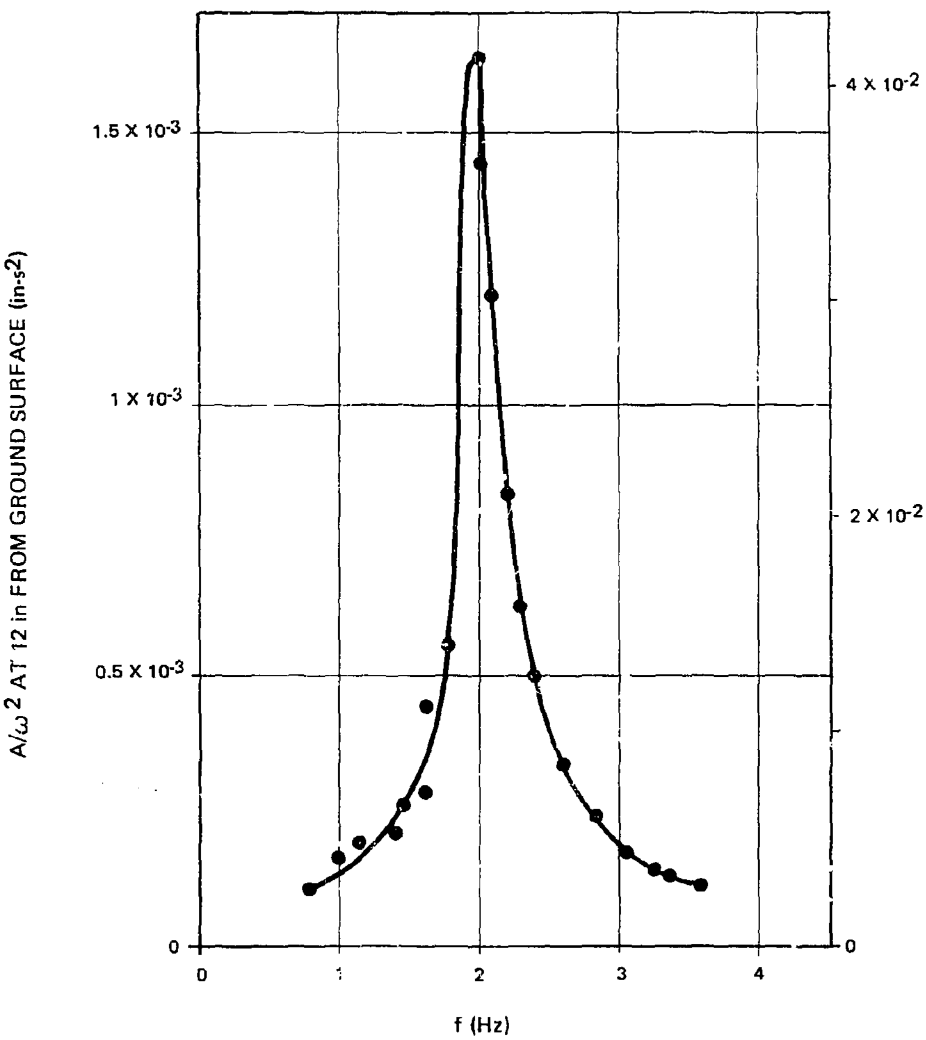
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 7
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81

FIGURE D.5

Compiled by ____ Drawn by ____ Checked by ____ Approved by ____



$f_1 = 2.01$ Hz
$\beta = 5.5\%$
PEAK LATERAL FORCE AT 1 Hz: 367 LBF (1.63 kN)

Ertec
The Earth Technology Corporation

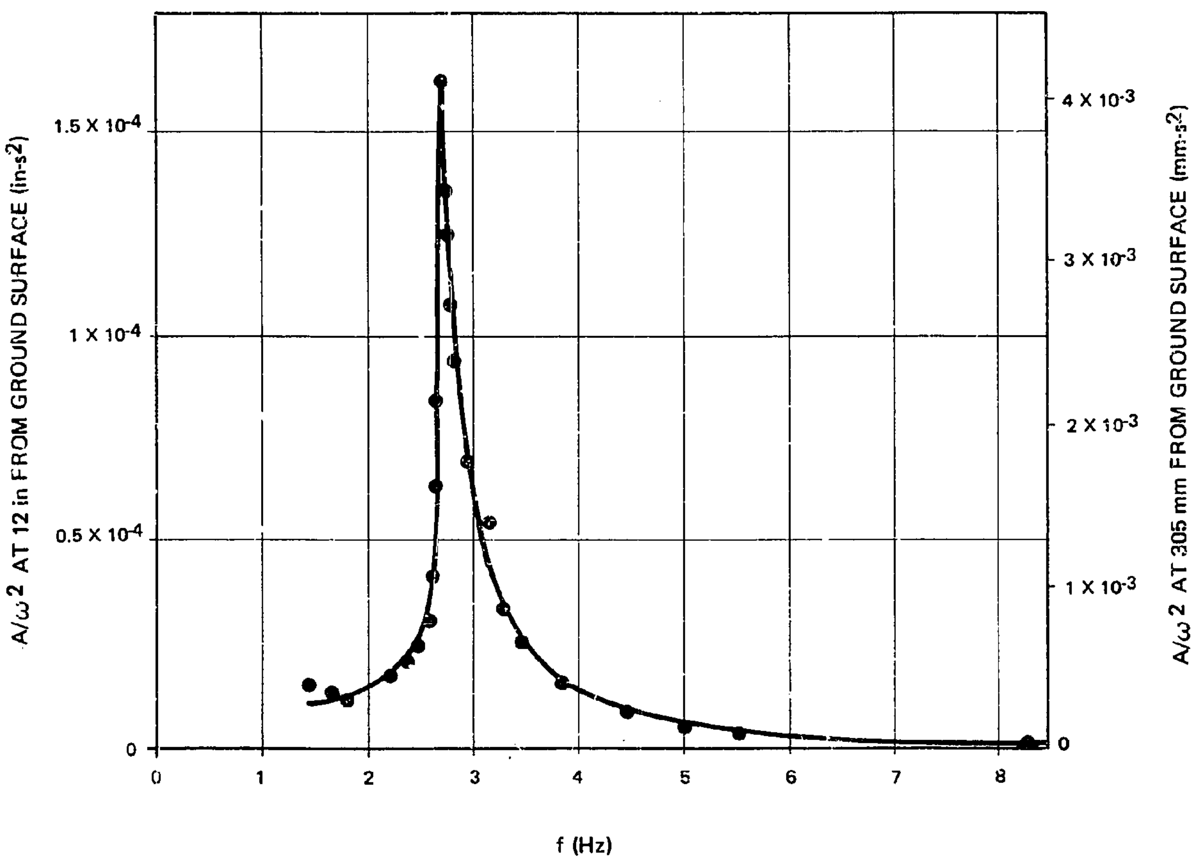
PROJECT NO.: 79-243

NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 9
NORMALIZED TO 367 LBF (1.63 kN)
PEAK LATERAL FORCE

8-81 FIGURE D.6

Compiled by ____ / Drawn by ____ / Checked by ____ / Approved by ____



f_1 = 2.67 Hz
β = 1.5%
PEAK LATERAL FORCE AT 1 Hz: 53 LBF (0.24 kN)

Compiled by ____ / Drawn by ____ / Checked by ____ / Approved by ____ /

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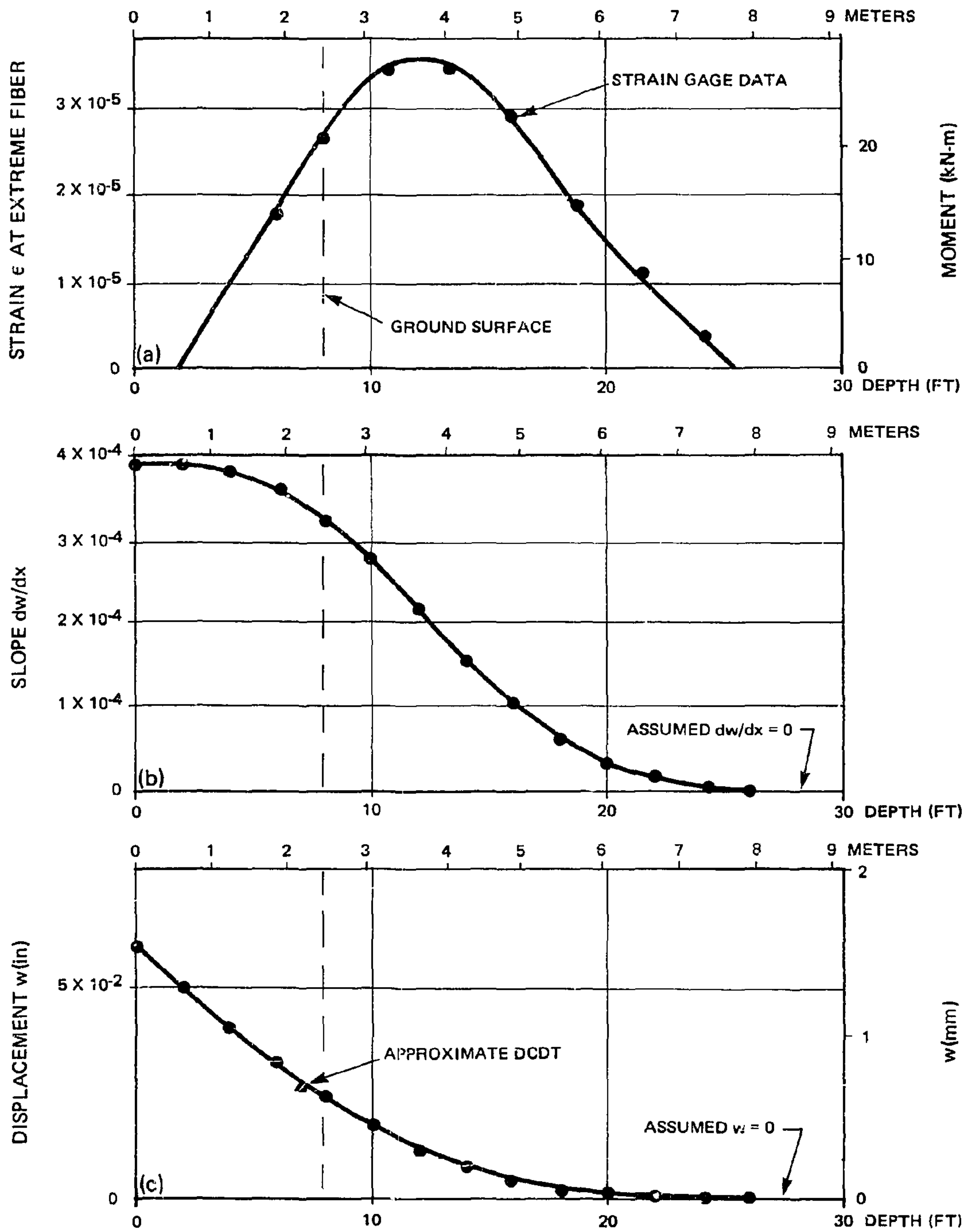
NSF PILE TEST

DISPLACEMENT RESPONSE CURVE
FOR TEST NO. 10
NORMALIZED TO 53 LBF (0.24 kN)
PEAK LATERAL FORCE

8-81

FIGURE D.7



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PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH FOR TEST NO. 1 AT RESONANCE, $f_1 = 2.62$ Hz

8-81

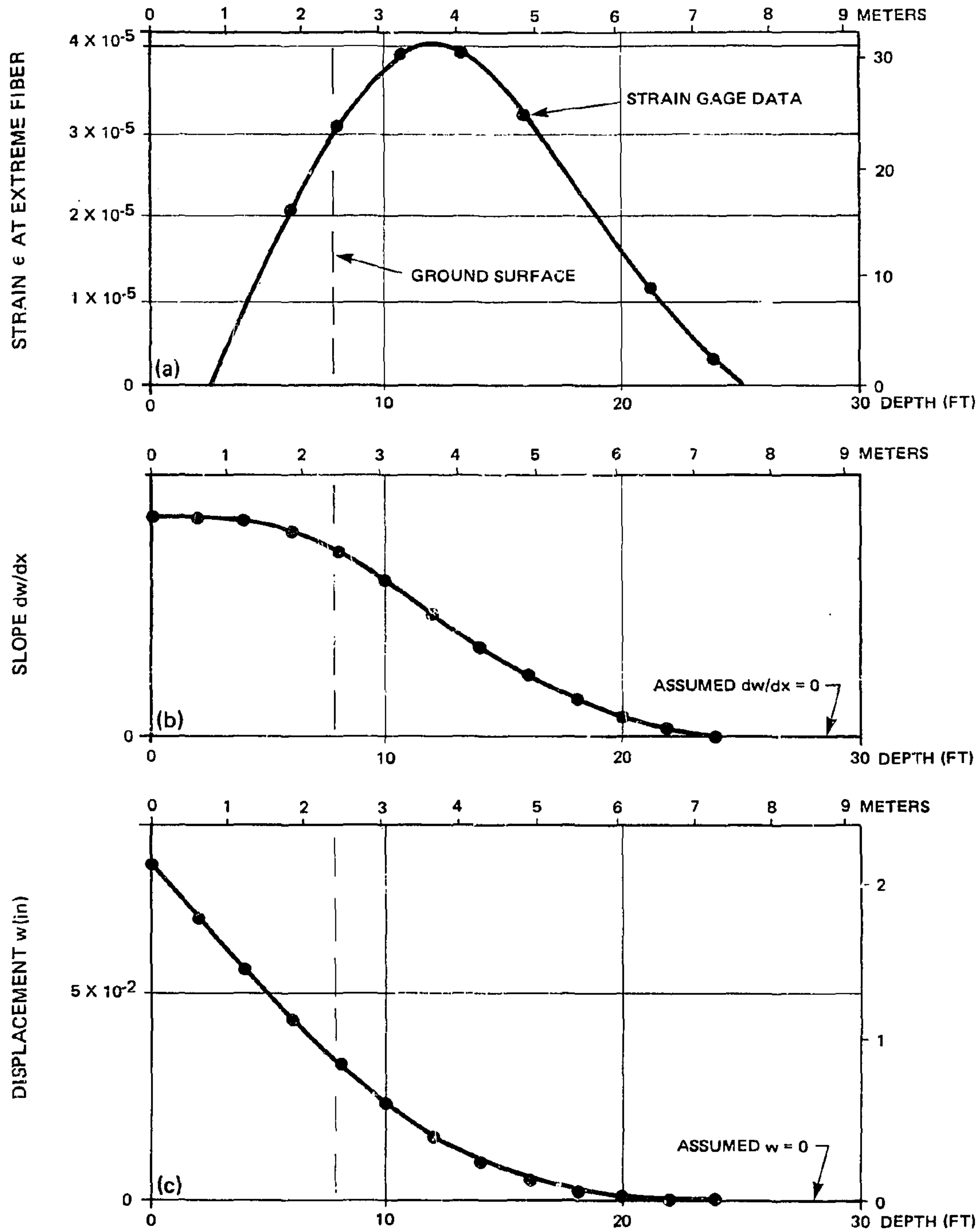
FIGURE D.8

Approved by

Checked by

Drawn by

Checked by



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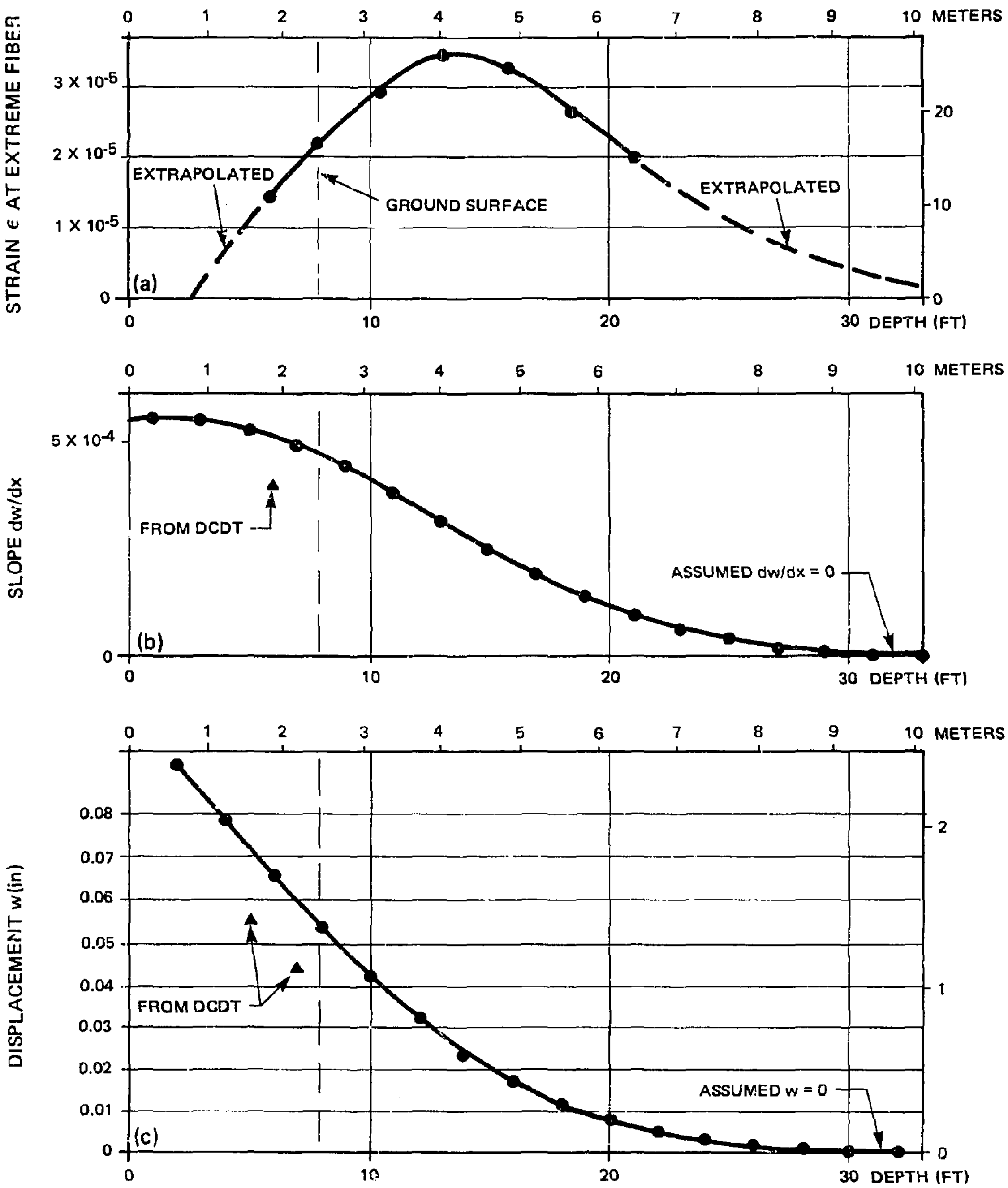
NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH FOR TEST NO. 2 AT RESONANCE, f_1 = 2.80 Hz

8-81

FIGURE D.9



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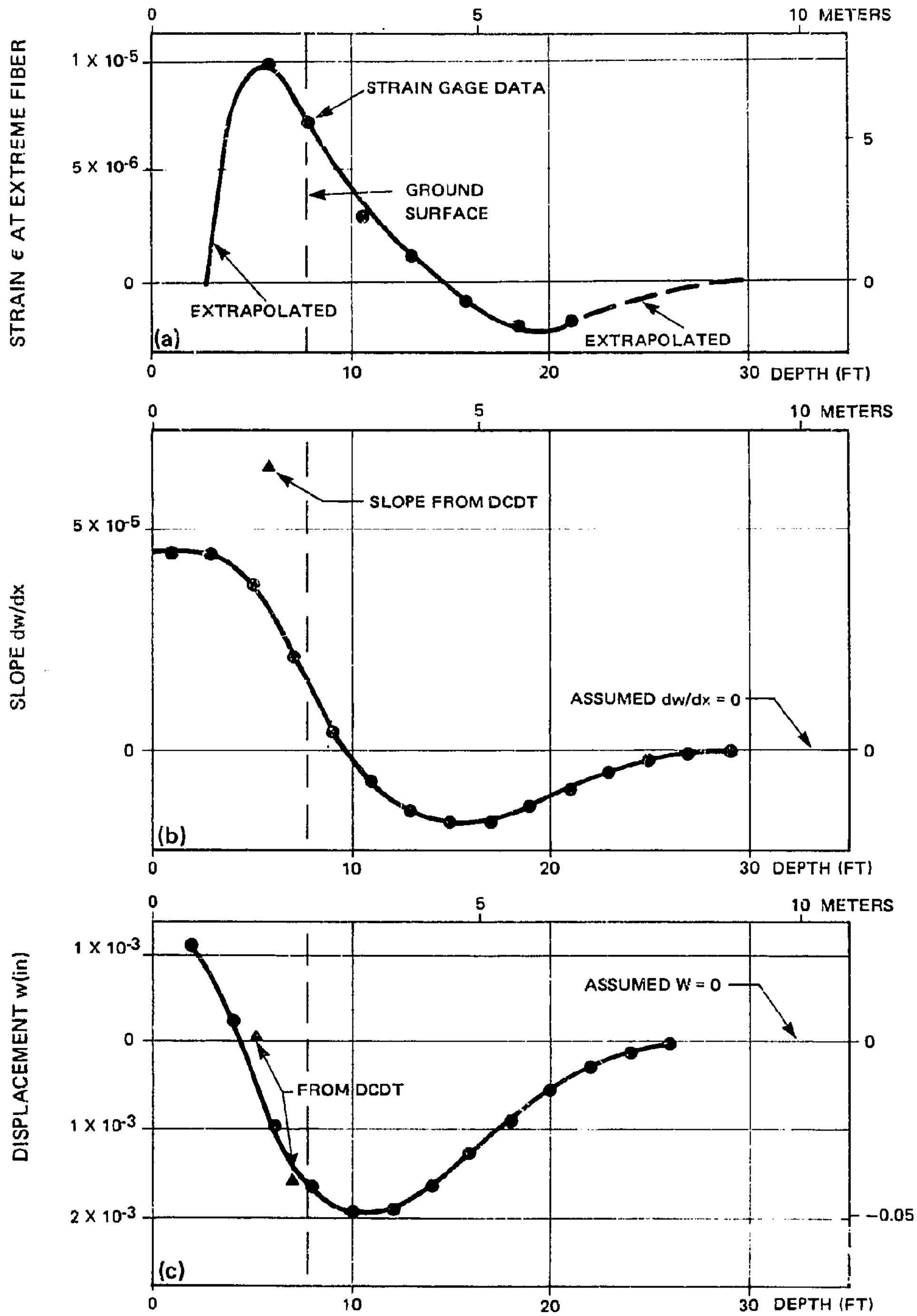
NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH FOR TEST NO. 6 AT RESONANCE, f_1 = 2.27 Hz

8-81

FIGURE D.10



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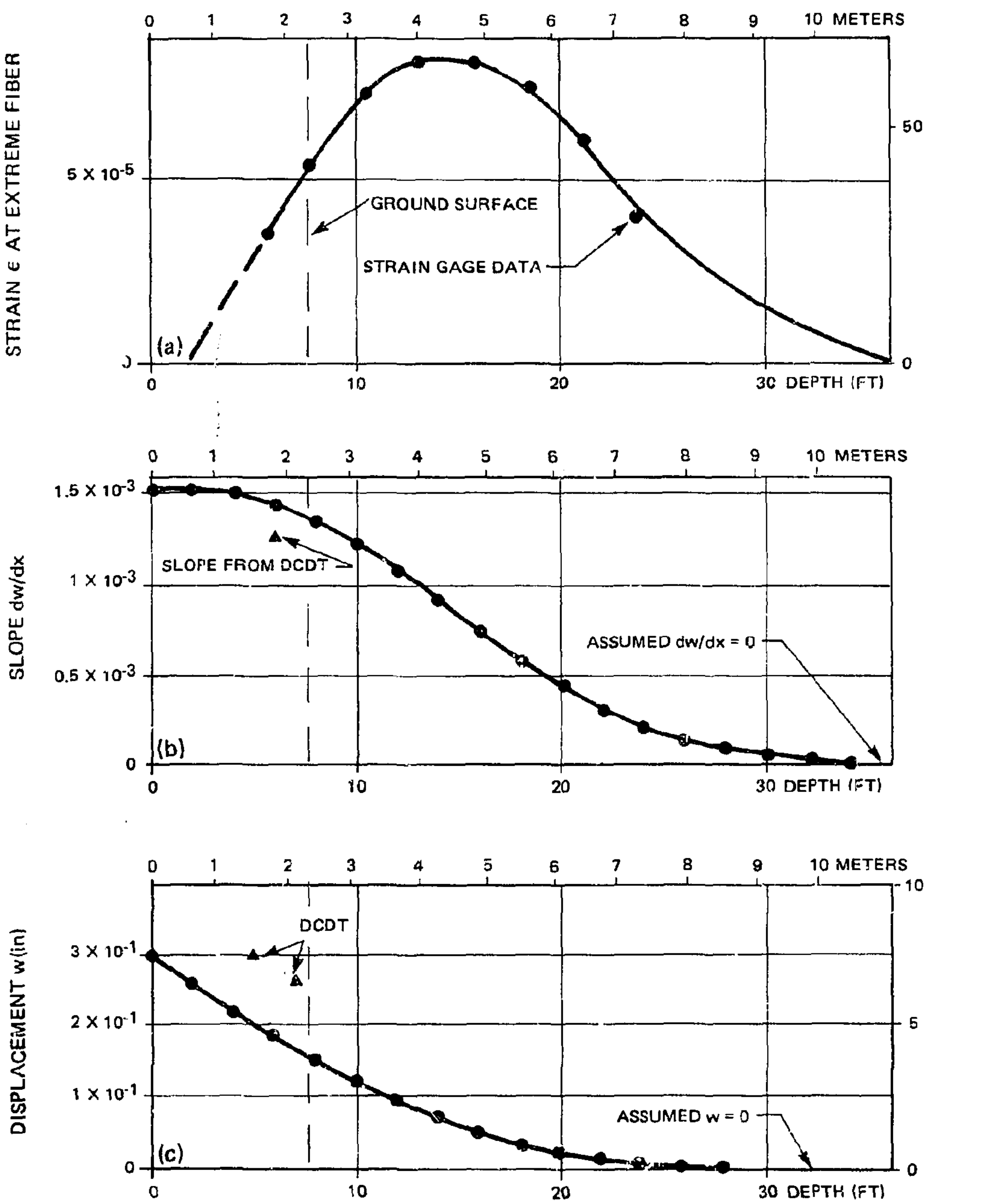
NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT PROFILES FOR TEST NO. 6 AT f = 8.62 Hz (NEAR 2ND MODE)

8-81

FIGURE D.11



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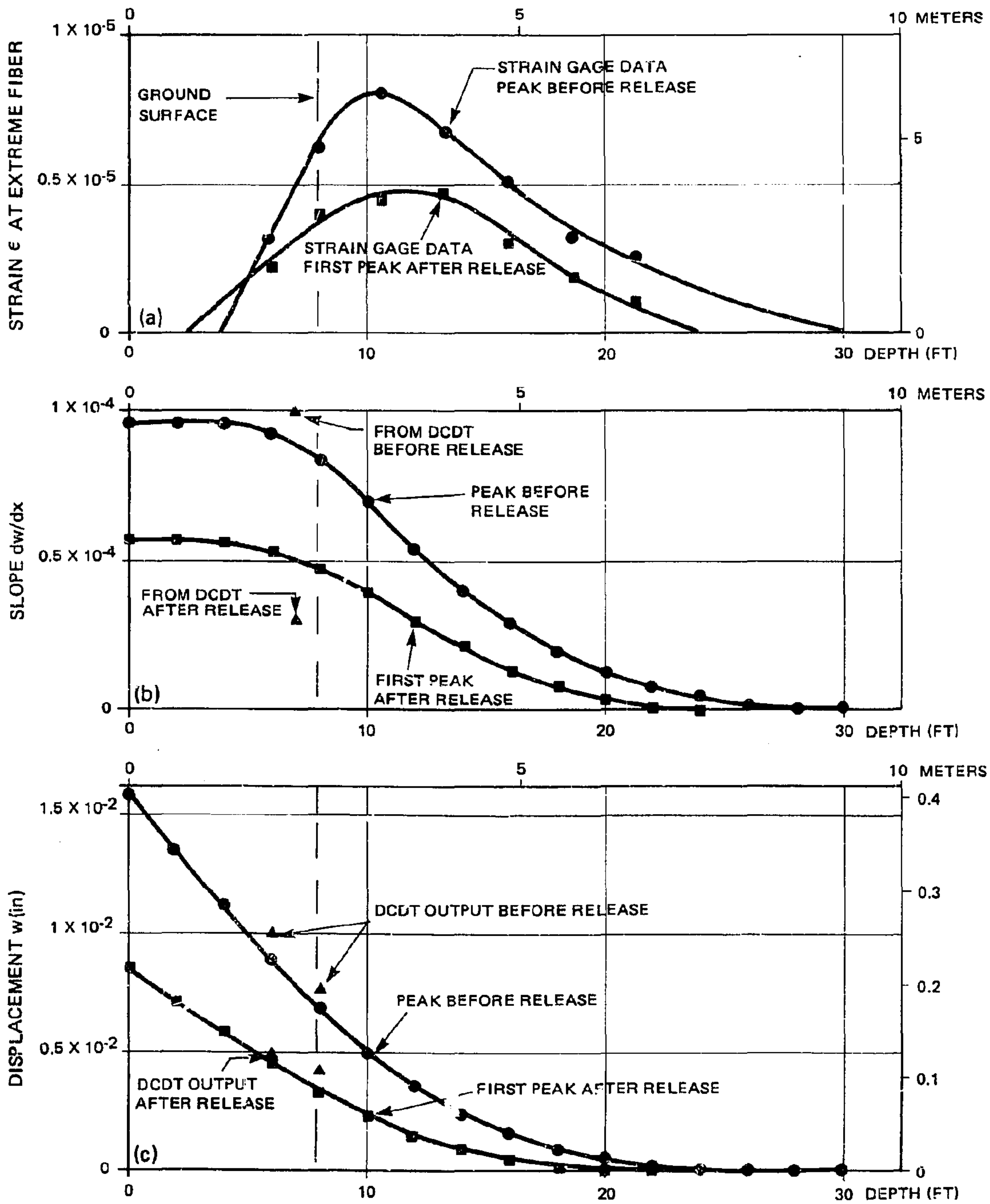
NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH FOR TEST NO. 9 DURING RESONANCE, $f_1 = 2.01$ Hz

8-81

FIGURE D.12



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NSF PILE TEST

PEAK STRAIN, SLOPE AND LATERAL DISPLACEMENT OF PILE VS DEPTH DURING RING-DOWN TEST NO. 16

8-81

FIGURE D.13

APPENDIX E

EQUIPMENT RECOMMENDATIONS

APPENDIX E

EQUIPMENT RECOMMENDATIONS

1. More horsepower and a higher frequency range is needed in the shaking machines.

2. The offset and related sensitivity problem on the pore pressure transducers needs to be solved.

3. The strain gages functioned well, but one or two gages a few feet further down the pile would be useful in the analysis.

4. Shorter stroke DCDT's would be adequate.

5. An effort should be made to calibrate the Ranger seismometers, so that they can be used for absolute readings.

6. The lead weights need to be strapped down more effectively.

7. The weight platform is too flexible.

8. More effort is required to make better pore pressure standpipes.

9. If only vibration peaks are required, a digital recording system is not necessary.