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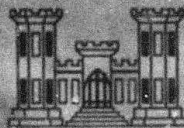


TECHNICAL REPORT S-72-5

AN EVALUATION OF AN EXISTING
PROCEDURE FOR DETERMINING SHEAR
MODULI AT DEPTHS BY IN SITU
VIBRATORY TECHNIQUE

by

J. Fowler



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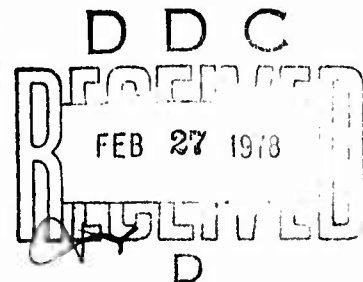
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FOREWORD

The investigation described herein was performed by the U. S. Army Engineer Waterways Experiment Station (WES) for the Office, Chief of Engineers, in accordance with the Instructions and Outlines for Development of Engineering Criteria for FY66 - FY68. This report describes the tests conducted at WES during March and April 1966.

Engineers of the Soil Dynamics Branch, WES, actively engaged in data collection, analysis, and report phases of this study were Messrs. R. W. Cunny, Z. B. Fry, R. F. Ballard, Jr., D. R. Casagrande and J. Fowler. The work was done under the general supervision of Mr. W. J. Turnbull, Chief (retired), and J. P. Sale, Chief, and Mr. A. A. Maxwell (deceased), Assistant Chief, and R. G. Ahlvin, Assistant Chief, Soils Division.

The report was prepared by Mr. Jack Fowler, and is essentially a thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering to the Mississippi State University, State College, Mississippi.

Directors of the Waterways Experiment Station during the preparation and publication of this report were COL John R. Oswalt, CE, COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Directors were Mr. J. B. Tiffany and Mr. F. R. Brown.

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ACKNOWLEDGEMENTS

The study presented in this thesis is an evaluation of the use of surface-induced vibrations to determine the dynamic soil properties at depth. This investigation was performed at the Waterways Experiment Station (WES) for the Office, Chief of Engineers, in accordance with the Instructions and Outlines for the Development of Engineering Criteria for FY 66 - FY 68. The author is grateful to the personnel and supervisors of the WES for allowing him to perform this study, as well as for their guidance and pertinent comments. Particular appreciation is expressed to Messrs. R. W. Cunny, Z. B. Fry, Jr., R. F. Ballard, Jr., and D. R. Casagrande. The author extends deep appreciation to Mr. H. C. Greer, III, Electrical Engineer, for his suggestions and assistance in assembling and operating the electronic instrumentation used in this study and to Mrs. Dorothy Booth and SP4 R. G. Guernsey for data reduction and preparation of report material. In addition, the writer extends his gratitude to his faculty advisor, Professor R. M. Scholtes, and the remainder of his graduate committee including Professor M. B. Clisby and Assistant Professor E. Springer. The work of Mrs. Gwendolyn W. Reid in assisting with the style and format and typing of the manuscript was invaluable. Appreciation is also extended to Mrs. Modell Holloway for her assistance in typing.

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CHAPTER I

INTRODUCTION

1.1 Background

The investigation described herein was directed toward evaluation of the use of dynamic test procedures for the determination of dynamic in situ soil characteristics - specifically to evaluate the concept that the velocity determined from propagation of a surface wave is applicable to a depth of equal to one half of the wavelength. The U. S. Army Engineer Waterways Experiment Station (WES) has conducted numerous foundation investigations using dynamic techniques to determine the characteristics of subsurface materials. The details of the dynamic test procedure are presented in Appendix A. Subsequently, the procedure was incorporated in Engineer Manual 1110-345-310, "Engineering and Design, Foundations Subjected to Vibratory Loads." This manual is used by all elements of the Corps of Engineers responsible for the design of military structures subjected to dynamic loads.

1.2 Purpose and Scope of Investigation

The purpose of this investigation was to determine the validity of the procedure described in Appendix A. Tests were conducted in an area where soil characteristics were considered to be uniform in both the horizontal and vertical planes. The tests included the determination of phase velocity between a controlled vibratory source and selected points on the surface and at various depths for a range of force and frequency input. In addition, the amplitude of motion

(particle velocity) and wave-shape characteristics were determined at the selected distances and depths for measurements made in vertical, radial, and transverse directions. The data obtained were compared to results of dynamic in situ tests conducted in the same area and with theoretical dispersion or attenuation of surface waves with distance and depth in a homogeneous medium to evaluate the concept of velocity determined from the surface wave being applicable to a depth equal to one half of the wavelength.

CHAPTER II

PRELIMINARY INVESTIGATION

2.1 Theoretical Basis

A solid that is in equilibrium generally requires 36 elastic constants to describe the stress-strain relations. For an isotropic body, the number of elastic constants reduces to two, and these may be taken to be the Lamé constants λ and G , or the shear and compression moduli E and G , or even the shear-wave and compression-wave v_s and v_p . Assuming that the ground at the WES test site is adequately represented by an isotropic elastic solid, and one of the purposes of this work is to determine E and G as a function of depth by a vibratory technique.

The two elastic parameters describe the linear relation between shear stress and shear strain and between pressure and simple compression. The existence of two independent elastic constants implies that for an infinite isotropic elastic solid, there are two fundamental and independent types of wave motion called shear waves and compression waves. Any wave motion is associated with a time-dependent displacement of the particles of the solid. A wave for which the particle displacement is in the tangent plane of the wave front is called a shear wave or S-wave. Historically the S refers to "secondary" arrivals on seismograph records of earthquakes. When the time-dependent displacement is perpendicular to the wave front, the waves are called compression waves or P-waves. The designation P refers to "primary" arrivals on earthquake seismograph records.

S-waves are not propagated in liquids because liquids do not sustain a shearing strain, while P-waves are propagated in liquids and solids.

The phase velocity of S- and P-waves is given by:

$$v_s = \sqrt{\frac{G}{\rho}} \quad (1)$$

$$v_p = \sqrt{\frac{\lambda + 2G}{\rho}} \quad (2)$$

where

ρ = mass density of soil

These expressions are only valid for infinite elastic media. If free surfaces are present, such as in the case of rods or plates, then different expressions for the P-wave velocity must be used.

When a free surface is present on a large elastic body, it is possible to have another kind of elastic wave propagation that has characteristics of both P- and S-waves (mainly S-waves) but which has the property of attenuating with depth and propagating along the surface. P- and S-waves propagate downward as well as in a horizontal direction. Surface waves with a vertical and a radial particle displacement producing a retrograde elliptical motion in time are called Rayleigh waves; while surface waves with a particle displacement that is transverse to the direction of propagation are called Love waves. Only Rayleigh waves will be discussed in this thesis.

If the ground is homogeneous, the phase velocity of the Rayleigh waves is independent of frequency and depends only on the elastic

constants of the ground. For example if Poisson's ratio ν has the value of $1/4$, then the Rayleigh wave velocity v_R equals $0.92 v_s$. It can be shown that in general $v_R \lesssim v_s$. If the elastic half-space is nonhomogeneous, the Rayleigh wave phase velocity is a function of frequency. This phenomenon of the velocity depending on the frequency is called dispersion, and the material in question is said to be dispersive. If the phase velocity is an increasing function of the wavelength, the situation is called normal dispersion. Such a situation occurs when the shear modulus is an increasing function of depth in an elastic half-space. If the shear modulus decreases with depth (such as in the case of a highway pavement overlying a soil base), the Rayleigh wave phase velocity is a decreasing function of wavelength and is referred to as the case of anomalous dispersion. The WES test site is a good example of normal dispersion. The calculation of a dispersion curve giving $v_R(\lambda)$ involves the solution of the wave equation, which attenuates with depth and which satisfies the boundary conditions of zero stress at the free surface. This calculation can be quite involved for nonhomogeneous media. For homogeneous media, theory shows that the Rayleigh wave phase velocity is always less than the shear-wave velocity; in fact, $0.875 v_s < v_R < 0.955 v_s$ for the full variation of Poisson's ratio $0 < \nu < 1/2$. Experimental data taken by the WES shows that for the case of normal dispersion, the shape of the dispersion curve is very similar to that of shear-wave velocity versus depth curve. The possibility, therefore, arises that for the

case of normal dispersion, it may be possible to reproduce the shear-wave velocity versus depth curve from the dispersion curve by taking $v_R \sim v_S$ and assuming the depth to be a simple multiple of the wavelength. Empirically, we find that such a factor does exist and has the value $1/2$, so that depth = $1/2$ wavelength may be taken as an experimentally verified approximation. In this way, the shear modulus is determined as a function of depth from an experimentally determined dispersion curve.

2.2 Location of Tests

The site selected for the investigation was a level area on the WES reservation where the turf and top soil had been removed for previous studies (figure 1). Although considerable information was available on general characteristics of the material at the site, a more detailed investigation was deemed necessary to better define the area. Borings were made in the area to obtain samples for both conventional and dynamic laboratory tests. In addition, refraction seismic and vibratory traverses were conducted at the site in accordance with the procedures presented in Appendix A. Details and results of the conventional soils tests conducted are presented in Appendix B; the dynamic laboratory and field tests are reported in Appendix C.

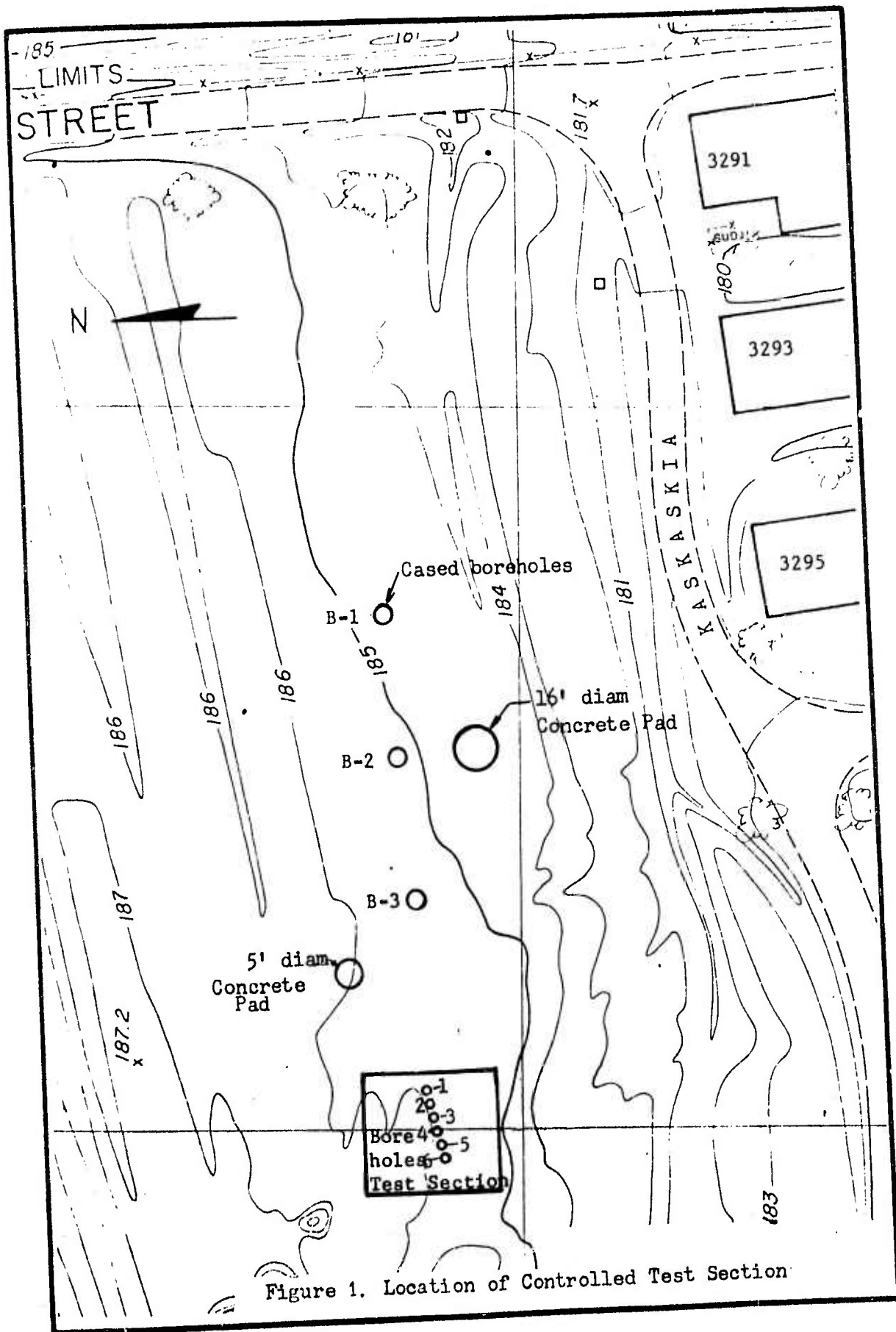


Figure 1. Location of Controlled Test Section

CHAPTER III

TESTS CONDUCTED

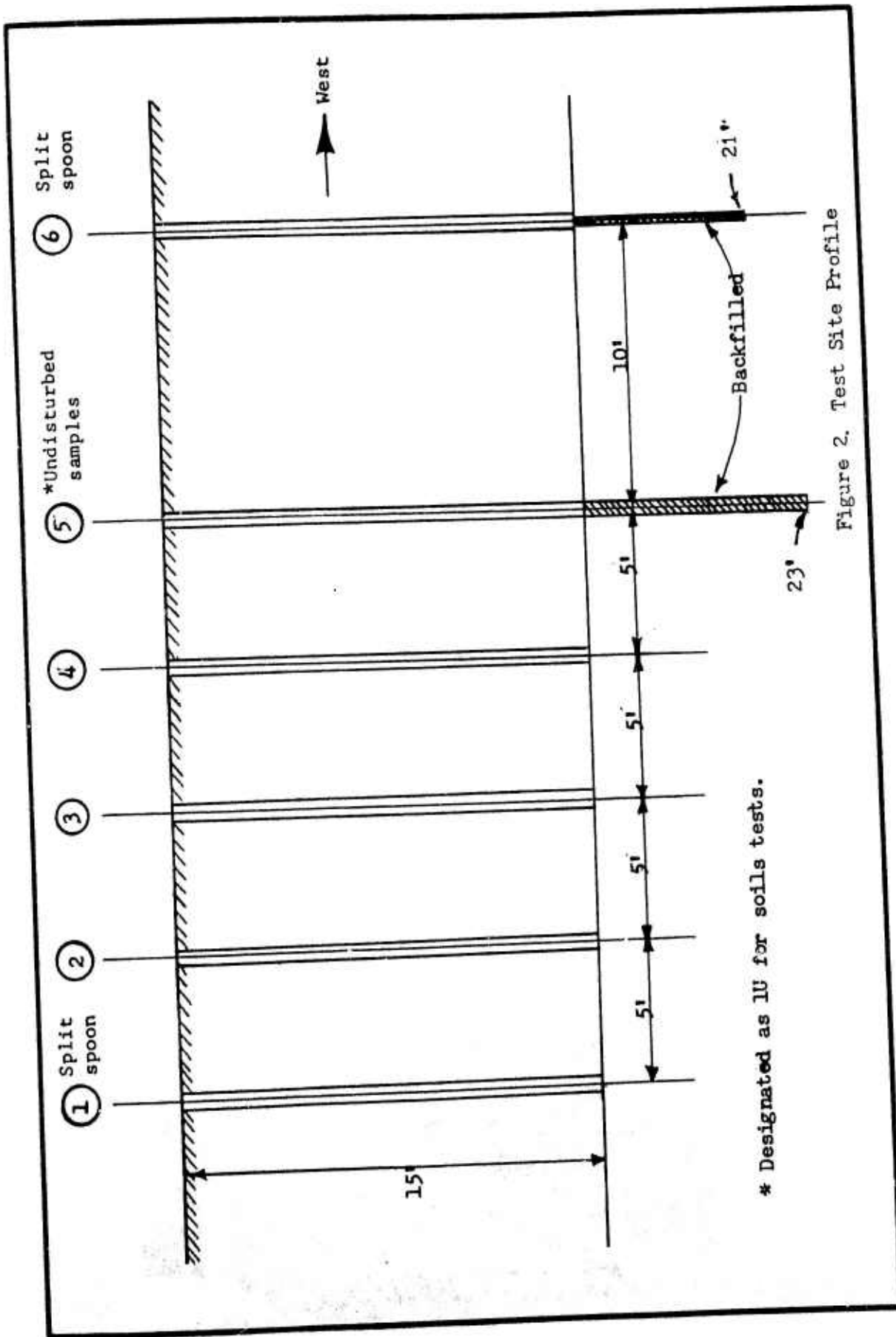
3.1 Test Plan

After an extensive examination of the properties of the soil in the test area by conventional field exploration and laboratory tests, as well as the results of field seismic and vibratory testing, a test plan was developed. The test plan required five 6-in.-diam holes 15 ft deep equally spaced 5 ft apart oriented on an east-west line. Two of the original boreholes drilled for soil testing were used and three additional holes were placed as shown in figure 2 (holes 1-5). The plan called for six three-directional velocity-type transducers to be placed on the surface and at depths for various horizontal and vertical distances from a vibratory source located on the ground surface.

Wave propagation was to be induced by a vibratory source located on the ground surface, and the response of the velocity transducers was to be recorded for a wide range of frequencies and force outputs. The vibrator utilized during these tests was an eccentric rotating-mass vibrator capable of producing both vertical and torsional forces (figure 3). Table 1 shows the frequencies and forces generated by the vibrator in the vertical and torsional modes.

3.2 Tests Performed and Instrumentation

3.2.1 Tests. The instrumentation recording capabilities were limited to 19 data channels; therefore, the tests were separated into groups A, B, and C. The first test group, group A, consisted of



* Designated as LU for soils tests.

Figure 2. Test Site Profile

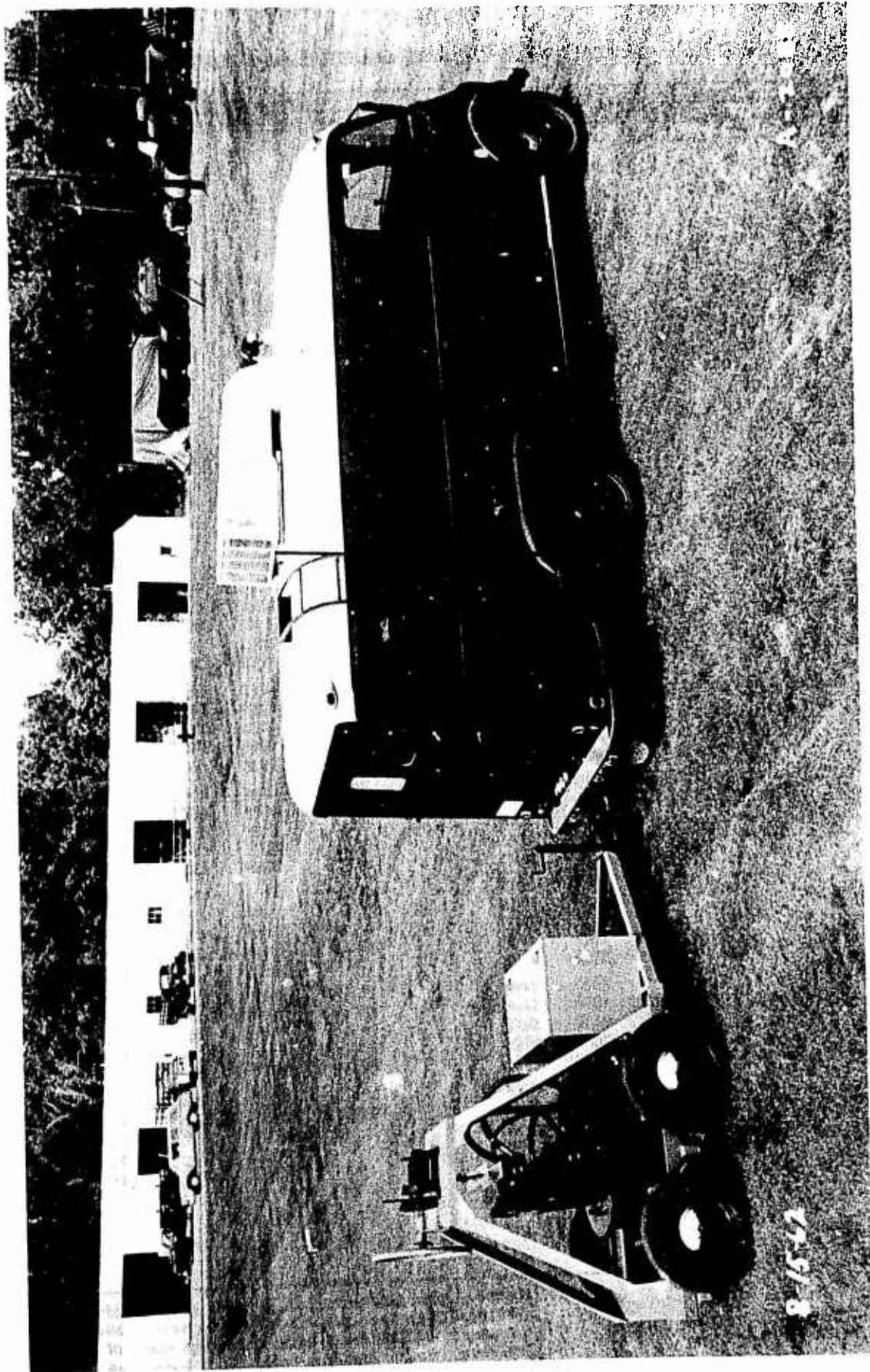


Figure 3. Eccentric Rotating Mass Vibrator and Instrumentation Van

Table 1

Torsional and Vertical Forces Generated by Variable-Mass Hydraulic Vibrator

Fre- quency Hz	Torsional Mode*					Vertical Mode*				
	Total Wt of Eccentric Masses, lb**					Total Wt of Eccentric Masses, lb**				
	5.00	9.60	14.48	19.00	40.00	5.00	9.60	14.48	19.00	40.00
1	1.2	2.3	3.5	4.6	9.7	2.1	4.0	6.0	7.9	16.6
2	4.8	8.1	14.0	18.4	38.7	8.3	13.9	24.0	31.5	66.2
3	10.7	20.6	31.1	40.7	85.8	18.4	35.3	53.2	69.8	147
4	19.0	36.6	55.1	72.4	152	32.6	62.6	94.4	124	261
5	29.8	57.2	86.4	113	238	51.0	97.9	148	194	408
6	42	82	124	162	343	73.5	141	213	279	588
7	58	112	169	221	467	100	192	290	380	801
8	76	146	220	289	609	130	250	378	496	1,043
9	96	185	279	366	773	165	317	478	627	1,321
10	119	228	345	453	955	204	392	591	776	1,634
11	144	277	418	546	1,153	247	475	716	939	1,977
12	171	330	497	653	1,373	294	565	852	1,118	2,354
13	201	386	582	766	1,610	345	662	998	1,310	2,758
14	233	448	676	888	1,868	400	768	1,158	1,519	3,199
15	268	515	778	1,020	2,146	459	882	1,330	1,746	3,675
16	305	586	885	1,160	2,442	523	1,004	1,514	1,986	4,182
17	344	662	1,000	1,310	2,757	590	1,134	1,709	2,243	4,722
18	386	742	1,120	1,468	3,088	661	1,269	1,914	2,512	5,288
19	429	827	1,245	1,634	3,440	736	1,414	2,133	2,799	5,892
20	476	916	1,380	1,812	3,814	816	1,568	2,364	3,103	6,532
21	526	1,010	1,525	1,998	4,207	901	1,730	2,608	3,422	7,205
22	577	1,110	1,671	2,193	4,618	989	1,899	2,863	3,756	7,909
23	630	1,210	1,825	2,394	5,042	1,079	2,072	3,125	4,101	8,634
24	686	1,320	1,987	2,608	5,491	1,175	2,257	3,404	4,467	9,404
25	745	1,430	2,157	2,831	5,960	1,276	2,450	3,694	4,848	10,207
26	806	1,550	2,334	3,062	6,447	1,380	2,651	3,997	5,244	
27	870	1,670	2,518	3,304	6,957	1,489	2,860	4,312	5,658	
28	935	1,794	2,705	3,550	7,474	1,600	3,072	4,633	6,079	
29	1,000	1,924	2,902	3,808	8,018	1,716	3,296	4,970	6,522	
30	1,070	2,060	3,107	4,077	8,584	1,837	3,529	5,321	6,982	
31	1,145	2,200	3,318	4,354	9,167	1,962	3,768	5,682	7,456	
32	1,220	2,345	3,536	4,640	9,769	2,091	4,016	6,055	7,946	
33	1,300	2,491	3,757	4,930	10,381	2,222	4,267	6,434	8,443	
34	1,380	2,646	3,989	5,235		2,359	4,531	6,832	8,965	
35	1,460	2,804	4,228	5,548		2,500	4,802	7,241	9,501	
36	1,550	2,967	4,474	5,871		2,646	5,081	7,662	10,054	
37	1,633	3,134	4,727	6,203		2,796	5,368	8,095		
38	1,721	3,304	4,983	6,539		2,947	5,659	8,533		
39	1,812	3,481	5,249	6,888		3,104	5,962	8,989		
40	1,907	3,662	5,522	7,247		3,266	6,272	9,457		
41	2,013	3,832	5,807	7,627		3,447	6,563	9,944		
42	2,112	4,022	6,094	8,003		3,617	6,887	10,435		
43	2,213	4,215	6,387	8,389		3,791	7,219	10,938		
44	2,318	4,413	6,687	8,783		3,970	7,558			
45	2,424	4,617	6,995	9,188		4,152	7,906			
46	2,533	4,824	7,309	9,600		4,339	8,261			
47	2,645	5,036	7,631	10,022		4,530	8,624			
48	2,759	5,253	7,959			4,725	8,995			
49	2,875	5,474	8,294			4,923	9,374			
50	2,993	5,700	8,636			5,126	9,761			

* Vertical mode - force, lb; torsional mode - torque, ft-lb.
 ** Eccentricity of the unbalanced mass is 4.0 inches.
 Note: Maximum operating force level 10,000 pound.
 Recommend 5,000 pound for continuous operation.

surface measurements made on a line away from the vibrator at constant vibratory forces and various frequencies. Group A was divided into two series with the vibrator operated in the vertical and torsional modes for series I and II, respectively. Group B consisted of series III through VIII with vertical and torsional tests conducted for each series. The transducers were positioned at different locations and elevations so that complete coverage of the borehole profile could be obtained for frequencies ranging from 10 to 50 Hz. The vibrator was operated at various locations. Group C was similar to the series of tests conducted for group A except that the transducers were equally spaced in one borehole instead of on the surface. The vibrator was operated over a wide range of frequencies and forces. The tabulation below shows the plan of tests.

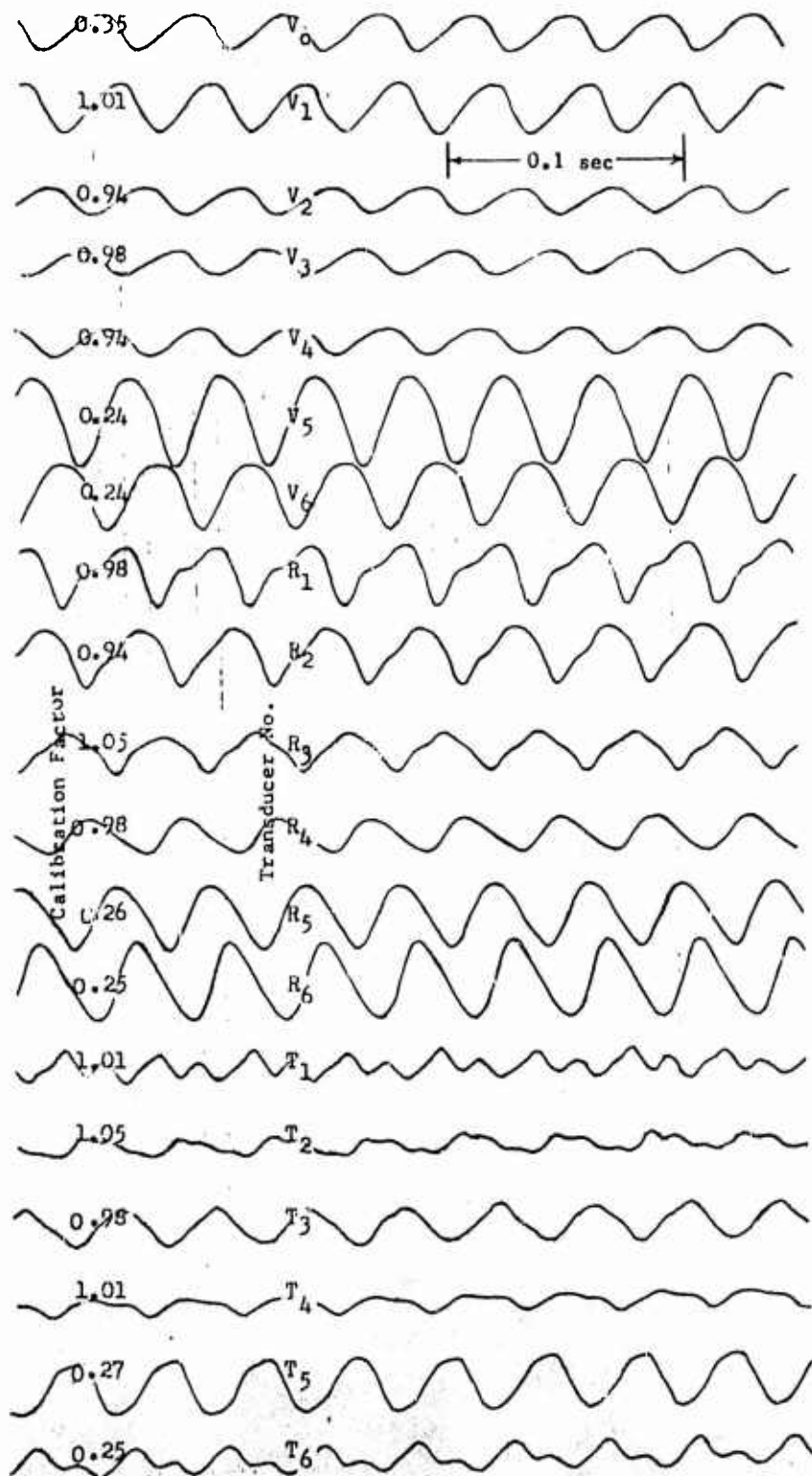
Group	Description of Tests	Series	
		No.	Mode of Vibration
A	Surface measurements	I	Vertical
		II	Torsional
B	Subsurface measurements with transducers at various depths:		
	Major transducers on surface and at 15-ft depth	III	Vertical and Torsional
	Major transducers at 15-ft depth	IV	Vertical and Torsional
	Major transducers at 12-ft depth	V	Vertical and Torsional
	Major transducers at 9-ft depth	VI	Vertical and Torsional
	Major transducers at 6-ft depth	VII	Vertical and Torsional
	Major transducers at 3-ft depth	VIII	Vertical and Torsional

<u>Group</u>	<u>Description of Tests</u>	<u>Series No.</u>	<u>Mode of Vibration</u>
C	Subsurface measurements with major transducers at various depths in hole 1. Vibrator 10 ft from hole 1.	IX	Vertical and Torsional
	Vibrator over hole 1.	X	Vertical and Torsional

3.2.2 Instrumentation. A thirty-six channel, direct-readout oscillograph recorder was used to record the signal from the velocity-type transducers used during the tests. The signals were amplified with DC amplifiers and terminated at the recorder. After the vibrator frequency and force level were set for each test, the recorder was turned on for approximately 5 to 10 seconds. A typical oscillogram of the resulting measured motion is shown in figure 4.

Selection of the transducer to be used to measure the displacements, velocities, frequencies, and phase relationships was given careful consideration. Moving-coil velocity-type pickups were chosen for sensitivity, uniform phase relationships, and frequency response. A total of 19 transducers, each with a natural frequency of 2.5 Hz and a sensitivity of 96.3 millivolt/ips, were used throughout the test program.

One vertical transducer was located about 6 in. from the base of the vibrator during the entire test program. Six groups of transducers were each positioned on aluminum mounts (figure 5) to monitor movements in the vertical, radial, and transverse directions as related to the vibrator. Each triaxial array was sealed in a watertight



Peak Particle Velocity

$$Vel = \frac{(C)(m)}{2}$$

C = Calibration, inches per sec/inch
 m = Peak-to-Peak amplitude measured
 from record.

Figure 4. Typical Oscillogram Record
 Test 5, 25 Hz

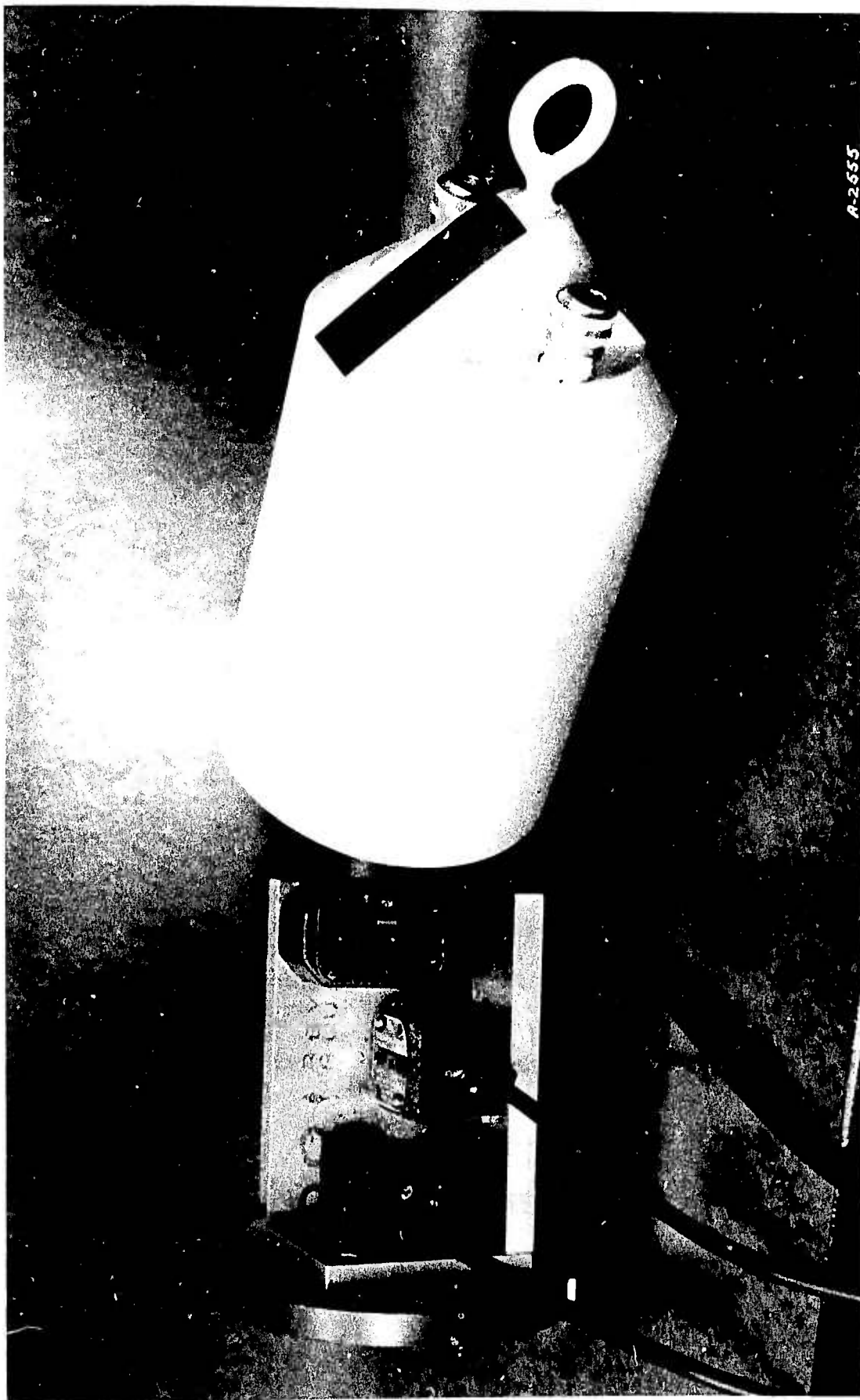


Figure 5. Velocity Transducers and Canister

canister. Prior to each test, the canisters that housed the transducers were seated on clean, well-graded sand on the ground surface or in the boreholes. A compacted sand backfill was used to raise the canisters to different elevations in the boreholes.

CHAPTER IV

TEST RESULTS

4.1 Surface Measurements - Group A

4.1.1 Procedure and Operational Data. The first tests conducted were measurements on the ground surface. An attempt was made to maintain a constant force at various frequencies in the vertical and torsional modes. The data obtained during the torsion tests were not analyzed in this report. The vibrator was positioned 10 ft east of borehole 1 with the transducers positioned as shown in the diagram below:

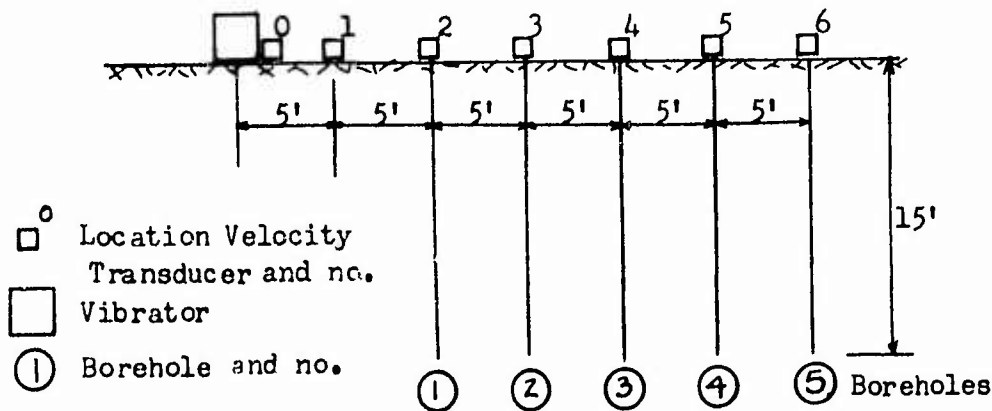


Figure 6. Group A test layout

The tests conducted were designated as series I and II for the vertical and torsional tests, respectively. Tabulated below are details of the tests.

Series I - Vertical			Series II - Torsional		
Test No.	Frequencies Hz	Force lb	Test No.	Frequencies Hz	Torque ft-lb
1	9-13	1321-2758	6	9-13	773-1610
2	13-18	1310-2512	7	13-18	766-1468
3	15-21	1330-2608	8	15-21	778-1525
4	18-26	1269-2651	9,9a	18-26	742-1550
5	25-36	1276-2616	10	25-36	745-1550

4.1.2 Measurements. The data recorded on the oscillograph were reduced by manual scaling techniques in terms of peak particle velocities and are tabulated in Appendix D, table D-1. Where no data are shown in the table, either the magnitude of the signal was not large enough to register or there was an instrumentation failure during the test. Maximum particle velocity on the surface was about 0.30 ips and occurred about 5 ft from the vibrator operating in the vertical mode between 19 and 20 Hz. Motion away from the vibrator generally increased to a point 5 ft from the vibrator and then decreased. The measurements in the torsional mode were less than the vertical velocities by a factor of about 10.

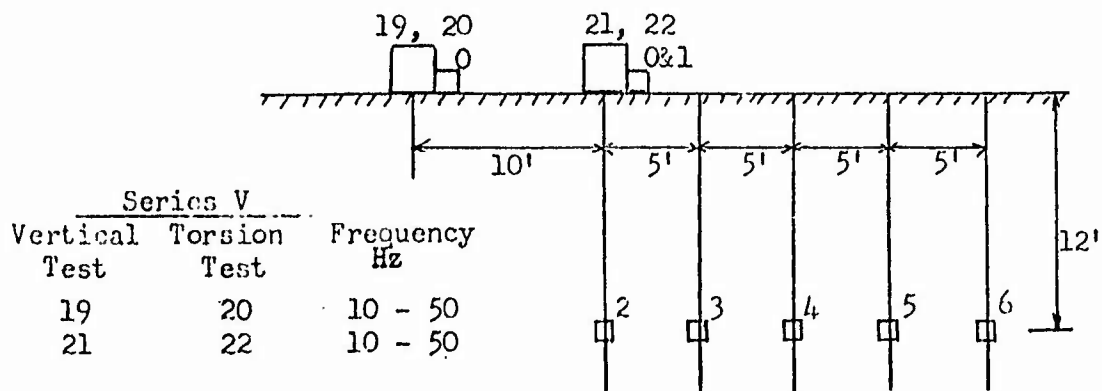
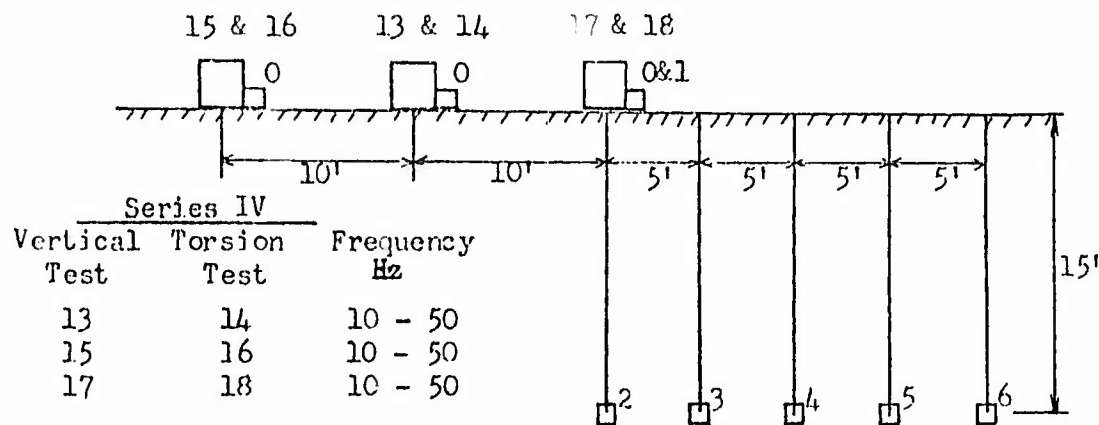
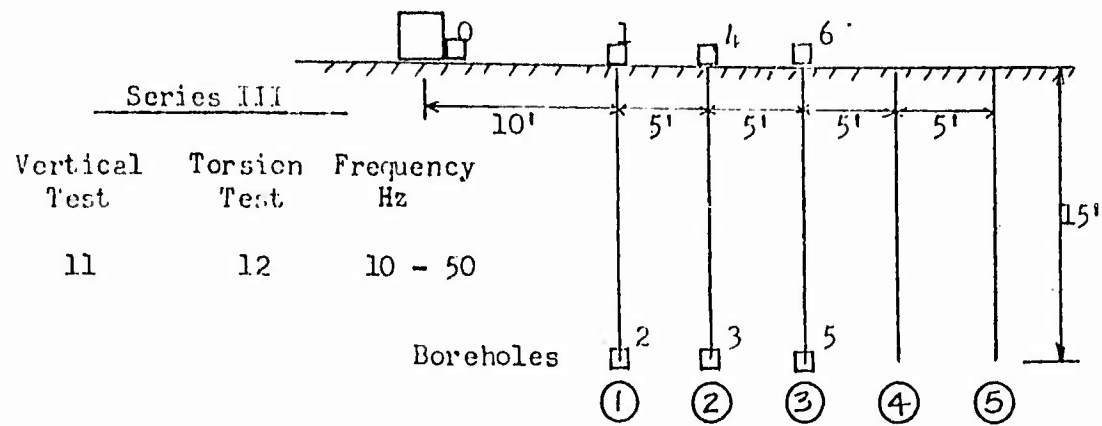
Surface velocities, determined by measuring the phase lag or time between successive peaks or troughs of the signal from the vertical transducer and by dividing this value into the distance between transducers, are shown in Appendix E. The accuracy of phase-velocity measurements determined from oscillograph records is about \pm five percent. Appendix E compares shear-wave velocities measured by the procedure outlined in Appendix A and the phase-velocity measurements obtained from the oscillograph records recorded on the surface and at various depths. The phase velocities on the surface ranged from 380 to 500 fps for frequencies ranging from 10 to 36 Hz.

4.2 Subsurface Measurements - Group B

4.2.1 Procedure and Operational Data. The second group of tests consisted of subsurface measurements made with the vibrator located on the surface over borehole 1 and 10 ft and 20 ft away from hole 1. Vertical and torsional tests were conducted through a frequency range of 10 to 50 Hz with vibratory forces ranging from 392 to 9790 lb in the vertical mode and torques ranging from 228 to 5720 ft-lb in the torsional mode. Test group B was divided into test series III - VIII with each series representing a different location of the vibrator and/or the six triaxial arrays of velocity transducers. The velocity transducers were located so that complete coverage of the borehole profile could be made. The transducers were removed from the boreholes and the boreholes were backfilled with a clean, well-graded sand to the new elevation for each test series. Figures 7 and 8 show the location of the vibrator and the transducers in the boreholes for each test series and a tabulation of tests conducted and frequency ranges.

4.2.2 Measurements. The particle-velocity measurements for group B were reduced from the oscillograph records in the same manner as those in group A and are tabulated in Appendix D, table D-2. The magnitudes of the particle velocities during the vertical tests were greater near the surface and near the first borehole than at any other place in the borehole profile.

Phase-velocity measurements made between adjacent transducers are

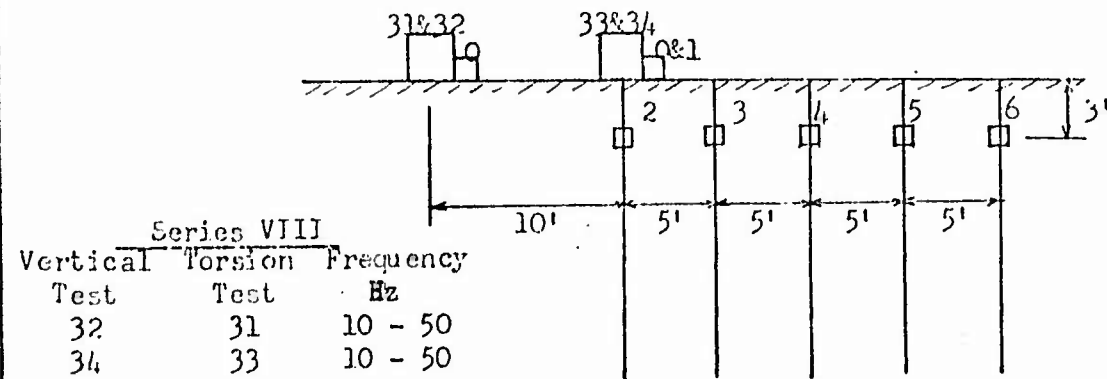
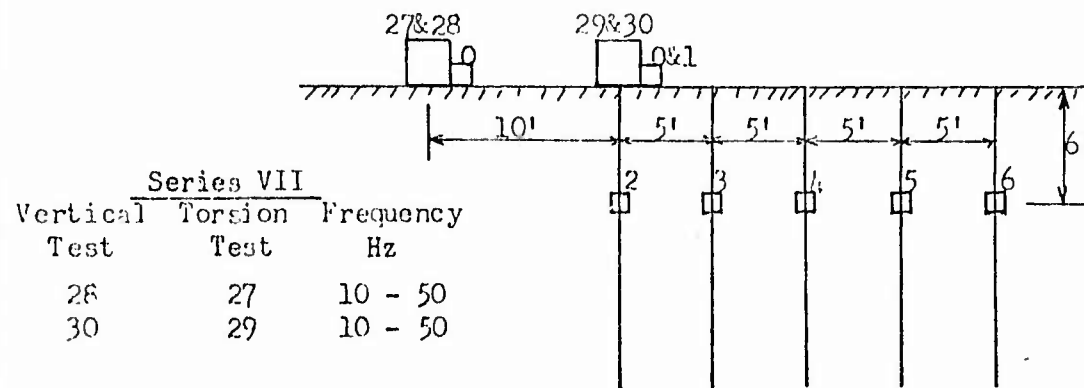
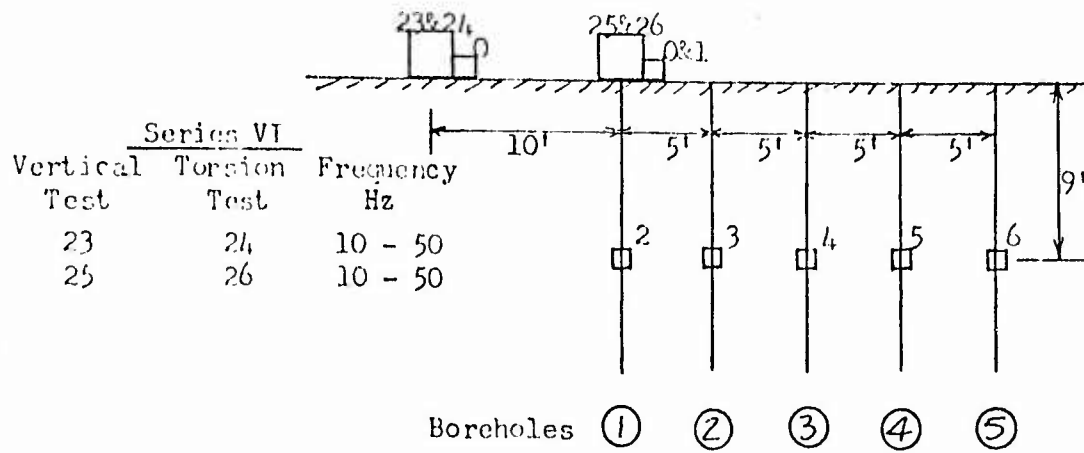


□⁰ Location Velocity Transducer and number

□ Vibrator

① Borehole and number

Figure 7 Test Group B
Series III-V



- Location Velocity Transducers and number
- Vibrator
- ① Borehole and number

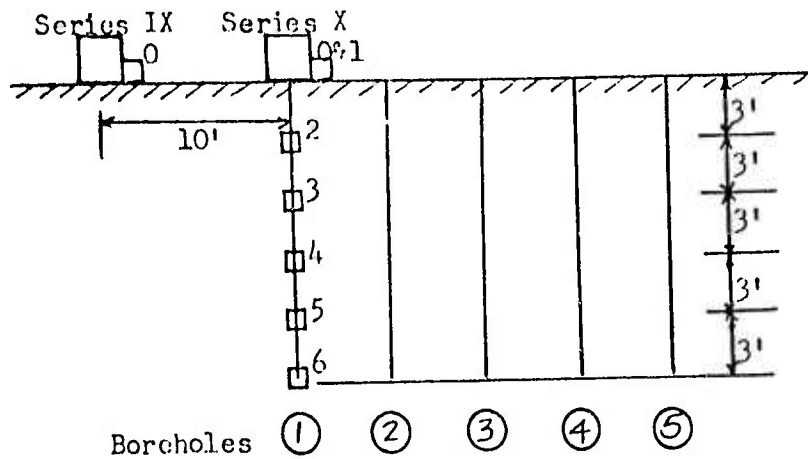
Figure 8. Test Group B
Series VI-VIII

displayed versus depth on a grid pattern of the boreholes and transducer locations in Appendix E. Each figure in Appendix E presents data at a single frequency and compares the one-half wavelength depths and shear-wave velocity computed by the procedure discussed in Appendix A with phase-velocity values listed in the grid pattern. The phase velocities from the grid system varied from 120 to 900 ips for frequencies ranging from 15 to 50 Hz.

4.3 Subsurface Measurements - Group C

4.3.1 Procedure and Operational Data. Test group C was very similar to group A except that the transducers were located in one borehole at 3-ft intervals from the surface to a depth of 15 ft. Borehole 1 was backfilled with a clean, well-graded sand, and the sand was tamped with a rod and plate to the desired elevation for each test. The vibrator was located both 10 ft from the hole and over the hole for test series IX and X, respectively, and operated in the vertical and torsional mode at each location. The vibrator frequency ranged from 9 to 50 Hz, the force in the vertical mode varied from 776 to 9790 lb, and torque for the torsion tests varied from 453 to 5700 ft-lb. Figure 9 shows the location of the vibrator and transducer for each test series and a tabulation of tests conducted and frequency ranges.

4.3.2 Measurements. The subsurface particle velocity data acquired during the conduct of test series IX and X are tabulated in Appendix D, table D-3. Recorded particle-velocity measurements for the vertical and torsional modes were in the same magnitude as those recorded



Series IX			
Vertical Test	Frequency, Hz	Torsion Test	Frequency, Hz
35	10 - 50	41	25 - 36
36	12 - 14	42	18 - 26
37	9 - 12	43	10 - 50
38	15 - 21	44	13 - 18
39	18 - 26	45	9 - 13
40	25 - 36	46	15 - 21

Series X			
Vertical Test	Frequency, Hz	Torsion Test	Frequency, Hz
53	25 - 36	47	15 - 21
54	18 - 26	48	18 - 26
55	10 - 50	49	10 - 50
56A	13 - 18	50	13 - 18
57	15 - 21	51	9 - 13
58	9 - 12	52	25 - 36

- ⁰ Location Velocity Transducer and number
- Vibrator
- ① Borchhole and number

Figure 9. Test Group C
Series IX-X

during groups A and B. The particle velocities in general decreased with depth. No phase-velocity measurements were made for this test group.

CHAPTER V

ANALYSIS OF RESULTS

An analysis of the conventional soil tests and dynamic field and laboratory tests was made to determine relative homogeneity of the loess test section to a depth of 15 ft. An analysis was also made of the particle-velocity measurements, shear-wave velocities determined by the procedure outlined in Appendix A, and the phase velocities determined between adjacent vertical transducers located in the bore-hole profile. The analysis was made to determine the relationship between the Rayleigh-wave velocity, particle velocity, and shear-wave velocity and the half-wave depth relationship of the shear-wave velocity.

5.1 Conventional Soil Tests

Conventional soils tests that included visual classifications, gradation analysis, Atterburg limits, densities, and water contents did not indicate any major differences or changes in the soil materials to a depth of 20 ft. Counts of the blows per ft taken during the split spoon sample tests indicated that the standard penetration resistance varied from 5 to 19 for a depth of about 20 ft. The visual classification and gradation indicated that the 15-ft-deep test section was a silty clay material (loess). The water content varied from about 23-33 percent; the wet density varied from about 110 to 120 pcf; the liquid limit varied from 28 to 36, the plastic limit varied from about 23 to 26; and plasticity index varied from about 4 to 13. The

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unconsolidated-undrained Q-test indicated shear strengths to be uniform with depth, varying from about 0.20 to 0.53 tons/sq ft from the surface to a depth of 20 ft. Direct shear strengths were also uniform with depth varying from about 0.20 to 0.60 tons/sq ft. The unconfined compression strength ranged from 0.6 to 1.2 tons/sq ft. A comparison of the conventional static strength parameters is shown in a composite plot in figure 10; the parameters include standard penetration resistance, direct shear, and unconfined compressive strength versus depth. These data indicate that the soil materials are fairly homogeneous in strength from the surface to a depth of about 20 ft.

5.2 Dynamic Laboratory and Field Moduli Tests

Dynamic laboratory and field tests, conducted to determine the elastic and shear moduli of the soil, indicated that the field and laboratory test exhibited about the same values. The dynamic moduli determined in the laboratory and field test are shown versus depth in figure 10. The laboratory shear moduli varied from about 4200 to 8000 psi, and the field shear moduli varied from 5500 to 7200 psi from near the surface to a depth about 20 ft. The average compression (Young's) moduli were about three times the shear moduli. The close agreement of the laboratory and field measurements of the dynamic moduli is considered to be excellent. Both the dynamic moduli and the conventional soil measurements demonstrate that the test section soil is fairly homogeneous to a depth of about 20 ft.

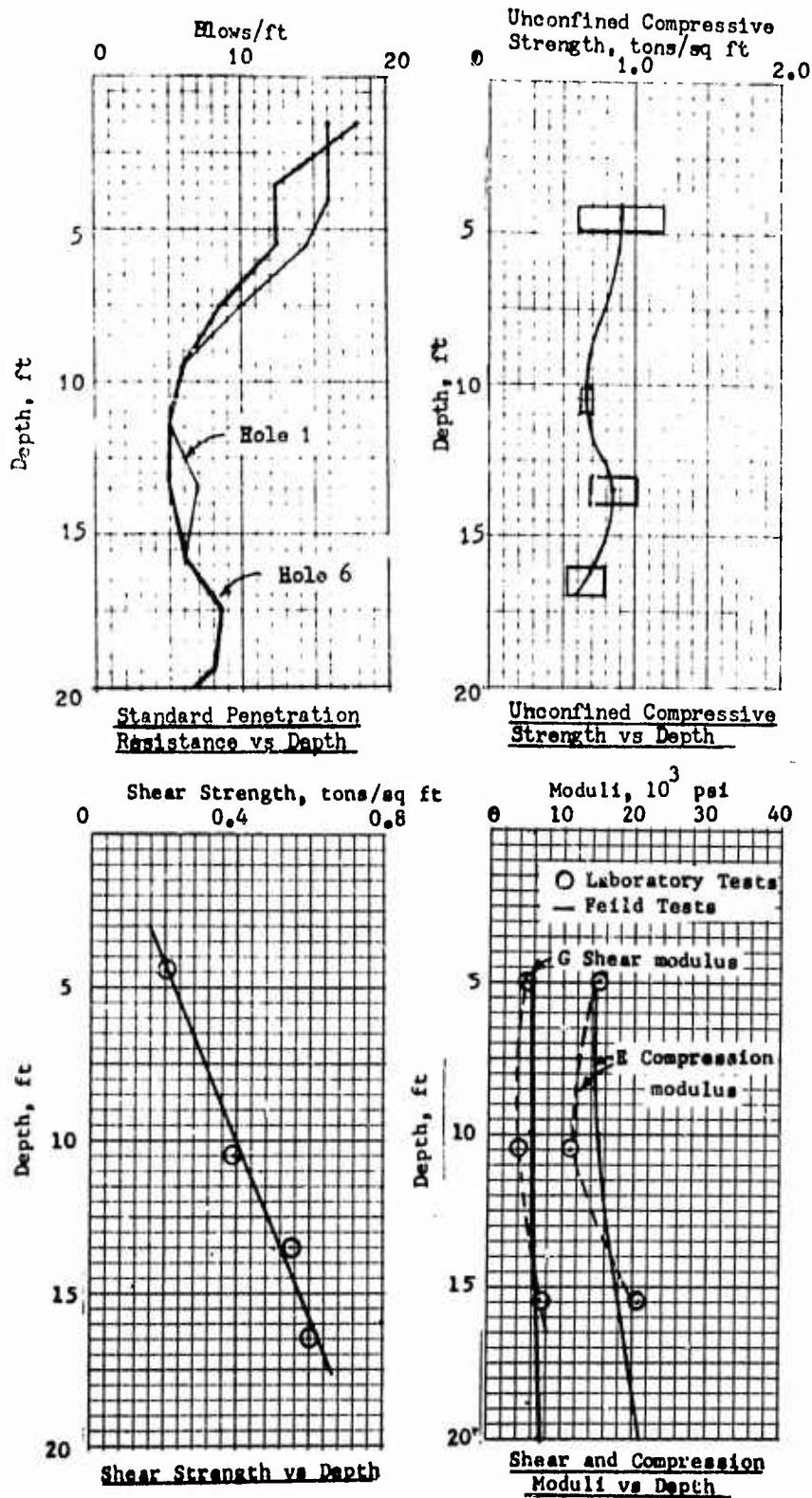


Figure 10. Comparison of Strength Parameters

5.3 Phase-Velocity Measurements

Comparisons were made of phase velocities from the dynamic laboratory tests, surface wave tests described in Appendix A, and from the velocity pickups buried at various depths in the test section. A comparison of the phase velocities determined from each method is shown plotted versus depth in figure 11. The laboratory and surface velocities were plotted at one half the wave length for individual frequencies from 15 to 50 Hz. Subsurface phase velocities were determined at a depth of one half the wavelength (Appendix A) from the phase-velocity profiles shown in Appendix E. The phase velocities varied from about 450 fps near the surface to about 530 fps at a depth of about 20 ft. The laboratory, surface, and subsurface phase velocities shown plotted in figure 11 indicate that each method exhibited about the same velocity for the depth of the test section. The phase-velocity data indicate that a fairly homogeneous material exists beneath the test section but does not necessarily confirm the concept that the surface wave propagates through a depth equal to one half the wavelength. The phase-velocity data (conventional and dynamic laboratory tests and field tests) confirm the homogeneity of the test section.

5.4 Particle-Velocity Measurements

The peak particle-velocity measurements versus distances for various depths below the ground surface for the vertical, radial, and transverse direction are plotted in Appendix F, figures F-1 through F-8, for a range of frequencies from 15 to 50 Hz. Each figure

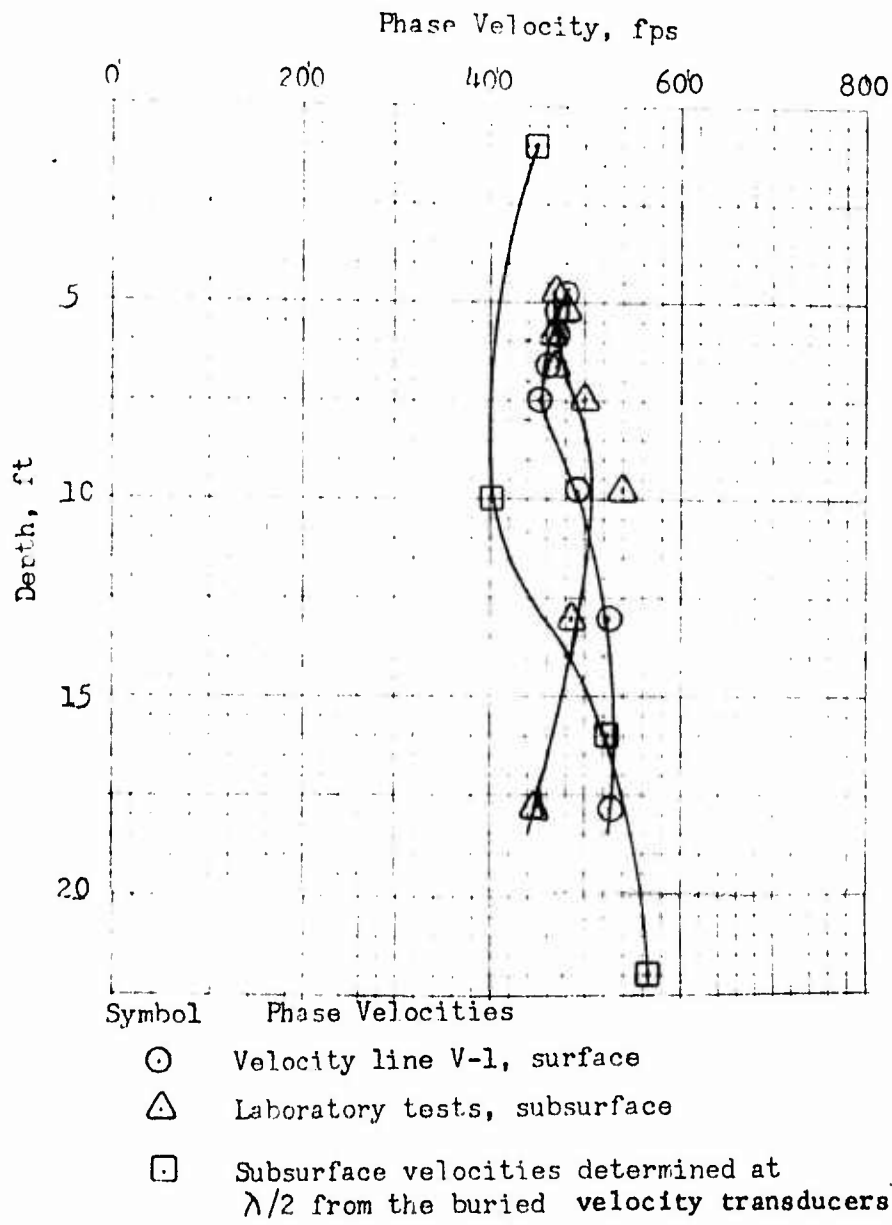


Figure 11. Comparison of Phase Velocities

presents data on particle velocities at a single frequency that was taken from test runs where the vibrator was located 10 ft from hole 1 and operated at about the same eccentric load. The peak particle velocities for the vertical, radial, and transverse directions with respect to vibrator location are plotted on the same vertical axis at each borehole location. The vertical particle velocities measured from the vibrator generally attenuated with distance; whereas, the radial and transverse measurement showed very little attenuation with distance.

Since figures F-1 through F-8 only show particle velocity versus distance at given depths, a second series of plots were made that included particle velocity versus depth measured simultaneously in hole 1. Figures F-9 through F-11 include measurements made in the vertical, radial, and transverse directions, respectively, for a range of frequencies and eccentric loadings with the vibrator located 10 ft from hole 1 and then located directly over the hole. The vertical particle velocity versus depth measured 10 ft from the vibrator attenuated with depth for all frequencies except 50 Hz. The particle velocity directly beneath the vibrator changed very little with depth at the lower frequencies but the values observed above 20 Hz showed an increase at a depth of about 3 ft and then a decrease. The radial component plotted versus depth (Figure F-10) for the vibrator located 10 ft from the hole showed very little change with depth. The radial particle velocity change directly beneath the vibrator attenuated to a depth of about 6 ft and then remained constant

to 15 ft. The transverse component plotted in figure F-11 for the vibrator located 10 ft away from the hole indicated an increase in movement on the surface with an increase in frequency but remained fairly constant in amplitude from a depth of 3 to 15 ft. The transverse component measured directly beneath the vibrator decreased very little in magnitude from the surface to a depth of 15 ft.

The vertical and radial measurements discussed in the previous paragraphs are plotted in figures 12 and 13 as particle velocity ratio versus depth for the Rayleigh wave. The ratio of the particle velocity at some depth z to the surface particle velocity was plotted versus the depth-to-wavelength ratio for each frequency. The theoretical dispersion curves shown in figure 12 for different Poisson's ratios and points for the vertical components measured 10 ft from the vibrator are in very close agreement. This close agreement would suggest that the soil at the site of the investigation is a fair example of a homogeneous elastic half space. The radial components shown in figure 13 are also compared with the theoretical curves for different Poisson's ratio, except that these values were taken from measurements directly beneath the vibrator. The radial components 10 ft from hole 1 were so scattered that only 10 percent of the values would plot on the graph. The field data for the radial components agree well with the theoretical curve. The dotted lines represent an envelope of the negative curves. The horizontal particle velocity depth-surface ratio and the depth-wavelength ratio fall into a narrow

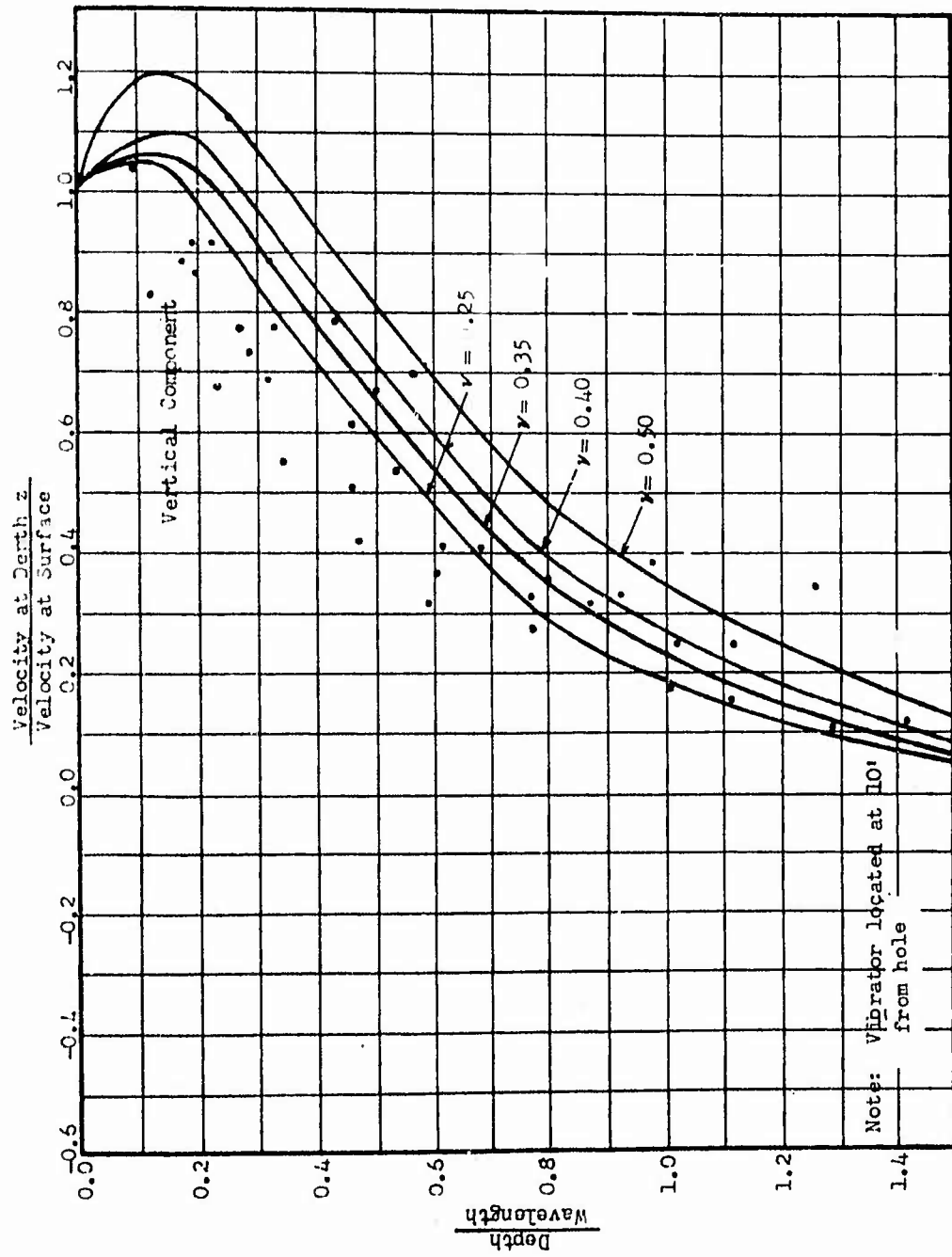


Figure 12. Vertical Particle Velocity Ratio versus Depth for Rayleigh Wave

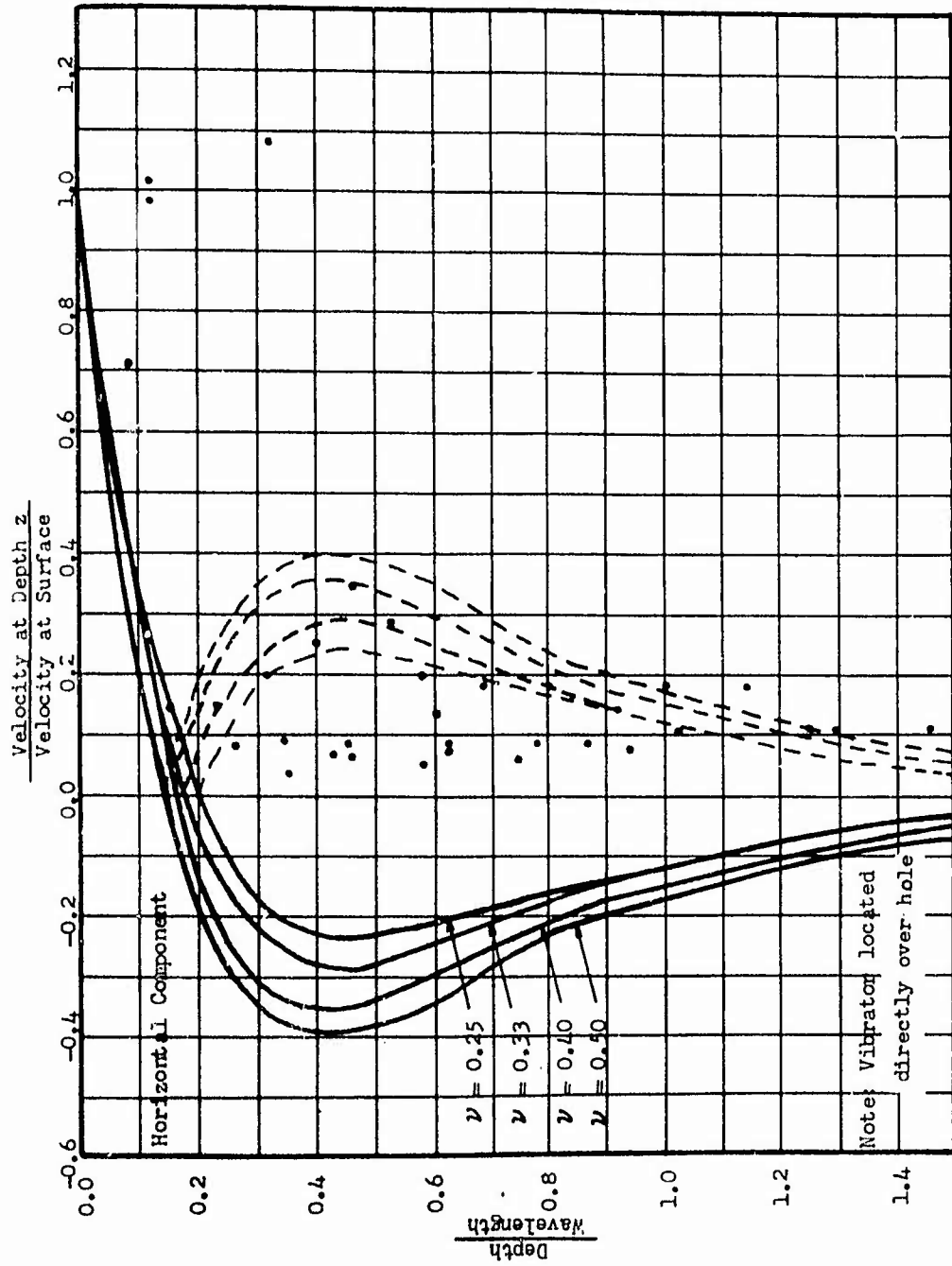


Figure 13. Radial Particle Velocity Ratio versus Depth for Rayleigh Wave

pattern with very little scatter. The particle-velocity data do not suggest any physical relationship to exist to support the theory that surface waves penetrate the soil for a depth equal to one half the wavelength. The vertical particle velocity attenuated with an increase in depth from the vibrating source and there was relatively no change in the radial component with depth.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

It was concluded from the particle-velocity measurements that the soil at the site of the investigation represents a fairly homogeneous elastic half-space that causes very little dispersion of a vibrating wave in the medium. These data provide no explanation of or proof to support the theory that surface waves penetrate the soil for a depth equal to one half the wavelength. However, comparison of phase-velocity measurements reduced from the records from surface and subsurface transducers in the borehole profile and the shear-wave velocities derived by the method outlined in Appendix A indicates excellent agreement of the methods of measurement.

It was further concluded from the analysis of the test results that the existing concept that the surface-wave propagates through a depth approximately equal to half the wavelength cannot be verified from tests of a homogeneous medium where there is practically no dispersion of the wave.

The shear-wave velocity varied from about 450 fps near the surface to about 530 fps at a depth of about 20 ft. The conventional and dynamic laboratory and field tests of soil from the site indicated the soil represent a homogeneous elastic half-space. It is, therefore, concluded that the dynamic procedures outlined in Appendix A provide a reliable description of subsurface conditions at a site and accurate values of the in situ moduli of the medium and should be

used in conjunction with conventional soils exploration and laboratory tests for the design of foundations subject to vibratory loads.

6.2 Recommendations

It is recommended that further studies of similar nature should be conducted and analysis of accumulated data be continued. Based on the agreement of the measurements and values obtained from the field test section, it is believed that additional studies are justified to further evaluate present criteria. Controlled test sections could be constructed or areas with various soil conditions could be selected that would include more dispersive materials or layered systems. The information thus obtained would be beneficial in evaluating present procedures for determining in situ soil characteristics.

ABSTRACT

Jack Fowler, Master of Science, 1972

Department: Civil Engineering

Major: Soil Mechanics

Title of Thesis: An Evaluation of An Existing Procedure For
Determining Shear Moduli at Depths by In Situ
Vibratory Technique

Directed by Professor Robert M. Scholtes

Pages in Thesis, 123. Words in Abstract, 474.

ABSTRACT

The Corps of Engineers has been determining the dynamic characteristic of soil and pavement systems by the use of surface-induced vibrations for approximately 10 years. This non-destructive vibratory technique is utilized by all federal agencies involved in planning the design of space-craft launch and operation facilities, antiballistic missile sites, heavy weapons, air field and road construction, radar towers or structures to house delicate electronic equipment when accurate and quick information pertaining to the ability of the soils to resist deformation is required. The Corps of Engineers has been actively engaged in improving and developing equipment and procedures for determining the elastic moduli (and thus the resistance to deformation) of foundation soils by vibratory techniques. ←

The basic theory assumes that the Rayleigh waves generated by the vertical oscillations of a vibrator on the soils surface are for all practical purposes considered to be shear-waves. When the waves are propagated at a known frequency, the wave velocity is determined by the product of the wave length and frequency. The shear-wave velocity propagation is assumed to take place within a depth equal to about one half the wavelength.

The purpose of this investigation was to validate the existing theory by determining shear moduli from the displacements, velocities, frequencies, phase relationship, and wave shape characteristics recorded versus distance and depth from a vibratory source placed on the ground surface. Specifically to evaluate the concept that the surface wave propagates through a depth about equal to one half the wavelength.

The results of this investigation indicated that the test section represented a fairly homogeneous elastic half-space which causes very little dispersion of a vibrating wave. Phase velocity measurements determined from surface and subsurface transducers were in excellent agreement. The conventional and dynamic laboratory and field tests of the soil from the site also indicated that the soil represented a homogeneous elastic half-space. The data collected offered no explanation or proof to support the theory that surface waves penetrate the soil for a depth equal to one half the wavelength.

It was concluded from the analysis of the existing concept, that the surface wave propagates through a depth approximately equal to half the wavelength cannot be verified from tests of a homogeneous medium where there is practically no dispersion of the wave. It was also concluded, however, that the vibratory technique does provide a reliable description of the subsurface condition and that the accurate values of in situ moduli of the medium should be used in conjunction with conventional soils exploration and laboratory tests for the design of foundations subjected to dynamic loads.

It was recommended that similar tests be conducted on test sections that would include more dispersive soil materials or layered systems. Additional information from similar tests would be beneficial in verifying the present procedures for determining in situ soil characteristics.

APPENDIX A
DETERMINATION OF ELASTIC CHARACTERISTICS OF
SOILS BY DYNAMIC FIELD TECHNIQUES

APPENDIX A
DETERMINATION OF ELASTIC CHARACTERISTICS OF
SOILS BY DYNAMIC FIELD TECHNIQUES

This appendix is a resumé of the usual tests performed and methods employed in the investigation of in situ soils by dynamic techniques. While every test described herein may not be applicable to the study described in the main text, they are presented to describe better the methods and the basic theories on which the procedures are based.

Seismic Tests

Seismic tests^{1,2} are made to determine the velocity of compression waves in the soil; this velocity value is used in conjunction with other data to determine Poisson's ratio. The seismic data are collected first so that the presence of unusual subsurface conditions, if any, can be revealed, and on the basis of this information, the vibration lines can be located to the best advantage. Data obtained with seismic equipment consist principally of the time required for a compression wave to travel from a seismic source to a point of measurement.

Data are plotted in graphic form as impulse distance versus travel time. The reciprocal of the slope of the line drawn to connect the plotted points indicates the velocity of the wave through each subsurface medium encountered. A distinct break in the slope of the

line indicates that the wave has probably passed through an interface between two subsurface layers having different velocities. The depth below the surface at which the first interface occurs can be calculated from the following equation:

$$D_1 = \frac{X_1}{2} \sqrt{\frac{v_2 - v_1}{v_2 + v_1}} \quad (\text{reference 1}) \quad (A1)$$

where

D_1 = depth, in feet, from the surface to the first interface, L

X_1 = distance, in feet, from seismic source to point at which first change in slope occurs, L

v_1 = compression-wave velocity, in feet per second, in first layer of material encountered, LT^{-1}

v_2 = compression-wave velocity, in feet per second, in second layer of material encountered, LT^{-1}

If a second interface is encountered, the depth can be calculated from the following equation:

$$D_2 = \frac{5}{6} D_1 + \frac{X_2}{2} \sqrt{\frac{v_3 - v_2}{v_3 + v_2}} \quad (\text{reference 1}) \quad (A2)$$

where

D_2 = depth, in feet, from the surface to the second interface, L

X_2 = distance, in feet, from seismic source to point at which second change in slope occurs, L

v_3 = compression-wave velocity, in feet per second, in third layer of material encountered, LT^{-1}

If additional interfaces occur, equations are available to determine their depth.

Vibration Tests

Vibration tests are conducted to determine both the frequency and the velocity of shear waves in the soil. The vibration tests are performed with a high-frequency electromagnetic vibrator and low-frequency counterrotating-mass vibrator, in accordance with the procedure outlined in WES Miscellaneous Paper No. 4-577, A Procedure for Determining Elastic Moduli of Soils by Field Vibratory Techniques.

The referenced report explains in detail how wavelengths of propagated Rayleigh waves (treated as shear waves) of known frequency are used to determine the shear and compression moduli of subsurface materials.

Wave Propagation

When sustained vibrations are induced into a soil, concentric waves are propagated outward from the source. If the waves are propagated at a known frequency f , then $V = \lambda f$ (A3)

where

v = wave velocity, LT^{-1}

λ = wavelength, L

f = frequency of the vibrator, cycles T^{-1}

Shear wave velocity and surface (Rayleigh) wave velocity are related by Poisson's ratio. For Poisson's ratios ranging from 0.2 to 0.5, the difference in velocities is less than 6 percent.

Therefore, for practical purposes, shear waves can be considered to have the same velocity as surface waves. Thus, shear velocities can be determined by the vibratory tests described.

Computation of elastic
moduli and Poisson's ratios

Wave velocity is dependent upon the waveform and the ratio of the elasticity of the medium to its mass density ρ . The relation of shear modulus of elasticity G and shear wave velocity v_s is as follows:

$$G = v_s^2 \rho \quad (\text{reference 3}) \quad (A4)$$

where

G = shear modulus of soil, FL^{-2}

v_s = shear-wave velocity, LT^{-1}

ρ = mass density of soil, γ/g , $FL^{-4}T^{-2}$

γ = wet unit weight of soil, FL^{-3}

g = acceleration of gravity, LT^{-2}

With the assumption that compression-wave velocity and shear-wave velocity were determined for comparable materials, Poisson's ratio can be calculated from the ratio of velocities v_r :

$$v_r = \frac{v_c}{v_s} \quad (\text{reference 4}) \quad (A5)$$

Poisson's ratio ν is then

$$\nu = \frac{v_r^2 - 2}{2(v_r^2 - 1)} \quad (\text{reference 4}) \quad (A6)$$

The compression modulus E (Young's modulus) can be determined by:

$$E = 2(1 + \nu) G \quad (\text{reference 5}) (A7)$$

It should be noted here that equations A4 through A7 are for homogeneous, isotropic, elastic materials.

Method of Computation

The preceding paragraphs have discussed the theory and mathematics utilized in the seismic and vibratory techniques for obtaining Poisson's ratio and the elastic moduli of soils. To present and utilize properly the data collected, a definite pattern, or sequence of computation, has evolved as experience progressed.

The basic data obtained by means of the vibratory technique are plotted as distances from vibrator to successive half or whole wavelengths versus the number of wavelengths. The wavelength for a particular frequency is determined as the reciprocal of the slope of the line through the plotted data points. The velocity of the shear wave is then determined by equation A3. A compression-wave velocity, which is determined by use of equations A1 and A2, is used together with the shear-wave velocity to determine a Poisson's ratio by equation A6. Caution should be exercised to ensure selection of the shear-wave and compression-wave velocities for corresponding media or depths. Based on experience of the WES, it appears that variations in E and G correlate best with conventional exploration methods when it is assumed that the depth for the computed

value of E and G is one half the length of the surface wave. Therefore, the computed values of E and G are considered to be the elastic moduli at such a depth. Using one half the length of the surface wave as the depth at which the velocity of propagation of that wave occurs, and the velocity of the compression wave at a corresponding depth, a Poisson's ratio can be obtained for materials at different depths. The elastic moduli, E and G , can then be computed by means of equations A7 and A4, respectively. It is usually convenient to plot E and G versus depth, which again is considered to be one half the wavelength of the surface wave. Such a plot provides a visual picture of the change in soil characteristics with increasing depth. The depth can also be expressed as overburden pressure in pounds per square inch, thus providing a comparison of E and G with increasing pressure.

APPENDIX B
CONVENTIONAL SOILS TESTS

APPENDIX B

CONVENTIONAL SOILS TESTS

Drilling, collection, and classification of soil samples for the field and laboratory soil tests were conducted by personnel of the WES Inspection Exploration Section and Soils Test Section. Collection of undisturbed soil samples, split spoon sampling and recording blows per ft, and soil classification were conducted during drilling operations. Conventional laboratory tests that were conducted on split spoon samples, included visual classification, gradation analysis, Atterburg limits, and water content determinations. Specific gravity tests and triaxial unconsolidated-undrained compressional (Q) tests were performed on undisturbed samples taken from Shelby tubes.

Data from the field and laboratory tests are shown in tables B-1 through B-3. The soil at the site ranged from light to dark brown and gray silty clay (CL) to a depth of about 13.0 ft, with variation of soft and firm, brown and gray clayey silts (ML) from 13.0 to 125 ft with a limestone layer at about 125 ft (determined during other investigations near the test area). Gradation curves for the subsurface materials at different depths are shown in figure B-1. The largest particle size was about 0.1 mm with 98 percent by weight finer than the No. 200 sieve size (0.074 mm). The standard penetration resistance blow count per ft for two holes, one 15 and one 22 ft deep, are shown in figure B-2. A profile of the test area is shown in figure B-3, with the location of boreholes used for soil sampling and for instrumentation.

The following ranges of soil properties were determined for the soil from the surface to a depth of about 22 ft.

Specific gravity	2.68 - 2.71
Water content, %	23.0 - 33.8
Dry unit weight, pcf	84 - 95
Void ratio	0.83 - 0.93
Liquid limit	28 - 36
Plastic limit	23 - 26
Plasticity index	4 - 13
Degree of saturation, %	71.1 - 100

The plasticity index versus liquid limit of samples from boring 1 are plotted on figure B-4.

Four specimens of undisturbed soil from average depths of 4.5, 10.5, 13.5, and 16.5 ft, respectively, were used to determine the unconfined shear strength. The test specimens were nominally 1.4 in. in diam and 3.0 in. long and were tested under controlled-strain conditions. The results of these tests are shown in table B-1 and figures B-5 through B-8. The unconfined compressive strengths were very uniform from 4 ft to a depth of about 17 ft. The variation of compressive strength with depth is shown in figure B-9. The unconfined compressive strength ranged from about 0.6 to 1.2 tons/sq ft or 8.3 to 16.6 psi.

A total of six unconsolidated-undrained Q-tests were performed on undisturbed specimens from average depths of 4.5, 7.8, 10.5, 13.5,

16.4, and 18.3 ft. Specimens were nominally 1.4 in. in diam and 3.0 in. high. The data from these tests are shown in table B-2 and figures B-10 through B-15. The cohesion C ranged from 0.2 to 0.5 tons/sq ft (2.8 to 6.9 psi) and the angle of friction ϕ decreased with depth, ranging from 12 deg at the surface to 0 deg below a depth of about 13 ft. The shear strength varied with depth from 0.20 to 0.65 tons/sq ft or 2.8 to 9.0 psi.

Direct shear tests were conducted on four undisturbed samples taken from average depths of 4.4, 10.5, 13.5, and 16.4 ft. Initially, specimens were 3 in. square and 0.502 in. thick. The cohesion C ranged from 0.02 to 0.06 tons/sq ft (0.28 to 0.82 psi) and the angle of friction ϕ ranged from 31 to 34 deg. The data from these tests are presented in table B-3 and in figure B-16 through B-19. Figure B-20 shows that shear strength increased as depth increased.

Table B-1

Unconfined Compression Tests

Depth ft*	Material Description	Unit Weight pcf		Water Content %	Void Ratio	Degree of Saturation %	Specific Gravity	Unconfined	
		Wet	Dry					Compressive Strength tons/sq ft	Undrained Shear Strength tons/sq ft
4.1-4.8	Silty clay (CL), brown	112	91	24	0.86	74.0	2.68	0.60-1.19	0.30-0.60
10.1-10.9	Silty clay (CL), brown	116	87	33	0.93	96.0	2.68	0.64-0.70	0.35-0.32
13.0-14.0	Clayey silt (ML), brown	120	91	33	0.87	100.0	2.71	0.68-1.02	0.34-0.51
16.0-16.9	Clayey silt (ML), brown	121	93	31	0.84	99.0	2.71	0.52-0.78	0.26-0.39

*Borehole IU
Data from Figures B5-B8

Table B-2

Triaxial Q Tests

Borehole Location	Depth ft	Material Description	Approximate Dry Unit Wt pcf	Approximate Water Content %	Angle of Friction ϕ , deg	Cohesion, C tons/sq ft	Approximate		Chamber Pressure psi	Compressive Strength tons/sq ft	Shear Strength tons/sq ft
							Pressure tons/sq ft	Avg Overburden Pressure tons/sq ft			
1U	4.1-4.8	Silty clay (CL), brown	91	23	12	0.3	0.25	3.5	0.98	0.53	
								7.0	1.00	0.53	
								21.0	1.58	0.65	
	7.5-8.0	Silty clay (CL), brown	93	28	5	0.2	0.44	6.2	0.43	0.20	
								21.0	0.62	0.21	
	10.1-10.9	Silty clay (CL), brown	86	32	3	0.3	0.60	8.5	0.65	0.32	
								21.0	0.72	0.33	
								42.0	0.88	0.33	
	13.0-14.0	Clayey silt (ML), brown	91	32	0	0.5	0.78	10.8	1.01	0.49	
								21.0	0.94	0.49	
								42.0	0.99	0.49	
	16.0-16.9	Clayey silt (ML), brown	93	31	0	0.4	0.96	13.2	0.60	0.42	
								21.0	0.78	0.42	
								42.0	1.12	0.42	
	17.6-19.0	Silt (ML), grey	95	29	0	0.5	1.14	7.0	0.86	0.47	
								15.0	0.91	0.47	
								21.0	1.03	0.47	

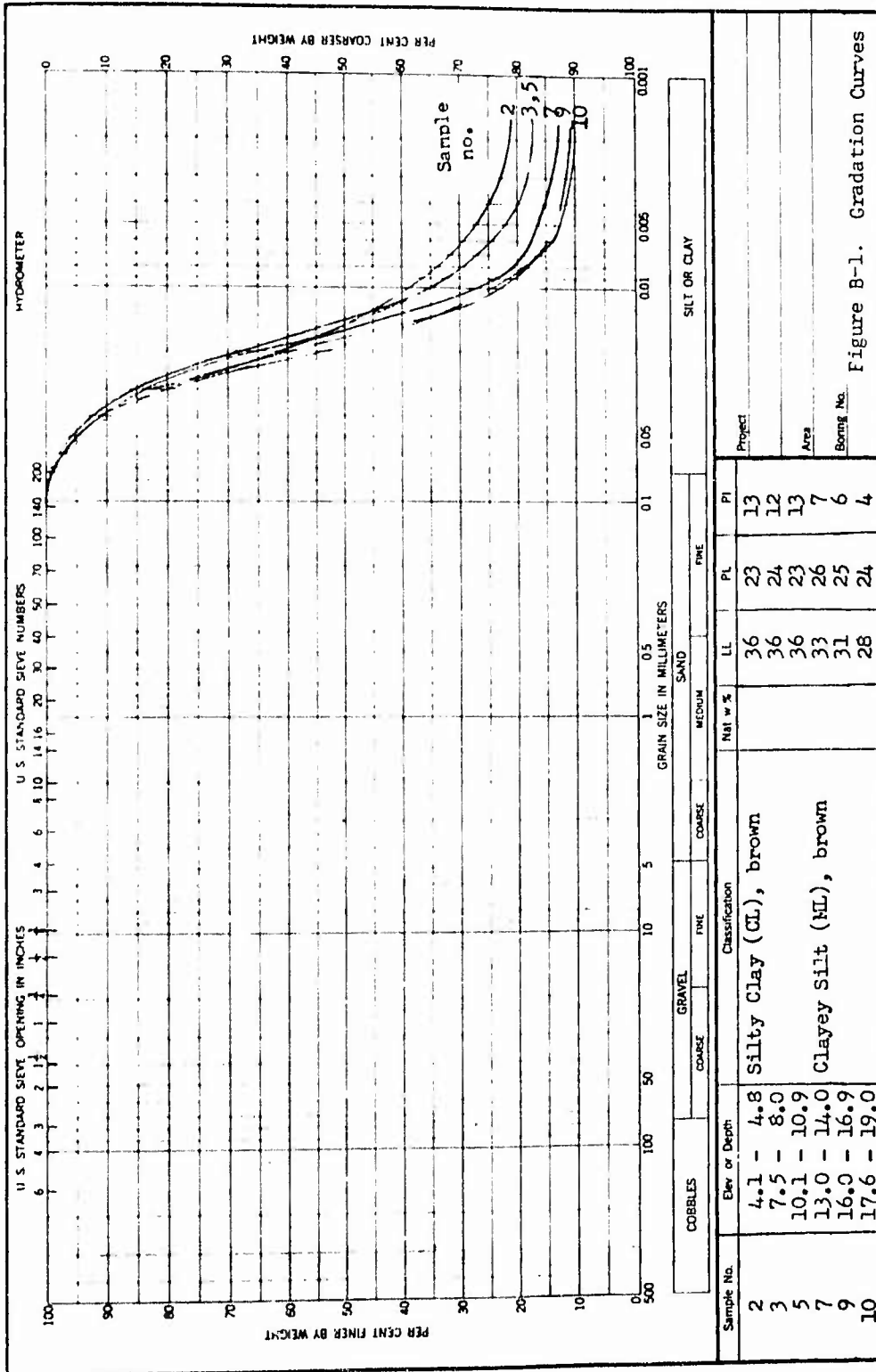
Data obtained from Figures B11-B15

Table B-3

Direct Shear Tests

Borehole Location	Depth ft	Material Description	Approximate		Angle of Friction ϕ , deg	Cohesion C tons/sq ft	Approximate		Normal Stress tons/sq ft	Shear Strength tons/sq ft	Shear Strength at Overburden Pressure tons/sq ft
			Unit Wt pcf	Water Content %			Avg Overburden Pressure tons/sq ft	Normal Stress tons/sq ft			
1U	4.1-4.8	Silty clay (CL), brown	88	23	31	0.06	0.25	0.5	0.38	0.20	
	10.1-10.9	Silty clay (CL), brown	84	32	33	0.02	0.58	0.5	0.35	0.38	
	13.0-14.0	Clayey silt (ML), brown	87	31	34	0.03	0.75	0.5	0.37	0.55	
	16.0-16.9	Clayey silt (ML), brown	90	30	32	0.02	0.93	0.5	0.34	0.60	

Data from Figures B16-319



Sample No.	Elev or Depth	Classification	GRAVEL			SAND			SILT OR CLAY		
			COARSE	FINE	COARSE	MEDIUM	FINE	PL	PI	Project	
2	4.1 - 4.8	Silty Clay (CL), brown			36	23	13				
3	7.5 - 8.0		36	24	12						
5	10.1 - 10.9		36	23	13						
7	13.0 - 14.0	Clayey Silt (ML), brown			33	26	7				
9	16.0 - 16.9		31	25	6						
10	17.6 - 19.0		28	24	4						

Figure B-1. Gradation Curves

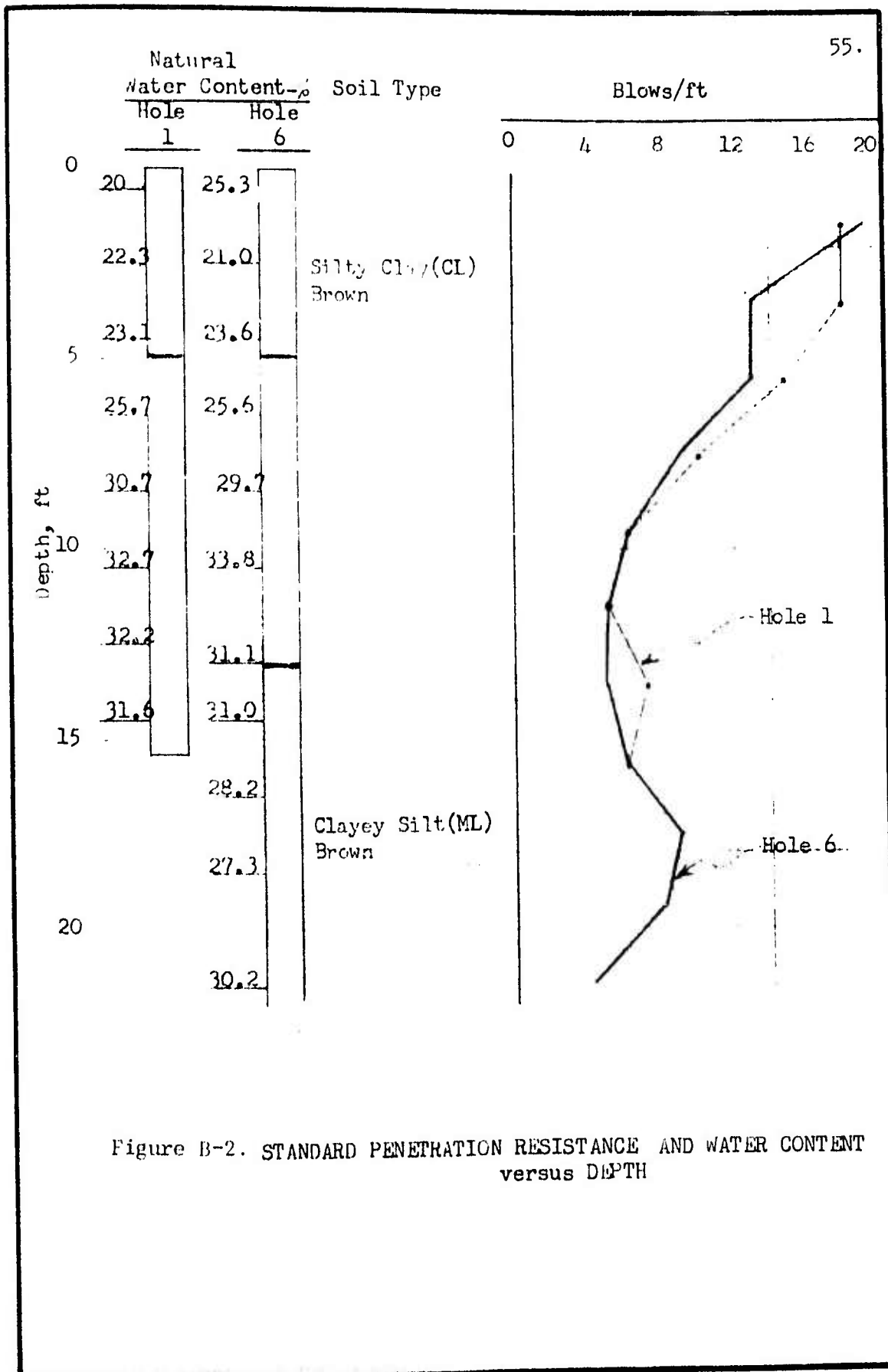
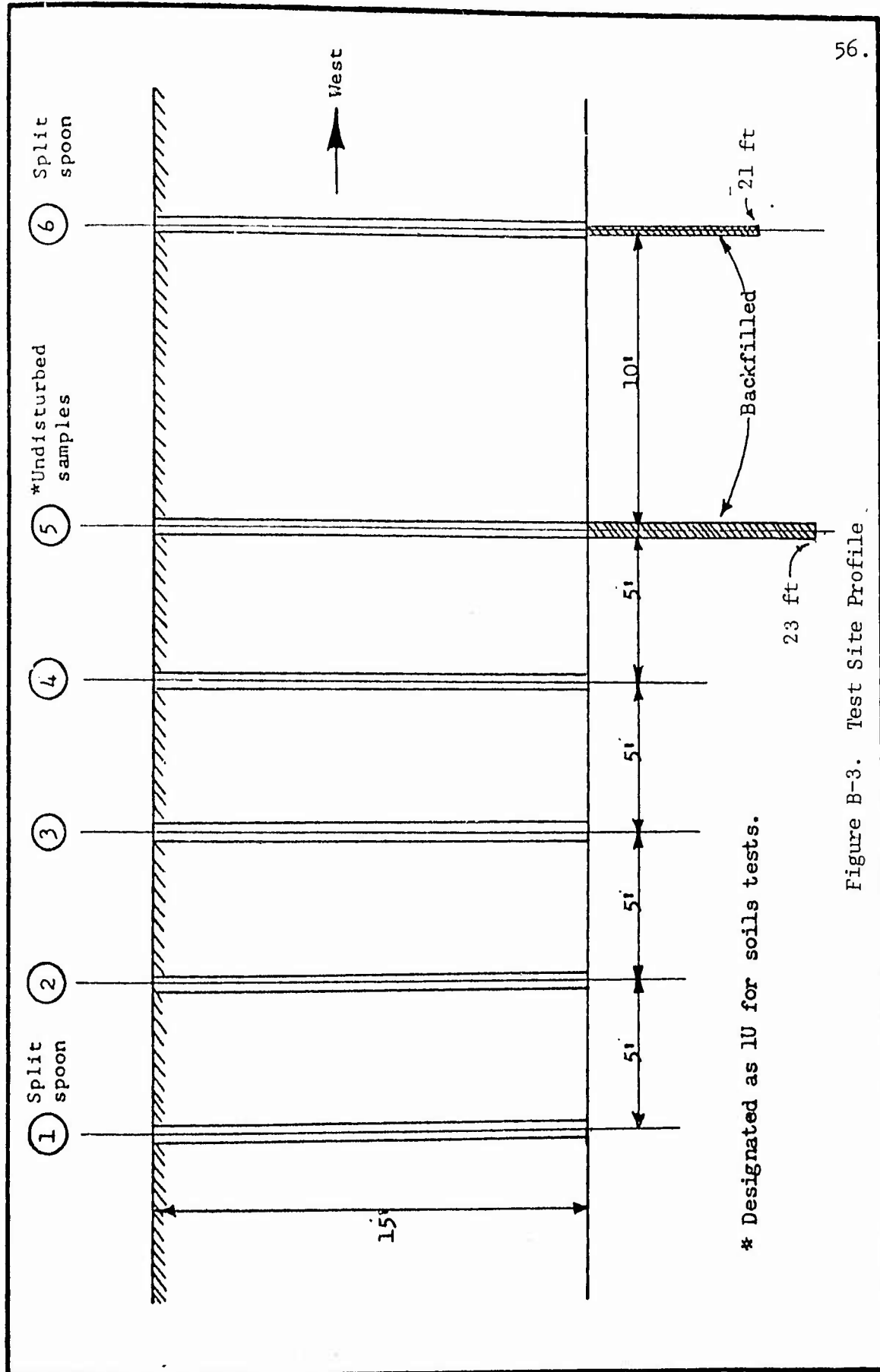


Figure B-2. STANDARD PENETRATION RESISTANCE AND WATER CONTENT versus DEPTH



* Designated as LU for soils tests.

Figure B-3. Test Site Profile

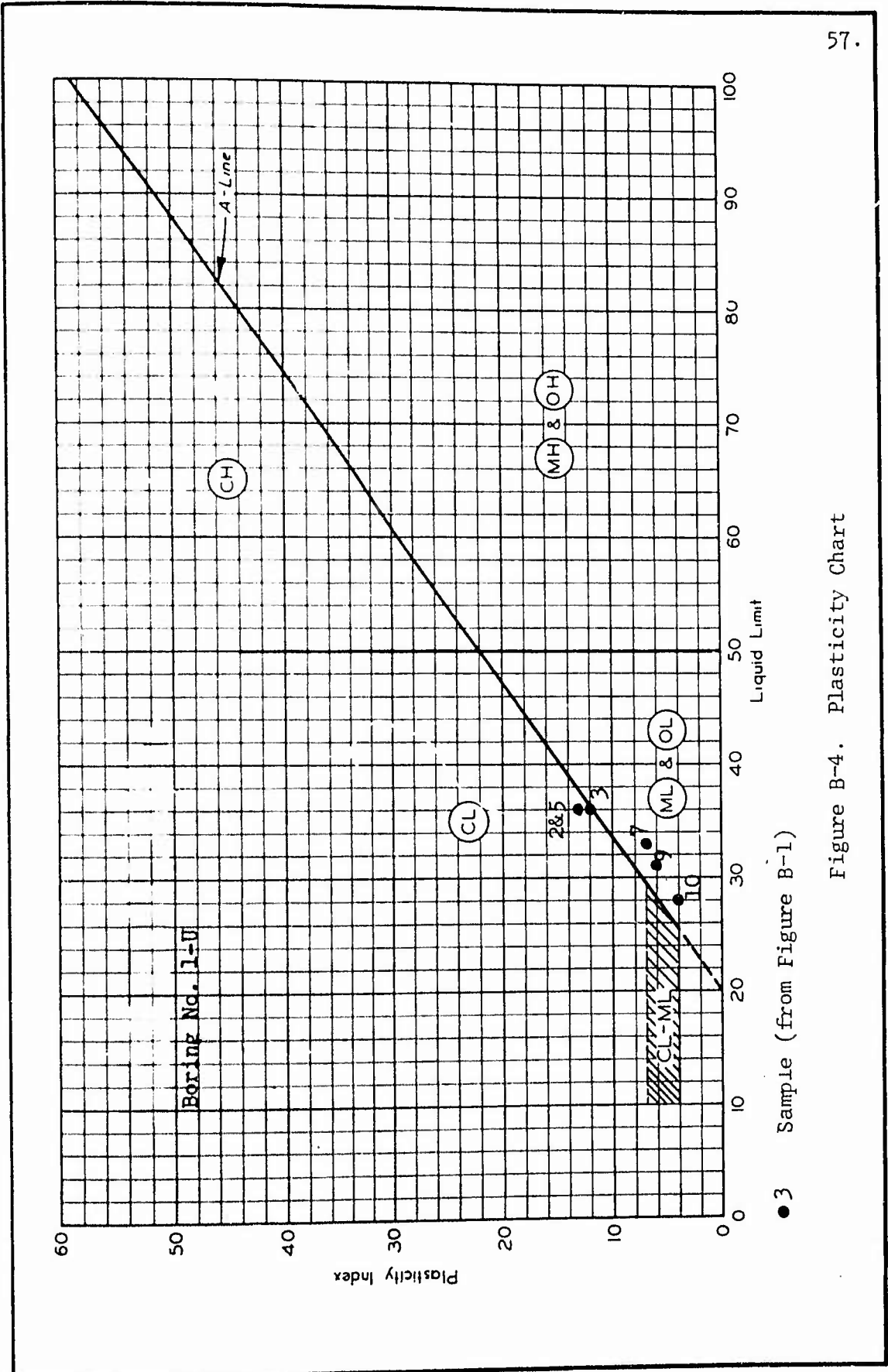
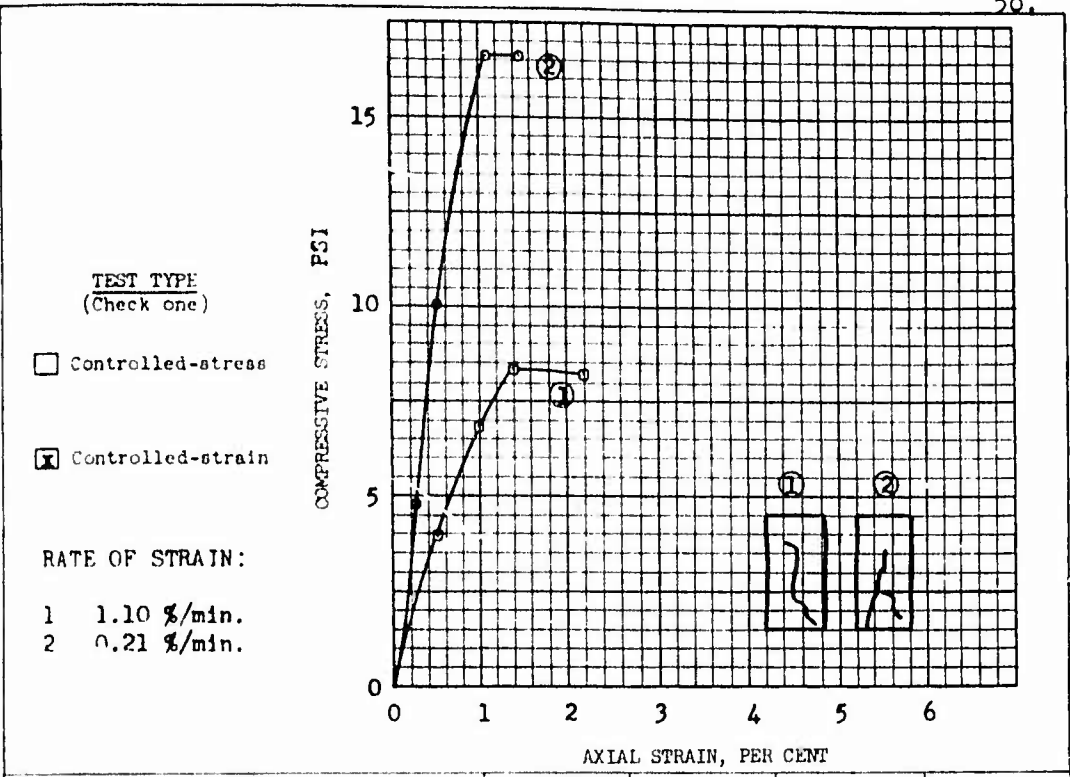


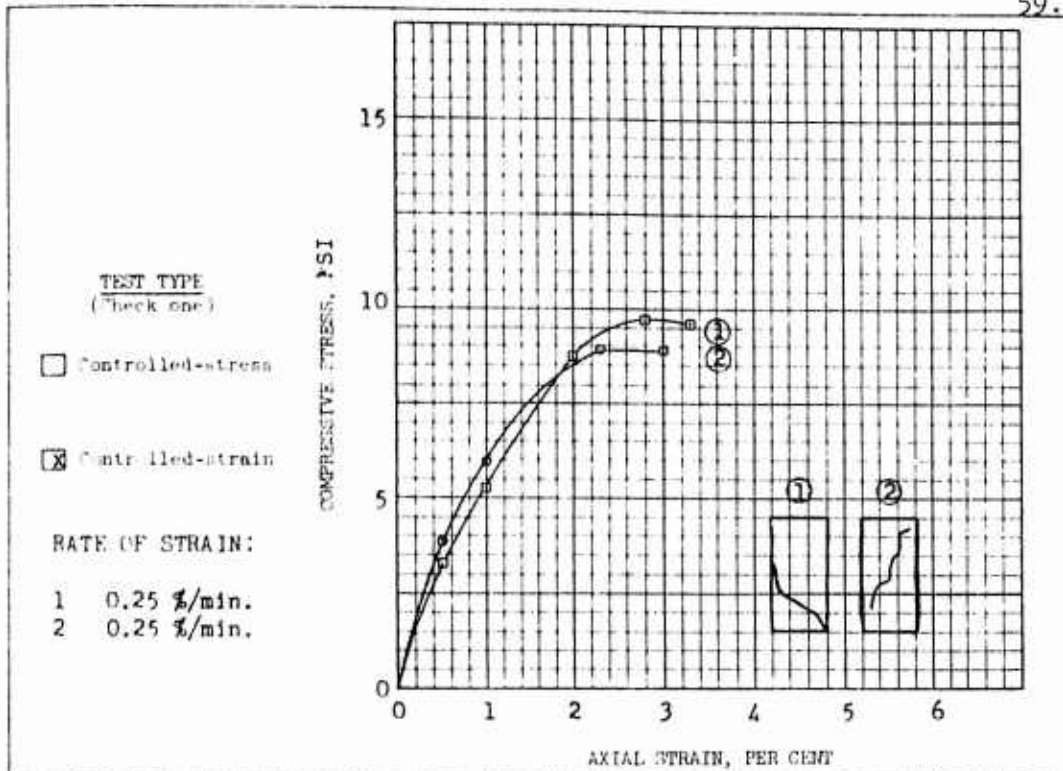
Figure B-4. Plasticity Chart

● 3 Sample (from Figure B-1)



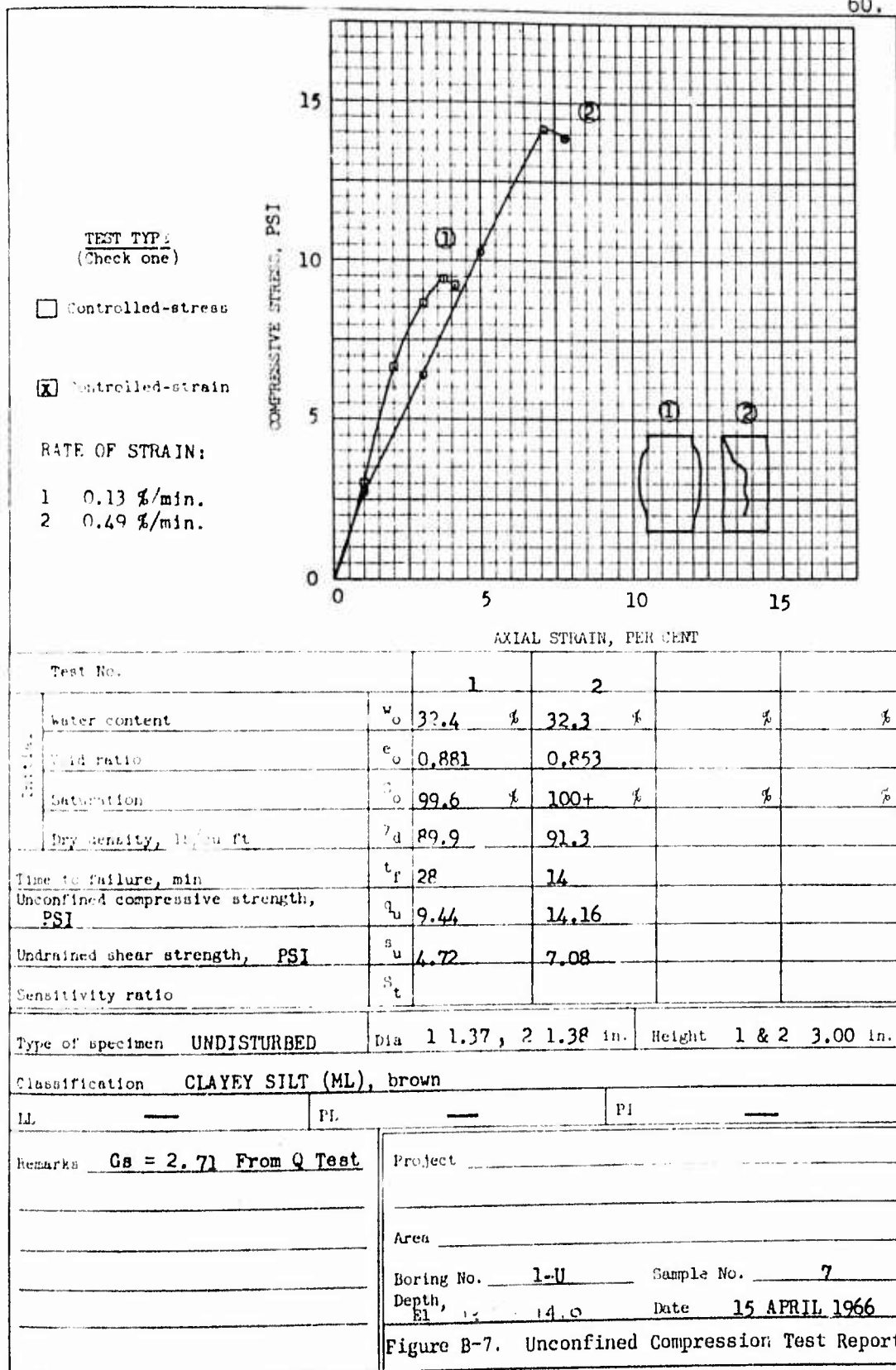
Test No.		1	2		
Initial	Water content	w_o 23.3 %	23.8 %	%	%
	Void ratio	e_o 0.878	0.828		
	Saturation	S_o 71.1 %	77.0 %	%	%
	Dry density, lb/cu ft	γ_d 89.1	91.5		
Time to failure, min		t_f 1	5		
Unconfined compressive strength, PSI		q_u 8.35	16.57		
Undrained shear strength, PSI		s_u 4.17	8.28		
Sensitivity ratio		S_t			
Type of specimen UNDISTURBED		Dia 1 & 2 1.41 in.		Height 1 & 2 3.00 in.	
Classification SILTY CLAY (CL), brown, fissured					
LL —		PL —		FI —	
Remarks $G_s = 2.68$ From Q Test		Project _____			
Insufficient material for		Area _____			
check tests		Boring No. 1-U		Sample No. 2	
		Depth, 4.1' - 4.8'		Date 13 APRIL 1966	

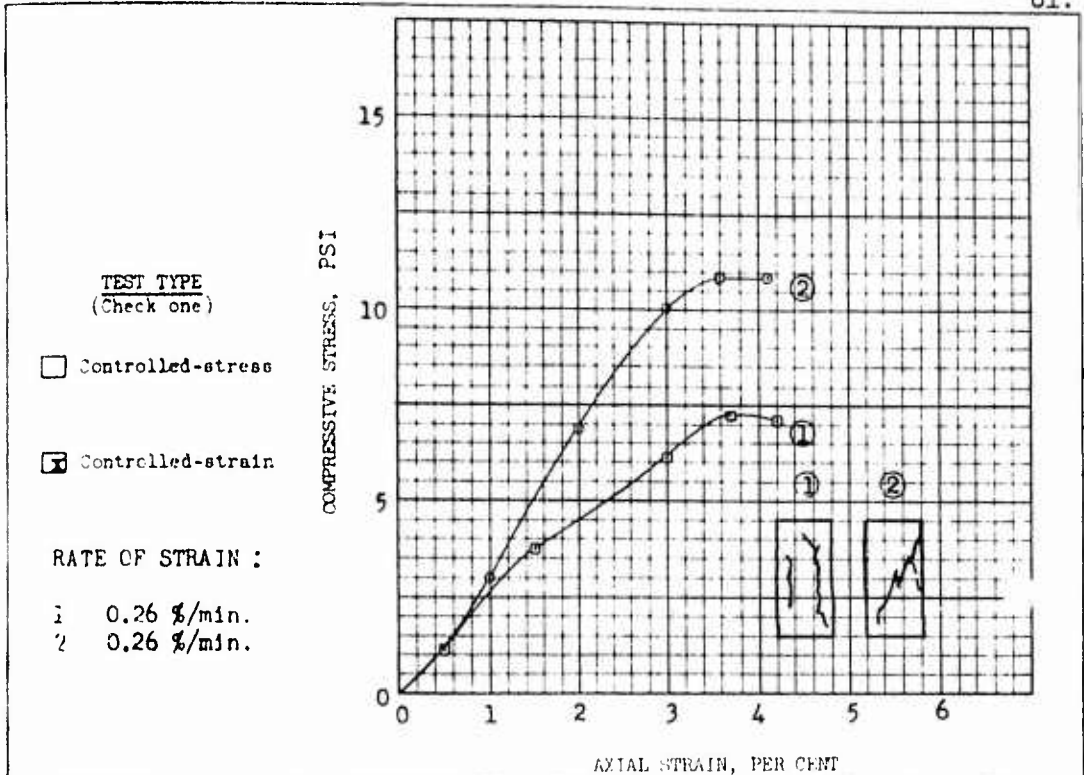
Figure B-5. Unconfined Compression Test Report



TEST TYPE (Check one)					
<input type="checkbox"/> Controlled-stress					
<input checked="" type="checkbox"/> Controlled-strain					
RATE OF STRAIN:					
1 0.25 %/min.					
2 0.25 %/min.					
Test No.		1	2		
Initial	Water content	w_o 33.4 %	33.0 %	%	%
	Void ratio	e_o 0.921	0.930		
	Saturation	S_o 97.2 %	95.1 %	%	%
	Dry density, lb/cu ft	γ_d 87.1	86.7		
Time to failure, min		t_f 11	9		
Unconfined compressive strength, PSI		q_u 9.73	8.92		
Undrained shear strength, PSI		s_u 4.86	4.46		
Sensitivity ratio		S_t			
Type of specimen UNDISTURBED		Dia 1 1.38, 2 1.39 in.		Height 1 & 2 3.00 in.	
Classification SILTY CLAY (CL), brown					
LL	—	PL	—	PI	—
Remarks G_s = 2.68 From Q Test		Project _____			
		Area _____			
		Boring No. 1-U		Sample No. 5	
		Depth, ft 16.1 C-1		Date 13 APRIL 1966	

Figure B-6. Unconfined Compression Test Report





Test No.		1	2		
Initial	Water content	w_o 30.9 %	30.4 %	%	%
	Void ratio	e_o 0.827	0.835		
	Saturation	S_o 100 %	98.7 %	%	%
	Dry density, lb/cu ft	γ_d 92.6	92.2		
Time to failure, min		t_f 14	14		
Unconfined compressive strength, PSI		q_u 7.22	10.87		
Undrained shear strength, PSI		u_u 3.61	5.43		
Sensitivity ratio		S_t			

Type of specimen **UNDISTURBED** Dia 1 1.37, 2 1.38 in. Height 1 & 2 3.00 in.

Classification **CLAYEY SILT (ML), brown**

LL PL PI

Remarks Gs = 2.71 From Q Test

Project _____

Area _____

Boring No. 1-U Sample No. 9

Depth, 16.5 ft Date 15 APRIL 1966

Figure B-8. Unconfined Compression Test Report

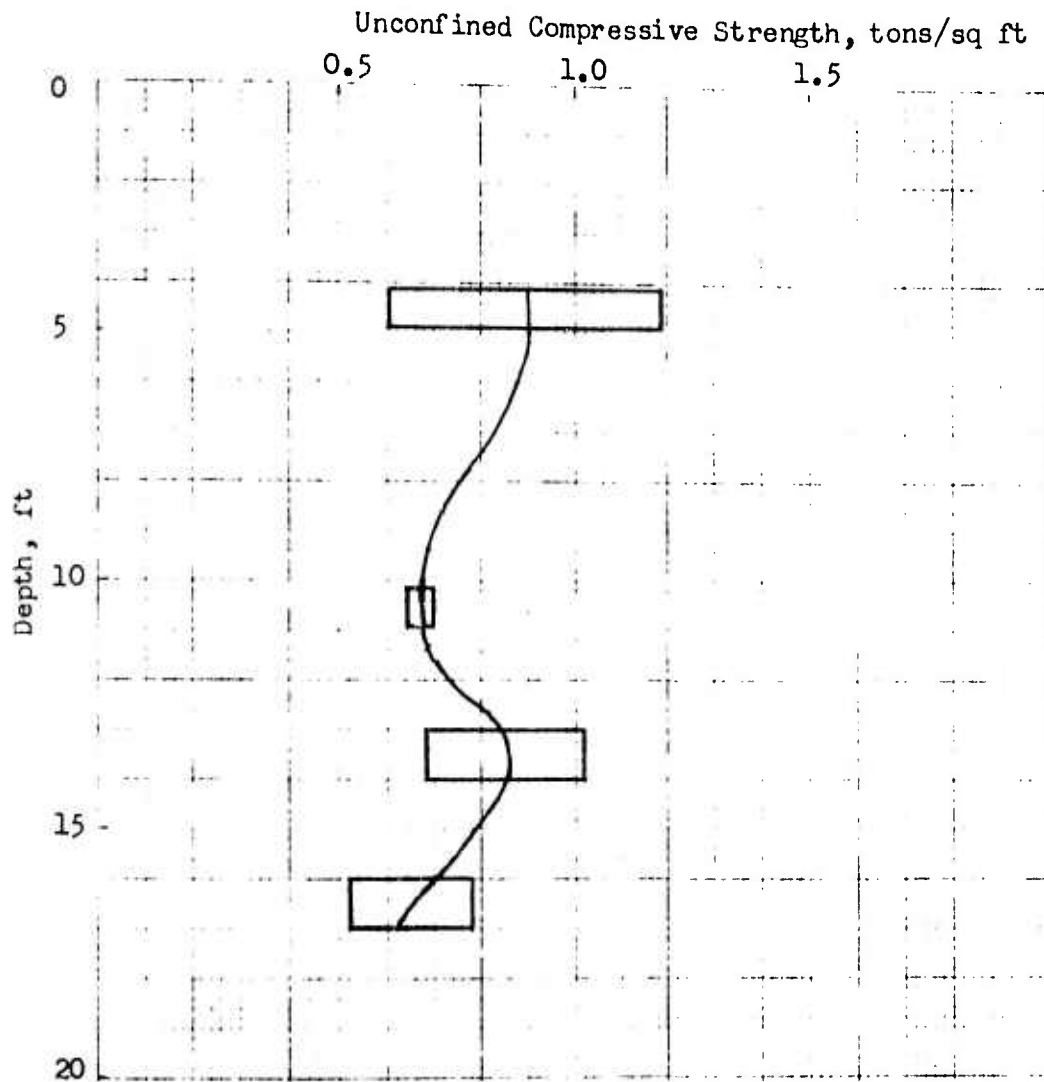
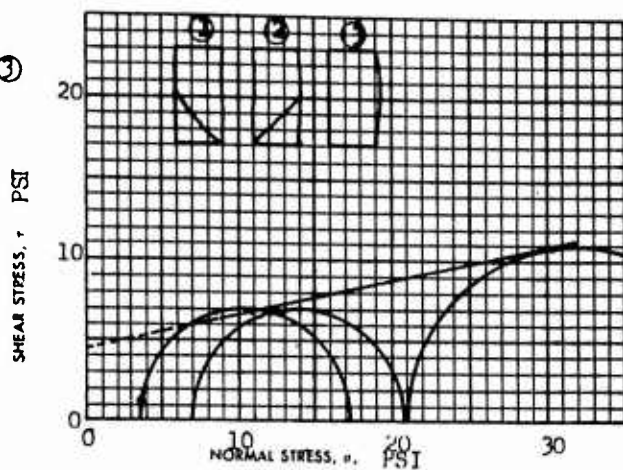
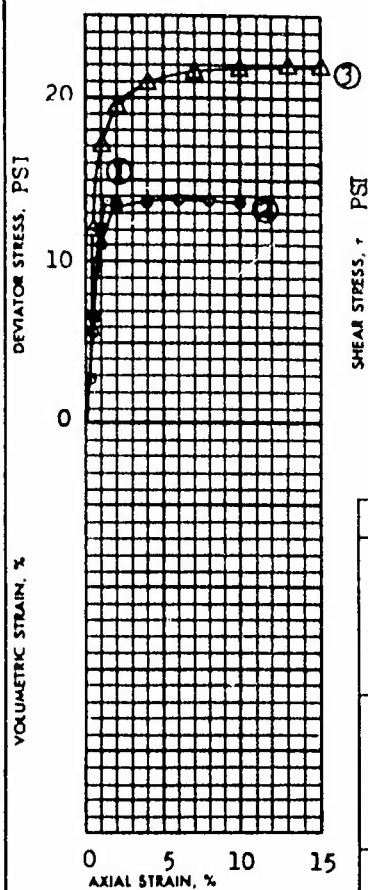


Figure B-9. Unconfined Compressive Strength Versus Depth



VOLUMETRIC STRAIN, %

0 5 10 15

SHEAR VALUES

$c = 12$
 $\tan \phi = 0.213$
 $c = 4.4$ PSI

TEST NO.		1	2	3	
INITIAL	WATER CONTENT	w_o 23.8 %	27.9 %	23.3 %	%
	VOID RATIO	e_o 0.847	0.828	0.838	
	SATURATION	s_o 75.3 %	74.1 %	74.5 %	%
	DRY DENSITY LB/CU FT	γ_d 90.6	91.5	91.0	
BEFORE TEST	WATER CONTENT	w_c	%	%	%
	SATURATION	s_c	%	%	%
	CONSOLIDATION PRESS., TONS/SQ FT	p_c			
	VOID RATIO	e_c			
FINAL	WATER CONTENT	w_f	%	%	%
	VOID RATIO	e_f			
MAJOR PRINCIPAL STRESS, PSI	σ_1	17.24	20.82	42.96	
MINOR PRINCIPAL STRESS, PSI	σ_3	3.5	7.0	21.0	
TIME TO FAILURE, MIN		6	24	52	
INITIAL DIAMETER, in.		1.39	1.39	1.39	
INITIAL HEIGHT, H_o , in.		3.00	3.00	3.00	

TYPE TEST 0

METHOD OF SATURATION

 CONTROLLED STRESS CONTROLLED STRAIN

TYPE OF SPECIMEN

UNDISTURBED

RATE OF STRAIN

0.25

0.25

0.25

1/8 MIN

CLASSIFICATION

SILTY CLAY (CL), brown, fissured

u

36

n

23

n

13

 $G_s = 2.68$ REMARKS Insufficient materialfor check tests

PROJECT

AREA

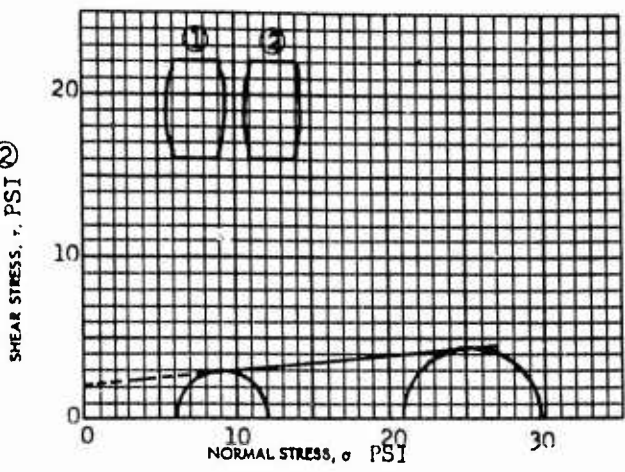
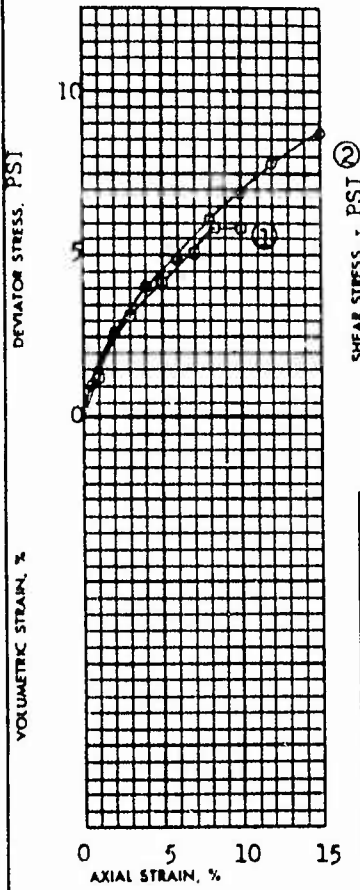
BORING NO. 1-U

SAMPLE NO. 2

DEPTH
EL 4.1 - 4.8'

DATE 21 APRIL 1966

Figure B-10. Triaxial Compression Test Report



VOLUMETRIC STRAIN, %

0 5 10 15

SHEAR VALUES

$\tau = 5$
 $\tan \phi = 0.087$
 $c = 2.2$ PSI

TEST NO		1	2		
INITIAL	WATER CONTENT	w _o 28.0 %	28.0 %	%	%
	VOID RATIO	e _o 0.797	0.791		
	SATURATION	s _o 94.2 %	94.9 %	%	%
	DRY DENSITY (LB/CU FT)	\gamma _d 93.1	93.4		
BEFORE TEST	WATER CONTENT	w _c %	%	%	%
	SATURATION	s _c %	%	%	%
	CONSOLIDATION PRESS., TONS/SQ FT	\sigma _c			
	VOID RATIO	e _c			
FINAL	WATER CONTENT	w _f %	%	%	%
	VOID RATIO	e _f			
MAJOR PRINCIPAL STRESS, PSI	\sigma ₁	12.06	29.67		
MINOR PRINCIPAL STRESS, PSI	\sigma ₃	6.2	21.0		
TIME TO FAILURE, MIN		33	60		
INITIAL DIAMETER, in.		1.38	1.37		
INITIAL HEIGHT, H _o , in.		3.00	3.00		

TYPE TEST Q CONTROLLED STRESS CONTROLLED STRAIN

METHOD OF SATURATION _____ RATE OF STRAIN 0.25 0.25 % / MIN

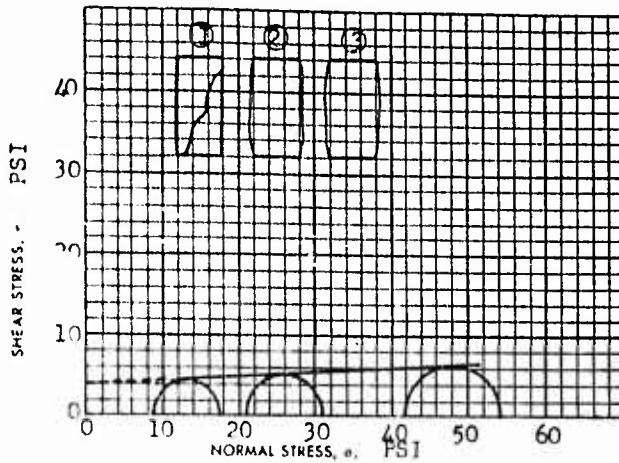
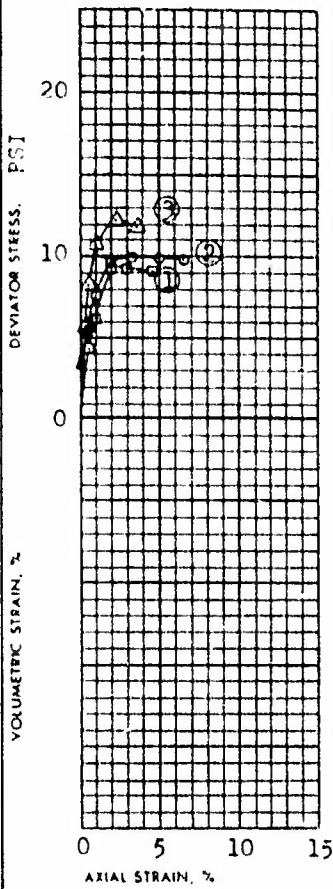
TYPE OF SPECIMEN UNDISTURBED CLASSIFICATION SILTY CLAY (CL), brown, contains voids

u 36 n 21 n 12 C_B = 2.68

REMARKS Some difficulty was encountered in trimming specimens due to voids. Insufficient material for test at 42 psi. C_B

PROJECT _____
 AREA _____
 BORING NO. 1-U SAMPLE NO. 3
 DEPTH 7.5' - 8.0' DATE 13 APRIL 1966

Figure B-11. Triaxial Compression Test Report



SHEAR VALUES
 $\phi = 3$
 $\tan \phi = 0.052$
 $c = 4.0$ PSI

TEST NO		1	2	3	
INITIAL	WATER CONTENT	w_o 32.5 %	32.4 %	32.3 %	%
	VOID RATIO	e_o 0.934	0.950	0.930	
	SATURATION	S_o 93.2 %	91.4 %	93.1 %	%
	DRY DENSITY	γ_d 86.5	85.8	86.7	LB/CU FT
BEFORE TEST	WATER CONTENT	w_c	%	%	%
	SATURATION	S_c	%	%	%
	CONSOLIDATION PRESS.	σ_c			TONS/SQ FT
	VOID RATIO	e_c			
FINAL	WATER CONTENT	w_f	%	%	%
	VOID RATIO	e_f			
MAJOR PRINCIPAL STRESS, PSI	σ_1	17.71	30.93	54.18	
MINOR PRINCIPAL STRESS, PSI	σ_3	8.5	21.0	42.0	
TIME TO FAILURE, MIN		8	13	9	
INITIAL DIAMETER, in.		1.39	1.40	1.39	
INITIAL HEIGHT, H_o , in.		3.00	3.00	3.00	

TYPE TEST Q
 METHOD OF SATURATION _____ CONTROLLED STRESS CONTROLLED STRAIN

TYPE OF SPECIMEN UNDISTURBED RATE OF STRAIN 0.25 0.25 0.25 %/MIN

CLASSIFICATION SILTY CLAY (CL), brown

" 36 " 23 " 13 $G_B = 2.68$

REMARKS	PROJECT	
	AREA	
	BORING NO. <u>1-U</u>	SAMPLE NO. <u>5</u>
	DEPTH EL <u>10.1</u>	DATE <u>16 APRIL 1966</u>

Figure B-12. Triaxial Compression Test Report

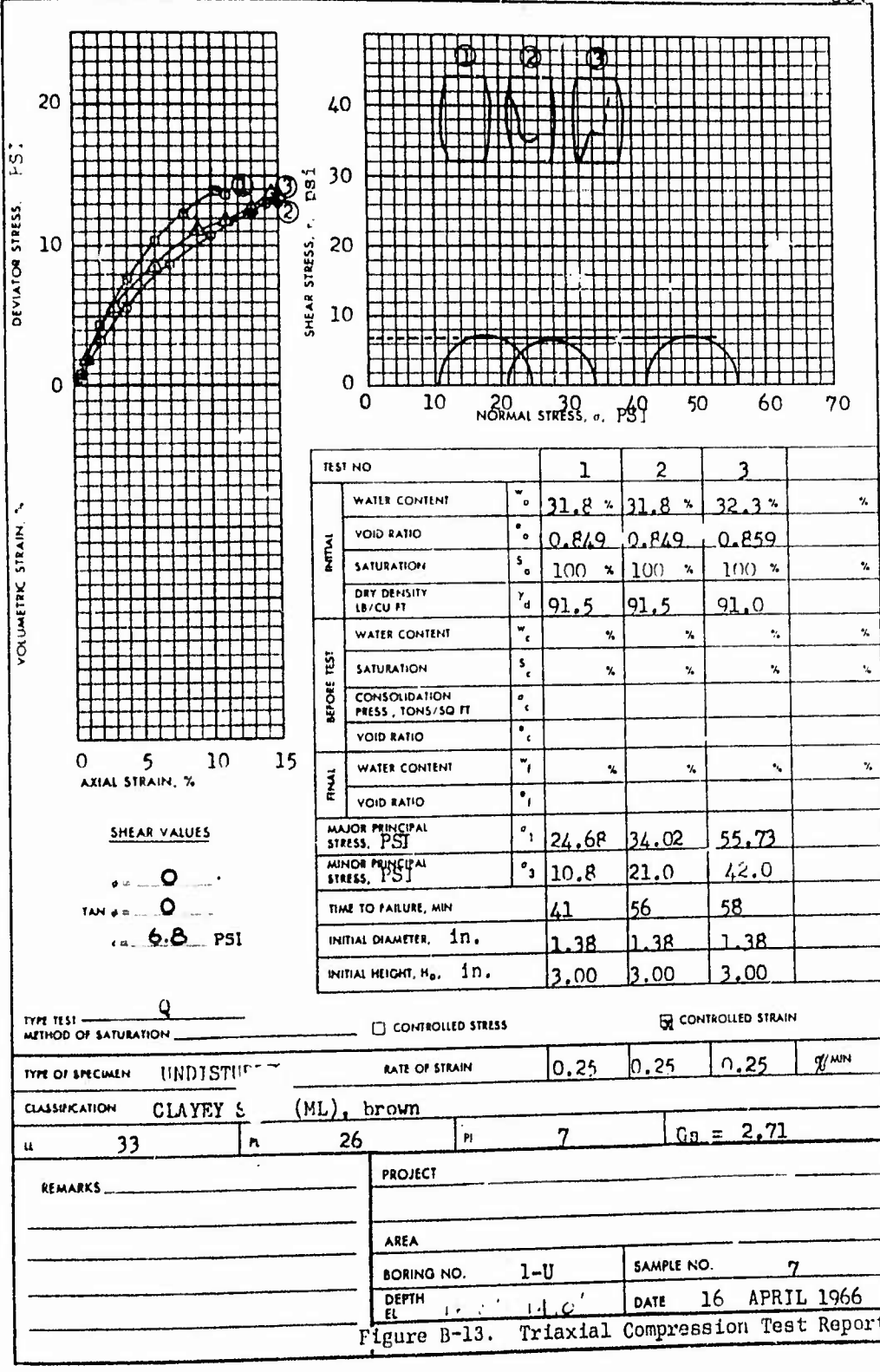
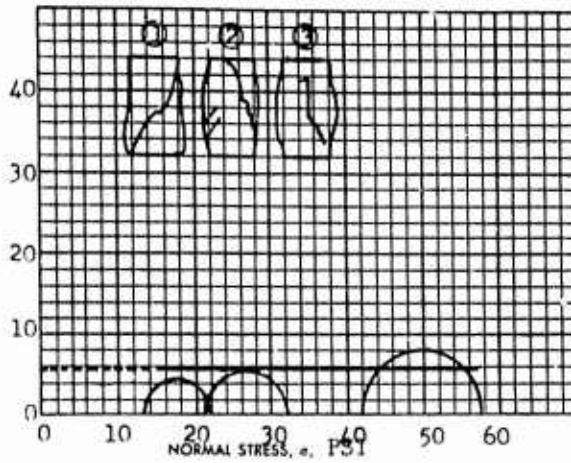
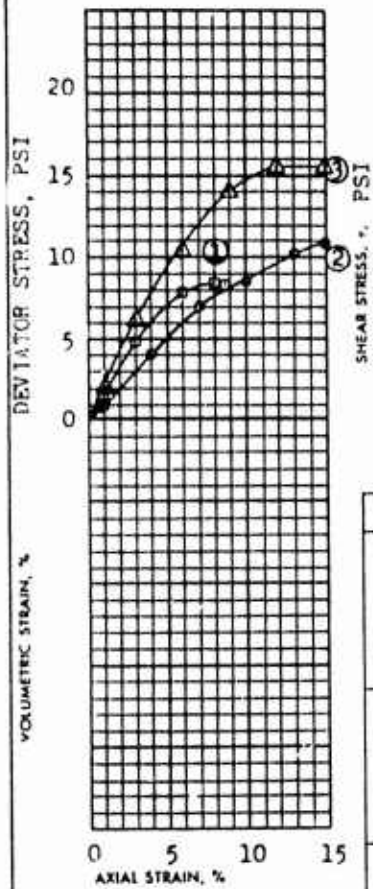


Figure B-13. Triaxial Compression Test Report



SHEAR VALUES

$\sigma = 0$
 $\tau = 0$
 $c = 5.8 \text{ PSI}$

TEST NO		1	2	3	
INITIAL	WATER CONTENT %	30.9	30.9	30.4	%
	VOID RATIO	0.833	0.819	0.798	
	SATURATION	100	100+	100+	%
	DRY DENSITY LB/CU FT	92.3	93.0	94.1	
BEFORE TEST	WATER CONTENT %				%
	SATURATION				%
	CONSOLIDATION PRESS., TONS/SQ FT				
	VOID RATIO				
FINAL	WATER CONTENT %				%
	VOID RATIO				
MAJOR PRINCIPAL STRESS, PSI	σ_1	21.64	31.88	57.60	
MINOR PRINCIPAL STRESS, PSI	σ_3	13.2	21.0	42.0	
TIME TO FAILURE, MIN		32	60	48	
INITIAL DIAMETER, in.		1.38	1.38	1.38	
INITIAL HEIGHT, in.		3.00	3.00	3.00	

TYPE TEST Q CONTROLLED STRESS CONTROLLED STRAIN

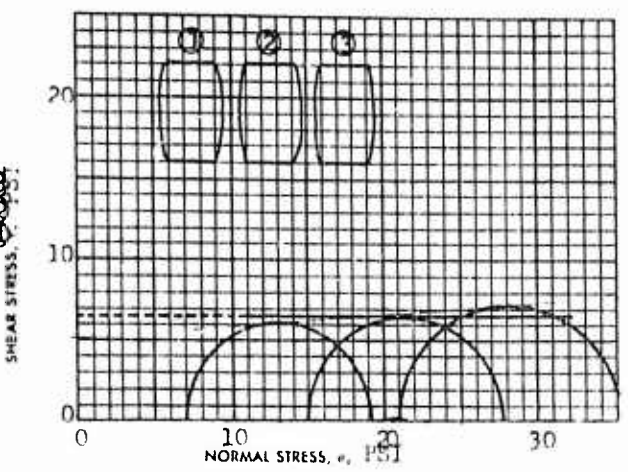
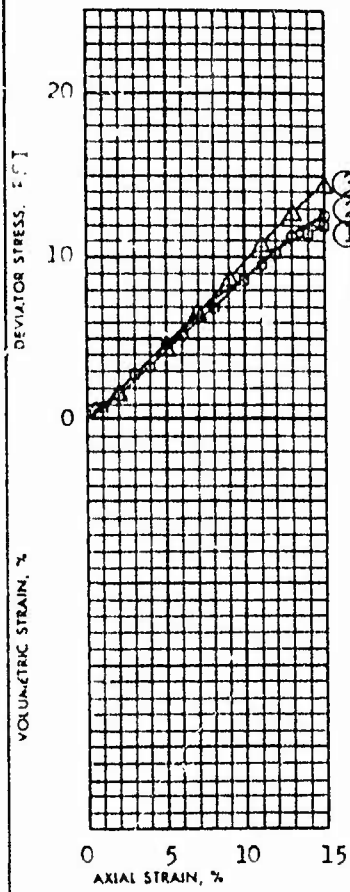
TYPE OF SPECIMEN UNDISTURBED RATE OF STRAIN 0.25 0.25 0.25 %/MIN

CLASSIFICATION CLAYEY SILT(M.), brown

31 25 6 Q_B = 2.71

REMARKS	PROJECT	
	AREA	
	BORING NO. <u>1-U</u>	SAMPLE NO. <u>9</u>
	DEPTH EL <u>16.0-16.1</u>	DATE <u>15 APRIL 1966</u>

Figure B-14. Triaxial Compression Test Report



SHEAR VALUES

$\phi = 0$

$\tan \phi = 0$

$c = 6.5$ PSI

TEST NO		1	2	3	
INITIAL	WATER CONTENT w_o	29.5 %	29.6 %	29.0 %	%
	VOID RATIO e_o	0.802	0.772	0.759	
	SATURATION s_o	99.7 %	100+ %	100+ %	%
	DRY DENSITY γ_d LB/CU FT	93.9	95.5	96.2	
BEFORE TEST	WATER CONTENT w_c	%	%	%	%
	SATURATION s_c	%	%	%	%
	CONSOLIDATION PRESS. TONS/SQ FT p_c				
	VOID RATIO e_c				
FINAL	WATER CONTENT w_f	%	%	%	%
	VOID RATIO e_f				
MAJOR PRINCIPAL STRESS, PSI σ_1	19.02	27.66	35.31		
MINOR PRINCIPAL STRESS, PSI σ_3	7.0	15.0	21.0		
TIME TO FAILURE, MIN	60	60	60		
INITIAL DIAMETER, in.	1.38	1.38	1.39		
INITIAL HEIGHT, H_o , in.	3.00	3.00	3.00		

TYPE TEST Q CONTROLLED STRESS CONTROLLED STRAIN

METHOD OF SATURATION _____

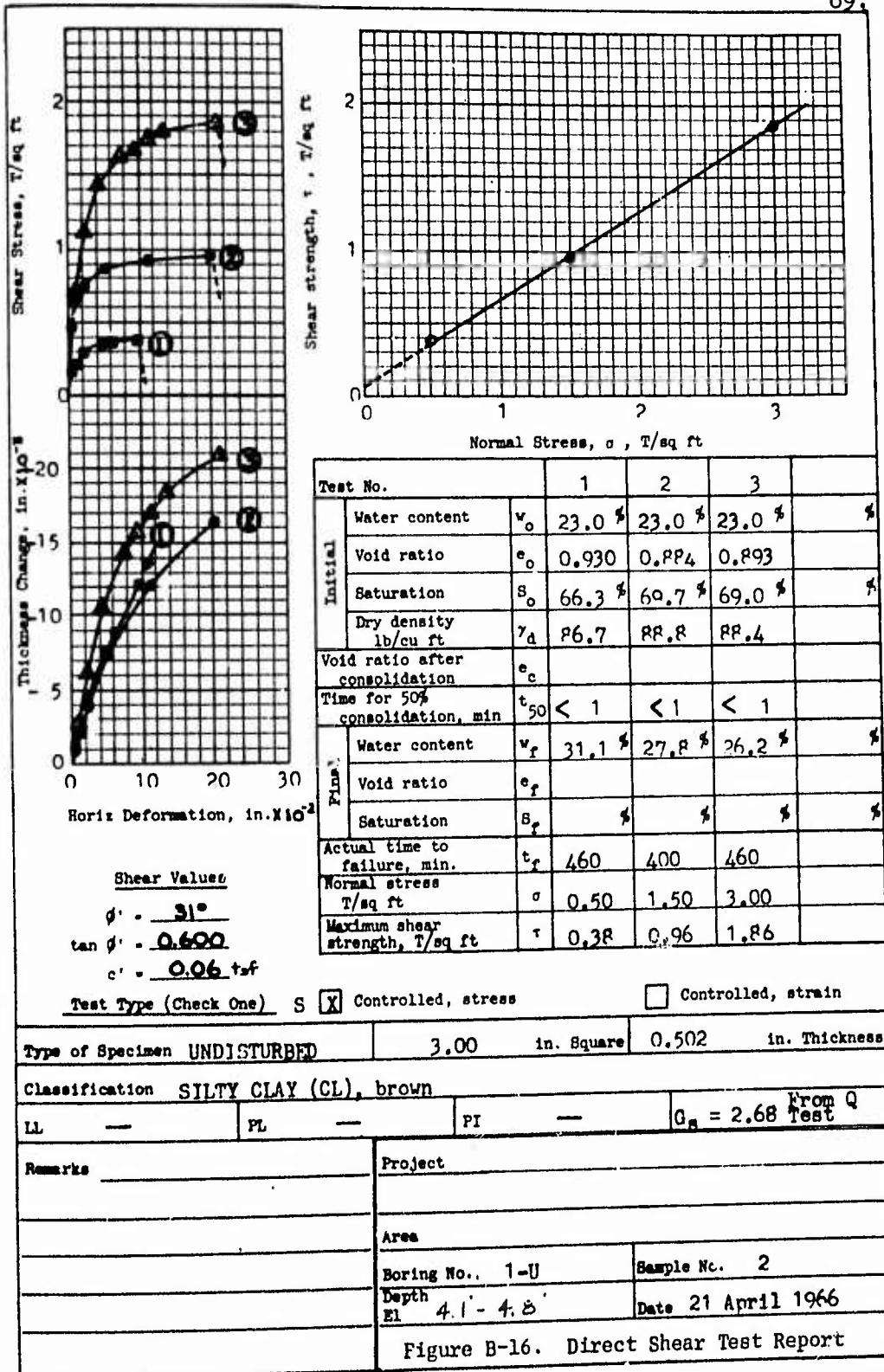
TYPE OF SPECIMEN UNDISTURBED RATE OF STRAIN 0.25 0.25 0.25 %/MIN

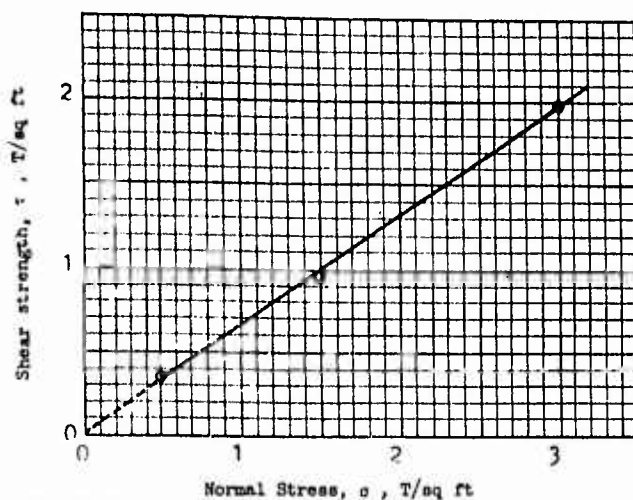
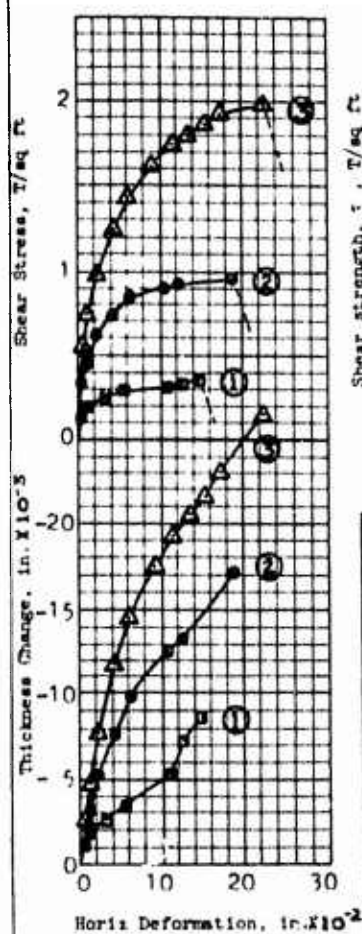
CLASSIFICATION Silt (ML), Grey

$u = 28$ $n = 24$ $n = 4$ $G_s = 2.71$

REMARKS _____	PROJECT _____	
	AREA _____	
	BORING NO. <u>1-U</u>	SAMPLE NO. <u>10</u>
	DEPTH EL. <u>110-1</u>	DATE <u>20 APRIL 1966</u>

Figure B-15. Triaxial Compression Test Report





Horiz. Deformation, in. $\times 10^{-2}$

Shear Values

$\phi' = 33^\circ$

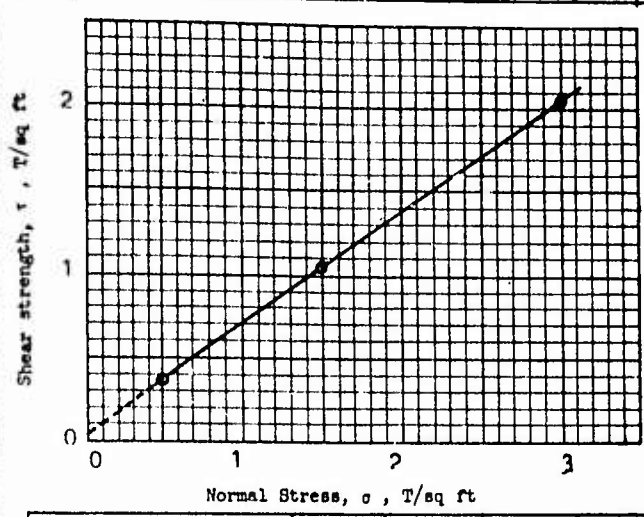
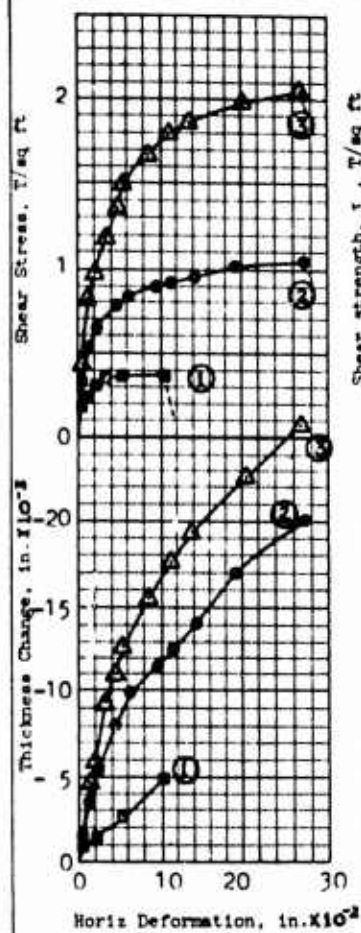
$\tan \phi' = 0.653$

$c' = 0.02$

Test Type (Check One) S Controlled, stress Controlled, strain

Test No.	1	2	3		
Initial	Water content	v_o 31.9%	32.5%	31.5%	%
	Void ratio	e_o 0.964	0.992	0.985	
	Saturation	S_o 88.7%	87.8%	85.7%	%
	Dry density	γ_d 85.2	84.0	84.3	lb/cu ft
Void ratio after consolidation	e_c				
Time for 50% consolidation, min	t_{50}	< 1	< 1	< 1	
Final	Water content	v_f 32.9%	30.5%	28.4%	%
	Void ratio	e_f			
	Saturation	S_f	%	%	%
Actual time to failure, min.	t_f	380	400	460	
Normal stress T/sq ft	σ	0.50	1.50	3.00	
Maximum shear strength, T/sq ft	τ	0.35	0.96	1.98	

Type of Specimen	UNDISTURBED	3.00 in. Square	0.502 in. Thickness
Classification	SILTY CLAY (CL), brown		
LL	—	PL	—
PT	—	$G_s = 2.68$ From Q Test	
Remarks	Project		
	Area		
	Boring No. 1-U	Sample No. 5	
	Depth El. 10.1 - 10.1'	Date 23 April 1966	
	Figure B-17. Direct Shear Test Report		

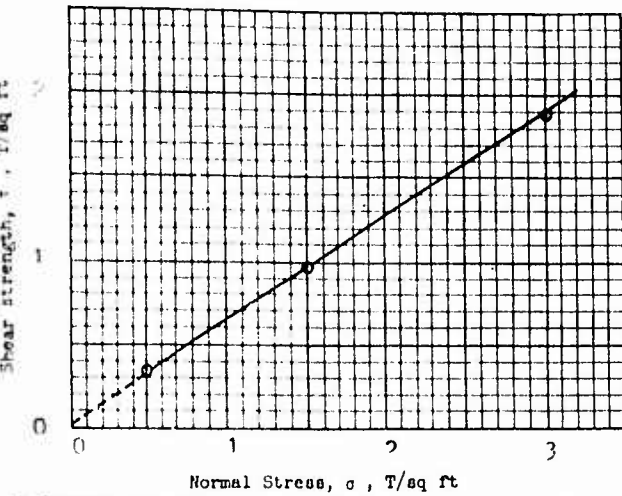
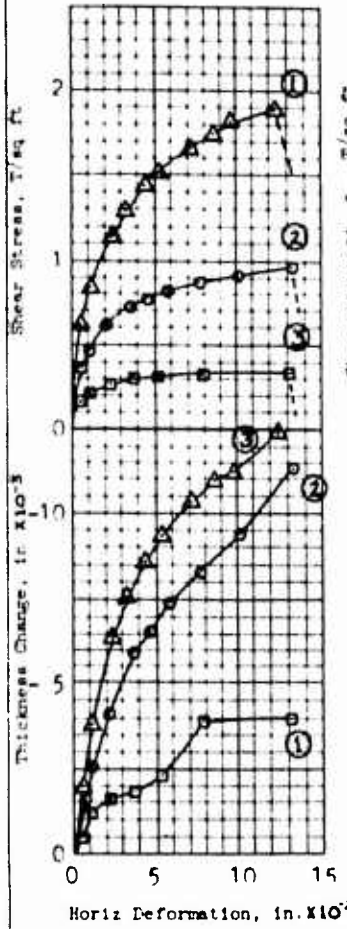


Shear Values
 $\phi' = 34^\circ$
 $\tan \phi' = 0.680$
 $c' = 0.03$

Test No.		1	2	3	
Initial	Water content w_o	31.0 %	30.9 %	31.5 %	%
	Void ratio e_o	0.918	0.946	0.951	
	Saturation S_o	91.2 %	88.2 %	89.4 %	%
	Dry density γ_d lb/cu ft	87.9	86.6	86.6	
Void ratio after consolidation e_c					
Time for 50% consolidation, min t_{50}		< 1	< 1	< 1	
Final	Water content w_f	32.7 %	31.2 %	29.6 %	%
	Void ratio e_f				
	Saturation S_f	%	%	%	%
Actual time to failure, min t_f		400	360	400	
Normal stress T/sq ft σ		0.50	1.50	3.00	
Maximum shear strength, T/sq ft τ		0.37	1.05	2.05	

Test Type (Check One) S Controlled, stress Controlled, strain

Type of Specimen	UNDISTURBED	3.00 in. Square	0.502 in. Thickness
Classification CLAYEY SILT (ML), brown			
LL	—	PL	—
PI	—	$G_s = 2.71$ From Q Test	
Remarks	Project		
	Area		
	Boring No. 1-U	Sample No. 7	
	Depth El 12.0' - 14.0'	Date 26 April 1966	
Figure B-18. Direct Shear Test Report			



Shear Values
 $\phi' = 32^\circ$
 $\tan \phi' = 0.630$
 $c' = 0.02$

Test No.		1	2	3	
Initial	Water content	w_o 28.6 %	29.9 %	30.4 %	%
	Void ratio	e_o 0.861	0.890	0.882	
	Saturation	S_o 90.0 %	91.0 %	93.4 %	%
	Dry density	γ_d 90.9	89.5	89.9	lb/cu ft
Void ratio after consolidation		e_c			
Time for 50% consolidation, min		t_{50} < 1	< 1	< 1	
Final	Water content	w_f 31.5 %	29.9 %	27.6 %	%
	Void ratio	e_f			
	Saturation	S_f	%	%	%
Actual time to failure, min.		t_f 300	280	380	
Normal stress T/sq ft		σ 0.50	1.50	3.00	
Maximum shear strength, T/sq ft		τ 0.34	0.97	1.88	

Test Type (Check One) S Controlled, stress Controlled, strain

Type of Specimen	UNDISTURBED	3.00	in. Square	0.502	in. Thickness
Classification	CLAYEY SILT (ML), brown				
LL	_____	PL	_____	PI	_____
				$G_s = 2.71$	from Q Test
Remarks	Project _____				
Area _____					
Boring No. 1-11			Sample No. 9		
Depth El 120 - 161			Date 4 May 1966		
Figure B-19. Direct Shear Test Report					

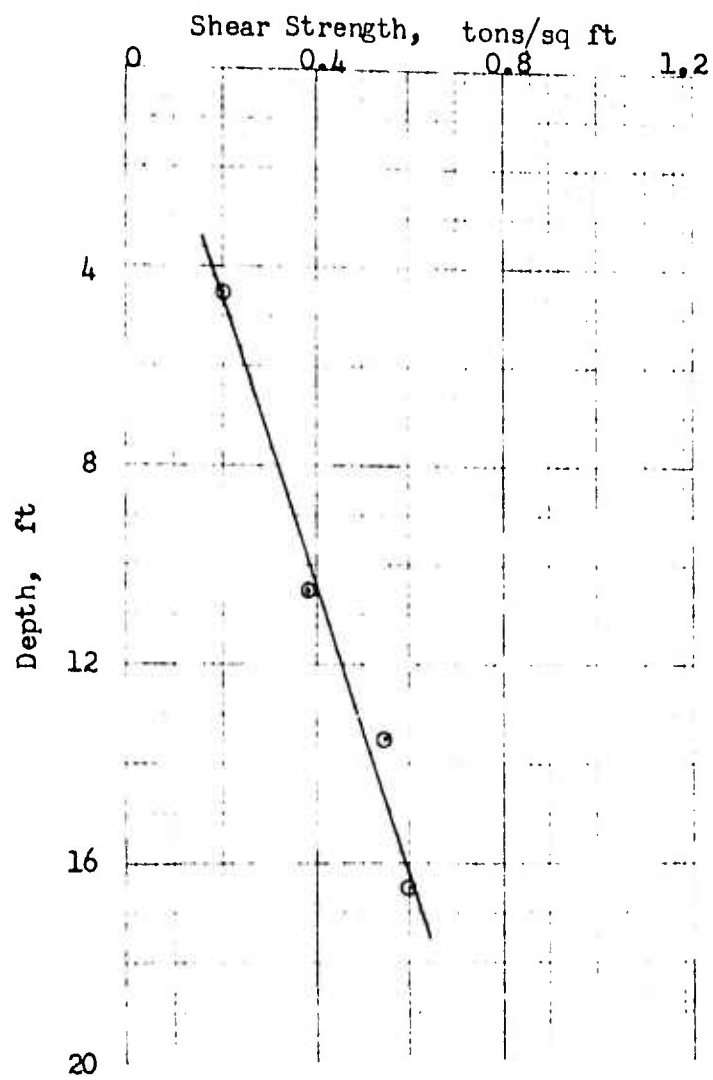


Figure B-20. Shear Strength Versus Depth

APPENDIX C
DYNAMIC LABORATORY AND FIELD TESTS

APPENDIX C

DYNAMIC LABORATORY AND FIELD TESTS

Dynamic Laboratory Tests

Vibratory tests were conducted on four undisturbed soil specimens in the WES soil dynamics laboratory to determine dynamic moduli for comparison with those moduli determined in the field. The test procedure and apparatus are described in detail in U. S. Army Material Command Cold Regions Research and Engineering Laboratory (CRREL) Technical Report No. 173, "Measurement of Complex Moduli and Damping of Soils under Dynamic Loads: Laboratory Test Apparatus, Procedure, and Analysis."

The test procedure is referred to as the resonant column technique. When this method is used, the sample response is monitored for a range of frequencies in both the longitudinal and torsional modes of excitation to determine the resonant frequency of the soil column. The fundamental resonant frequency of the sample is assumed to be the lowest frequency at which the ratio of the acceleration at the top of the sample to that at the bottom of the sample is a maximum, and the phase angle between the sine wave at the top and bottom of the sample approaches 90 deg. The desired dynamic soil properties can then be found by means of the following relations:

$$G = 16f_t^2 L^2 \rho \quad (C1)$$

$$E = 16f_l^2 L^2 \rho \quad (C2)$$

$$v_s = 4f_t L \quad (C3)$$

$$v_1 = rf_1 L \quad (C4)$$

$$v_c = v_1 \left(\frac{1 - \nu}{(1 + \nu)(1 - 2\nu)} \right)^{1/2} \quad (C5)$$

$$\nu = \frac{E}{2G} - 1 \quad (C6)$$

where

G = shear modulus

f_t = fundamental torsional resonant frequency

L = length of the sample

ρ = mass density of the sample

E = compression (Young's) modulus

f_1 = fundamental longitudinal resonant frequency

v_s = shear wave velocity

v_1 = longitudinal wave velocity

v_c = compression wave velocity

ν = Poisson's ratio

The equations given above are simplified versions that consider the elastic case only. Equations are available, however, that consider the soil to be a viscoelastic material and yield a value for the damping property and values of moduli and velocity that account for material damping.

The results obtained from this type of laboratory test, like those obtained from any other conventional laboratory tests, are dependent upon the quality and representativeness of the sample. Ideally, it is desirable to have undisturbed samples; if the samples

are remolded, the water content and density should be the same as those of the undisturbed material. Also, the stress conditions for either the undisturbed or remolded sample should be the same as those for the material in situ. Under most circumstances, it is generally impossible to obtain an accurate estimate of the state of stress of either in situ material or remolded specimens. Therefore, in situ residual stresses generally are not reproduced in remolded samples, and the confining pressure for the laboratory specimens can only approximate that for the in situ material.

The test specimens, 4 in. in diam and about 15 in. long, were inclosed in a rubber membrane and placed upright in a triaxial chamber. Confining pressure was applied by gas pressure in the chamber. The tests were conducted by vibrating each specimen from the bottom in longitudinal and torsional modes and measuring the amplitudes of the induced waves by means of accelerometers attached to the bottom and top of specimen. Velocities and moduli were then calculated from the ratio of the amplitudes measured by the top and bottom accelerometers, the specimen length, and wave frequency at resonance.

Four undisturbed soil specimens from hole 1 (described in Appendix B) were tested at confining pressures representative of the pressure of overburden material to a depth of about 22 ft from the ground surface.

The data obtained from the dynamic laboratory tests are plotted in figure C-1 as the laboratory moduli versus depth. The average

laboratory shear moduli for samples from the ground surface to a depth of 22 ft ranged from 4200 to 8000 psi. Poisson's ratio varied from 0.37 to 0.49.

Field Tests

Refraction Seismic Tests

Refraction seismic tests were conducted to determine the compression-wave velocity of the soil in the test area. The compression-wave velocity and the shear-wave velocity are used together in determining Poisson's ratio and the elastic moduli of the soil. This method is outlined in Appendix A.

Four seismic lines run in the test area, shown in figure C-2, were oriented in north, south, east, and west directions. A geophone was placed on the ground surface near the boreholes, and a signal was produced by striking a circular steel plate with a sledgehammer. The steel plate was placed on the ground at various distances measured from the geophone, and the test repeated. The compression-wave velocity was computed from the time measured for the first wave arrival and the distance of the geophone from the energy source.

Compression-wave velocity data for the traverses are shown in table C-1. The average velocity was about 1060 fps for an average depth of about 13 ft. The average compression-wave velocity in the second zone was approximately 4600 fps. A selected plot of time versus distance for a refraction seismic traverse is shown in

figure C-3. Seismic traverses conducted during other tests near the test area indicated that the limestone layer was at a depth of about 125 ft and had a compression-wave velocity of about 10,000 fps.

Vibration Test

A low-frequency, hydraulic motor-driven, eccentric-mass vibrator with a maximum force level of 10,000 lbs and the capability of producing sinusoidal motion in the vertical direction within a frequency range of 1 - 50 Hz was used in determining the shear-wave velocities through the soil in accordance with the procedure outlined in Appendix A. A total of four vibration traverses, oriented north, south, east, and west, were conducted in the test area. Figure C-2 shows the relative location of the boreholes and vibrator and the direction of the traverses.

The shear-wave velocity data obtained from the four traverses are shown in table C-1 and are plotted versus depth in figure C-4. The data indicated that the average shear wave velocity increased very slightly from about 460 to 600 fps between about 5 to 30 ft.

Shear and Compression Moduli

Using the field data and the procedures described in Appendix A, shear and compression (Young's) moduli were calculated. These data are tabulated in table C-1 and are plotted versus depth in figure C-5. The average shear moduli increased uniformly from approximately 5500 to 9000 psi between depths of about 5 to 30 ft. The average compression moduli increased from about 15,000 psi at a depth of 5 ft to

80.

about 27,000 psi at a depth of 30 ft. Poisson's ratios ranged from 0.36 to 0.50.

Table C-1

In Situ Soil Tests, WES Test Area

Depth ft	Material Condition			Vibration Tests			Seismic Tests			Poisson's Ratio		Modulus $\text{psi} \times 10^3$		
	Description	Type of Deposition	Wet Unit Weight lb/cu ft	Water Content %	Line No.	Frequency f_v , cps	Wave Length λ_v , ft	Wave Velocity V_v , fps	Depth ft	Line No.	Wave Velocity V_s , fps	Depth ft	Shear Compression E	Modulus Compression E
0-18.5	Silty clay	Aeolian	115	27.2	V-1	50	9.5	486	4.8	S-1	1200	0-14.7	5.7	16.0
18.5-36	Clayey silt					45	10.4	470	5.2				5.5	15.3
						40	11.7	470	5.8				5.5	15.3
						35	13.2	460	6.6				5.2	14.7
						30	15.0	450	7.5		4800	14.7+	5.0	15.1
						25	19.7	495	9.9				6.1	18.2
						23	26.2	525	13.1				6.8	20.5
						15	35.5	530	17.8				7.0	20.9
						12	44.5	530	22.3				7.0	20.9
						10	61.0	610	30.5				9.2	27.7
					V-2	50	8.8	440	4.4	S-2	1000	0-11.6	4.8	13.4
						45	10.4	470	5.2				5.5	15.3
						40	11.8	470	5.9		4200	11.6+	5.5	15.3
						35	14.8	520	7.4				6.7	20.1
						30	15.5	470	7.8				5.5	16.4
						25	18.2	460	9.1				5.2	15.7
						20	26.4	530	13.2				7.0	20.9
						15	35.0	530	17.5				7.0	20.9
						12	46.8	560	23.4				7.8	23.3
						10	60.0	600	30.0				8.9	26.8
					V-3	50	9.3	465	4.7	S-3	1060	0-12.5	5.4	15.0
						45	10.4	470	5.2				5.5	15.3
						40	11.8	470	5.9				5.5	15.3
						35	13.2	460	6.6		4100	12.5-	5.2	15.7
						30	17.0	510	8.5				6.4	19.3
						25	20.0	500	10.0				6.2	18.6
						20	24.2	480	12.1				5.7	17.1
						18	26.2	470	13.1				5.5	16.4
						15	33.3	500	16.6				6.2	18.6
						13	41.0	535	20.5				7.1	21.3
						12	46.0	560	23.0				7.8	23.3
						10	56.0	560	28.0				7.8	23.3

(Continued)

Table C-1 (Concluded)

Depth ft	Material Condition			Vibration Tests				Seismic Tests						
	Description	Type of Deposition	Wet Unit Weight lb/cu ft	Water Con- tent %	Line No.	Pre- quency f, Hz	Wave Length λ , ft	Wave Velocity V, fps	Depth ft	Line No.	Wave Velocity V, fps	Depth ft	Poisson's Ratio ν	Modulus, psi $\times 10^3$
														Compression E
0-18.5	Silty clay	Aeolian	115	27.2	V-4	50	9.3	465	4.7	S-4	1000	0-13.2	0.36	14.5
18.5-30	Clayey silt					45	10.2	460	5.1				0.36	14.2
						40	11.6	465	5.8				0.36	14.5
						35	13.6	475	6.8		5000	13.2+	0.50	16.8
						30	16.0	480	8.0				0.50	17.1
						25	21.2	508	10.6				0.50	19.2
						20	25.0	500	12.5				0.50	18.6
						18	29.8	536	14.9				0.50	21.4
						15	34.0	510	17.0				0.50	19.3
						13	41.5	541	20.7				0.50	21.8
						12	49.5	595	24.7				0.50	26.3
						10	61.4	614	30.7				0.50	28.0

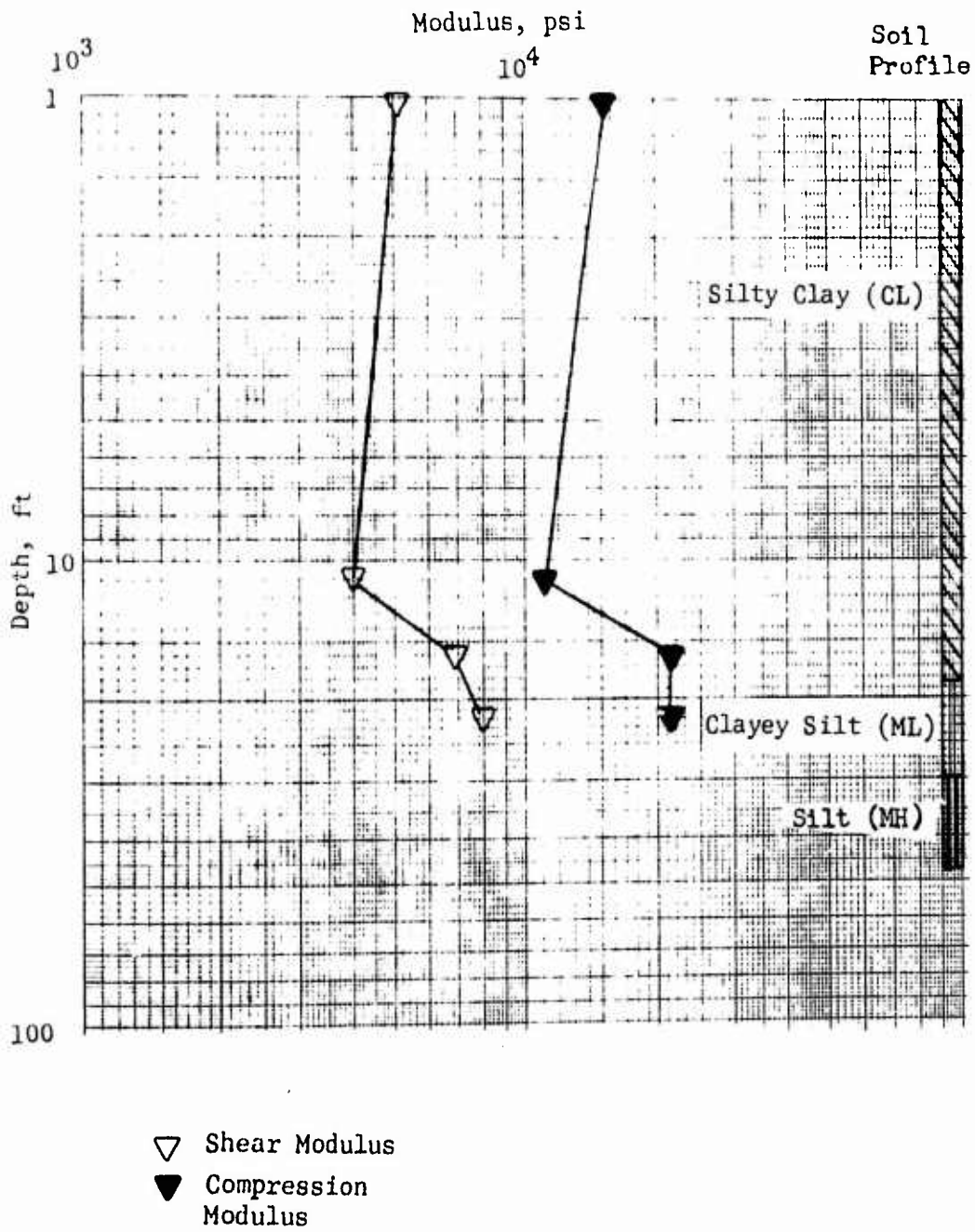


Figure C-1. Laboratory Shear and Compression Modulus Versus Depth and Soil Profile

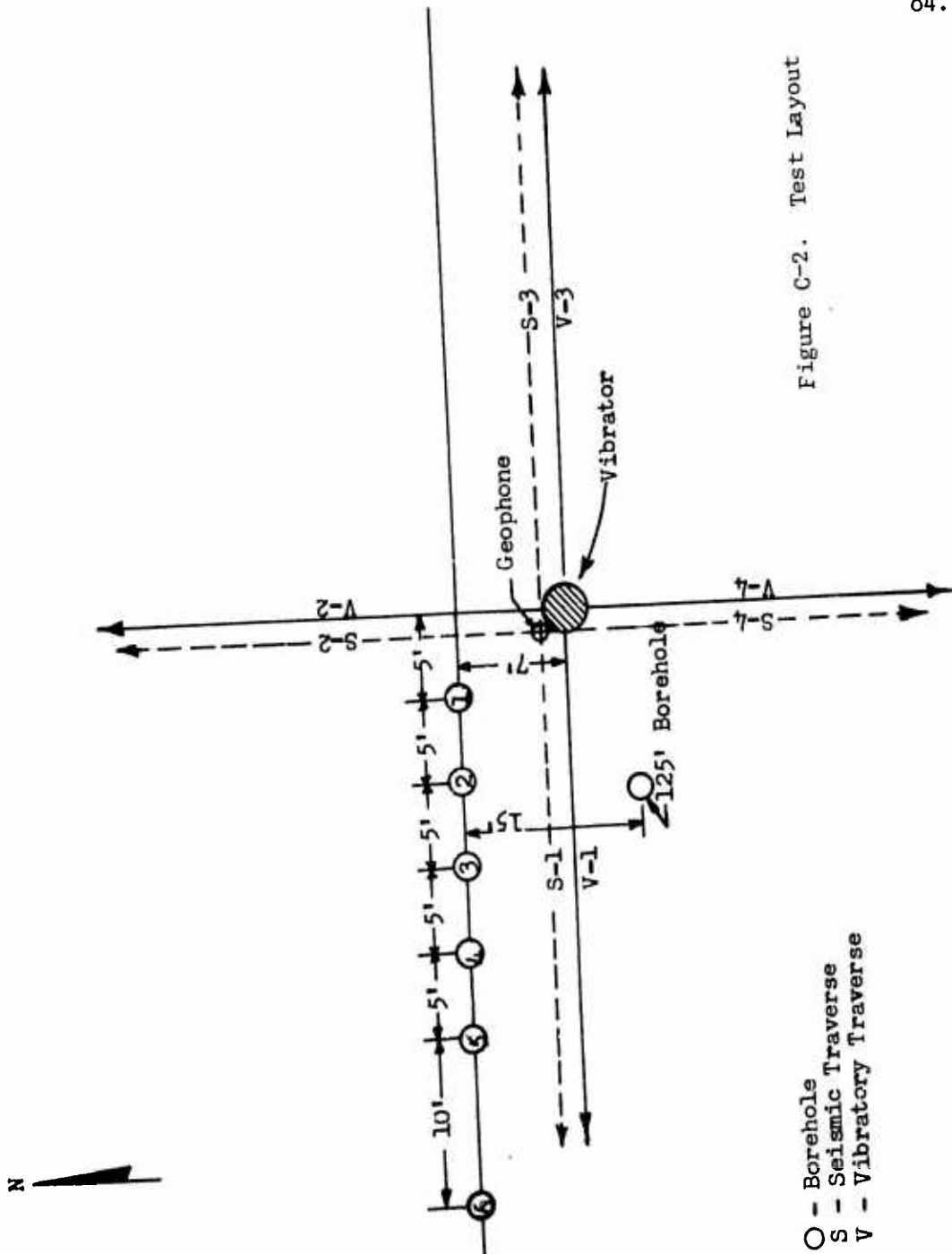


Figure C-2. Test Layout

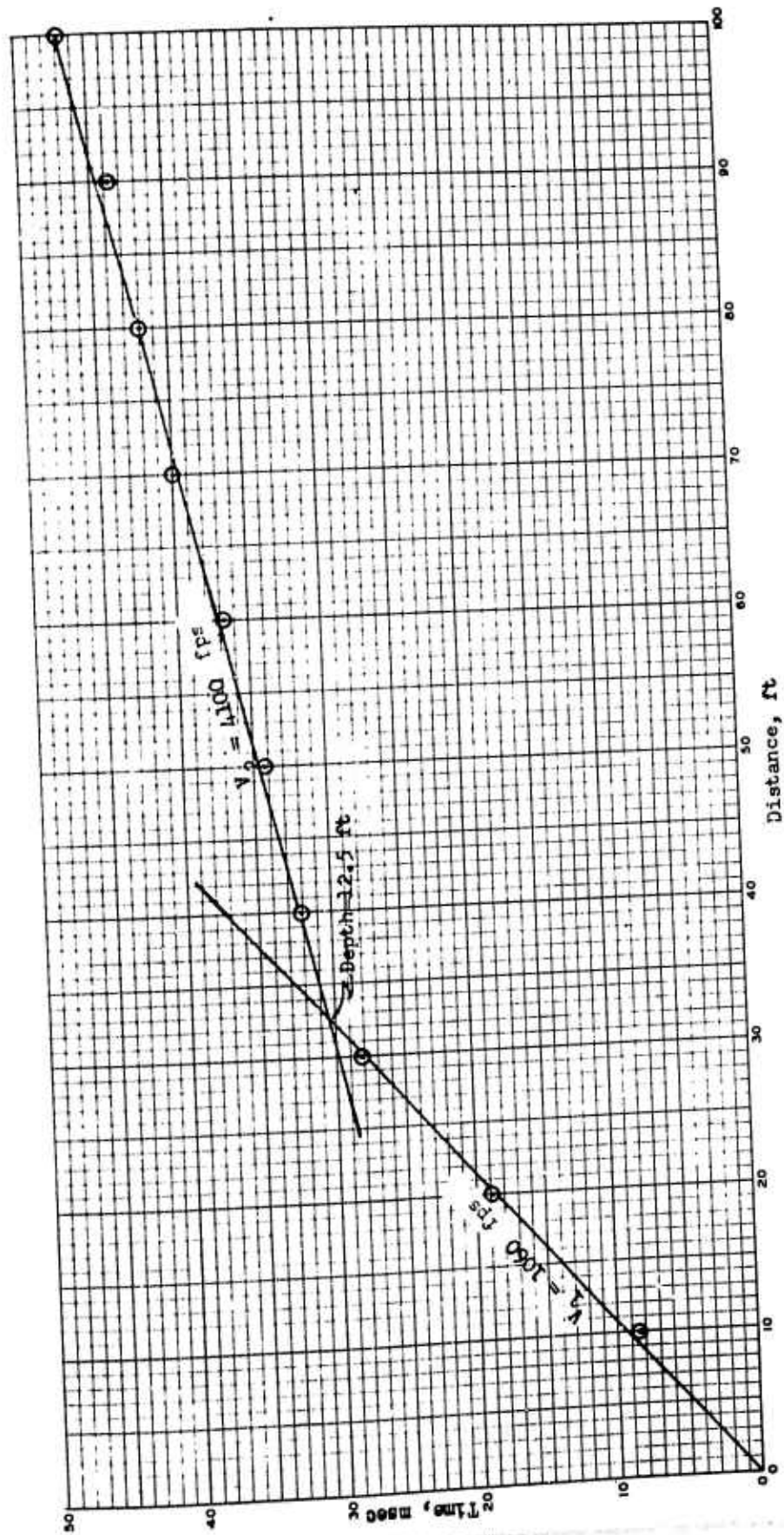


Figure C-3. Typical Refraction Seismic Traverse

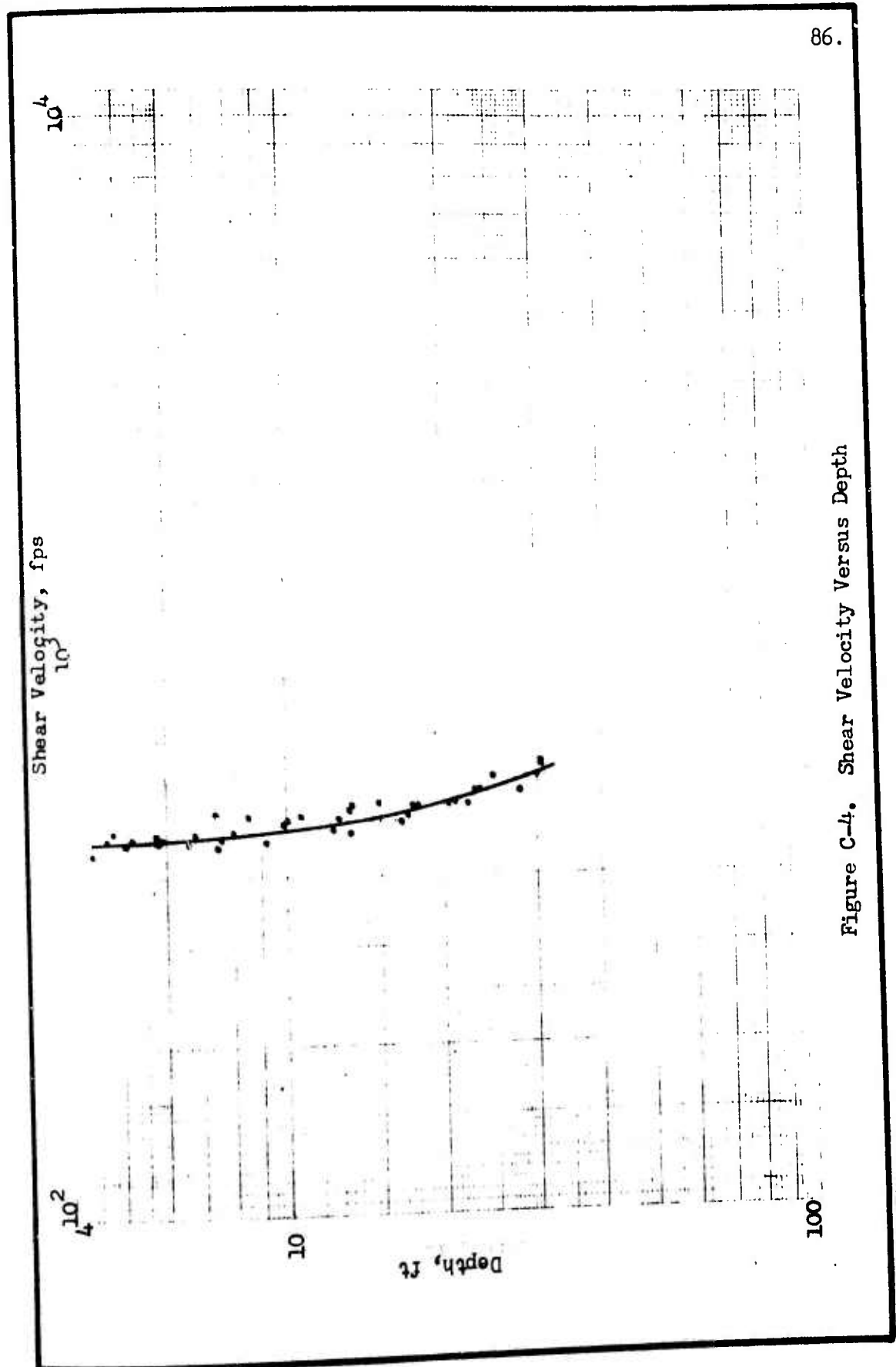


Figure C-4. Shear Velocity Versus Depth

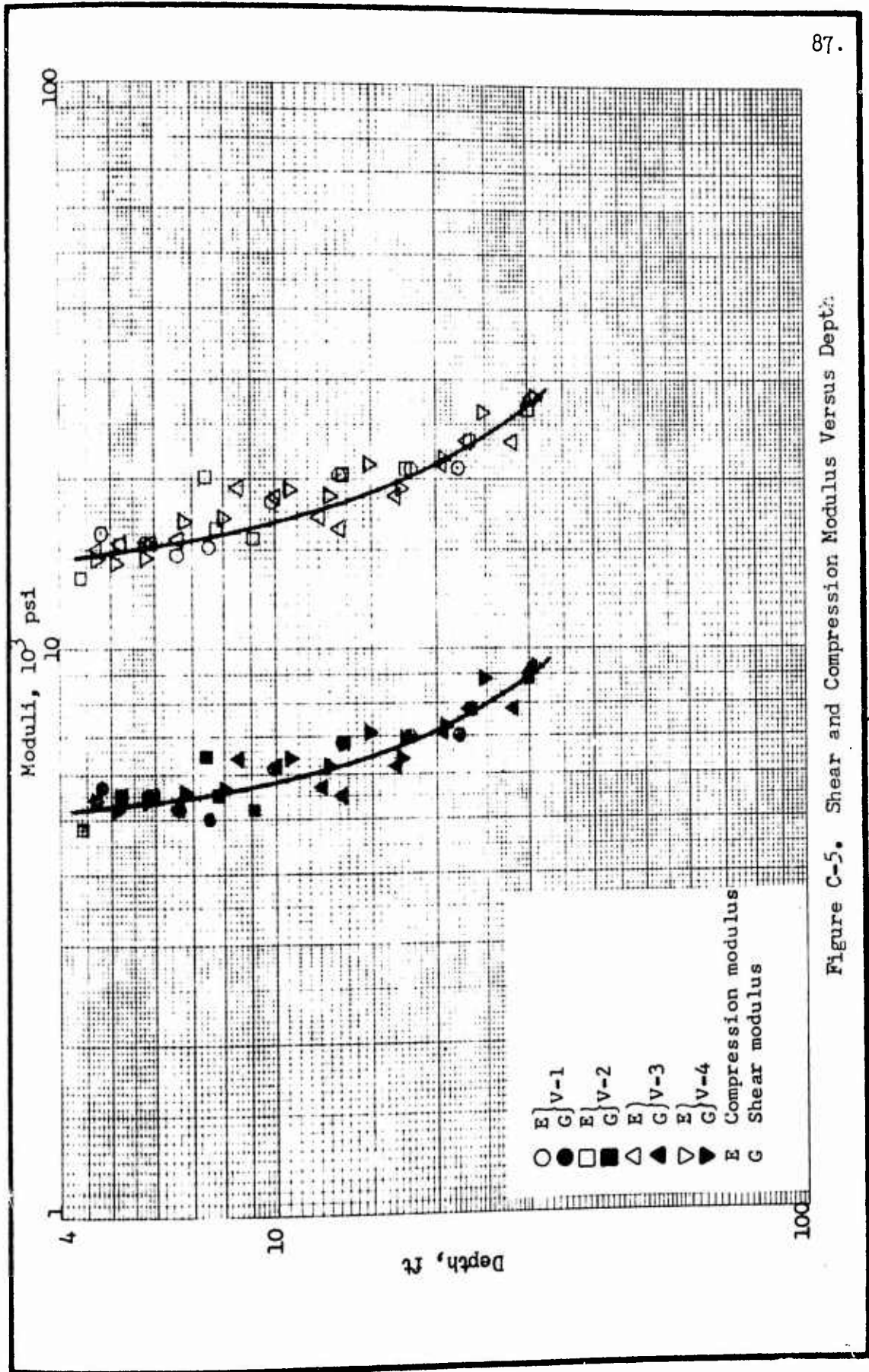


Figure C-5. Shear and Compression Modulus Versus Depth.

APPENDIX D

PEAK PARTICLE VELOCITY MEASUREMENTS

Table B-1
 Part II
 Vertical Vibration in Vertical Mode

Dist. from Hole 1, ft.	Vibr. Mode	Excav. Rate, ft./hr.	No. of Tests	Vibration in Vertical Mode																					
				V ₀	V ₁	V ₂	V ₃	V ₄	V ₅	V ₆	V ₇	V ₈	V ₉												
1	Vertical	40.00	9	1371	0.0064	0.0075	0.0086	0.0097	0.0108	0.0119	0.0130	0.0141	0.0152	0.0163	0.0174	0.0185	0.0196	0.0207	0.0218	0.0229	0.0240				
				1372	0.0065	0.0076	0.0087	0.0098	0.0109	0.0120	0.0131	0.0142	0.0153	0.0164	0.0175	0.0186	0.0197	0.0208	0.0219	0.0230	0.0241	0.0252	0.0263		
				1373	0.0066	0.0077	0.0088	0.0099	0.0110	0.0121	0.0132	0.0143	0.0154	0.0165	0.0176	0.0187	0.0198	0.0209	0.0220	0.0231	0.0242	0.0253	0.0264	0.0275	
				1374	0.0067	0.0078	0.0089	0.0100	0.0111	0.0122	0.0133	0.0144	0.0155	0.0166	0.0177	0.0188	0.0199	0.0210	0.0221	0.0232	0.0243	0.0254	0.0265	0.0276	
				1375	0.0068	0.0079	0.0090	0.0101	0.0112	0.0123	0.0134	0.0145	0.0156	0.0167	0.0178	0.0189	0.0200	0.0211	0.0222	0.0233	0.0244	0.0255	0.0266	0.0277	
				1376	0.0069	0.0080	0.0091	0.0102	0.0113	0.0124	0.0135	0.0146	0.0157	0.0168	0.0179	0.0190	0.0201	0.0212	0.0223	0.0234	0.0245	0.0256	0.0267	0.0278	0.0289
				1377	0.0070	0.0081	0.0092	0.0103	0.0114	0.0125	0.0136	0.0147	0.0158	0.0169	0.0180	0.0191	0.0202	0.0213	0.0224	0.0235	0.0246	0.0257	0.0268	0.0279	0.0290
				1378	0.0071	0.0082	0.0093	0.0104	0.0115	0.0126	0.0137	0.0148	0.0159	0.0170	0.0181	0.0192	0.0203	0.0214	0.0225	0.0236	0.0247	0.0258	0.0269	0.0280	0.0291
				1379	0.0072	0.0083	0.0094	0.0105	0.0116	0.0127	0.0138	0.0149	0.0160	0.0171	0.0182	0.0193	0.0204	0.0215	0.0226	0.0237	0.0248	0.0259	0.0270	0.0281	0.0292
				1380	0.0073	0.0084	0.0095	0.0106	0.0117	0.0128	0.0139	0.0150	0.0161	0.0172	0.0183	0.0194	0.0205	0.0216	0.0227	0.0238	0.0249	0.0260	0.0271	0.0282	0.0293
2	Vertical	15.00	13	1310	0.0133	0.0144	0.0155	0.0166	0.0177	0.0188	0.0199	0.0210	0.0221	0.0232	0.0243	0.0254	0.0265	0.0276	0.0287	0.0298	0.0309	0.0320			
				1311	0.0134	0.0145	0.0156	0.0167	0.0178	0.0189	0.0200	0.0211	0.0222	0.0233	0.0244	0.0255	0.0266	0.0277	0.0288	0.0299	0.0310	0.0321	0.0332		
				1312	0.0135	0.0146	0.0157	0.0168	0.0179	0.0190	0.0201	0.0212	0.0223	0.0234	0.0245	0.0256	0.0267	0.0278	0.0289	0.0300	0.0311	0.0322	0.0333	0.0344	
				1313	0.0136	0.0147	0.0158	0.0169	0.0180	0.0191	0.0202	0.0213	0.0224	0.0235	0.0246	0.0257	0.0268	0.0279	0.0290	0.0301	0.0312	0.0323	0.0334	0.0345	0.0356
				1314	0.0137	0.0148	0.0159	0.0170	0.0181	0.0192	0.0203	0.0214	0.0225	0.0236	0.0247	0.0258	0.0269	0.0280	0.0291	0.0302	0.0313	0.0324	0.0335	0.0346	0.0357
				1315	0.0138	0.0149	0.0160	0.0171	0.0182	0.0193	0.0204	0.0215	0.0226	0.0237	0.0248	0.0259	0.0270	0.0281	0.0292	0.0303	0.0314	0.0325	0.0336	0.0347	0.0358
				1316	0.0139	0.0150	0.0161	0.0172	0.0183	0.0194	0.0205	0.0216	0.0227	0.0238	0.0249	0.0260	0.0271	0.0282	0.0293	0.0304	0.0315	0.0326	0.0337	0.0348	0.0359
				1317	0.0140	0.0151	0.0162	0.0173	0.0184	0.0195	0.0206	0.0217	0.0228	0.0239	0.0250	0.0261	0.0272	0.0283	0.0294	0.0305	0.0316	0.0327	0.0338	0.0349	0.0360
				1318	0.0141	0.0152	0.0163	0.0174	0.0185	0.0196	0.0207	0.0218	0.0229	0.0240	0.0251	0.0262	0.0273	0.0284	0.0295	0.0306	0.0317	0.0328	0.0339	0.0350	0.0361
				1319	0.0142	0.0153	0.0164	0.0175	0.0186	0.0197	0.0208	0.0219	0.0230	0.0241	0.0252	0.0263	0.0274	0.0285	0.0296	0.0307	0.0318	0.0329	0.0340	0.0351	0.0362
3	Vertical	14.48	15	1318	0.0190	0.0201	0.0212	0.0223	0.0234	0.0245	0.0256	0.0267	0.0278	0.0289	0.0300	0.0311	0.0322	0.0333	0.0344	0.0355	0.0366	0.0377			
				1319	0.0191	0.0202	0.0213	0.0224	0.0235	0.0246	0.0257	0.0268	0.0279	0.0290	0.0301	0.0312	0.0323	0.0334	0.0345	0.0356	0.0367	0.0378	0.0389		
				1320	0.0192	0.0203	0.0214	0.0225	0.0236	0.0247	0.0258	0.0269	0.0280	0.0291	0.0302	0.0313	0.0324	0.0335	0.0346	0.0357	0.0368	0.0379	0.0390		
				1321	0.0193	0.0204	0.0215	0.0226	0.0237	0.0248	0.0259	0.0270	0.0281	0.0292	0.0303	0.0314	0.0325	0.0336	0.0347	0.0358	0.0369	0.0380	0.0391		
				1322	0.0194	0.0205	0.0216	0.0227	0.0238	0.0249	0.0260	0.0271	0.0282	0.0293	0.0304	0.0315	0.0326	0.0337	0.0348	0.0359	0.0370	0.0381	0.0392		
				1323	0.0195	0.0206	0.0217	0.0228	0.0239	0.0250	0.0261	0.0272	0.0283	0.0294	0.0305	0.0316	0.0327	0.0338	0.0349	0.0360	0.0371	0.0382	0.0393		
				1324	0.0196	0.0207	0.0218	0.0229	0.0240	0.0251	0.0262	0.0273	0.0284	0.0295	0.0306	0.0317	0.0328	0.0339	0.0350	0.0361	0.0372	0.0383	0.0394		
				1325	0.0197	0.0208	0.0219	0.0230	0.0241	0.0252	0.0263	0.0274	0.0285	0.0296	0.0307	0.0318	0.0329	0.0340	0.0351	0.0362	0.0373	0.0384	0.0395		
				1326	0.0198	0.0209	0.0220	0.0231	0.0242	0.0253	0.0264	0.0275	0.0286	0.0297	0.0308	0.0319	0.0330	0.0341	0.0352	0.0363	0.0374	0.0385	0.0396		
				1327	0.0199	0.0210	0.0221	0.0232	0.0243	0.0254	0.0265	0.0276	0.0287	0.0298	0.0309	0.0320	0.0331	0.0342	0.0353	0.0364	0.0375	0.0386	0.0397		
4	Vertical	9.50	18	1265	0.0230	0.0241	0.0252	0.0263	0.0274	0.0285	0.0296	0.0307	0.0318	0.0329	0.0340	0.0351	0.0362	0.0373	0.0384	0.0395	0.0406	0.0417			
				1266	0.0231	0.0242	0.0253	0.0264	0.0275	0.0286	0.0297	0.0308	0.0319	0.0330	0.0341	0.0352	0.0363	0.0374	0.0385	0.0396	0.0407	0.0418	0.0429		
				1267	0.0232	0.0243	0.0254	0.0265	0.0276	0.0287	0.0298	0.0309	0.0320	0.0331	0.0342	0.0353	0.0364	0.0375	0.0386	0.0397	0.0408	0.0419	0.0430		
				1268	0.0233	0.0244	0.0255	0.0266	0.0277	0.0288	0.0299	0.0310	0.0321	0.0332	0.0343	0.0354	0.0365	0.0376	0.0387	0.0398	0.0409	0.0420	0.0431		
				1269	0.0234	0.0245	0.0256	0.0267	0.0278	0.0289	0.0300	0.0311	0.0322	0.0333	0.0344	0.0355	0.0366	0.0377	0.0388	0.0399	0.0410	0.0421	0.0432		
				1270	0.0235	0.0246	0.0257	0.0268	0.0279	0.0290	0.0301	0.0312	0.0323	0.0334	0.0345	0.0356	0.0367	0.0378	0.0389	0.0400	0.0411	0.0422	0.0433		
				1271	0.0236	0.0247	0.0258	0.0269	0.0280	0.0291	0.0302	0.0313	0.0324	0.0335	0.0346	0.0357	0.0368	0.0379	0.0390	0.0401	0.0412	0.0423	0.0434		
				1272	0.0237	0.0248	0.0259	0.0270	0.0281	0.0292	0.0303	0.0314	0.0325	0.0336	0.0347	0.0358	0.0369	0.0380	0.0391	0.0402	0.0413	0.0424	0.0435		
				1273	0.0238	0.0249	0.0260	0.0271	0.0282	0.0293	0.0304	0.0315	0.0326	0.0337	0.0348	0.0359	0.0370	0.0381	0.0392	0.0403	0.0414	0.0425	0.0436		
				1274	0.0239	0.0250	0.0261	0.0272	0.0283	0.0294	0.0305	0.0316	0.0327	0.0338	0.0349	0.0360	0.0371	0.0382	0.0393	0.0404	0.0415	0.0426	0.0437		
5	Vertical	5.00	26	1276	0.0390	0.0401	0.0412	0.0423	0.0434	0.0445	0.0456	0.0467	0.0478	0.0489	0.0500	0.0511	0.0522	0.0533	0.0544	0.0555	0.0566	0.0577			
				1277	0.0391	0.0402	0.0413	0.0424	0.0435	0.0446	0.0457	0.0468	0.0479	0.0490	0.0501	0.0512	0.0523	0.0534	0.0545	0.0556	0.0567	0.0578	0.0589		
				1278	0.0392	0.0403	0.0414	0.0425	0.0436	0.0447	0.0458	0.0469	0.0480	0.0491	0.0502	0.0513	0.0524	0.0535	0.0546	0.0557	0.0568	0.0579	0.0590		
				1279	0.0393	0.0404	0.0415	0.0426	0.0437	0.0448	0.0459	0.0470	0.0481	0.0492	0.0503	0.0514	0.0525	0.0536	0.0547	0.0558	0.0569	0.0580	0.0591		
				1280	0.0394	0.0405	0.0416	0.0427	0.0438	0.0449	0.0460	0.0471	0.0482	0.0493	0.0504	0.0515	0.0526	0.0537	0.0548	0.0559	0.0570	0.0581	0.0592		
				1281	0.0395	0.0406	0.0417	0.0428	0.0439	0.0450	0.0461	0.0472	0.0483	0.0494	0.0505	0.0516	0.0527	0.0538	0.0549	0.0560	0.0571	0.0582	0.0593		
				1282	0.0396	0.0407	0.0418	0.0429	0.0440	0.0451	0.0462	0.0473	0.0484	0.0495	0.0506	0.0517	0.0528	0.0539	0.0550	0.0561	0.0572	0.0583	0.0594		
				1283																					

Table 1
 Figure 1
 Laboratory Measurements of Peak Particle Velocity Transducers at Various Depths

Vibrator Excite- tion Rate, Hz	Vibrator Excite- tion Frequency, cps	Vibrator Excite- tion Force, lb	Peak Particle Velocity, Ips., at Various Depths																			
			0	1	2	3	4	5	6	7	8	9										
12	10	Vertical	9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
			9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097
9.60	0.007	0.012	0.017	0.022	0.027	0.032	0.037	0.042	0.047	0.052	0.057	0.062	0.067	0.072	0.077	0.082	0.087	0.092	0.097			

Vibrator Excite- tion Rate, Hz	Vibrator Excite- tion Frequency, cps	Vibrator Excite- tion Force, lb	Peak Particle Velocity, Ips., at Indicated Depths*																		
			0	1	2	3	4	5	6	7	8	9									
13	10	Vertical	9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181
			9.60	0.013	0.021	0.031	0.041	0.051	0.061	0.071	0.081	0.091	0.101	0.111	0.121	0.131	0.141	0.151	0.161	0.171	0.181

* Vertical mode - force, 10; horizontal mode - torque, 80-lb.
 ** V_0 located on ground surface, 0.5 ft. from base of vibrator; V_1, V_2, V_3 and V_4 located on ground surface, 5 ft. from hole 1.
 V_5 - vertical
 V_6 - horizontal
 V_7 - transverse

(Continued)

Table 12
 Surface Measurements of Free Field in Vertical Plane of Direction of Vibration

Dist. from Pt.	Vibrator	Excit. Force, Pk-Pk, Lb.	Pre-cessory Hz.	Pace or Direction	Dist. from Pt. (ft.)																																																																				
					0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36																																
35	10	Vertical	15.00	775	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001
36	10	Vertical	19.00	1115	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001
37	10	Vertical	19.00	1315	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001
38	10	Vertical	14.48	1296	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001
39	10	Vertical	9.60	1444	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001
40	10	Vertical	5.00	1276	0.4560	0.0061	0.0069	0.0068	0.0067	0.0065	0.0064	0.0062	0.0061	0.0060	0.0059	0.0058	0.0057	0.0056	0.0055	0.0054	0.0053	0.0052	0.0051	0.0050	0.0049	0.0048	0.0047	0.0046	0.0045	0.0044	0.0043	0.0042	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026	0.0025	0.0024	0.0023	0.0022	0.0021	0.0020	0.0019	0.0018	0.0017	0.0016	0.0015	0.0014	0.0013	0.0012	0.0011	0.0010	0.0009	0.0008	0.0007	0.0006	0.0005	0.0004	0.0003	0.0002	0.0001

(Continued)

* Vertical mode - force, lb; horizontal mode - torque, ft.-lb.
 ** V_0 located on ground surface, 0.5 ft. from base of vibrator; V_1 , V_2 , and V_3 located on ground surface, 5 ft. from hole 1.
 V = vertical
 H = horizontal
 T = transverse

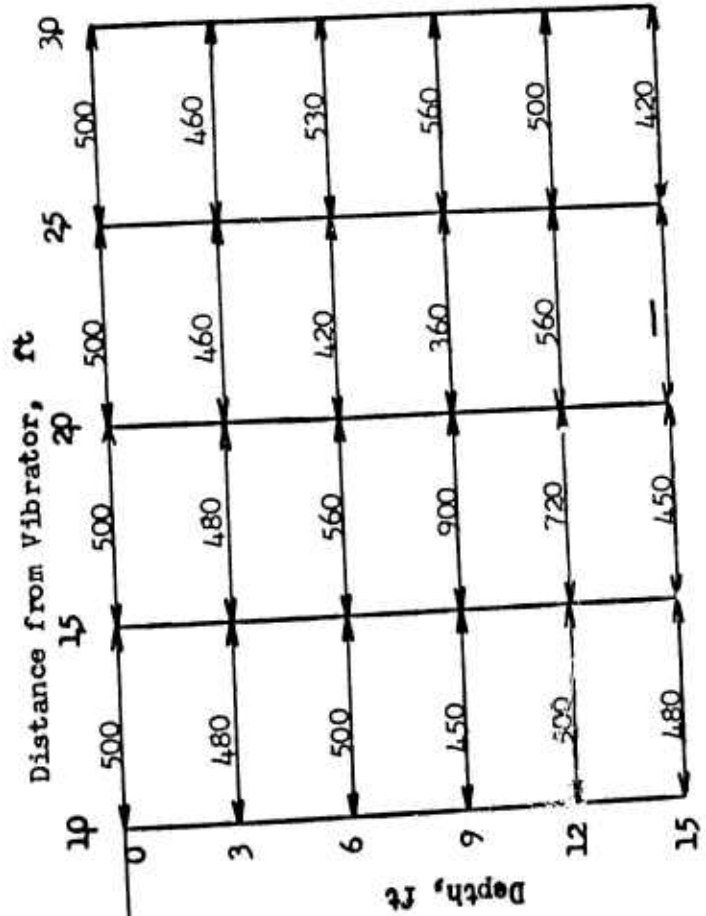
Table (a) - (Cont.)

Test No.	Distance From Hole 1, Feet	Vibration Mode	Excursion, In.	Frequency, Cycles Per Sec.	Peak Particle Velocity, In. Per Second																			
					1	2	3	4	5	6	7	8	9	10	11	12								
41	10	Torsional	5.00	27	1.1600	0.0086	0.0071	0.0058	0.0044	0.0034	0.0027	0.0021	0.0016	0.0012	0.0009	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001			
			5.00	28	1.3200	0.0079	0.0065	0.0052	0.0040	0.0031	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001		
			5.00	29	1.4800	0.0072	0.0059	0.0047	0.0036	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001		
			5.00	30	1.6400	0.0065	0.0053	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	31	1.8000	0.0058	0.0047	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	32	1.9600	0.0051	0.0041	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	33	2.1200	0.0044	0.0035	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	34	2.2800	0.0037	0.0029	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	35	2.4400	0.0030	0.0023	0.0017	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001		
			5.00	36	2.6000	0.0023	0.0017	0.0012	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001		
			42	10	Torsional	9.60	16	1.6500	0.0118	0.0097	0.0078	0.0061	0.0047	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0007	0.0005	0.0004	0.0003	0.0002
						9.60	19	1.8100	0.0111	0.0091	0.0073	0.0057	0.0044	0.0034	0.0026	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001
9.60	20	1.9700				0.0104	0.0084	0.0067	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001		
9.60	21	2.1300				0.0097	0.0078	0.0062	0.0048	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001		
9.60	22	2.2900				0.0090	0.0072	0.0057	0.0044	0.0034	0.0026	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001		
9.60	23	2.4500				0.0083	0.0066	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001		
9.60	24	2.6100				0.0076	0.0060	0.0047	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001		
9.60	25	2.7700				0.0069	0.0054	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001		
9.60	26	2.9300				0.0062	0.0048	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001		
43	10	Torsional				19.00	10	2.4500	0.0097	0.0078	0.0061	0.0047	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001
						19.00	12	2.6100	0.0090	0.0072	0.0057	0.0044	0.0034	0.0026	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001
						19.00	15	2.7700	0.0083	0.0066	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001
			19.00	20	2.9300	0.0076	0.0060	0.0047	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001			
			19.00	25	3.0900	0.0069	0.0054	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001			
			19.00	30	3.2500	0.0062	0.0048	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001			
			19.00	35	3.4100	0.0055	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001			
			19.00	40	3.5700	0.0048	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001			
			19.00	45	3.7300	0.0041	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001			
			19.00	50	3.8900	0.0034	0.0024	0.0017	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001			
			44	10	Torsional	19.00	13	2.4500	0.0097	0.0078	0.0061	0.0047	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001
						19.00	15	2.6100	0.0090	0.0072	0.0057	0.0044	0.0034	0.0026	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001
19.00	17	2.7700				0.0083	0.0066	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001			
19.00	20	2.9300				0.0076	0.0060	0.0047	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001			
19.00	25	3.0900				0.0069	0.0054	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001			
19.00	30	3.2500				0.0062	0.0048	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001			
19.00	35	3.4100				0.0055	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001				
19.00	40	3.5700				0.0048	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001				
19.00	45	3.7300				0.0041	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001				
19.00	50	3.8900				0.0034	0.0024	0.0017	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001				
45	10	Torsional				40.00	9	3.0900	0.0107	0.0086	0.0068	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001
						40.00	10	3.2500	0.0100	0.0080	0.0063	0.0048	0.0037	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001
			40.00	11	3.4100	0.0093	0.0074	0.0057	0.0043	0.0033	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001			
			40.00	12	3.5700	0.0086	0.0068	0.0052	0.0040	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001			
			40.00	13	3.7300	0.0079	0.0062	0.0047	0.0036	0.0027	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001			
			46	10	Torsional	14.48	15	2.1000	0.0161	0.0129	0.0099	0.0074	0.0056	0.0042	0.0032	0.0024	0.0018	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002
						14.48	16	2.2600	0.0154	0.0122	0.0092	0.0068	0.0051	0.0038	0.0028	0.0021	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001
						14.48	17	2.4200	0.0147	0.0115	0.0085	0.0062	0.0046	0.0034	0.0025	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001
						14.48	18	2.5800	0.0140	0.0108	0.0078	0.0056	0.0041	0.0030	0.0022	0.0016	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001
						14.48	19	2.7400	0.0133	0.0101	0.0071	0.0050	0.0036	0.0026	0.0019	0.0014	0.0010	0.0007	0.0005	0.0004	0.0003	0.0002	0.0001	0.0001
						14.48	20	2.9000	0.0126	0.0094	0.0064	0.0044	0.0032	0.0023	0.0017	0.0012	0.0009	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	0.0001
						14.48	21	3.0600	0.0119	0.0087	0.0057	0.0038	0.0028	0.0020	0.0015	0.0011	0.0008	0.0006	0.0004	0.0003	0.0002	0.0001	0.0001	

APPENDIX E

TABULATION OF PHASE VELOCITY MEASUREMENTS BETWEEN
VERTICAL TRANSDUCERS AT VARIOUS DEPTHS

Average Phase Velocities
fps



$v_s = 530$ fps
 $\frac{\lambda}{2} = 17.8$ ft

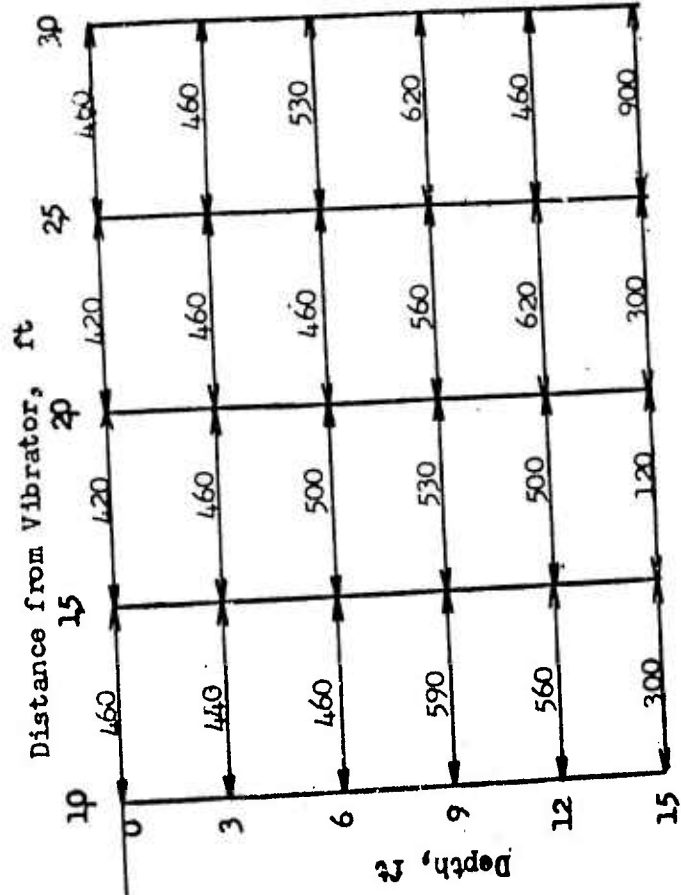
Boreholes ① ② ③ ④ ⑤

NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-1. Phase Velocity of Vertical Components at 15 Hz Frequency.

Average Phase
Velocities
fps

440
450
490
580
540
410

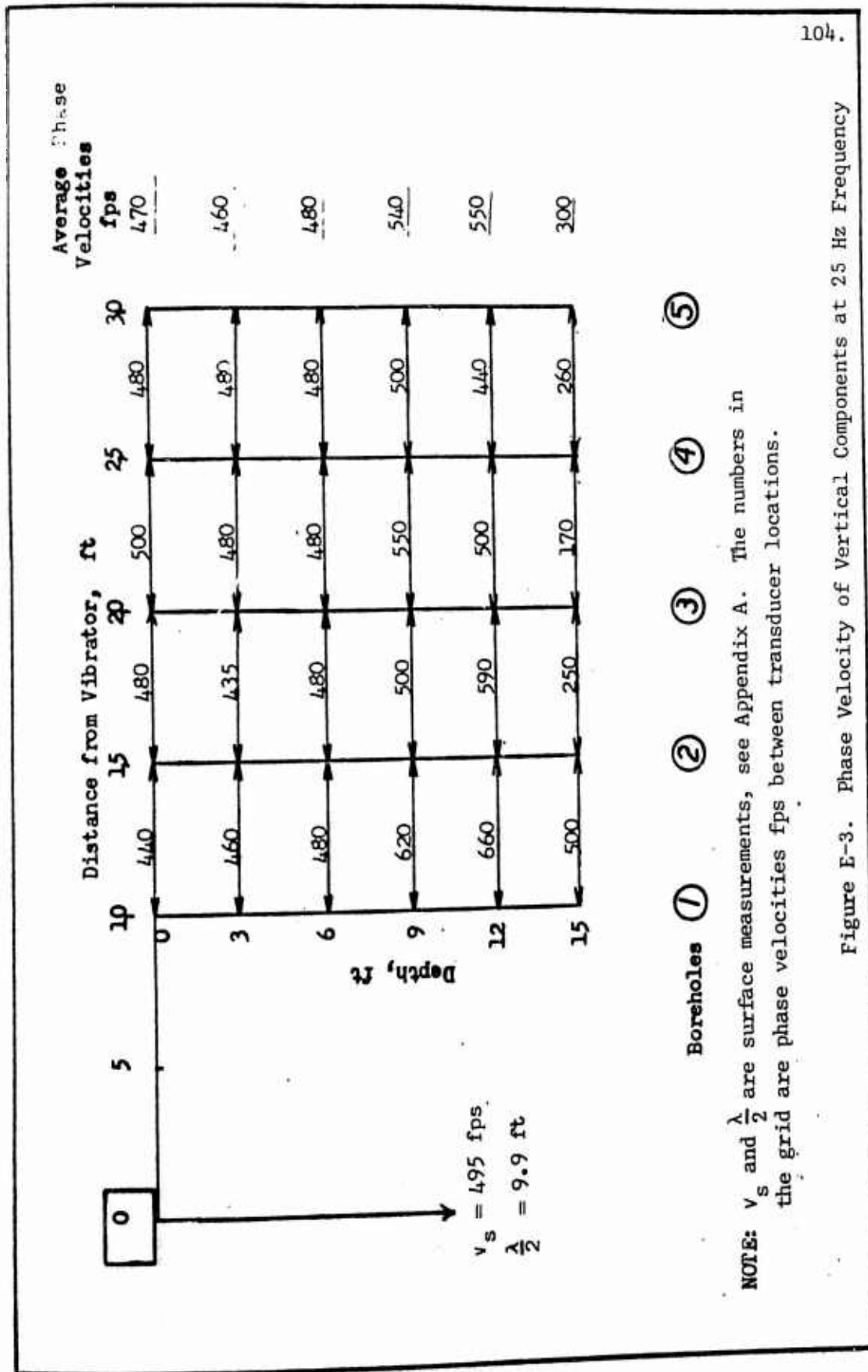


$v_s = 525$ fps
 $\frac{\lambda}{2} = 13.1$ ft

Boreholes ① ② ③ ④ ⑤

NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-1. Phase Velocity of Vertical Components at 20 Hz Frequency

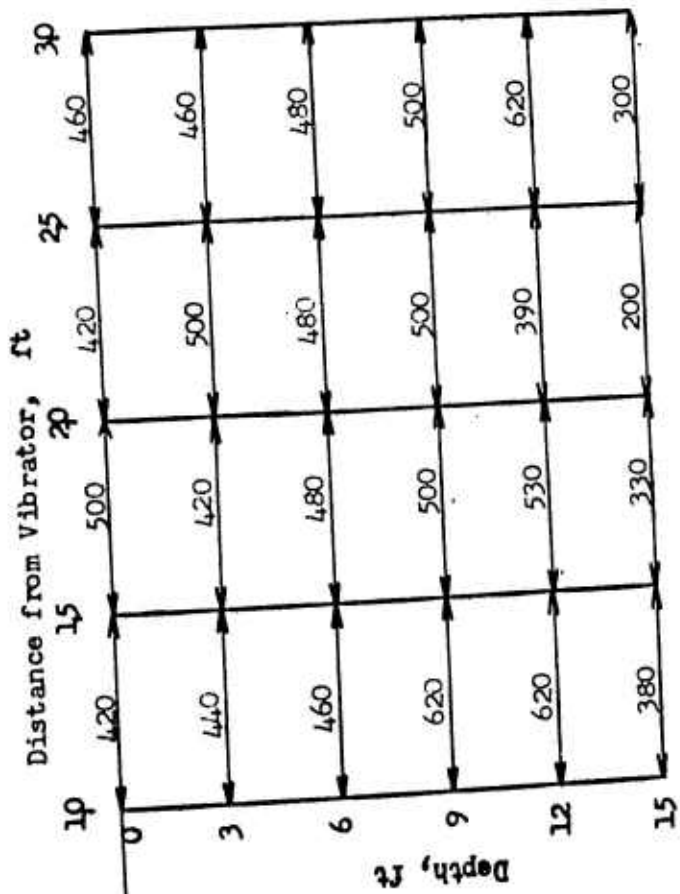


NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-3. Phase Velocity of Vertical Components at 25 Hz Frequency

Average Phase Velocities
fps

450
450
480
530
540
300

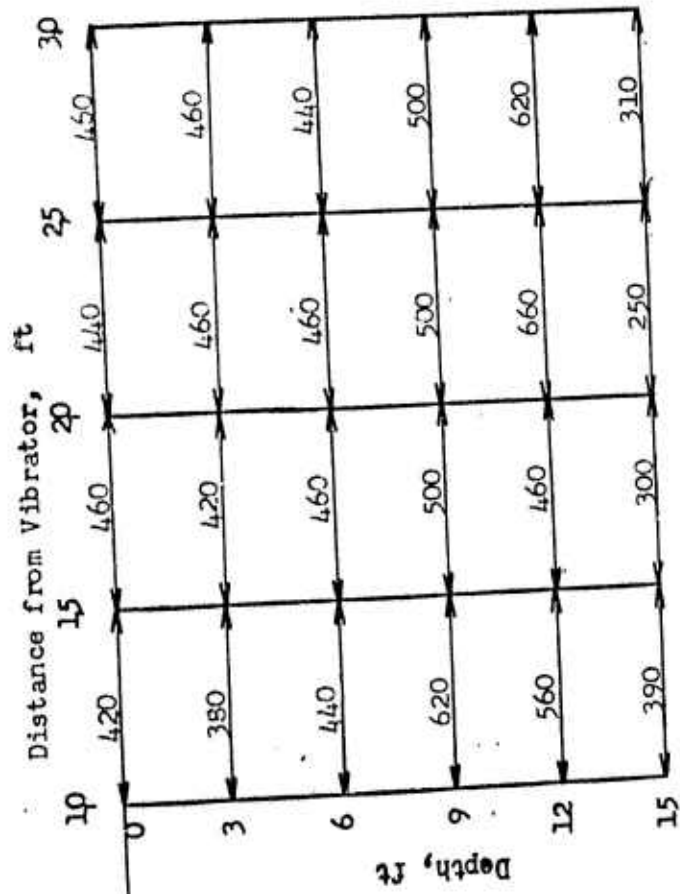


Boreholes ① ② ③ ④ ⑤

NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-4. Phase Velocity of Vertical Components at 30 Hz Frequency

Average Phase Velocities
fps



Boreholes ① ② ③ ④ ⑤

NOTES: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-5. Phase Velocity of Vertical Components at 35 Hz Frequency

Average Phase Velocities
fps

420

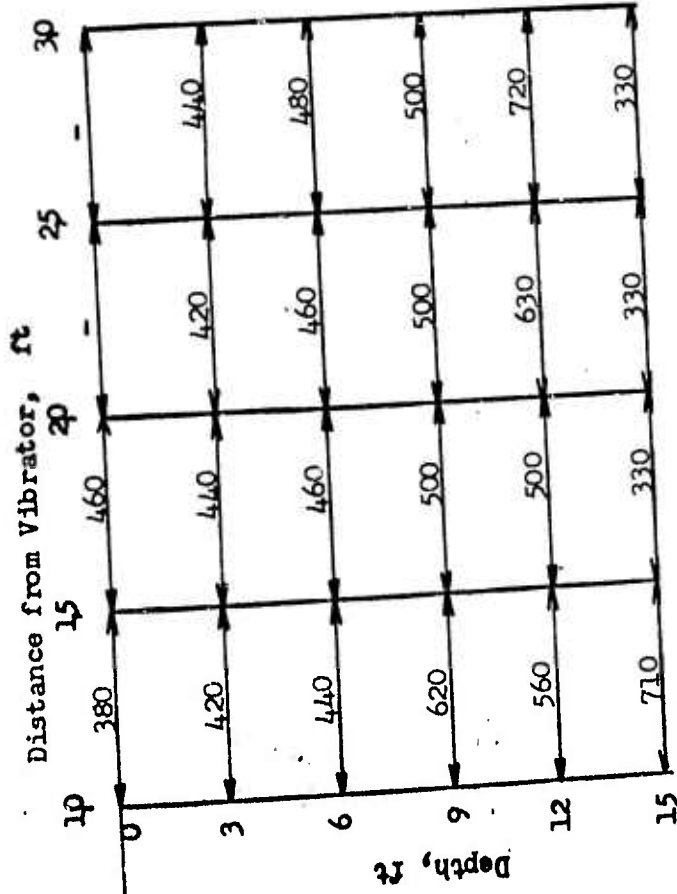
430

460

530

600

420



$v_s = 470$ fps
 $\frac{\lambda}{2} = 5.8$ ft

Boreholes ①

②

③

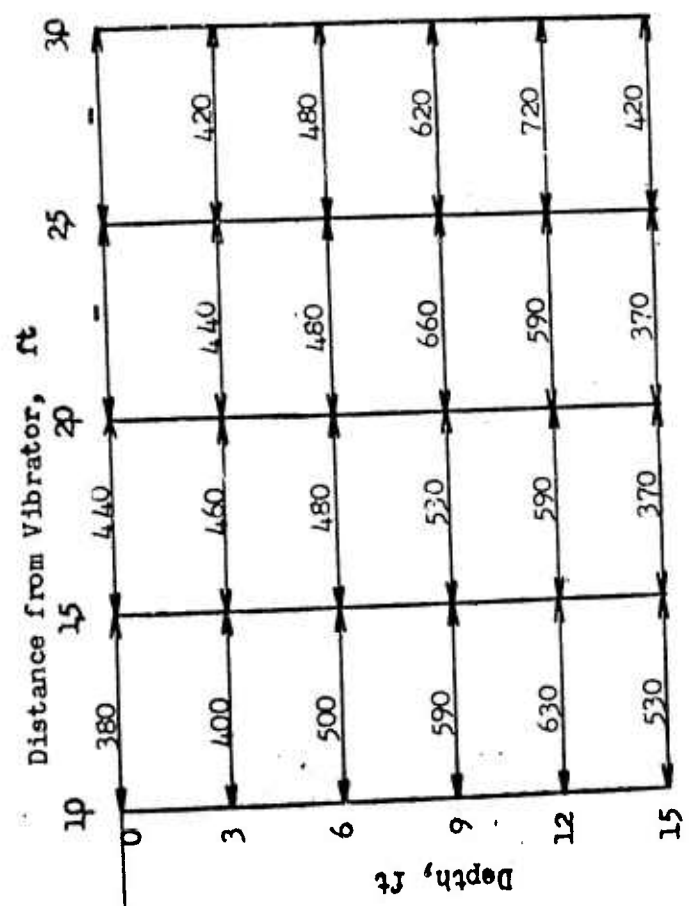
④

⑤

NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, See Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-6. Phase Velocity of Vertical Components at 40 Hz Frequency

Average Phase
Velocities
fps



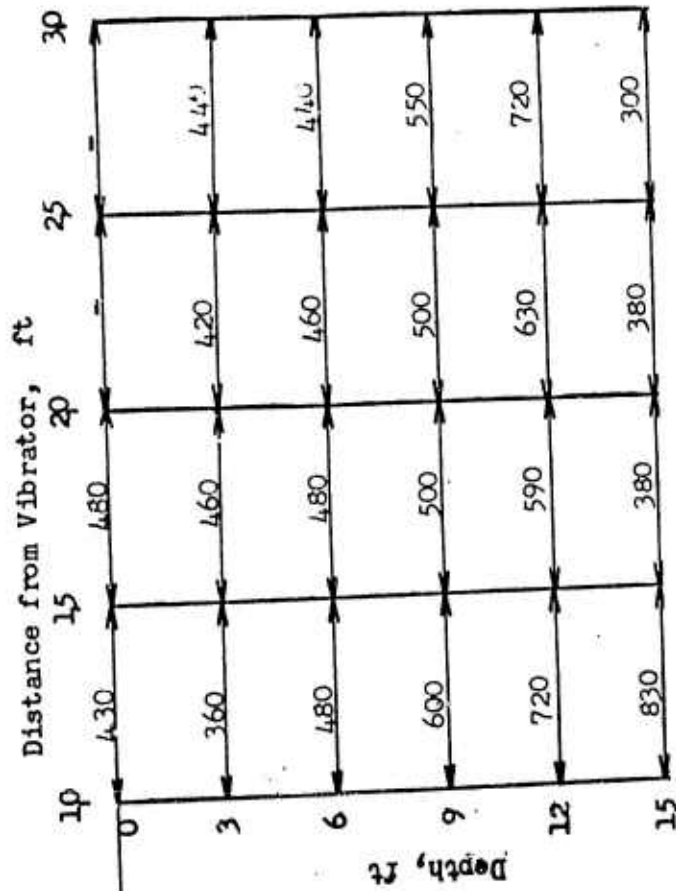
$v_s = 470 \text{ fps}$
 $\frac{\lambda}{2} = 5.2 \text{ ft}$

Boreholes ① ② ③ ④ ⑤

NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-7. Phase Velocity of Vertical Components at 45 Hz Frequency

Average : nase
Velocities
fps



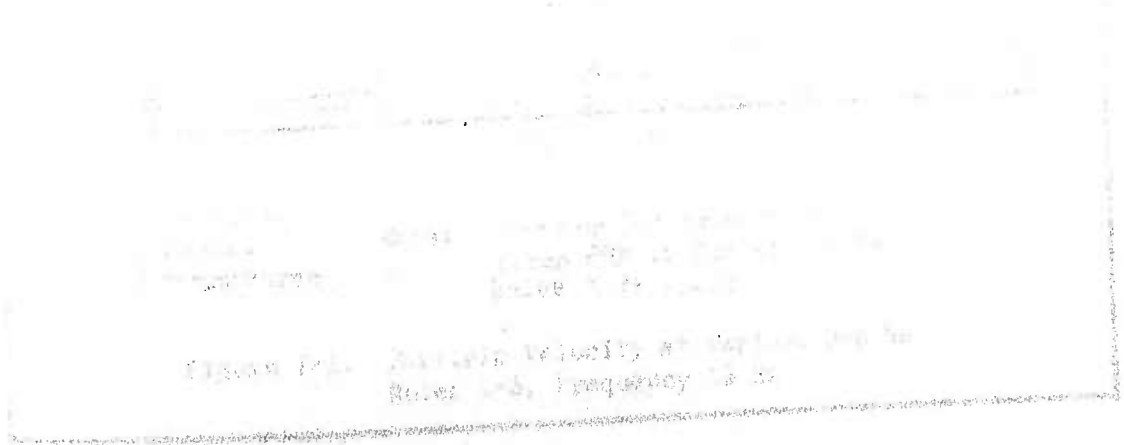
Boreholes ① ② ③ ④ ⑤

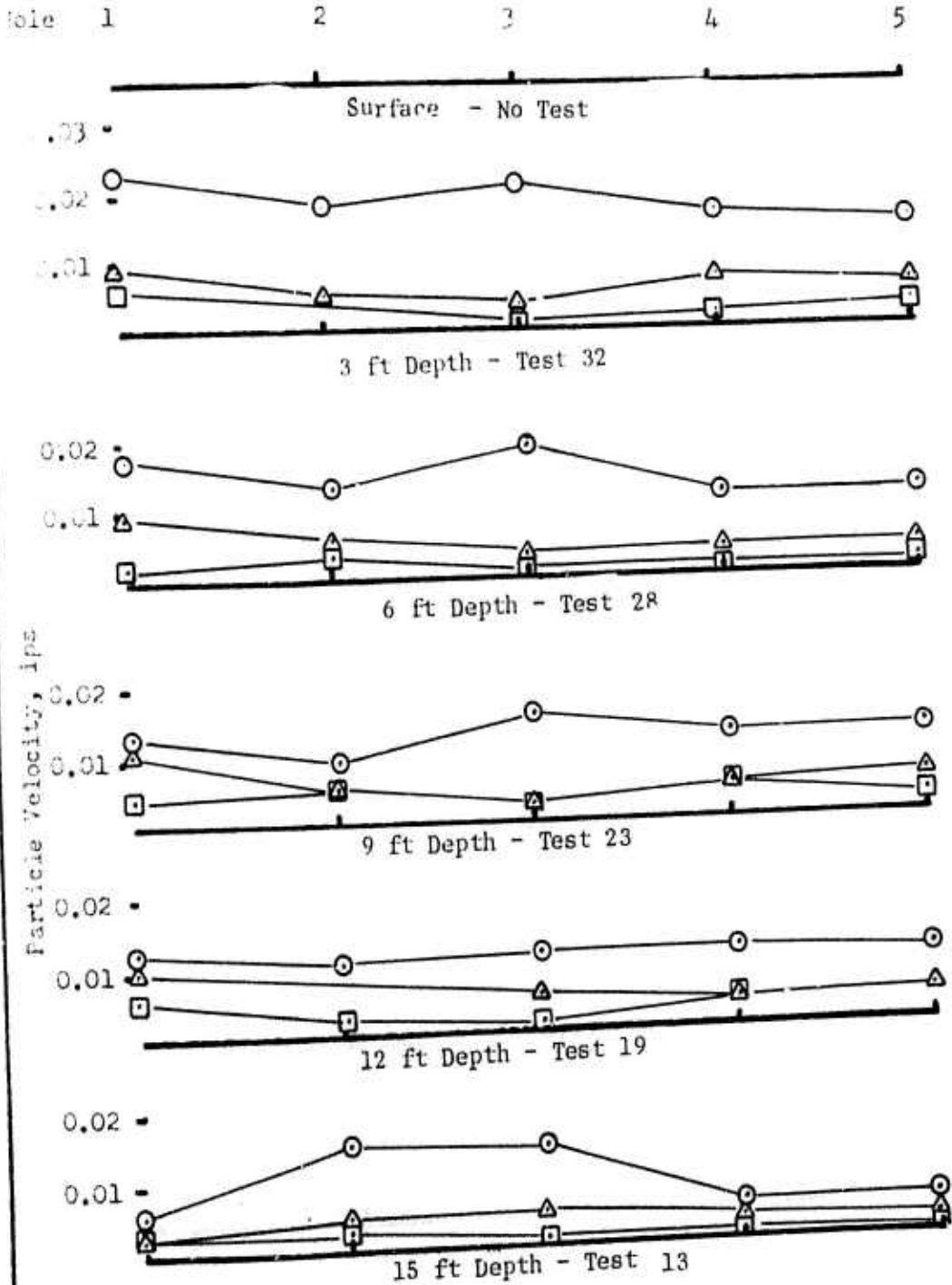
NOTE: v_s and $\frac{\lambda}{2}$ are surface measurements, see Appendix A. The numbers in the grid are phase velocities fps between transducer locations.

Figure E-8. Phase Velocity of Vertical Components at 50 Hz Frequency

APPENDIX F

PLOTS OF PARTICLE VELOCITY AT VARIOUS DEPTHS





○ vertical
 △ radial
 □ transverse

Note: Vibrator 10' from hole 1
 Force 882 lb for all tests
 holes 5 ft apart

Figure F-1. Particle Velocity at Various Depths
 Holes 1-5, Frequency 15 Hz

Hole 1 2 3 4 5

Note: Vibrator 10' from hole 1.
Force 1568 lbs for all tests
holes 1 ft apart

○ vertical
△ radial
□ transverse

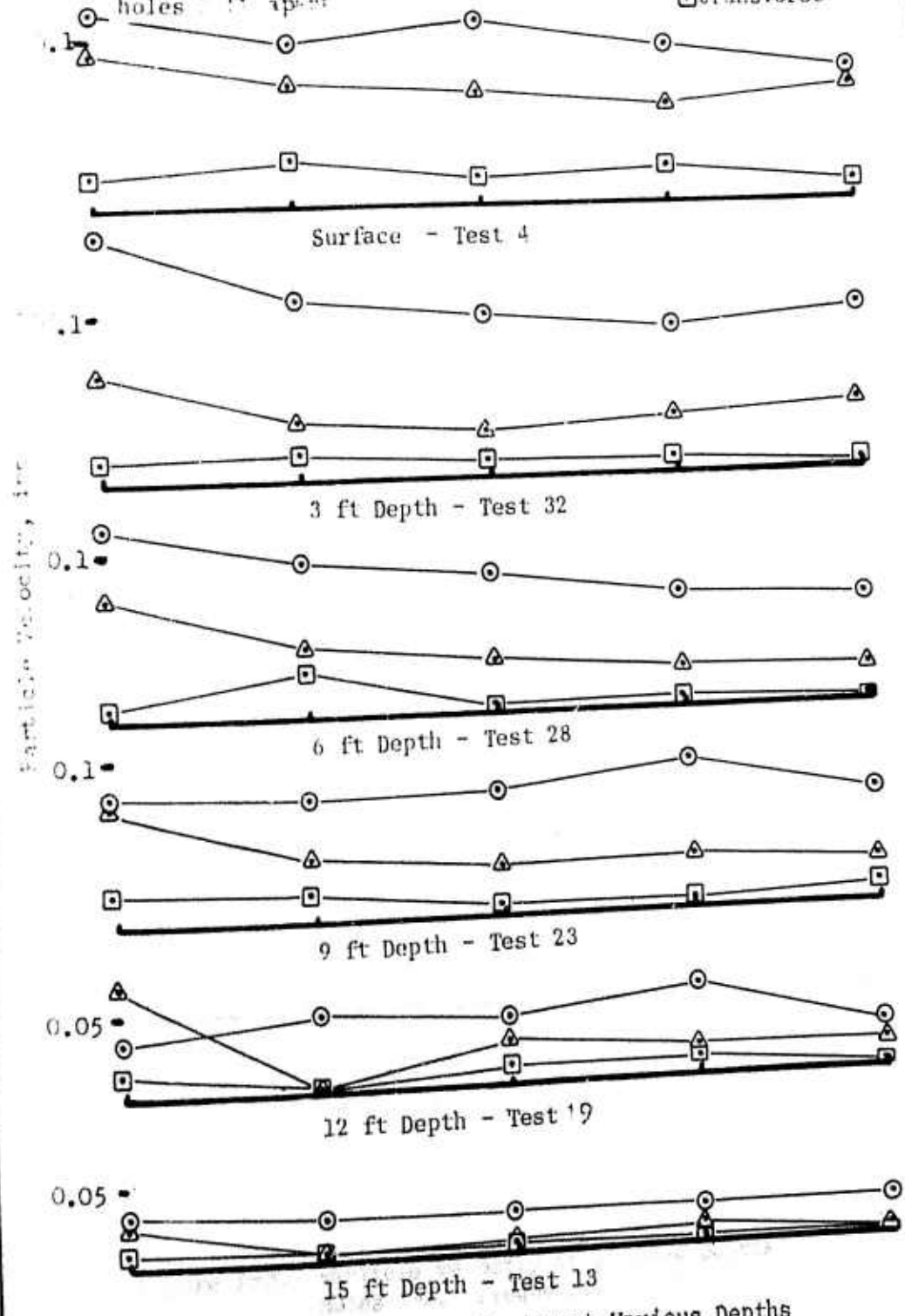


Figure F-2. Particle Velocity at Various Depths
Holes 1-5, Frequency 20 Hz

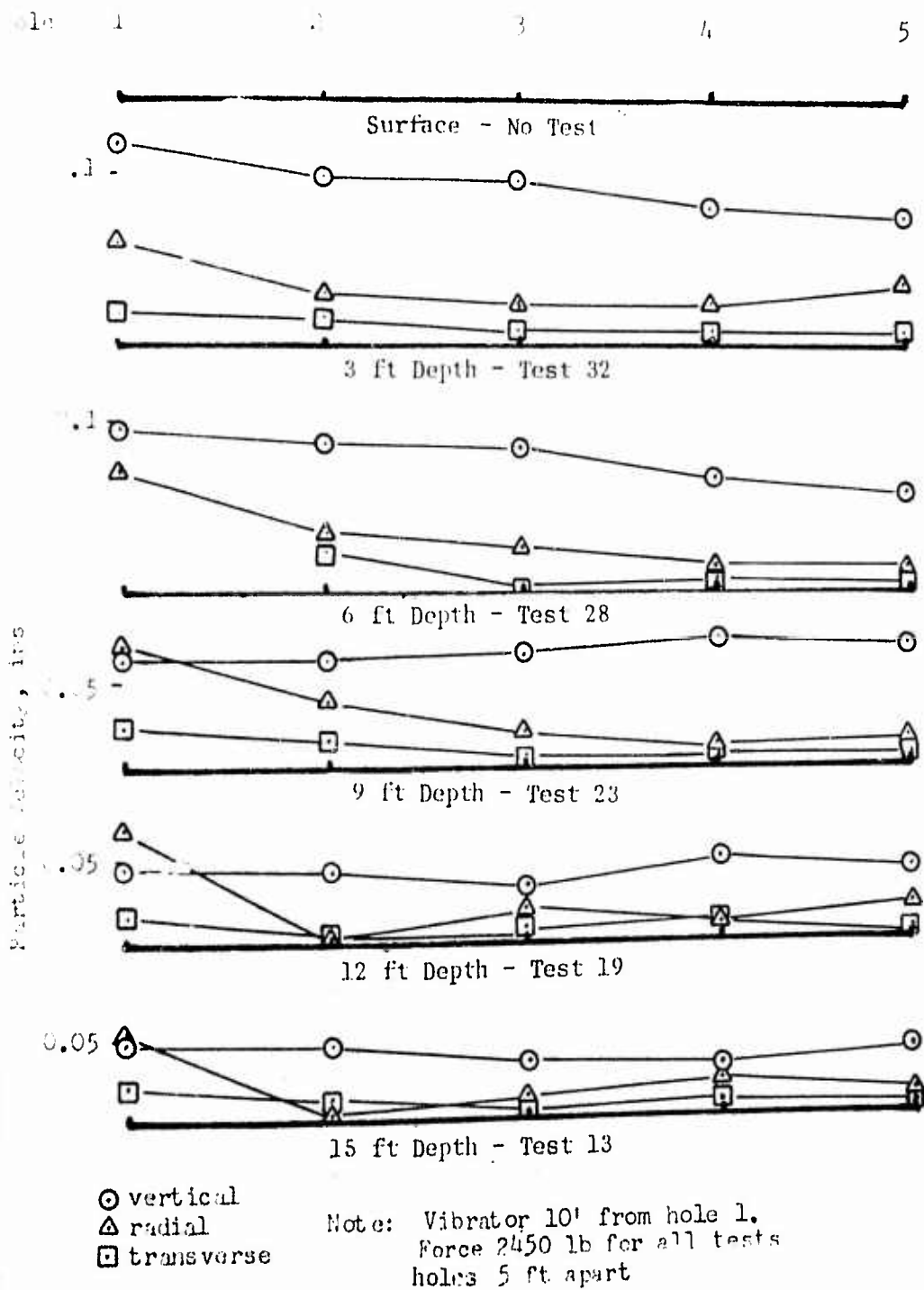
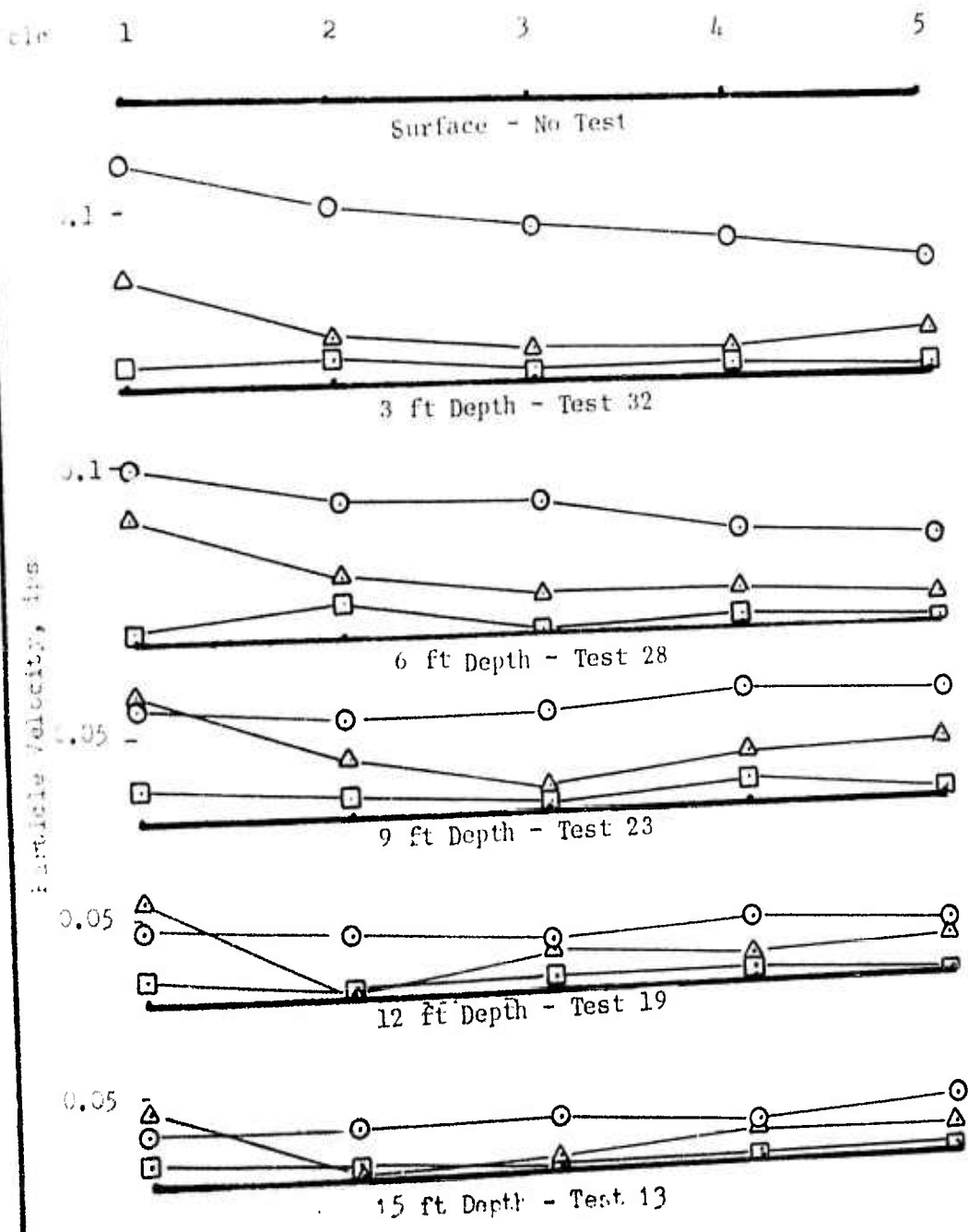


Figure F-3. Particle Velocity at Various Depths
Holes 1-5, Frequency 25 Hz

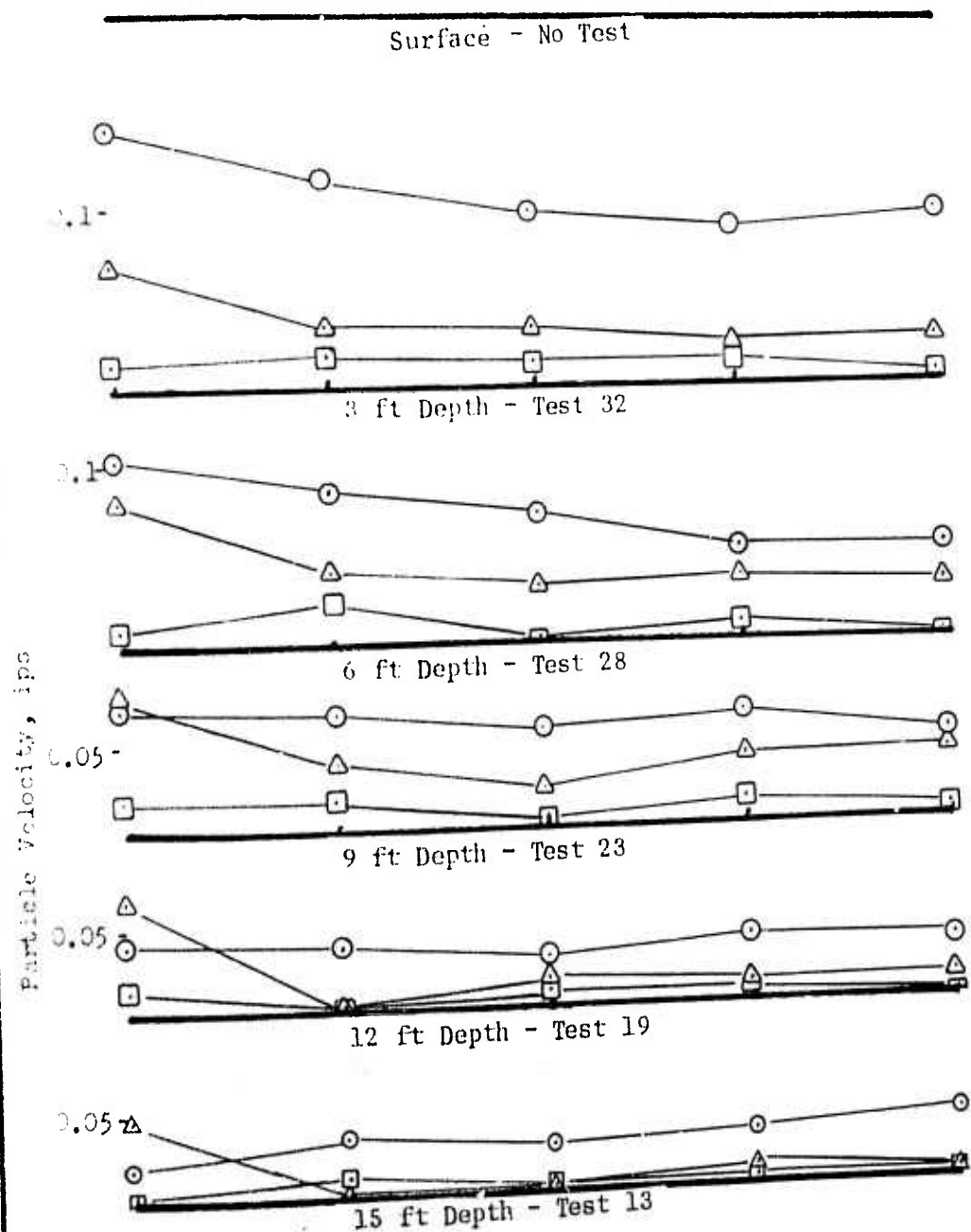


○ Vertical
 △ Radial
 □ Transverse

Note: Vibrator 10' from hole 1.
 Force 3529 lb for all Tests
 All Holes 5 ft apart

Figure P-4. Particle Velocity at Various Depths
 Holes 1-5, Frequency 30 Hz

ole 1 2 3 4 5 115.

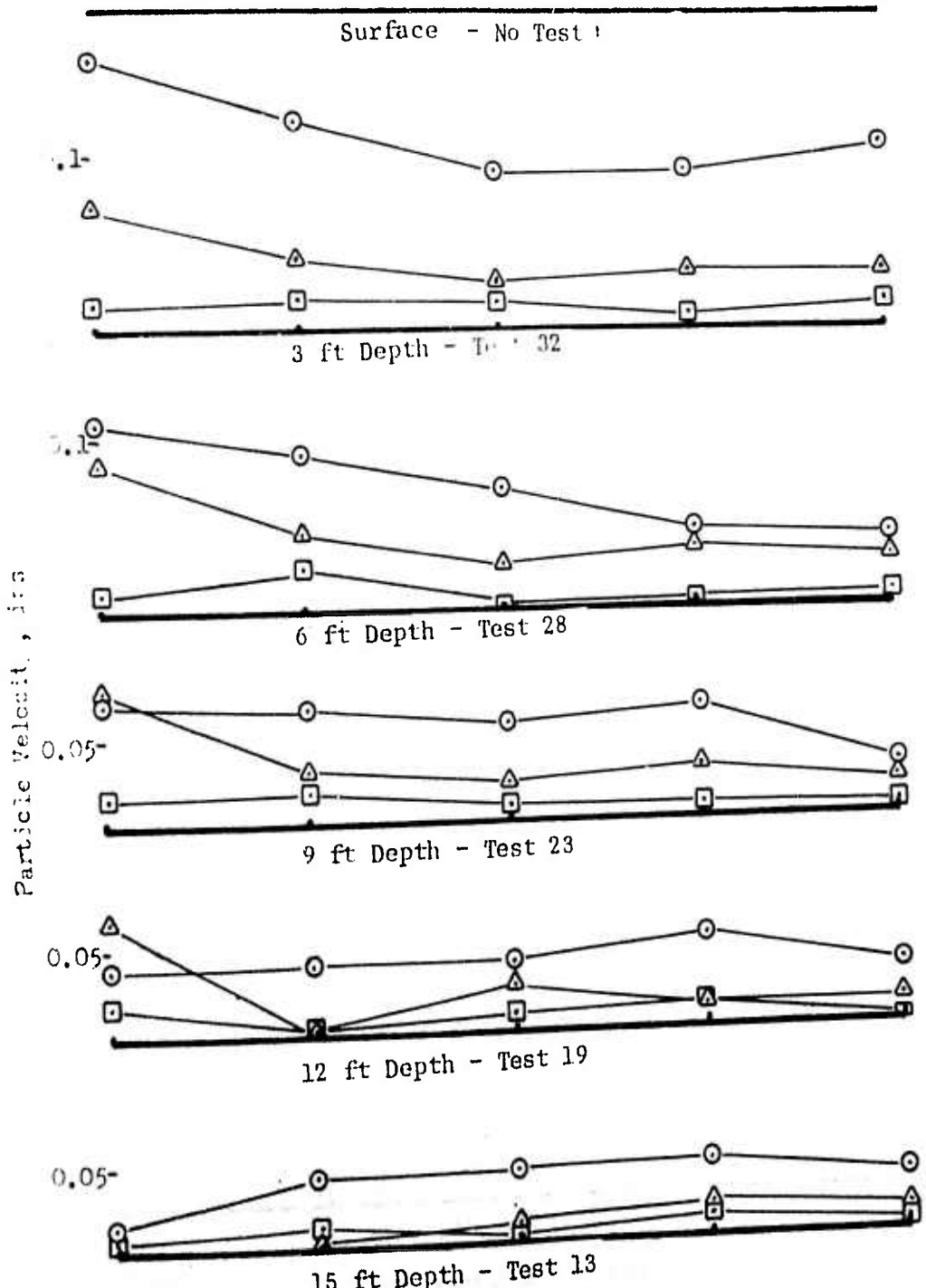


○ vertical
 △ radial
 □ transverse

Note: Vibrator 10' from hole 1.
 Force 4802 lb for all Tests
 All holes 5 ft apart

Figure P-5. Particle Velocity at Various Depths
 Holes 1-5, Frequency 35 Hz

Hole 1 2 3 4 5 116.



Particle Velocity, in/s

○ vertical
 △ radial
 □ transverse

Note: Vibrator 10' from hole 1.
 Force 6272 lb for all Tests
 All holes 5 ft apart

Figure F-6. Particle Velocity at Various Depths
 Holes 1-5, Frequency 40 Hz

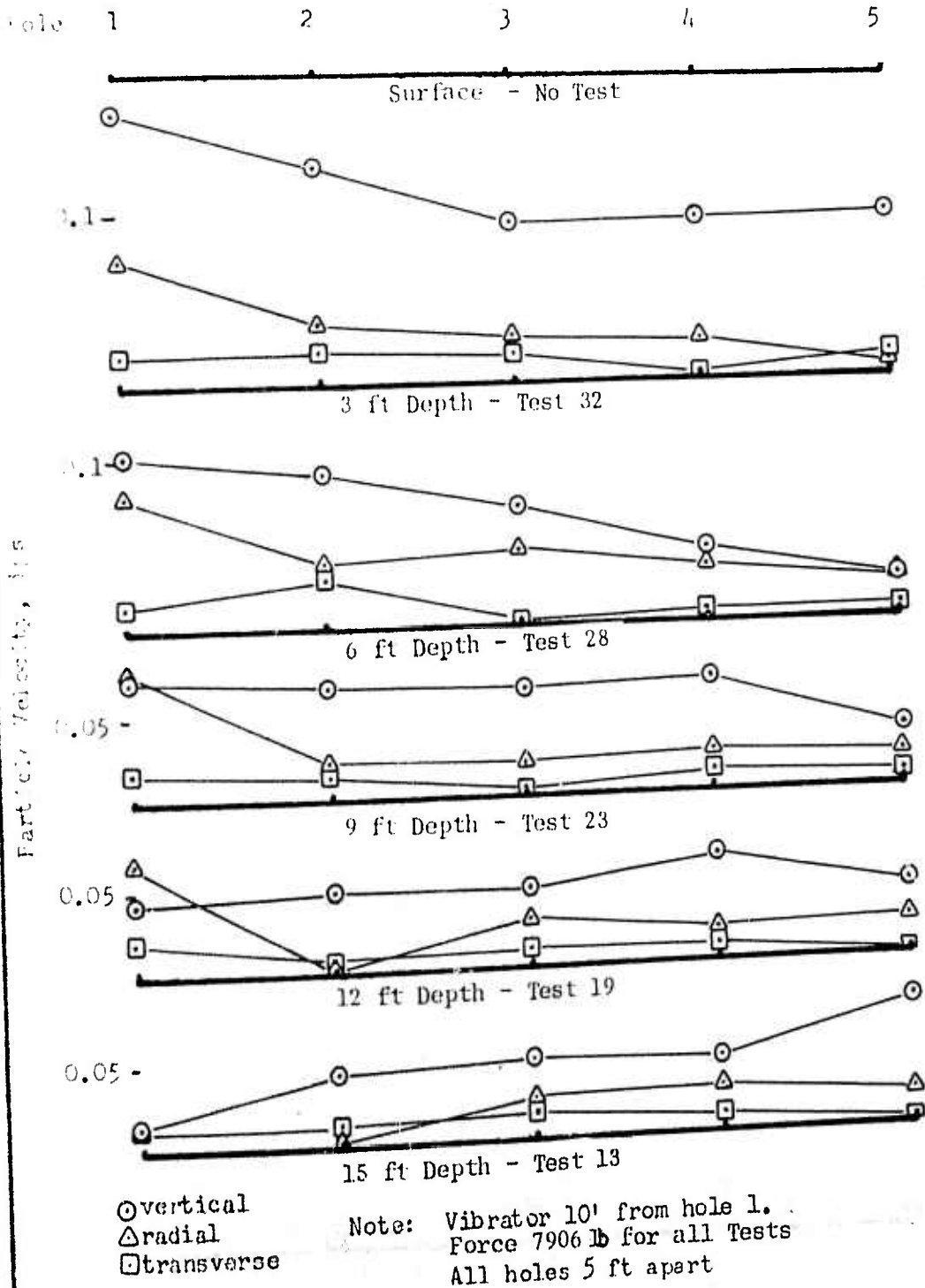
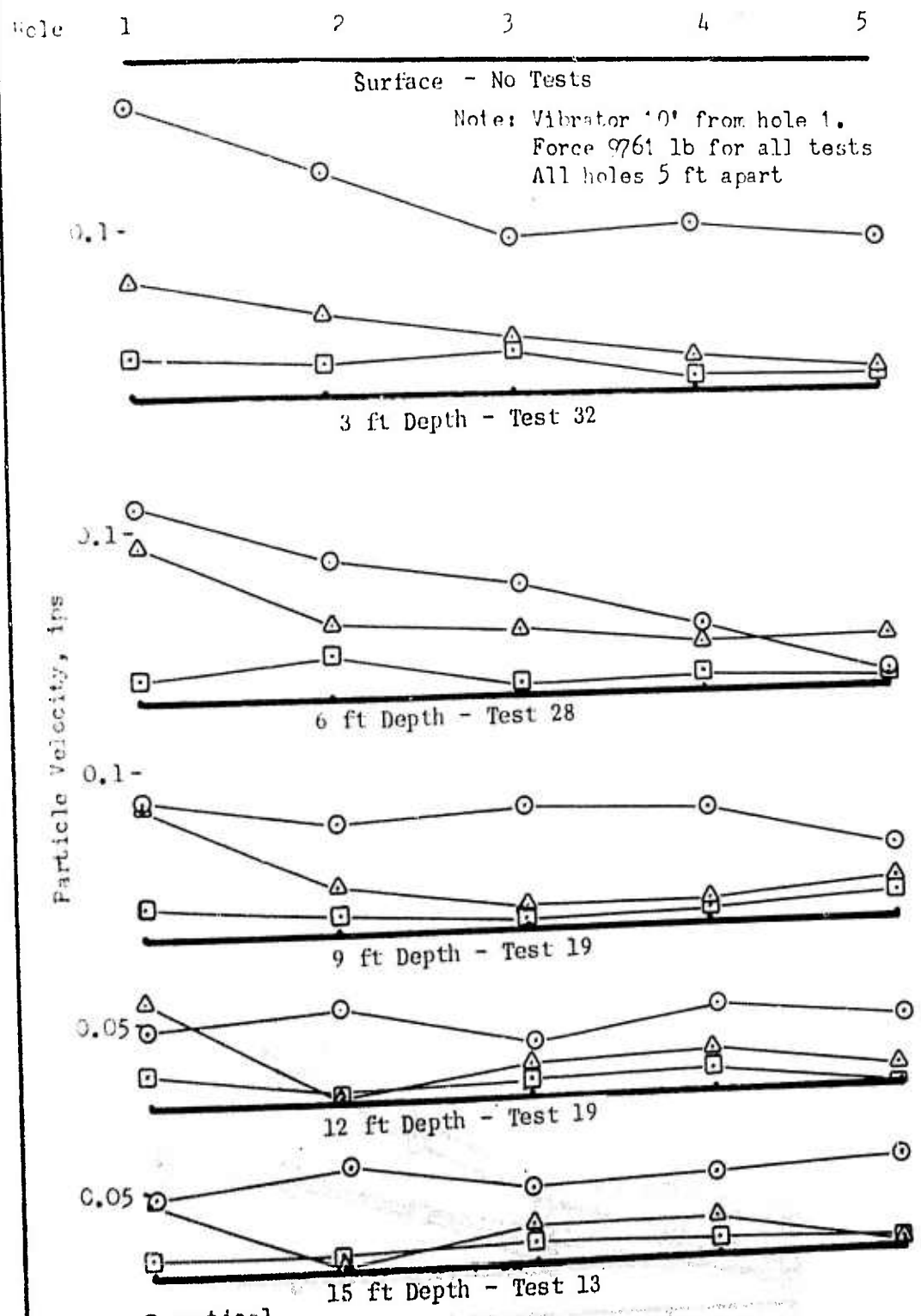


Figure F-7. Particle Velocity at Various Depths
 Holes 1-5, Frequency 45 Hz



Hole 1 2 3 4 5

Particle Velocity, ips

○ vertical
 △ radial
 □ transverse

Figure F-8. Particle Velocity at Various Depths
 Holes 1-5, Frequency 50 Hz

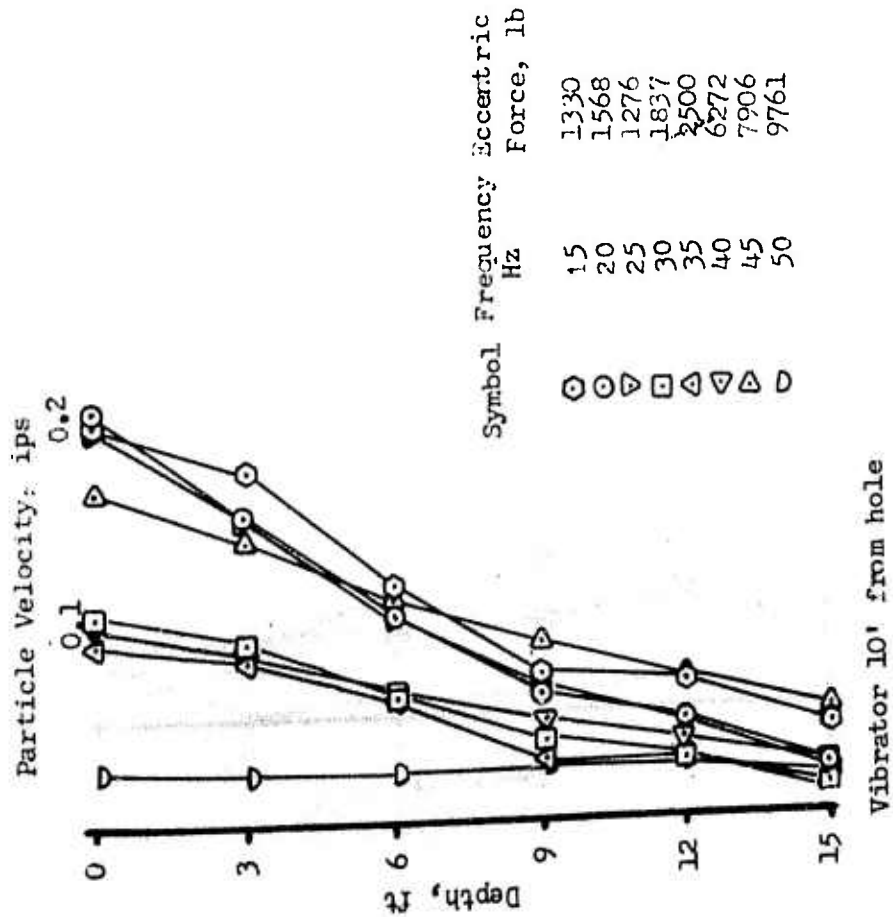
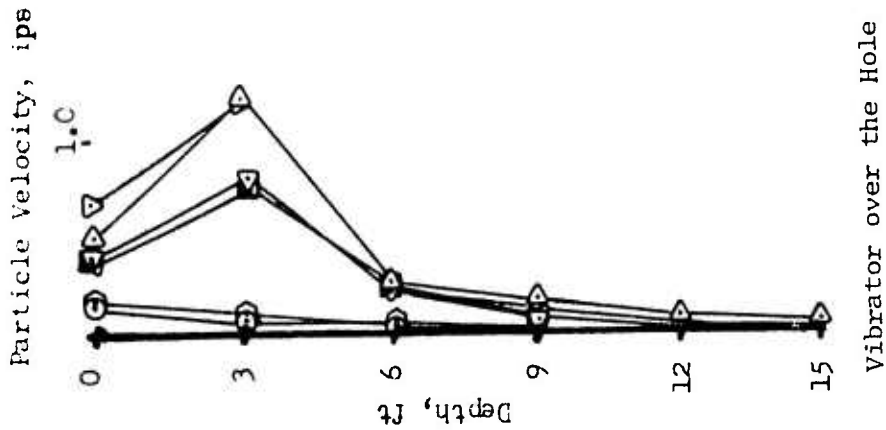
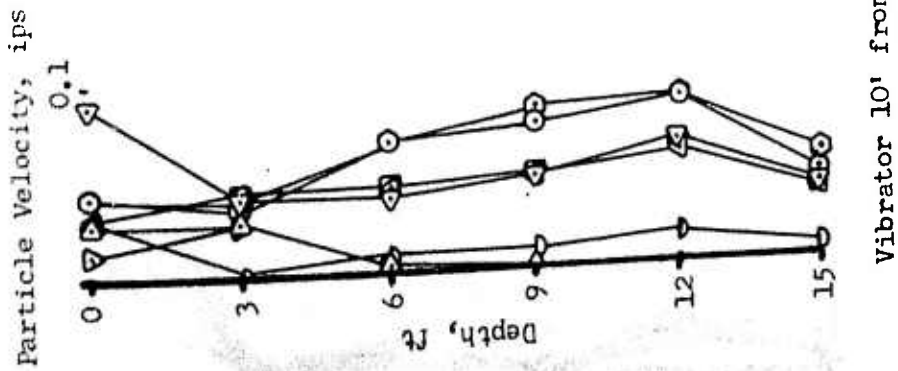
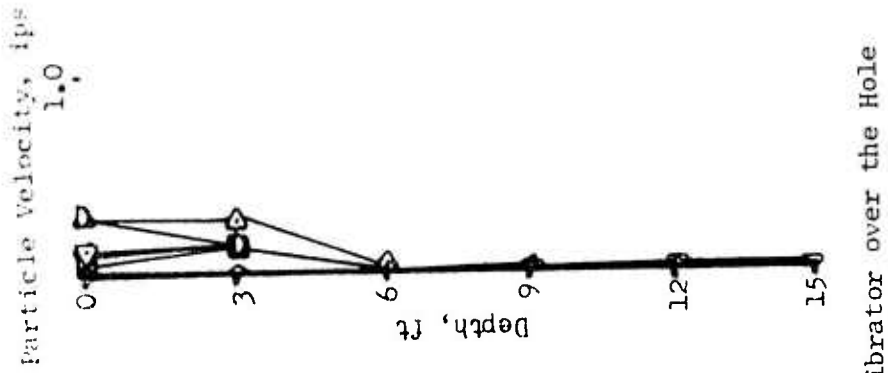
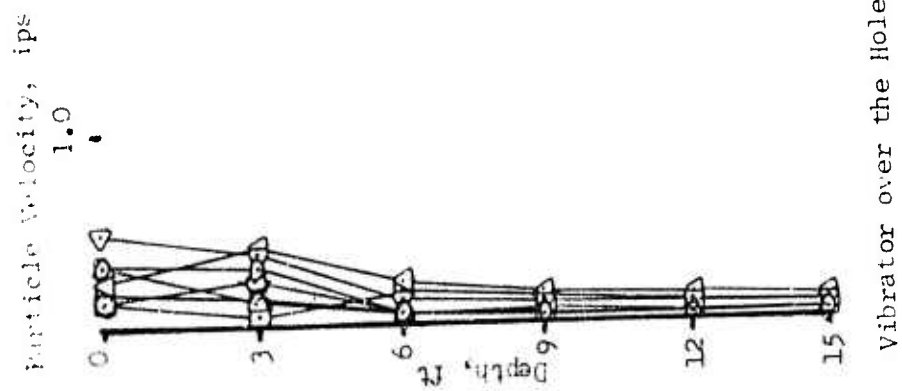


Figure F-9. Vertical Particle Velocity Versus Depth

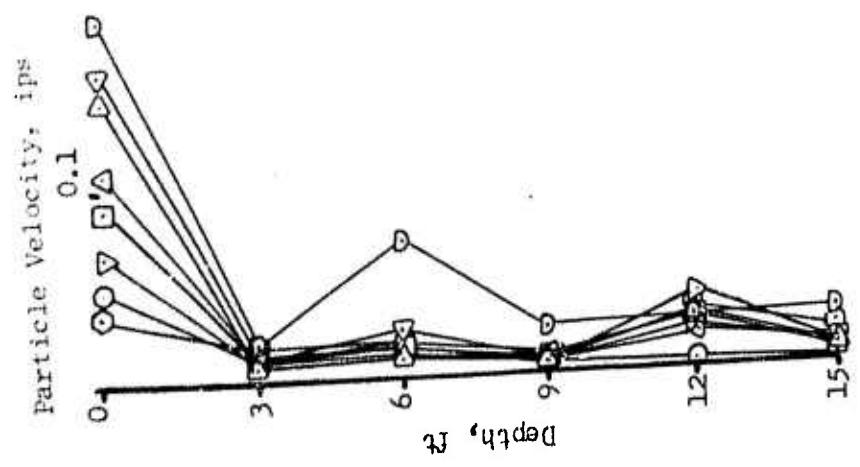


Symbol	Frequency Hz	Eccentric Force, lb
○	15	1330
○	20	1568
▽	25	1276
□	30	1827
△	35	2500
▽	40	6272
△	45	7906
▽	50	9761

Figure F-10. Radial Particle Velocity Versus Depth



Vibrator over the Hole



Vibrator 10' from hole

Symbol	Frequency Hz	Eccentric Force, lb
○	15	1330
○	20	1563
▽	25	1276
□	30	1837
△	35	2500
▽	40	6272
△	45	7906
▷	50	9761

Figure F-11. Transverse Particle Velocity Versus Depth

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13. ABSTRACT The Corps of Engineers has been determining the dynamic characteristic of soil and pavement systems by the use of surface-induced vibrations for approximately 10 years. This non-destructive vibratory technique is utilized by all federal agencies involved in planning the design of space-craft launch and operation facilities, antiballistic missile sites, heavy weapons, air field and road construction, radar towers or structures to house delicate electronic equipment when accurate and quick information pertaining to the ability of the soils to resist deformation is required. The Corps of Engineers has been actively engaged in improving and developing equipment and procedures for determining the elastic moduli (and thus the resistance to deformation) of foundation soils by vibratory techniques. The basic theory assumes that the Rayleigh waves generated by the vertical oscillations of a vibrator on the soils surface are for all practical purposes considered to be shear-waves. When the waves are propagated at a known frequency, the wave velocity is determined by the product of the wavelength and frequency. The shear-wave velocity propagation is assumed to take place within a depth equal to about one half the wavelength. The purpose of this investigation was to validate the existing theory by determining shear moduli from the displacements, velocities, frequencies, phase relationship, and wave shape characteristics recorded versus distance and depth from a vibratory source placed on the ground surface. Specifically to evaluate the concept that the surface wave propagates through a depth about equal to one half the wavelength. The results of this investigation indicated that the test section represented a fairly homogeneous elastic half-space which causes very little dispersion of a vibrating wave. Phase velocity measurements determined from surface and subsurface transducers were in excellent agreement. The conventional and dynamic laboratory and field tests of the soil from the site also indicated that the soil represented a homogeneous elastic half-space. The data collected offered no explanation or proof to support the theory that surface waves penetrate the soil for a depth equal to one half the wavelength. It was concluded from the analysis of the existing concept, that the surface wave propagates through a depth approximately equal to half the wavelength cannot be verified from tests of a homogeneous medium where there is practically no dispersion of the wave. It was also concluded, however, that the vibratory technique does provide a reliable description of the subsurface condition and that the accurate values of in situ moduli of the medium should be used in conjunction with conventional soils exploration and laboratory tests for the design of foundations subjected to dynamic loads. It was recommended that similar tests be conducted on test sections that would include more dispersive soil materials or layered systems. Additional information from similar tests would be beneficial in verifying the present procedures for determining in situ soil characteristics.		

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