VIBRATION COMPACTION OF COHESIVE SOILS

CALIFORNIA INSTITUTE OF TECHNOLOGY

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VIBRATION COMPACTION
OF
COHESIVE SOILS

A study of the basic laws
governing compaction of soil
by oscillating surface loads

A research project sponsored by
U. S. Navy Bureau of Yards and Docks,
U. S. Naval Civil Engineering and Research Laboratory
Port Hueneme, California

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ERRATA

Page xi.	Description of Figure No. 32 should be . . . . 1/6% Daxad 23 added.
Page 6.	The heading for column 5 should be $\frac{(F + W)}{A}$
Page 8.	In the third to the last line of Section 1.61, the reference should be to Appendix C on plates 19 through 52.
Facing Page 25.	Plate 2.2. The two photographs should be exchanged in position, caption (A) referring to the present lower photograph and caption (B) referring to the present upper photograph.
Page 29.	Fig. 2.10. L1, L2, L3 should be L8, L9, L10.
Page 32.	Fig. 2.12. The Frequency axis is identified incorrectly. The range should be from 6.8 to 18.8 c.p.s. in place of the present 8 to 28 c.p.s.
Page 37.	Fig. 2.18. The legend should be exchanged in position, the solid blocked circles indicating the moisture content range 11.0% to 13.7%, and the open circles indicating the moisture content range 7.9% to 9.9%.
Page 40.	Equation at bottom of page should read: $\gamma_d = 74.2 + 36.2\, p^{-0.655}$
Facing Page 40.	The caption under Photograph (A) should read: Density Measurement by Sand-Replacement Method.
Page 42.	Equation at top of page should read: $\gamma_d = 74.9 + 34.6\, p^{-0.589}$
Page 109.	Equation 5.29 and the following relationship should read: $E = E'(h + z)$ where $E' = \frac{dE}{dz}$ . . . .

PREFACE

Object

The object of this investigation was to determine the basic laws governing the compaction of cohesive soil by vibration.

Authority

The research was conducted under Contract No. NOy-22271 between the U.S. Navy Bureau of Yards and Docks, U.S. Naval Civil Engineering Research and Evaluation Laboratory, Construction Battalion Center, Port Hueneme, California, and the California Institute of Technology, Pasadena, California.

Personnel

For the Navy, this investigation was under the general direction of W. E. Davidson, Commander (CEC) U.S.N. Officer-in-Charge. Mr. John A. Bishop, Head, Soil Mechanics Laboratory, supervised the work and Mr. Charles White was contact engineer.

For the California Institute of Technology, the project was staffed by Professor Frederick J. Converse, Project Director, Professor George W. Housner, Consultant. Mr. William F. Jones, Project Engineer during most of the investigation, and Mr. John Nacos, Project Engineer during the early part of the work. Others who contributed to the investigation as technicians and special investigators were William Anderson, Del Hausmann, William Linville, Frank Lowry, Bent Lundbye, George Madsen, Carl Mattinson, Rudolf Ribbens, Robert Riley, and David Wilson.

History

The search for the basic rules governing the compaction of soils by vibration has been carried on at the California Institute of Technology for a number of years and has included both theoretical analysis and experimental investigation. Reports on the vibration compaction of sand were published in 1950 and 1952, covering the basic work on the vibration compaction of sand and the design and construction of a large vibrator. The success of the large vibrator in compacting sand led to the continuance of the studies into the field of cohesive soils.

Synposis

Preliminary field tests with a large vibrator resulted in the conclusion that the laws developed for sand did not apply to cohesive soil. A new theoretical approach to the determination of resonance of the vibrator soil system for cohesive soil was developed, but even though the vibrator was operated at resonance in a manner similar to that used for compacting sand, the degree of compaction of the cohesive soil was not satisfactory to these early tests.

A program of basic studies on a small scale was therefore undertaken at the California Institute of Technology laboratories. The general behavior of the soil was first studied by vibrating small quantities in a 4 inch diameter cylinder with dead weights acting on the surface of the soil. Later larger quantities in a box 4 ft. x 4 ft. x 2 ft. deep were used, with a Lazan Oscillator as the compacting force. More extensive studies were also carried on in a pit 4 ft. x 8 ft. x 3 ft. deep.

The preliminary field studies appeared to indicate that rather large forces were required to cause compaction of the cohesive soil. This led to a study of possibilities of reducing the force required to shear the soil by the introduction of chemicals to help break down the surface tension of the soil moisture and reduce the electrostatic bonds between the soil particles. A series of experiments including over fifty different chemicals resulted in the choice of Daxad 23 and sodium sulphate as offering the greatest possibilities for reducing the shearing resistance and permitting greater ease of compaction. During the laboratory experiments with the small scale equipment it was observed that the optimum moisture content for compaction by vibration was higher than that given by the standard Proctor compaction method. This proved to be very important to later successful tests on a larger scale.

A careful series of experiments was run in the field on a cohesive sandy loam using the Lazan oscillator, with unit dead weights on the order of 670 pounds. The variables were frequency, moisture content, the ratio of dynamic force to dead weight, and unit soil pressure. It was found that by a proper combination of these parameters it was possible to get excellent compaction, either with or without chemicals, with dead weight unit soil pressures on the order of 4 to 5 psi. It was established that former failures to compact cohesive soils were caused by the incorrect combination of the above variables. The results of the tests with the Lazan Oscillator were checked in the pit by the large Navy vibrator having a 3 ft x 5 ft base plate. A very rough field check at Port Hueneme indicated that these particular soils, at least, could be compacted by vibration.

All of the tests to date have been upon disturbed soils, that is, soils which are not in their natural structural condition but have been reworked and placed as fills. Sufficient work has not been done to thoroughly establish the variations in compaction resulting from changes over the complete range of each of the parameters, nor have the limits of cohesiveness at which reasonable compaction can be obtained been established. However, results to date are encouraging and appear to indicate that the vibrator compactor may eventually become a more useful tool for compacting soil than it has been in the past.

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NOTATION

The notation, with additions, is the same as that used in the report of January 1952, reproduced on page 98 of Appendix A of this report. The following include the additional terms, and also certain of the original notation, required for the work conducted since January 1952.

A	area of base-plate of vibrator or oscillator, or a constant coefficient
B	a constant coefficient
c	damping ratio β/β_c
f	unit pressure
F	dynamic force
h	total soil surcharge height
h_v	equivalent soil surcharge height due to the weight of the vibrator or oscillator
h_s	equivalent soil surcharge height due to the cohesiveness of the soil
p	rate of penetration of penetrometer
V	volume
w	moisture content of the soil
W	dead weight
γ_d	dry density of the soil
ω	frequency of vibration
ω_n	natural or resonant frequency

CHAPTER 1

PRELIMINARY FIELD INVESTIGATIONS

1.1 Object

The tests described below were made in order to determine whether or not the rules governing the compaction of sand by vibration could be applied directly to the compaction of cohesive soils.

1.2 Equipment

The vibrator used in these tests was basically the one previously used for the vibration compaction of sand and described in our report of January 1952, but there had been numerous modifications since that time.

The original base plate for the vibrator was 3 ft. x 5 ft. in plan and the sides were flared outward at an angle of about 30° so that it could be towed in any direction through loose sand. The new plate was 5 ft. by 6 ft. in plan, with three sides vertical and one end flared. The major outward differences between the original and modified vibrators are shown on Plate 1.1.

Four hydraulic motors were provided in the first unit to drive the shafts on which the eccentric weights are mounted. This was done in order to provide the flexibility desired for the initial research work. Since it was found that two motors could handle the load under practical working conditions, the two motors mounted at the rear, together with a great deal of piping which had been necessary to connect them into the system, were removed. This reduction made possible the elimination of one of the two oil pumps and the reduction of the oil storage capacity from 300 gallons to 75 gallons. These reductions in turn resulted in the elimination of the heavy trailer previously used and the placing of all of the driving equipment, including engine, storage tank, and pumps, on a small sled. The sled may be attached to the tractor and the vibrator compactor may then be attached to the sled, thus making a convenient train and enabling the vibrator to operate with less loss due to damping than occurred when the vibrator was attached directly to the tractor and was followed by the trailer.

The increased base plate area and the elimination of piping made it possible to add more weight and to lower the center of gravity of the vibrator.

The weights of the revised unit were as follows:

Oscillator Unit and Base Plate	- - - -	9,600 pounds
Removable Ballast Plates	- - - -	8,000 pounds
Maximum Dead Load	- - - -	17,600 pounds

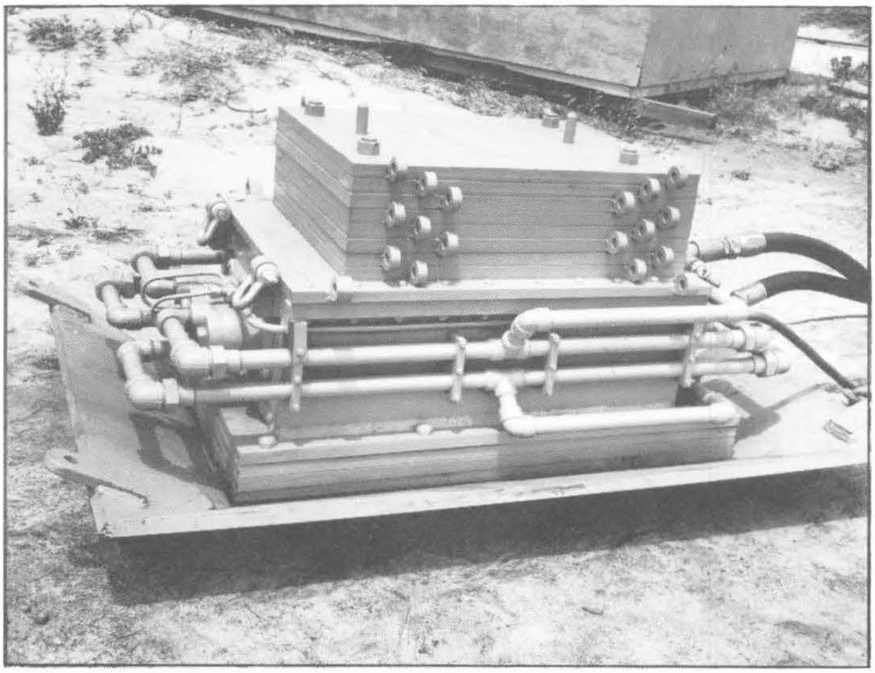
1.3 Theory of Resonance of Vibrator-Soil Systems

In the previous work on sand it was found that the best results were obtained when the vibrator-soil system was at resonance. A theory for the determination of the natural frequency of a vibrator soil system for cohesionless soil was presented in our report of January 1952 and is reproduced as appendix A of this report. In order to adopt the same ideas to cohesive soils modifications have been made, resulting in a general theory applicable to any type of soil. The modifications are presented in appendix B.

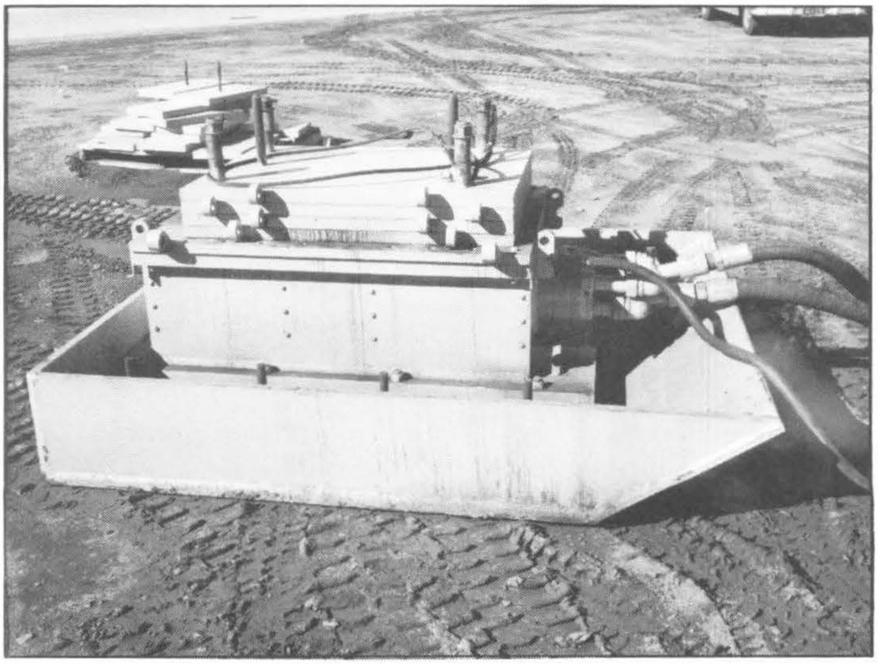
1.4 Test Procedure

The tests were performed at the Construction Battalion Center, Port Hueneme, California on a sandy loam soil having the following properties based on the U.S. Bureau of Soils classification: sand 70%, silt 20%, clay 10%, specific gravity 2.66, liquid limit 24, maximum density as determined by 25 blows of a 10 pound hammer on each of five layers in a 1/30th cubic foot cylinder was 124.5 pounds per cubic foot at an optimum moisture content of 10.5%. This soil is described as Port Hueneme sandy loam in the remainder of this report.

The sandy loam varied in thickness from 24 to 30 inches and below this was a three foot layer of more plastic silty loam. Moderately clean beach sand was found below the cohesive materials and extended to an unknown depth. The water table was approximately eight feet below the surface.



(A) Original Vibrator Mounted on 3 ft x 5 ft Base with Sloping Sides.



(B) Vibrator Modified and Mounted on 5 ft x 6 ft Base.

PLATE 1.1

The first group of tests was made with the vibrator at one location. The variables were dead weight and frequency. Since the dynamic force varied with the frequency as well as with the number of eccentrics on the rotating shafts, it was impossible to keep the dynamic force constant while the frequency was varied. However, since there were nine eccentrics on each shaft it was possible to vary the dynamic force at any given frequency and therefore to study the effects of the three variables, dead weight, dynamic force, and frequency.

The details of the procedure were as follows. Starting with three eccentrics on each shaft and using a dead load of 12,800 pounds, the vibrator was operated through a frequency range of approximately 7 to 18 cycles per second. In general the frequency was increased in increments of about 0.5 cps, measurements of the dynamic displacements being taken at each speed level. This type of run was repeated with dead loads of 14,800 pounds and 16,800 pounds.

The procedure outlined above was repeated three times using three eccentrics, six eccentrics, and nine eccentrics. The maximum frequency in each run was limited by the allowable dynamic force of 20,000 pounds.

Displacements of the vibrator were measured by means of a velocity pickup (Consolidated Engineering Corporation type 4-102A) and a vibration meter (Consolidated type 1-110B) on which either average velocities or peak to peak displacements could be read directly. The pickup was mounted within the vibrator box directly over the center of the base plate.

Frequencies were measured by means of a DuMont cathode-ray oscilloscope and a Hewlett-Packard audio oscillator type 202D. The frequencies were determined by matching the output of the audio oscillator against the output of the unknown frequency from the vertical velocity pickup. When the signals from both sources were of the same frequency a circle or ellipse was observed on the oscilloscope and the vibrator frequency was then read directly on the dial scale of the audio oscillator. The audio oscillator was calibrated before and after each test run by matching its output against a 60-cycle test signal on the oscilloscope. Frequencies of 10, 15, 20, 30, 40, and 60 cycles were conveniently checked by this method. Calibration curves were then plotted giving the correct vibrator frequency in terms of the audio oscillator readings.

1.5 Test Results

The details of the data and result sheets from these tests are shown on plates 7 to 53 in Appendix C. As an example, consider run number 12 on plate 7. This is for a vibrator dead load of 13,600 pounds and with 7 eccentrics on each shaft. The curve in the upper left hand corner of the sheet is the first plot of data from the test. Peak to peak displacement in mils is plotted against frequency and cycles per second. Except for the fact that the dynamic force changes with frequency, the frequency corresponding to maximum displacement would be considered resonant frequency. However, since the dynamic force varies as the square of the frequency, and since displacement varies directly as dynamic force as well as frequency ratio, the true resonant frequency is not given by this curve.To overcome this,the values of peak to peak displacement at any frequency are divided by the dynamic force at that frequency, thus giving a value that may be considered as the displacement per thousand pounds of dynamic force. The curve in the upper right corner of plate 7 is that which would be expected if the dynamic force were held constant at 1000 pounds. The frequency at the maximum displacement on this curve is therefore considered the resonant frequency.

In order to determine the damping ratio, which is the ratio of the actual damping to the critical damping, the damping curves from test results are compared with the theoretical curves. In order to exaggerate these curves so that a closer comparison can be made, the following procedure has been used. For any given value of "c" the amplification factor at any frequency has been divided by the ordinate corresponding to the frequency ratio $\omega/\omega_n=1$. This gives the normalized damping curves shown in the lower left hand corner. The normalized damping curve from the test, as shown by the heavy line, is sometimes skewed and sometimes does not follow a single theoretical normalized damping ratio line throughout its whole length. This indicates non-linearity in damping.

The curve at the lower right hand corner represents horsepower lost due to damping plotted against frequency in cycles per second. The maximum horsepower loss occurs at a frequency close to that of resonance but usually at a slightly higher value.

Table No. 1.1 represents a summary of the preliminary tests showing values taken from the curves together with calculated results. The first 11 runs are not shown since they were made on a different type of soil. Runs 12 and 13 were made using the 5 ft. by 6 ft. base plate.

1.6 Density Tests

A major objective of these preliminary tests was to determine whether or not the vibrator could compact cohesive soil when operating in the manner used for sand. The first tests made on natural soil with maximum vibrator weight and maximum dynamic force obtainable at resonant frequency resulted in no visible compaction of the soil, although a telephone pole some 200 ft away was observed to sway in rhythm. Since the soil was quite wet and had a firm structure it was decided to break it up with a bulldozer, windrow it, and see if it could be compacted in the disturbed state. The equipment at hand made it impossible to obtain uniform density, and the moisture content could not be controlled. While the resonant frequency, damping factor, and lost power were determined, there was apparently no appreciable compaction of the soil below the vibrator. The first assumption as to the reason for this was that the unit soil pressures were not high enough to shear the cohesive soil. As a rough way of trying this out rather than waiting for the construction of a new smaller base plate, two skids 5 ft. long and 12 inches wide by 6 ft. deep were bolted to the bottom of the vibrator, reducing the bearing area from 30 sq. ft. to 10 sq. ft.

The tests were made on disturbed material pushed into a windrow by a bulldozer, but since this soil was very loose, the skids sank deeply into it, and it is probable that the base plate itself was bearing on the soil at least part of the time. In order to eliminate this difficulty, the 6 inch by 12 inch skids were replaced by 12 inch by 12 inch skids. Runs 21 through 26 were made with 6 inch by 12 inch skids, and runs 31 through 39M, inclusive, were made with 12 inch by 12 inch skids. In runs T 37 to T 40 inclusive, skids were also used, but their form was modified as shown on the result sheets.

TABLE 1.1

VIBRATOR-SOIL DYNAMIC FACTORS

SUMMARY OF PRELIMINARY TESTS WITH

NAVY VIBRATOR ON SANDY LOAM AT PORT HUENEME

Run No.	Dead Weight W Kips	Dead Weight psi	Dynamic Force F Kips	F + W psi	F/W	Resonant Frequency ω_n c.p.s.	Damping Ratio c	Power Lost in Damping H.P.	U** or R
				5 ft. x 6 ft. Base Plate					
12	13.6	3.14	8.8	5.16	0.646	10.5	0.08	4.8	R
13	15.6	3.60	10.1	5.91	0.647	10.0	0.08	6.2	R
		5 ft. x 6 ft. Plate with 2 skids each 12 in. wide, 6 in. deep, 5 ft. long.							
21	16.8	11.68 3.66*	9.4	18.2 5.7*	0.559	9.6	0.06	5.0	R
22	16.8		7.5	17.1 5.4*	0.446	9.6	0.10	3.4	R
23	12.8	8.88 2.97	7.5	14.1 4.71	0.587	9.6	0.10	3.7	R
24	12.8		6.0	13.1 4.36*	0.469	10.1	0.08	1.9	R
25	9.6	6.66 2.22*	6.1	10.89 3.63*	0.635	10.2	0.20	3.3	R
26	16.8	11.68 3.66*	6.5	16.20 5.07*	0.387	10.5	0.15	2.0	R
		5 ft. x 6 ft. Plate with 2 skids each 12 in. x 12 in. x 5 ft. long.							
31	12.8	8.88	5.1	12.4	0.398	12.0	0.20	1.7	U
32	14.8	10.12	6.1	14.4	0.418	13.1	0.21	1.9	U
33	16.8	11.68	5.4	15.4	0.321	12.3	0.15	1.7	U
34	12.8	8.88	9.0	15.1	0.705	11.3	0.18	5.0	U
35	14.8	10.3	8.9	16.4	0.600	11.2	0.10	3.4	U
36	16.8	11.68	8.2	17.4	0.488	10.9	0.10	3.1	U
37	12.8	8.88	9.3	15.4	0.727	9.5	0.13	6.2	U
38	14.8	10.3	11.0	17.9	0.741	10.3	0.12	7.9	U
39	16.8	11.68	11.0	19.5	0.664	10.3	0.05	7.3	U
39M	16.8	11.68	9.4	18.3	0.569	9.7	0.03	6.8	R

(concluded next page)

TABLE 1.1 (CONCLUDED)

VIBRATOR-SOIL DYNAMIC FACTORS

SUMMARY OF PRELIMINARY TESTS WITH

NAVY VIBRATOR ON SANDY LOAM AT PORT HUENEME

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10
		5 ft. x 6 ft. Plate with 2 skids each 5 ft. long with 30° slope.							
30°T37	12.8	8.88 2.97*	11.0	16.5 5.53	0.860	10.3	0.10	8.3	R
30°T38	14.8	10.31 3.42*	12.5	19.02 6.31	0.844	11.0	0.15	8.2	R
30°T39	16.8	11.68 3.66*	10.3	18.85 5.91	0.614	10.1	0.13	6.2	U
		5 ft. x 6 ft. Plate with 2 skids each 5 ft. long with 15° slope.							
15°T40	11.6	2.69*	10.3	5.07	0.889	10.1	0.10	8.7	R
		5 ft. x 6 ft. Plate with 2 skids each 5 ft. long with 45° slope.							
45°T40	11.6	8.04 2.69*	8.9	14.2 4.75	0.767	8.8	0.08	6.0	R

*Because of the shallow skids which sank into the soil, the entire base plate may have been acting.
**U = undisturbed soil, R = reworked soil.

1.61 Compaction Test Procedure

A test area about 30 ft square was prepared by the bulldozer. Three parallel lines were staked out on this area about 10 feet apart, and four positions were marked on each line about equally spaced from each other. An indication of the initial density of the soil was obtained by driving a penetrometer into the soil at several points in each location.

A test was run by first bringing the vibrator up to resonant frequency and then towing it with a tractor along one of the lines. As the vibrator reached the first position, its forward motion was stopped for 5 seconds while vibration continued. It was then towed on to the second position and its forward motion stopped for a period of 10 seconds. At the next position its forward motion was stopped for 15 seconds, and at the last position for 20 seconds.

After this first run the speed of vibration was decreased to 80% of the resonant frequency. The vibrator was then towed along the second line stopping as before for 5, 10, and 20 seconds at each of the marked positions. This procedure was repeated a third time along the third line with the vibrator oscillating at a frequency 120% of the resonant frequency. Penetrometer readings and field densities by the sand method were taken at each of the positions where the vibrator stopped. The penetrometer readings before and after vibration compaction were plotted and are shown in the Appendix on plates 19 through 52. There appears to be no consistent pattern to these curves and no indications of good compaction except in occasional isolated instances.

1.7 Conclusions

As a result of these experiences it was obvious that work of a more basic nature was necessary, in which the conditions of moisture and initial density could be carefully controlled. Work with a large vibrator was therefore discontinued and the studies were transferred to small scale equipment at the laboratory.

Although the density tests were disappointing, some facts of interest may be gleaned from the results:

1. The ratio of the dynamic force to the dead weight was low in all cases. This was later found to be a major cause of lack of compaction.

2. The sum of the dynamic force plus the dead weight in pounds per square inch was quite high in most cases. While high unit soil pressure is an advantage, it is not as important as other factors if it is above a certain minimum for the particular soil being tested.

3. The resonant frequency lay between 9.5 and 11.3 cycles per second for all except three cases, where the F/W ratio was very low, and one case with the 45° skids.

4. The average value of the resonant frequency of all except the four cases mentioned in section 3 above was 10.3 cycles per second, and the maximum variation from average was 10%.

5. The three high values of resonant frequency (12.0, 13.1, 12.3 cps) mentioned as exceptions in section 3 above, had F/W ratios of 0.398, 0.418, and 0.321. One low value of resonant frequency (8.8) was for the vibrator with 45° skid shoes.

6. The damping ratios varied from 0.03 to 0.21, with an average value of 0.116.

7. The power lost in damping varied from 1.7 horsepower to 8.7 horsepower, with an average of 5 horsepower.

CHAPTER 2

FIELD INVESTIGATIONS OF BASIC LAWS FOR COHESIVE SOILS

2.1 Object

The tests described in the following text were made in order to provide basic resonance and compaction data from which optimum field operating conditions for a large vibrator compacting cohesive soils may be derived.

2.2 Scope

The principle of maximum compaction at resonance was assumed, and the effect on the resonant frequency of each of the following variables was investigated, within the limits outlined below.

(1) Soil moisture content - w;

(2) Dead weight of the vibrator - W;

(3) Dynamic force of the vibrator - F;

(4) Ratio of dynamic force to dead weight - F/W;

(5) Nominal contact pressure (the sum of dead weight plus dynamic force divided by base-plate area) between the soil and the base-plate of the vibrator - f.

In addition to the above variables, the effect on compaction of the addition of two chemicals to the soil was observed.

2.3 Investigations

The investigations were divided into three phases, as follows:

Phase 1: Preliminary tests to establish resonance data, and consisting of Tests L1, L2, and L3.

Phase 2: Preliminary tests to investigate compaction, and consisting of Tests L4 to L7.

Phase 3: Comprehensive tests to investigate compaction and consisting of

Tests L8 to L10, plus Tests HF3 incorporating a high-frequency vibrator.

A soil similar to the Port Hueneme sandy loam was found to exist on the campus of the California Institute of Technology, having the following physical and mechanical properties (shown also on Figs. 2.1 and 2.2):

Description (U. S. Bureau of Soils Classification): Sandy Loam

Particle size distribution:

58% sand, 28% silt, 14% clay (U. S. Bureau of Soils)

54% sand, 41% silt, 5% clay (M.I.T. Classification)

Liquid limit: 25

Plasticity index: 4

Specific gravity: 2.67

Max. density: 123.4 pounds/cubic foot at an optimum moisture content of 10.3% (10 pound hammer; 25 blows per lift; 18 in. drop; 5 lifts; 1/30 cu. ft. mould).

This soil is referred to in the remainder of the text as C.I.T. sandy loam.

2.4 Phase 1. Preliminary tests to establish resonance data.

Test No.	Location of Test	Variables	Chemical Added	Location of Data and Plates
L1	4 ft. x 4 ft. x 2 ft. box	w, **W**, F, f	None	Appendix D
L2	do.	w, **W**, F, f	Sod. Sulph.	do.
L3	do.	w, **W**, F, f	None	do.

2.4.1 Equipment and Preparation

A box was constructed using the natural ground as the base, the sides being made of 2 in. x 8 in. boards spanning horizontally between 4 in. x 4 in. corner posts firmly embedded in the ground, forming a square box with sides 4 ft. long by 2 ft. high.

C.I.T. sandy loam was placed in this box in batches of 500 pounds, each batch being uniformly tamped to a measured thickness of 3-3/4 inches, to give an initial density of

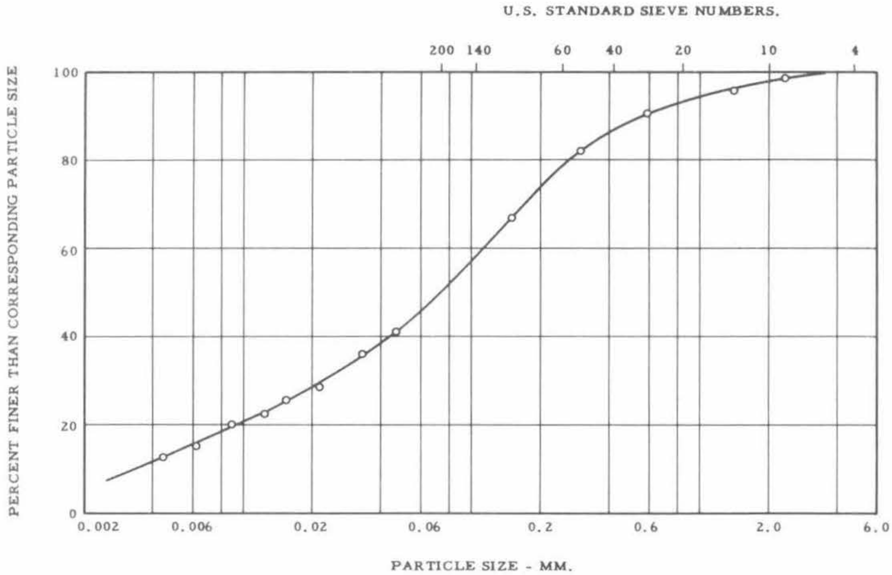


Fig. 2.1 PARTICLE SIZE DISTRIBUTION CURVE, C.I.T. SANDY LOAM.

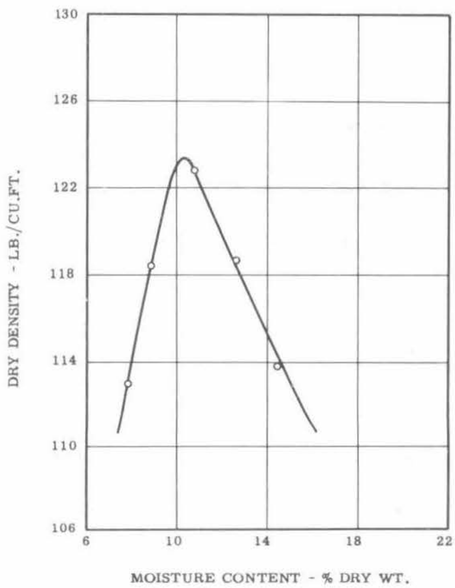


Fig. 2.2 DRY DENSITY vs MOISTURE CONTENT RELATIONSHIP (MOD. A.A.S.H.O.). C.I.T. SANDY LOAM.

100 pounds per cubic foot (81% of miximum). In Tests L1, the desired moisture content was attained by mixing measured quantities of water and soil by shovel, but in L2 and all subsequent tests, mixing was done in a plaster mixer. After filling, the box was covered by a tarpaulin and allowed to stand over-night in order to permit capillary movement of moisture and a consequent more uniform moisture distribution.

Sodium sulphate was added to the soil for Tests L2. In accordance with conclusions from the laboratory vibration-table tests, see Chapter 4, the chemical was dissolved in water before being added to the soil.

Moisture contents at various levels were determined after the tests by the use of a Volumeter. No density measurements were made.

The Lazan oscillator was supported in all the tests on a steel plate 19.2 inches in diameter bearing directly on the soil surface. Steel plates sliding on rods screwed vertically into the top of the oscillator enabled the dead weight to be varied. Lateral stability of the whole assembly was provided by three bars radiating from the Lazan to posts set in the natural ground, the horizontal angle between the bars being 120°. Turnbuckles were provided to enable the bars to be adjusted to any desired position.

2.4.2 Procedure

Having chosen a ratio of dynamic force to dead weight for a particular test, the lowest possible frequency obtainable with the oscillator to give the required force was determined and adjustment made to give that dynamic force output. The oscillator was then brought up to speed rapidly and the speed was maintained for 10 seconds. At the end of 10 sec., the speed was reduced to zero and the oscillator readjusted to give the required force at a speed 1 cps above the previous test. The oscillator was then brought up to speed and held for 10 seconds, as before. The process was repeated in steps of 1 cps until the full range of the oscillator had been covered. Vertical displacements of the Lazan were determined by means of a Consolidated Engineering Velocity Pick-up and Meter.

The complete series of tests was run on the same soil and the final resonant frequency was that for the soil after the cumulative compactive effect had occurred.

TABLE 2.1

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L1, L2, AND L3

SOIL-C.I.T. SANDY LOAM

Test No.	Moisture Content at 10 in. % Dry Wt.	Chem. Content	Base Plate Area sq.in.	Dead Wt. W lbs.	Dynamic Force at Resonance F lbs.	Ratio F/W	Nominal Force F + W lbs.	Max.Nominal Contact Pressure p.s.i.	Resonant Frequency c.p.s.
L1-1	6.5	None	289	384.5	231	0.60	616	2.13	28.4
-2	6.5	do.	do.	384.5	365	0.95	750	2.60	28.4
-3	6.5	do.	do.	795.1	476	0.60	1,271	4.40	29.2
-4	6.5	do.	do.	795.1	754	0.95	1,549	5.36	26.0
-5	6.5	do.	do.	1,170	701.5	0.60	1,872	6.48	23.3
-6	6.5	do.	do.	1,170	1,111	0.95	2,281	7.90	18.3
-7	10.0	do.	do.	384.5	231	0.60	616	2.13	28.7
-8	10.0	do.	do.	384.5	365	0.95	750	2.60	27.8
-9	10.0	do.	do.	795.1	476	0.60	1,271	4.40	29.2
-10	10.0	do.	do.	795.1	754	0.95	1,549	5.36	22.2
-11	10.0	do.	do.	1,170	701.5	0.60	1,872	6.48	24.2
-12	10.0	do.	do.	1,170	1,111	0.95	2,281	7.90	18.0
-13	15.0	do.	do.	384.5	231	0.60	616	2.13	28.5
-14	15.0	do.	do.	384.5	365	0.95	750	2.60	23.0
-15	15.0	do.	do.	795.1	476	0.60	1,271	4.40	23.6
-16	15.0	do.	do.	795.1	754	0.95	1,549	5.36	18.0
-17	15.0	do.	do.	1,170	701.5	0.60	1,872	6.48	14.5
-18	15.0	do.	do.	1,170	1,111	0.95	2,281	7.90	19.2
L2-1	10.0	Na_2SO_4	289	589	443	0.752	1,032	3.57	25.3
-2	do.	do.	do.	589	502	0.853	1,091	3.77	23.0
-3	do.	do.	do.	589	560	0.951	1,149	3.98	21.3
-4	do.	do.	do.	791	594	0.751	1,385	4.79	21.3
-5	do.	do.	do.	791	672	0.849	1,463	5.06	19.8
-6	do.	do.	do.	791	751	0.949	1,542	5.34	19.0

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TABLE 2.1 (CONTINUED)

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L1, L2, AND L3

SOIL-C.I.T. SANDY LOAM

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10
L2-7	10.0	Na_2SO_4	289	1,042	782	0.750	1,824	6.31	19.8
-8	do.	do.	do.	1,042	886	0.850	1,928	6.67	17.8
-9	do.	do.	do.	1,042	989	0.949	2,031	7.03	18.0
-10	12.0	do.	do.	589	443	0.752	1,032	3.57	22.5
-11	do.	do.	do.	589	502	0.853	1,091	3.77	22.2
-12	do.	do.	do.	589	560	0.951	1,149	3.98	20.8
-13	do.	do.	do.	791	594	0.751	1,385	4.79	20.3
-14	do.	do.	do.	791	672	0.849	1,463	5.06	19.0
-15	do.	do.	do.	791	751	0.949	1,542	5.34	16.8
-16	do.	do.	do.	1,042	782	0.750	1,824	6.31	19.3
-17	do.	do.	do.	1,042	886	0.850	1,928	6.67	17.0
-18	do.	do.	do	1,042	989	0.949	2,031	7.03	14.0
-19	15.0	do.	do.	589	443	0.752	1,032	3.57	19.3
-20	do.	do.	do.	589	502	0.853	1,091	3.77	17.3
-21	do.	do.	do.	589	560	0.951	1,149	3.98	14.0
-22	do.	do.	do.	791	594	0.751	1,385	4.79	14.0
L3-1	7.2	None	289	589	443	0.752	1,032	3.57	23.0
-2	do.	do.	do.	589	502	0.853	1,091	3.77	22.3
-3	do.	do.	do.	589	560	0.951	1,149	3.98	19.8
-4	do.	do.	do.	791	594	0.751	1,385	4.79	20.6
-5	do.	do.	do.	791	672	0.849	1,463	5.06	18.7
-6	do.	do.	do.	791	751	0.949	1,542	5.34	16.8
-7	do.	do.	do.	1,042	782	0.750	1,824	6.31	19.7
-8	do.	do.	do.	1,042	886	0.850	1,928	6.67	17.0
-9	do.	do.	do.	1,042	989	0.949	2,031	7.03	18.0

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TABLE 2.1 (CONCLUDED)

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L1, L2, AND L3

SOIL-C.I.T. SANDY LOAM

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10
L3-10	8.0	None	289	589	443	0.752	1,032	3.57	22.3
-11	do.	do.	do.	589	502	0.853	1,091	3.77	21.2
-12	do.	do.	do.	589	560	0.951	1,149	3.98	20.4
-13	do.	do.	do.	791	594	0.751	1,385	4.79	20.6
-14	do.	do.	do.	791	672	0.849	1,463	5.06	20.3
-15	do.	do.	do.	791	751	0.949	1,542	5.34	17.7
-16	do.	do.	do.	1,042	782	0.750	1,824	6.31	18.8
-17	do.	do.	do.	1,042	886	0.850	1,928	6.67	17.0
-18	do.	do.	do.	589	443	0.752	1,032	3.57	26.4
-19	do.	do.	do.	589	502	0.85	1,091	3.77	25.0
-20	do.	do.	do.	589	560	0.95	1,149	3.98	23.0
-21	do.	do.	do.	791	198	0.25	989	3.42	27.0
-22	do.	do.	do.	791	277	0.35	1,068	3.70	27.0
-23	do.	do.	do.	791	356	0.45	1,147	3.97	27.0
-24	do.	do.	do.	791	435	0.55	1,226	4.25	26.4
-25	do.	do.	do.	791	515	0.651	1,306	4.52	24.0
-26	do.	do.	do.	791	594	0.751	1,385	4.79	22.0
-27	do.	do.	do.	791	672	0.849	1,463	5.06	20.5
-28	do.	do.	do.	791	751	0.949	1,542	5.34	17.9
-29	do.	do.	do.	1,042	782	0.750	1,824	6.31	20.0
-30	do.	do.	do.	1,042	886	0.850	1,928	6.67	18.0

2.4.3 Test Results

The details of the data and result sheets from these tests are shown in Table 2.1 and Appendix D, Plates 1 to 17. In many cases, due to the limitations of the Lazan oscillator, the resonant frequency could not be obtained for a particular combination of dead weight and dynamic force. Thus, in Test L2-9, Appendix D, Fig. 2.45, the minimum operating frequency of the Lazan to give a dynamic force output of 989 pounds was 18.0 cps, which, for the conditions of that test, was higher than resonant frequency. Similarly, in Test L3-23, Appendix D, Fig. 2.81, the maximum allowable operating frequency of the Lazan, 27.0 cps, was attained without passing through resonance, which, for the conditions of that test was clearly greater than 27.0 cps.

The four independent variables in these tests reduced to three, since the base-plate was of constant area, and hence the nominal unit contact pressure was directly proportional to the total dead weight plus dynamic force. To establish any relationship between the resonant frequency and the three variables, it was necessary to fix two of these at some constant value, and consider the relation between resonance and the third variable. Unfortunately, due to resonance being unattainable in some tests, it was possible in only a few cases to obtain the minimum of three points required to establish, or even suggest, any form of relationship. Where this was possible, the data have been plotted and are shown on Figs. Nos. 2.3, 2.4, and 2.5.

2.4.4 Conclusions

Within the limits of this investigation the following tentative conclusions may be drawn.

Effect of Soil Moisture Content. As soil moisture content increases, the resonant frequency decreases, as is clearly shown on Fig. 2.3. The majority of the remaining data, (shown on Figures 2.19 to 2.88) although incomplete, exhibit the same trend.

Effect of Total Dead Weight Plus Dynamic Force. As the total dead weight plus dynamic force increases, the resonant frequency decreases. Although in this series of tests the nominal unit contact pressure was directly proportional to the total weight, it cannot be considered established that increasing the unit pressure decreases the resonant frequency, since only one size of base plate was used in all of the tests.

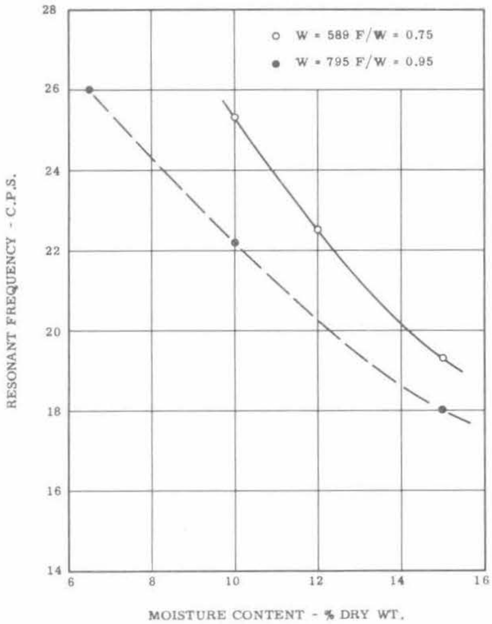


Fig. 2.3 RESONANT FREQUENCY vs MOISTURE CONTENT RELATIONSHIP. TESTS L1, L2, L3.

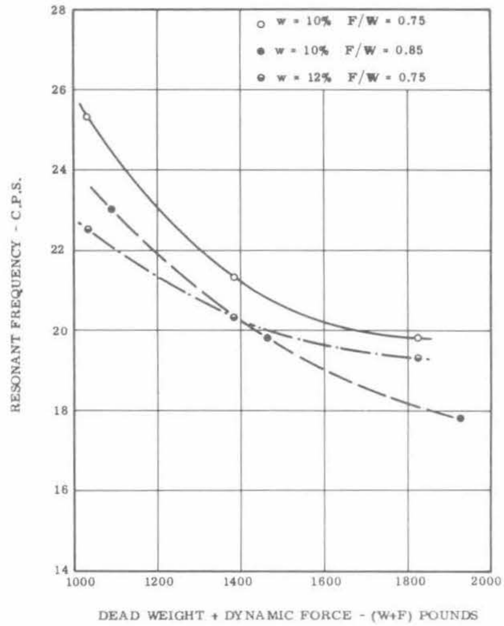


Fig. 2.4 RESONANT FREQUENCY vs DEAD WEIGHT PLUS DYNAMIC FORCE RELATIONSHIP. TESTS L1, L2, L3.

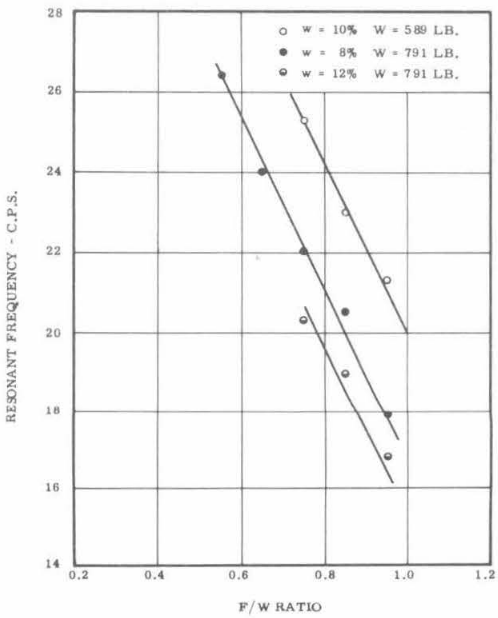


Fig. 2.5 RESONANT FREQUENCY vs F/W RATIO RELATIONSHIP. TESTS L1, L2, L3.

Effect of the Ratio of Dynamic Force to Dead Weight. The effect on the resonant frequency of the ratio dynamic force to dead weight, or F/W, is the most pronounced, there being apparently a linear relationship in which resonant frequency decreases as the ratio F/W increases. The relationship has the form

$$\omega_n = A - B(F/W)$$

$$\text{where } \omega_n = \text{resonant frequency}$$

$$A \,\&\, B = \text{constants}$$

The constant 'A' appears to be a function of the moisture content and dead weight, giving

$$\omega_n = f(w, W) - B(F/W)$$

Insufficient data are available to establish the numerical values in the equation. The most significant aspect is that if the shape of the base plate, in this instance circular, is held constant, the relation between the resonant frequency and the ratio F/W appears to be linear, with a constant slope for all values of the remaining independent variables (moisture content and dead weight), see Figure 2.5.

2.5 Phase 2 - Preliminary tests to investigate compaction.

Test No.	Location of Test	Variables	Chemical Added	Location of Data and Plates
L4	4 ft. x 4 ft. x 2 ft. Box	w, F, f	None	Appendix D
L5	do.	w, F, f	Sod. Sulph.	do.
L6	do.	F, f	do.	do.
L7	do.	F, f	do.	do.

2.5.1 Equipment and Procedure

The tests were conducted in the same box described in Section 2.4.1 of this report, and the soil was prepared and placed in the same manner. In addition, a Proctor needle was used to take density measurements at various levels as the soil was being placed and later being removed after conducting the test.

For each test a constant dead weight and operating frequency was chosen on

TABLE 2.2

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L4, L5, AND L6

SOIL - C.I.T. SANDY LOAM

Test No.	Moisture Content 10'' below G.L.	Chem. Content	Base Plate Area	Dead Weight	Dynamic Force	Ratio F/W	Nominal Total Force (W+F)	Max. Nominal Contact Pressure (W+F)/A	Operating Frequency
	% dry wt.		sq.in.	W pounds	F pounds		pounds	p.s.i.	c.p.s.
L4-1	11.25	None	289	794	754.0	0.95	1,548	5.35	20.5
L4-2	10.2	do.	do.	794	Varied	0.6 - 1.7	Varied	4.67	21.0
L5-1	8.9	Sod. Sulph.	do.	1,111	do.	0.6 - 1.4	do.	5.38	22.0
L5-2	10.0	do.	do.	794	do.	0.6 - 1.7	do.	4.67	21.0
L5-3	12.0	do.	do.	1,111	do.	0.6 - 1.4	do.	5.38	22.0
L6-1	11.3	do.	193	1,339	do.	0.6 - 1.2	do.	8.32	22.0
L7-1	10.0	do.	60*	1,339	do.	0.6 - 1.2	do.	26.77	22.0

Note: *indicates use of two 3 in. x 3 in. x 10 in. skids as a base.

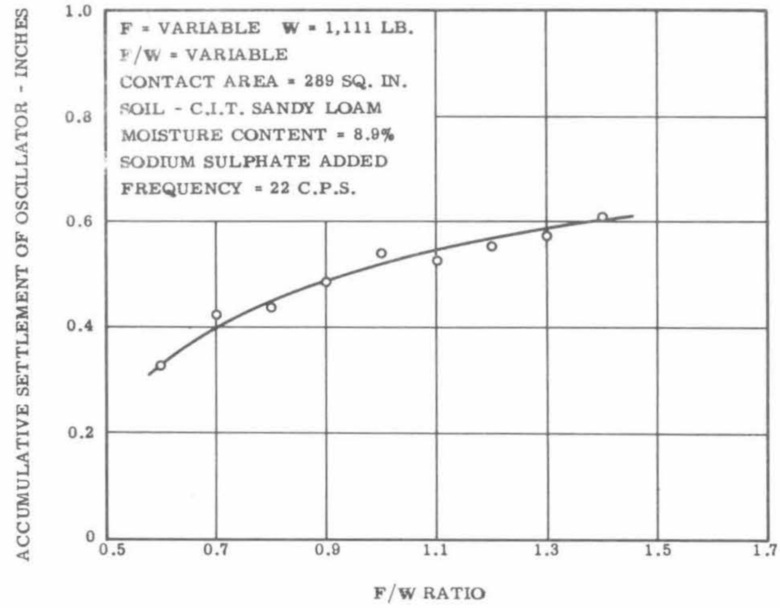


Fig. 2.6 SETTLEMENT vs F/W RATIO RELATIONSHIP
TEST L5-1.

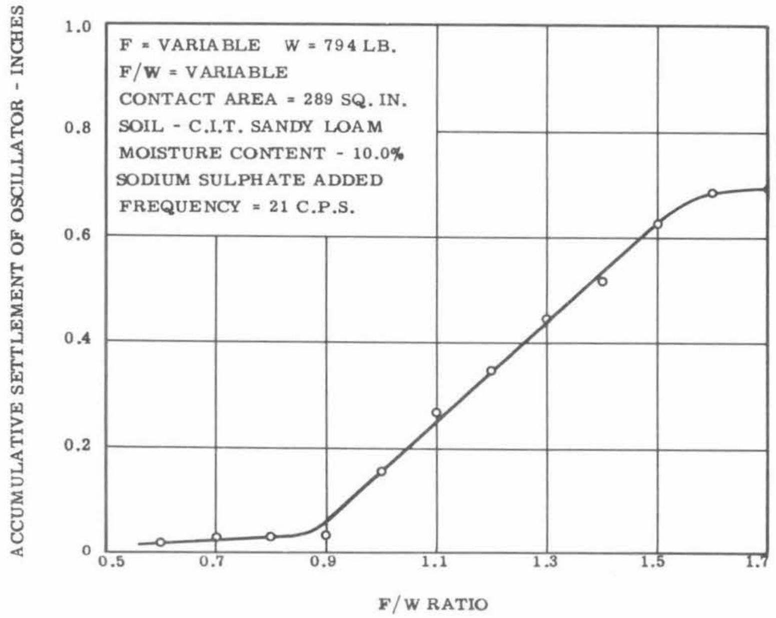


Fig. 2.7 SETTLEMENT vs F/W RATIO RELATIONSHIP.
TEST L5-2.

basis of the preliminary tests described in Section 2.4 of this report. The dynamic force was varied to give a range in the ratio F/W from 0.6 to 1.7, and in each test, vibration was carried out for 10 seconds at each value of the ratio F/W, the settlement of the oscillator being noted after each period of vibration. The settlements were determined with the use of a surveyor's level reading a scale divided into 0.01 inches and attached to a guide pole screwed to the top of the vibrator.

To obtain the effect on compaction of contact pressure, a smaller diameter base-plate was used in one test and two wooden skids, 3 in. x 3 in. x 10 in., in a second test, see Plate 2.1, (A).

2.5.2 Test Results

The results proved to be inconclusive and disappointing. The settlements obtained were small and the approximate densities indicated by Proctor needle tests and stainless steel cylinder tests were considerably less than 90% of maximum. In addition, it had been expected that when the F/W ratio was approximately unity, there would be a marked increase in the rate of settlement of the oscillator, but except in Test L5-2, this was not the case.

The lack of success in these tests is attributed in the incorrect combination of the controlled variables, in particular the operation at a constant frequency over the whole range of the F/W ratio.

The results of the tests are shown in Table 2.2 and the settlement curves for Tests L5-1 and L5-2 in Figs. 2.6 and 2.7.

2.5.3 Conclusions

Despite the utilization of relatively high dead weight, dynamic force, and contact pressure, densities equal to, or greater than 90% of maximum were not attained. It is clear that the correct combination of the parameters mentioned, together with resonant frequency and suitable soil moisture content, is required before successful compaction can be achieved.

2.6 Phase 3. Comprehensive tests to investigate compaction.

Test No.	Location of Test	Variables	Chemical Added	Location of Data and Plates
L8	Pit	w, W, F, f	None	Appendix D
L9	do.	w, F, f	Daxad 23	do.
L10	do.	w, W, F, A, f	Sod. Sulph.	do.
HF3	do.	W, F, A, f	do.	do.

2.6.1 Test Procedure.

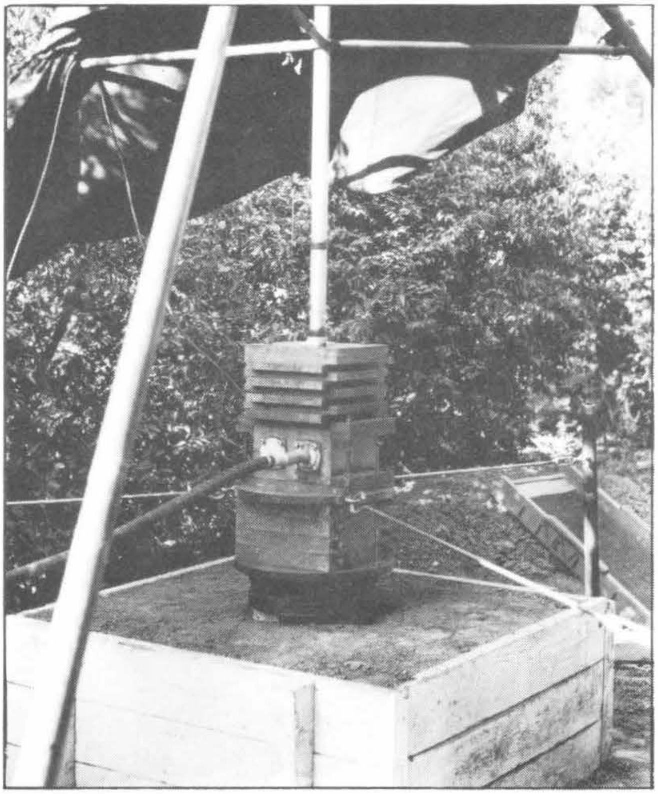
Test Area

In place of the raised box previously used, a pit measuring 3 ft. deep by 4 ft. wide by 8 ft. long was constructed in the natural ground at the California Institute of Technology campus. End slopes 3 ft. long were provided from the ground surface to the bottom of the pit to facilitate rolling of the test soil during placing. In order to form a cushion of loose soil below the more carefully prepared test soil, some of the excavated soil was replaced in the pit and loosely compacted, until the final depth of the pit had been reduced to between 2 ft. and 2 ft. 6 in. Choice of a pit of the given dimensions was made so that there was sufficient space for two compaction tests to be run, with any variables due to soil conditions eliminated.

Preparation of Soil in Pit. Soil with water added to produce the proper moisture content was mixed in a plaster mixer in batches of 100 pounds, and when sufficient soil had been placed in the pit to form a lift approximately 4 in. thick, it was raked and rolled to a density of approximately 75% of maximum. The pit with the mixer in operation is shown on Plate 2.1, (B). Repeated determinations of soil moisture in the storage bin and of the soil being placed in the pit were made during these operations, by use of a quick-drying apparatus (a device which blows warm air through a sample retained on a multi-layer fine screen).

The amount of water added to the soil being mixed was carefully controlled in an attempt to obtain a uniform moisture content. Nevertheless, a certain amount of variation in moisture content with depth occurred, although this tended towards uniformity by allowing the soil to stand over-night before conducting the tests. Daxad 23 and Sodium Sulphate were added for some tests, in the proportions 1/6% by weight and 1/3% by weight respectively, and in both cases were dissolved in water before being mixed with the soil.

After the final rolling and levelling of the surface of the pit, a grid-work of lines forming squares of 10 in. sides was sçratched on the surface with a spatula. The cone



(A) Lazan Oscillator mounted on Skids for Tests L7-1

(B) Mixing Soil for Preparation of Test Pit at C.I.T.

PLATE 2.1

(A) Lazan Oscillator in position in Pit at C.I.T. before Vibration

(B) Lazan Oscillator in position in Pit at C.I.T. after Vibration

PLATE 2.2

penetrometer described in Chapter 3 of this report was used to obtain penetration readings in the center of each alternate grid square before conducting the test, and at the points formed by the interesection of each of the grid lines after the test. In addition, at least one density measurement by the sand-replacement method was made after each test, and two moisture content samples were taken at each of four or five depths below ground level.

Test Procedure. From consideration of previous tests, it was thought that a dead weight of 800 pounds was the maximum that could be used with a ratio **F/W** of unity, and result in a resonant frequency within the range of the oscillator. This value, however, proved to be too high, and the standard dead weight was reduced to 670 pounds, used in conjunction with the smallest base-plate available, one of 15.7 in. diameter (193 sq. inches).

The test was run as in 2.4.2, the frequency being increased in increments of 1 cps over the whole range of the oscillator, and vertical displacement readings being taken at each frequency. After completion of the cycle, a second cycle was occasionally run, but generally a 30-second period of vibration at resonance was the final operation.

During vibration the Lazan tended to move laterally, and since the earlier method of tie-bars attached to posts was impracticable with the new arrangement of the test area, difficulty was found in preventing the occurrence of considerable lateral movement. Photographs (A) and (B), Plate 2.2, taken before and after one particular test show, by the amount of disturbed soil thrown up, the extent of this movement. However, the problem was solved in a very simple manner when it was found that a slight force applied by hand, as high as possible on the guide post, and in the direction in which the vibrator was moving, caused the motion to reverse itself. In this manner it was possible to restrict the lateral movement to a very small quantity, and at the same time dispense with the use of the large steel tripod seen in Plate 2.2.

2.6.2 Test Results

The results of these tests proved to be highly satisfactory. Of sixteen conducted, six resulted in densities in excess of 90% of maximum, and of the remainder, the cause of low densities was usually quite clear. The results are shown in Table 2.3 and Appendix D, Figures 2.89 to 2.111.

Based on the Proctor-type curve (Fig. 2.2) for this soil, and on the premise that the best moisture content for ease of compaction should be somewhat higher than the laboratory-determined optimum, it was desired that the field value should be 12% $\pm$ 1%. Apart from

TABLE 2.3

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L8, L9, L10, HF3

SOIL-C.I.T. SANDY LOAM

Test No.	Moisture Content 10" Below G.L. % dry wt.	Chem-ical Content	Base Plate Area sq.in.	Dead Weight W lbs.	Dynamic Force F lbs.	Ratio F/W	Nominal Total Force F+W lbs.	Max. Nominal Contact Pressure (F+W)/A lbs/sq.in.	Res. Frequency cps	Max. Dry Density % of Max.
L8-1	14.6	None	193	800	800	1.0	1600	8.29	14.5	85.8
L8-2	14.45	do.	193	716	716	1.0	1432	7.42	15.5	83.9
L8-3	12.9	do.	193	420	420	1.0	840	4.35	(c)	90.9
L8-4	12.55	do.	193	670	670	1.0	1340	6.94	17.3	92.1
L9-1	12.34	Daxad 23	193	670	670	1.0	1340	6.94	17.2	96.1
L9-2	12.34	do.	193	670	670	1.0	1340	6.94	16.7	94.0
L9-3	11.6	do.	193	670	670	1.0	1340	6.94	16.2	98.5 (f)
L9-4	11.5	do.	193	670	335	0.5	1005	5.21	22.8	78.4
L10-1	10.8	Sod. Sulph.	193	670	670	1.0	1340	6.94	15.7	94.0 (d)

(concluded next page)

TABLE 2.3 (CONCLUDED)

SUMMARY OF DATA FROM TESTS WITH LAZAN OSCILLATOR

TESTS L8, L9, L10, HF3

SOIL-C.I.T. SANDY LOAM

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11
L10-2	10.5	Sod. Sulph.	193	670	670	1.0	1340	6.94	15.2	87.5 (d)
L10-3	10.8	do.	193	420	420	1.0	840	4.35	17.0	77.7 (e)
L10-4	10.5	do.	288	670	670	1.0	1340	4.65	17.0	74.1 (e)
L10-5	12.1	do.	193	445	445 (a)	1.0	890	4.61	14.0	75.0 (e)
L10-6	12.5	do.	193	670	500	0.75	1270	6.06	17.3	79.6 (e)
HF-3-1	10.5	do.	114	151	57	0.38	208	1.83	27.2	62.6 (e)
HF-3-2	10.6	do.	193	640	640 (b)	1.0	1280	6.63	15.7	75.3 (e)

Notes:

(a) In horizontal plane
(b) Output of Lazan. Viber H. F. unit operating simultaneously at 8000 RPM with output of 1450 pounds.
(c) Instruments broke down
(d) Determined approximately from penetration data.
(e) Determined by sand-replacement method.
(f) Penetrometer approximate at this value.

Tests L8-1 and -2, which were initially set up only as trial tests, and the L10-1 to -4 and HF3-1 and -2 tests, this condition was fulfilled. It should be noted that two factors operated to cause the variation from the values aimed at, namely:

(i) The addition of sodium sulphate to the soil caused difficulty in attaining consistent values for moisture content, the value invariably being less than would be expected on the basis of the amount of moisture added to the soil during mixing. Why this should be so is not clear, but an hypothesis is put forward in the section of this report devoted to the effect of chemicals on the soil. One reason for the attainment of the desired moisture contents in Tests L10-5 and L10-6 is attributed to the fact that these two were conducted some weeks after Tests L10-1 to -4 and HF3-1 and -2, by which time the effect of the sodium sulphate had almost disappeared.

(ii) The extremely hot and drying conditions under which the tests in question were conducted. Additional water was added during mixing to allow for evaporation losses, but this apparently was not sufficient for the purpose.

A significant feature of the tests was the marked increase in compaction when a certain combination of the variables was obtained. This is illustrated particularly well in the series of Tests L8-4 to L9-3. Following this, when the variables were altered to determine their respective effect on compaction, there was a marked decrease in the resulting densities except in Tests L10-1 and L10-2, where the previously successful combination was again used. Furthermore, as Figures 2.8 to 2.11 indicate, the results tend largely to conform to definite patterns indicating the existence of definite laws relating the variables. However, insufficient tests were conducted, with each variable controlled within previously defined close limits, to establish these laws.

2.6.3. Conclusions

General. Compaction of a moderately cohesive soil to a density greater than 90% of maximum can be obtained if the correct combination of parameters is made. This combination is (a) F/W ratio near unity, (b) soil moisture slightly above laboratory optimum (c) operation at resonant frequency and (d) unit dead weight soil pressure at least 3 psi.

Effect of Soil Moisture Content on Compaction. The amount of compaction is dependent on the moisture content of the soil. This value should be somewhat higher than the laboratory determined value based on the modified AASHO method. For the soil tested, the range in values in the field was approximately 10.4% to 12.4% with an optimum at 11.6%, compared with the laboratory-determined optimum of 10.3%.

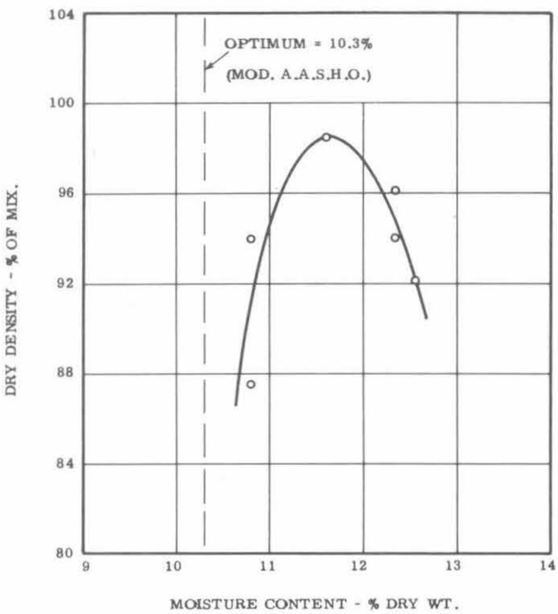


Fig. 2.8 DRY DENSITY vs MOISTURE CONTENT RELATIONSHIP. TESTS L8, L9, L10, HF3.

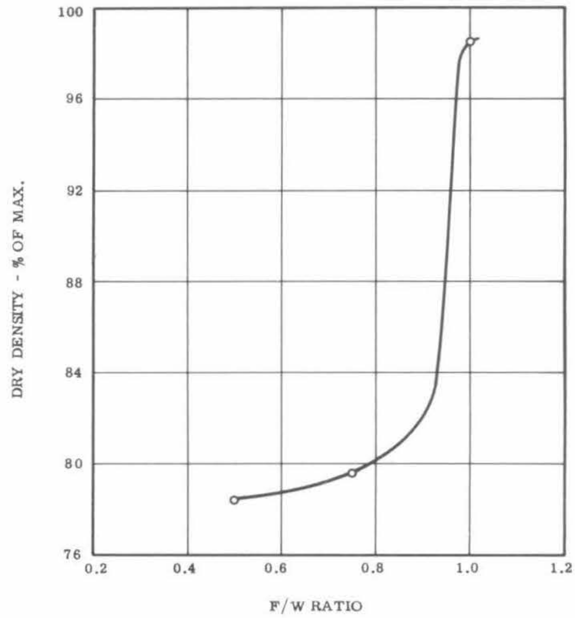


Fig. 2.9 DENSITY vs F/W RATIO RELATIONSHIP. TESTS L9, L10.

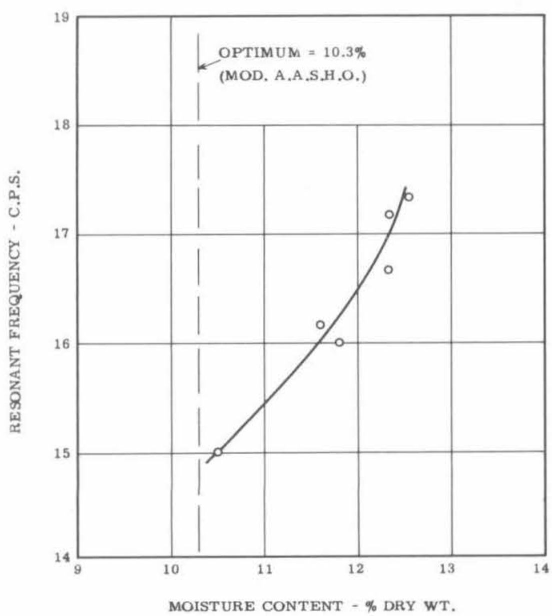


Fig. 2.10 RESONANT FREQUENCY vs MOISTURE CONTENT RELATIONSHIP. TESTS L1, L2, L3.

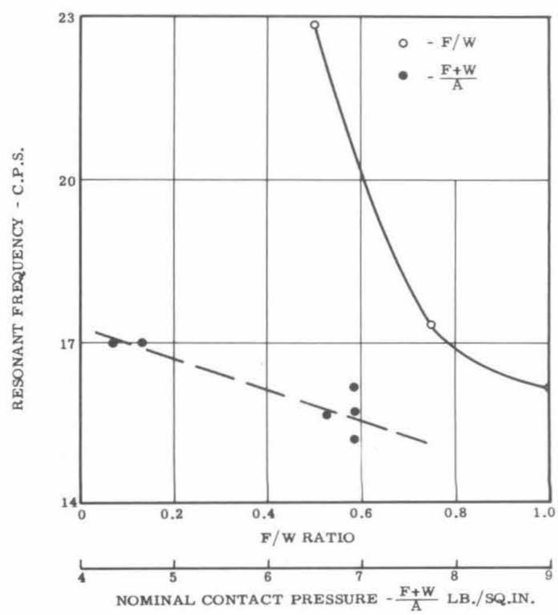


Fig. 2.11 RELATIONSHIPS BETWEEN RESONANT FREQUENCY AND F/W RATIO AND BETWEEN RESONANT FREQUENCY AND NOMINAL CONTACT PRESSURE. TESTS L8, L9, L10, HF3.

Effect of Total Force and Nominal Contact Pressure on Compaction. From the meager data obtained to date there does not appear to be any specific relationships between the density obtained and the total force (dead weight plus dynamic force) of the oscillator, except insofar as an increase in total force on a given area increases the nominal contact pressure. An increase in nominal contact pressure causes an increase in compaction if the other parameters are correctly chosen.

Effect of the F/W Ratio on Compaction. There is a marked increase in the amount of compaction obtained as the value of the F/W ratio approaches unity.

Effect of Chemical Content on Compaction. The addition of the chemicals Daxad 23 and Sodium Sulphate, the most promising ones chosen on the basis of laboratory tests, does not so materially increase the resulting compaction that their use should be considered absolutely essential. It is possible that the rate with which compaction takes place is increased, although this has not been definitely established. It is also possible that the penetrating action of Daxad 23 will be desirable in difficult mixing conditions.

2.7 Confirmatory Compaction Tests with Large Vibrator

To check the conclusions of the field investigations made with the Lazan oscillator, a number of tests were conducted with the large U. S. Navy vibrator operating on the 3 ft. x 5 ft. base. This work was done in two phases, the first being conducted in the pit at the California Institute of Technology, and the second at the Port Hueneme Naval Base.

2.7.1 Tests at the California Institute of Technology

Test Procedure

The pit in which the tests with the Lazan oscillator had been conducted was enlarged to a width of 6 ft. The soil was mixed to a moisture content of about $12\frac{1}{2}\%$, and was placed in the same manner as in the previous work.

Based on earlier work with the large vibrator, a combination of 8 eccentrics per shaft with a dead weight of 9,200 pounds was expected to result in a dynamic force of about 9,200 lb. at resonance. However, the dynamic force at resonance proved to be much higher than this, and the resulting vibrator motion was very unsteady, with oscillations occurring about both horizontal axes. The test was halted to add further dead weight, resulting in an F/W ratio more nearly equal to unity, and less oscillation, see Table 2.4, Tests V1-A and V1-B. A second test was conducted with sodium sulphate added to the soil.

TABLE 2.4

SUMMARY OF DATA FROM TESTS WITH LARGE VIBRATOR AT C.I.T.

TESTS V1-A, V1-B, V2

SOIL - C.I.T. SANDY LOAM

Test No.	Moisture Content 10" below G.L. % dry wt.	Chemical Content	Base Plate Area A sq. in.	Dead Weight W lb.	Dynamic Force at Resonance F lb.	Ratio F/W	Nominal Contact Force (F+W) lb.	Maximum Nominal Contact Pressure (F+W)/A lb./sq.in.	Resonant Frequency c.p.s.	Maximum Dry Density % of max.
V1-A	12.4	None	2,160	9,200	9,830	1.07	19,030	8.81	10.3	
V1-B	12.4	do.	2,160	11,200	13,070	1.17	24,270	11.23	11.9	95.6
V2	13.9	Sod. Sulph.	2,160	11,200	11,140	1.00	22,340	10.35	11.0	91.7

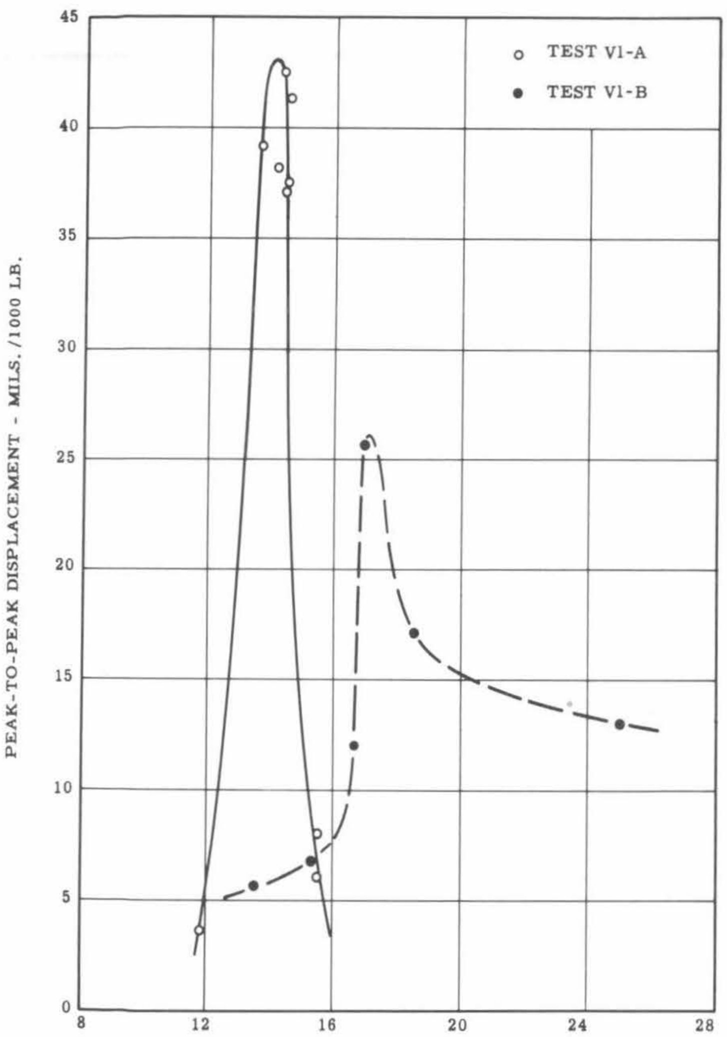


Fig. 2.12 DISPLACEMENT vs FREQUENCY RELATIONSHIP. TESTS V1-A AND V1-B WITH LARGE VIBRATOR.

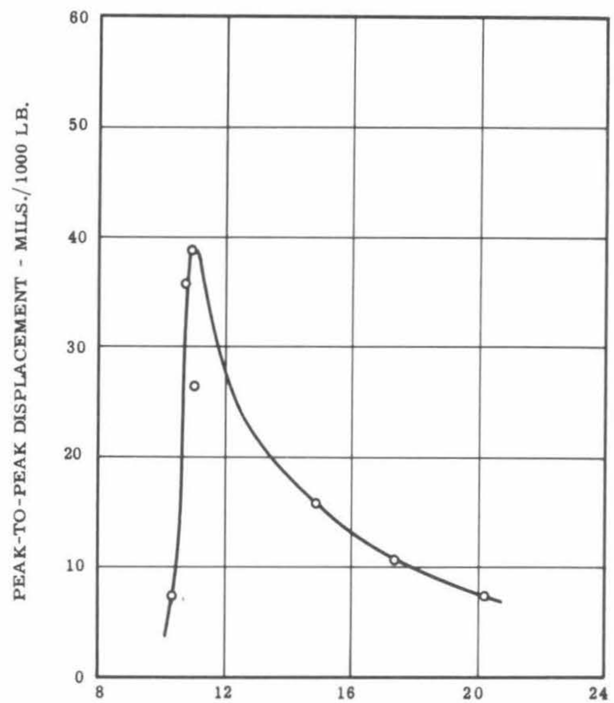
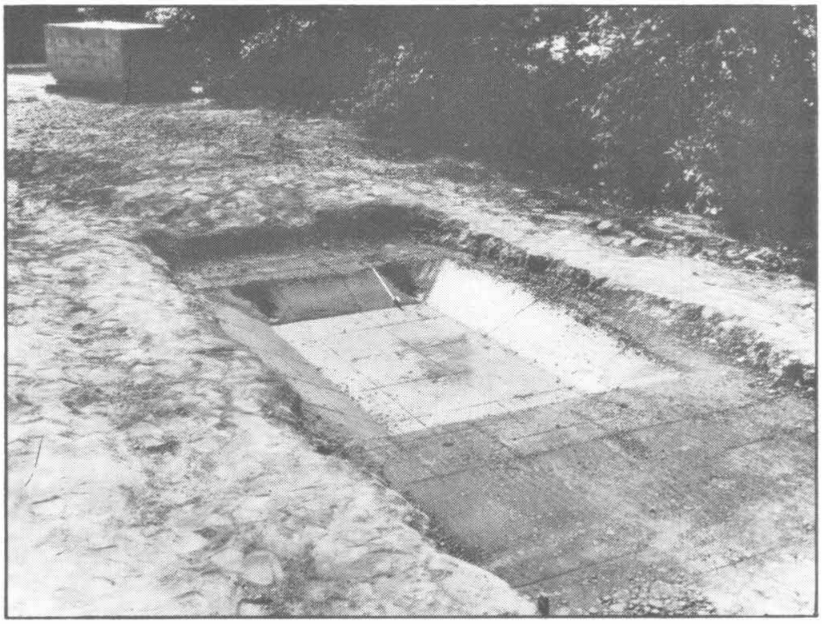


Fig. 2.13 DISPLACEMENT vs FREQUENCY RELATIONSHIP. TEST V2 WITH LARGE VIBRATOR.



(A) Compacted area in Pit at C.I.T. after Tests V1 with Large Vibrator

In all the tests, penetrometer readings were made on a grid before and after vibration. The form of the grid, re-marked after the test, and the amount of settlement obtained is shown on Plate 2.3, (A). Densities were also measured by the sand-replacement method.

Test Results and Conclusions. The results proved highly satisfactory, in both cases densities in excess of 90% of maximum being obtained. That a lower density was obtained with sodium sulphate present in the soil was attributed to the high moisture content of the soil in this test, which in addition resulted in heaving of the soil round the vibrator, a condition which had not been observed in any of the previous tests.

While these tests showed definite agreement with the conclusions arising from the investigations with the Lazan vibrator, a certain amount of confinement was present due to the relatively hard, dry condition of the sides of the pit. Consequently further tests were desirable under more normal field conditions.

2.7.2 Tests at the U.S. Naval Base, Port Hueneme, California

Test Procedure and Results

The tests were conducted at the equipment-handling instruction area of the Port Hueneme Naval Base, the soil consisting of a sandy loam with the following properties:

U.S. Bureau of Soils Classification - Sandy loam with 7% gravel, 59% sand, 27% silt, and 7% clay.

Liquid Limit - 28

Plasticity Index - 10

Specific Gravity - 2.64

Maximum density 126.3 lb./cu. ft. at optimum moisture content of 10.2% (10 lb. hammer, 5 lifts, 25 blows per lift, 18 in. drop, 1/30 cu. ft. mould).

With the wholehearted cooperation of the personnel of the equipment instruction school, an area 160 ft. x 40 ft. was prepared for the tests by sprinkling with water and disturbing the soil with a rooter to a depth of between 24 in. and 30 in. Unfortunately neither equipment nor time was available for careful scarafying and mixing to get a uniform mass of soil of the proper moisture content. The tests are therefore considered as only roughly indicative of what could be obtained under usual construction procedure.

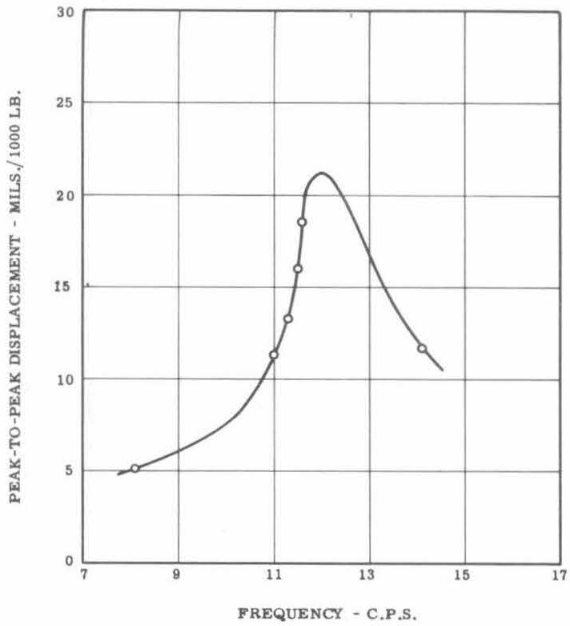


Fig. 2.14 DISPLACEMENT vs FREQUENCY RELATIONSHIP. PRELIMINARY TEST NO. 1 WITH LARGE VIBRATOR.

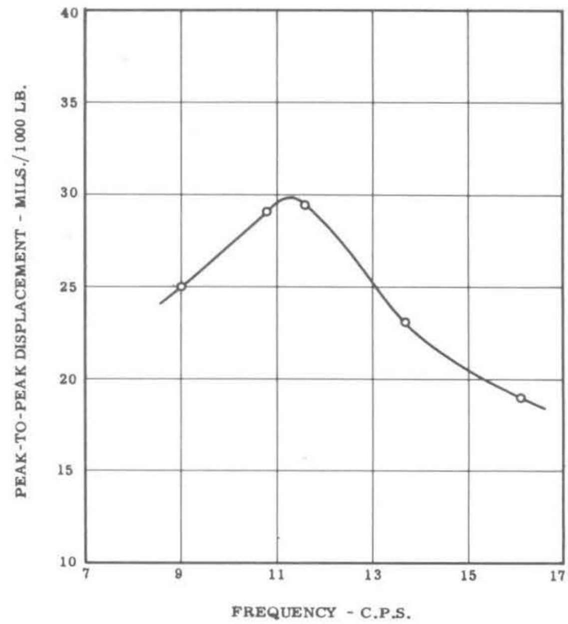


Fig. 2.15 DISPLACEMENTS vs FREQUENCY RELATIONSHIP. PRELIMINARY TEST NO. 2 WITH LARGE VIBRATOR.

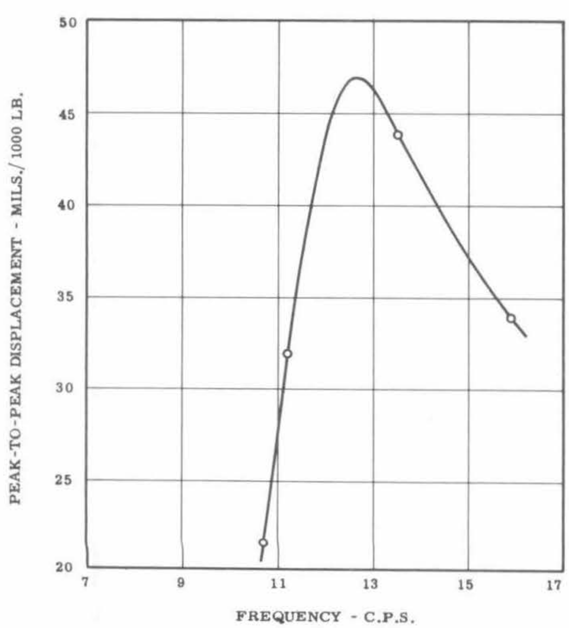


Fig. 2.16 DISPLACEMENTS vs FREQUENCY RELATIONSHIP. PRELIMINARY TEST NO. 3 WITH LARGE VIBRATOR.

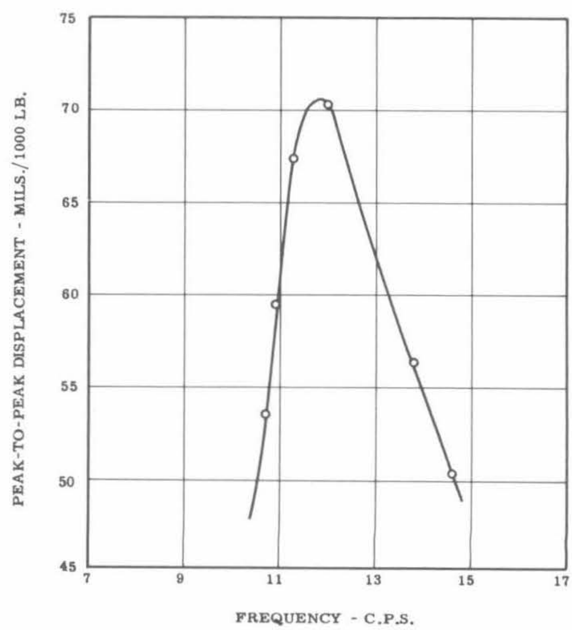


Fig. 2.17 DISPLACEMENT vs FREQUENCY RELATIONSHIP. TESTS V3 TO V7 WITH LARGE VIBRATOR.

A first series of three tests were made on undisturbed, relatively dry soil, to obtain an idea as to the resonant frequency of the vibrator-soil mass. Based on the tests at the California Institute of Technology it was anticipated that the combination of 8 eccentrics per shaft with 11,200 lb. dead weight would result in a dynamic force of about 11,200 lbs. at resonance. However, a dynamic force of 13,300 lb. occurred at resonance, as can be seen from Fig. 2.14. The dead weight was therefore increased to 13,200 lbs. and both a stationary and a towing test were made, see Figures 2.15 and 2.16. As a consequence of this work a dead weight of 13,200 lbs. was used in a second series of four stationary tests made on ground which had been watered and rooted until the moisture appeared to be adequate. Resonance for the more moist soil condition was established in the first test of the four at 11.7 cps, see Fig. 2.17, the dynamic force at this speed being 12,620 lb. and the F/W ratio 0.956. Densities before and after the four tests, Numbers V3 to V6, were determined by the sand-replacement method. Tables 2.5 and 2.6 show the results of the first and second series.

TABLE 2.5

PRELIMINARY TESTS

Test No.	Eccentrics per Shaft	Dead Weight W. lb.	Stationary or Towing	Resonant Frequency c.p.s.	Dynamic Force at Resonance F. lb.	F/W
1	8	11,200	Stationary	12.0	13,300	1.19
2	8	13,200	Stationary	11.2	11,500	0.87
3	8	13,200	Towing	12.7	14,800	1.12

TABLE 2.6

TESTS V3 TO V6

Dead Weight = 13,200 lb.
Resonant Frequency = 11.7 cps (established in V3)
Dynamic Force at resonance = 12,600 lbs.
F/W ratio = 0.96
Nominal unit contact pressure = 11.95 psi
Type of test = Stationary

Test No.	Moisture Content	Time of Vibration	Maximum recorded density
	% dry wt.	sec.	% of max.
V3	9.90	360	89.0
V4	7.90	10	80.0
V5	9.79	30	82.8
V6	8.26	60	87.0

It was clear from these tests that the soil moisture content was too low, a value being desired somewhat higher than the laboratory-determined optimum of 10.2%. The site was therefore resprinkled and reworked. Also, noting that resonant frequency increases if the vibrator is being towed, the following tests, V7 to V10, were run at 12.0 cps, which, while not being exactly at resonance, was sufficiently close for the purpose of the investigations and at the same time gave a dynamic force equal to the dead weight.

TABLE 2.7

TESTS V7 TO V10

Test No.	Moisture Content % dry wt.	Stationary or Towing	No. of Passes	Time of Vibration* sec.	Max. recorded Density % of max.
V7	13.65	Stationary	---	30.0	92.2
V8	15.11	Towing	1	2.7	89.1
V9	11.00	do.	2	5.4	79.8
V10	12.69	do.	3	8.1	87.7

*In the towing tests, the times of vibration correspond to a towing speed of 1.85 fps and a vibrator base 5 ft. long.

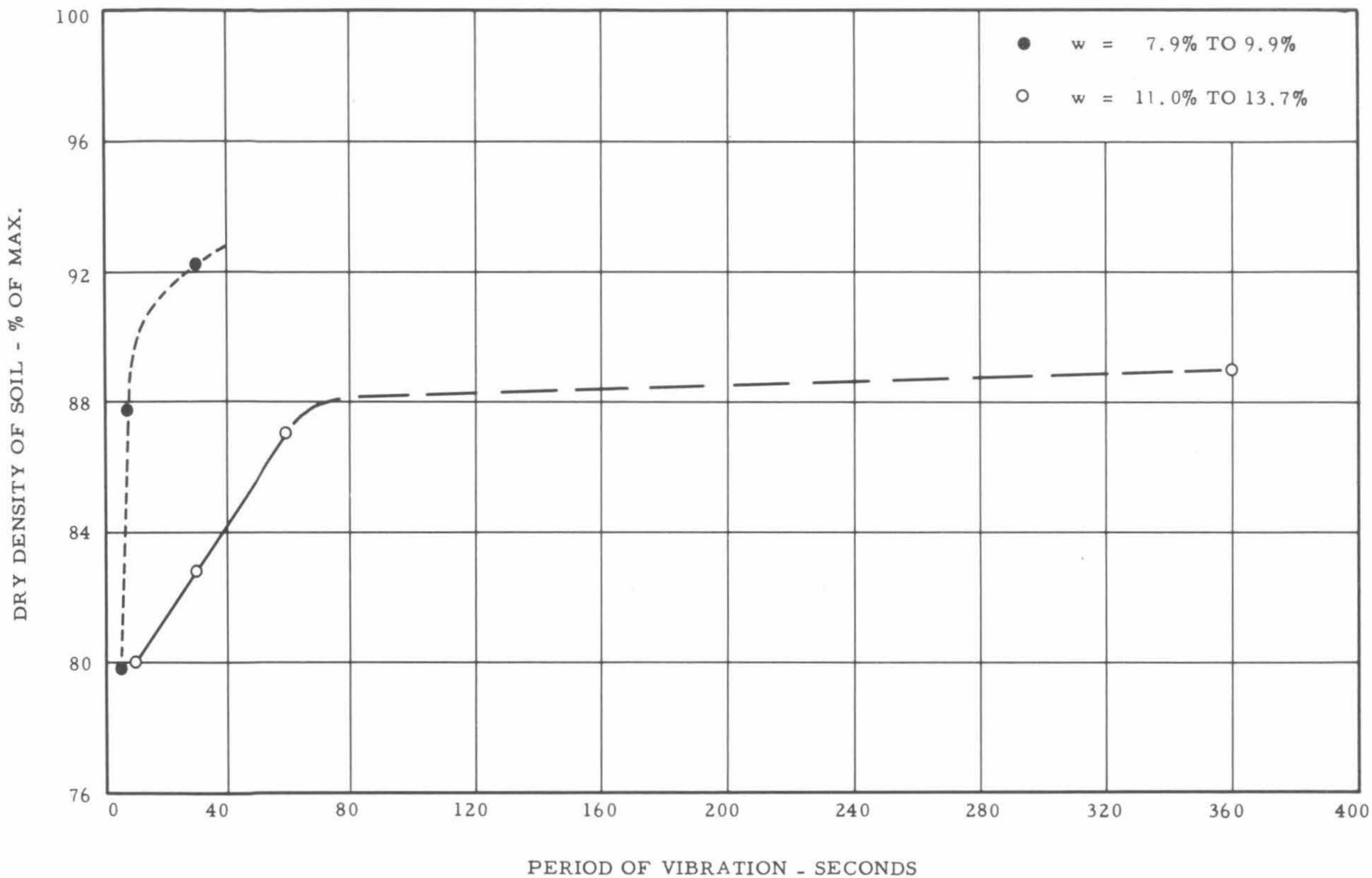


Fig. 2.18 EFFECT OF PERIOD OF VIBRATION ON DENSITY OF SOIL.

TESTS V3 TO V10 WITH LARGE VIBRATOR AT PORT HUENEME.

The results of tests V3 to V10 are shown graphically on Fig. 2.18 and indicate that there was marked effect of both moisture content and the time of vibration on the density obtained. It is apparent from Fig. 2.18 that the number of passes to obtain a specified density can be kept to a minimum by working the soil at the correct moisture content. Laboratory tests indicate that the number of passes may be cut still more by using small amounts of Daxad 23 or sodium sulphate in the mixing water.

Conclusions

1. Satisfactory compaction is possible under field conditions provided the correct combination of parameters is employed, even though the soil is prepared in a crude manner. It appears likely that under the more uniform conditions obtainable with the usual construction methods, greater compaction could be obtained with the same effort.

2. The length of time of vibration is very important. Similar results are obtained either by increasing the time in a stationary test or by making a large number of passes in a towing test. At the correct moisture content, a total time of contact between 10 and 20 seconds appears necessary.

CHAPTER 3

MEASUREMENT OF IN-PLACE PROPERTIES OF SOIL

Methods for the measurement of the in-place properties (moisture content and density) of soils were given much consideration due to the necessity of:

(a) Speed;

(b) Accuracy;

(c) Measurement of density in a very localized area in order to obtain a picture of the change in density with increasing distance from the source of vibration.

The widely accepted method of sand replacement was used for basic density measurements. However, since it did not quite meet requirements (a) and (c) above, this method was, for many experiments, used only to check or augment measurements made by the penetration method. The five methods used or investigated are described below.

3.1 Sand-Replacement Method of Measuring Density

3.1.1 Description

A hole about 6 inches in diameter is excavated with suitable tools to a depth of about 6 inches, the weight of soil removed is determined and a moisture content sample (for oven-drying at 105°C) taken. Sand is run into the hole from a container and the weight of sand in the hole determined from the difference in weight of the cylinder before and after filling the hole, allowing for the sand contained in the cone below the valve (see Plate 3.1). The volume of the soil required to fill the hole is determined from the known weight and bulk density of the sand, which is calibrated by filling a can of known volume under similar conditions.

3.1.2 Investigations and Calibration

No investigation of this method was made due to its wide acceptance and known accuracy (provided the initial calibration is accurate). The limitations of lack of speed and its giving the density of a mass relatively large for all except field tests with the large U. S. Navy vibrator were, however, recognized.

3.2 Penetrometer Method of Measuring Density

3.2.1 Description

The penetrometer consists of a steel shaft 0.75 inches in diameter, 6 feet long, with a tapered point 0.6 inches long and 0.8 inches diameter. It is driven into the soil by the impact of a 20 pound hammer falling 6 inches, guided by a piston sliding inside the shaft. In determining densities, the penetrometer is driven vertically into the soil, through a hole in a 50 pound steel surcharge weight 12-1/4 inches diameter and 1-1/4 inches thick. A sufficient number of blows are struck to cause a penetration of at least 3 cm., and an attempt is made to limit the penetration to 6 cm. The latter figure is often exceeded with only one blow of the hammer in soft soil. Density is determined by comparing the rate of penetration with a calibrating curve determined from tests with soil of known density.

3.2.2 Investigation and Calibration

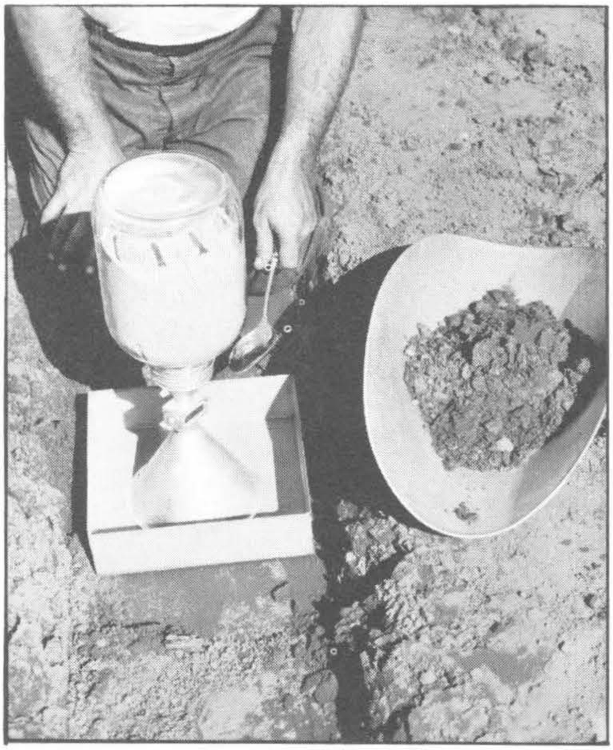
Comprehensive investigation was made of this instrument with the Port Hueneme and Caltech sandy loams, to determine whether or not consistent results could be obtained. At the same time, the resulting data were used to calibrate the penetrometer. The investigations consisted of uniformly compacting the soil to various densities at various chosen moisture contents, and driving the penetrometer to obtain a value of penetration in cm. per blow corresponding to the given soil density. At Port Hueneme, the soil sample was contained in a small pit in the natural ground; at the California Institute of Technology a can 26 inches deep by 13 inches diameter was used.

Due probably to the non-plastic nature of the soil, and to the stone content, the Port Hueneme sandy loam gave inconsistent results, and the penetrometer was used only as a means of indicating the change in density with depth, and the relative change occurring before and after vibration.

The C.I.T. sandy loam was much more suitable for use with this device. A variant analysis for homogeneity showed that for depths greater than 4 inches below ground level, the rate of penetration was independent of moisture content within the operating range. The following relationships were shown to exist:

(1) C.I.T. sandy loam - no chemical added:

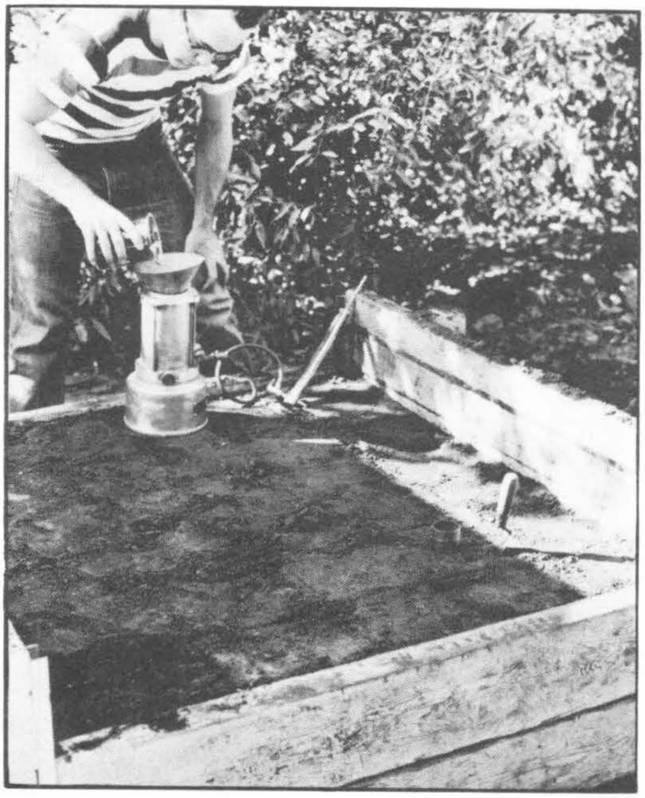
$$\gamma_d = 74.2 = 36.2\ p^{-0.655} \text{ (see Fig. 3.1)}$$



(A) Density Measurement of Sand-Replacement Method

(B) Density Measurement by Penetrometer Method

PLATE 3.1



(A) Moisture Content Measurement by Volumeter Method

PLATE 3.2

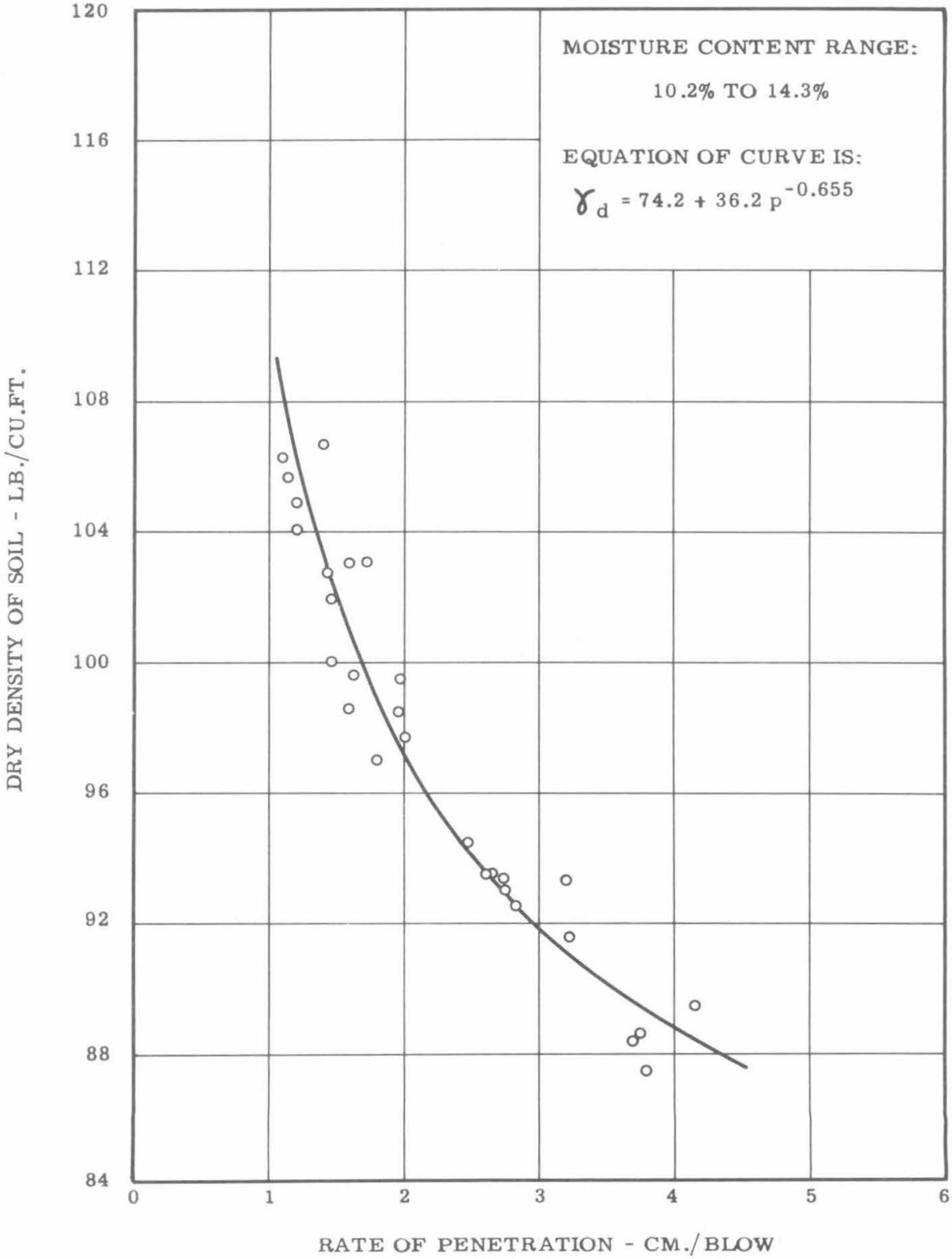


Fig. 3.1 PENETROMETER CALIBRATION CURVE.

C.I.T. SANDY LOAM - NO CHEMICAL ADDED.

(2) C.I.T. sandy loam = 1/6% Daxad 23 added:

$$\gamma_d = 74.9 + 34.6\ p^{-.0589} \text{ (see Fig. 3.2)}$$

where γ_d = dry density of soil in lb/cu. ft.

p = rate of penetration in cm/blow.

The results with C.I.T. sandy loam containing 1/3% Sodium Sulphate were too inconsistent and incapable of being repeated, in spite of very careful control during the calibration tests, to enable the penetrometer to be used. In addition, it was evident there was considerable variation in rate of penetration with small variations in moisture content, as shown on Fig. 3.3.

3.3 Proctor Needle Method of Measuring Density

3.3.1 Description

A needle attached to a spring-loaded plunger, the stem of which is calibrated to read in pounds up to a maximum of 110 pounds, is pushed by hand vertically into the soil to a depth of 3 inches in 6 seconds. A series of points is provided so that a wide range of penetration resistances can be measured.

3.3.2 Investigation and Calibration

A quantity of dry C.I.T. sandy loam was thoroughly mixed at various moisture contents. Samples were formed by compacting the soil uniformly with a 5-1/2 lb. hammer falling specified distances on thin soil layers, in standard C.B.R. moulds until depths of about 6 in. were reached. Five samples, each of different density, were formed for each moisture content. A minimum of six penetrations were made in each sample, the average being taken as the correct value for the given density.

Fifty-five tests were made, of which the results from six showed excessive scatter. The remaining results are shown plotted on Fig. 3.4. Inspection of these data clearly showed a region of instability in the moisture content range 10.3% to 12.9%. Unfortunately this was the range considered most favorable for compaction purposes, and in which the field tests were to be conducted, hence it was decided that with the C.I.T. sandy loam, the Proctor needle apparatus could not be used as the means for absolute measurement of density, although it was used to establish the pattern of variation in density in certain of the field work.

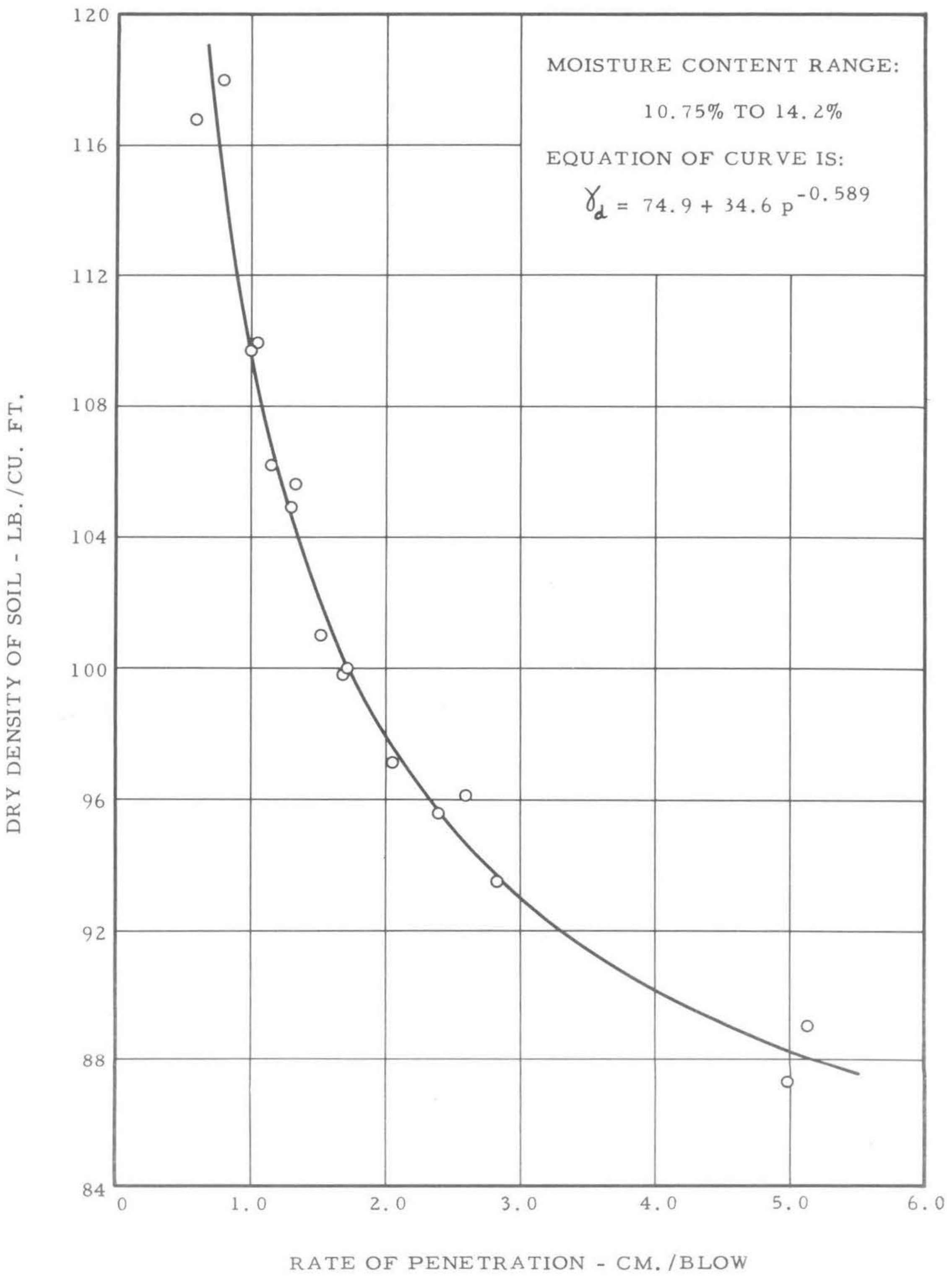


Fig. 3.2 PENETROMETER CALIBRATION CURVE FOR C.I.T. SANDY LOAM WITH 1/6% DAXAD 23 ADDED.

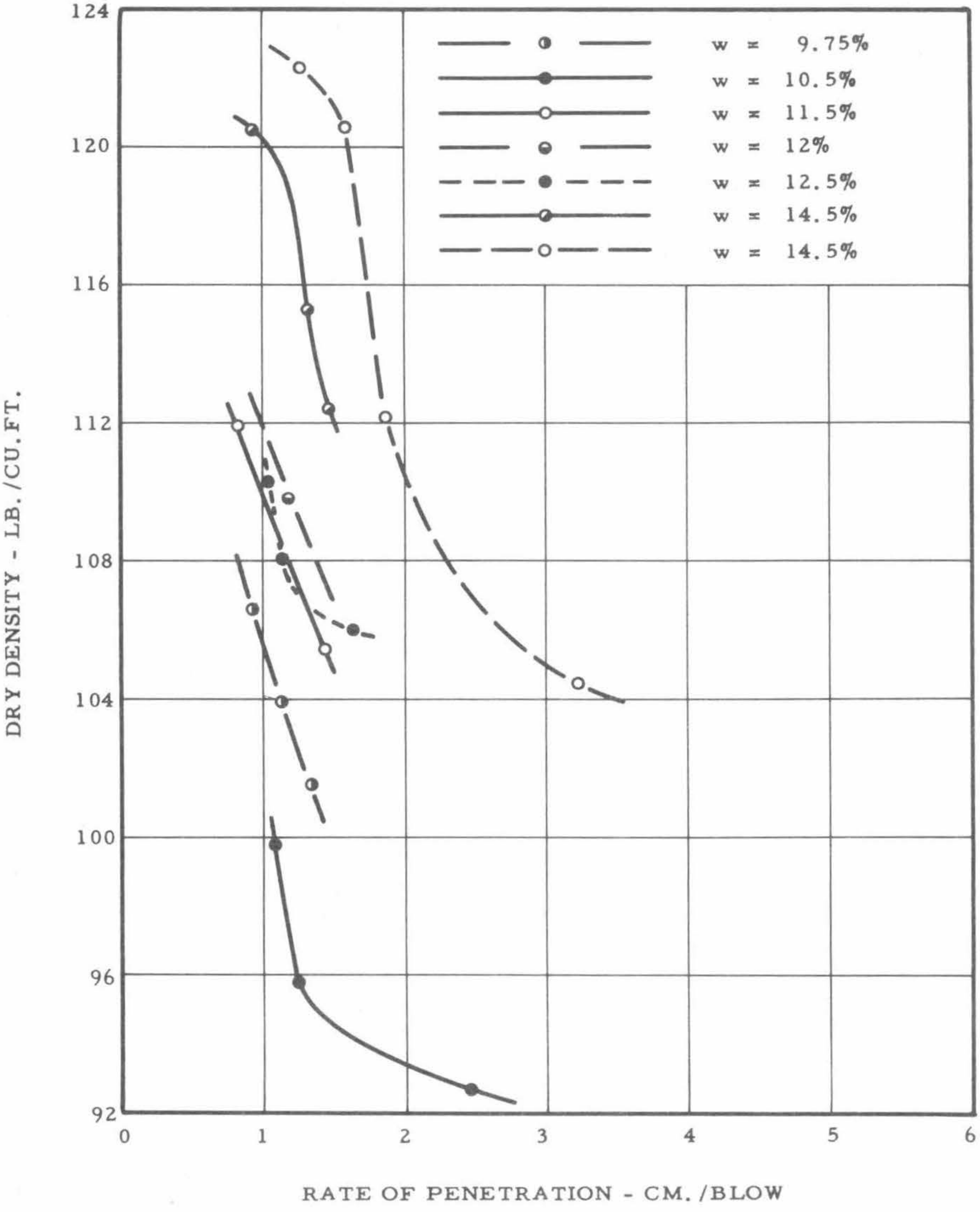


Fig. 3.3 PENETROMETER CALIBRATION CURVES.

C. I. T. SANDY LOAM - 1/3% SODIUM SULPHATE ADDED.

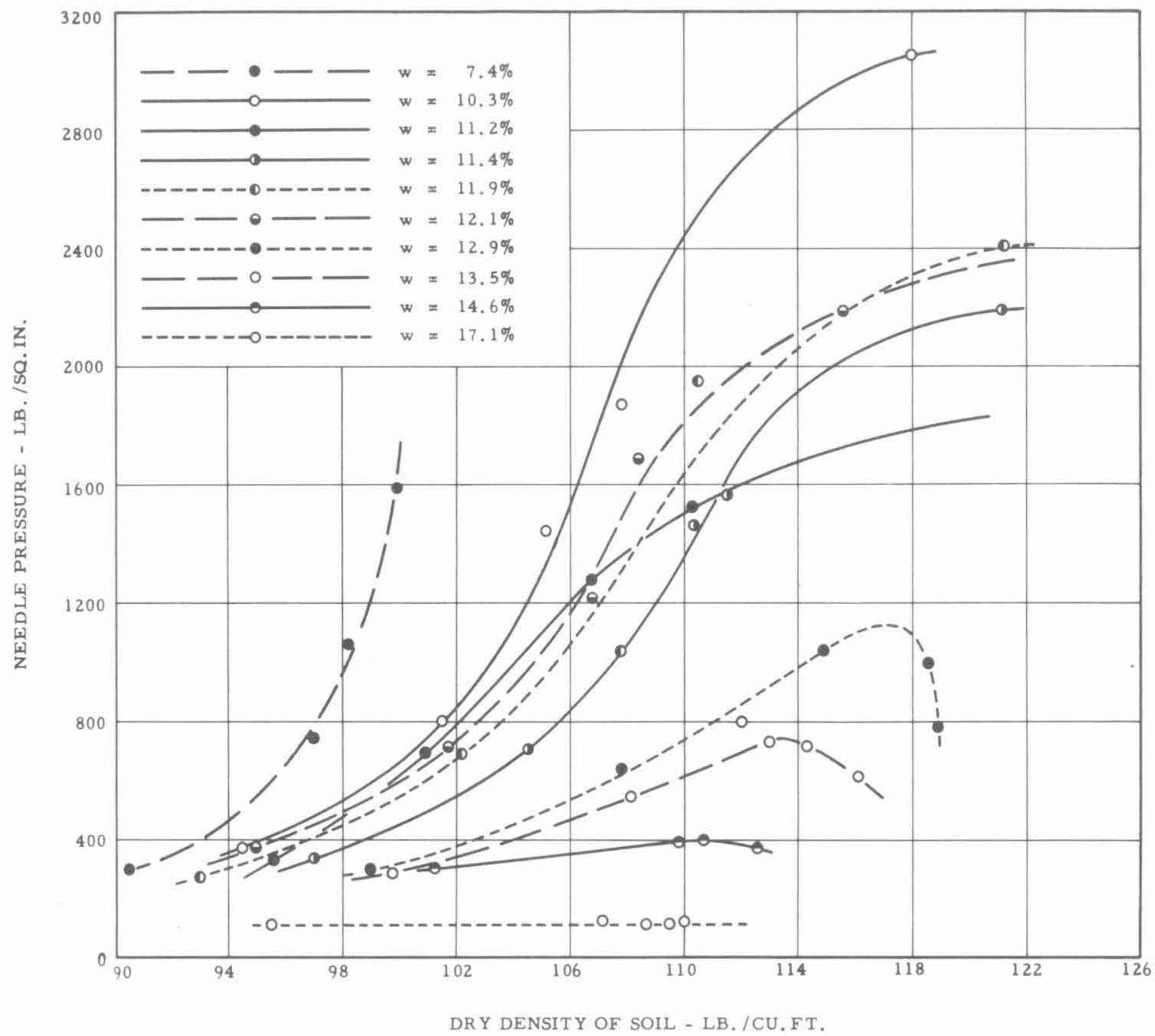


Fig. 3.4 CURVES OF PLASTICITY NEEDLE (PROCTOR TYPE) PRESSURE vs DRY DENSITY OF SOIL AT VARIOUS MOISTURE CONTENTS FOR CALTECH SANDY LOAM, NO CHEMICAL ADDED.

TABLE 3.1

SUMMARY OF EVALUATION TESTS WITH TWO VOLUMETERS

SOIL TYPES: NO. 1 - C.I.T. SANDY LOAM

NO. 2 - PORT HUENEME SANDY LOAM

Test* No.	Volumeter No.	Soil Type	Wt. of Moist Sample gm.	Vol. of Inserts c.c.	Moisture Content Volumeter	Moisture Content Oven-drying
					% dry wt.	% dry wt.
1A,B	A2	No. 1	303.7	301.1	6.84	11.76
2A,B	do.	do.	478.3	301.1	9.11	11.60
3A,B	do.	do.	570.4	301.1	8.95	11.25
4A,B	do.	do.	704.6	200.7	8.57	11.15
5A,B	do.	do.	951.3	100.4	9.22	11.58
6A,B	do.	do.	966.9	-	8.20	11.27
7	do.	do.	834.5	200.7	4.13	4.58
8	do.	do.	875.1	200.7	6.75	7.43
9	do.	do.	877.1	200.7	9.97	10.60
10	do.	do.	1138.9	-	12.20	13.25
11	do.	do.	1291.7	-	17.12	17.63
12	3	No. 2	724.1	-	1.05	1.51
13	do.	do.	741.1	-	4.55	6.69
14	do.	do.	698.5	-	9.35	10.77
15	do.	do.	751.8	-	13.20	13.64
16	do.	do.	704.1	-	14.90	15.93
17	do.	do.	330.6	301.4	9.40	10.90
18	do.	do.	410.1	301.4	8.70	11.05
19A,B	do.	do.	549.1	301.4	9.40	10.80
20A,B	do.	do.	855.8	100.0	10.20	10.70
21A,B	do.	do.	1110.0	-	9.40	10.60

*Suffixes A, B indicate the test was conducted twice without removing sample from the volumeter.

3.4 Density Measurement by Core-Cutting

Density measurement by core cutting samples with small cylinders was attempted. For this purpose three stainless steel seamless cylinders, 2 in. O.D. by #18 gauge by 2 in. long, were obtained, with one end sharpened to a knife-edge. These were driven into the soil by applying a vertical force to a board placed on top of the cylinder, the moist density being obtained by weighing the cylinder after striking and levelling off the soil at each end.

This method did not prove successful mainly because of the small size of the cylinders and the type of soil being investigated. Any small variation in the direction of the force during the driving of the cylinder caused tilting and consequent loss of accuracy; different operators were found to obtain densities varying by as much as 5% in soil of uniform density; and consequently this method was used in a few tests only.

3.5 "Volumeter" Method of Measuring Moisture Content

3.5.1 Description

The "Volumeter" is a device for determining volumes of substances by means of air displacement, the action being based on the well-known physical relationship for gases:

$$\frac{P \times V}{T} = \text{constant}$$

where P = pressure

V = volume

T = absolute temperature.

As used in determining the properties of soils, the temperature is essentially constant, and the relationship reduces to:

$$P \times V = \text{constant}$$

Plate 3.2 shows a known weight of moist soil being placed in a small inner chamber of the volumeter which is then sealed. Air is pumped into a second chamber of constant volume until the pressure reaches a set value. This air is then released to the first chamber containing the soil and the drop in pressure noted. From previously determined calibration curves, the volume of soil and moisture present in the first chamber is known and the moisture content is determined from the relationship:

$$w = \frac{G - (W/V)}{G\left[(W/V) - 1\right]}$$

where w = moisture content

G = specific gravity

W = weight of moist soil

V = absolute volume of moist soil.

3.5.2 Investigation and Calibration

Discrepancies in the moisture contents as determined by oven-drying and by volumeter in Lazan tests No's. L1 to L6 led to a general evaluation of the instrument. The relationship between the residual pressure in the chambers (after allowing air to escape under pressure from the second chamber to the one containing the soil under investigation) and the volume of the soil was established by the use of inserts of known volume. A number of carefully controlled tests were then made, comparing the moisture content given by the volumeter with that given by oven-drying. Both C.I.T. sandy loam and Port Hueneme sandy loam were tested, at moisture contents varying from 1.51% to 17.63% of dry weight, with varying quantities of soil, and with various numbers of inserts of known volume introduced into the chamber with the soil. Tests were made for accuracy and repeatability, and a summary of the data obtained with two volumeters is shown in Table 3.1.

As a result of these tests, it was concluded that the Volumeter was not a suitable device for use in determining moisture contents of the soils used in this project.

CHAPTER 4

LABORATORY INVESTIGATIONS

4.1 Introduction

The failure of the preliminary field tests at Port Hueneme, described in Chapter 1, to cause compaction even with large forces and high unit contact pressures at the soil surface, resulted in the creation of a program of basic studies on a small scale in the California Institute of Technology laboratories.

The object of the studies was to determine what was happening within the soil mass when subjected to vibrational forces. This did not imply, at least initially, any approach to the question of the "best" combination of parameters such as those discussed in Chapter 2 of this report, except where the soil conditions were concerned.

It was realized at the outset that compaction of the soil requires the application of forces large enough to overcome the shearing resistance of the soil and permit displacement within the soil mass. Underlying the whole problem is the basic difference between the shear strengths exhibited by coarse- and fine-grained soils when subjected to vibration. Contact pressures between the particles of coarse-grained soils become very low when the mass is vibrating, permitting easy readjustment to a more compact form, but the particles of fine-grained soils are held together by cohesion, and considerable force is required to break this bond. Consequently the prime considerations during the period of basic studies and laboratory work were to determine:

(a) A means by which cohesion may be reduced or altered so as to permit easier movement between particles of fine-grained soils. It is known that shearing displacements in loose non-saturated fine grained soils break the cohesive bonds and result in greater density. It was therefore assumed that any method which could reduce cohesion would aid compaction.

(b) The manner in which the optimum soil conditions for ease of compaction can be created.

4.2 Theoretical Studies

4.2.1 Surface Tension

The first concept considered was by Haines(1),(2) and Nichols(3)*, which postulates that cohesion is due to the surface tension of the water surrounding the soil particles and

*The numbers in parentheses refer to the list of references at the end of this chapter.

bridging between them. Fig. 4.1 shows two spherical particles of radius 'a', with the water in an annular ring formed around the point of contact. If r_1 and r_2 are the radii of curvature of the film in its two principal directions, then the pressure inside the meniscus produced by surface tension, T, is

$$P = T \frac{1}{r_1} - \frac{1}{r_2} = T \frac{r_2 - r_1}{r_1 r_2}$$

But $r_1 = a \tan 2\theta \tan\theta$

and $r_2 = a \tan 2\theta(1 - \tan\theta)$

Also, the total pull F between the spheres is given by:

$F = P \times \text{area of ring}$

Thus

$$F = 2\pi a\ T. \frac{1 - 2\tan\theta}{1 + \tan\theta} \qquad \text{..........(4.1)}$$

This expression is for the pull existing at one point of contact. For close packing, the condition giving the highest density, there are twelve contacts for each particle, and the total pull across unit cross-sectional area is

$$F_t = 7.25 \frac{T}{a} \frac{1 - 2\tan\theta}{1 + \tan\theta} \qquad \text{..........(4.2)}$$

For open packing the number of contacts per particle is six, and the expression for the cohesion per unit cross-sectional area is

$$F_t = \pi \frac{T}{a} \frac{1 - 2\tan\theta}{1 + \tan\theta} \qquad \text{..........(4.3)}$$

Both Haines and Keen[4] have considered that as the moisture content increases, the cohesion increases after the individual menisci coalesce until it attains a maximum value at saturation. This is based on the following:

(a) The validity of the equation for all values of r_1;

(b) The assumption of spherical particles;

(c) A number of experiments conducted by Haines.

However, the equation is not valid unless the menisci are discontinuous, and also particles of soil are generally not spherical, the majority of fine particles being flat and disc-shaped. Nevertheless, the existence of surface tension as a cause of cohesion is certain, provided the limitations of the theoretical approach given above are recognized.

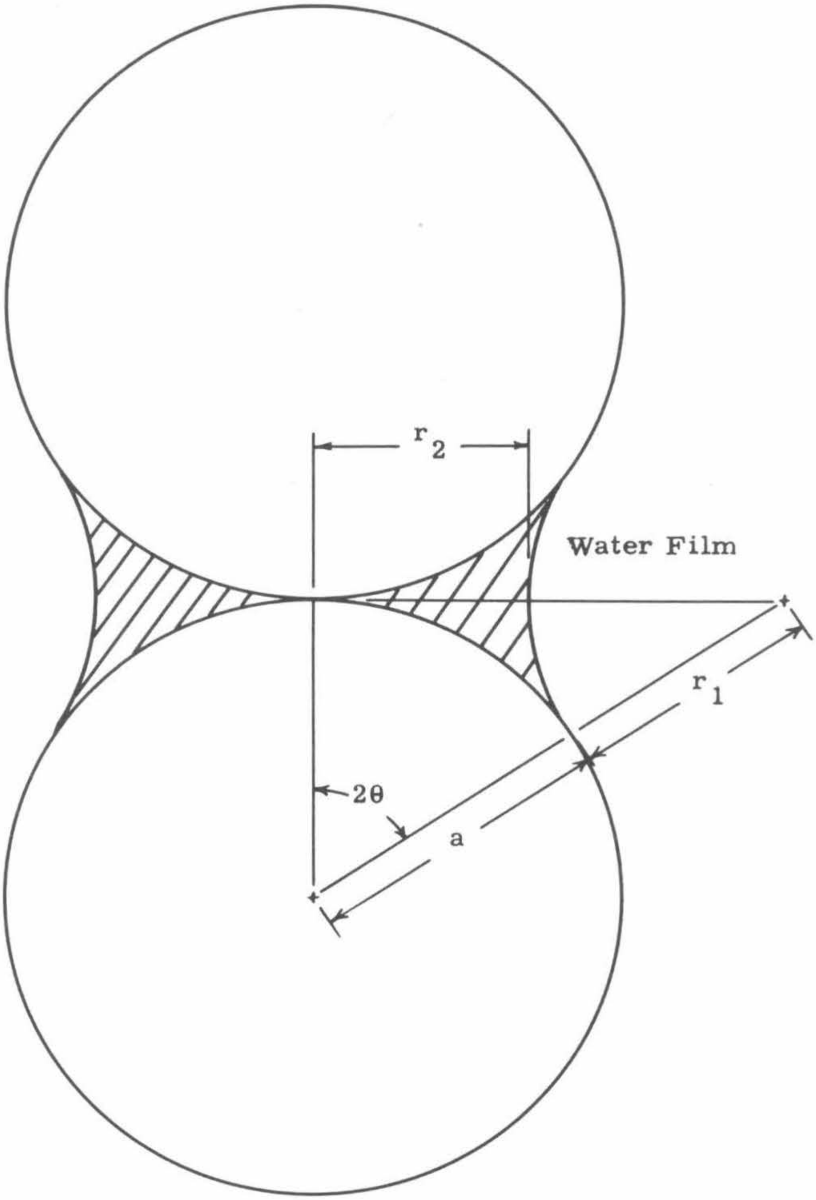


Fig. 4.1 SURFACE TENSION EFFECT BETWEEN SPHERICAL PARTICLES.

Wetting Agents

Wetting agents are substances which are added to surface coatings, water, or oils to increase spreading and penetrating action. They belong to a group of a more general class of materials called "surface active agents," and while there is no direct relationship between wetting powers and surface tension values, as a general rule a good wetting agent is also effective in reducing surface tension.

Surface activity results from the combination of a strongly polar group, the hydrophilic group which is attracted to water, and a non-polar group, the hydrophobic group which is attracted to non-aqueous materials. Many materials exhibiting surface activity are by nature electrolytes, dissociating in an aqueous solution. However, the common inorganic electrolytes, such as sodium chloride, do not possess surface-active properties because no portion of the molecule is hydrophobic in character. If one of the ions is replaced by a hydrophobic ion, then a wetting agent is produced, of which the following are three typical examples:

$$RSO_3Na \rightleftharpoons Na^+ + (RSO_3)^- \qquad \text{..........(I)}$$

$$R(CH_3)_2\,(C_6H_5CH_2)N\,Cl \rightleftharpoons R(CH_3)_2(C_6H_5CH_2)N^+ + Cl^- \qquad \text{..........(II)}$$

$$R(OCH_2CH_2)_xOH \qquad \text{..........(III)}$$

Type (I) is known as Anionic because its activity is due to the oil-soluble (hydrophobic) anion or acid radical.

Type (II) is known as Cationic because its activity is due to the long-chain hydrophobic cation.

Type (III) is known as Non-ionic because it is a non-electrolyte, ionizes in water to a negligible extent, and is effective because of a proper balance between the hydrophilic and hydrophobic groups.

Note that in the above equations R represents the active end of the hydrophobic group, one example being 2-ethyl-hexane.

4.2.2 Chemical Linkage by Electrostatic Phenomena

A second concept is that of chemical linkage by electrostatic phenomena, involving electrically charged ions with the soil moisture acting as a bonding agent(5). Hardy(6) has stated that while film tension probably plays an important part in conferring plasticity on wet clays and soils, and, as Wilsdon(7) has indicated, is probably augmented by the presence of colloidal matter in a state of active imbibation, it is evident that surface films alone cannot, in every case, account completely for the cohesiveness of these materials. Hardy hypothesized that the intrinsic property of clay particles to adhere to one another is due to the presence of emulsoid colloidal matter that coats, more or less thickly, the individual particles of the clay, or binds together and envelopes a number of particles to form separate clay aggregates. This hypothesis, however, does not arrive at the basis of the problem, for it does not explain how emulsoid colloidal matter coats the soil particles, nor is it in accord with the results of later investigations.

Ducleaux[8] has considered Kossel's explanation of adsorptive forces on solid surfaces as due to surface ions or scattered valences, giving electric fields around solid particles. The surface of the solid particle in water may release ions to the water because of their affinity to the latter; or may collect ions from the water; or may attract and orient water dipoles. If the particle is surrounded by ions, these together with adsorbed dipoles may be so strongly attached to the solid as to form part of a micelle, which is the solid and the surrounding swarm ions. Under the assumption that kinetic dispersing forces and the electric attraction forces must balance each other, Ducleaux gives the concentration of ions as

$$C = A \frac{Pp^2}{KT} \cdot \frac{1}{r} \qquad \text{..........(4.4)}$$

where

A = a constant (which, during derivation of the formula has absorbed

e - the base of natural logarithm;

E - the elementary electric charge;

N - Avogadro's Number;

R - the gas constant)

P = number of swarm ions

p = charge of swarm ions

C = number of ions per unit volume

K = di-electric constant of the liquid

T = absolute temperature

r = distance from surface of a spherical particle.

Recognizing that most fine particles have a plate-like structure, Winterkorn[9] calculated the distribution of ions on a uniformly charged plane surface using the same energy assumptions as Ducleaux, arriving at:

$$C = Br^k \qquad \text{..........(4.5)}$$

where B and k are coefficients.

'k' is very small and r^k is almost equal to unity, hence the ion concentration is not a function of distance from the surface.

The important feature of equations (4.4) and (4.5) is that they have been derived on the basis of a logical mathematical approach, hence the existence of a concentration of swarm ions is probable, and is in fact substantiated by other phenomena such as osmosis. Also, the magnitude of the force acting between two particles must be a function of the concentration, which, on the basis of equation (4.4) may be reduced by

(a) decreasing the charge of swarm ions.

(b) increasing the size of the particles.

4.2.3 Discussion

The significance of these concepts in relation to the problems under consideration (Section 4.1) is derived from the fact that since cohesion is functionally connected with the thickness and structure of the internal moisture films, a change of structure of these films by changing the electric surface field is bound to result in a change of the cohesion of the soil. Surface tension is included by this statement for it is itself a phenomenon associated with electric activity. Thus Adam[10] has said .... "The term 'surface tension' has often been strained to imply that liquids have in their surfaces some mechanism like a stretched membrane pulling parallel to the surface..... The view that there is some skin in the surface, pulling parallel to it, leads to great difficulties when the structure of the supposed skin is considered in terms of molecules..... The forces round the molecules, which give rise to the phenomena of capillarity, are identical with those which cause chemical reaction and solution, as well as all the phenomena of adhesion and cohesion. Most frequently, in liquids, the attractive forces between molecules are of the type known as the van der Waal's forces; pure electrostatic attractions or repulsions are, however, often superposed on these, particularly when electrolytically dissociated groups are present in the molecules. In the case of solid surfaces, forces of the covalent type are often the predominant factors in determining the nature and amount of cohesion."

Due to mutual interference, the structure of moisture films in a soil mass is different from that of a single particle suspended in water, and quantitative predictions of behavior based on considerations of the single particle meet as yet insurmountable difficulties. Nevertheless, qualitative predictions can be, and have been, made[11], being summarized as follows:

(a) Particles of like and uniform electric surface fields will repel each other;

(b) Particles with uniform but unlike fields will attract each other;

(c) Particles with non-uniform surface fields <u>may</u> attract each other.

To what extent these attractions and repulsions will take place depends upon the amounts and valencies of adsorbed ions which, together with the water dipoles, are the carriers of the electric field.

4.2.4 <u>Conclusions</u>

The cohesion between the soil particles may be reduced by:

(i) Reducing the surface tension of the liquid present in the soil mass;

(ii) Reducing the concentration of swarm ions round the individual particles;

(iii) Increasing the effective size of the soil particles, which will also reduce the concentration of swarm ions;

(iv) Creating like and uniform electric surface fields on the particles.

<u>Laboratory Studies</u>

The programme of laboratory studies was divided into two major phases, namely:

Phase 1, in which samples of soil containing various chemicals were vibrated in a cylinder placed on a small vibration-table; and

Phase 2, in which direct shear tests were conducted on samples containing selected chemicals.

The object of Phase 1 was primarily to determine the relative ease by which soil may be compacted under dynamic conditions, denoted in the following text as compactibility, when various chemicals were present in the soil. Secondary considerations were the effects of:

(a) the frequency and displacement of the vibration-table;

(b) the amount of chemical added to the soil;

(c) the manner by which the chemical was added to the soil;

(d) the moisture content of the soil;

(e) the length of time during which the sample was vibrated;

(f) the time elapsing between the addition of the chemical to the soil and the commencement of vibration.

The object of Phase 2 was primarily to test the underlying assumption that compactibility was an inverse function of shear strength, in particular of cohesional shear strength. Consideration was given to the effects of:

(a) different cations and anions;

(b) the amount of chemical added to the soil;

(c) the moisture content of the soil;

(d) the time elapsing between the addition of the chemical to the soil and the shearing of the sample;

(e) the molecular weight of the added chemical.

4.3 Phase 1 - Studies with the Vibration-Table

4.3.1 Apparatus and Procedure

The vibration-table consisted of a small platform driven by an electric motor operating through pulleys and a V-belt. Vertical simple harmonic oscillations were imparted to the platform by a positive-motion cam, the eccentricity of which could be controlled to give any desired value of peak-to-peak displacement within the range 0 to 0.25 inches, at any frequency within the range 0 to 30 cycles per second. Special containers to hold the test samples consisted of "Lucite" cylinders 3 in. internal diameter and 6 in. high, closed at one end by an aluminum base. Steel weights, slightly smaller than the Lucite cylinders, were provided to superimpose upon the soil sample.

The general procedure was to mix a determined quantity of water and chemical with 600 gm. of oven-dried soil, which was then placed in a cylinder firmly screwed to the platform of the vibration-table. After a measured period of two minutes from the completion of mixing, the whole assembly was vibrated at the chosen frequency and displacement for consecutive periods of 10 seconds, 20 seconds, and 30 seconds. Measurements of the height of soil sample before vibration and at the end of each period of vibration enabled the corresponding soil density to be determined. Moisture content was obtained by oven-drying a sample taken from the cylinder after the test.

4.3.2 Initial Experiments

For comparison purposes it was necessary that conditions of frequency, displacement, superimposed surface weight, moisture content, and size of soil sample be held constant, or controlled in a pre-determined manner.

The frequency-displacement relationship was chosen so that the superimposed weights showed signs of rotation about a vertical axis through the center of the cylinder. This was taken to indicate that 'tamping', or separation of the weights from the soil, was just beginning. A few preliminary experiments were made to determine if there was any particular combination of frequency and displacement (analogous with resonant frequency) at which maximum density would be obtained. As a result, see Figure 4.2, all the initial comparative tests were made at a frequency of 12.8 c.p.s. with a displacement of 0.1625 inches peak-to-peak. Later experiments conducted to investigate the frequency-density relationship were made with the condition that the acceleration applied to the soil sample was exactly equal to gravity, thus ensuring that, theoretically at least, no tamping was taking place.

The superimposed surface weights were required to be as large as possible, but, using steel, they could not exceed about 6 pounds without becoming rather unwieldly and top-heavy for the apparatus. The actual value of the special weights was 5.76 pounds, or 2615 gm., giving a nominal contact pressure (weight divided by contact area) of 0.81 pound per square inch at the soil surface.

A basic value of 10.5% moisture content was chosen, this being the optimum value (see section 1.4, chapter 1) for the Port Hueneme sandy loam used in the investigations. Some variations in this were made from time to time, and as a result it became clear that an optimum value, much higher than 10.5%, existed for this vibration apparatus. This was expected from the known fact that the moisture-density relationship is effected by the manner in which the soil is compacted, but it was not anticipated that the optimum in this instance would be almost doubled to a value apparently in the range 16% to 22% moisture. The basic value of the moisture content was thus raised to 16.5%, this being the highest value at which stability of the soil was generally maintained, stability indicating the condition during vibration of zero soil movement up into the annular space between the superimposed weights and the " Lucite " cylinder.

A soil sample of 600 gm. (dry weight) was found to be suitable for the apparatus, the cylinders being just filled by the loose moist sample of this dry weight.

The quantity of chemical to use in each test was quite unknown at the outset. A number of quantities were tried according to some arbitrarily chosen relationship, such as 1 gm. per pound of soil (see test no. 102), and it was found that about 2 gm. of chemical was effective in the soil sample of 600 gm., although generally an additional test was conducted with at least one other quantity of each chemical.

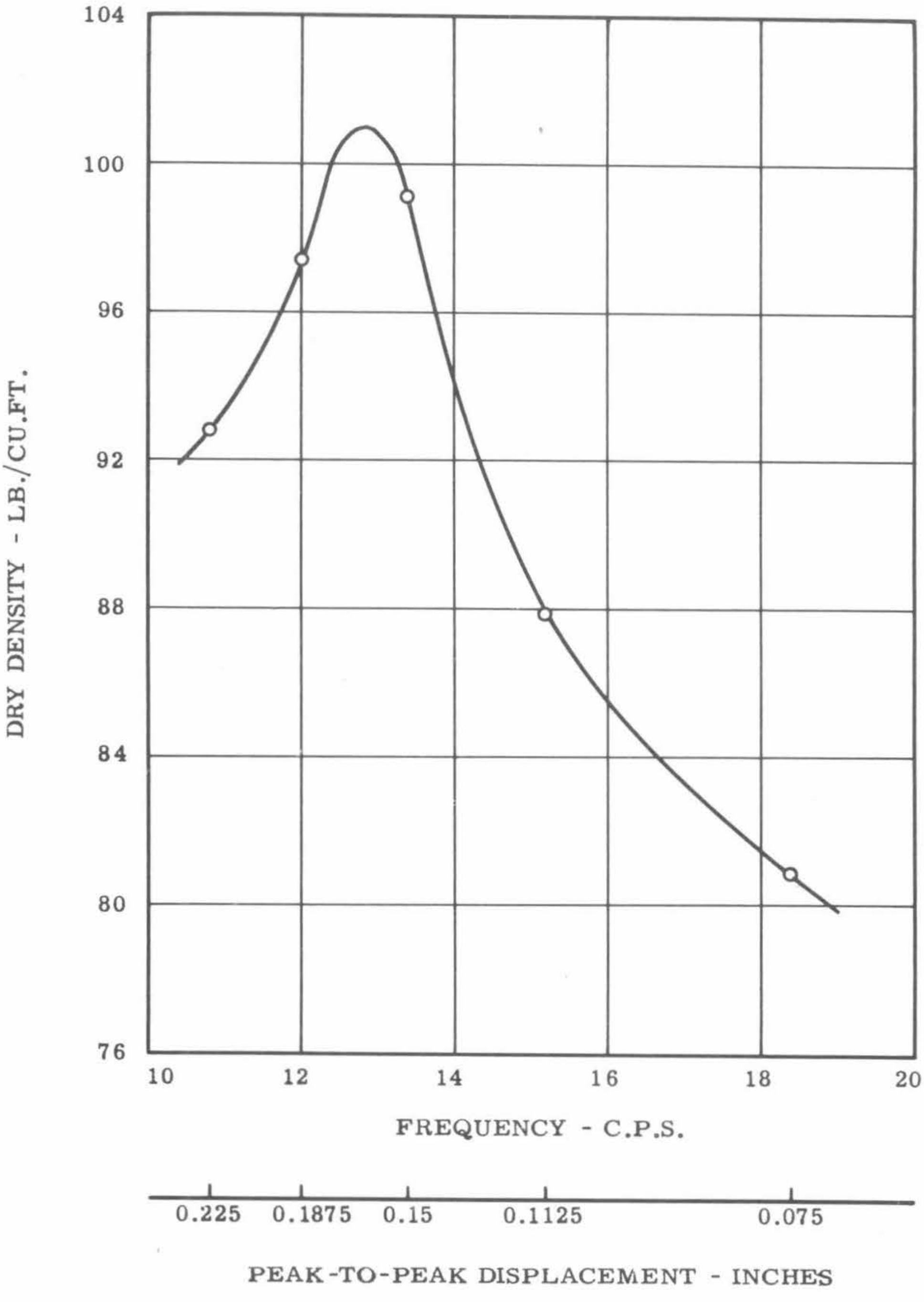


Fig. 4.2 DENSITY - FREQUENCY CURVE SHOWING OPTIMUM OPERATING CONDITIONS FOR VIBRATION-TABLE TESTS.

In many of the earlier experiments the pH value of the moist soil 30 seconds after mixing was measured by means of Beckmann apparatus. The time interval of 30 seconds was chosen as sufficient for a stable condition to be attained. The reason for the measurement was to determine if a correlation existed between pH and density of the soil, the original thought being that the density after vibration would increase with increasing pH. This thought originated from work by Meyer[12], who propounded that by far most of the reactions of clay particles depend on acidic and basic residues and take place at particle surfaces. Defining isoelectric pH as that pH at which the positive charges on the particles equal the negative charges, and noting that the lower the ratio of the acidic residue to the basic residue of the particle the higher will be the isoelectric pH, Meyer considered that a low ratio of acidic to amphoteric constituents corresponds to a low micellar ion density, resulting in maximum particle growth arising from the combination of acid and basic residues of adjacent particles. Since clays tend to weather to the composition which has the isoelectric pH equal to the pH of the weathering solution, it appeared that treatment of the soil with a chemical solution of high pH would give the desired effect of a fine-grained soil behaving like a coarse-grained soil, which in effect it would become by virtue of particle growth arising from a low micellar ion density.

4.3.3 Results and Conclusions of Initial Experiments

A list of all the chemicals used in the tests is shown in Appendix E, and details of the results of the experiments in Table 4.1.

A number of qualitative conclusions were obtained from the initial experiments, as follows:

(i) The addition of chemicals to the soil can result in increased compaction, although this is not necessarily true for all chemicals nor for all soils.

(ii) All of the most successful chemicals were sodium salts, and those which were wetting agents were all anionic, i.e., depended on the acid radical for their wetting activity. The most successful chemicals, in order of their success, were Sodium Sulphate, Darvan #1 (known also as Daxad 11), Darvan #2 (Daxad 23), Aerosol OT, Nopco 1067-A, Victawet 35B, Sodium Chloride, and Aerosol IB. It was the presence of a non-wetting agent at the head of the list that prompted investigation of chemical linkage by electrostatic phenomena, see section 4.2.2.

(iii) Mixing an electrolyte and a wetting-agent, each of which may be a reasonably successful chemical, does not increase the density after vibration, and often causes a decrease.

TABLE 4.1

DETAILS OF INITIAL EXPERIMENTS

Experiment No.	Reference No. of Added Chemical (see Appendix E)	Quantity added to 600 gm. Soil Sample gm.	Moisture Content of Sample % dry wt.	Dry Density after 60 sec. Vibration lb./cu.ft.	pH Value of Sample 30 sec. after Mixing
101	Water only		9.84	86.2	
102	38	1.3(a)*	10.7	85.7	
103	8	3.42(b)*	10.4	83.5	
104	83	1.3(a)*	10.9	90.0	
105	83	0.6 *	10.7	89.4	
106	81	1.3(a)*	10.9	88.0	
107	81	1.3	10.5	88.9	
108	60	5.0	10.4	88.0	
109	5	0.06	10.4	86.5	
110	5	0.06	10.0	86.4	
111	5	0.6	10.0	89.0	
112	62	6.0	10.2	83.8	
113	61	1.3(a)*	9.6	82.7	
114	86	6.0	9.4	77.5	
115	14	6.0	10.5	84.7	
116	86	30.0	6.0	80.5	
117	14	30.0	9.9	85.2	
118	83	2.0 *	10.4	94.6	
119	83	4.0 *	10.6	96.6	
120	74	4.0 *	10.4	92.2	
121	17	4.0 *	10.3	90.8	
122	83	8.0 *	10.4	94.6	
123	59	4.0 *	10.4	86.3	
124	74	8.0 *	10.4	82.5	

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
125	17	2.0 *	10.0	83.2	
126	58	4.0 *	9.8	86.9	
127	Machine out of adjustment.				
128	5 64	0.6 3.5	10.4	92.2	
129	5 83	0.6 4.0	9.1	88.9	
130	48	1.0	10.9	88.9	
131	48	4.0	10.4	91.6	
132	57	1.0	10.8	89.0	
133	57	4.0	10.7	88.9	
134	23	1.0	11.1	86.8	
135	23	4.0	10.6	88.4	
136	73	1.0 *	10.7	90.4	
137	73	4.0 *	10.8	92.0	
138	7	1.0 *	10.5	90.5	
139	7	4.0 *	10.7	91.5	
140	85	1.0 *	10.4	90.8	
141	85	4.0 *	10.5	91.0	
142	83	6.0 *	10.8	88.2	
143	64	3.5 *	10.6	87.6	
144	93	1.0	10.3	88.5	7.35
145	93	4.0	10.9	90.7	7.2
146	33	1.0	10.2	87.9	6.9
147	33	4.0	10.9	88.3	6.8
148	87	1.0	9.9	87.0	7.0
149	87	4.0	10.2	89.1	7.2
150	31	1.0	9.7	88.1	6.8

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
151	31	4.0	10.0	90.4	6.9
152	80	1.0	10.1	88.1	6.9
153	80	4.0	10.0	89.9	7.0
154	9	4.0	10.5	86.8	7.65
155	9	1.0	10.3	87.0	7.75
156	82	1.0 *	10.7	87.0	6.7
157	82	4.0 *	8.8	84.4	6.7
158	30	1.0 *	10.5	90.0	6.7
159	30	4.0 *	10.4	91.8	6.8
160	94	1.0	10.4	91.1	6.76
161	94	4.0	10.5	92.1	7.1
162	95	1.0 *	10.9	86.1	7.65
163	95	4.0 *	10.5	86.2	8.4
164	71	1.0 *	10.3	85.6	7.12
165	71	4.0 *	10.4	85.1	7.18
166	7	4.0	10.5	92.7	7.42
167	73	4.0	10.3	94.3	7.03
168	30	4.0	10.3	93.9	6.98
169	74	4.0	10.2	92.4	7.26
170	83	4.0	10.5	92.8	6.86
171	73 64	4.0 3.5	10.4	94.5	6.49
172	45	1.0	10.4	92.5	6.90
173	45	4.0	10.2	93.4	6.77
174	46	1.0	10.4	93.8	6.96
175	47	4.0	10.7	94.2	6.96
176	91	1.0	10.6	91.2	7.02

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
177	91	4.0	10.4	91.4	7.10
178	92	1.0	10.6	90.8	6.82
179	92	4.0	10.8	91.1	6.45
180	89	1.0	9.6	88.7	6.93
181	89	4.0	Chemical would not dissolve.		
182	15	1.0	10.8	88.2	7.16
183	15	4.0	10.7	85.5	7.08
184	69	1.0	10.2	85.9	7.15
185	69	4.0	9.6	83.7	7.28
186	67	1.0	9.4	84.1	5.97
187	67	4.0	9.6	84.5	4.93
188	46 15	4.0 1.0	10.3	94.4	6.90
189	73 15	4.0 1.0	10.4	94.4	7.11
190	30 15	4.0 1.0	10.2	93.8	6.92
191	7 15	4.0 1.0	10.4	93.0	7.40
192	94 15	4.0 1.0	10.3	92.8	7.20
193	48 15	4.0 1.0	10.5	92.9	8.67
194	5 15	4.0 1.0	10.3	94.3	7.11
195	5	4.0	10.1	94.0	7.06
196	41	1.0	10.5	86.7	7.88
197	41	4.0	10.4	84.6	9.6
198	42	1.0	10.6	86.6	7.99

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
199	42	4.0	10.5	84.9	9.78
200	43	1.0	10.0	85.8	7.86
201	43	4.0	10.2	84.6	9.80
202	29	1.0	10.4	91.1	6.92
203	29	4.0	10.2	92.0	6.89
204	5 64	4.0 3.5	10.2	96.1	6.44
205	5 67	4.0 6.7	10.6	94.0	4.27
206	83	4.0	10.3	90.8	6.82
207	83	4.0	10.3	92.3	6.74
208	46	4.0	8.0	92.1	6.71
209	46	4.0	9.0	92.5	6.82
210	46	4.0	11.1	94.6	6.86
211	46	4.0	12.2	97.6	6.86
212	46	4.0	13.3	99.6	7.08
213	46	4.0	14.4	98.7	7.06
214	46	2.85	10.6	93.0	6.89
215	46	3.08	10.7	92.8	6.93
216	46	3.34	10.6	92.9	6.90
217	46	3.63	10.4	91.6	6.93
218	46	2.0	13.7	97.0	7.07
219	46	6.0	13.4	98.7	7.08
220	Water only		6.7	84.0	6.70
221	Water only		8.7	84.3	6.80
222	Water only		12.5	90.6	7.02
223	Water only		14.6	93.9	7.19

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
224	Water only		16.5	98.9	7.24
225	Water only		18.4	105.3	7.17
226	Water only		20.3	102.2	7.15
227	46	4.0	18.5	99.9	
228(c)	Water only		6.8	99.5	
229(c)	Water only		8.7	101.8	
230(c)	Water only		10.9	104.9	
231(c)	Water only		12.9	108.9	
232(c)	Water only		14.8	110.1	
233(c)	Water only		16.8	108.2	
234	46	4.0	16.9	102.0	
235	49	1.0	10.2	90.4	7.01
236	49	4.0	10.3	92.1	7.06
237	50	1.0	9.7	89.0	7.12
238	50	4.0	9.8	92.8	7.22
239	52	1.0	10.0	91.4	7.01
240	52	4.0	10.1	93.0	7.09
241	51	1.0	10.0	87.0	6.91
242	51	4.0	9.8	87.2	6.90
243	88	1.0	9.9	87.8	7.10
244	88	4.0	10.2	88.2	6.93
245	46	1.0	15.8	101.1	7.18
246	84	4.0	6.4	88.5	
247	84	4.0	8.5	87.9	
248	84	4.0	10.6	92.2	

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
249	84	4.0	12.7	96.8	
250	84	4.0	14.2	101.1	
251	84	4.0	16.7	103.2	
252	84	4.0	18.9	103.3	
253	84	4.0	20.2	100.2	
254	Water only		18.9	104.3	
255	Water only		19.9	102.5	
256	Water only		20.7	101.9	
257	84	4.0	15.9	100.3	
258	84	4.0	17.5	104.1	
259	84	4.0	20.3	100.8	
260	53	12.0	17.0	104.0	
261	5 64	2.0 3.5	6.8	92.3	
262	5 64	2.0 3.5	12.7	99.5	
263	5 64	2.0 3.5	17.3	103.2	
264	5 64	2.0 3.5	20.2	99.3	
265	53	24.0	17.3	104.0	
266	53	48.0	16.3	97.5	
267	5	2.0	7.5	95.0	
268	5	2.0	13.5	101.5	
269	5	2.0	16.2	105.6	
270	5	2.0	18.4	98.4	
271	88 47	2.0 2.0	10.8	92.9	

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
272	16 22	2.0 1.0	16.7	99.2	
273	16	2.0	16.8	104.0	
274	16 22 72	2.0 1.0 1.0	16.6	102.3	
275	16 22 70	2.0 1.0 1.0	16.1	104.1	
276	16 22 72 70	2.0 1.0 1.0 1.0	16.8	101.5	
277	16 72	2.0 1.0	16.5	104.9	
278	22	2.0	16.5	89.9	
279	72	2.0	16.5	108.2	
280	16 72 22	1.0 2.0 0.5	16.3	101.0	
281	16 72 22	1.0 2.0 1.0	16.3	98.5	
282	16 72 22 70	1.0 2.0 0.5 0.5	16.4	99.8	
283	16 72 22 70	1.0 2.0 1.0 0.5	16.5	97.0	
284	16 22 70	2.0 1.0 1.0	17.0	100.0	
285	16 22 70 72	2.0 1.0 1.0 1.0	16.5	102.4	

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
286	72	2.0	16.6	107.1	
287		2.0	16.8	106.3	
288		0.84	16.7	105.0	
289	16 72	1.0 2.0	17.2	103.9	
290	16 72	2.0 2.0	17.0	102.0	
291	5 72	1.0 2.0	16.8	102.2	
292	5 72	2.0 2.0	16.2	103.7	
293	65	2.0	16.5	96.4	
294	66	2.0	16.9	96.1	
295	63	2.0	16.4	92.8	
296	54	2.0	16.6	93.0	
297	55	2.0	16.6	92.9	
298	56	2.0	16.5	93.6	
299	10	2.0	16.7	96.5	
300	11	2.0	16.5	98.7	
301	72	2.0	16.4	107.8	
302	66	2.0	16.5	97.2	
303	12	2.0	16.8	94.7	
304	13	2.0	16.5	91.5	
305	24	2.0	17.0	99.1	
306	18	2.0	16.6	96.9	
307	37	2.0	16.6	96.2	
308	34	2.0	16.9	92.1	

(continued next page)

TABLE 4.1 (CONTINUED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
309	35	2.0	16.6	90.5	
310	36	2.0	16.4	95.6	
311	40	2.0	16.8	96.6	
312	39	2.0	16.9	97.2	
313	19	2.0	16.5	100.3	
314	20	2.0	16.7	92.3	
315	21	2.0	16.8	98.7	
316	79		16.8	98.5	
317	25	2.0	16.8	107.1	
318	26	2.0	16.4	106.0	
319	7	2.0	16.8	102.2	
320	53	12.0	16.7	94.0	
321	53	24.0	16.3	92.5	
322	30	2.0	16.6	100.1	
323	28	2.0	16.8	101.0	
324	31	2.0	16.8	100.0	
325	45	2.0	16.8	102.8	
326	48	2.0	16.6	102.2	
327	49	2.0	17.4	103.7	
328	50	2.0	17.0	101.0	
329	52	2.0	17.0	100.0	
330	73	2.0	17.1	98.5	
331	74	2.0	16.4	98.0	
332	84	2.0	16.5	103.0	
333	91	2.0	16.5	97.8	

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TABLE 4.1 (CONCLUDED)

DETAILS OF INITIAL EXPERIMENTS

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
334	92	2.0	16.5	105.2	
335	93	2.0	16.6	99.4	
336	1	2.0	17.2	93.1	
337	2	2.0	16.8	104.9	
338	3	2.0	17.1	89.5	
339	4	2.0	17.2	98.0	
340	49	2.0	17.6	105.2	
341	6	2.0	19.2	98.6	
342	90	2.0	19.3	99.4	
343	32	2.0	19.5	100.2	
344	77	2.0	16.2	103.8	
344A	77	2.0	15.8	102.8	
344B	77	2.0	16.4	105.0	
345	76	2.0	15.6	91.7	
346	6	2.0	16.0	94.5	
347	78	2.0	16.3	93.0	
348	68	2.0	16.4	93.6	
349	44	2.0	15.7	91.8	
350	75	2.0	15.9	92.1	
351	27	2.0	16.0	93.6	

Notes:
* indicates chemical mixed dry with the dry soil before adding moisture.
(a) 1 gm. chemical per 1 lb. dry soil.
(b) 1/10 N solution.
(c) 23.9 c.p.s.

(vi) Chemical added as an aqueous solution is more effective than chemical added to and mixed with the dry soil before moisture is added.

(vii) Considerable variation in the density of the soil after vibration occurs if the moisture content of the soil is varied, there appearing to be an optimum value lying between 16% and 22%, depending upon the chemical present in the soil.

(viii) Most of the soil compaction occurs within the first 10 seconds of vibration, but no decrease in density is obtained if the vibration is continued for long periods. This is felt to be unique for the conditions of these tests, as field experience with surface vibrators has generally shown otherwise.

(ix) There is no correlation between the pH value of the soil and the density obtained after vibration, see Fig. 4.3. However, it is possible that some correlation may be found in a series of carefully controlled experiments aimed only at investigation of the relation between pH and density, in which pH be measured more accurately than is possible by the use of electrodes placed in soil at 10.5% moisture content.

4.3.4 Effect of the Frequency and Displacement of the Vibration-Table

A comprehensive series of displacement frequency tests were conducted with the Port Hueneme sandy loam containing water only, sodium sulphate, or Aerosol OT. For a chosen frequency, the displacement was computed theoretically so that the maximum acceleration applied to the soil was equal to gravity, a condition maintained throughout these and following tests. From an examination of the data it was considered that the greatest densities would be obtained at a frequency of 14.6 cycles per second with a displacement of 0.092 inches peak-to-peak. Unfortunately after the series was completed, a fault in the displacement-setting mechanism was detected at the commencement of the tests described in section 4.3.6 of this chapter. Inasmuch as the point at which this fault developed was not known, it rendered unreliable the whole of the data. Fortunately, although the maximum applied acceleration was less than gravity, the resulting frequency-displacement combination proved to be a suitable one, without being necessarily the best one. Consequently this combination was retained throughout all the subsequent experimental work. A few tests were made with sodium sulphate in the soil at two different moisture contents, with correct settings of the vibration-table, indicating that maximum density would be attained at a frequency of about 13 c.p.s. with a corresponding displacement of 0.116 inches peak-to-peak, see Fig. 4.4.

It may be concluded that as far as this type of vibration test is concerned there is an optimum combination of frequency and displacement at which maximum densities are obtained.

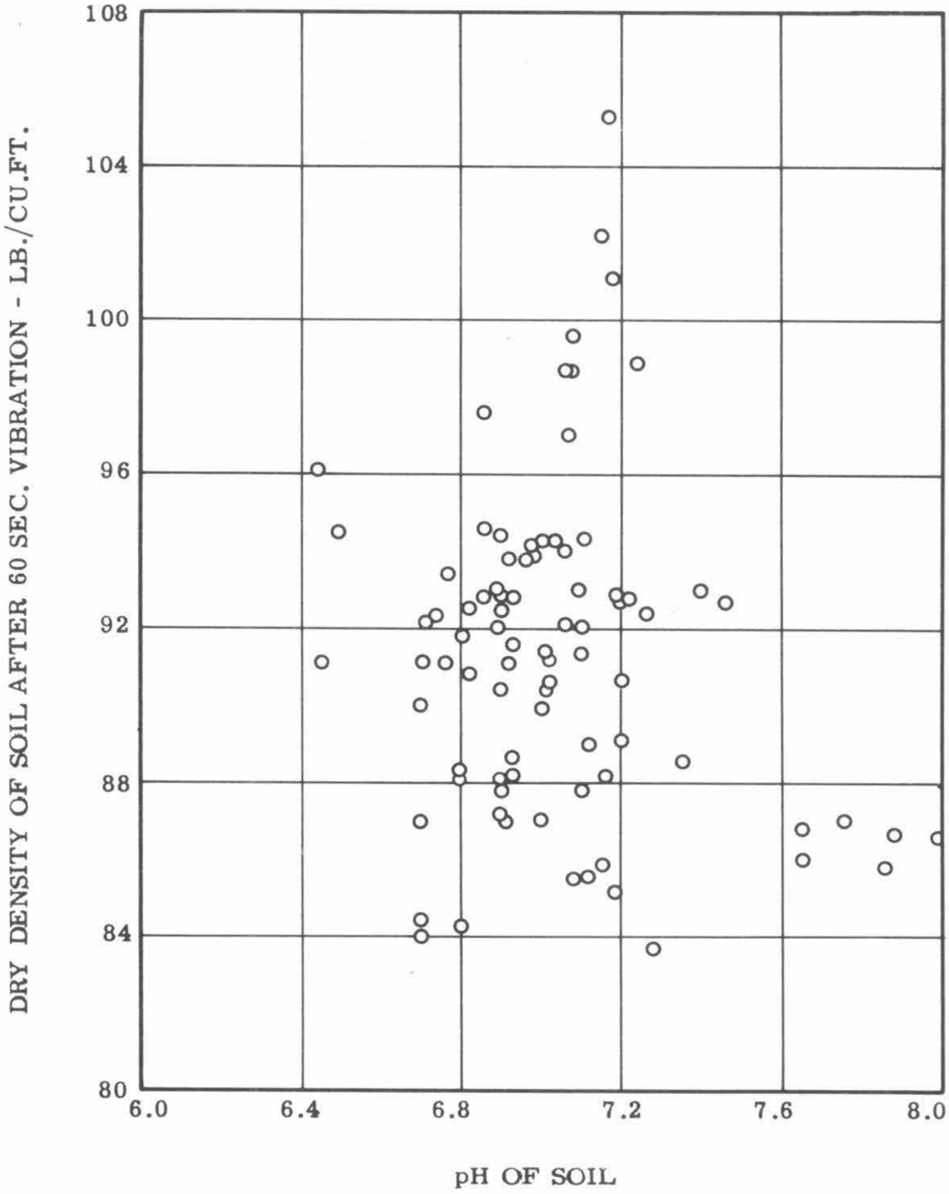

Fig. 4.3 RELATION BETWEEN SOIL pH AND DENSITY AFTER VIBRATION.

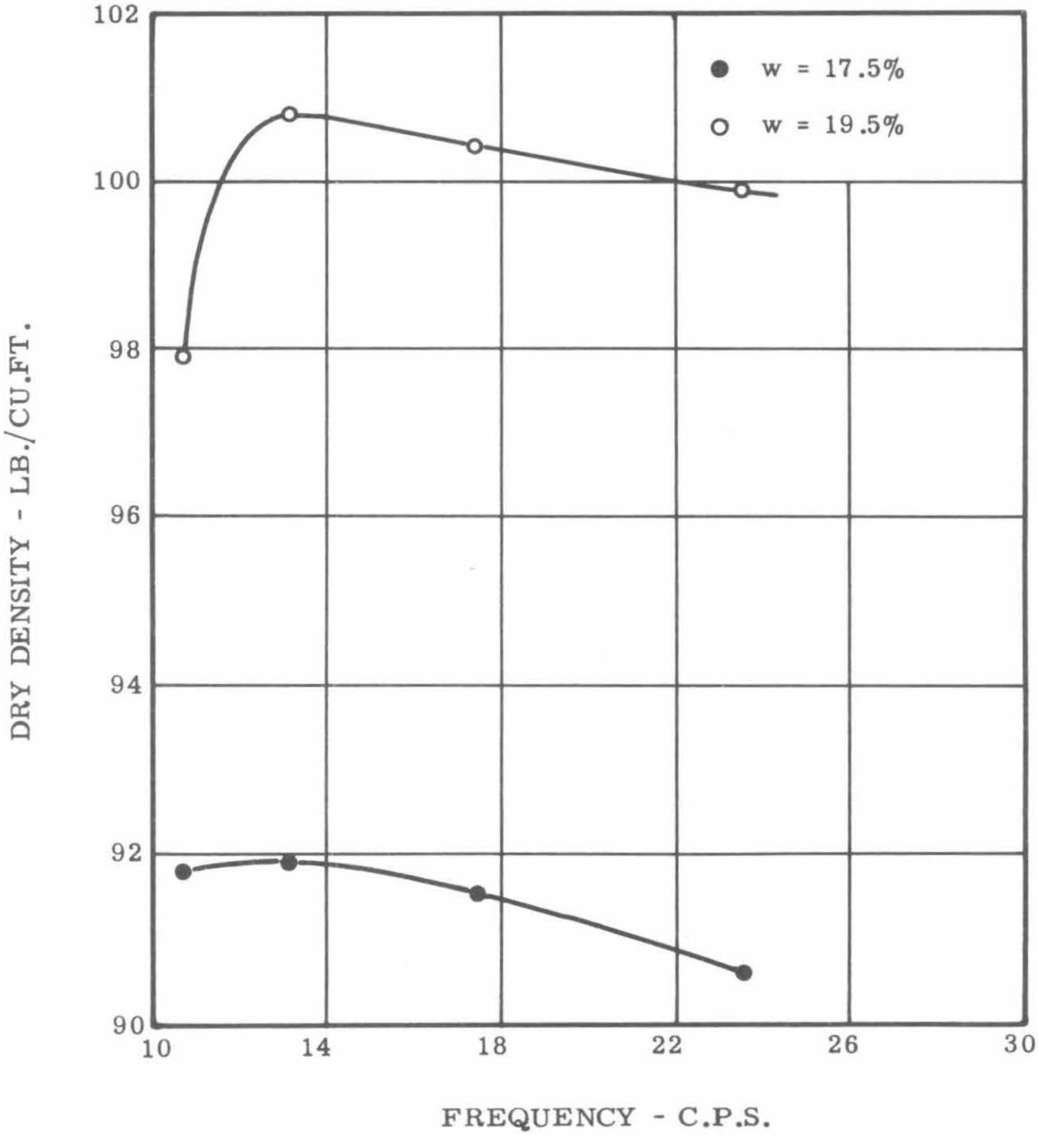

Fig. 4.4 DENSITY - FREQUENCY RELATIONSHIP IN VIBRATION-TABLE TESTS.

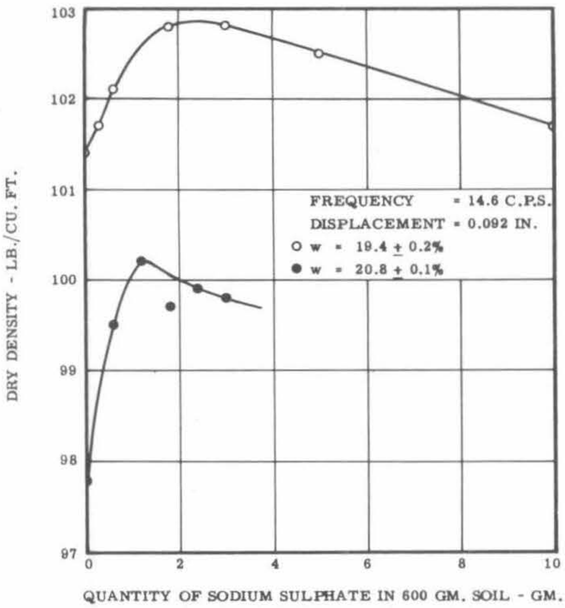

Fig. 4.5 EFFECT OF QUANTITY OF SODIUM SULPHATE ADDED TO SOIL ON DENSITY OF SOIL AFTER VIBRATION-TABLE TEST.

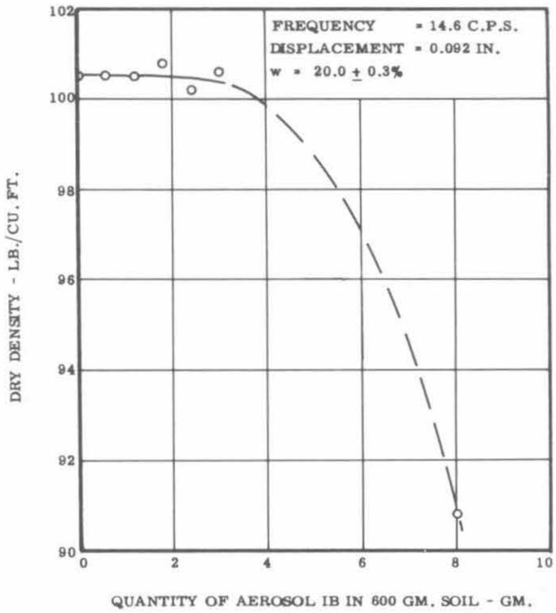

Fig. 4.6 EFFECT OF QUANTITY OF AEROSOL IB ADDED TO THE SOIL ON DENSITY OF SOIL AFTER VIBRATION-TABLE TEST.

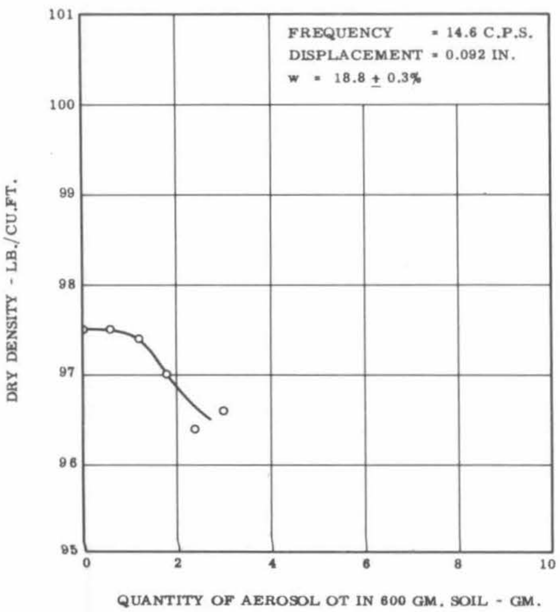

Fig. 4.7 EFFECT OF QUANTITY OF AEROSOL OT ADDED TO THE SOIL ON DENSITY OF SOIL AFTER VIBRATION-TABLE TEST.

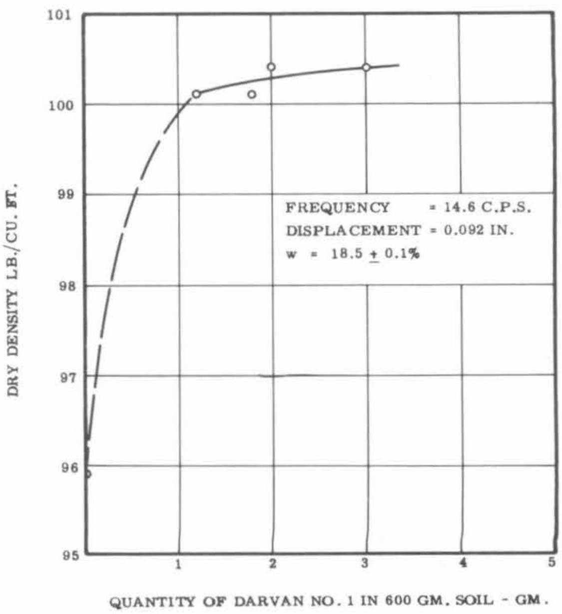

Fig. 4.8 EFFECT OF QUANTITY OF DARVAN NO. 1 ADDED TO THE SOIL ON DENSITY OF SOIL AFTER VIBRATION-TABLE TEST.

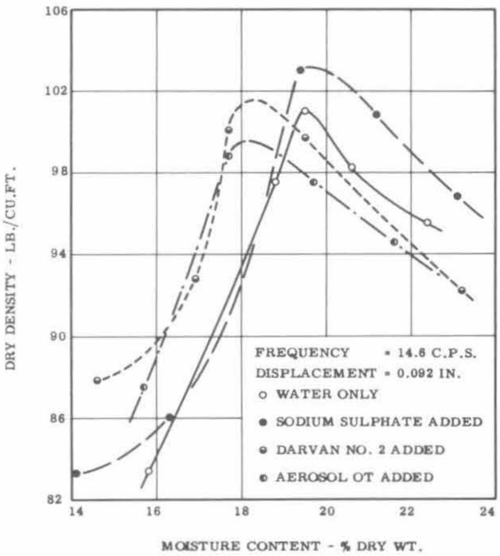

Fig. 4.9 DRY DENSITY vs MOISTURE CONTENT RELATIONSHIPS IN VIBRATION-TABLE TESTS WITH PORT HUENEME SANDY LOAM CONTAINING VARIOUS CHEMICALS.

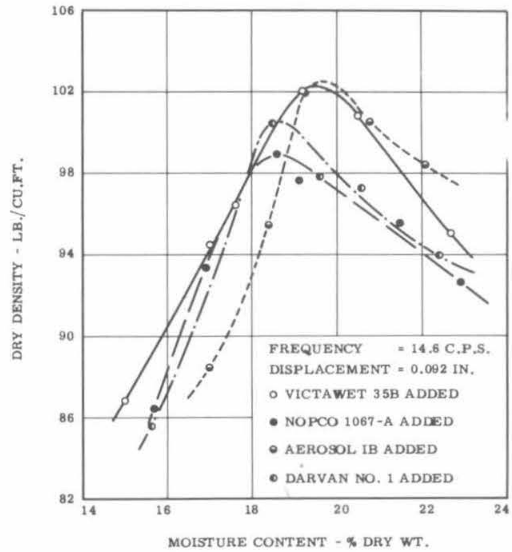

Fig. 4.10 DRY DENSITY vs MOISTURE CONTENT RELATIONSHIPS IN VIBRATION-TABLE TESTS WITH PORT HUENEME SANDY LOAM CONTAINING VARIOUS CHEMICALS.

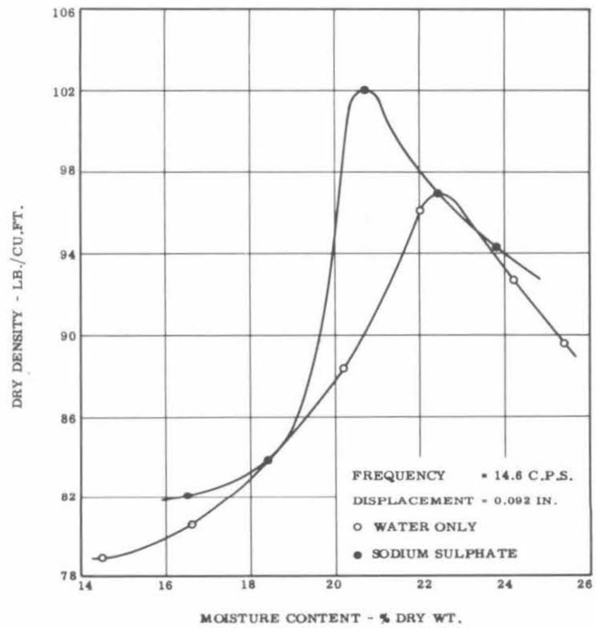

Fig. 4.11 DRY DENSITY vs MOISTURE CONTENT RELATIONSHIPS IN VIBRATION-TABLE TESTS WITH CALTECH SANDY LOAM CONTAINING SODIUM SULPHATE.

This combination is found to vary according to the chemical added to the soil, and it was observed that the curve relating density with frequency is more peaked at high moisture contents than at low moisture contents.

4.3.5 Effect of the Quantity of Chemical Added to the Soil

Tests to determine the effect of the quantity of chemical added to the soil were conducted with Sodium Sulphate, Aerosol OT, Aerosol IB, and Darvan #1. The moisture content in each series of tests was fixed at a value at, or close to, that determined in the investigations for the effect of moisture content, described in section 4.3.6 of this chapter. Table 4.2 and Figures Nos. 4.5 to 4.8 show details of the results of the experiments.

It may be concluded that the amount of chemical added to the soil has an effect on the resulting densities after vibration, the maximum effect being obtained when about 2 gm. of chemical is added to 600 gm. dry soil, i.e. 1/3% by weight. It should be noted that the two Aerosols which had, in previous tests at lower moisture contents, resulted in an increase in density compared with samples containing water only, exhibited a decrease in density as the quantity of chemical added was increased. The reason for this is not quite clear, but may possibly be due to a considerable thickening of the anionic layer surrounding the soil particles, with the sodium ions in the atmosphere surrounding the anionic layer causing an increase in the repulsive forces.

4.3.6 Effect of Moisture Content

Each of the seven most successful chemicals of the series of initial experiments was added, in the proportion 2 gm. to 600 gm. dry soil, to Port Hueneme sandy loam at various moisture contents within the range 14% to 24%. Vibration tests were conducted in the usual manner at 14.6 c.p.s. with 0.092 inches peak-to-peak displacement. The resulting densities are shown in Table 4.3 and are plotted as a function of moisture content on Figures 4.9 and 4.10. Similar tests with sodium sulphate added to Caltech sandy loam were conducted, the results being shown in Table 4.3 and Figure 4.11.

Proctor-type curves were obtained in every case, although as anticipated from consideration of the amount of energy involved, the optimum moisture content was higher, and the maximum density lower, than is obtained by the normal laboratory procedure. The chemicals have some effect on the relationship between density and moisture content, although the 'spread', or peakedness of the curves is hardly changed, and the maximum density varies only within the range 98.8 lb./cu.ft. to 103.2 lb./cu.ft. and the optimum moisture content within the range 18% to 19.5%.

TABLE 4.2

VIBRATION-TABLE EXPERIMENTS TO DETERMINE

EFFECT OF CHEMICAL CONCENTRATION

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak, providing a maximum acceleration equal to gravity. *

Experiment No.	Chemical Added	Quantity added to 600 gm. Soil Sample gm.	Moisture Content of Sample % dry wt.	Dry Density after 60 sec. Vibration lb./cu.ft.
C-1	None - Water only		19.4	101.4
C-2	Sod.Sulphate	0.3	19.3	101.7
C-3	do.	0.6	19.5	102.1
C-4	do.	1.8	19.2	102.8
C-5	do.	3.0	19.4	102.8
C-6	do.	5.0	19.3	102.5
C-7	do.	10.0	19.3	101.7
C-8	Aerosol IB	0.6	20.0	100.5
C-9	do.	1.2	19.8	100.5
C-10	do.	1.8	20.0	100.8
C-11	do.	2.4	20.0	100.2
C-12	do.	3.0	20.3	100.6
C-13	do.	8.0	20.0	90.8
C-14	Aerosol OT	0.6	18.8	97.5
C-15	do.	1.2	19.1	97.4
C-16	do.	1.8	18.9	97.0
C-17	do.	2.4	19.1	96.4
C-18	do.	3.0	18.8	96.6

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TABLE 4.2 (CONCLUDED)

VIBRATION-TABLE EXPERIMENTS TO DETERMINE

EFFECT OF CHEMICAL CONCENTRATION

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak, providing a maximum acceleration equal to gravity. *

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
C-19	Darvan #1	0.6	18.6	100.8
C-20	do.	1.2	18.5	100.1
C-21	do.	1.8	18.5	100.1
C-22	do.	3.0	18.6	100.4
C-23	Sod.Sulphate	0.6	20.9	99.5
C-24	do.	1.2	20.8	100.2
C-25	do.	1.8	20.7	99.9
C-26	do.	2.4	20.8	99.9
C-27	do.	3.0	20.8	99.8

Note: *See section 4.3.4.

TABLE 4.3

VIBRATION-TABLE EXPERIMENTS TO DETERMINE

EFFECT OF MOISTURE CONTENT

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak, providing a maximum acceleration equal to gravity.*

Experiment No.	Chemical Added	Quantity added to 600 gm. Soil Sample gm.	Moisture Content of Sample % dry wt.	Dry Density after 60 sec. Vibration lb./cu.ft.
M-1	None - Water only		15.8	83.4
M-2	do.		18.8	97.5
M-3	do.		20.6	98.2
M-4	do.		22.4	95.5
M-5	do.		19.5	101.0
M-6	Darvan #1	2.0	15.6	85.5
M-7	do.	do.	18.5	100.4
M-8	do.	do.	20.6	97.2
M-9	do.	do.	19.6	97.8
M-10	do.	do.	22.4	93.9
M-11	Darvan #2	2.0	15.7	87.5
M-12	do.	do.	17.7	98.8
M-13	do.	do.	19.7	97.5
M-14	do.	do.	21.6	94.5
M-15	Sod.Sulphate	2.0	14.1	83.3
M-16	do.	do.	16.3	86.0
M-17	do.	do.	19.4	103.0
M-18	do.	do.	21.2	100.8
M-19	do.	do.	23.2	96.8

(continued next page)

TABLE 4.3 (CONTINUED)

VIBRATION-TABLE EXPERIMENTS TO DETERMINE

EFFECT OF MOISTURE CONTENT

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak, providing a maximum acceleration equal to gravity. *

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
M-20	Aerosol IB	2.0	19.5	101.9
M-21	do.	do.	17.0	88.4
M-22	do.	do.	18.4	95.4
M-23	do.	do.	20.8	100.5
M-24	do.	do.	22.1	98.4
M-26	Victawet 35B	2.0	15.0	86.8
M-27	do.	do.	17.0	94.4
M-28A	do.	do.	17.6	96.4
M-28	do.	do.	19.2	102.0
M-29	do.	do.	20.5	100.8
M-30	do.	do.	22.7	95.0
M-31	Aerosol OT	2.0	14.6	87.9
M-32	do.	do.	16.9	92.8
M-33	do.	do.	19.5	99.7
M-34	do.	do.	17.7	100.1
M-35	do.	do.	23.2	92.2
M-36	Nopco 1067-A	2.0	19.1	97.5
M-37	do.	do.	16.9	93.3
M-38	do.	do.	15.7	86.4
M-39	do.	do.	21.5	95.5
M-40	do.	do.	22.9	92.6
M-41	do.	do.	18.6	98.8
M-42	do.	do.	19.1	97.6

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TABLE 4.3 (CONCLUDED)

VIBRATION-TABLE EXPERIMENTS TO DETERMINE

EFFECT OF MOISTURE CONTENT

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak providing a maximum acceleration equal to gravity.*

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
	The following tests were conducted with Caltech Sandy Loam instead of Port Hueneme Sandy Loam.			
M-43	None - Water only		14.5	78.9
M-44	do.		16.6	80.6
M-45	do.		20.2	88.4
M-46	do.		22.0	96.1
M-47	do.		24.2	92.7
M-48	do.		25.4	89.6
M-49	Sod.Sulphate	2.0	16.5	82.1
M-50	do.	do.	18.4	83.9
M-51	do.	do.	20.7	101.0
M-52	do.	do.	22.4	97.0
M-53	do.	do.	23.8	94.3

Note: *See section 4.3.4.

4.3.7 Effect of Time between Mixing and Vibration

Previous tests having shown the superiority of sodium sulphate to the other chemicals under examination, this compound was selected for investigation of its time effect on the soil. For this purpose, a number of samples were mixed with 1 gm. of sodium sulphate to 600 gm. of dry soil, and vibration tests conducted at chosen intervals after the mixing was completed. Three such series of tests were conducted, two with Port Hueneme sandy loam at moisture contents of 19.7% $\pm$ 0.5% and 21.2% $\pm$ 0.2%, and one with Caltech sandy loam at a moisture content of 12.8% $\pm$ 0.3%. The results are given in Table 4.4 and on Figures 4.12 and 4.13.

It was observed that with the Port Hueneme soil there was, at both moisture contents, a definite time interval of between 100 and 120 minutes after mixing at which a maximum density was obtained, and that the curve at the higher moisture content was the more peaked. No peak was observed with the Caltech soil, the total range of densities in a 6-hour period being from 81.1 lb./cu.ft. to 82.1 lb./cu.ft., but at the end of that period the tendency was towards an increase in density.

These results prove to be possibly the most significant, from the point of view of application to field compaction procedure, of all the vibration-table investigations. If sodium sulphate is used in compaction work, then a peak of activity, as far as compaction is concerned, is reached some two hours after application of the chemical to the soil, this activity subsequently diminishing. If a similar characteristic is exhibited with other important physical properties, such as shear strength and permeability, this would indicate a very useful tool for compaction work by a vibratory process.

4.4 Phase 2 - Studies with the Direct Shear Machine

4.4.1 Object and Scope

The object of this phase was to determine the effect of chemicals on the shearing strength of the soil, in order to ascertain whether or not a correlation exists between the laboratory shearing strength and the compactibility as determined by other phases of the research. For this purpose a program of direct shear tests was formulated, which, while not attempting to give a complete answer to the many questions arising from the use of chemicals in the soil, would give an indication of their effect on the properties of the soil. In view of the limited time and effort available for this program the number of chemicals involved was fixed at the minimum considered necessary to furnish useful data.

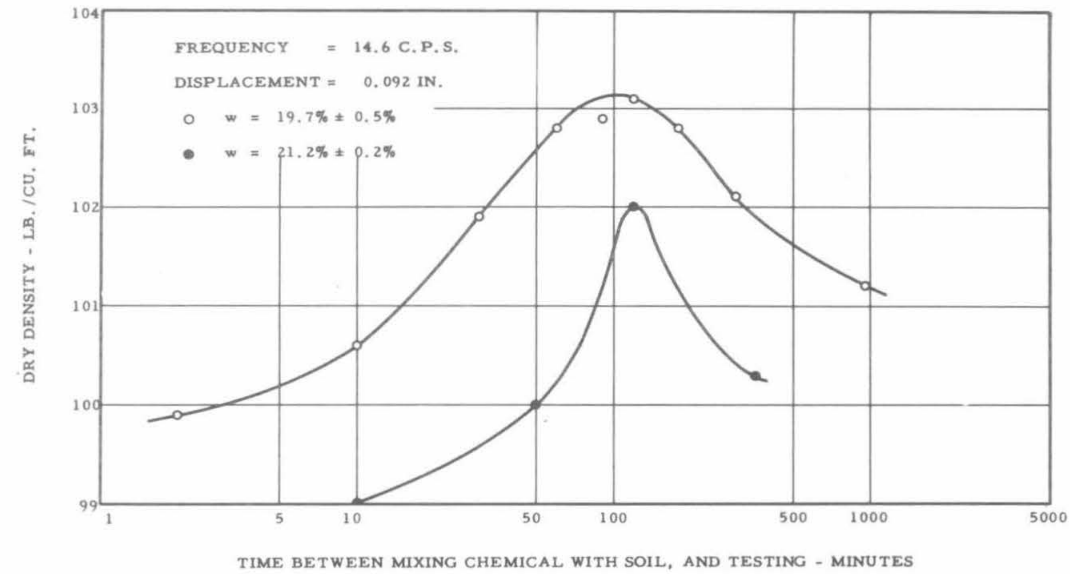

Fig. 4.12 CURVES RELATING DRY DENSITY OF SOIL AND THE TIME BETWEEN MIXING SODIUM SULPHATE WITH THE SOIL AND TESTING. PORT HUENEME SANDY LOAM.

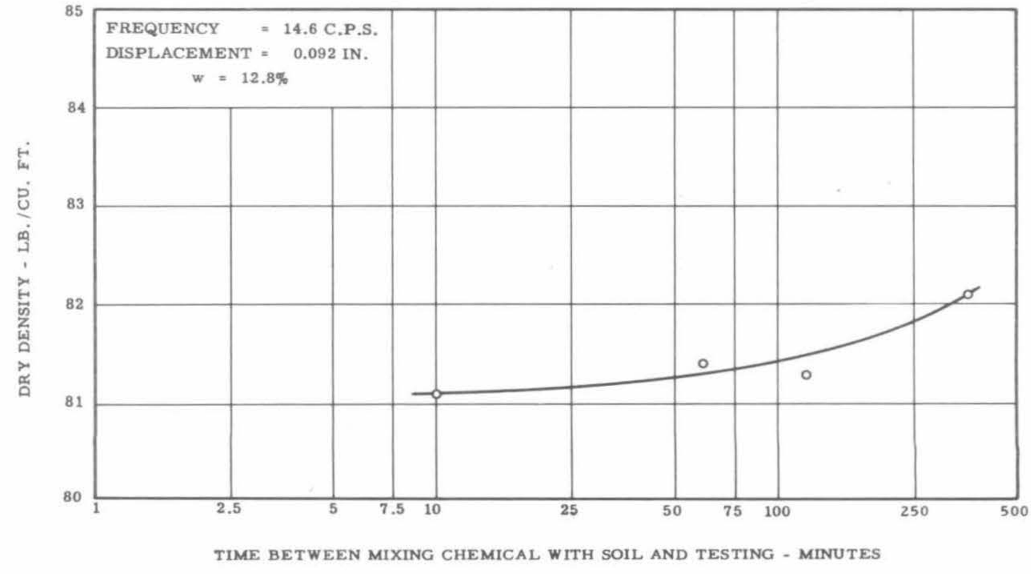

Fig. 4.13 CURVE RELATING DRY DENSITY OF SOIL AND THE TIME BETWEEN MIXING SODIUM SULPHATE WITH THE SOIL AND TESTING. CALTECH SANDY LOAM.

TABLE 4.4

VIBRATION-TABLE TESTS TO DETERMINE

EFFECT OF TIME BETWEEN MIXING AND TESTING

All experiments conducted at a frequency of 14.6 c.p.s., with a displacement of 0.092 inches peak-to-peak providing a maximum acceleration equal to gravity.* Soil tested was Port Hueneme Sandy Loam containing 1 gm. of Sodium Sulphate in 600 gm. of dry soil.

Experiment No.	Time between Mixing in Chemical Sol^n. and Vibration min.	Moisture Content of Sample % dry wt.	Dry Density after 60 sec. Vibration lb./cu.ft.
T-1	10	19.4	100.6
T-2	60	19.2	102.8
T-3	2	19.7	99.9
T-4	300	19.7	102.1
T-5	960	20.2	101.2
T-6	30	20.2	101.9
T-7	90	19.9	102.9
T-8	120	19.7	103.1
T-9	180	19.9	102.8
T-10	10	21.3	99.0
T-11	60	21.4	100.1
T-12	120	21.0	102.0
T-13	360	21.1	100.3
In the following tests, 1 gm. of Sodium Sulphate was added to 600 gm. of Caltech Sandy Loam			
T-14	10	12.6	81.1
T-15	60	12.7	81.4
T-16	120	13.1	81.3
T-17	360	12.8	82.1

Note: *See section 4.3.4.

These chemicals were:

Sodium Sulphate

Aerosol IB (Di-iso-butyl sodium sulphosuccinate)

Aerosol MA (Di-hexyl sodium sulphosuccinate)

Aerosol OT (Di-octyl sodium sulphosuccinate)

Aerosol TR (Di-iso-tridecyl sodium sulphosuccinate)

Zinc Aerosol OT (Di-octyl zinc sulphosuccinate)

Barium Aerosol OT (Di-octyl barium sulphosuccinate)

The group of Aerosols was chosen because of the excellent results obtained with Aerosol OT and IB in the vibration-table tests, and because all are strongly active wetting agents. The sodium sulphate was included as it was the best chemical in the vibration-table tests. Zinc and barium Aerosol OT were added because they were identical to the Aerosol OT except for the presence of zinc and barium, respectively, instead of sodium.

With this group, it was anticipated that most of the effects outlined in section 4.3 could be determined at least qualitatively.

4.4.2 Apparatus and Procedure

The direct shear machine consisted of a square brass box designed to hold a soil sample $2\frac{1}{2}$ in. in diameter by 1 in. thick and split horizontally at the level of the center of the soil sample. Porous stories were placed on the top and bottom of the sample. Normal load was provided by a lever arm and suspended weights. The upper part of the box was held stationary while the lower part was moved at a constant rate of 0.1 inch per minute giving rise to a shear load on the specimen, gradually rising to a maximum at which the sample failed to shear.

The general procedure was to mix a determined quantity of water and chemical with sufficient sieved soil to make up four test samples, this number being considered the minimum required to establish the Mohr envelope for determining the constants known as "cohesion" and "internal friction" of the soil under the given conditions of moisture and chemical content.

The sieving was necessary as early tests with unsieved soil exhibited an undesirable scatter of results. Also it was recommended in the A.S.T.M. Procedures for Testing Soils, 1944 edition, that the maximum grain size of the soil should be one-twentieth that of the diameter of the shear ring. Passing the soil through a number 14 sieve effected more uniform results and satisfied the grain-size recommendation.

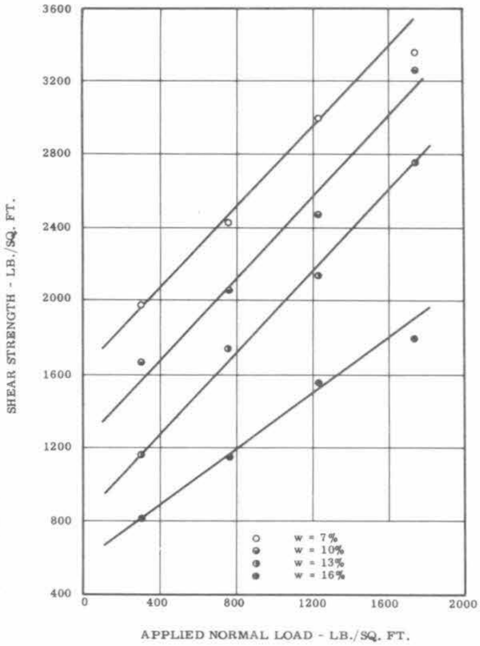

Fig. 4.14 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. NO CHEMICAL ADDED.

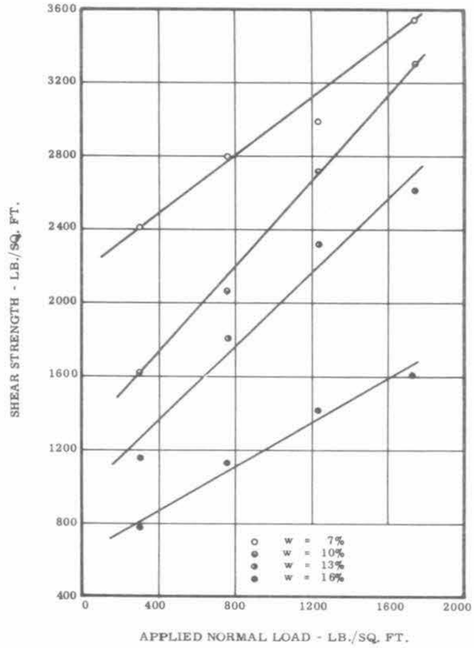

Fig. 4.15 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. SODIUM SULPHATE ADDED.

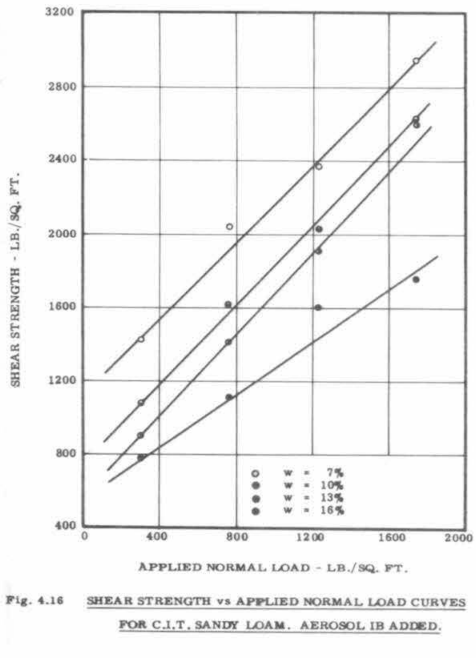

Fig. 4.16 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. AEROSOL IB ADDED.

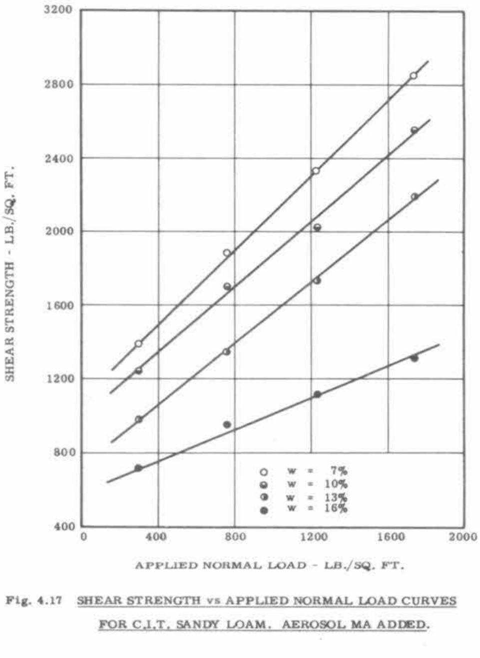

Fig. 4.17 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. AEROSOL MA ADDED.

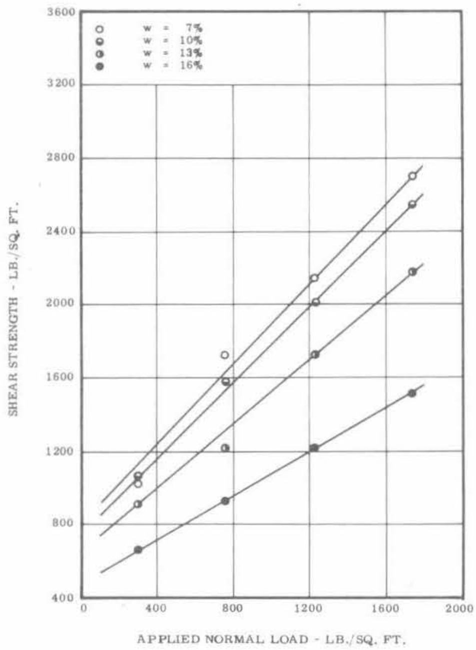

Fig. 4.18 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. AEROSOL OT ADDED.

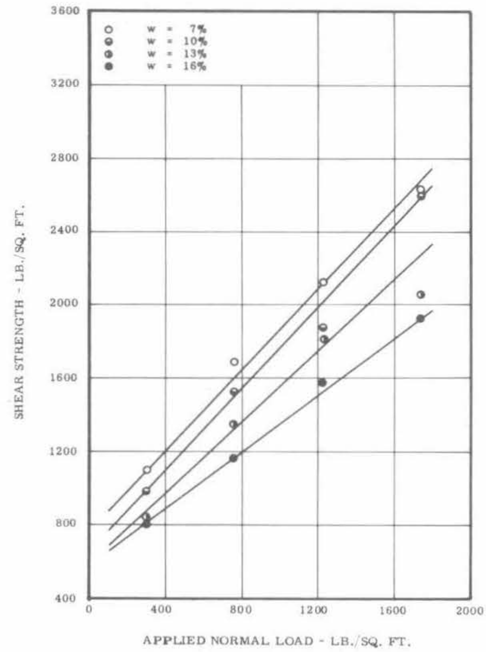

Fig. 4.19 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. AEROSOL TR ADDED.

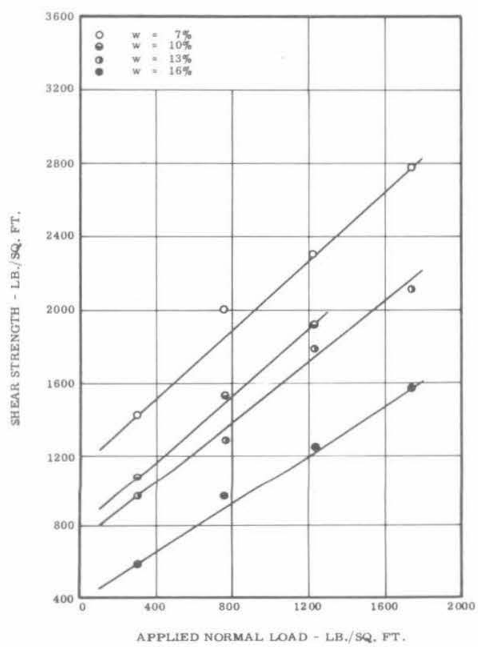

Fig. 4.20 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. ZINC AEROSOL OT ADDED.

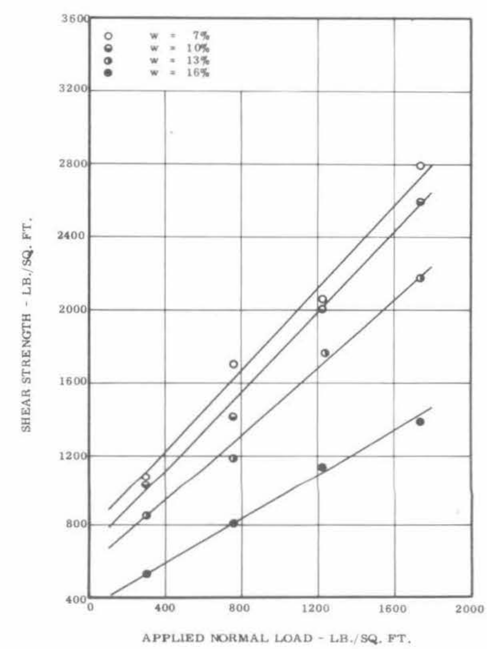

Fig. 4.21 SHEAR STRENGTH vs APPLIED NORMAL LOAD CURVES FOR C.I.T. SANDY LOAM. BARIUM AEROSOL OT ADDED.

The mixed soil was then placed in a brass ring 1 in. high by 2.42 in. internal diameter and tamped to form a sample filling the brass ring to a density of 90% of maximum. This density was chosen because the conditions represented by soil at this density must be overcome before adequate compaction in the field can be attained. After preparation of the sample in this manner, it was stored between glass plates until required.

Normally, unless the time-effect of the added chemicals was being investigated, a batch of four samples was made up and tested within a period of 60 minutes. This, however, was not always possible, and certain vibrations in the results were attributed to undue extension of the time interval.

4.4.3 Limitations of Results

The effect of the amount of chemical added to the soil was not investigated, 1/3% by weight being added in all the tests, since this quantity had proved effective in the small scale vibration tests. The effect of the time-interval between mixing and testing was investigated, using periods from 1 hour to 28 days. However, it was found that although there was a noticeable effect existing, the results were very irregular. On many occasions when a series of tests was run a second time, the resulting relationship from the second series was completely the reverse of that arising from the first series, maximum shear values in the one corresponding with minimum shear values in the other, and vice-versa. The reason for this is probably because the moisture content of the relatively dense samples was never higher than 14% (tests were made at 10%, 12%, and 14% moisture contents). Thus any movement of ions or other electrically charged particles such as dipoles would be severely restricted. This is to be compared with the vibration-table tests, section 4.3, in which very regular results were obtained with samples prepared and stored in a relatively loose state at high moisture contents.

The results of other tests are shown in Table 4.5 and Figures Nos. 4.14 to 4.21.

4.4.4 Discussion of Results

The most significant of the results is that sodium sulphate causes an increase in shear strength, except at the highest moisture content involved. This is made more significant by the fact that at the low moisture content values, the presence of sodium sulphate in the soil during the vibration-table tests caused a considerable increase in the density of the soil. The reason for this is not clear. One possible explanation is that the form of bond between the particles becomes thixotropic when the sodium sulphate is added to the soil. Thixotropy is

TABLE 4.5

EFFECT OF CHEMICALS ON PROPERTIES OF SOIL

Soil: C.I.T. Sandy Loam

Chemical	Gram-Molecular Weight gm.	Moisture Content of Soil 7%		10%		13%		16%	
		C	ϕ	C	ϕ	C	ϕ	C	ϕ
Water		1.63	48	1.23	48	0.82	48.5	0.59	47.5
Sodium Sulphate	142	2.17	48.5	1.28	49	0.96	45	0.63	31
Aerosol IB	332	1.12	46	0.75	47.5	0.57	48	0.55	35.5
Aerosol MA	388	1.08	45.5	0.99	41.5	0.73	40	0.59	23.5
Aerosol OT	444	0.80	47.5	0.74	46	0.66	41	0.49	31
Aerosol TR	584	0.76	48	0.66	48	0.59	44	0.59	37.5
Zinc Aerosol OT	907	1.15	43	0.80	42			0.38	34
Barium Aerosol OT	980	0.78	48	0.68	47.5	0.58	42.5	0.34	32

Note: C in Kips per Sq. Ft.
ϕ in degrees.

---

defined as the property or phenomena, exhibited by certain systems, of becoming fluid when shaken, that is, of showing decreasing viscosity with increasing rate of shear. Lambe[13] explains thixotropy, in conjunction with a colloidal suspension, by means of a plot of potential energy versus the distance apart of the soil particles. This, however, is not applicable in the case of the shear samples, and some other explanation of the cause of thixotropy is required before it can be considered as the reason for the behaviour of the sodium-sulphate-treated soil.

The increase in shear strength of the soil when treated with sodium sulphate may be partly explained by a reduction in the 'solid' water present in the soil. Solid water is the water held so closely to the solid particles that it behaves like a solid itself, i.e. is very highly viscous. If this solid water were decreased in content, two easily discernable physical quantities would change. Firstly, the apparent moisture content would increase and secondly, the liquid limit and plasticity index would increase as a result of the smaller effective particle diameter. This was found to be the case in these tests. The data for the effect on the L.L. and P.I. of the C.I.T. Sandy Loam are given in Table 4.6.

TABLE 4.6

LIQUID LIMIT AND PLASTICITY INDEX OF C.I.T. SANDY LOAM

Chemical Additive	Plastic Limit	Liquid Limit	Plasticity Index
None	21	25	4
Sodium Sulphate	17	27	10

The raising of the liquid limit on the addition of sodium sulphate to the soil is contrary to the concept of thixotropy.

Effect of Different Anions and Cations, and of the Molecular Weight of the Chemical. The effect of the different anions is difficult to assess, being closely connected with other parameters such as wetting-power and molecular weight. As far as can be judged, however, as the anion increases in complexity, and thus as the molecular weight of the chemical increases, the greater is the reduction in the shear strength of the soil, see Fig. No. 4.22. The effect of the cation is more easily seen by comparison between the sodium, barium, and zinc aerosol OT, although the wetting-power is also a factor to be considered. The conclusion is that further tests with a wider range of cations is necessary, there being no significant trend other than a slight increase in shear strength given by the addition of zinc aerosol OT.

Effect of the Moisture Content of the Soil. A gradual decrease in shearing strength with increasing moisture content was noted in all cases. As shown in figures nos. 4.23 to 4.26, there appears to be a peak value of shear strength at a moisture content of less than 9%, that is, less than the optimum moisture content of the moisture-density relationship for this soil. This is in agreement with the work of others, such as Golder[14].

4.4.5 Conclusions

The shearing resistance of the soil with the addition of the chemicals of this series was lower than that of the soil without chemicals. An exception to this was obtained with Sodium Sulphate. The addition of this chemical produced greater shearing resistance at a soil moisture content of 7%, a slightly greater resistance at moisture contents 10% and 13%, and slightly smaller value at 16% moisture.

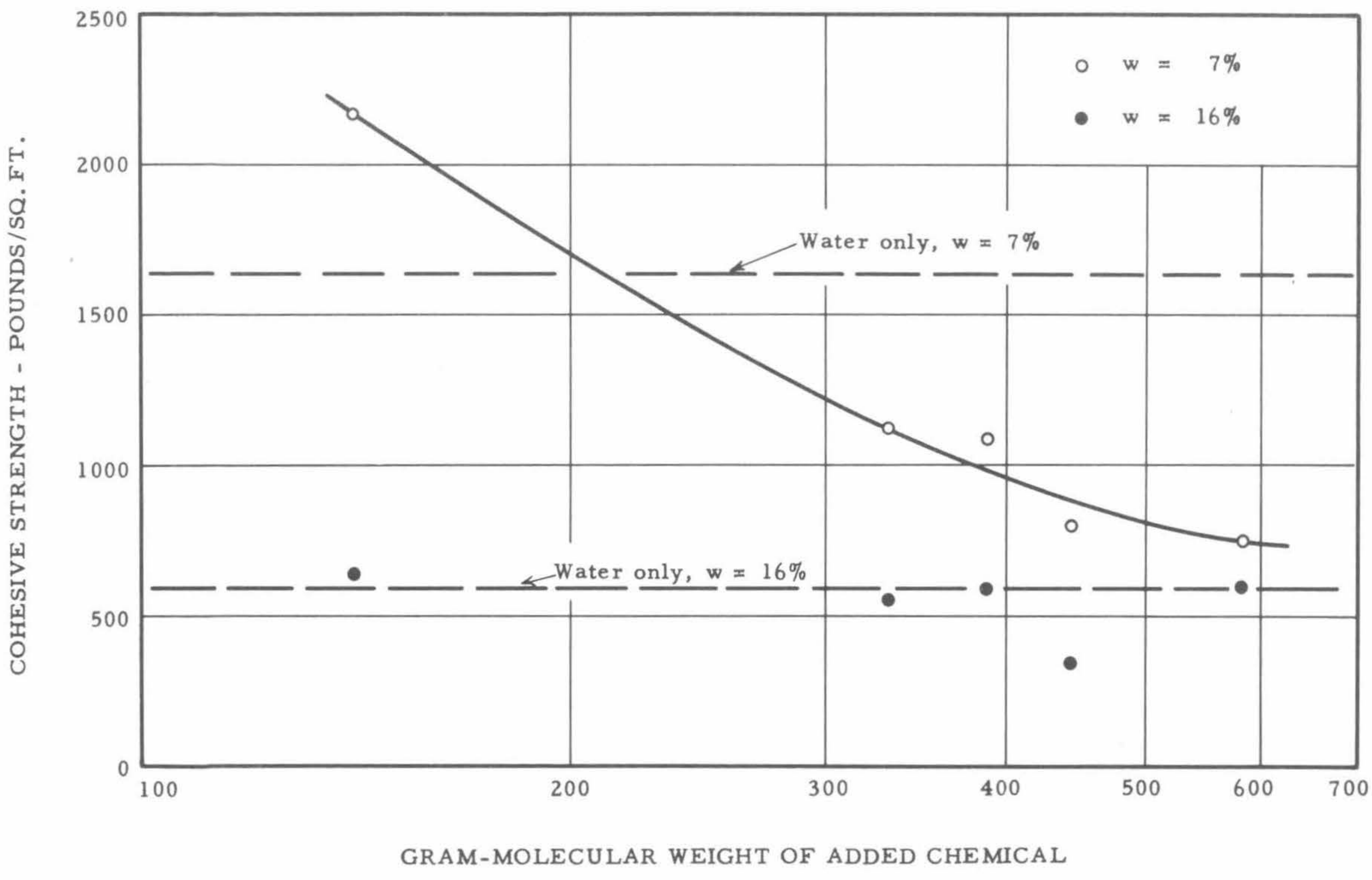

Fig. 4.22 EFFECT OF THE MOLECULAR WEIGHT OF THE ADDED CHEMICAL ON THE COHESIVE STRENGTH OF THE SOIL.

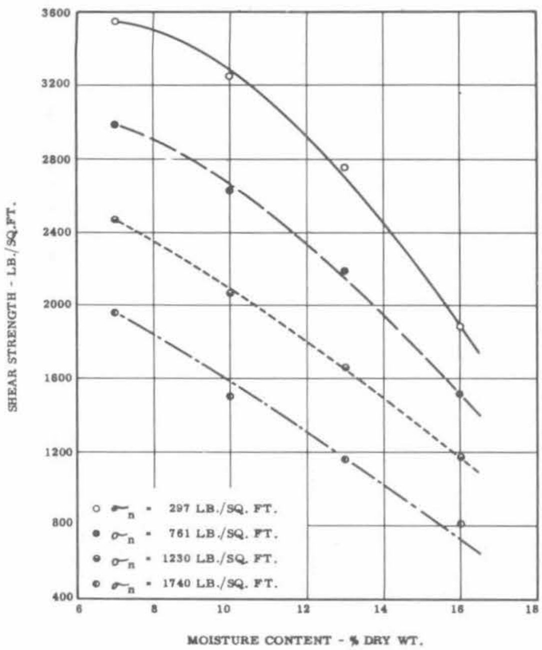

Fig. 4.23 SHEAR STRENGTH vs MOISTURE CONTENT CURVES FOR CALTECH SANDY LOAM. NO CHEMICAL ADDED.

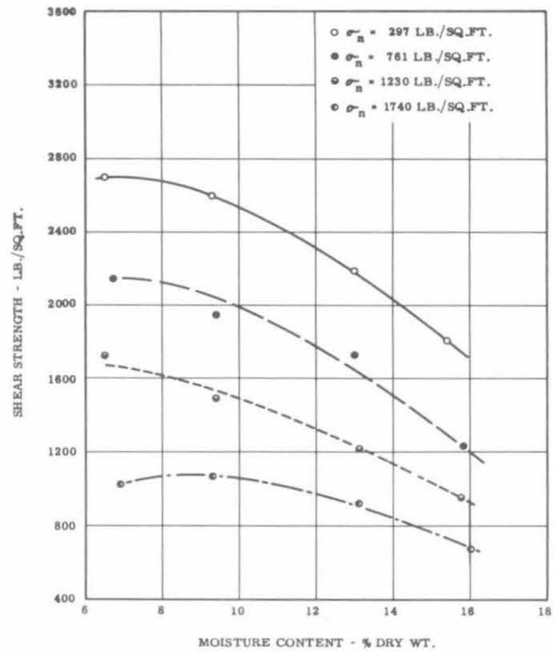

Fig. 4.24 SHEAR STRENGTH vs MOISTURE CONTENT CURVES FOR CALTECH SANDY LOAM. AEROSOL OT ADDED.

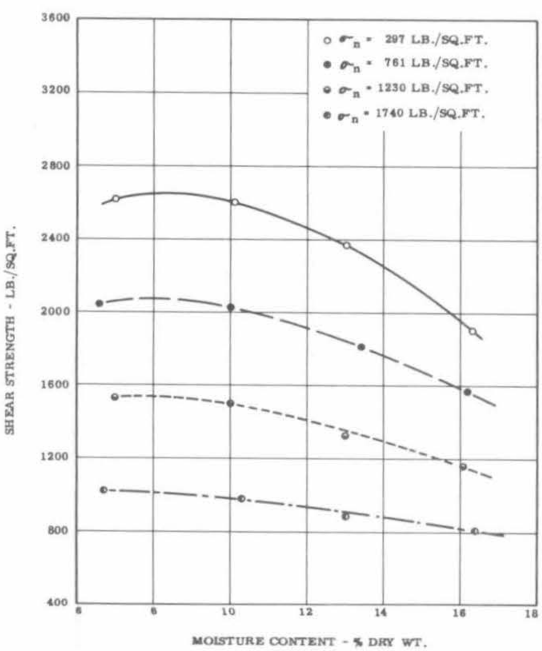

Fig. 4.25 SHEAR STRENGTH vs MOISTURE CONTENT CURVES FOR CALTECH SANDY LOAM. AEROSOL TR ADDED.

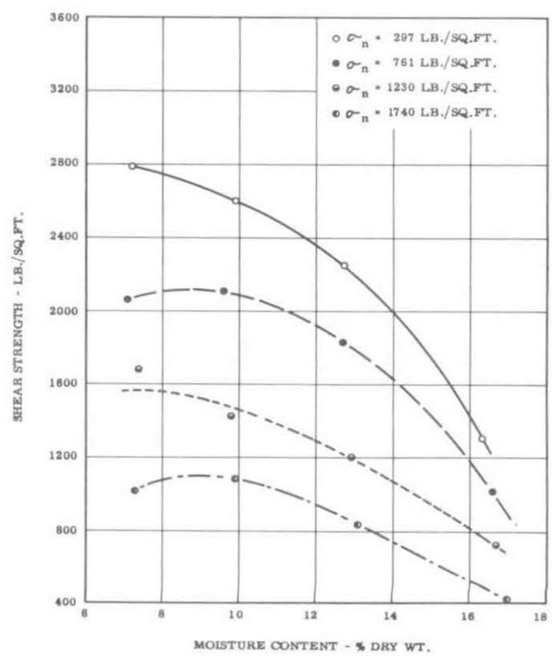

Fig. 4.26 SHEAR STRENGTH vs MOISTURE CONTENT CURVES FOR CALTECH SANDY LOAM. BARIUM AEROSOL OT ADDED.

4.5 General Conclusions

By nature of the scope and purpose of this research project, the theoretical studies and laboratory investigations outlined in this chapter were necessarily limited in extent. The most significant aspect of the investigations is the indication given of changes that may be made in soil properties by treatment with chemicals. This is, of course, not new in the realm of soil mechanics, but the majority of work up to the present has been concerned with either soil stabilization from the solidification point of view, or with base-exchange effects mainly on the Atterberg Limits, permeability, etc. The main concern of the present work was to see whether the addition of chemicals would enable compaction to be obtained more easily. In this respect, the investigations were undoubtedly useful, resulting in two conclusions which subsequently aided the field compaction of soil by vibration, namely:

(a) Moisture content is important, and should preferably be slightly higher than the optimum value given by the modified A.A.S.H.O. laboratory test;

(b) Chemicals can be useful in aiding compaction, as shown by the beneficial effect of Daxad 23 in subsequent tests with the Lazan oscillator.

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CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 General

Compaction of moderately cohesive soil by both the Lazan oscillator and the Navy-Caltech vibrator has been accomplished. The process may be more accurately termed "resonant impact compaction," than "vibration compaction," since the mechanics of compaction is quite different from that which occurred in the densification of sand.

Only a limited number of field experiments have been made on sandy loam soils with either the Lazan oscillator or the Navy-Caltech vibrator, but on the basis of these tests the requirements for successful compaction may tentatively be stated as follows:

1. Unit dead weight contact pressure is important, and the minimum effective value is probably dependent upon the cohesion of the soil. Unit pressures on the order of three to four psi were effective on the sandy loam soils tested.

2. The dynamic force should be at least equal to the dead weight of the oscillator. It is probable that a dynamic force greater than the dead weight may be desirable.

3. The soil-oscillator system should be in resonance.

4. The soil moisture may be of considerable importance. In these tests, the best results were obtained with the soil moisture slightly in excess of the optimum as determined by the modified A.A.S.H.O. method, in which a ten pound hammer is dropped 25 times from a height of 18 inches on each of three layers of soil in a 1/30th cubic foot container.

5. Time in one location or speed of towing is important. In these tests at least ten seconds were required to obtain the desired increase in density in a stationary location, and three passes of the Navy-Caltech vibrator at 30 ft/min resulted in continually increasing densities.

5.2 Chemical Additives

Some chemicals will reduce the surface tension of the soil moisture and the chemical bond between the particles to such an extent that less energy is required to compact the soil. Only very small quantities (on the order of 1/6 to 1/3 percent by weight) of either Daxad 23 or Sodium Sulphate were required. Best results were obtained when a curing period of several hours was allowed after mixing, in order to permit diffusion of the moisture through the soil.

5.3 Suggestions for Future Investigations

In order to determine whether or not the conclusions arrived at on the basis of the limited number of tests so far performed are correct, and to extend the knowledge of cohesive soils under vibratory impact, the following further investigations are recommended using small-scale field tests with the Lazan oscillator or equipment of similar size:

1. Determine the effect of moisture on the compaction of soils.
2. Determine the effect of the ratio of dynamic force to dead weight.
3. Determine the effect of unit soil pressure on the degree of compaction.

The conclusions resulting from the small-scale tests should be checked in the field with a large vibrator of the Navy-Caltech type. It is especially desirable to compare stationary tests at different time intervals with the results of various numbers of passes at different speeds.

The theory of compaction of cohesive soils should be checked with test results, and a laboratory method developed for estimating the resonant frequency of the oscillator soil system closely enough so that a small amount of judgment is required in the field by the operator.

APPENDIX A

THEORY OF VIBRATIONS
APPLIED TO A
VIBRATOR-SOIL SYSTEM

NOTATION

A	amplitude of oscillation
A_n	amplitude at natural frequency
A_z	area of a section at depth z
a	length of vibrator base
b	width of vibrator base
C	constant
C_k	spring factor coefficient
c	(1) damping ratio β/β_c; (2) two times the tangent of an angle defining a zone of pressure distribution
d	diameter of vibrator base
E	modulus of elasticity
E'	increase in the modulus of elasticity per unit depth
e	(1) eccentricity; (2) base of natural logarithms
F_o	exciting force
G	shearing modulus of elasticity
G'	increase in the shearing modulus of elasticity per unit depth
g	acceleration of gravity
h	equivalent surcharge height
i	mass moment of inertia
i'	apparent mass moment of inertia of soil
k	spring factor
M	moment
m	mass
m'	(1) apparent soil mass; (2) eccentric mass
m_v	mass of vibrator
m_k	inertia parameter ($k = 1, 2, 3, \ldots 6$)
N	frequency ratio ω/ω_n
P	total surface load
P_i	power input
P_d	power dissipated in damping
P_z	vertical load at depth z
q	unit surface load
q_k	generalized independent coordinate ($k = 1, 2, 3, \ldots 6$)
r	ratio a/b
S	shearing force
s	ratio ch/b
T	kinetic energy
t	time
U	potential energy
u	shearing deformation
x	(1) linear displacement; (2) coordinate axis
y	coordinate axis
z	(1) depth; (2) coordinate axis
z'	ratio cz/b
$\bar{z}$	centroidal distance
β	damping factor
β_c	critical damping factor
δ_o	total surface deformation
δ_z	vertical deformation at depth z
δ_{st}	static deflection
θ_o	angular displacement of the soil surface
θ_z	angular displacement of a section at depth z
Λ	coefficient for power dissipated in damping
μ	poisson's ratio
ρ	unit weight of soil
ϕ	phase angle
ω	frequency
ω_n	natural frequency

THEORY OF VIBRATIONS APPLIED TO A VIBRATOR-SOIL SYSTEM

5.1 Synopsis

The purpose of this chapter is to develop a simplified analysis of vibrator-soil systems, using the theory of linear vibrations for systems with finite numbers of degrees of freedom. The problem is treated as a linear one by using equivalent values of the viscous damping ratio, spring factor, and effective mass.

In Parts I and II, the theory of vibration for single and multiple degrees of freedom systems is reviewed. Equations are developed for predicting the natural frequencies of the vibrator-soil system treated as a simplified equivalent undamped system having six degrees of freedom. To utilize the equations derived in Part II, the equivalent spring factors of the soil and the inertia parameters of the system must be determined. Procedures for computing approximate values of these parameters are considered in Parts III and IV. These procedures are based on several simplifying assumptions, principal of which are:

(1) the effective modulus of elasticity of the soil increases linearly with depth,
(2) only a truncated cone or pyramid of soil directly beneath the vibrator is effective in distributing the vibrator load.

The results of Parts III and IV are presented in graphical form to facilitate evaluation of the required parameters.

5.2 General Principles

Mechanical oscillations occur when a body displaced from its position of static equilibrium is acted upon by inertia and restoring forces. In the case of a vibrator placed in contact with a semi-infinite mass of soil, the inertia is provided by the mass of the vibrator plus a portion of the soil which moves with it, and the restoring force is furnished by the elasticity of the soil.

Unless external energy is supplied, a disturbed system will gradually come to rest due to energy losses. The sources of these losses in the vibrator-soil system are dissipation due to radiation of energy into the soil mass, and damping resulting from friction between the soil particles. The combined effect of these energy losses is most conveniently expressed in terms of an equivalent damping force producing the same energy losses in the system. It is customary to assume that the damping is viscous, and therefore the damping force is equal to an equivalent damping factor β times the velocity of the vibrator. The assumption of viscous damping yields results which are satisfactory for most vibration analyses.

A vibration system may have several degrees of freedom, that is, oscillation may occur about or parallel to a number of axes in the system. The vibrator under consideration may be treated as a rigid rectangular unit for which there are six degrees of freedom, namely translation parallel to the three principal centroidal axes of the system, and rotation about these axes. Equations for this system are developed in Part II

PART I

SINGLE DEGREE OF FREEDOM SYSTEMS

5.3 Equivalent Spring System

The simplest mechanical analogy to the vibrator-soil system is shown schematically in Figure 5.1.

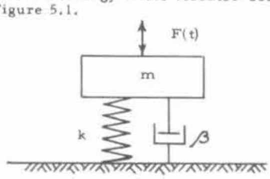


Figure 5.1

The mass of the vibrator plus the effective portion of the soil moving with it constitute an equivalent mass m oscillating on a damped spring. The differential equation for this single degree of freedom system is

$$m\ddot{x} + \beta\dot{x} + kx = F(t) \tag{5.1}$$

5.4 Damped Free Oscillation

Consider first the case where the vibrator is given an initial displacement and then allowed to oscillate freely about its position of static equilibrium. The equation of motion is

$$m\ddot{x} + \beta\dot{x} + kx = 0 \quad , \tag{5.2}$$

for which the general solution is of the form

$$x = C_1 e^{\lambda_1 t} + C_2 e^{\lambda_2 t} \tag{5.3}$$

where

$$\lambda = \frac{-\beta \pm \sqrt{\beta^2 - 4km}}{2m} \tag{5.3a}$$

If $\beta^2 \geq 4km$, equation (5.3) contains no terms which vary periodically with time, and the vibrator will return to its initial position without oscillatory motion. When $\beta^2 = 4km$, the system is said to be critically damped. The critical damping factor is

$$\beta_c = 2\sqrt{km} \quad . \tag{5.4}$$

If $\beta^2 < 4km$, the values of λ are complex numbers, and by the use of the Euler Formulas, equation (5.3) may be written

$$x = e^{-\beta t/2m}\left[C_1' \cos\sqrt{k/m-(\beta/2m)^2}\; t + C_2' \sin\sqrt{k/m-(\beta/2m)^2}\; t\right] \tag{5.5}$$

where

$$C_1' = C_1 + C_2 \tag{5.5a}$$

$$C_2' = i(C_1 - C_2) \quad . \tag{5.5b}$$

Equation (5.5) represents a vibratory motion of decreasing amplitude. The frequency of oscillation is

$$\omega = \sqrt{k/m - (\beta/2m)^2} \quad \text{radians per second.} \tag{5.5c}$$

For the special case $\beta = 0$, (5.5) and (5.5c) reduce to the equations for free oscillation of an undamped system:

$$x = C_1' \cos\omega t + C_2' \sin\omega t \tag{5.6}$$

$$\omega = \sqrt{k/m} \tag{5.6a}$$

5.5 Forced Oscillation

Consider next the motion of a one degree of freedom system under the action of a periodic external force. The equation of motion for this case may be written

$$m\ddot{x} + \beta\dot{x} + kx = F_o \sin\omega t \quad . \tag{5.7}$$

In the application considered, we are interested only in the steady-state oscillation which is given by the particular solution

$$x = \frac{F_o}{\sqrt{(k - m\omega^2)^2 + \beta^2\omega^2}} \sin(\omega t - \phi) \tag{5.8}$$

where ϕ, the phase angle between the exciting force and the displacement of the vibrator is

$$\phi = \tan^{-1}\frac{\beta\omega}{k - m\omega^2} \quad . \tag{5.8a}$$

The natural frequency of the system (i. e., the frequency at which the system would oscillate freely with no damping) may be defined by

$$\omega_n = \sqrt{k/m} \quad . \tag{5.9}$$

Recalling that the critical damping factor is given by $\beta_c = 2\sqrt{km}$, and defining

$$N = \omega/\omega_n$$

$$c = \beta/\beta_c \quad ,$$

equations (5.8) and (5.8a) may be written

$$x = \frac{F_o/k}{\sqrt{(1 - N^2)^2 + (2cN)^2}} \sin(\omega t - \phi) \tag{5.10}$$

$$\phi = \tan^{-1}\frac{2cN}{1 - N^2} \quad . \tag{5.10a}$$

The maximum amplitude occurs when $\sin(\omega t - \phi) = 1$, that is,

$$A = \frac{F_o/k}{\sqrt{(1 - N^2)^2 + (2cN)^2}} \quad . \tag{5.11}$$

When the frequency ratio $N = 1$, the phase shift $\phi = 90°$ for all damping ratios, and (5.11) may be written

$$A_n = F_o/2ck \quad . \tag{5.11a}$$

Equation (5.11) may also be written

$$A/\delta'_{st} = \frac{1}{\sqrt{(1 - N^2)^2 + (2cN)^2}} \tag{5.11b}$$

where δ'_{st} is defined as the static deflection produced by the exciting force F_o, that is,

$$\delta'_{st} = F_o/k \quad . \tag{5.11c}$$

The ratio A/δ'_{st} is called the dynamic amplification factor and is plotted in Figure 5.2 for several damping ratios. It is apparent from these curves that an increase in damping appreciably reduces the amplification factor in the region $0.7 < \omega/\omega_n < 1.4$. The resonant frequency (i. e., the frequency at which the maximum amplitude of oscillation occurs) is slightly lower than the natural frequency of the system, and consequently, the amplitude at $\omega/\omega_n = 1$ is somewhat less than maximum. In most applications the damping ratio is small ($c < 0.2$), and the difference between the maximum amplitude and the amplitude at the natural frequency is negligible.
For these cases the terms resonant and natural frequency are practically synonymous.

The curves in Figure 5.2 may be normalized by dividing all ordinates by the ordinate corresponding to the frequency ratio $\omega/\omega_n = 1$. The resulting curves (Figure 5.3) may be used to determine the damping ratio of a system by comparing them with a normalized curve of measured displacements.

5.6 Power Considerations

In a system having sustained steady-state oscillations, the average power input must equal the average power dissipated in damping. Power is the rate of doing work, hence the input power is

$$P_i = F\frac{dx}{dt} = F_o \sin \omega t \cdot A\omega \cos(\omega t - \phi) \qquad (5.12)$$

where F_o is the magnitude of the exciting force and A is the amplitude of oscillation as given by equation (5.11). By a simple trigonometric transformation, it can be shown that

$$\sin\omega t \cdot \cos(\omega t - \phi) = 1/2\left[\sin\phi + \sin(2\omega t - \phi)\right] ,$$

therefore
$$P_i = \frac{F_o A \omega}{2}\left[\sin\phi + \sin(2\omega t - \phi)\right] \quad . \qquad (5.12a)$$

Inspection of equation (5.12a) shows that the input power oscillates at double the frequency of the exciting force about the mean level

$$P_{i_{ave}} = \frac{F_o A\omega}{2}\sin\phi \; . \qquad (5.13)$$

When $\omega = \omega_n$, $\sin\phi = 1$, and

$$P_{i_{ave}} = \frac{F_o A_n \omega_n}{2} \quad . \qquad (5.13a)$$

The power dissipated in damping is given by

$$P_d = F_d \cdot v = \beta A^2\omega^2 \cos^2(\omega t - \phi) \quad . \qquad (5.14)$$

The difference between (5.12a) and (5.14) represents the potential energy stored in the spring and the kinetic energy of the mass. The average value of the power dissipated is

$$P_{d_{ave}} = \frac{\beta A^2 \omega^2}{2} \qquad (5.15)$$

since the average value of $\cos^2(\omega t - \phi)$ is 1/2. Equating (5.13) and (5.15) we obtain

$$\beta = \frac{F_o \sin\phi}{A\omega} \; . \qquad (5.16)$$

In the compaction vibrator, rotating weights are used to produce the periodic exciting forces. These forces are proportional to the square of the frequency and are given by

$$F(t) = F_o \sin \omega t \qquad (5.17)$$

where
$$F_o = m' e \omega^2 \qquad (5.17a)$$

In (5.17a) m' is the mass and e the eccentricity of the rotating weights. Combining equations (5.11) (5.15) and (5.17a), the average power dissipated is found to be

$$P_{d_{ave}} = \frac{(m'e)^2\omega^6}{2k^2\ (1 - N^2)^2 + (2cN)^2} \quad .$$

Recalling that $\beta = c\beta_c = 2c\sqrt{km}$ and that $\omega_n = \sqrt{k/m}$ the above equation may be written

$$P_{d_{ave}} = k(m'e/m)^2\omega_n \Lambda \qquad (5.18)$$

where
$$\Lambda = \frac{cN^6}{(1 - N^2)^2 + (2cN)^2} \quad . \qquad (5.18a)$$

Λ is plotted as a function of the frequency ratio for several values of c in Figure 5.4.

PART II

MULTIPLE DEGREE OF FREEDOM SYSTEMS

5.7 Introduction

The soil vibrator is restrained only by the reaction of the soil, hence it has several degrees of freedom and can oscillate simultaneously about a number of axes. When using the vibrator as a compaction machine we are interested primarily in the vertical mode. However, the other modes are of some interest in that they may affect the stability of the machine at certain operating frequencies. For the design of compaction equipment it is therefore desirable to determine the resonant frequencies of these modes.

In the discussion of single degree of freedom systems, it is noted that damping causes only a small shift in the resonant frequency. Similarly, for a multiple degree of freedom system, frequencies of the undamped system are approximately equal to the resonant frequencies of the damped system. The following analysis is simplified by neglecting damping.

5.8 La Grange's Equations*

The differential equations for the system are most readily determined by the use of La Grange's equations of motion. To apply these equations to our problem, a set of generalized independent coordinates, $q_1, q_2, q_3, \ldots q_6$, are selected. These coordinates correspond to the number of degrees of freedom of the system. Since damping is neglected, the system is conservative, and La Grange's equations take the form

$$\frac{d}{dt}\left(\frac{\partial T}{\partial \dot{q}_k}\right) - \left(\frac{\partial (T - U)}{\partial q_k}\right) = 0 \qquad (5.19)$$

where T is the kinetic energy and U the potential energy of the system. Equation (5.19) may be further simplified by selecting the coordinate system so that in the equilibrium position

(a) the level of the potential energy is zero,
(b) $q_1 = q_2 = q_3 = \ldots = q_6 = 0$.

With these coordinates, and for small oscillations, La Grange's equations reduce to the simple form

$$\frac{d}{dt}\left(\frac{\partial T}{\partial \dot{q}_k}\right) + \frac{\partial U}{\partial q_k} = 0 \quad . \qquad (5.20)$$

* Karman, T. von, and Biot, M. A., 'Mathematical Methods in Engineering,' McGraw-Hill (1940). p.101., p.106, p.165.

5.9 Generalized Coordinates for the Vibrator

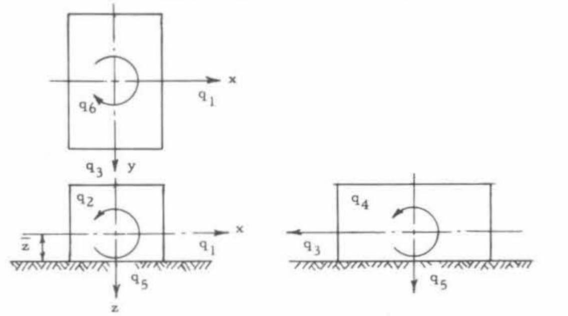


Figure 5.5

The requirements of Section 5.8 may be satisfied by selecting the coordinates as shown in Figure 5.5. The center of coordinates lies a distance $\bar{z}$ above the contact plane and is the center of the combined mass of the vibrator and the apparent mass of the soil moving with it.

5.10 Energy Equations

If the machine is depressed a unit distance into the soil, the soil will exert a force on the base of the machine which may be defined by k_z. This force is called the vertical spring factor of the soil. Similarly, for unit displacements in the x and y directions, the spring factors are k_x and k_y. For unit-angle rotations about the x, y, and z axes through the centroid of the contact plane, the spring constants are k_{yz}, k_{xz}, and k_{xy}, respectively.

The total potential energy is given by

$$U = \tfrac{1}{2}\Big[k_x\ (q_1 + \bar{z}q_2)^2 + k_y\ (q_3 - \bar{z}q_4)^2 + k_z q_5{}^2 + k_{xz} q_2{}^2 + k_{yz} q_4{}^2 + k_{xy} q_6{}^2\Big] . \qquad (5.21)$$

To calculate the kinetic energy the inertia parameters of the system are required. These parameters are determined from the mass of the machine and the apparent mass of the soil. Corresponding to q_1, q_3, q_5, we have

$$m_1 = m_3 = m_5 = m_v + m' \qquad (5.21a)$$

where m_v is the mass of the vibrator and m' is the apparent mass of the soil. Similarly, corresponding to the rotational coordinates, q_2, q_4, and q_6 we have

$$m_2 = i_y + i_y' \qquad (5.21b)$$

$$m_4 = i_x + i_x' \qquad (5.21c)$$

$$m_6 = i_z + i_z' \qquad (5.21d)$$

where i is the mass moment of inertia of the vibrator, i' is the mass moment of inertia of the soil, and the subscripts refer to the centroidal axes about which the moments of inertia are determined.

Since the motion may be considered to be that of a rigid system, and the coordinates selected are independent, the kinetic energy of the system is given by

$$T = \frac{1}{2}\sum_{k=1}^{k=6} m_k \dot{q}_k{}^2 \qquad (5.23)$$

5.11 Differential Equations of Motion

Applying equation (5.20) it is seen that the differential equations of motion for the free undamped system are:

$$m_1\ddot{q}_1 + k_x(q_1 + \bar{z}q_z) = 0 \qquad (5.24a)$$

$$m_2\ddot{q}_2 + \bar{z}k_x(q_1 + \bar{z}q_z) + k_{xz}q_2 = 0 \qquad (5.24b)$$

$$m_3\ddot{q}_3 + k_y(q_3 - \bar{z}q_4) = 0 \qquad (5.24c)$$

$$m_4\ddot{q}_4 - \bar{z}k_y(q_3 - \bar{z}q_4) + k_{yz}q_4 = 0 \qquad (5.24d)$$

$$m_5\ddot{q}_5 + k_z q_5 = 0 \qquad (5.24e)$$

$$m_6\ddot{q}_6 + k_{xy}q_6 = 0 \qquad (5.24f)$$

5.12 Frequency Equations

Inspection of equations (5.24e) and (5.24f) reveals that they are of the same form as the differential equation for a single degree of freedom system. The frequencies of oscillation for these modes are therefore given by

$$\omega_5 = \sqrt{k_z/m_5} \qquad (5.25a)$$

and

$$\omega_6 = \sqrt{k_{xy}/m_6} \qquad (5.25b)$$

Equations (5.24a), (5.24b), (5.24c), and (5.24d) are somewhat more complicated, and the motion for these modes can not be defined by a single coordinate. The motion of the vibrator is 'coupled' and consists of combined horizontal translation and rotation in the xz-and yz-planes.

To determine the natural frequencies for these modes of oscillation, we assume as a solution

$$q_1 = a_1 \sin \omega t$$

$$q_2 = a_2 \sin \omega t$$

Substituting in (5.24a) and (5.24b) we obtain

$$-\omega^2 a_1 m_1 + k_x a_1 + \bar{z}k_x a_2 = 0$$

and

$$-\omega^2 a_2 m_2 + \bar{z}k_x a_1 + (\bar{z}^2 k_x + k_{xz})a_2 = 0$$

These equations are compatible only if the determinant of the coefficients for a_1 and a_2 is identically equal to zero. Thus:

$$\begin{vmatrix} (k_x - \omega^2 m_1) & \bar{z}k_x \\ \bar{z}k_x & (\bar{z}^2 k_x + k_{xz} - \omega^2 m_2) \end{vmatrix} = 0$$

Solving for ω,

$$\omega^4 - \omega^2\left(\frac{k_x}{m_1} + \frac{\bar{z}^2 k_x}{m_2} + \frac{k_{xz}}{m_2}\right) + \frac{k_x k_{xz}}{m_1 m_2} = 0$$

therefore

$$\omega_{12}^2 = \frac{1}{2}\left[\frac{k_x}{m_1} + \frac{\bar{z}^2 k_x + k_{xz}}{m_2} \pm \sqrt{\left(\frac{k_x}{m_1} + \frac{\bar{z}^2 k_x + k_{xz}}{m_2}\right)^2 - 4\,\frac{k_x k_{xz}}{m_1 m_2}}\right] \qquad (5.25c)$$

From (5.25c) it is seen that there are two frequencies ω_{12} which satisfy the compatibility requirement. These are therefore the two natural frequencies for the coupled modes in the xz-plane.
Similarly the frequencies for the coupled modes in the yz-plane are given by

$$\omega_{34}^2 = \frac{1}{2}\left[\frac{k_y}{m_3} + \frac{\bar{z}^2 k_y + k_{yz}}{m_4} \pm \sqrt{\left(\frac{k_y}{m_3} + \frac{\bar{z}^2 k_y + k_{yz}}{m_4}\right)^2 - 4\,\frac{k_y k_{yz}}{m_3 m_4}}\right] \qquad (5.25d)$$

Summarizing, the six natural frequencies corresponding to the six degrees of freedom are:

(a) two frequencies, ω_{12}, for the coupled modes in the xz-plane (Eq. 5.25c)
(b) two frequencies, ω_{34}, for the coupled modes in the yz-plane (Eq. 5.25d)
(c) the frequency for the vertical mode, ω_5 (Eq. 5.25a)
(d) the frequency for the rotational mode in the xy-plane, ω_6 (Eq. 5.25b).

Only two of these modes were excited in the case of the soil vibrator, namely the vertical mode and the low-frequency coupled mode in the plane perpendicular to the longitudinal axis.

PART III

EQUIVALENT SOIL SPRING FACTORS

5.13 Introduction

To utilize the expressions developed in Part II for predicting the modes of oscillation of the vibrator, the dynamic spring factors of the soil must first be determined. A spring factor may be defined either as the force exerted on the vibrator by the soil when the vibrator is displaced a unit distance from the equilibrium position, or as the moment, when the vibrator is rotated through a unit angle.

Six spring factors must be evaluated for a complete solution of all possible modes of oscillation. However, for the solution of the two excited modes of the vibrator, only three spring factors need be determined.

5.14 Basic Assumptions

Soils are not homogeneous, isotropic, elastic materials and do not lend themselves to a rigorous mathematical treatment. Therefore some simplifying assumptions are made to permit an approximate evaluation of the required spring factors.

Modulus of Elasticity of Soil

The effective modulus of elasticity may be defined as the ratio between the vertical unit pressure and the corresponding vertical strain. For soil, the elastic modulus is not a constant, but is a function of depth. As a first approximation, the relation between the modulus, E, and the depth, z, may be expressed by

$$E = C_1(C_2 + z) \qquad (5.26)$$

where C_1 and C_2 are constants. In the case of sand, the elastic deformations occur rapidly after the application of a stress increment; hence, it is further assumed that the dynamic modulus may also be approximated by (5.26).

Shearing Modulus of Elasticity of Soil

The shearing modulus of elasticity, G, is involved in the development of the soil spring factor for horizontal displacement. In an isotropic, elastic medium, the relation between G and the modulus of elasticity, E, is given by

$$G = \frac{E}{2(1+\mu)} \qquad (5.27)$$

where μ = poisson's ratio. Although the validity of this relation has not been definitely established for soils, equation (5.27) has been assumed in the following development.

Stress Distribution

Further simplifications are made in assuming that:
(1) Only a truncated cone or pyramid of soil directly under the vibrator is effective in distributing the load.
(2) The lateral faces of the effective soil prism make angles $\tan^{-1}(c/2)$ with the vertical.
In evaluating the equations developed for the spring factors, it is assumed that c is equal to unity. This assumption is equivalent to the 2 to 1 stress distribution frequently used to approximate the boundaries of the Boussinesq vertical stress distribution.

Finally, it is assumed that vertical stresses on any horizontal section of the effective soil zone are proportional to the deformations, and that plane surfaces remain plane after the application of dynamic loads.

5.15 Spring Factor for Vertical Displacement (k_z)

Consider an elemental cube of soil at a depth z below the surface, having dimensions dz, and subjected to a vertical load dP_z producing a deformation $d\delta_z$ (Figure 5.6). The elastic modulus is

$$E = \frac{dP_z/dz^2}{d\delta_z/dz} = \frac{dP_z}{d\delta_z \cdot dz} \qquad (5.28)$$

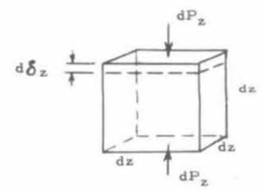


Figure 5.6

Consider next a rectangular vibrator of length a and width b applying uniform pressure q to the soil surface. Let the effective zone of pressure distribution be determined by the surface area ab and the planes sloping at an angle $\tan^{-1}(c/2)$ as shown in Figure 5.7a.

Recalling the basic assumption that the elastic modulus increases linearly with depth, and replacing the uniform surface load q with an equivalent depth of soil h, the effective modulus of elasticity may be written

$$E = E'(q/\rho + z) = E'(h + z) \qquad (5.29)$$

In equation (5.29) ρ is the unit weight of the soil, and E' is the increase in the modulus of elasticity per unit depth. This relation is shown graphically in Figure 5.7b.

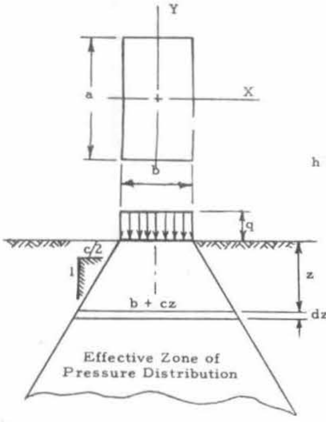


Figure 5.7a

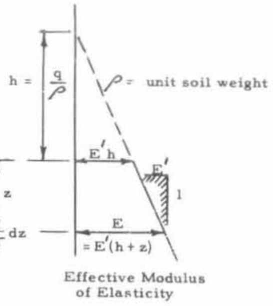


Figure 5.7b

The total pressure on any horizontal section of the effective zone is

$$P = \frac{A_z}{(dz)^2} \quad dP_z$$

where A_z, the area of the section is

$$A_z = (a + cz)(b + cz)$$

and dP_z is defined by equation (5.28). Substituting,

$$P = \frac{(a + cz)(b + cz)}{(dz)^2} \quad E \cdot dz \cdot d\delta_z \tag{5.30}$$

Solving for $d\delta_z$, and with E as defined in equation (5.29),

$$d\delta_z = \frac{P}{E'} \quad \frac{dz}{(a + cz)(b + cz)(h + z)} \quad .$$

Hence the total deformation of the contact surface is given by

$$\delta_o = \frac{P}{E'} \int_o^\infty \frac{dz}{(a + cz)(b + cz)(h + z)} \quad . \tag{5.31}$$

Defining $\quad r = a/b, \; a \geq b \quad$ (5.32)

and $\quad s = ch/b \quad ,\quad$ (5.33)

equation (5.31) may be rewritten in the form

$$\delta_o = \frac{P}{E'b^2} \int_o^\infty \frac{dz'}{(r + z')(1 + z')(s + z')} \tag{5.34}$$

where $\quad z' = cz/b \quad$ (5.34a)

By definition the spring factor k_z is given by

$$k_z = P/\delta_o \quad . \tag{5.35}$$

therefore $\quad k_z = E'b^2 C_{k_z} \quad$ (5.36)

where

$$\frac{1}{C_{k_z}} = \int_o^\infty \frac{dz'}{(r + z')(1 + z')(s + z')} \tag{5.36a}$$

Evaluating integral (5.36a), the following results are obtained:

$$C_{k_z} = \frac{r - s}{\frac{\log s}{s - 1} - \frac{\log r}{r - 1}} \quad , \quad r \neq s \neq 1 \tag{5.37a}$$

$$C_{k_z} = \frac{s - 1}{1 - \frac{\log s}{s - 1}} \quad , \quad r = 1, \; s \neq 1 \tag{5.37b}$$

$$C_{k_z} = \frac{r - 1}{1 - \frac{\log r}{r - 1}} \quad , \quad s = 1, \; r \neq 1 \tag{5.37c}$$

$$C_{k_z} = \frac{s - 1}{\frac{\log s}{s - 1} - \frac{1}{s}} \quad , \quad r = s \neq 1 \tag{5.37d}$$

$$C_{k_z} = 2 \quad , \quad r = s = 1 \tag{5.37e}$$

C_{k_z} is plotted as a function of s in Figure 5.8 for several values of r.

In the case of a vibrator with a circular base plate of diameter d, equation (5.30) takes the form

$$P = \frac{\pi}{4} \frac{(d + cz)^2}{dz^2} \quad E \cdot dz \cdot d\delta_z \quad . \tag{5.38}$$

Hence the expression for surface deformation is

$$\delta_o = \frac{4P}{\pi E'} \int_o^\infty \frac{dz}{(d + cz)^2 (h + z)} \quad . \tag{5.39}$$

Equation (5.39) reduces to the form of (5.34) for the case $r = 1$, except for the constant $\pi/4$. For this case therefore,

$$k_z = \frac{\pi}{4} E' d^2 C_{k_z} \tag{5.40}$$

where C_{k_z} has the same values as obtained from (5.37b) or (5.37e).

5.16 Spring Factor for Horizontal Displacement (k_x)

Consider an elemental cube of dimensions dz subjected to a shearing force dS causing a distortion du (Figure 5.9).

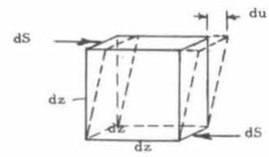


Figure 5.9

The shearing modulus of elasticity is

$$G = \frac{dS / dz^2}{du / dz} = \frac{dS}{du \cdot dz} \tag{5.41}$$

Recalling equation (5.27), it is seen that an expression for G may be written in the same form as (5.29), or

$$G = G'(h + z) \tag{5.42}$$

where $\quad G' = \frac{E'}{2(1 + \mu)} \quad . \quad$ (5.42a)

It is evident that the equation for the horizontal spring factor may be developed in the same manner as the expression for the vertical spring factor, or

$$k_x = G'b^2 C_{k_x} \tag{5.43}$$

where C_{k_x} is equal to the C_{k_z} of the previous section.

5.17 Spring Factor for Rotation about the Y-axis (K_{xz})

On the basis of the assumptions in section 5.14, the moment on any horizontal section of the effective zone is given by

$$M = 2 \int_o^{\frac{b + cz}{2}} x \cdot dP_{xz} \quad \frac{(a + cz)\,dx}{(dz)^2}$$

where $\quad dP_{xz} = E \cdot dz\,(x \cdot d\theta_z) \quad$ (cf. equation 5.28).

Substituting the expression for dP_{xz} we have

$$M = \left[\frac{2E\,(a + cz)}{dz} \int_o^{\frac{b + cz}{2}} x^2\,dx \right] d\theta_z$$

$$= \frac{E\,(a + cz)\,(b + cz)^3}{12\,dz} \; . \; d\theta_z \tag{5.44}$$

Solving for $d\theta_z$, and with E defined by equation (5.29),

$$d\theta_z = \frac{12M}{E'} \quad \frac{dz}{(a + cz)(b + cz)^3 (h + z)}$$

Hence the rotation of the contact surface is

$$\theta_o = \frac{12M}{E'} \int_o^\infty \frac{dz}{(a + cz)(b + cz)^3 (h + z)} \tag{5.45}$$

Recalling the definitions of r and s (equations 5.32 and 5.33), equation (5.45) may be written

$$\theta_o = \frac{12M}{E'b^4} \int_o^\infty \frac{dz'}{(r + z')(1 + z')^3 (s + z')} \tag{5.46}$$

where z' is again defined by (5.34a). The spring factor k_{xz} is defined

$$k_{xz} = M/\theta_o \quad . \tag{5.47}$$

Hence $\quad k_{xz} = E'b^4 C_{k_{xz}} \quad$ (5.48)

where

$$\frac{1}{C_{k_{xz}}} = 12 \int_o^\infty \frac{dz'}{(r + z')(1 + z')^3 (s + z')} \quad . \tag{5.48a}$$

Evaluation of integral (5.48a) yields the following results:

$$C_{k_{xz}} = \frac{1}{12}\left[\frac{(r-s)(s-1)(r-1)}{\frac{(r-1)}{(s-1)^2}\log s - \frac{(s-1)}{(r-1)^2}\log r + (r-s)\left(\frac{1}{2} - \frac{1}{r-1} - \frac{1}{s-1}\right)}\right], \qquad r \neq s \neq 1 \quad (5.49a)$$

$$C_{k_{xz}} = \frac{1}{12}\left[\frac{(s-1)^3}{1 - \frac{s-1}{2} + \frac{(s-1)^2}{3} - \frac{\log s}{s-1}}\right], \qquad r = 1,\ s \neq 1 \quad (5.49b)$$

$$C_{k_{xz}} = \frac{1}{12}\left[\frac{(r-1)^3}{1 - \frac{r-1}{2} + \frac{(r-1)^2}{3} - \frac{\log r}{r-1}}\right], \qquad s = 1,\ r \neq 1 \quad (5.49c)$$

$$C_{k_{xz}} = \frac{1}{12}\left[\frac{(r-1)^3}{\frac{3\log r}{r-1} - \frac{1}{r} - 2 + \frac{r-1}{2}}\right], \qquad r = s \neq 1 \quad (5.49d)$$

$$C_{k_x} = \frac{1}{3}, \qquad r = s = 1 \quad (5.49e)$$

$C_{k_{xz}}$ is plotted as a function of s in Figure 5.10 for several values of r.

PART IV

APPARENT MASS

5.18 Introduction

The inertia parameters of the vibrator-soil system must be determined before the theory of Part II can be applied. These parameters consist of the mass and mass moments of inertia of the system. The system may be considered analogous to a vibrator of mass m_v oscillating on damped springs of mass m_s. However, for the purposes of analysis it is convenient to replace the springs with weightless springs and to add an apparent mass m' to the mass of the vibrator. Thus for the vertical mode, the frequency equation

$$\omega_n = \sqrt{k/m}$$

may be written

$$\omega_n = \sqrt{\frac{k}{m_v + m'}} \qquad (5.50)$$

5.19 Apparent Soil Mass (m')

An estimate of the apparent or equivalent mass acting at the contact surface may be made by equating the kinetic energy of the apparent mass to the total kinetic energy of the soil in the effective zone. The kinetic energy of the apparent mass is

$$T = \tfrac{1}{2} m' \dot{\delta}_o^2 = \tfrac{1}{2} m\,\omega^2 \delta_o^2 \qquad (5.51)$$

and the kinetic energy of the soil in the effective zone is

$$T_s = \frac{\rho}{2g}\int_o^\infty A_z \dot{\delta}_z^2\,dz = \frac{\rho\omega^2}{2g}\int_o^\infty A_z \delta_z^2\,dz \qquad (5.52)$$

where A_z, the area of a horizontal section of the effective zone, is

$$A_z = (a + cz)(b + cz) \qquad (5.53)$$

and δ_z, the vertical displacement of the section, may be obtained from equation (5.31) by changing the lower limit to z. Thus

$$\delta_z = \frac{P}{E'}\int_z^\infty \frac{dz}{(a + cz)(b + cz)(h + z)} \quad . \qquad (5.54)$$

Equating (5.51) to (5.52) and solving for m we have

$$m' = \frac{\rho}{\delta_o^2\, g}\int_o^\infty A_z\,\delta_z^2\,dz \quad . \qquad (5.55)$$

Substituting (5.31), (5.53), and (5.54) in (5.55) and with

$$r = a/b\ , \quad a \geq b$$
$$s = ch/b$$
$$z' = cz/b$$

we obtain

$$m' = \frac{\rho b^3}{g\ c}\,C_m \qquad (5.56)$$

$$C_m = \frac{\int_o^\infty (r+z')(1+z')\left[\int_{z'}^\infty \frac{dz'}{(r+z')(1+z')(s+z')}\right]^2 \cdot dz'}{\left[\int_o^\infty \frac{dz'}{(r+z')(1+z')(s+z')}\right]^2} = C_{k_z}^2\int_o^\infty (r+z')(1+z')\left[\int_{z'}^\infty \frac{dz'}{(r+z')(1+z')(s+z')}\right]^2 dz' \qquad (5.56a)$$

and where C_{k_z} is defined by equation (5.37). For the special case $r = s = 1$, equation (5.56a) reduces to

$$C_m = \int_o^\infty \frac{dz'}{(1+z')^2} = 1 \quad .\ . \qquad (5.56b)$$

The coefficient C_m has been evaluated by numerical integration for several values of r and s and plotted in Figure 5.11.

For a circular base

$$A_z = \frac{\pi}{4}(d + cz)^2$$

therefore

$$m = \frac{\pi\rho\, d^3}{4\ g\ c}\,C_m \qquad (5.57)$$

where C_m is the coefficient for $r = 1$.

5.20 Apparent Mass Moment of Inertia

The apparent mass moment of inertia may be estimated in a similar manner. For rotation about the y-axis in the contact surface the kinetic energy of the apparent mass is

$$T = \tfrac{1}{2} i' \dot{\theta}_o^2 = \tfrac{1}{2} i' \omega^2 \theta_o^2 \qquad (5.58)$$

and the kinetic energy of the soil in the effective zone is

$$T_s = \frac{\rho}{2g}\int_o^\infty I_z \dot{\theta}_z^2\,dz = \frac{\rho\omega^2}{2g}\int_o^\infty I_z \theta_z^2\,dz \qquad (5.59)$$

where I_z, the moment of inertia of a horizontal section of the effective zone, is

$$I_z = \frac{(a + cz)(b + cz)^3}{12} \qquad (5.60)$$

and θ_z, the rotation of the section may be obtained from equation (5.45) by changing the lower limit to z. Thus

$$\theta_z = \frac{12\,M}{E'}\int_z^\infty \frac{dz}{(a + cz)(b + cz)^3(h + z)} \quad . \qquad (5.61)$$

Equating (5.58) to (5.59) and solving for i' we have

$$i' = \frac{\rho}{g\ \theta_o^2}\int_o^\infty I_z\,\theta_z^2\,dz \quad . \qquad (5.62)$$

Substituting (5.45), (5.60), and (5.61) in (5.62) and with r, s, and z' defined as in Section 5.19, we obtain

$$i' = \frac{\rho\ b^5}{12\ g\ c}\,C_i \qquad (5.63)$$

where

$$C_i = \frac{\int_o^\infty (r+z')(1+z')^3\left[\int_{z'}^\infty \frac{dz'}{(r+z')(1+z')^3(s+z')}\right]^2 dz'}{\left[\int_o^\infty \frac{dz'}{(r+z')(1+z')^3(s+z')}\right]^2} = C_{k_{xz}}^2\int_o^\infty (r+z')(1+z')^3\left[\int_{z'}^\infty \frac{dz'}{(r+z')(1+z')^3(s+z')}\right]^2 dz' \qquad (5.63a)$$

where $C_{k_{xz}}$ is defined by equation (5.49). For the special case $r = s = 1$, equation (5.63a) reduces to

$$C_i = \int_o^\infty \frac{dz'}{(1+z')^4} = \frac{1}{3} \quad . \qquad (5.63b)$$

The coefficient C_i has been evaluated by numerical integration for several values of r and s and plotted in Figure 5.12.

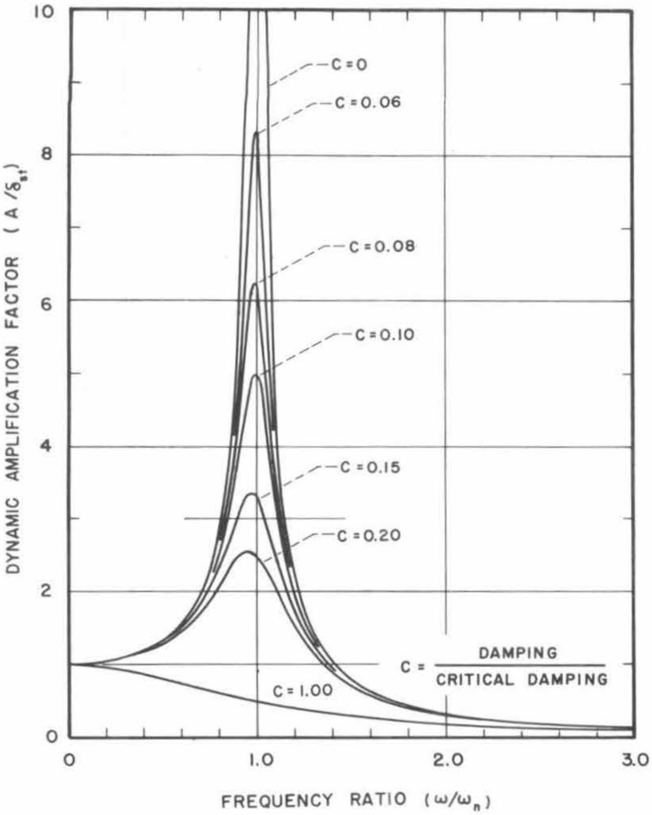


FIGURE 5.2

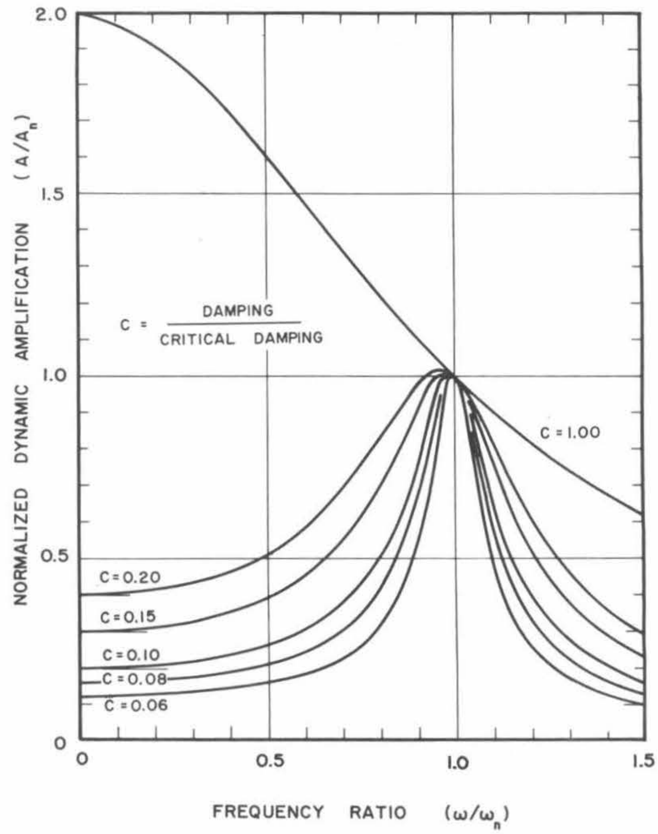


FIGURE 5.3

AVERAGE POWER DISSIPATED IN DAMPING

$P_{d_{ave.}} = \Lambda k \omega r^2$

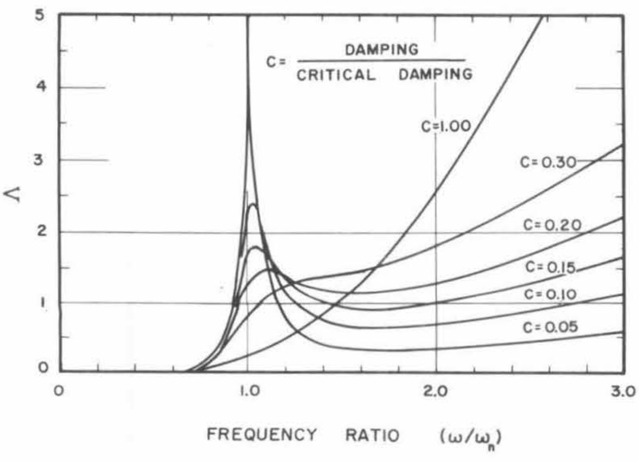
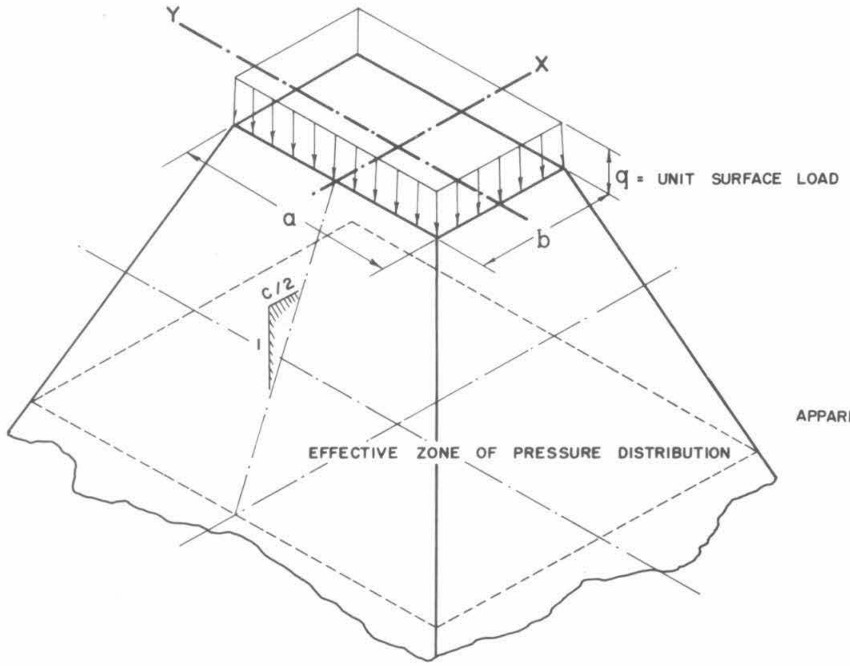


FIGURE 5.4

APPARENT SOIL MASS



APPARENT SOIL MASS

$$m' = \frac{\rho b^3}{g c} C_m$$

APPARENT MASS MOMENT OF INERTIA ABOUT Y-AXIS

$$i' = \frac{\rho b^5}{12 g c} C_i$$

g = ACCELLERATION DUE TO GRAVITY

$r = a/b$ $\quad a \geq b$

$h = q/\rho$ $\quad \rho$ = UNIT WEIGHT OF SOIL

$s = ch/b$

APPARENT MASS AND MASS MOMENT OF INERTIA COEFFICENTS

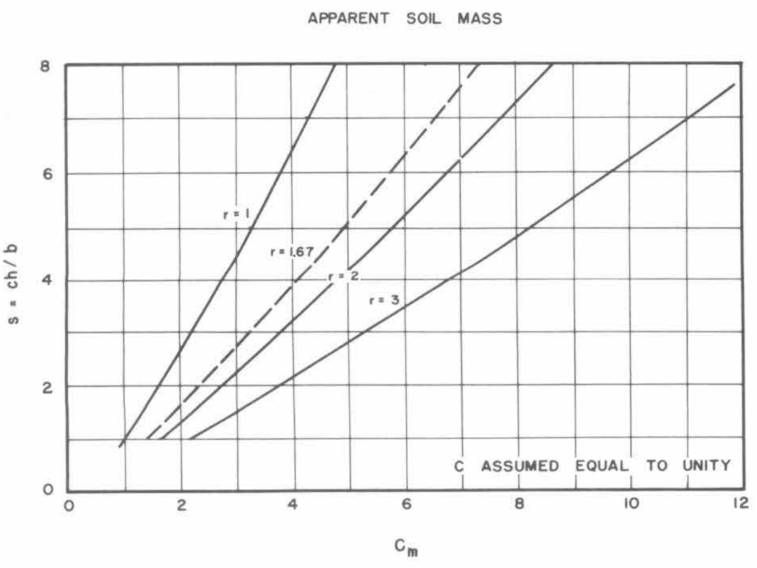


FIGURE 5.11

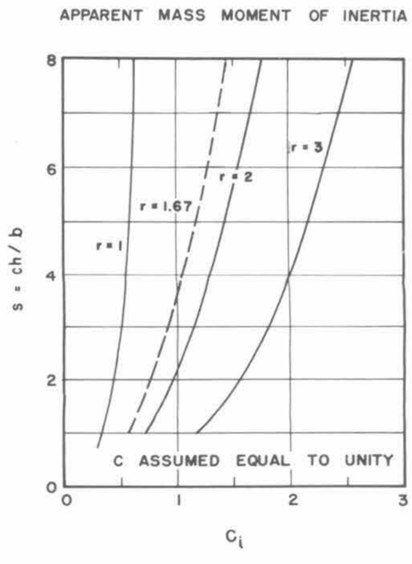


FIGURE 5.12

APPENDIX B

GENERAL THEORY
FOR THE
DETERMINATION OF THE NATURAL FREQUENCY
OF A
VIBRATOR-SOIL SYSTEM

The prediction of the natural frequency of vibration of a vibrator-soil system requires the computation of the soil spring-factor and the apparent mass of the system. The procedure for doing this is reproduced in Appendix A. However, due to the limitation imposed by one of the basic assumptions, this procedure is applicable only to a system including non-cohesive soil such as sand, and requires slight modification if it is to apply to all vibrator-soil systems.

B.1 Elastic Modulus of Soils

The theories of soil spring-factors and apparent soil mass presented in Parts III and IV of Appendix A are based on several simplifying assumptions, one of which is an approximate relationship between the modulus of elasticity of the soil, E, and the depth, z. The assumption made is that:

$$E = E\,(h+z) \qquad \text{.......(5.29)}$$

where $E = \frac{dE}{dz}$ = a constant for a given soil

and h = an equivalent soil surcharge height due to the weight of the vibrator.

Equation (5.29) applies only to a non-cohesive soil since at the surface of an unloaded non-cohesive soil E is zero, a value obtained by substituting $h = z = o$ in the equation.

Due to the cohesiveness of silty and clayey soils, the elastic modulii of such materials have finite values at the soil surface when the soils are not under. That is, E has a finite value even when z is zero, showing that h for a cohesive soil is finite also. Hence, the equivalent surcharge, h, must be considered composed of two parts, h_v and h_s, so that

$$h = h_v + h_s \qquad \text{.......(B.1)}$$

where $h_v = \frac{q}{\rho}$ an equivalent soil surcharge due to the weight of the vibrator,

q = unit surface load on the soil,

ρ = unit weight of the soil,

and h_s = an equivalent soil surcharge due to the cohesiveness of the soil.

Substituting equation (B.1) in equation (5.29), the general expression for the modulus of elasticity of any soil becomes

$$E = E'(h_v + h_s + z) \qquad \text{.......(5.29a)}$$

In the case of sand there is no cohesion and h_s is zero. Equation (5.29a) then becomes

$$E = E'(h_v + z)$$

which is identical to (5.29)

With the exception of a cohesionless soil, in which $h = h_v$ = a known quantity, the "equivalent surcharge" h_s must be evaluated before the theory in Parts III and IV of Appendix A can be applied.

B.2 Experimental Determination of Soil Constants E' and h_s.

Reference is made to Appendix A and to the sample calculations shown in Chapter 6 of the January 1952 report for the method of calculating the following:

Natural frequency:	ω_n
Spring factor:	k_z
Total mass:	m
Apparent soil mass:	m'

The calculations for E' and h_s are then based on the values of the above terms.

(1) $$m' = \frac{\rho \cdot b^3}{g \cdot c} C_m$$

whence $$C_m = \frac{m' g}{\rho b^3} \quad \text{for } c = 1$$

(2) From Figure 5.11, determine "S" using the calculated value of C_m.

(3) $$k_z = E' b^2 C_{k_2}$$

whence $$E' = \frac{k_2}{b^2 C_{k_2}}$$

(4) $$S = \frac{ch}{b} = \frac{c(h_s + h_v)}{b}$$

whence $$\underline{h_s = bs - h_v} \quad \text{for } c = 1$$

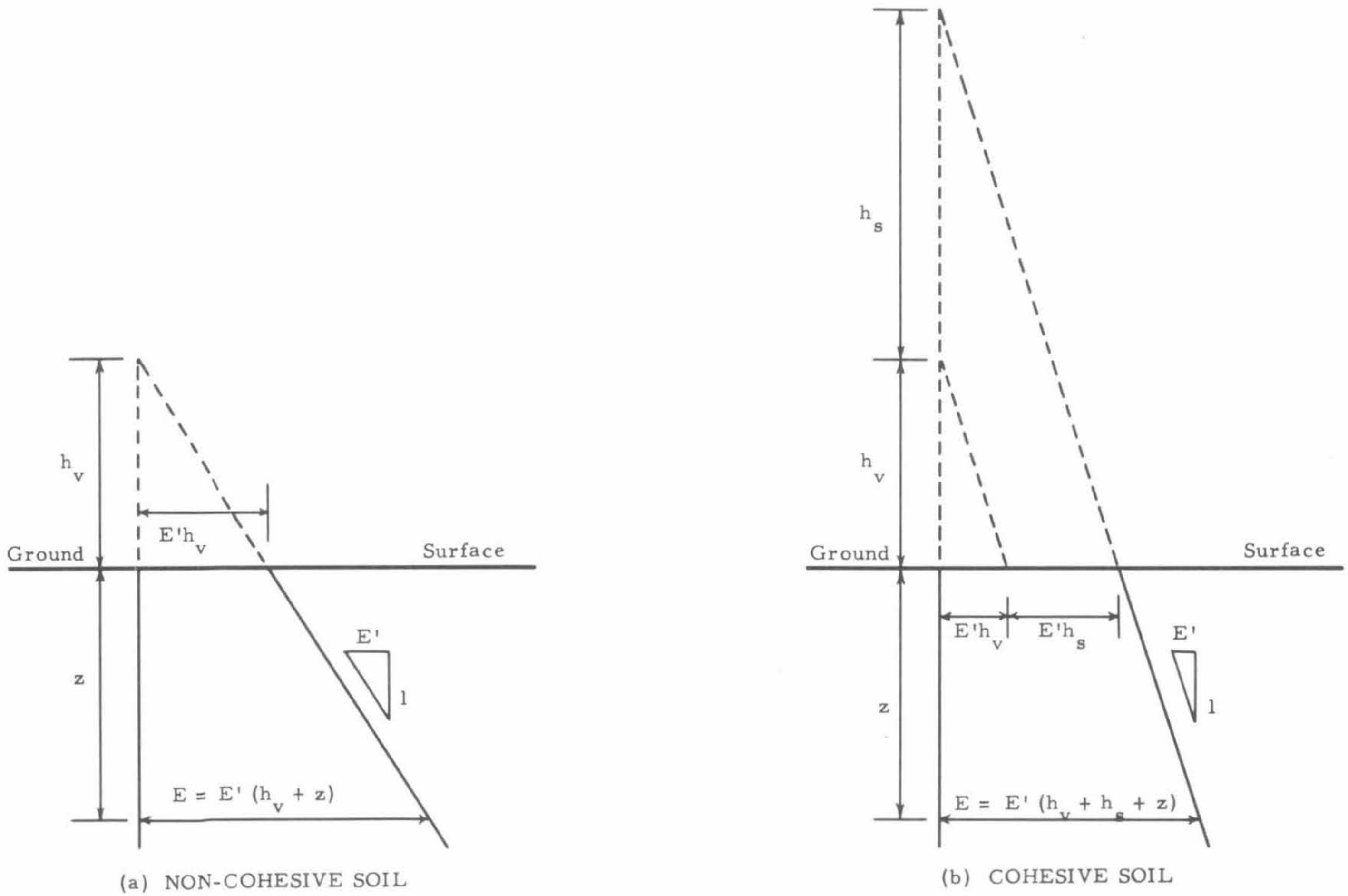


Fig. B.1 VARIATION IN MODULUS OF ELASTICITY OF SOIL WITH DEPTH BELOW GROUND SURFACE.

APPENDIX C

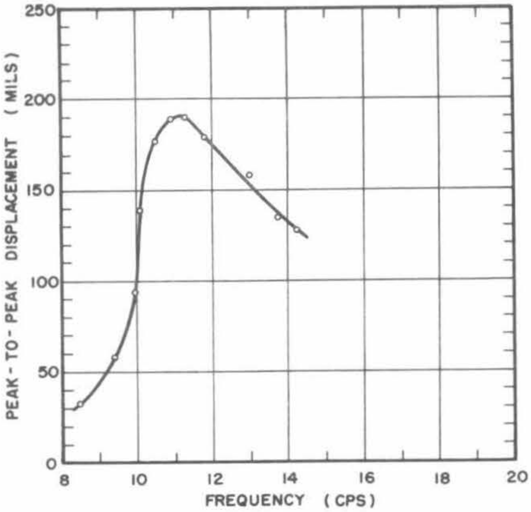
DATA FROM PRELIMINARY FIELD TESTS

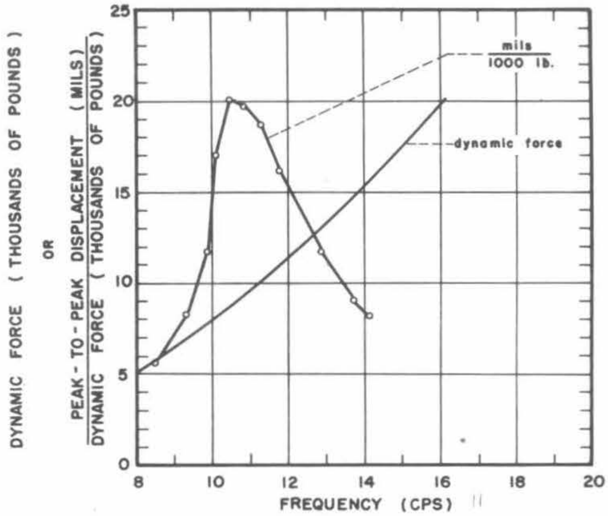
AT

PORT HUENEME, CALIFORNIA

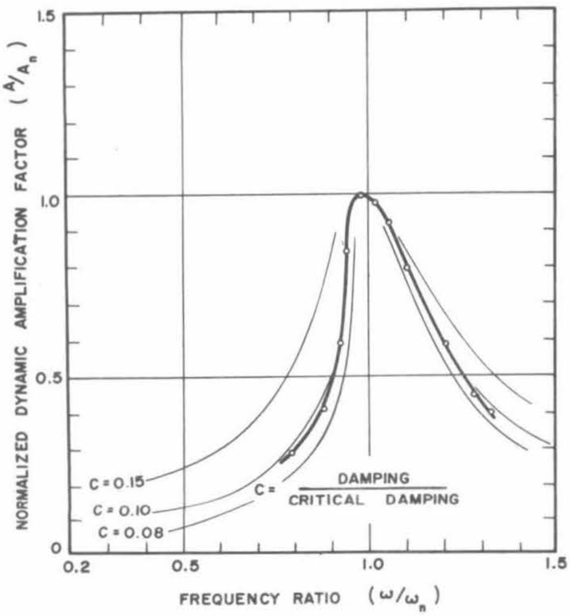
RUN NO. 12

VERTICAL DISPLACEMENT DATA

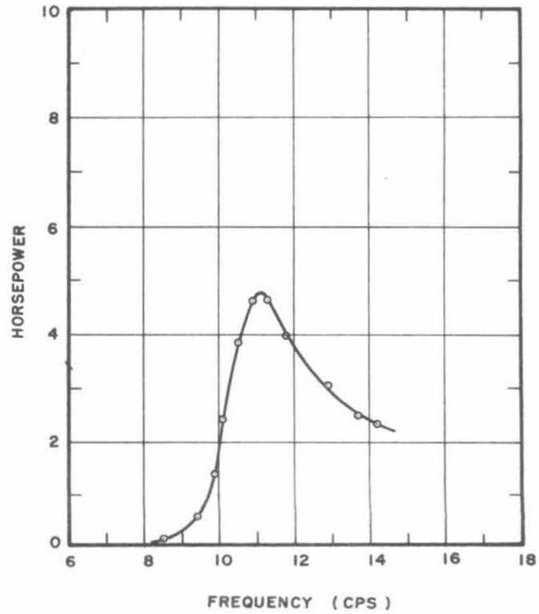




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



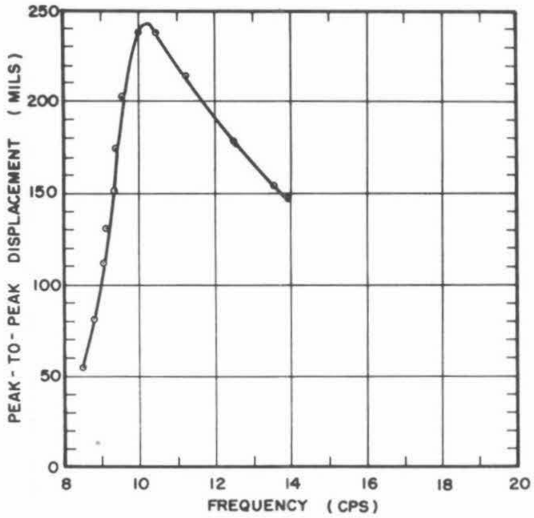
NOTES

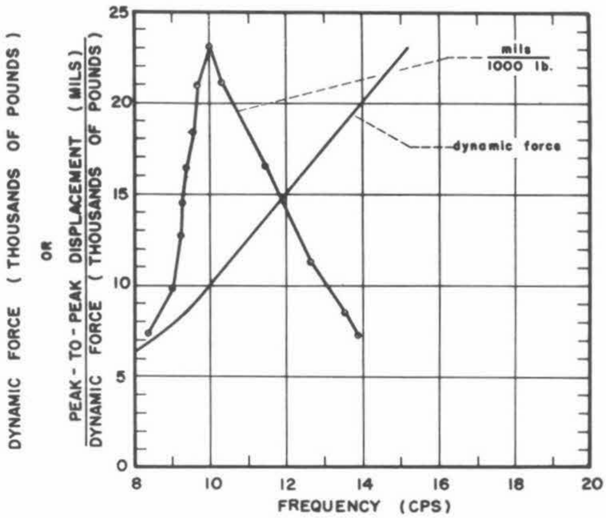
VIBRATOR DEAD LOAD = 13,600 POUNDS

SEVEN ECCENTRICS PER SHAFT

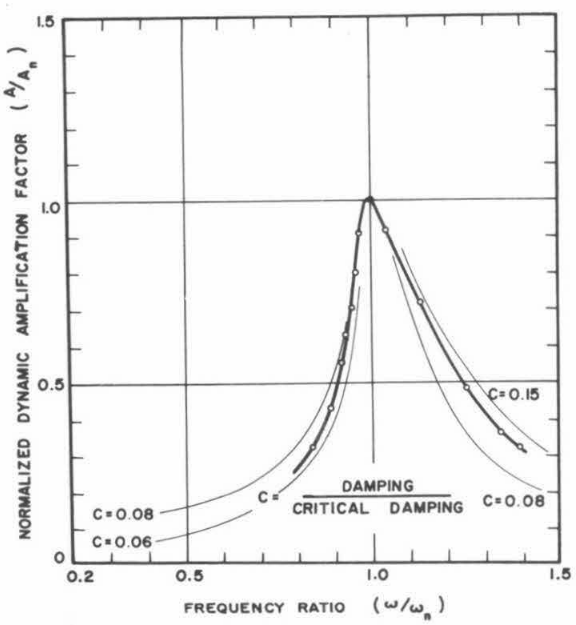
RUN NO. 13

VERTICAL DISPLACEMENT DATA

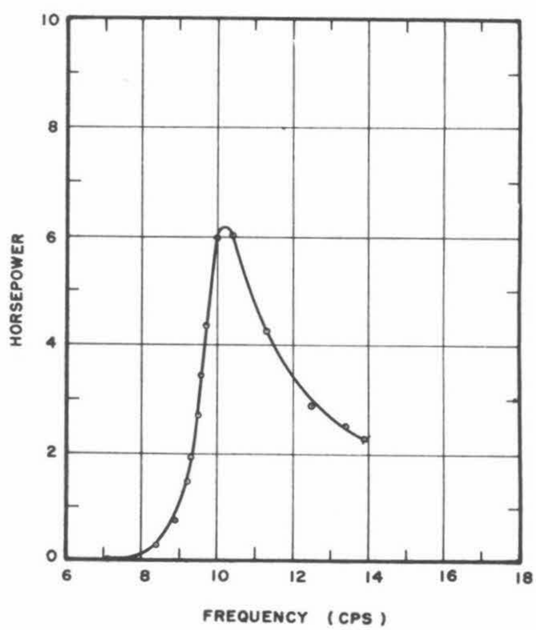




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



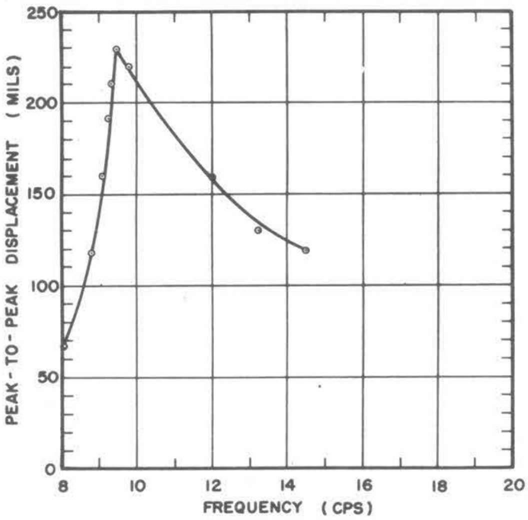
NOTES

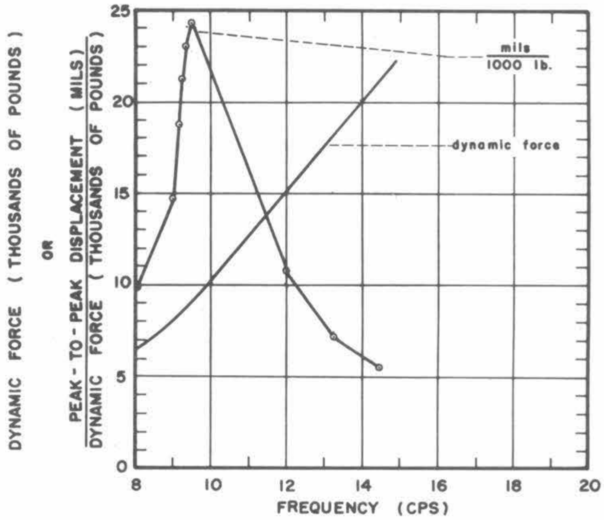
VIBRATOR DEAD LOAD = 15,600 POUNDS

NINE ECCENTRICS PER SHAFT

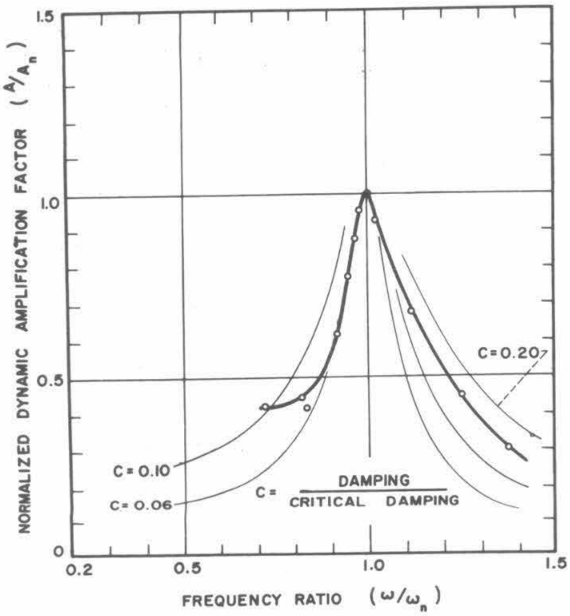
RUN NO. 21

VERTICAL DISPLACEMENT DATA

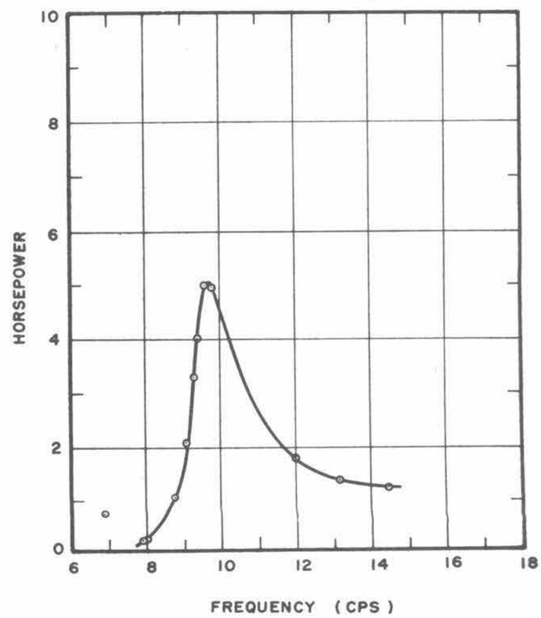




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



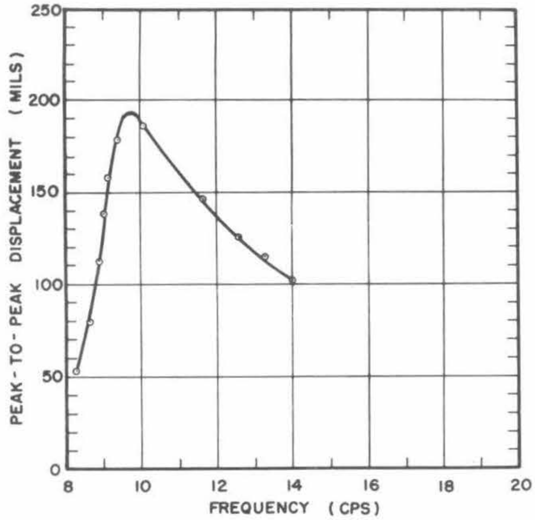
NOTES

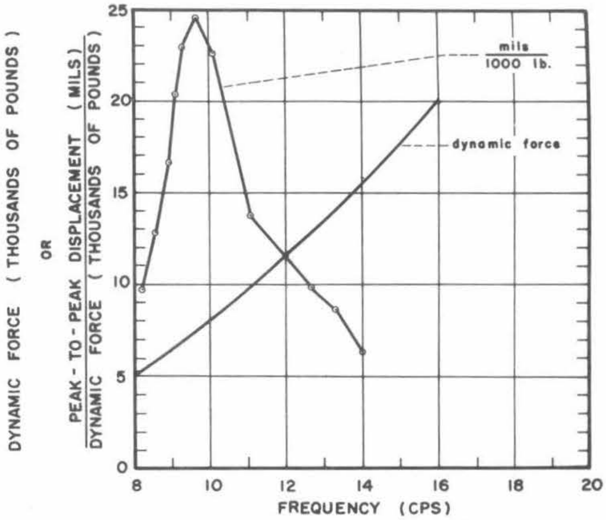
VIBRATOR DEAD LOAD = 16,800 POUNDS

NINE ECCENTRICS PER SHAFT

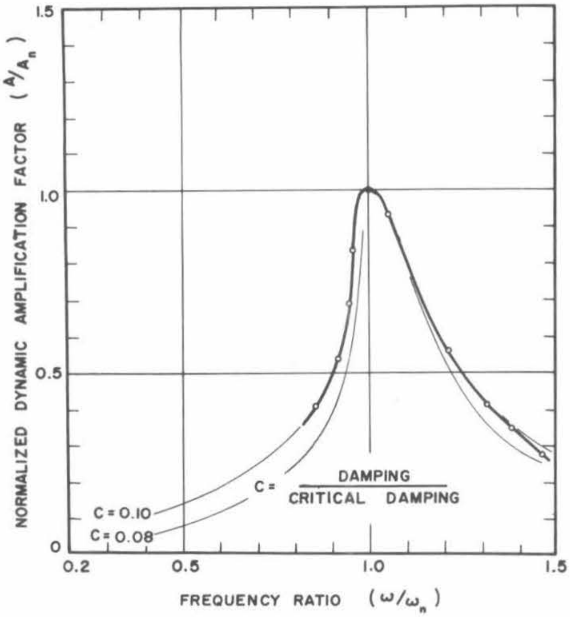
RUN NO. 22

VERTICAL DISPLACEMENT DATA

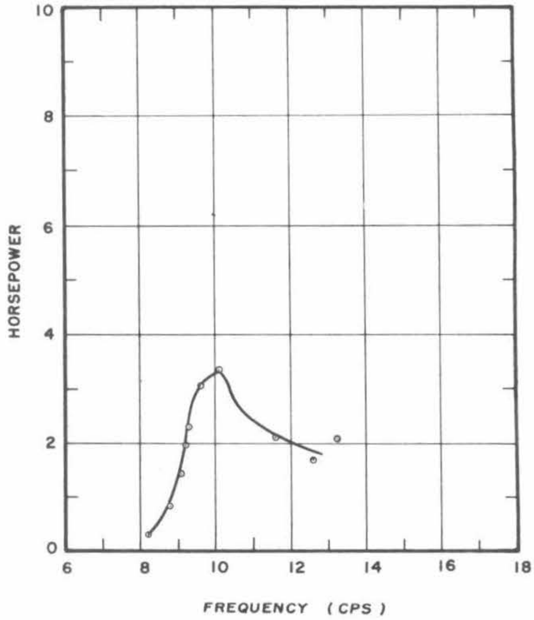




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



NOTES

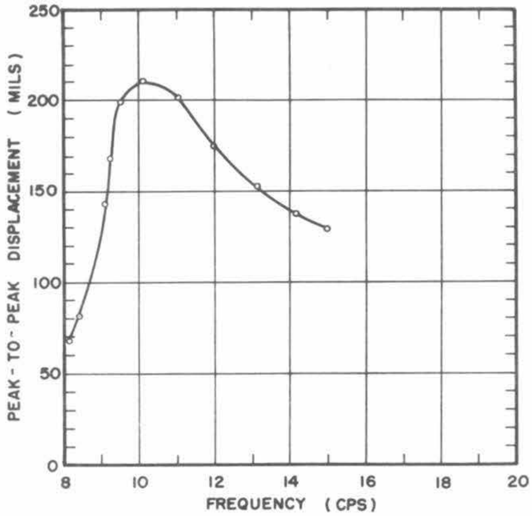
VIBRATOR DEAD LOAD = 16,800 POUNDS

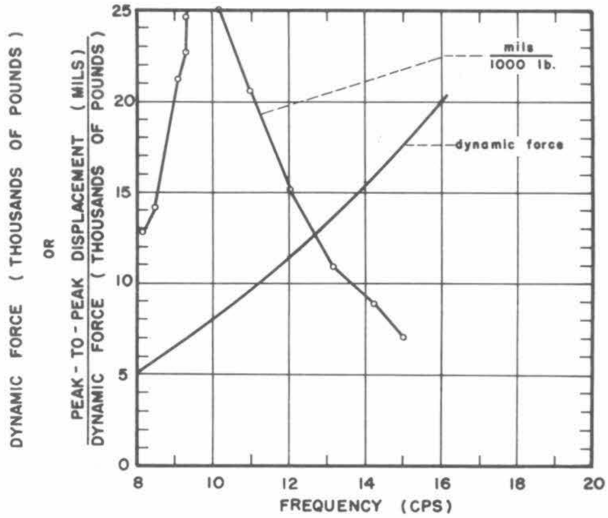
SEVEN ECCENTRICS PER SHAFT

PLATE 10

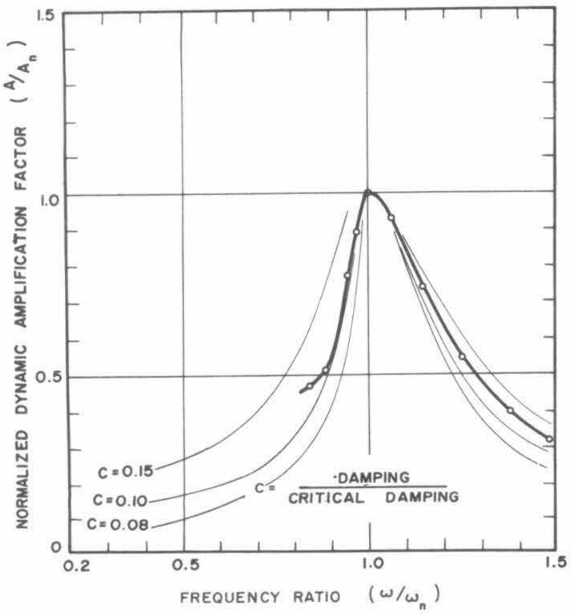
RUN NO. 23

VERTICAL DISPLACEMENT DATA

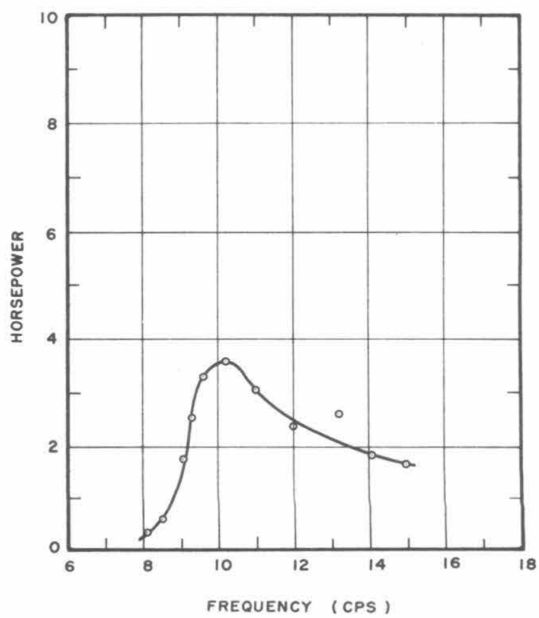




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



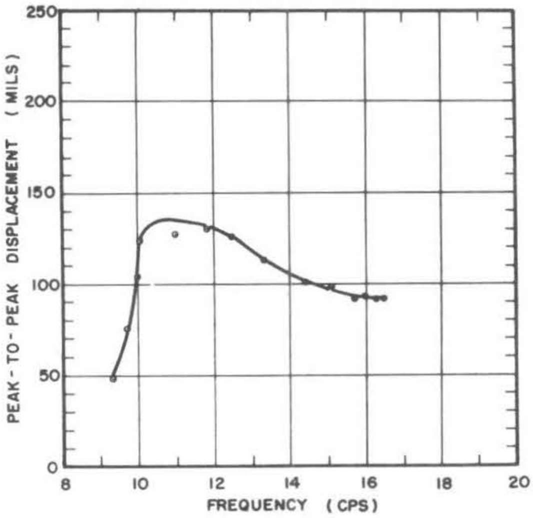
NOTES

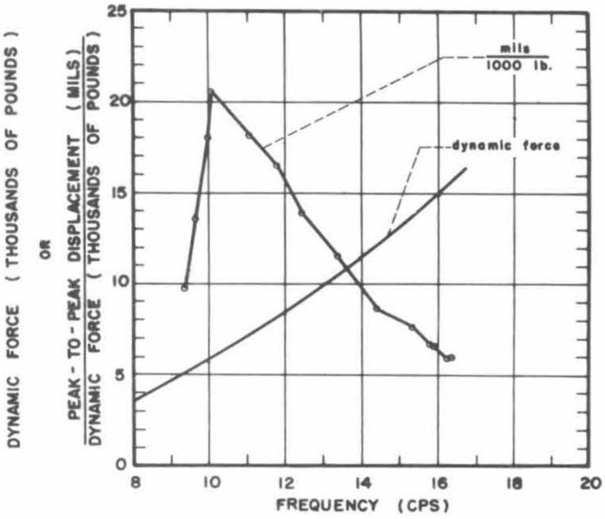
VIBRATOR DEAD LOAD = 12,800 POUNDS

SEVEN ECCENTRICS PER SHAFT

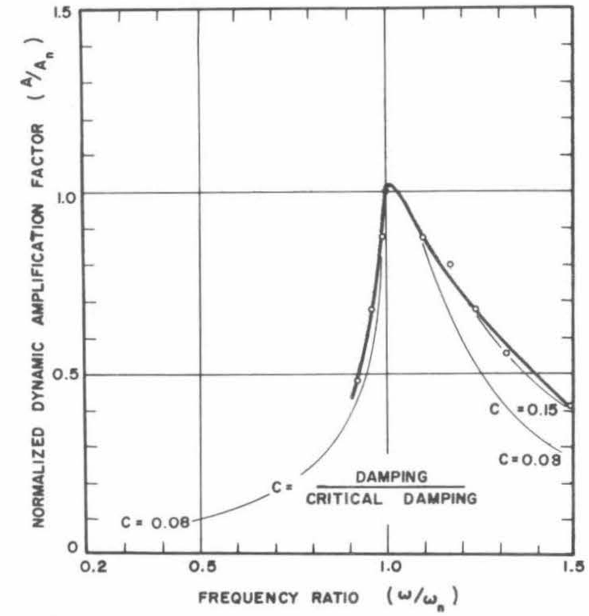
RUN NO. 24

VERTICAL DISPLACEMENT DATA

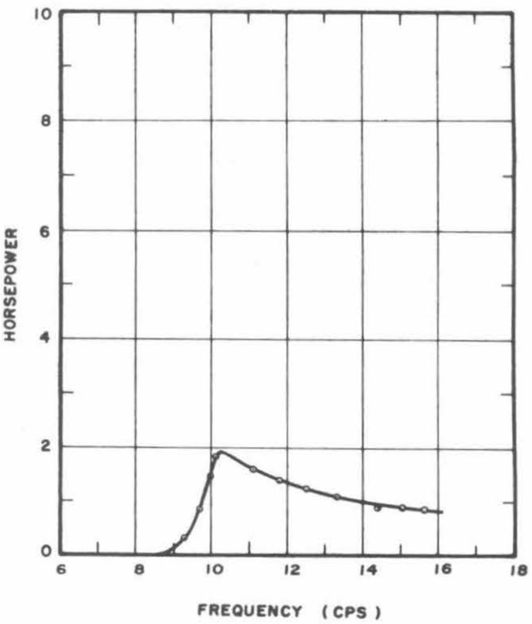




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



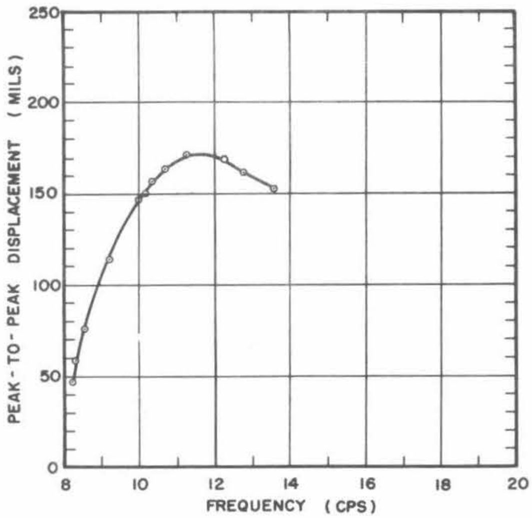
NOTES

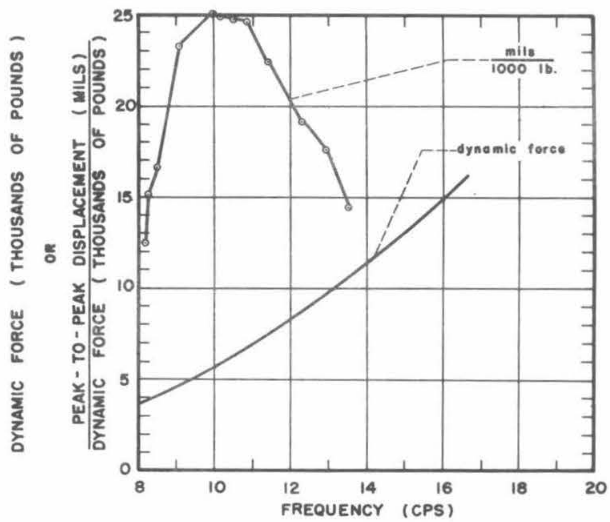
VIBRATOR DEAD LOAD = 12,800 POUNDS

FIVE ECCENTRICS PER SHAFT

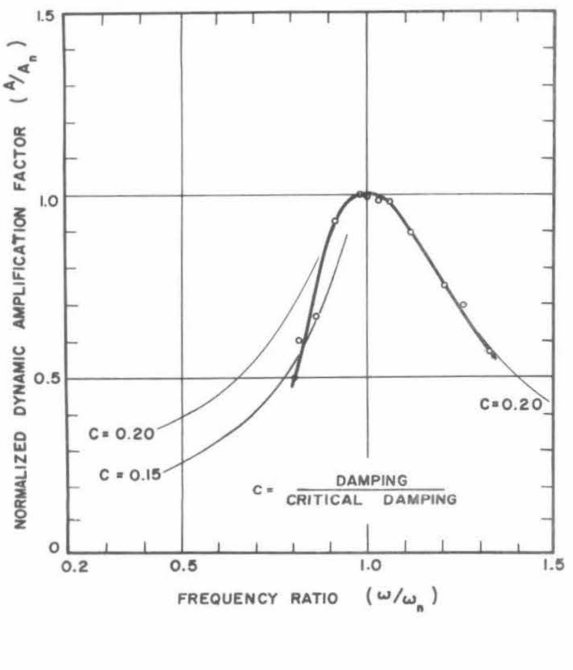
RUN NO. 25

VERTICAL DISPLACEMENT DATA

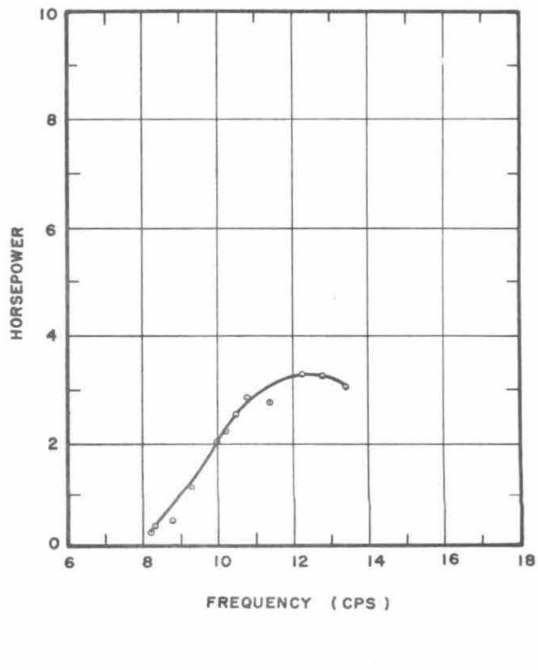




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



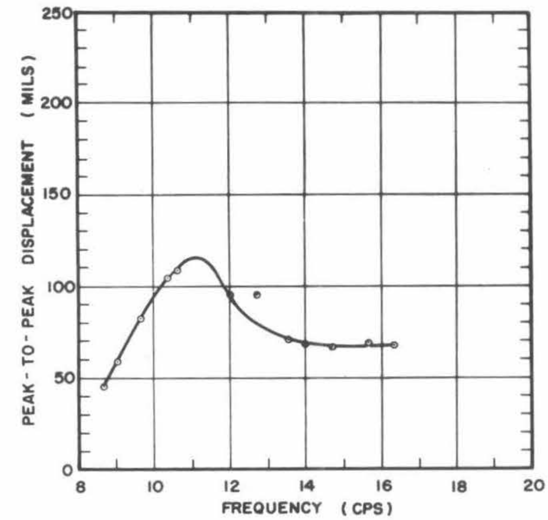
NOTES

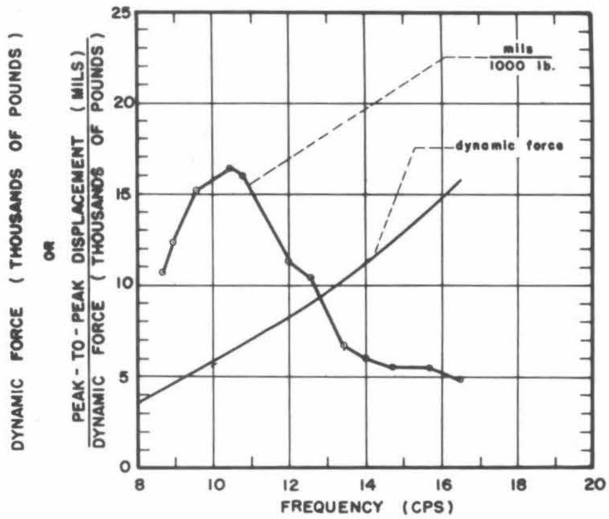
VIBRATOR DEAD LOAD = 9,600 POUNDS

FIVE ECCENTRICS PER SHAFT

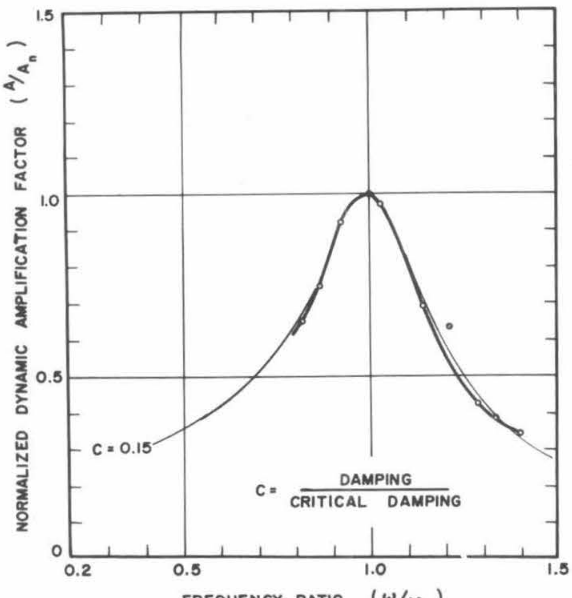
RUN NO. 26

VERTICAL DISPLACEMENT DATA

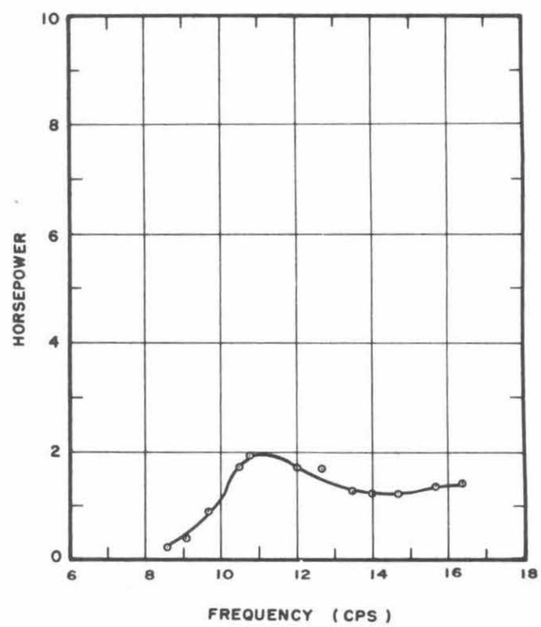




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



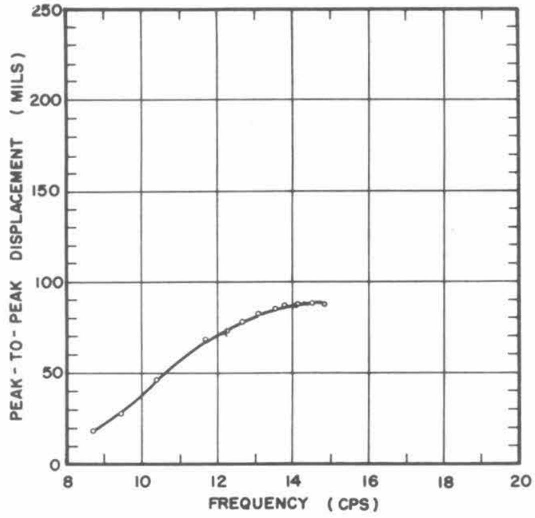
NOTES

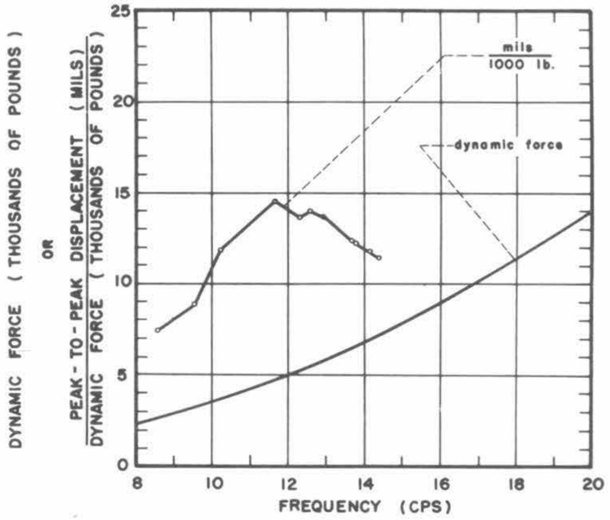
VIBRATOR DEAD LOAD = 16,800 POUNDS

FIVE ECCENTRICS PER SHAFT

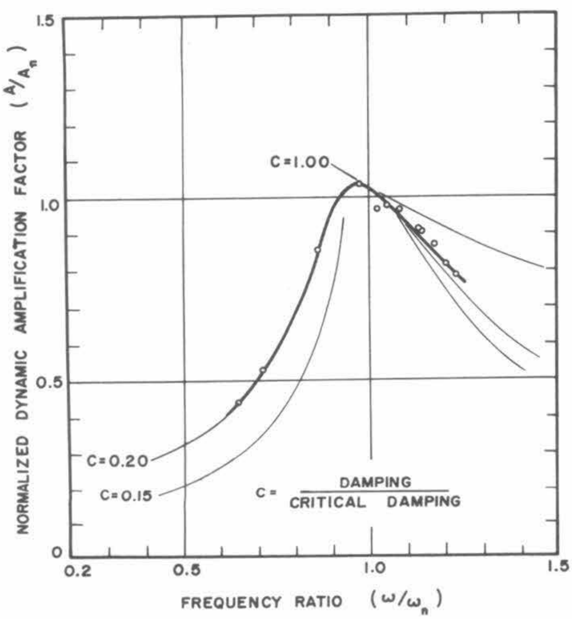
RUN NO. 31

VERTICAL DISPLACEMENT DATA

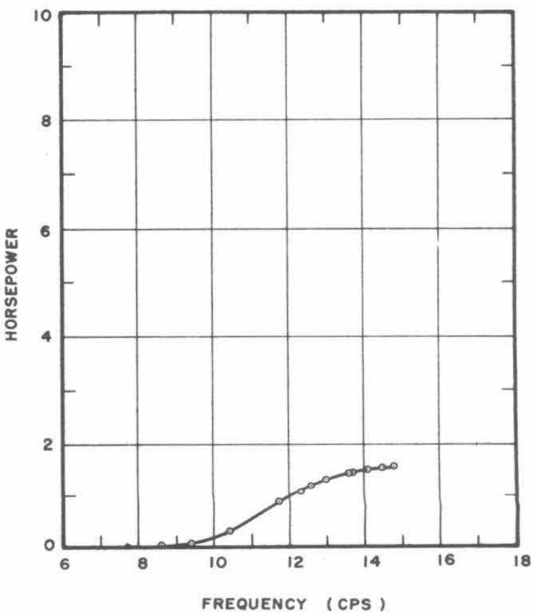




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



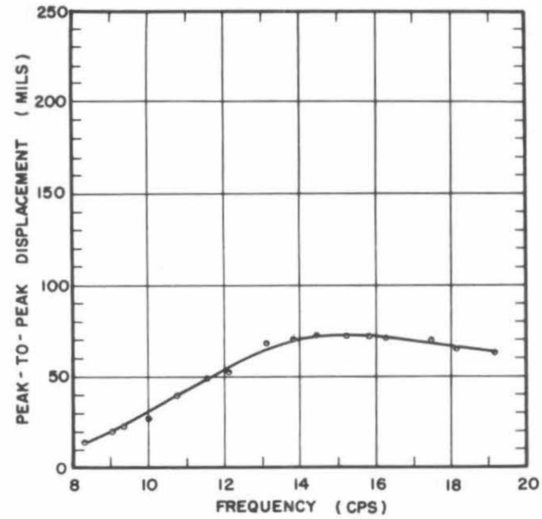
NOTES

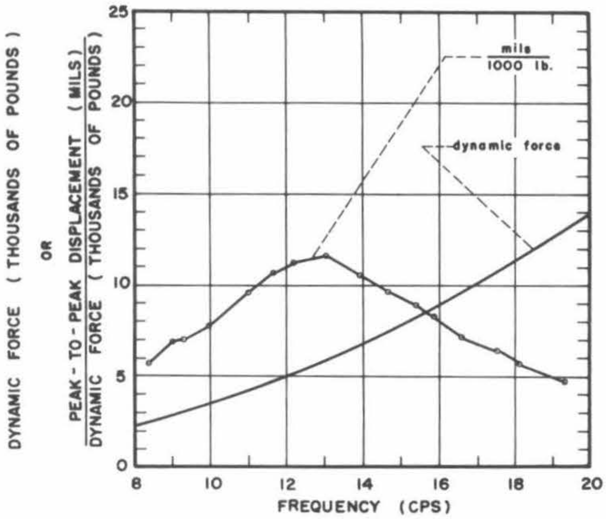
VIBRATOR DEAD LOAD = 12,800 POUNDS

THREE ECCENTRICS PER SHAFT

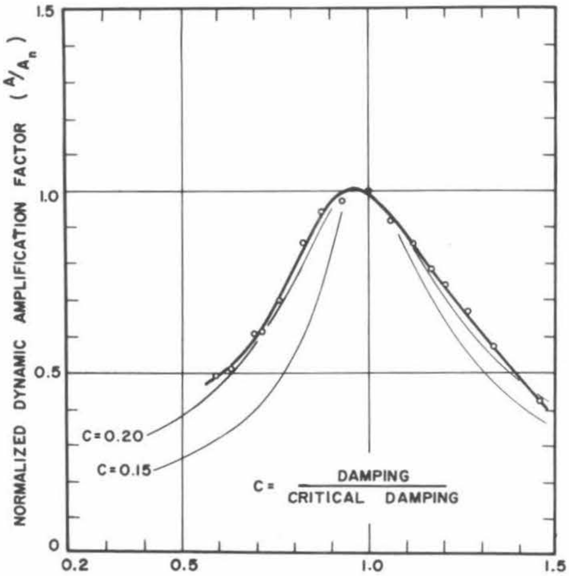
RUN NO. 32

VERTICAL DISPLACEMENT DATA

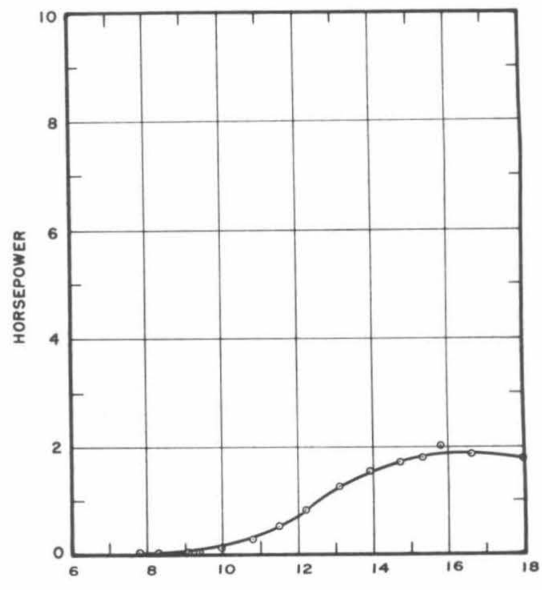




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



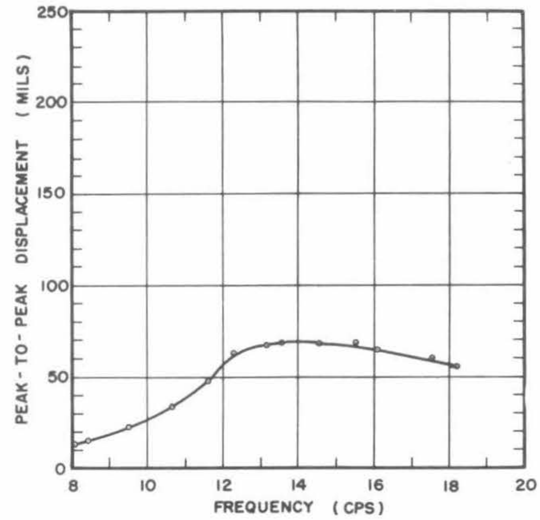
NOTES

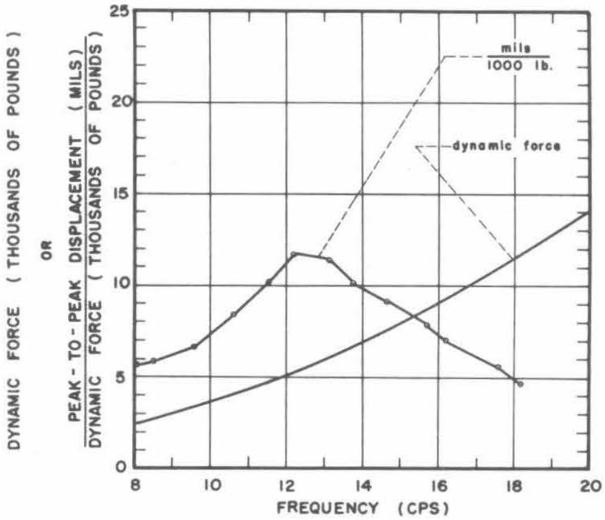
VIBRATOR DEAD LOAD = 14,800 POUNDS

THREE ECCENTRICS PER SHAFT

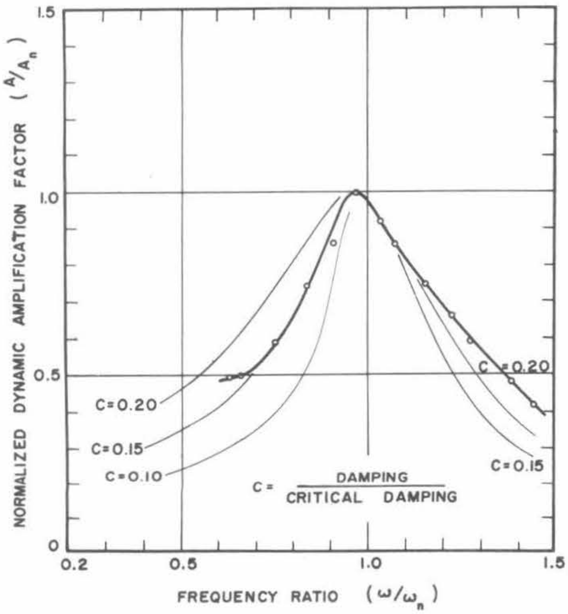
RUN NO. 33

VERTICAL DISPLACEMENT DATA

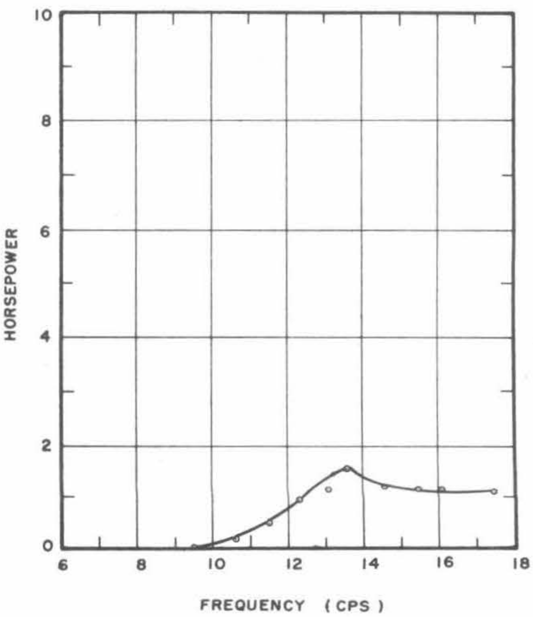




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



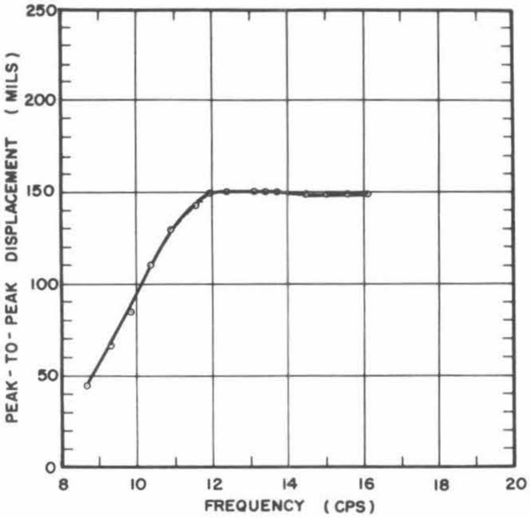
NOTES

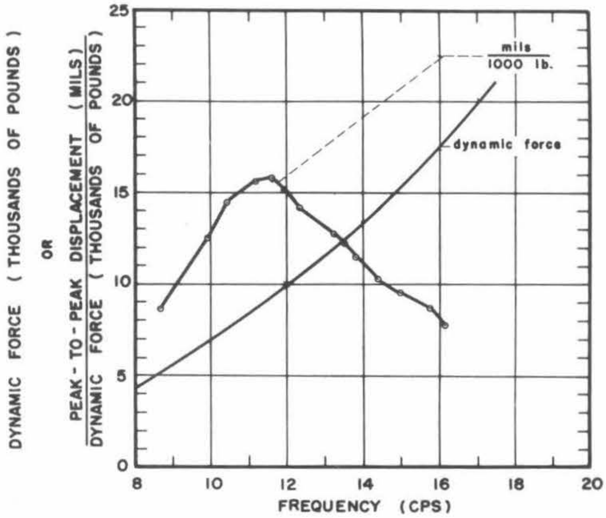
VIBRATOR DEAD LOAD = 16,800 POUNDS

THREE ECCENTRICS PER SHAFT

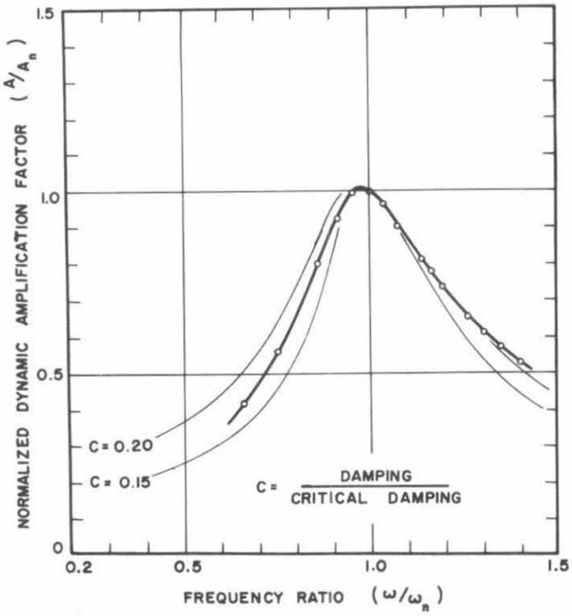
RUN NO. 34

VERTICAL DISPLACEMENT DATA

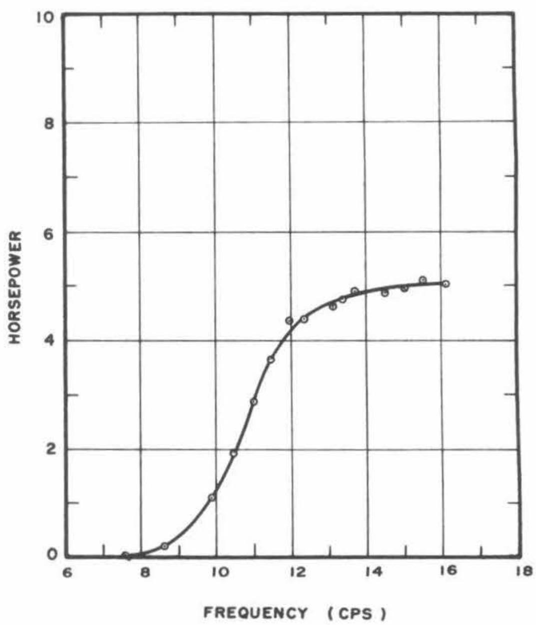




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

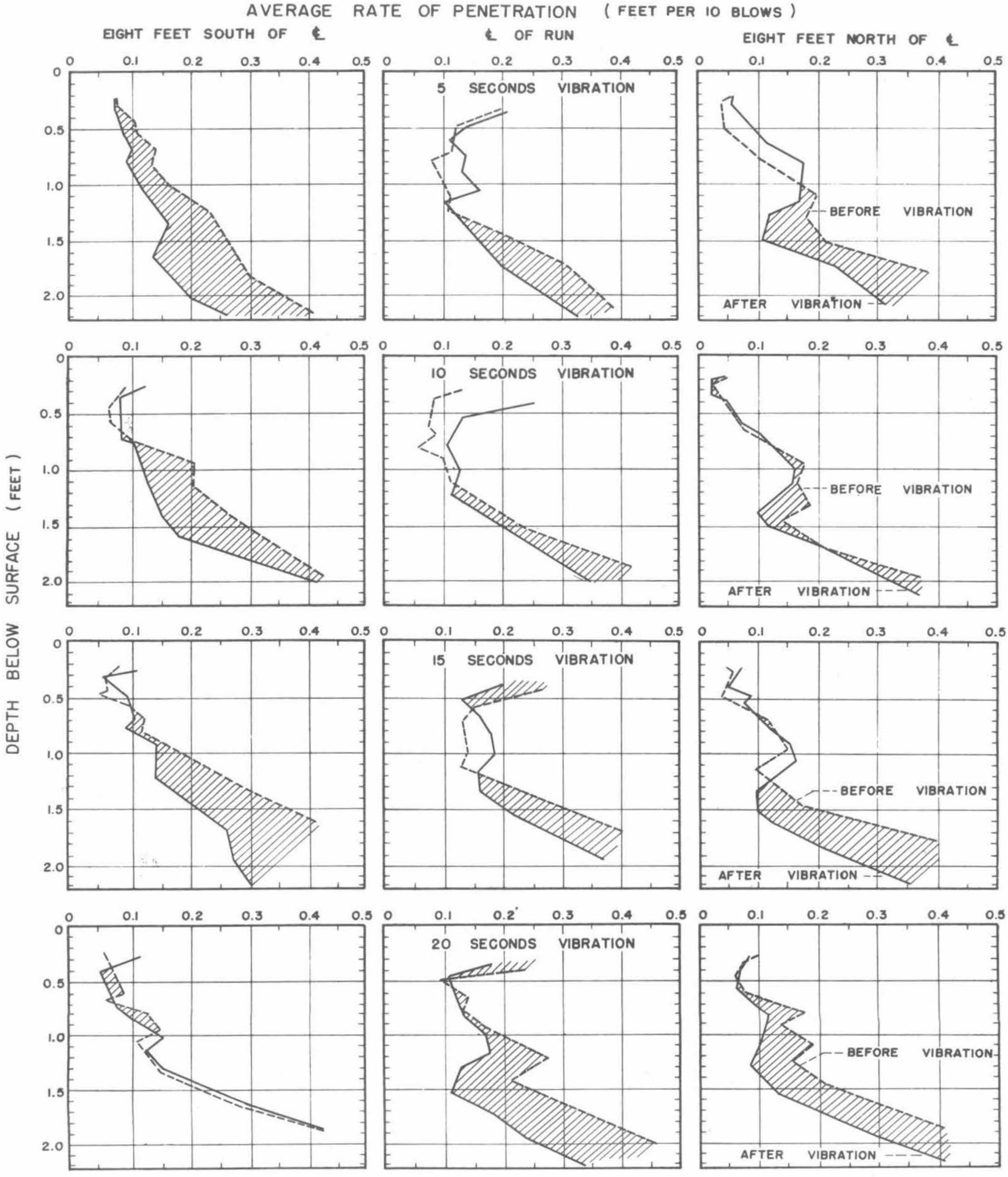


NOTES

VIBRATOR DEAD LOAD = 12,800 POUNDS

SIX ECCENTRICS PER SHAFT

COMPACTION TEST NO. 134



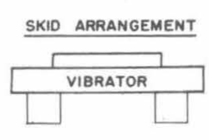
DEAD LOAD 12,800 LB.

DYNAMIC FORCE 10,000 LB.

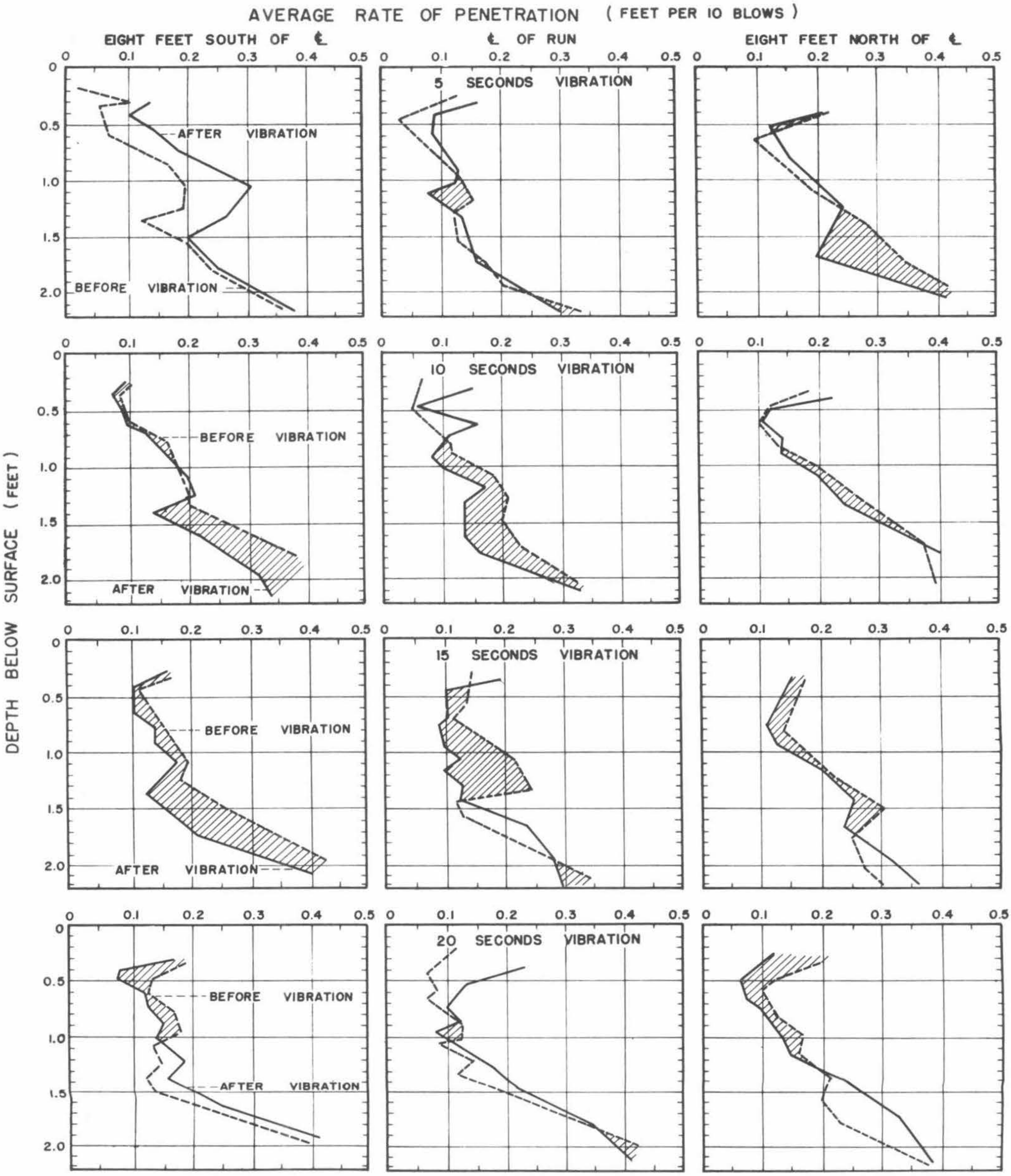
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

FREQUENCY 12.0 CPS.

ECCENTRICS 6 PER SHAFT

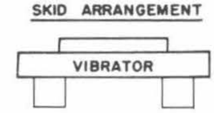


COMPACTION TEST NO. 134A

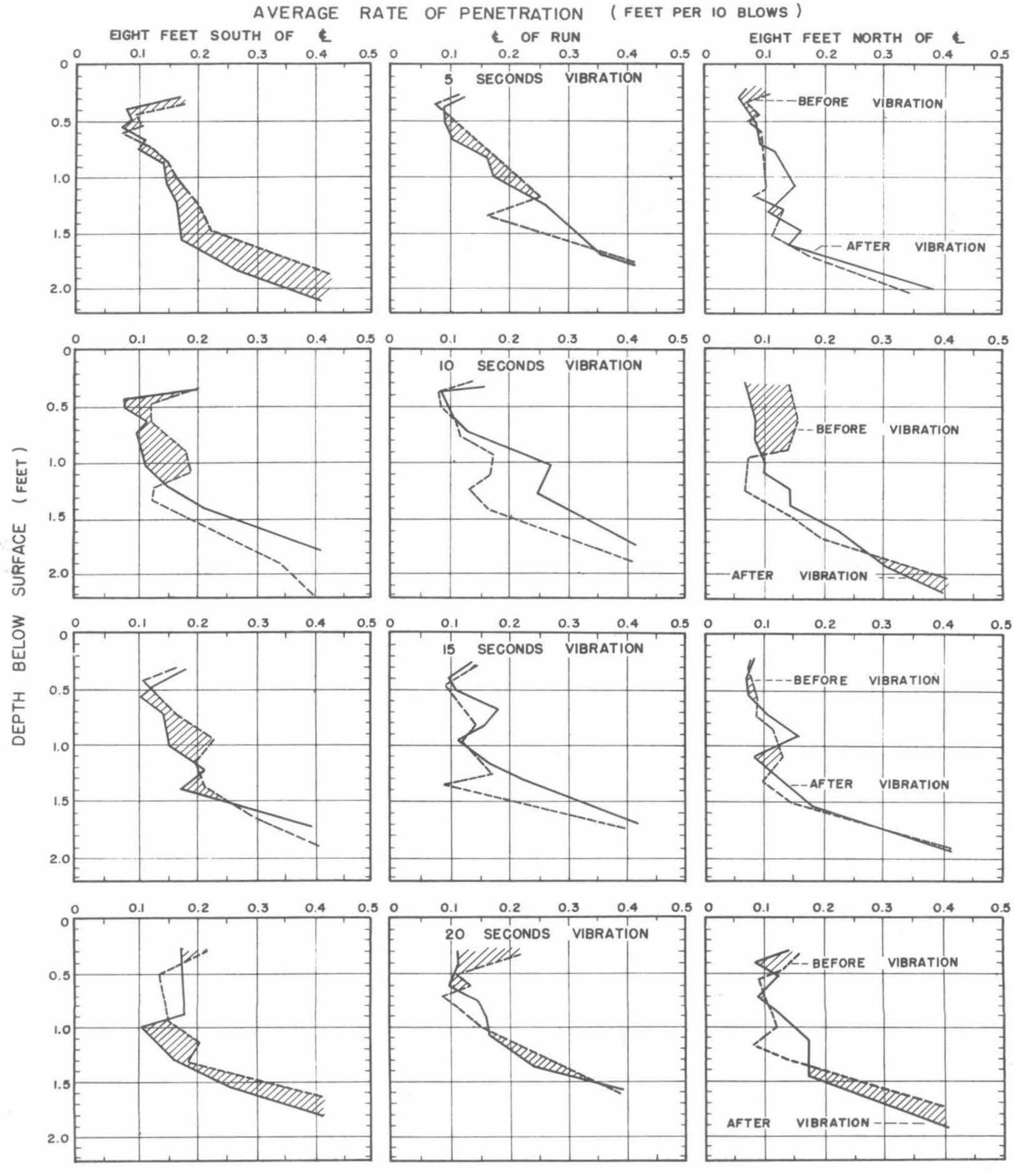


DEAD LOAD 12,800 LB. FREQUENCY 14.4 CPS.
DYNAMIC FORCE 14,300 LB. ECCENTRICS 6 PER SHAFT
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT



COMPACTION TEST NO. 134 B



DEAD LOAD 12,800 LB. FREQUENCY 9.6 CPS.

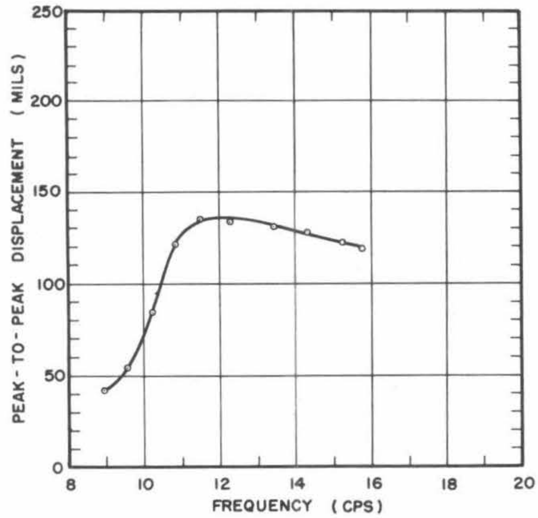
DYNAMIC FORCE 6,400 LB. ECCENTRICS 6 PER SHAFT

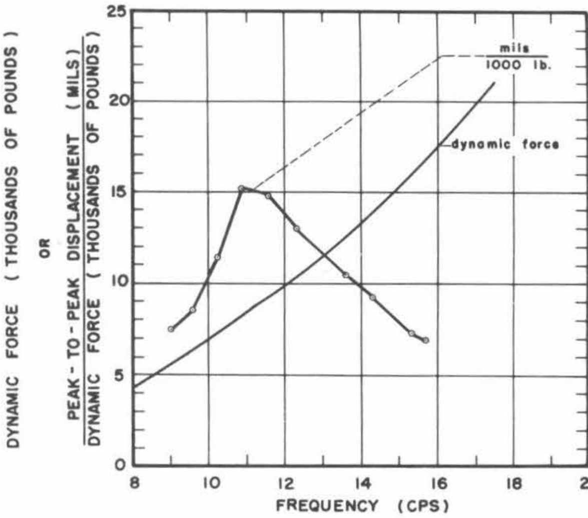
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT

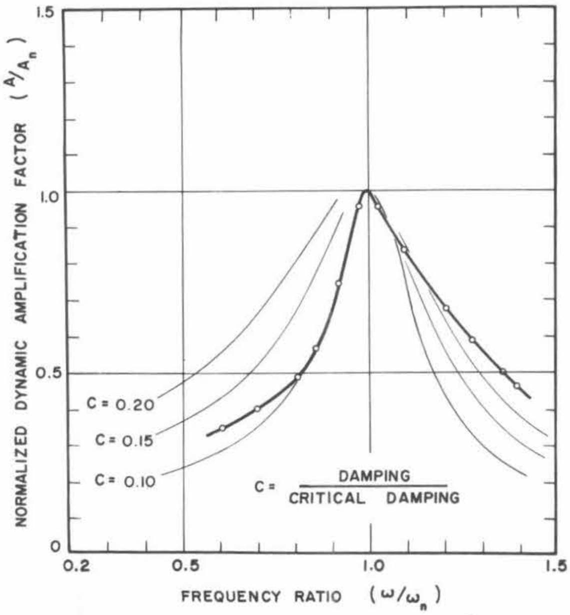
RUN NO. 35

VERTICAL DISPLACEMENT DATA

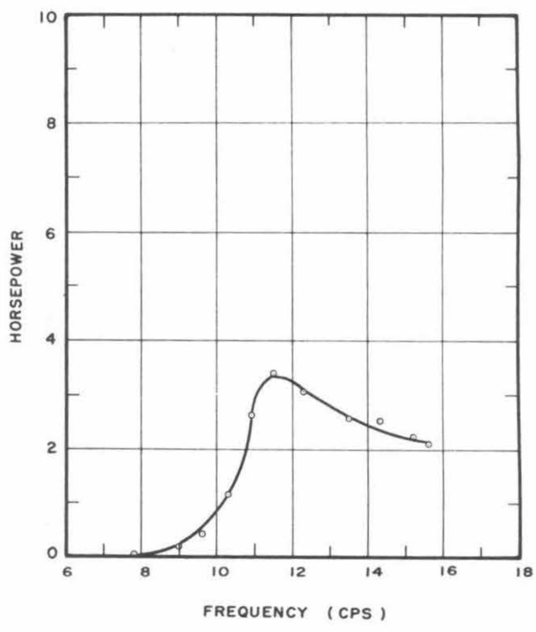




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

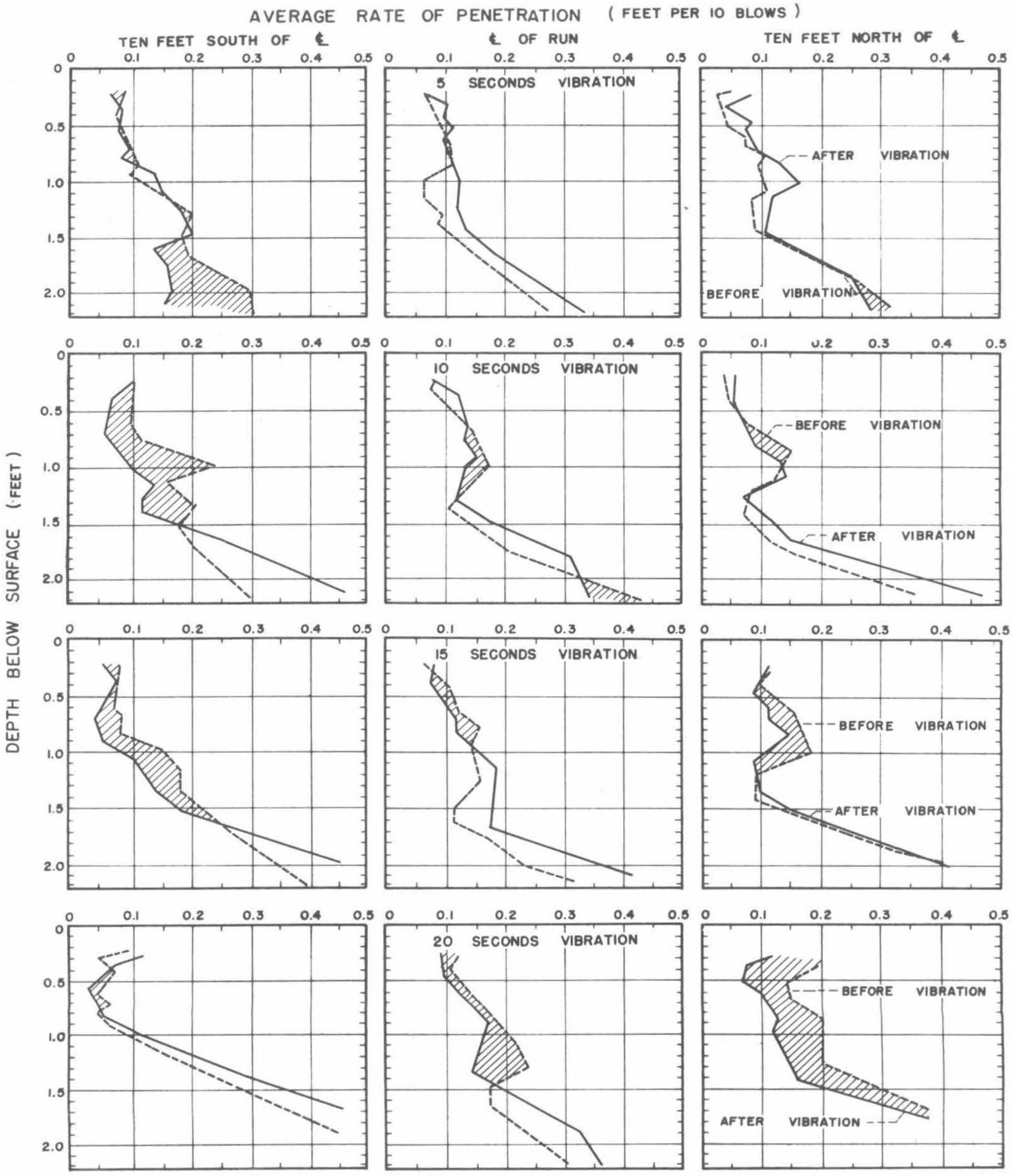


NOTES

VIBRATOR DEAD LOAD = 14,800 POUNDS

SIX ECCENTRICS PER SHAFT

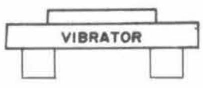
COMPACTION TEST NO. 135



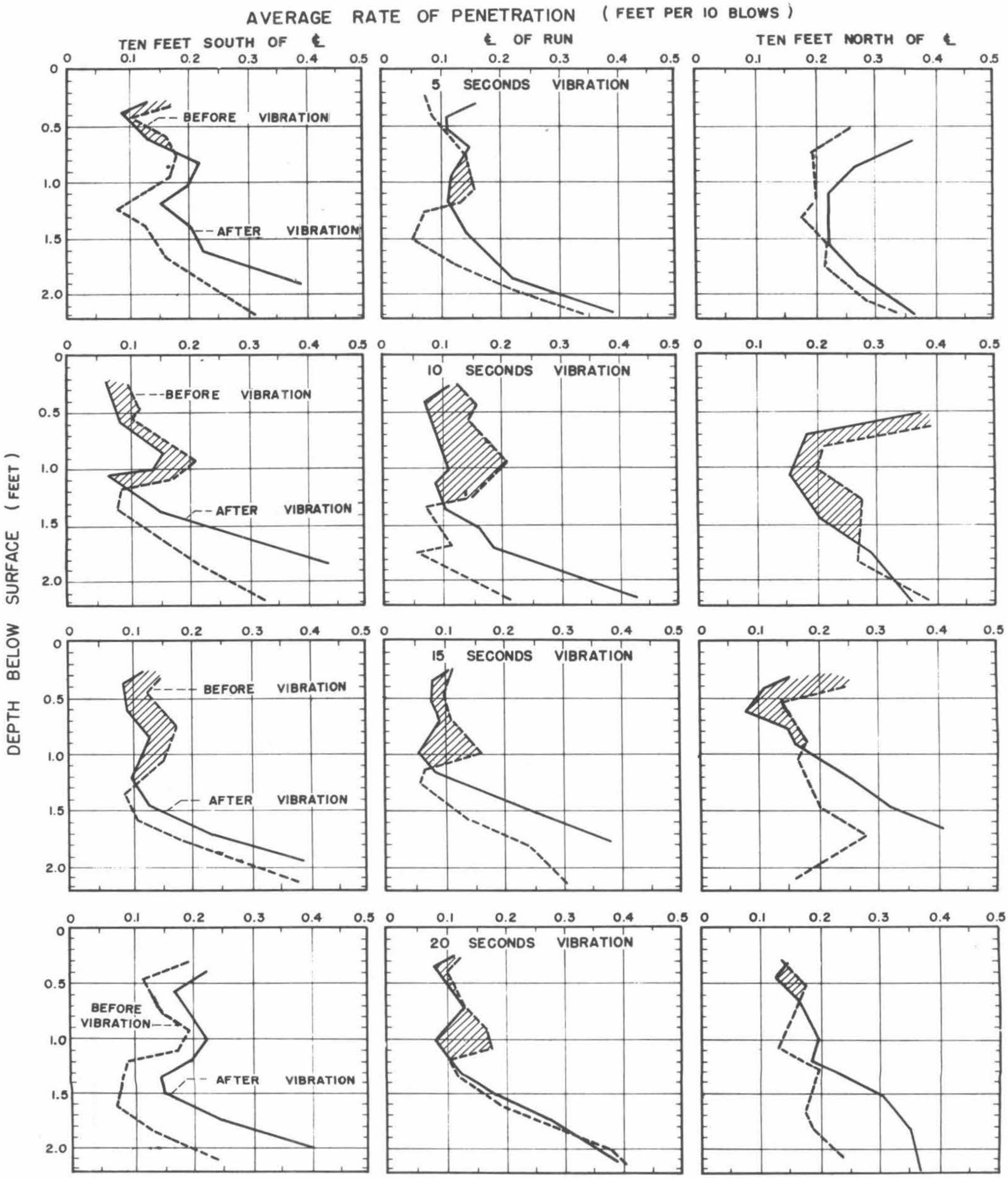
DEAD LOAD	14,800 LB.	FREQUENCY	11.2 CPS.
DYNAMIC FORCE	8,700 LB.	ECCENTRICS	6 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT



COMPACTION TEST NO. 135 A



DEAD LOAD	14,800 LB.	FREQUENCY	13.4 CPS.
DYNAMIC FORCE	12,400 LB.	ECCENTRICS	6 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

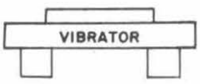
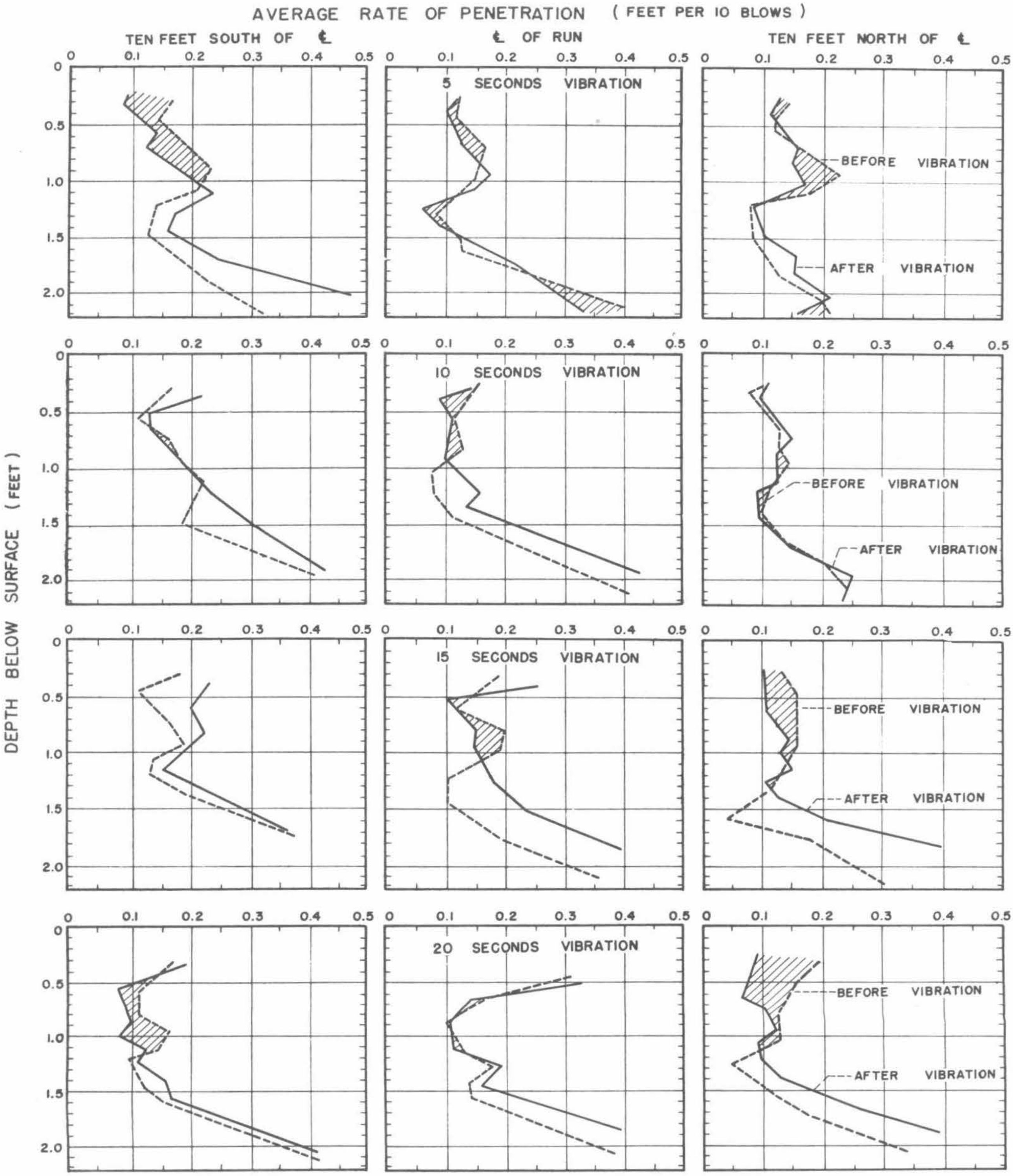


PLATE 24

COMPACTION TEST NO. 135 B

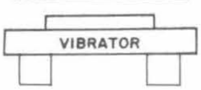


DEAD LOAD 14,800 LB. FREQUENCY 9.0 CPS.

DYNAMIC FORCE 5,600 LB. ECCENTRICS 6 PER SHAFT

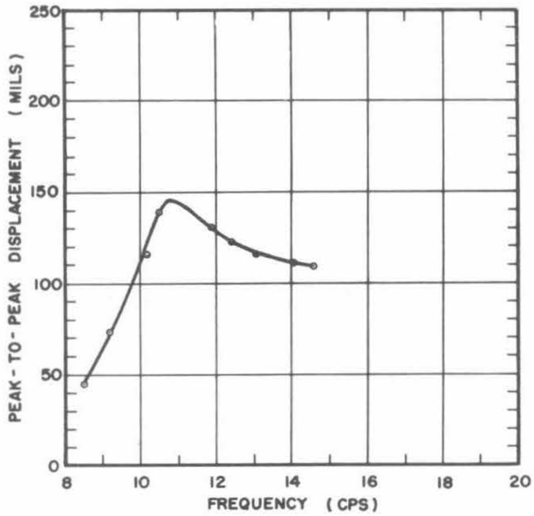
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

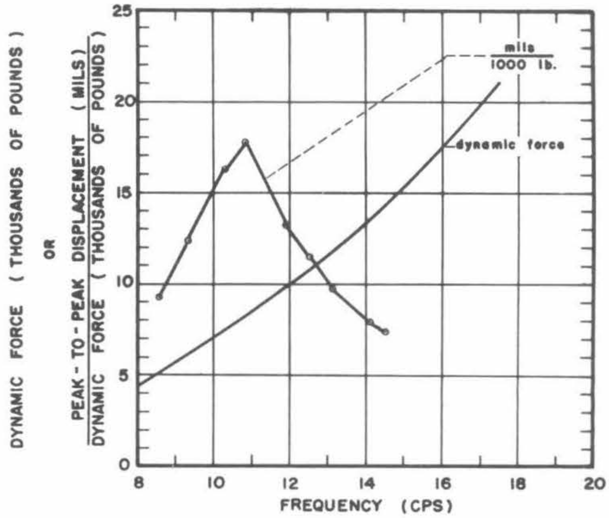
SKID ARRANGEMENT



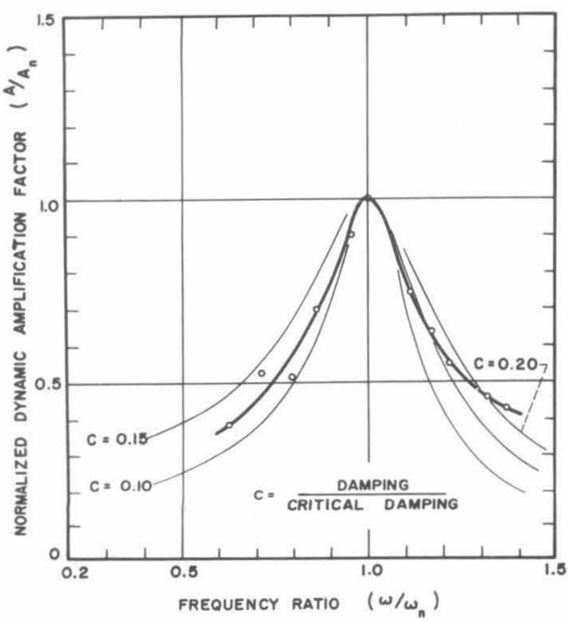
RUN NO. 36

VERTICAL DISPLACEMENT DATA

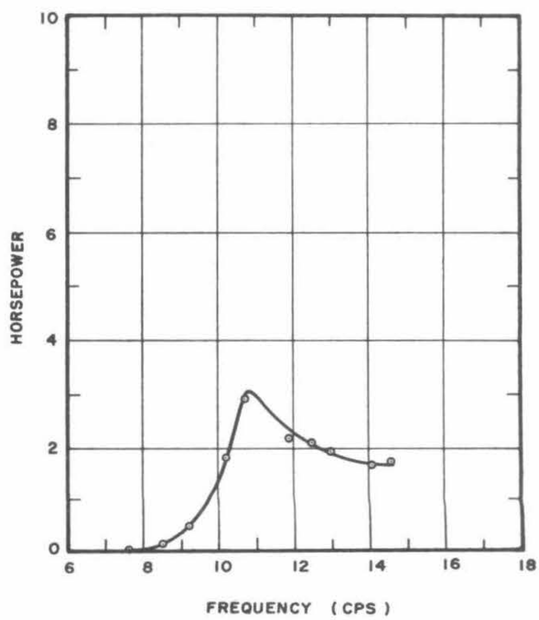




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



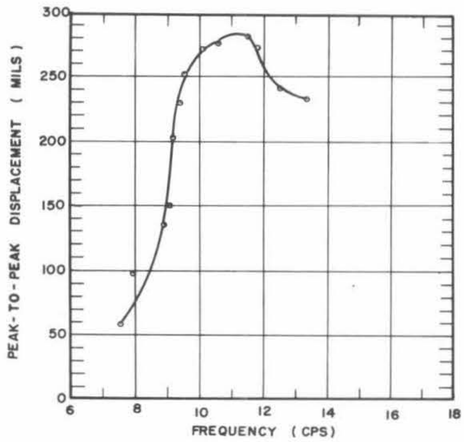
NOTES

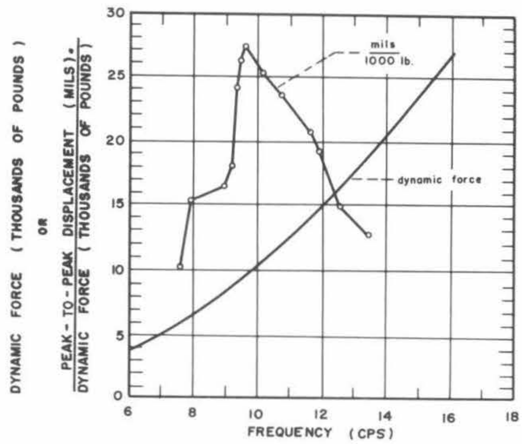
VIBRATOR DEAD LOAD = 16,800 POUNDS

SIX ECCENTRICS PER SHAFT

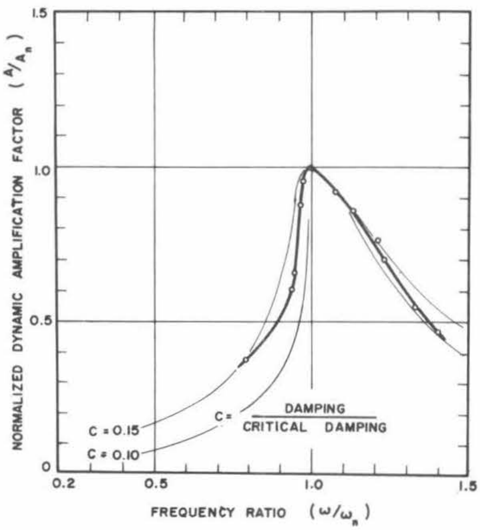
RUN NO. 37

VERTICAL DISPLACEMENT DATA

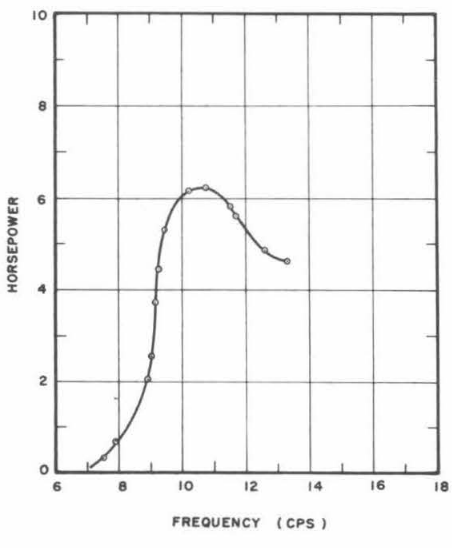




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



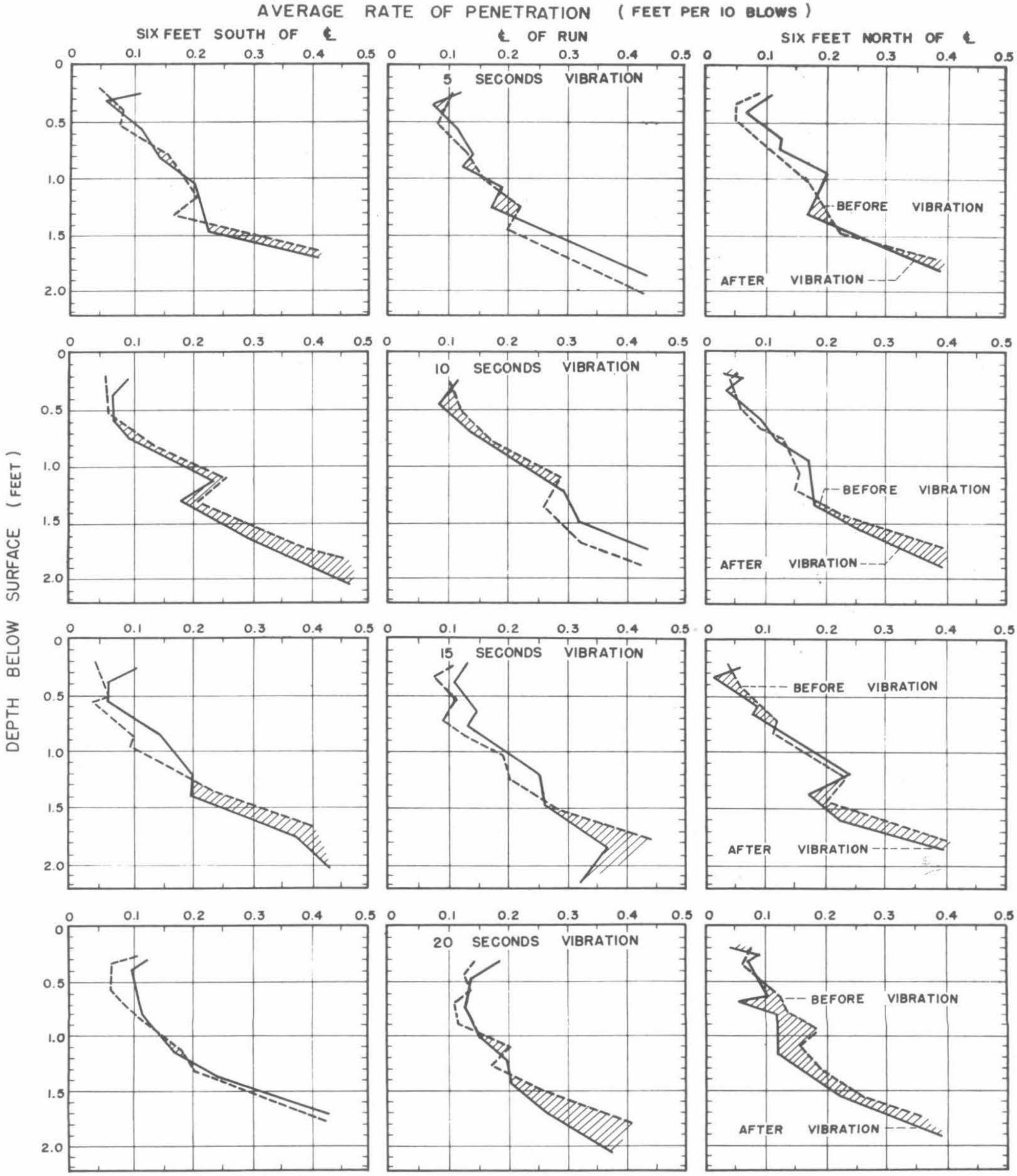
NOTES

VIBRATOR DEAD LOAD = 12,800 POUNDS

NINE ECCENTRICS PER SHAFT

PLATE 27

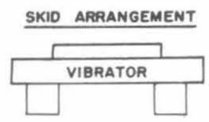
COMPACTION TEST NO. 137



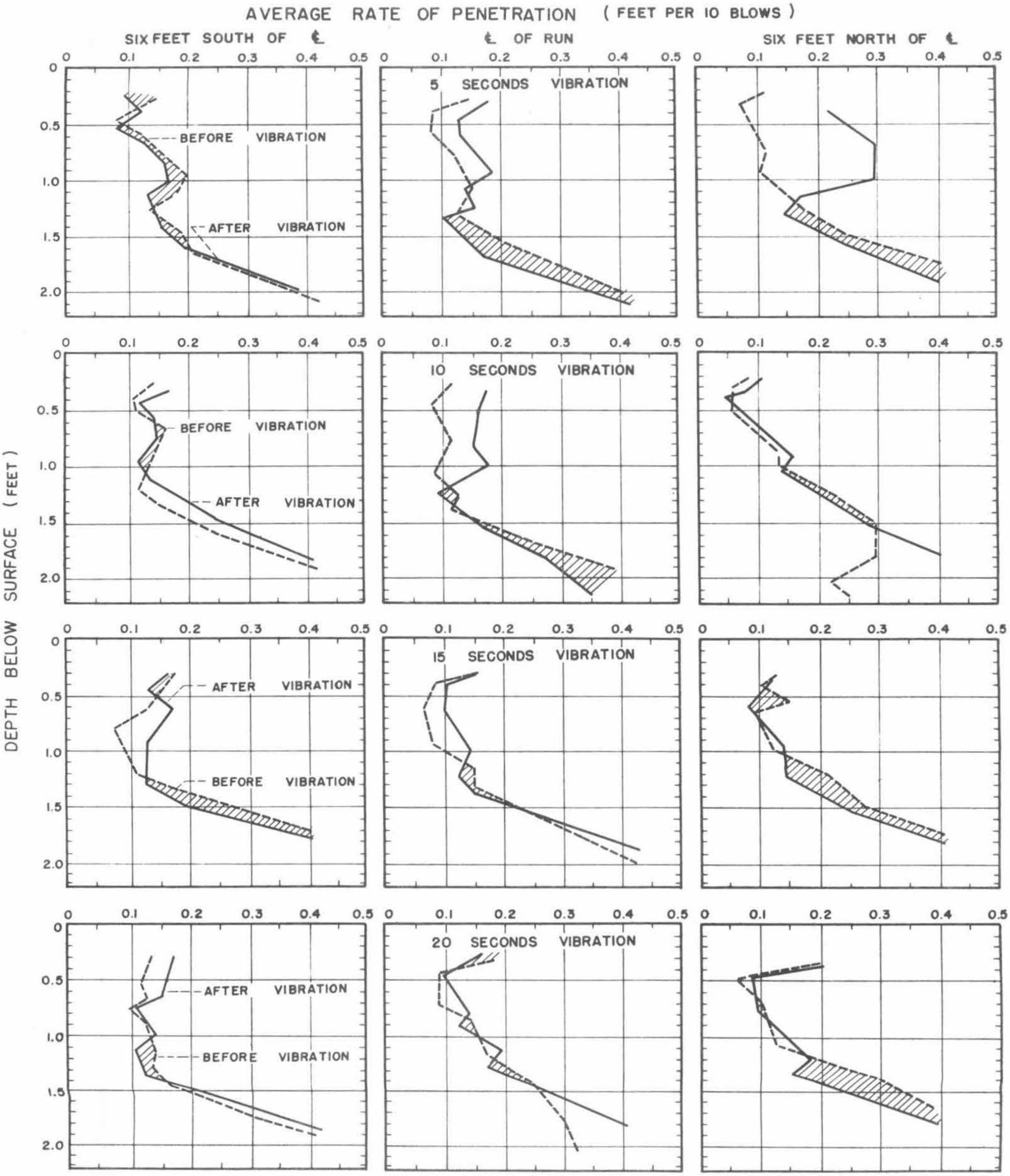
DEAD LOAD 12,800 LB. FREQUENCY 10.4 CPS.

DYNAMIC FORCE 11,100 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'



COMPACTION TEST NO. 137 A



DEAD LOAD 12,800 LB. FREQUENCY 12.5 CPS.

DYNAMIC FORCE 16,000 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - RECTANGULAR - 1'x 1'x 5'

SKID ARRANGEMENT

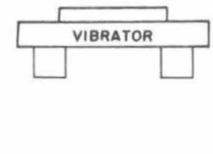
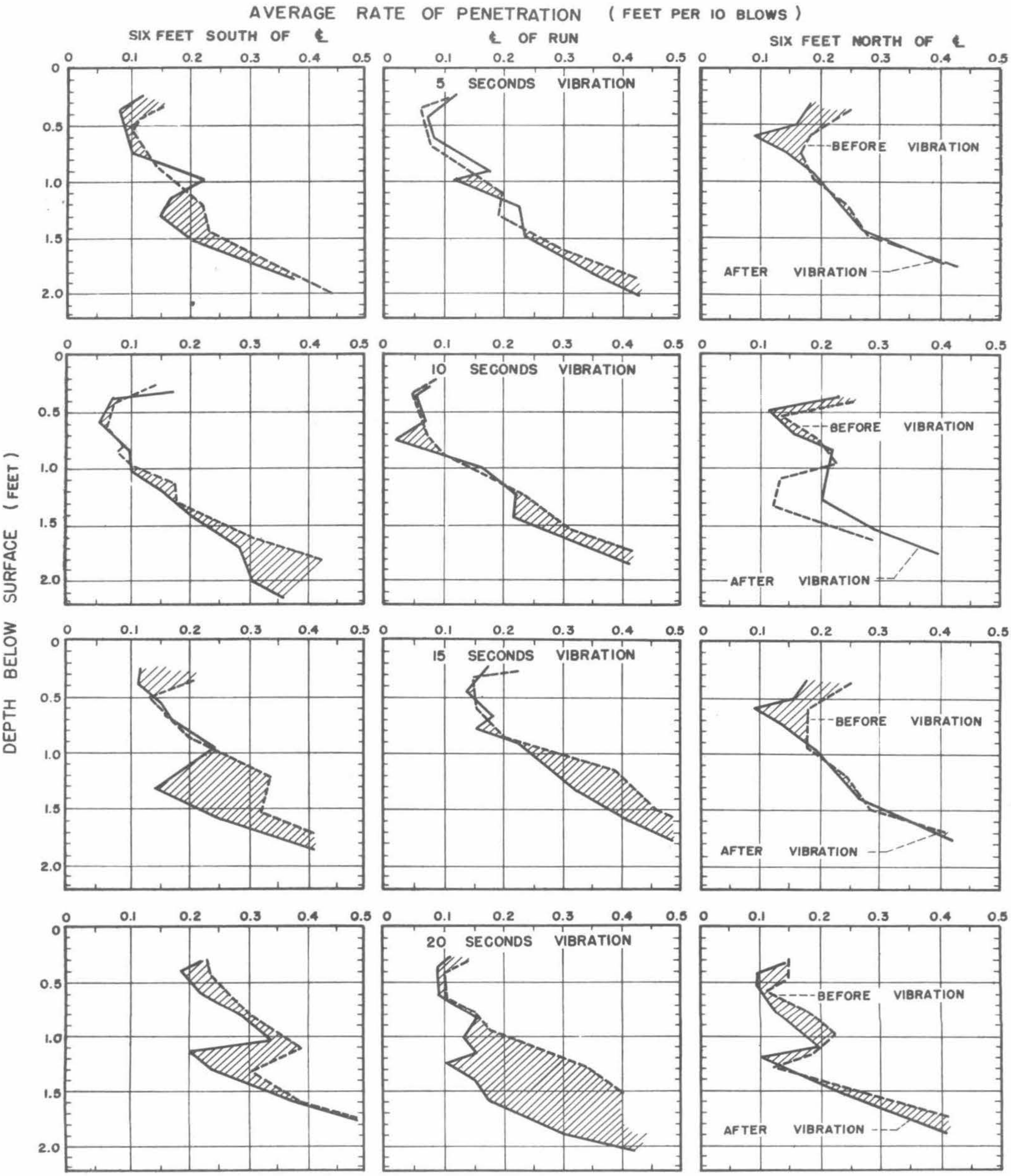


PLATE 29

COMPACTION TEST NO. 137 B

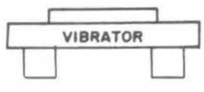


DEAD LOAD 12,800 LB. FREQUENCY 8.3 CPS.

DYNAMIC FORCE 7,000 LB. ECCENTRICS 9 PER SHAFT

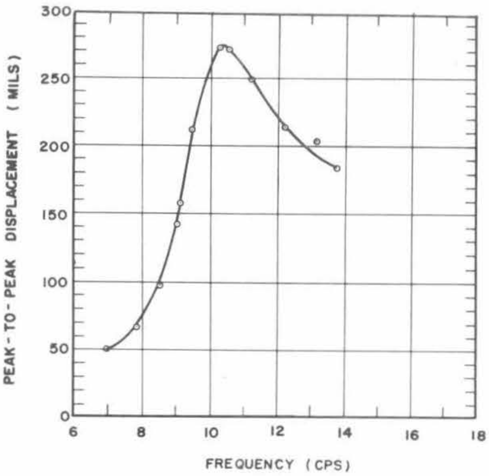
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

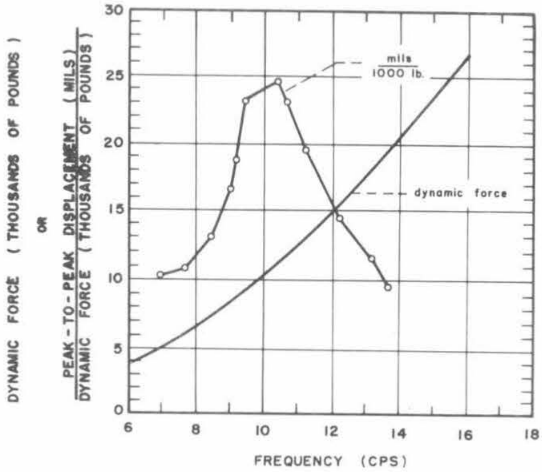
SKID ARRANGEMENT



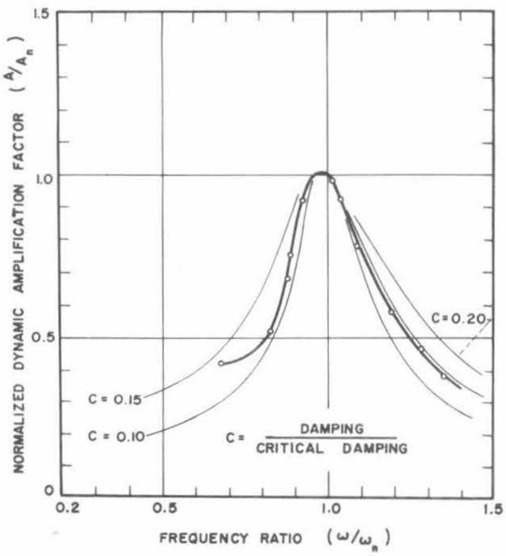
RUN NO. 38

VERTICAL DISPLACEMENT DATA

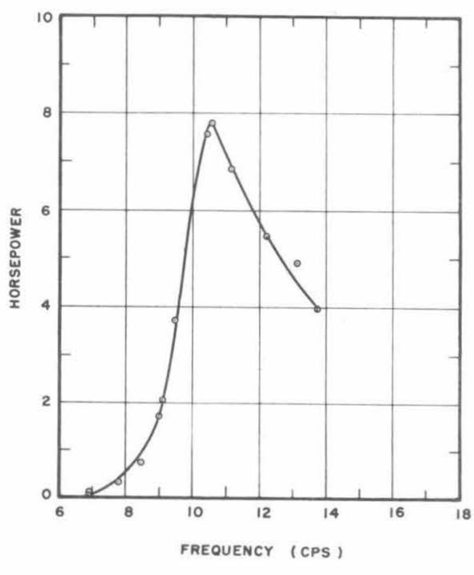




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

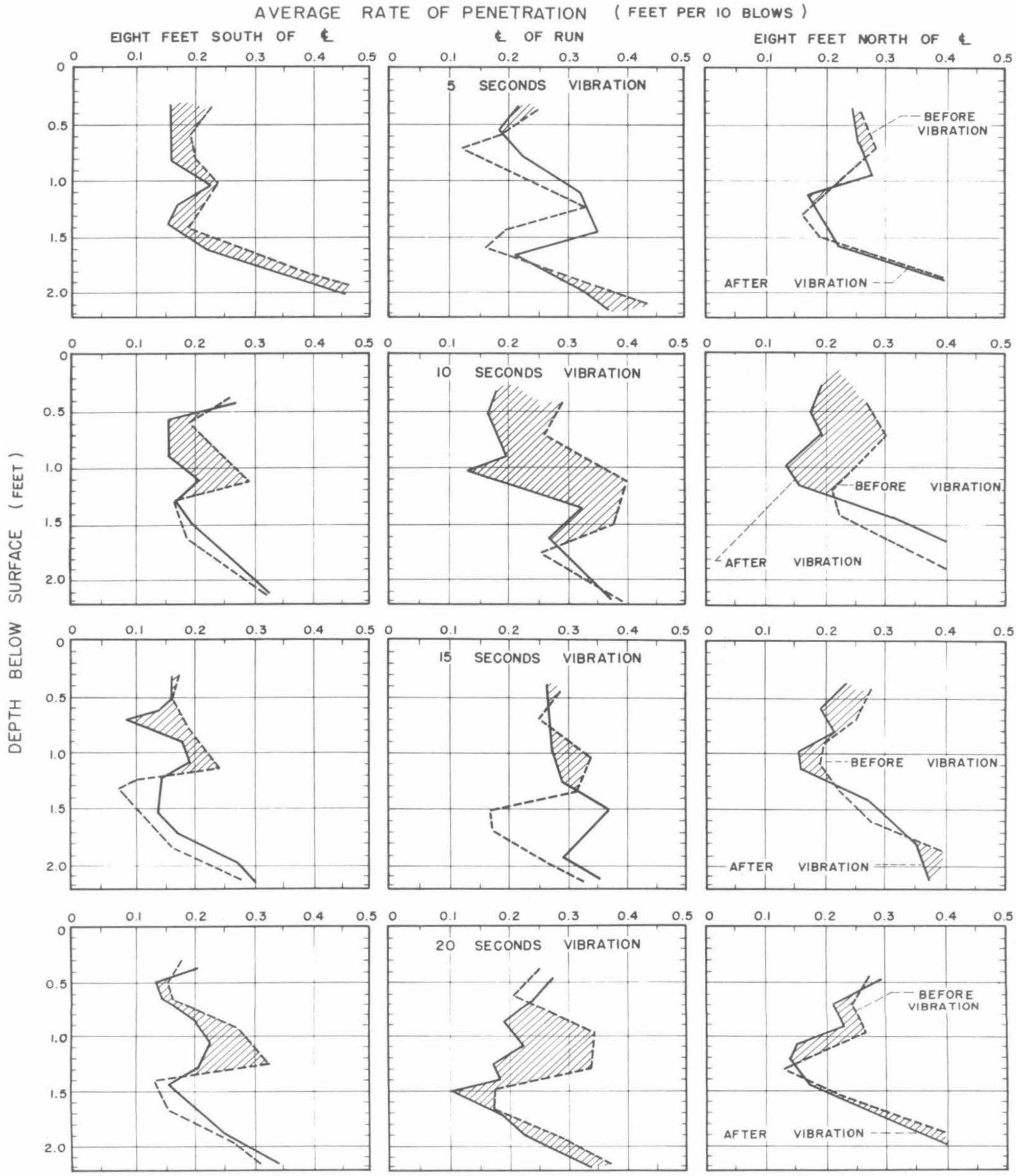


NOTES

VIBRATOR DEAD LOAD = 14,800 POUNDS

NINE ECCENTRICS PER SHAFT

COMPACTION TEST NO. 138



DEAD LOAD 14,800 LB. FREQUENCY 9.8 CPS.

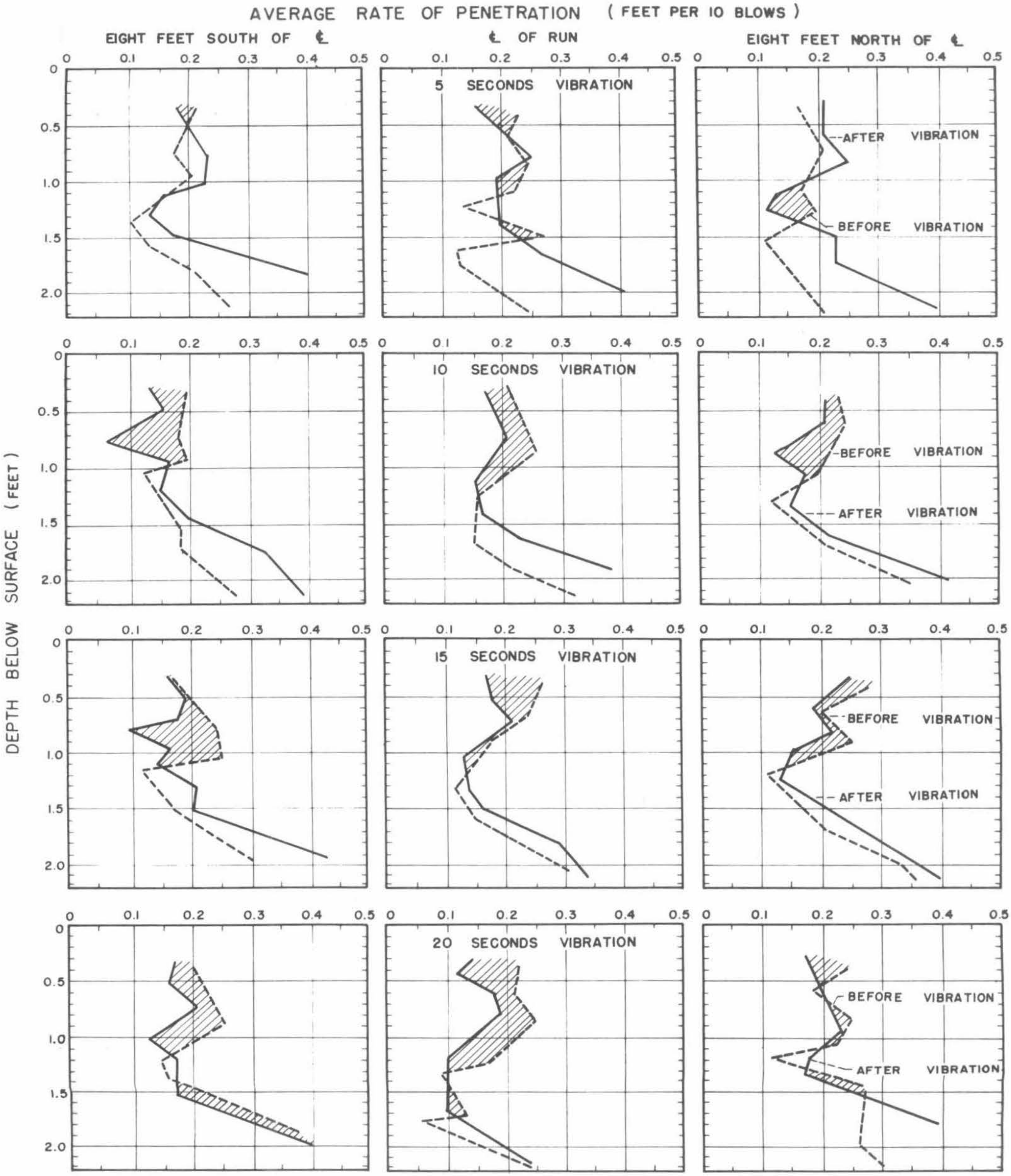
DYNAMIC FORCE 9,900 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT

PLATE 32

COMPACTION TEST NO. 138 A



DEAD LOAD 14,800 LB. FREQUENCY 11.8 CPS.

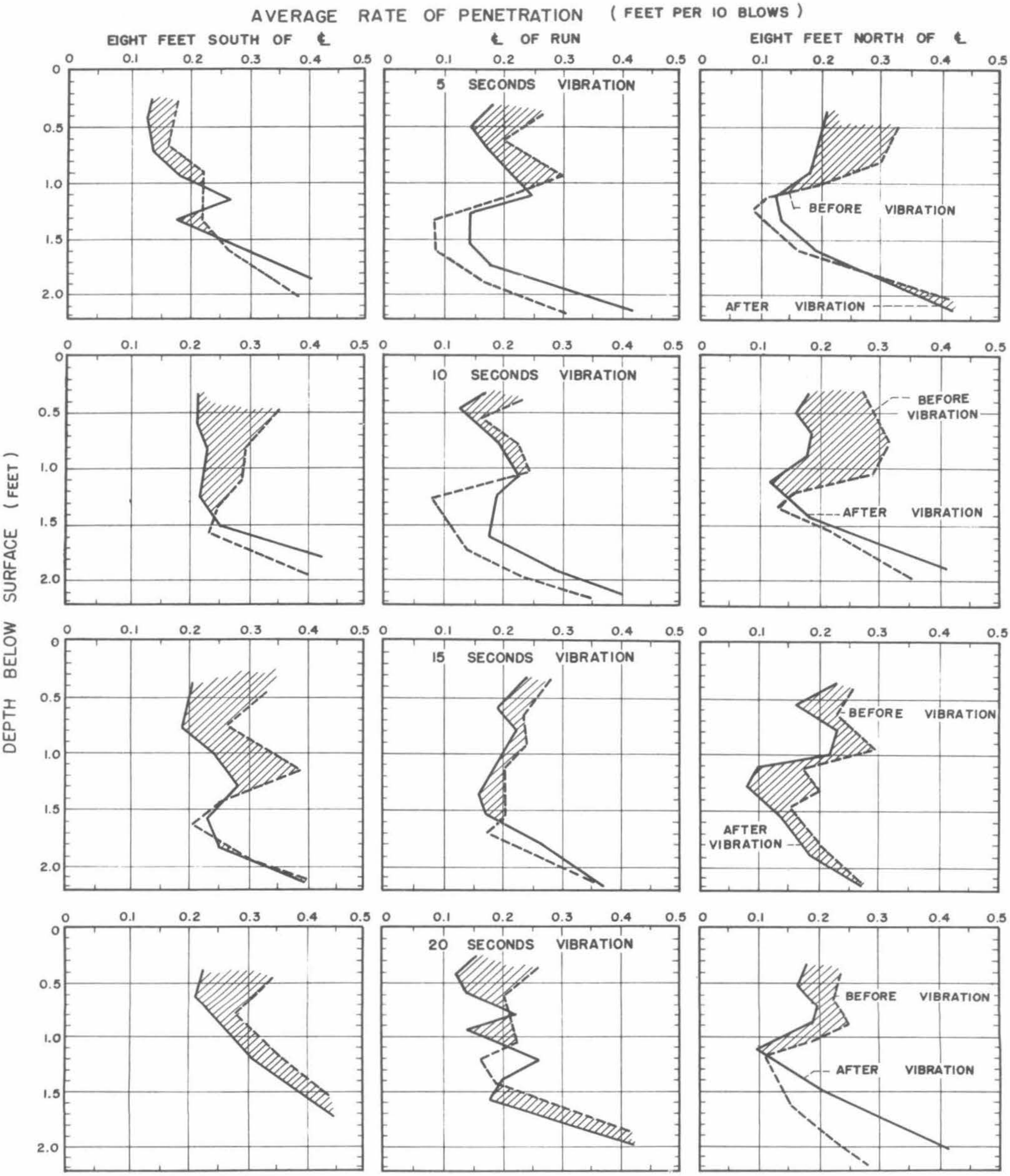
DYNAMIC FORCE 14,000 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - RECTANGULAR - 1'x 1'x 5'

SKID ARRANGEMENT

PLATE 33

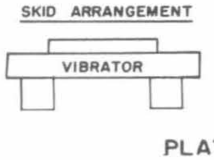
COMPACTION TEST NO. 138 B



DEAD LOAD 14,800 LB. FREQUENCY 7.8 CPS.

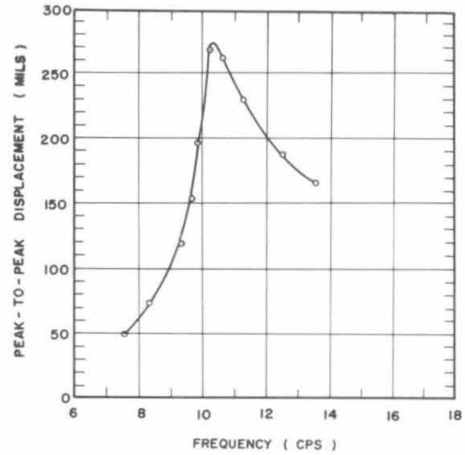
DYNAMIC FORCE 6,200 LB. ECCENTRICS 9 PER SHAFT

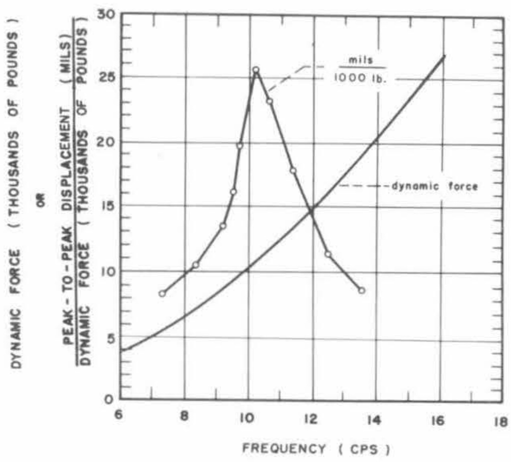
TYPE OF BASE PLATE TWO SKIDS - RECTANGULAR - 1'x 1'x 5'



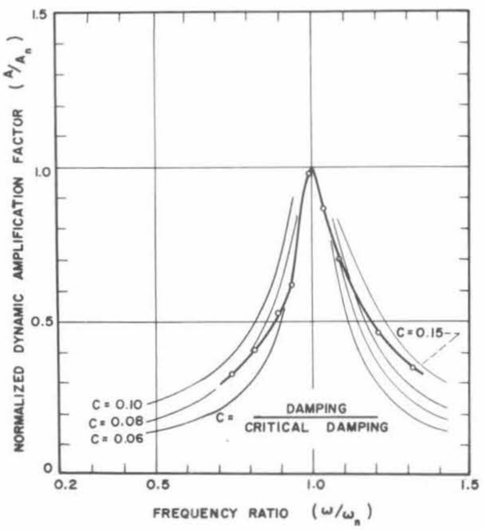
RUN NO. 39

VERTICAL DISPLACEMENT DATA

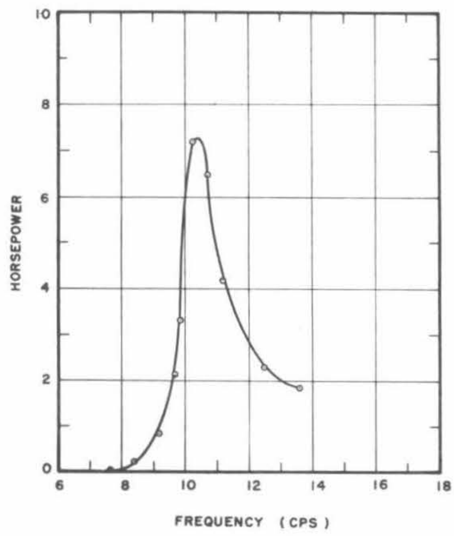




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

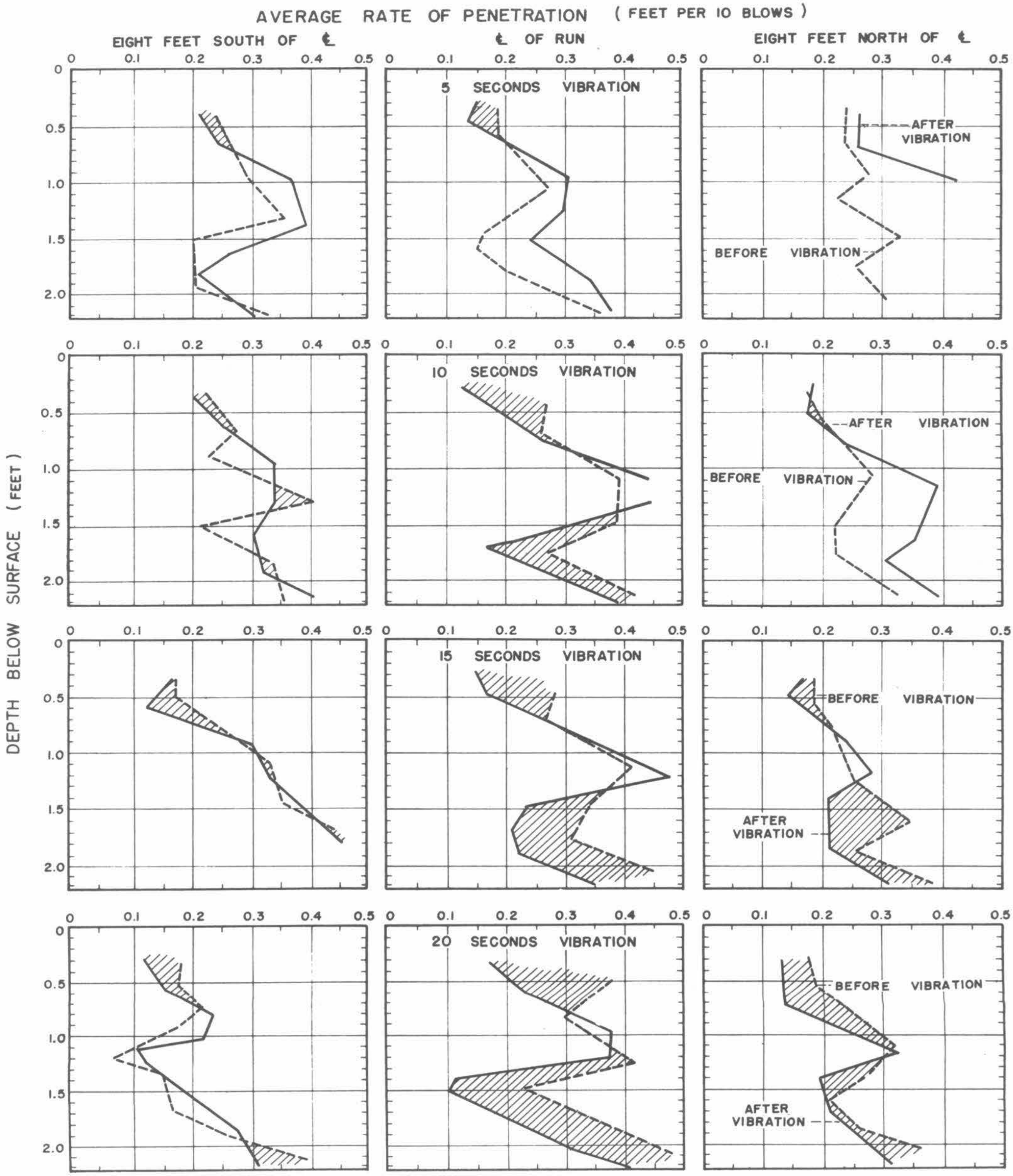


NOTES

VIBRATOR DEAD LOAD = 16,800 POUNDS

NINE ECCENTRICS PER SHAFT

COMPACTION TEST NO. 139



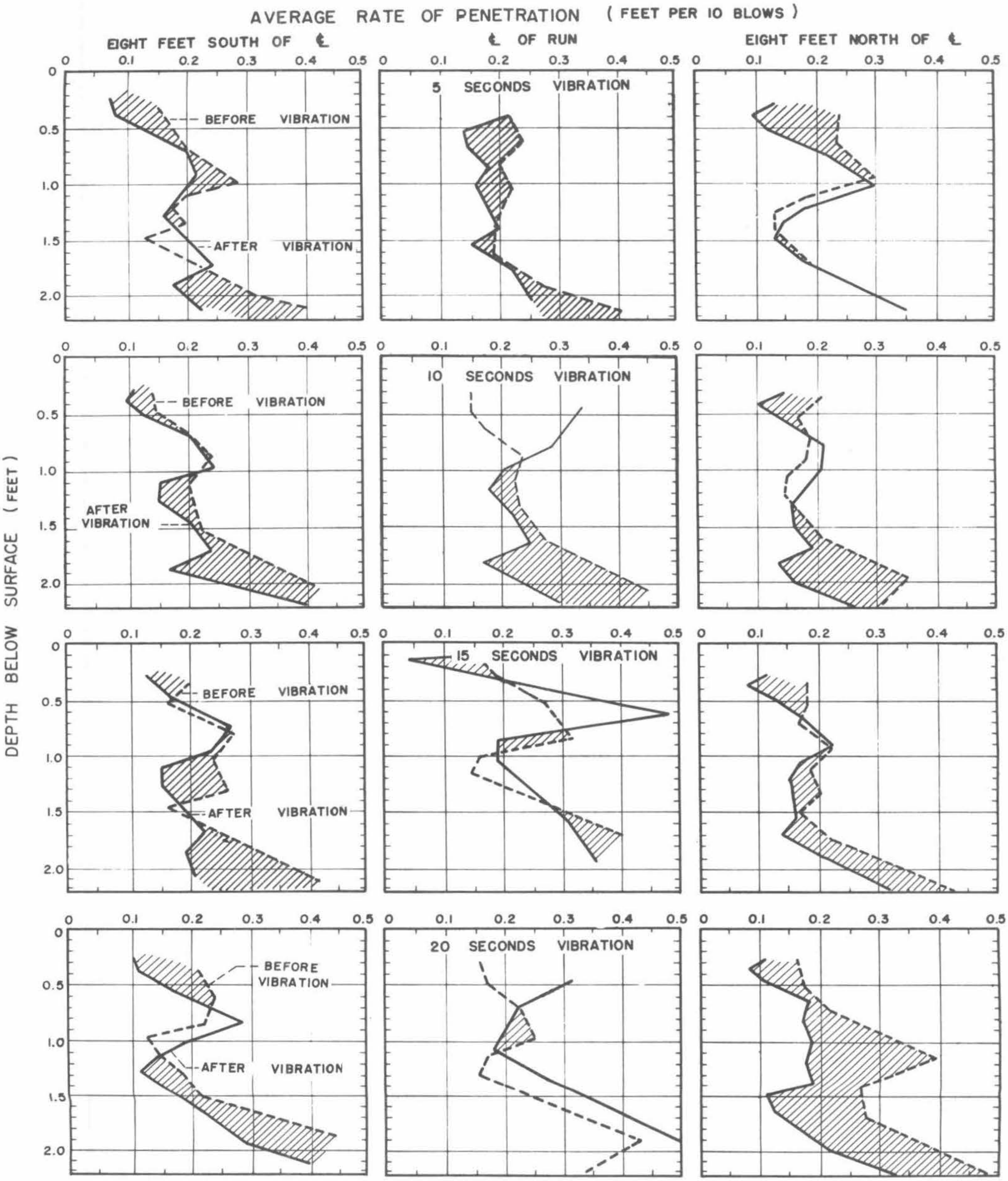
DEAD LOAD 16,800 LB. FREQUENCY 10.3 CPS.

DYNAMIC FORCE 10,400 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - RECTANGULAR - 1'x 1'x 5'

SKID ARRANGEMENT

COMPACTION TEST NO. 139 A

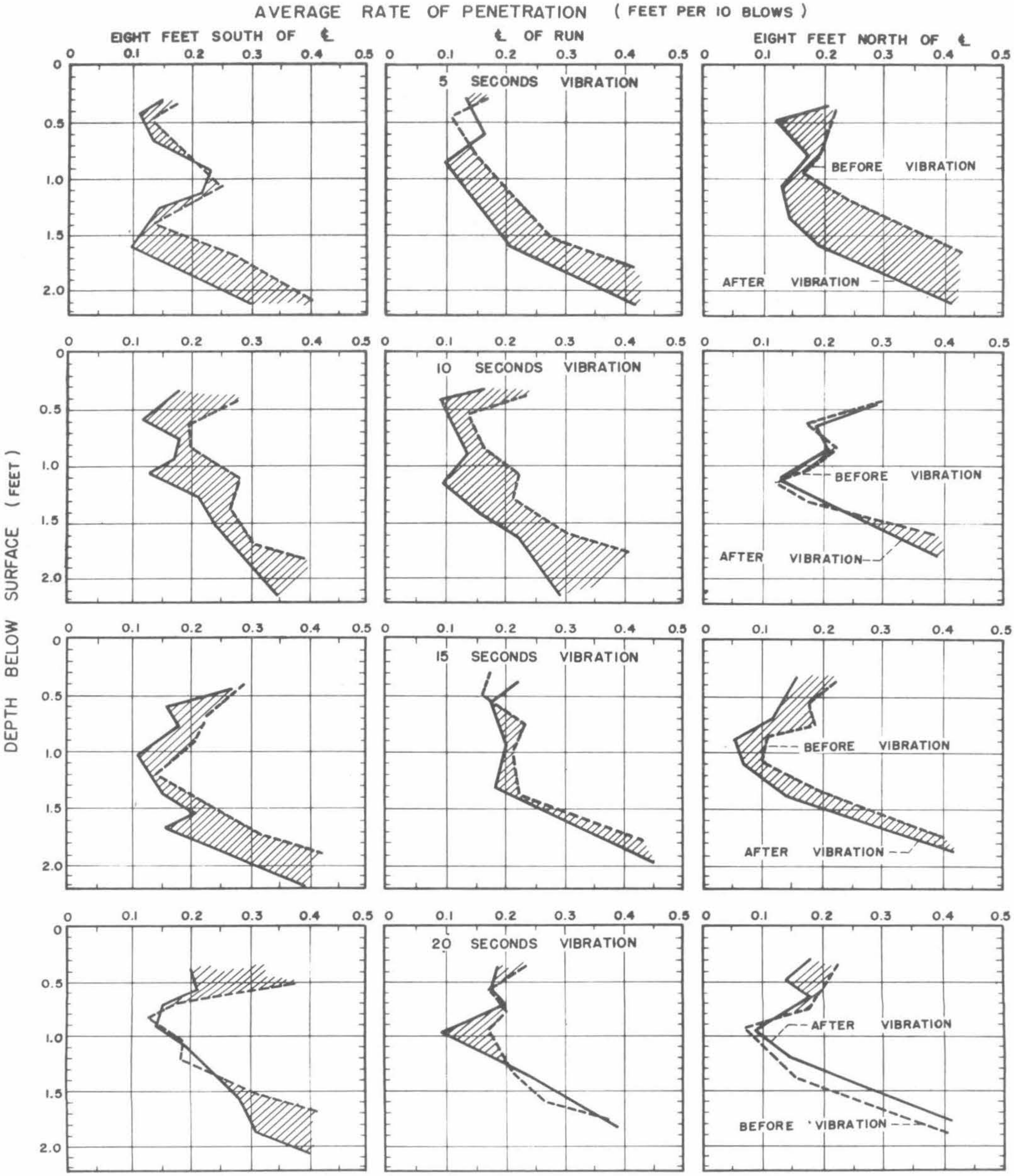


DEAD LOAD 16,800 LB. FREQUENCY 12.4 CPS.
DYNAMIC FORCE 16,000 LB. ECCENTRICS 9 PER SHAFT
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT

PLATE 37

COMPACTION TEST NO. 139 B



DEAD LOAD 16,800 LB. FREQUENCY 8.2 CPS.

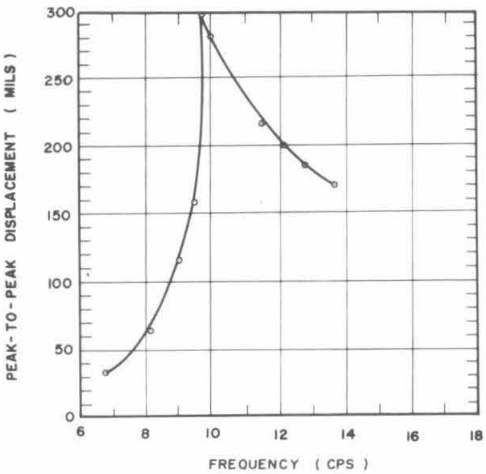
DYNAMIC FORCE 6,700 LB. ECCENTRICS 9 PER SHAFT

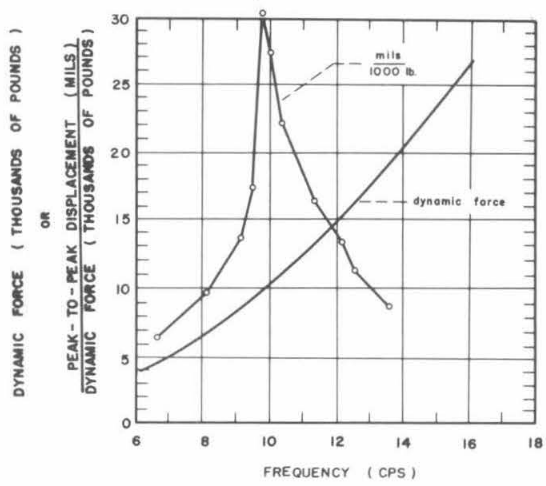
TYPE OF BASE PLATE TWO SKIDS – RECTANGULAR – 1'x 1'x 5'

SKID ARRANGEMENT

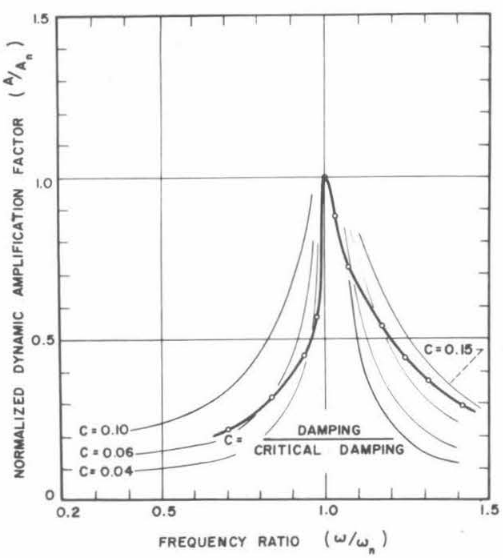
RUN NO. 39 M

VERTICAL DISPLACEMENT DATA

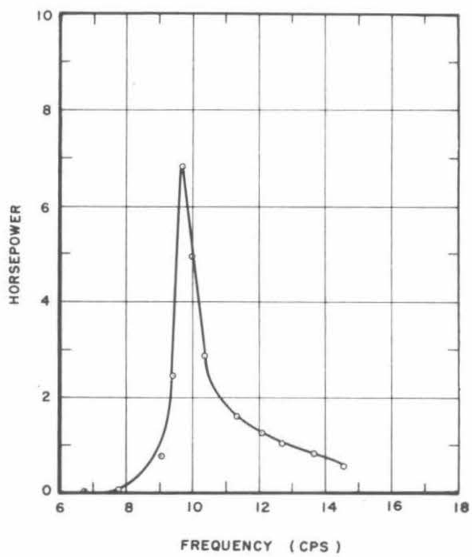




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



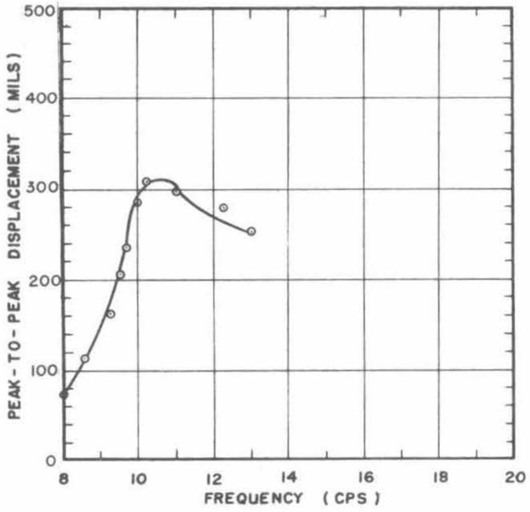
NOTES

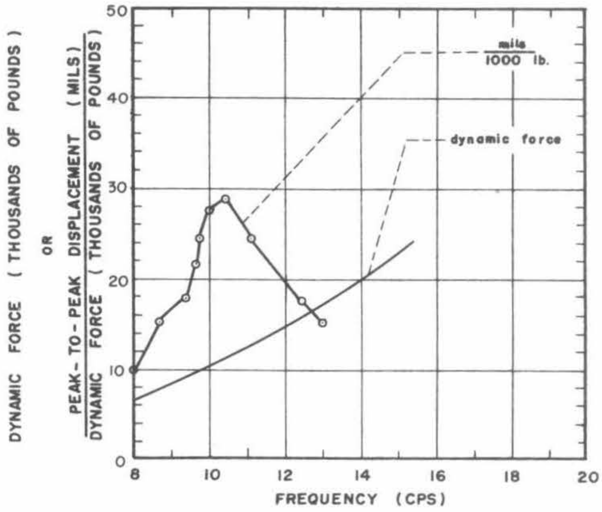
VIBRATOR DEAD LOAD = 16,800 POUNDS

NINE ECCENTRICS PER SHAFT

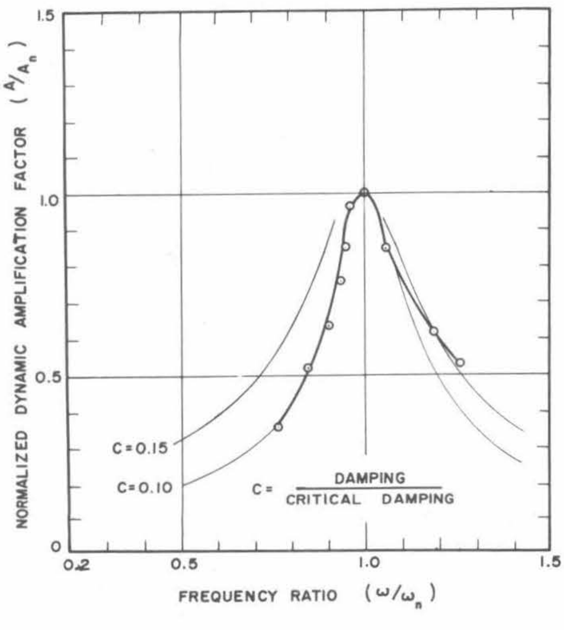
RUN NO. 30° T 37

VERTICAL DISPLACEMENT DATA

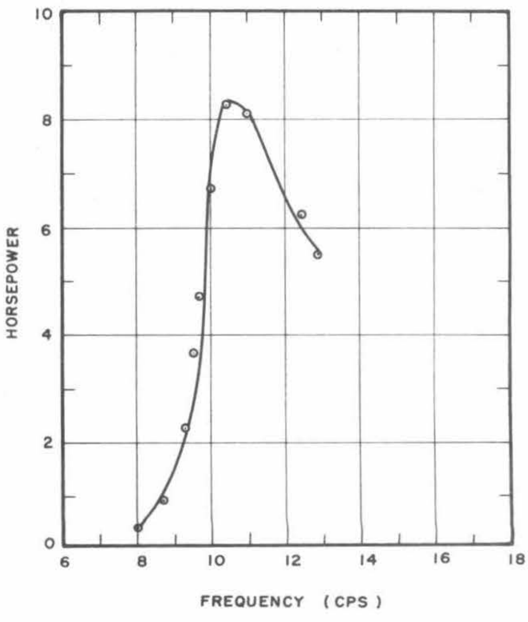




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

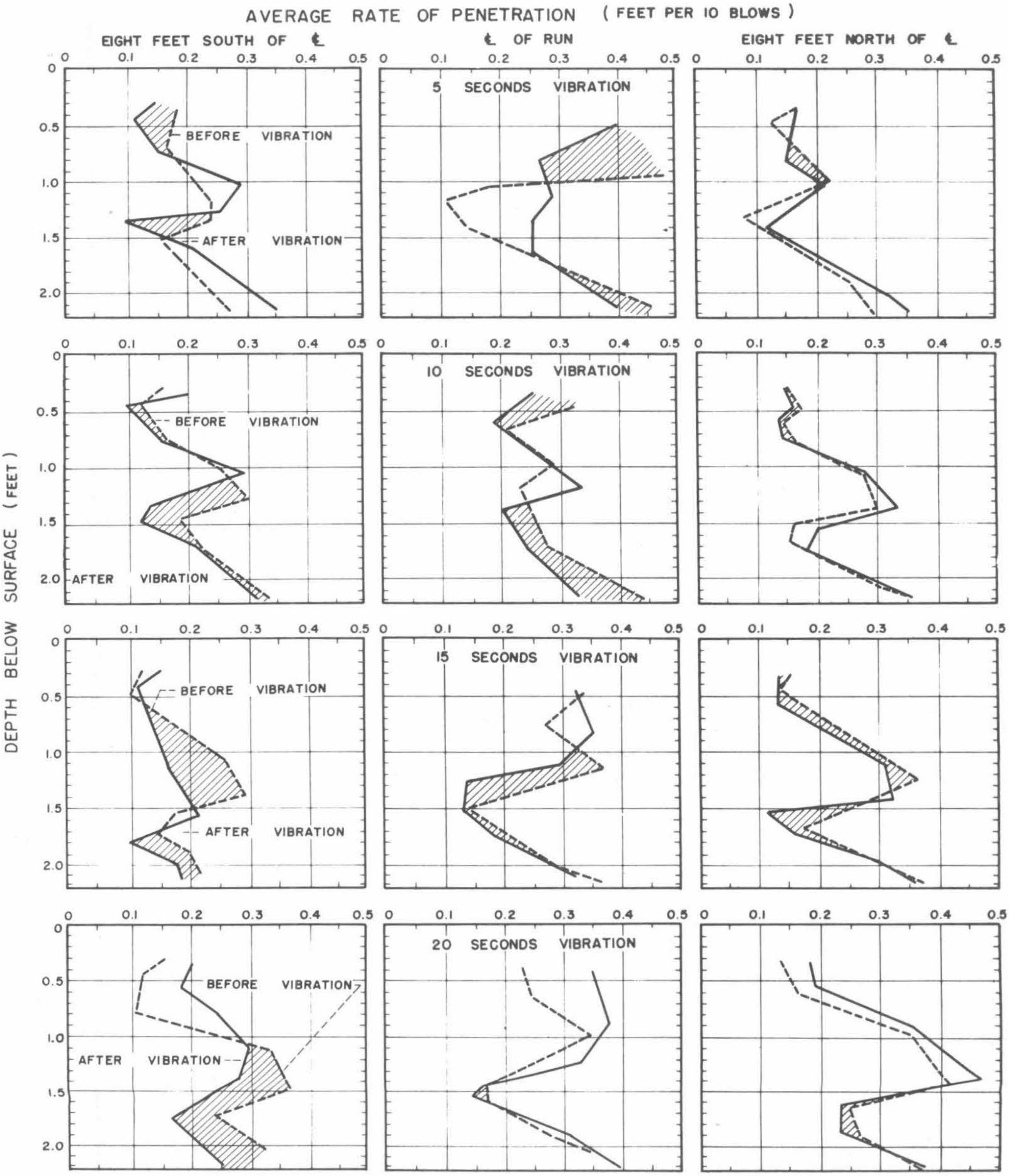


NOTES

VIBRATOR DEAD LOAD = 12,800 POUNDS

NINE ECCENTRICS PER SHAFT

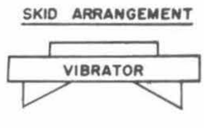
COMPACTION TEST NO. 30° T 137-1



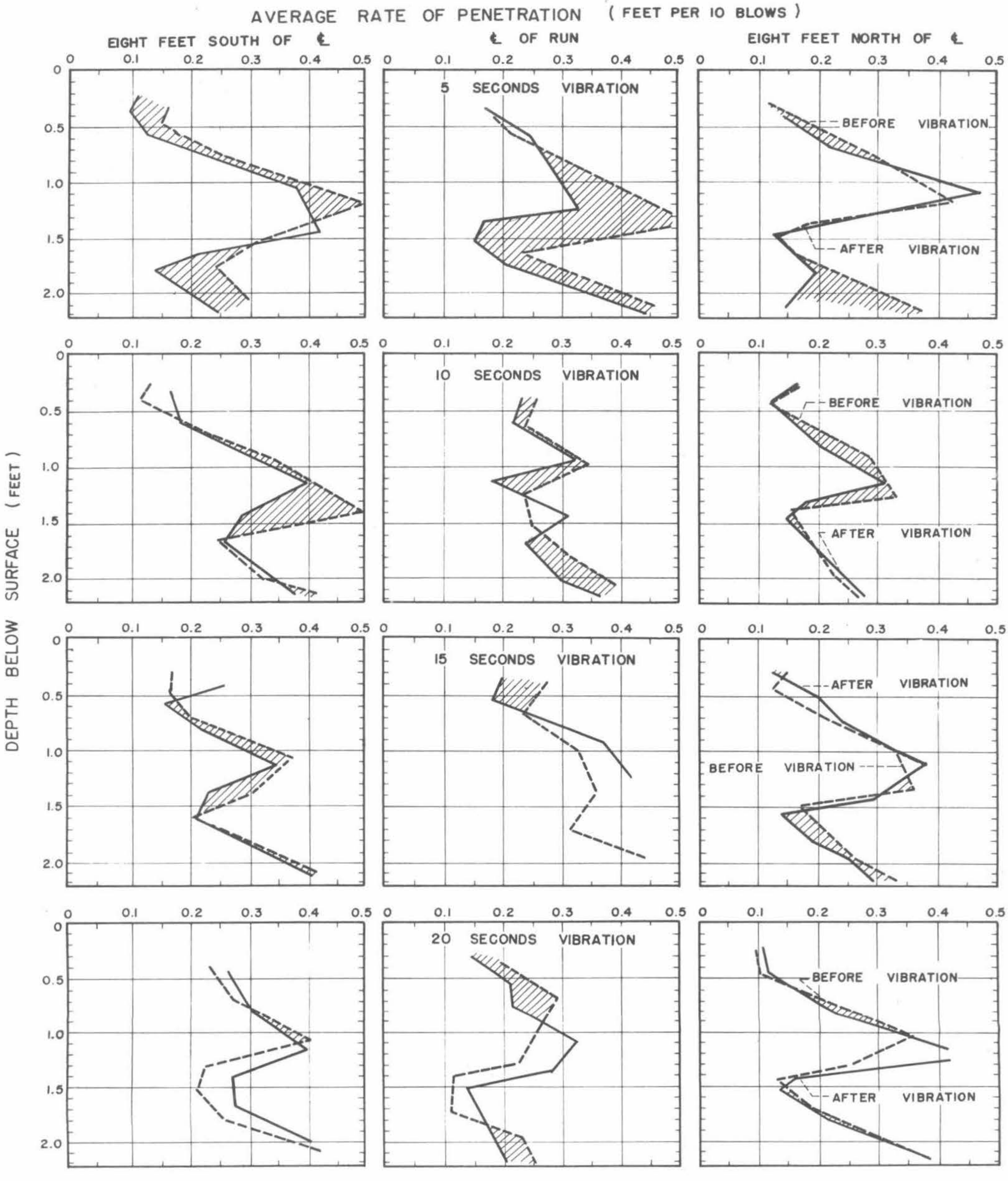
DEAD LOAD 12,800 LB. FREQUENCY 10.3 CPS.

DYNAMIC FORCE 10,800 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - TRIANGULAR



COMPACTION TEST NO. 30° T 137-2



DEAD LOAD 12,800 LB.

DYNAMIC FORCE 10,800 LB.

TYPE OF BASE PLATE TWO SKIDS – TRIANGULAR

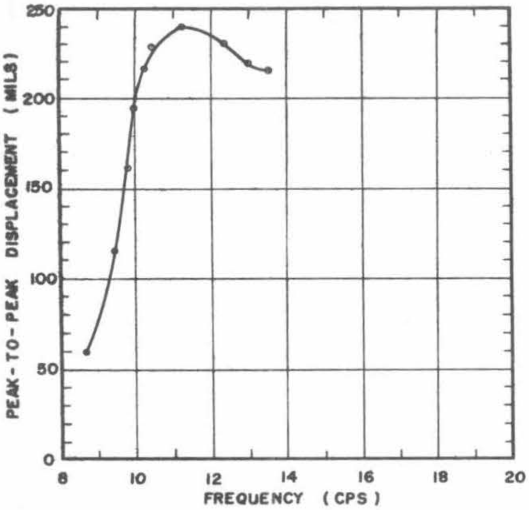
FREQUENCY 10.3 CPS.

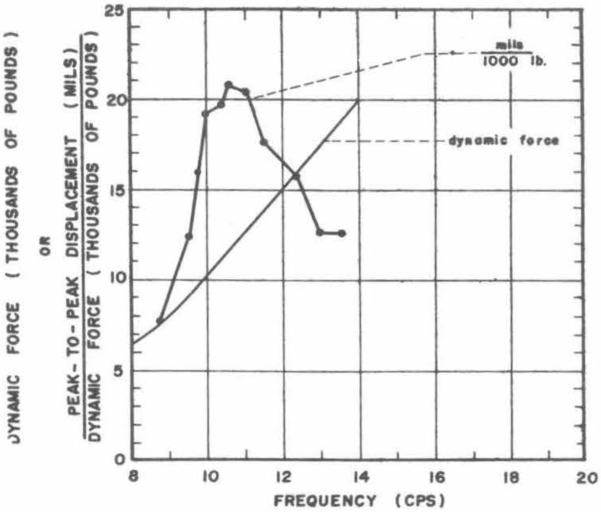
ECCENTRICS 9 PER SHAFT

SKID ARRANGEMENT

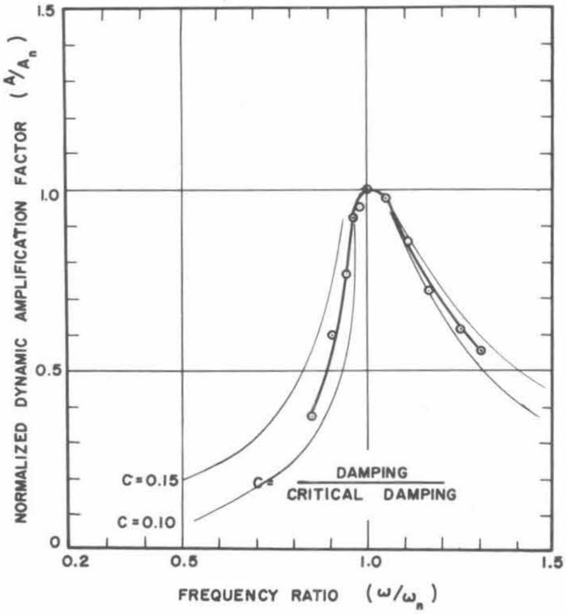
RUN NO. 30° T 38

VERTICAL DISPLACEMENT DATA

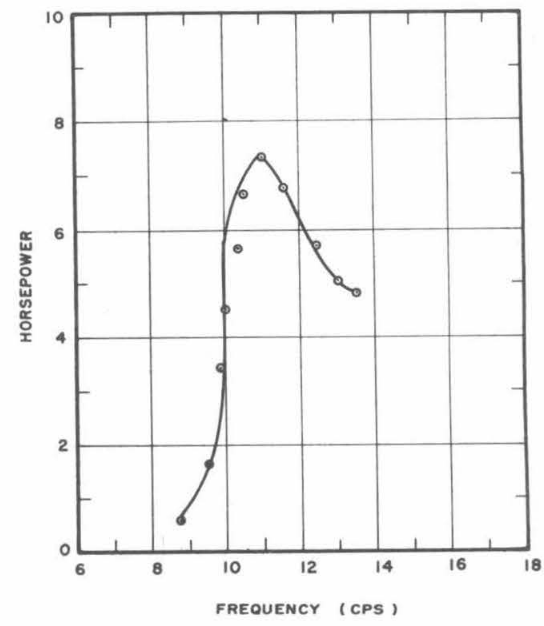




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



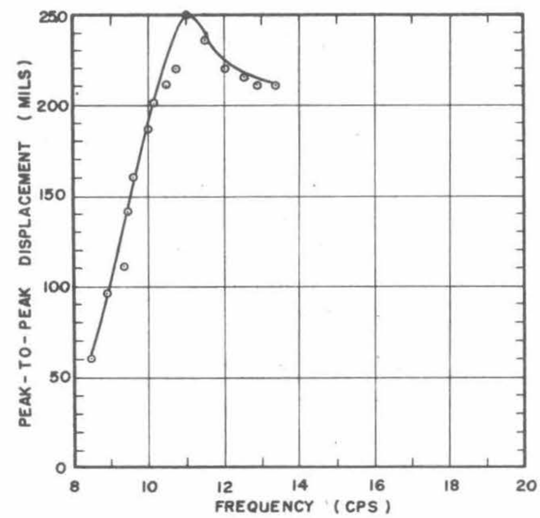
NOTES

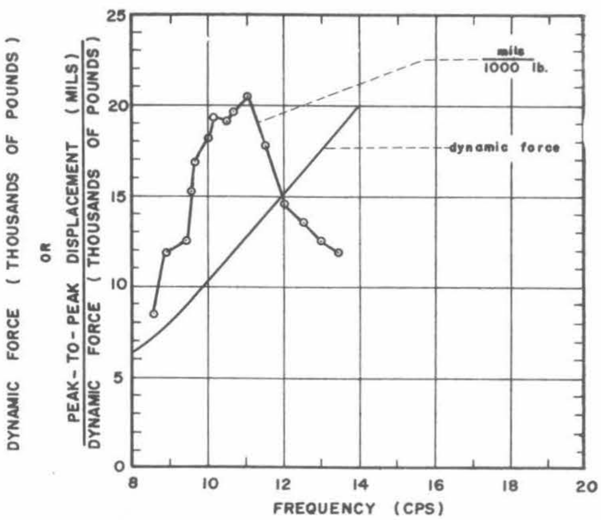
VIBRATOR DEAD LOAD = 14,800 POUNDS

NINE ECCENTRICS PER SHAFT

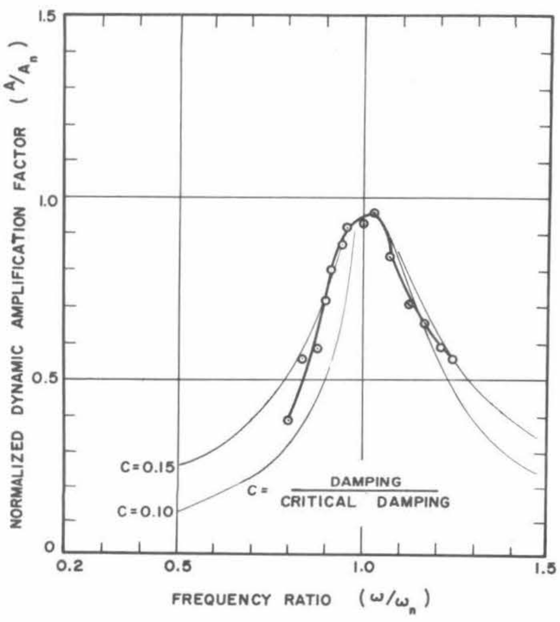
RUN NO. 30° T 38M

VERTICAL DISPLACEMENT DATA

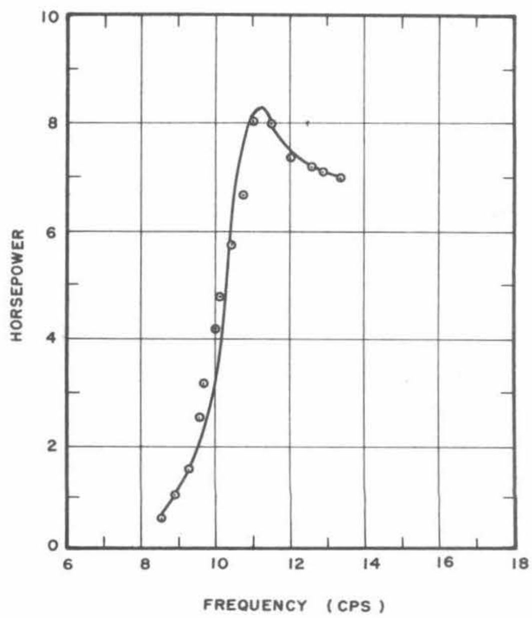




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

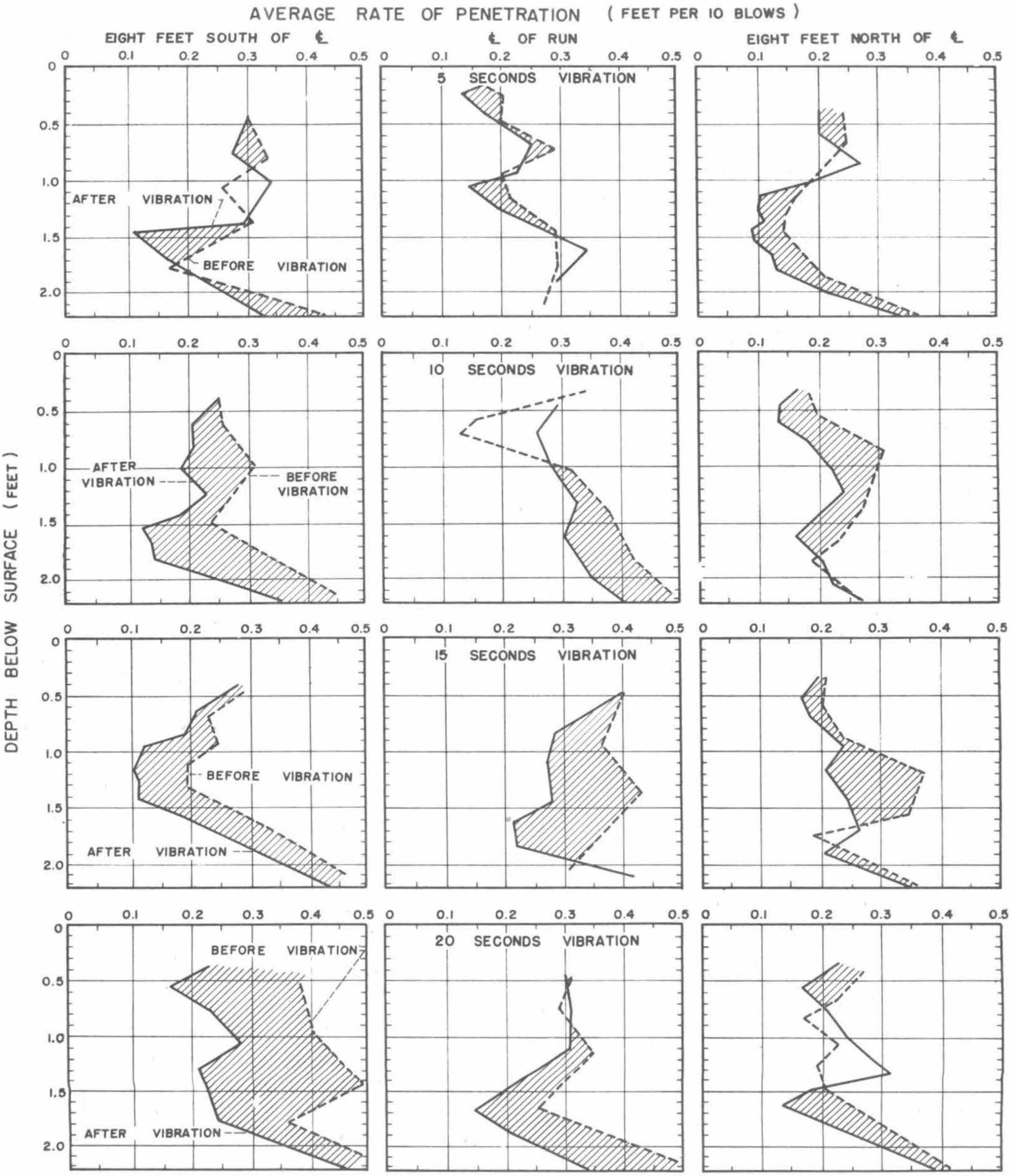


NOTES

VIBRATOR DEAD LOAD = 14,800 POUNDS

NINE ECCENTRICS PER SHAFT

COMPACTION TEST NO. 30° T 138-1



DEAD LOAD 14,800 LB. FREQUENCY 10.8 CPS.

DYNAMIC FORCE 11,800 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - TRIANGULAR

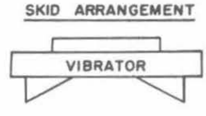
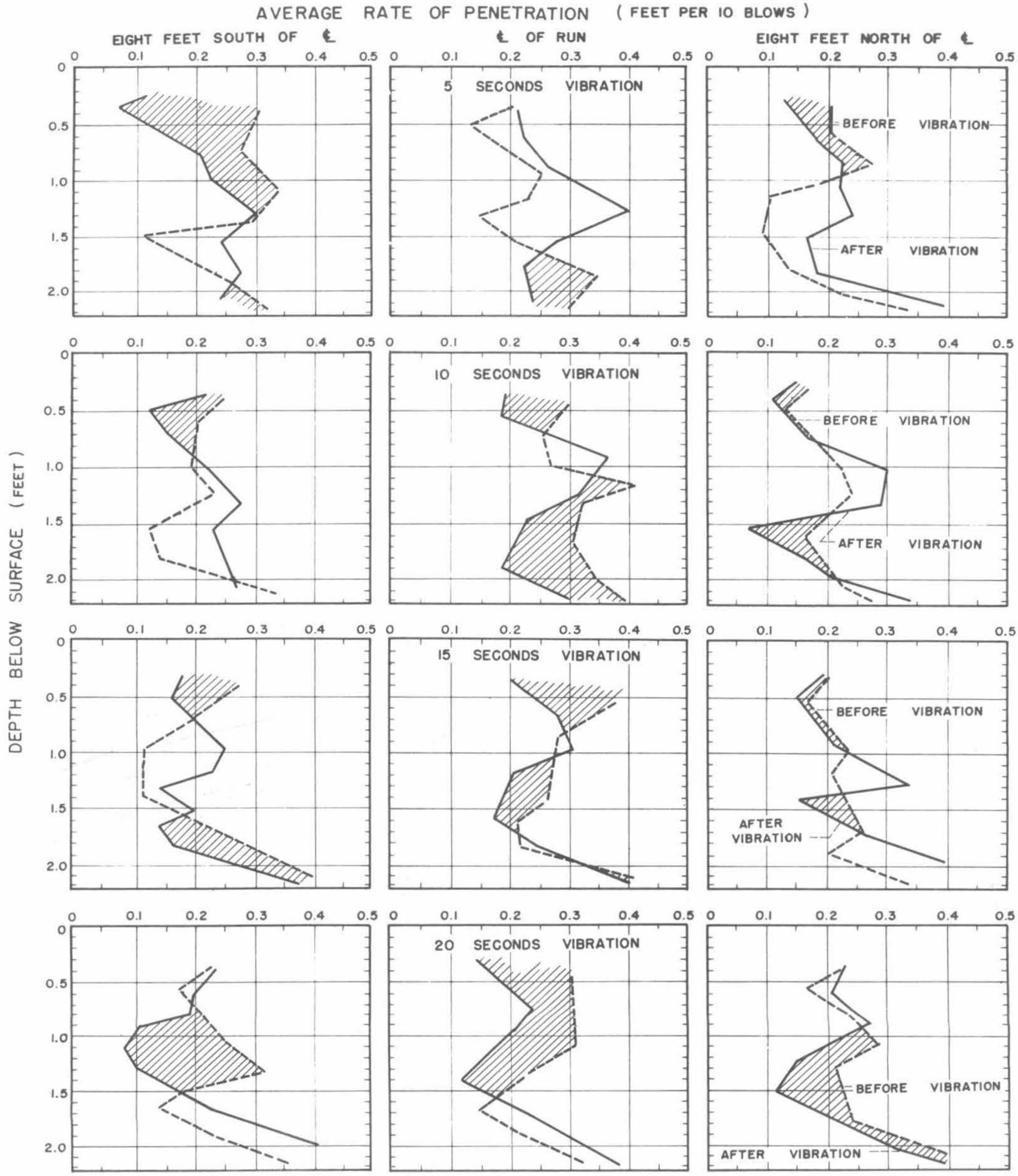


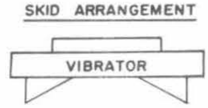
PLATE 45

COMPACTION TEST NO. 30° T 138-2



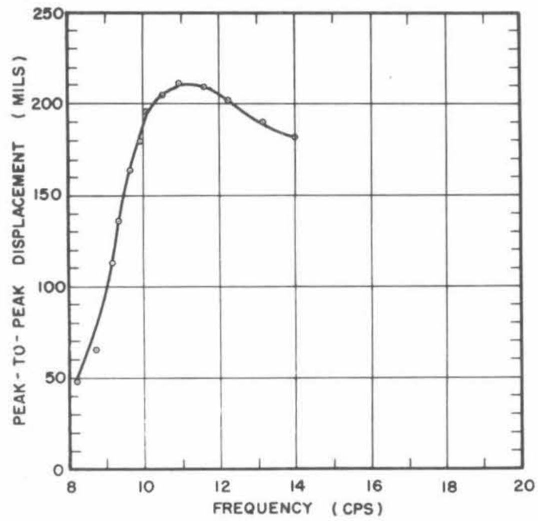
DEAD LOAD	14,800 LB.	FREQUENCY	10.8	CPS.
DYNAMIC FORCE	11,800 LB.	ECCENTRICS	9	PER SHAFT
TYPE OF BASE PLATE	TWO SKIDS – TRIANGULAR			

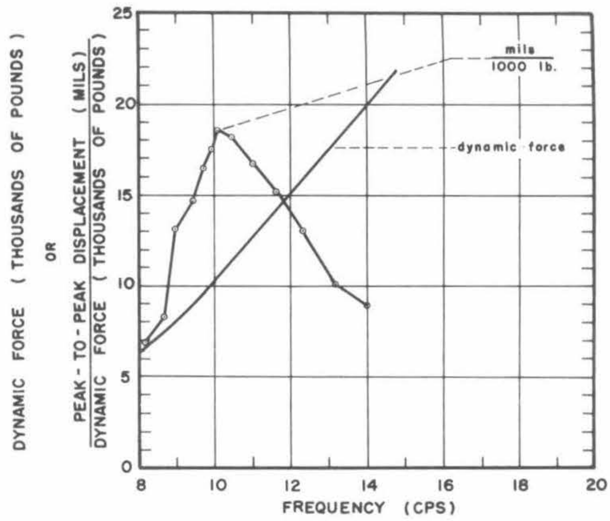
SKID ARRANGEMENT



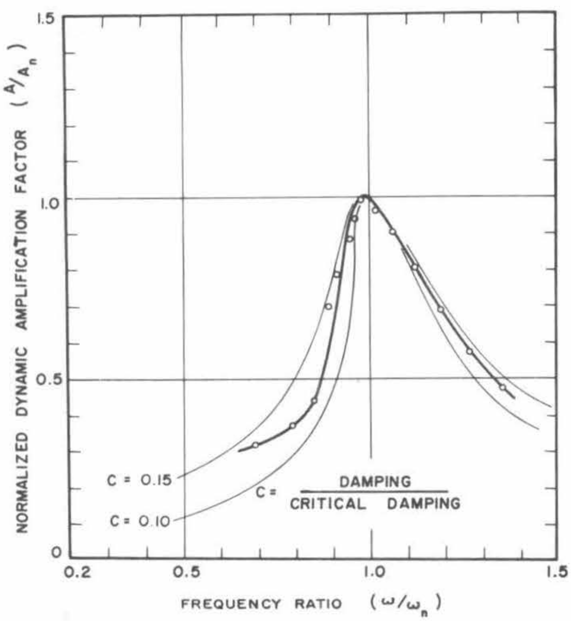
RUN NO. 30° T 39

VERTICAL DISPLACEMENT DATA

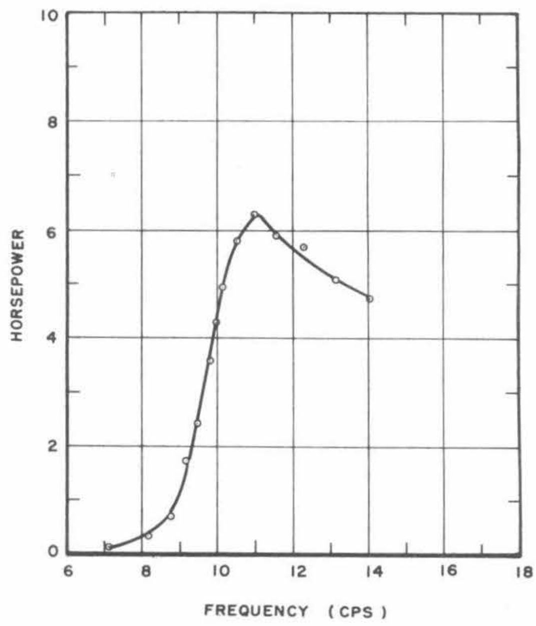




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

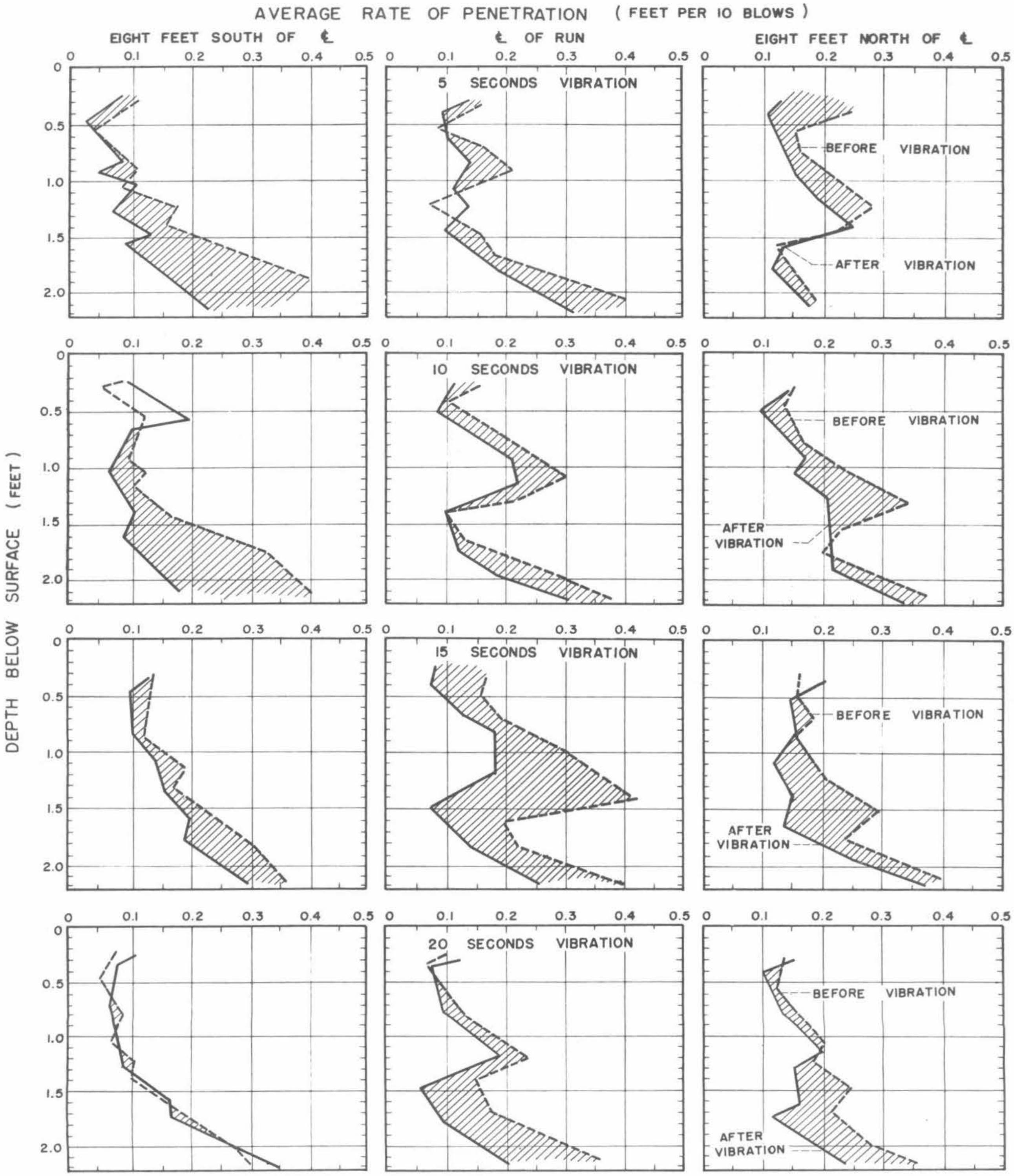


NOTES

VIBRATOR DEAD LOAD = 16,800 POUNDS

NINE ECCENTRICS PER SHAFT

COMPACTION TEST NO. 30° T 139-I



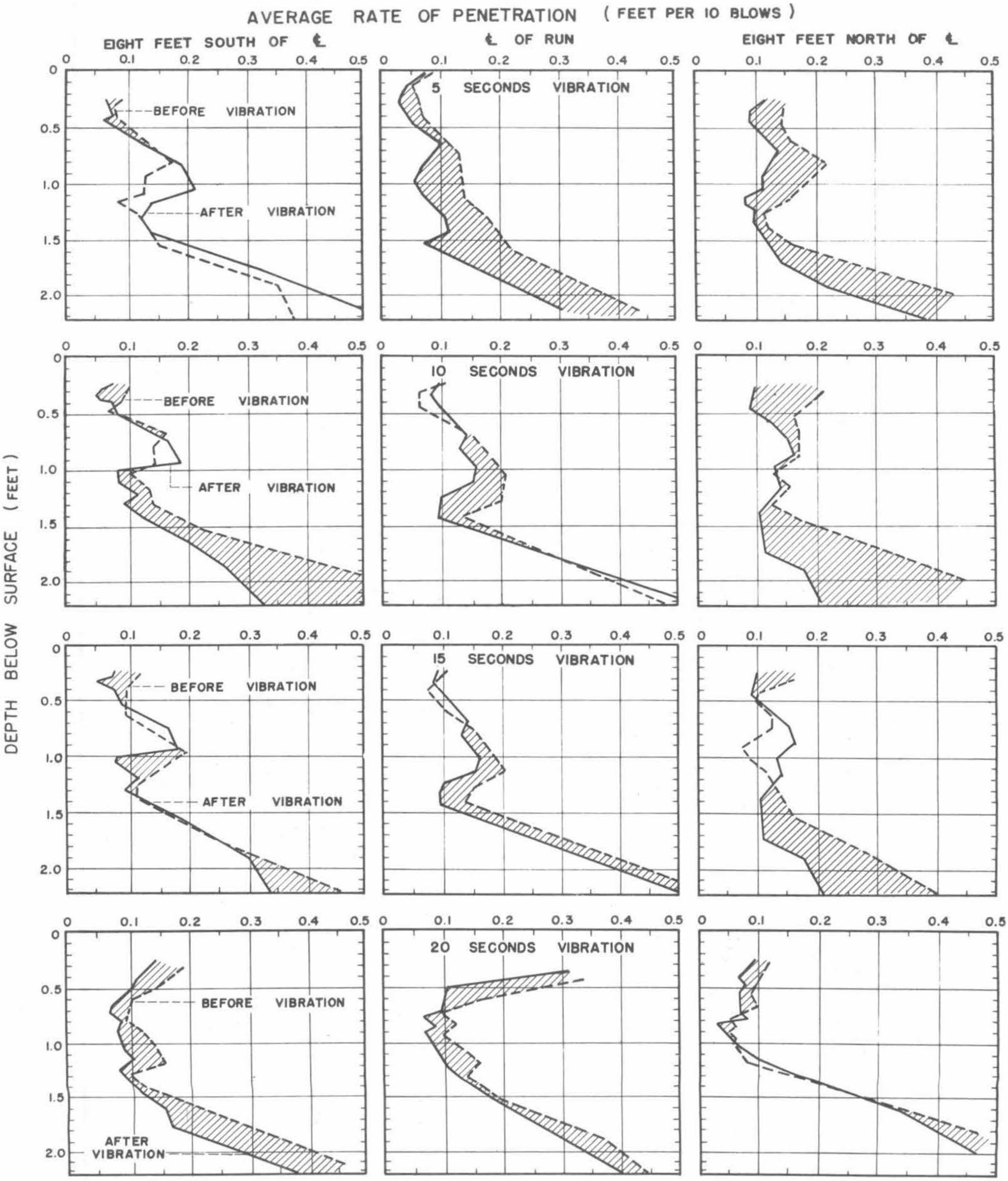
DEAD LOAD 16,800 LB. FREQUENCY 10.7 CPS.

DYNAMIC FORCE 11,900 LB. ECCENTRICS 9 PER SHAFT

TYPE OF BASE PLATE TWO SKIDS - TRIANGULAR

SKID ARRANGEMENT

COMPACTION TEST NO. 30° T 139-2

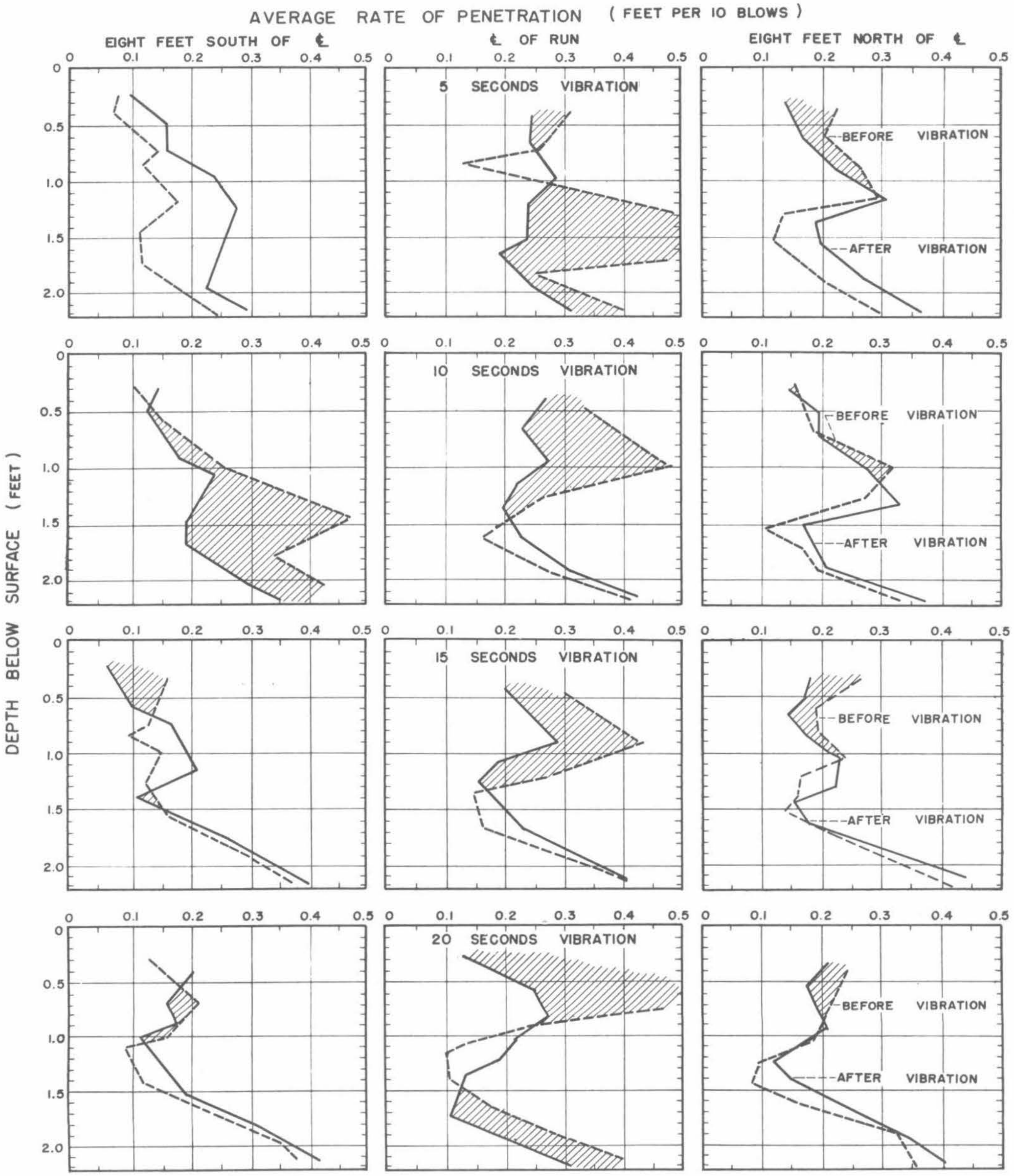


DEAD LOAD	16,800	LB.	FREQUENCY	10.7	CPS.
DYNAMIC FORCE	11,900	LB.	ECCENTRICS	9	PER SHAFT
TYPE OF BASE PLATE	TWO SKIDS - TRIANGULAR				

SKID ARRANGEMENT

PLATE 49

COMPACTION TEST NO. 30° T 139-3



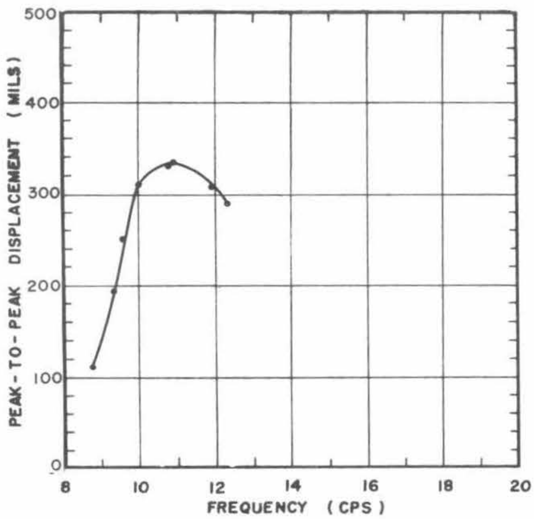
DEAD LOAD 16,800 LB.
DYNAMIC FORCE 11,900 LB.
TYPE OF BASE PLATE TWO SKIDS - TRIANGULAR

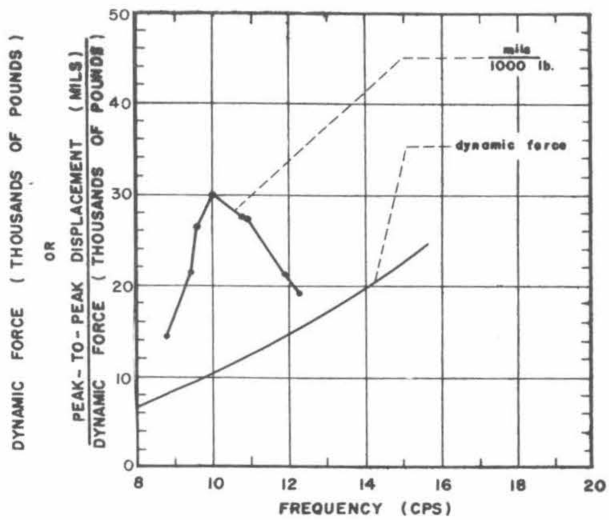
FREQUENCY 10.7 CPS.
ECCENTRICS 9 PER SHAFT

SKID ARRANGEMENT

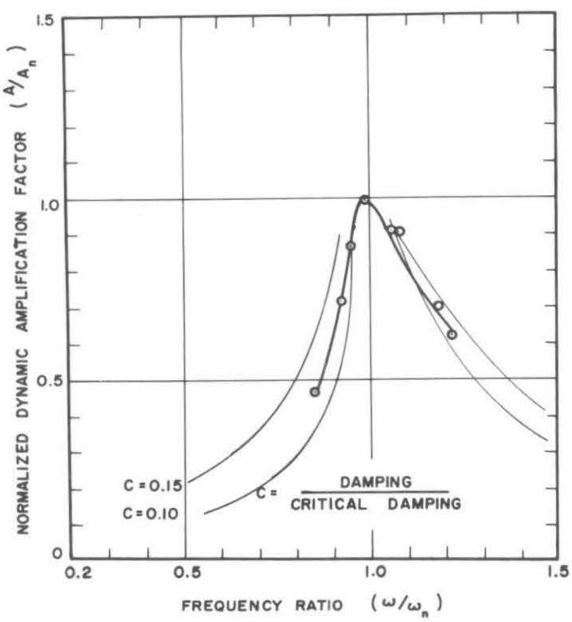
RUN NO. 15° T 40

VERTICAL DISPLACEMENT DATA

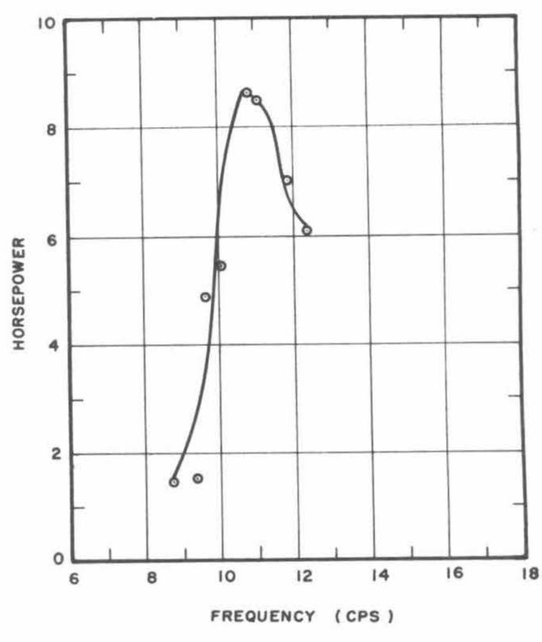




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL

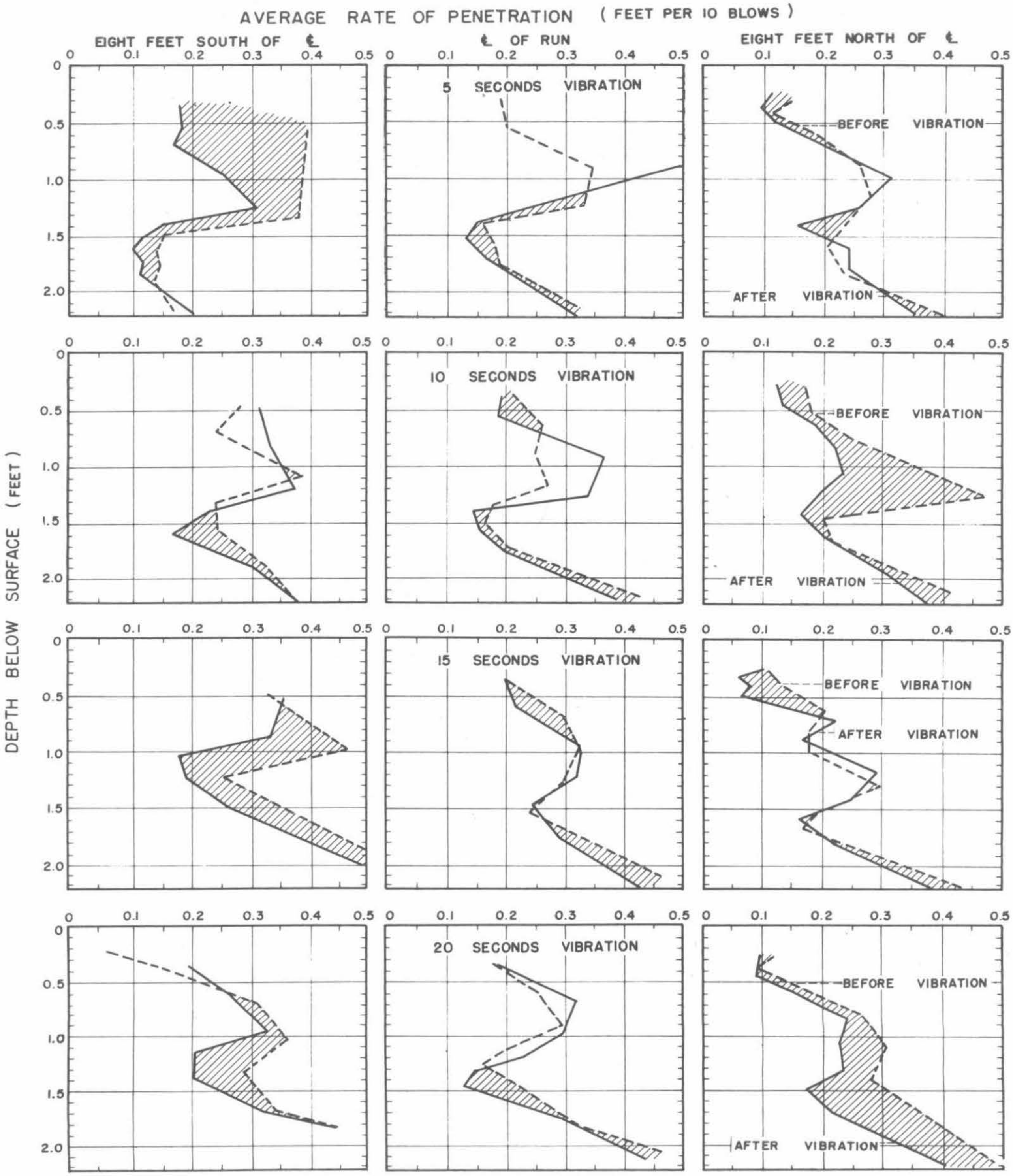


NOTES

VIBRATOR DEAD LOAD = 11,600 POUNDS

NINE ECCENTRICS PER SHAFT

COMPACTION TEST NO. 15° T 140

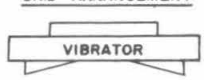


DEAD LOAD 11,600 LB. FREQUENCY 10.9 CPS.

DYNAMIC FORCE 12,100 LB. ECCENTRICS 9 PER SHAFT

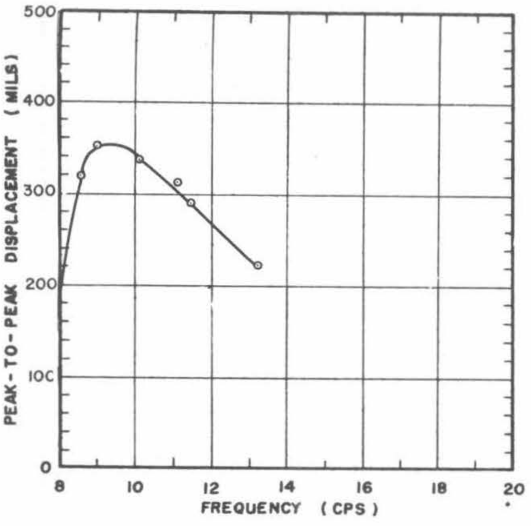
TYPE OF BASE PLATE TWO SKIDS – TRIANGULAR

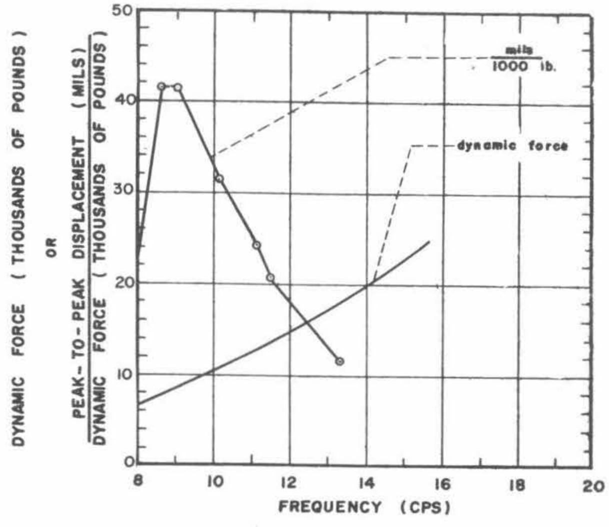
SKID ARRANGEMENT



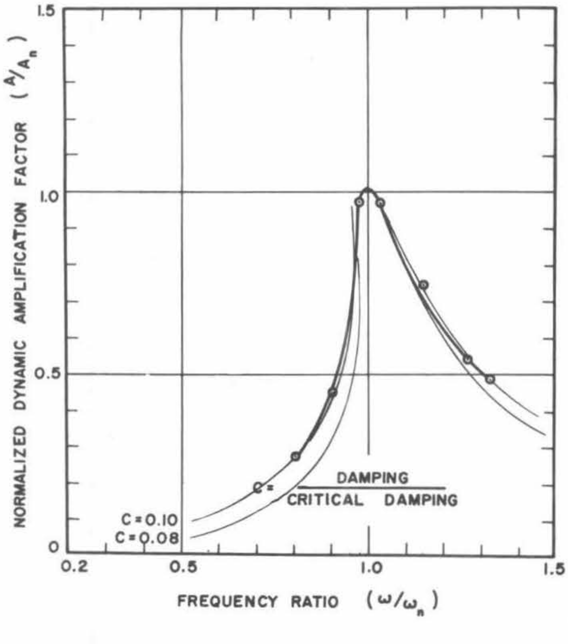
RUN NO. 45° T 40

VERTICAL DISPLACEMENT DATA

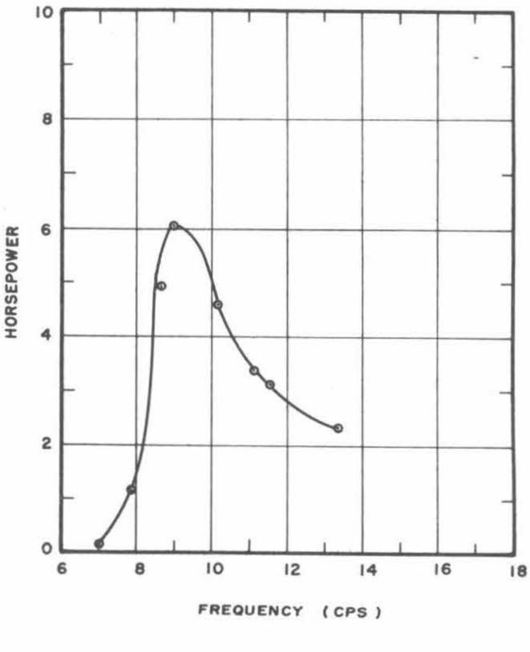




NORMALIZED DYNAMIC AMPLIFICATION FACTOR CURVE



AVERAGE POWER DISSIPATED IN DAMPING IN THE SOIL



NOTES

VIBRATOR DEAD LOAD = 11,600 POUNDS

NINE ECCENTRICS PER SHAFT

APPENDIX D

DATA FROM FIELD TESTS
AT
CALIFORNIA INSTITUTE OF TECHNOLOGY

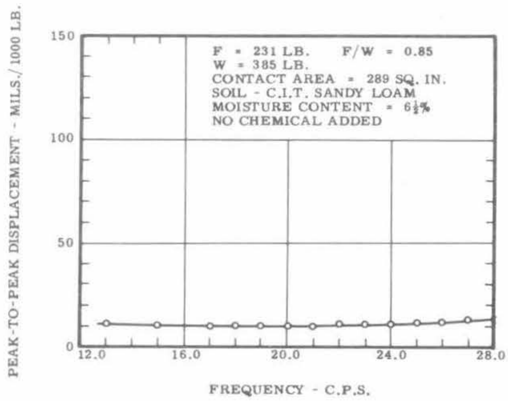


Fig. 2.19 TEST L1-1 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

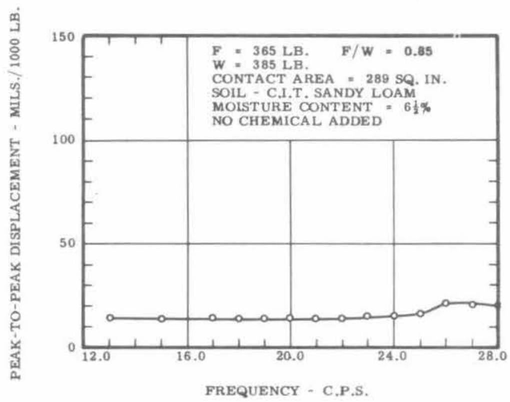


Fig. 2.20 TEST L1-2 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

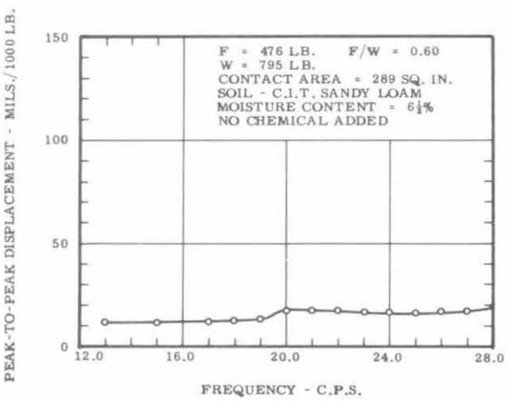


Fig. 2.21 TEST L1-3 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

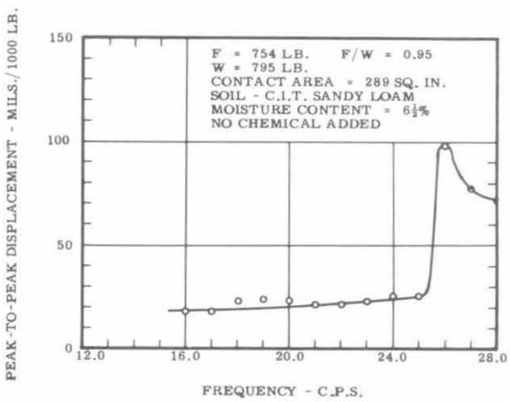


Fig. 2.22 TEST L1-4 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

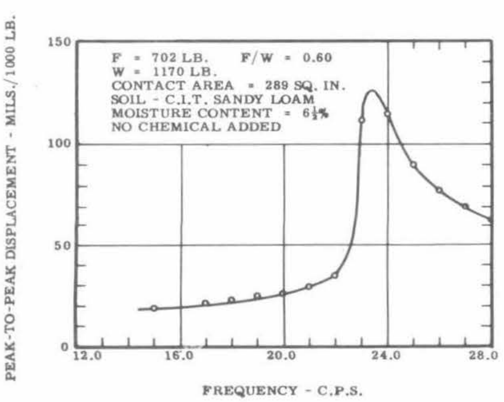


Fig. 2.23 TEST L1-5 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

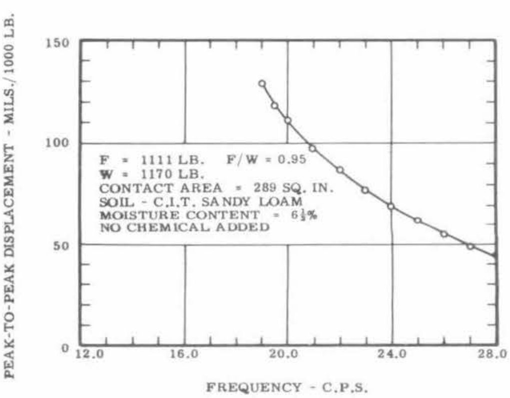


Fig. 2.24 TEST L1-6 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

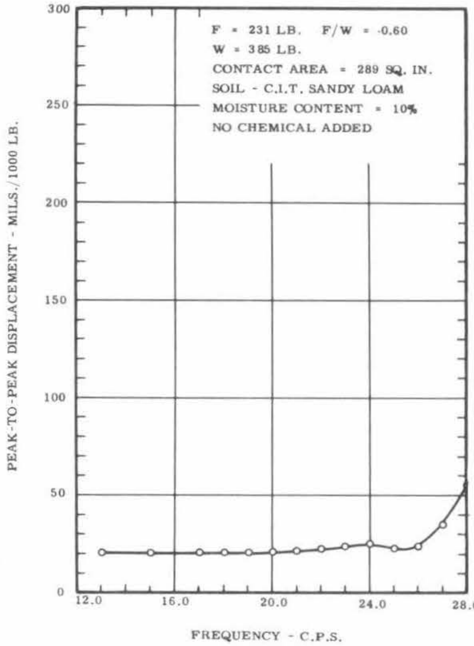


Fig. 2.25 TEST L1-7 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

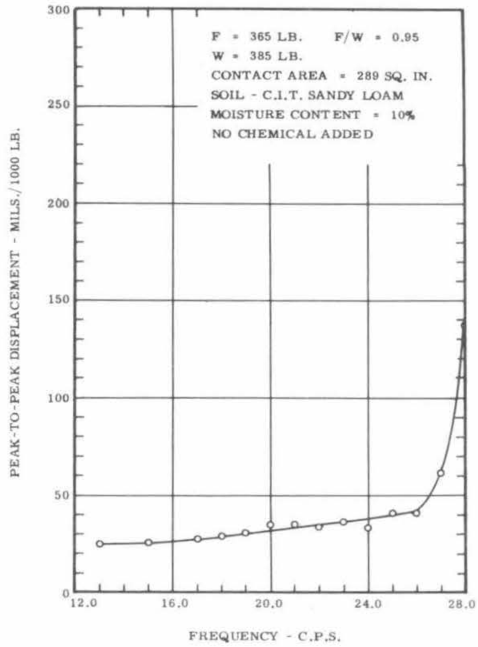


Fig. 2.26 TEST L1-8 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

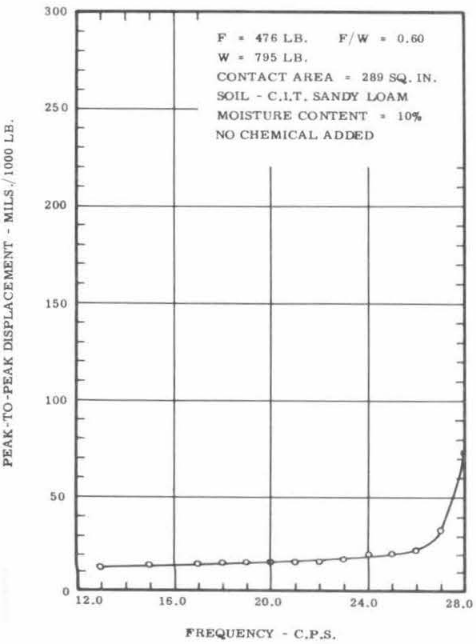


Fig. 2.27 TEST L1-9 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

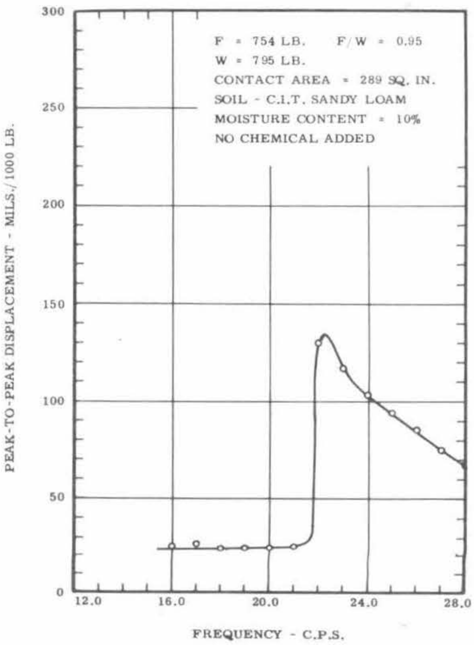


Fig. 2.28 TEST L1-10 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

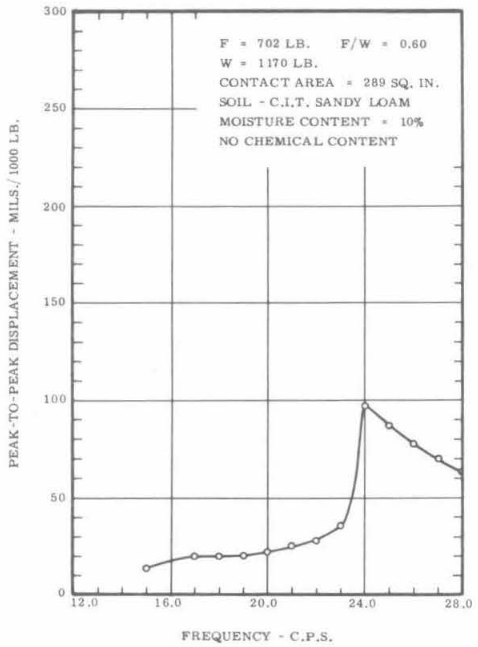


Fig. 2.29 TEST L1-11 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

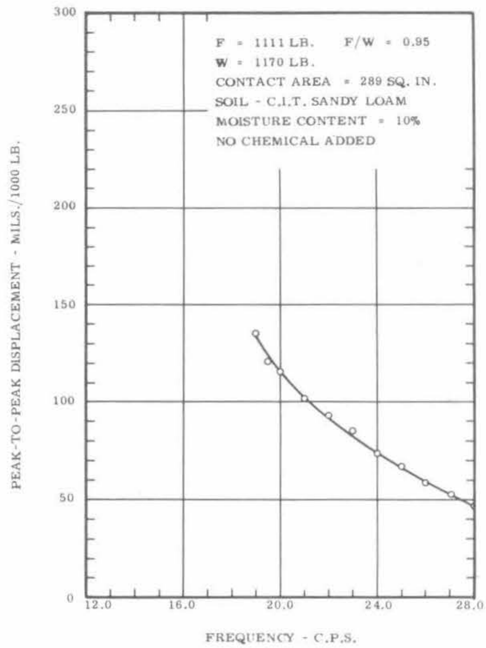


Fig. 2.30 TEST L1-12 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

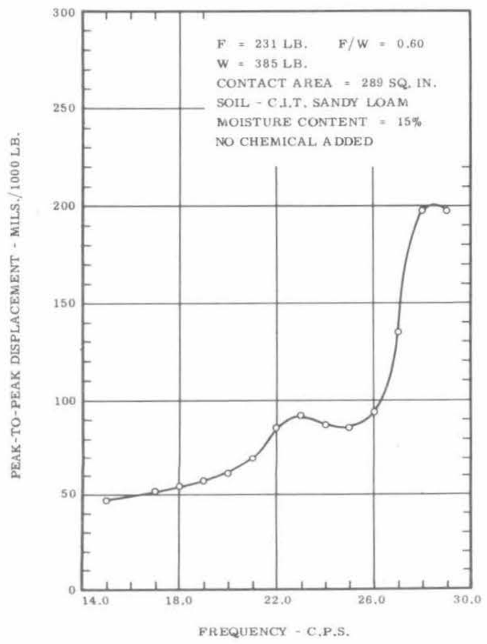


Fig. 2.31 TEST L1-13 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

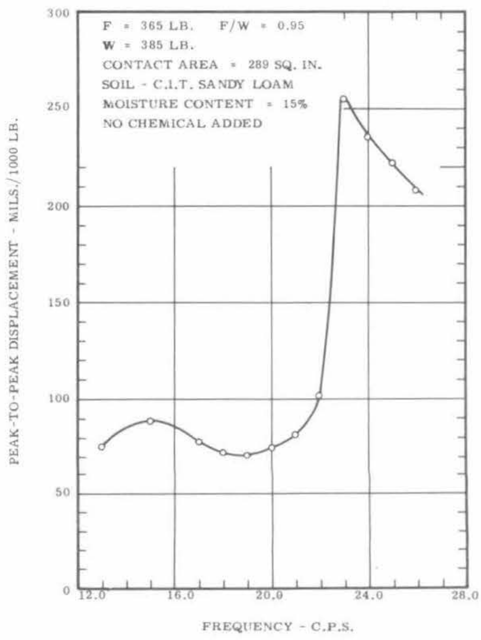


Fig. 2.32 TEST L1-14 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

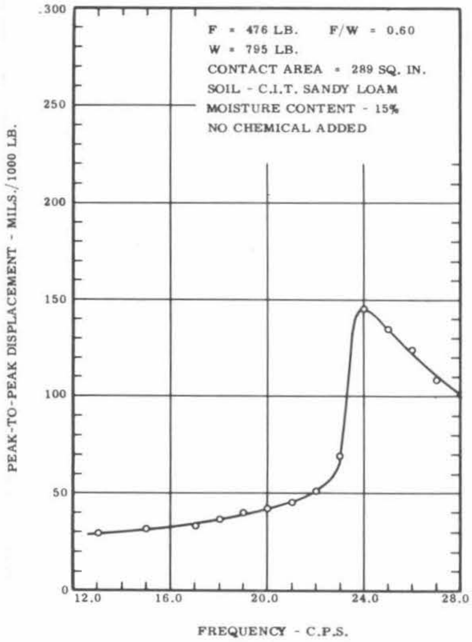


Fig. 2.33 TEST L1-15 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

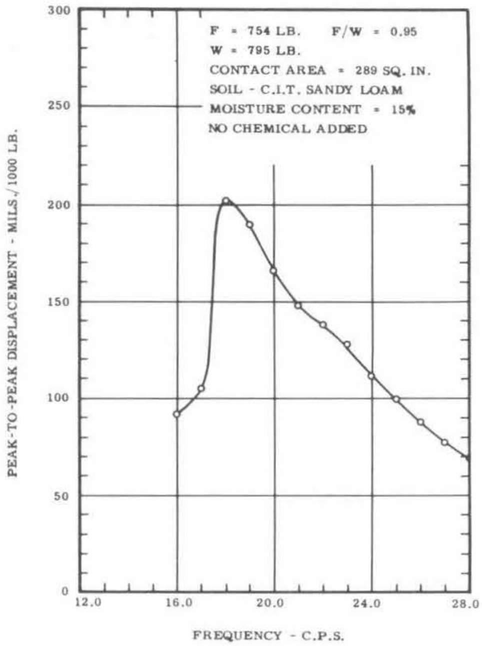


Fig. 2.34 TEST L1-16 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

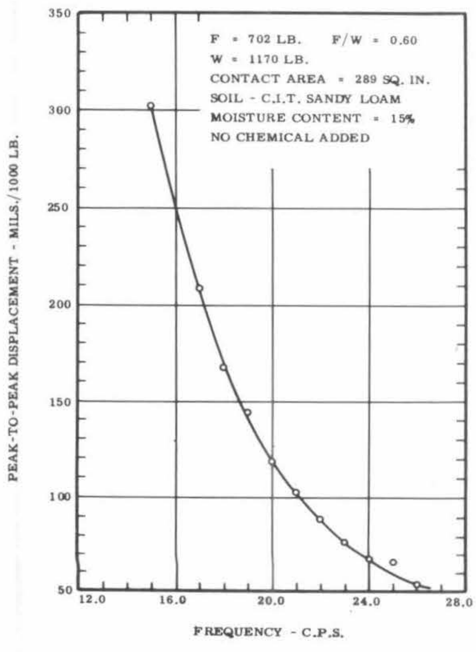


Fig. 2.35 TEST L1-17 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

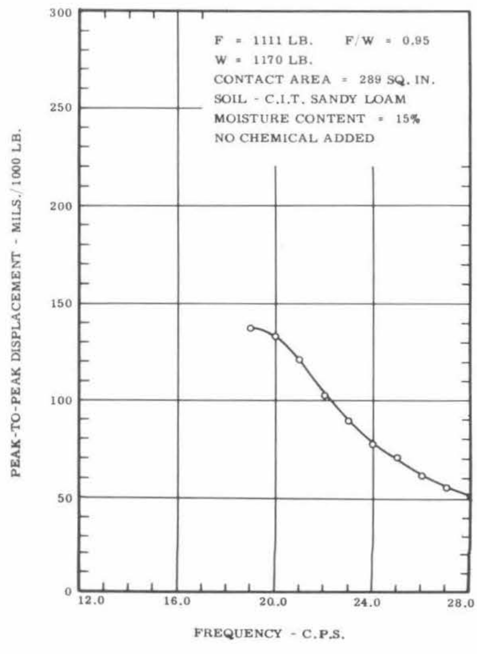


Fig. 2.36 TEST L1-18 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

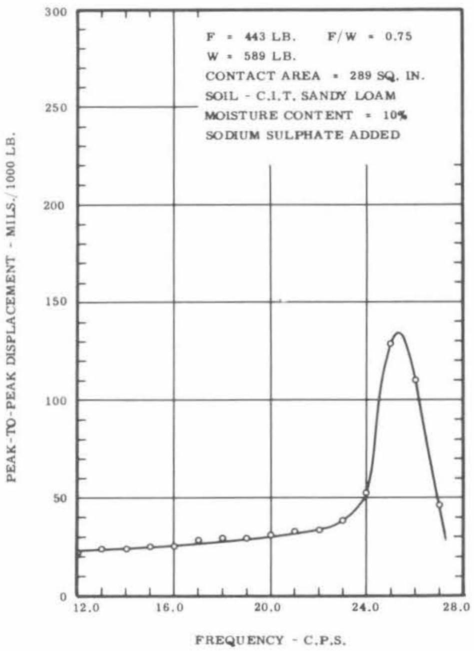


Fig. 2.37 TEST L2-1 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

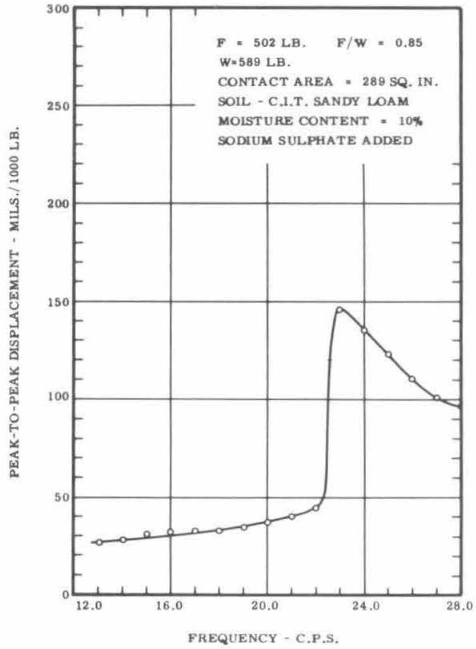


Fig. 2.38 TEST L2-2 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

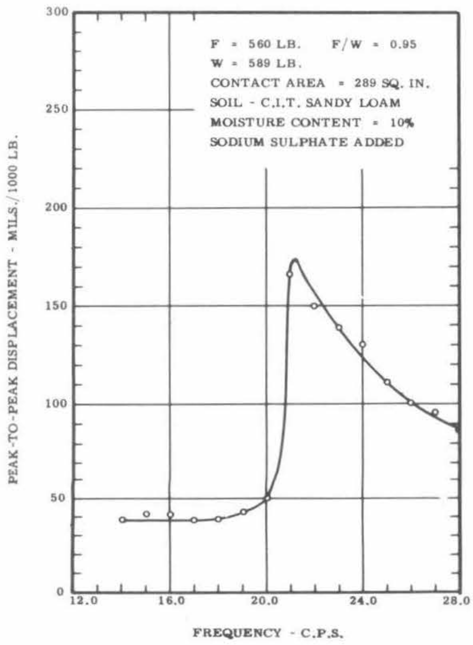


Fig. 2.39 TEST L2-3 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

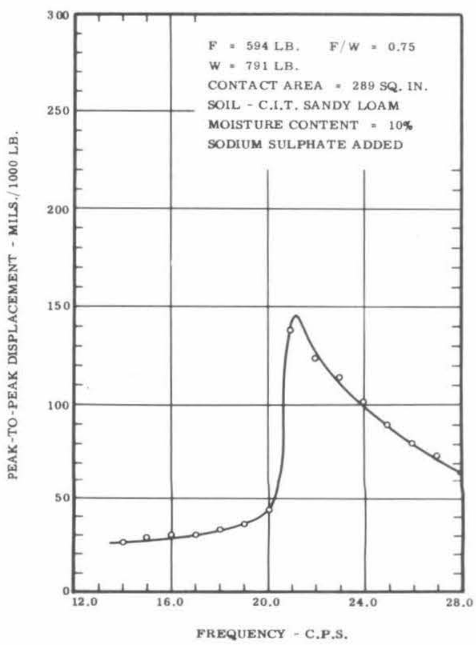


Fig. 2.40 TEST L2-4 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

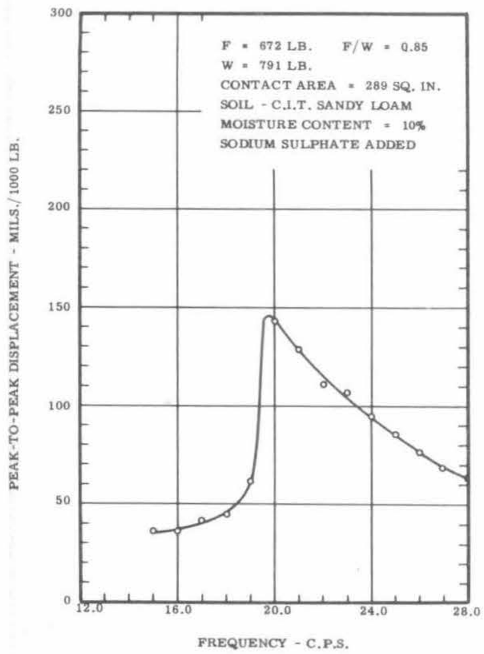


Fig. 2.41 TEST L2-5 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

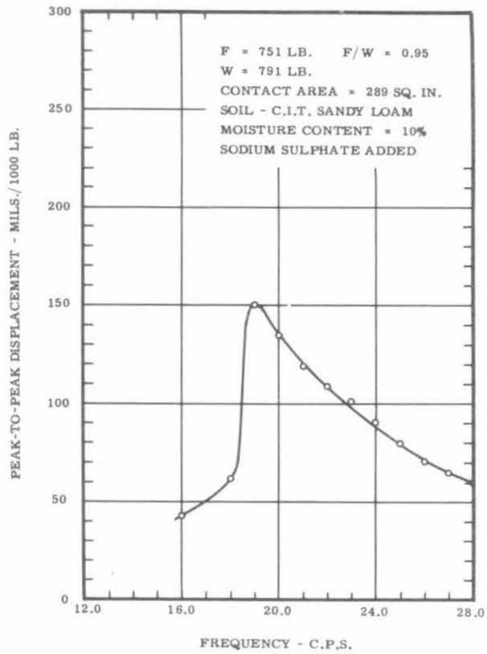


Fig. 2.42 TEST L2-6 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

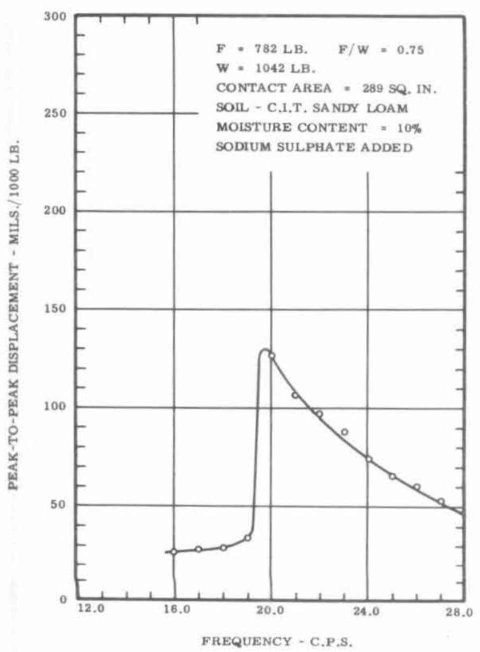


Fig. 2.43 TEST L2-7 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

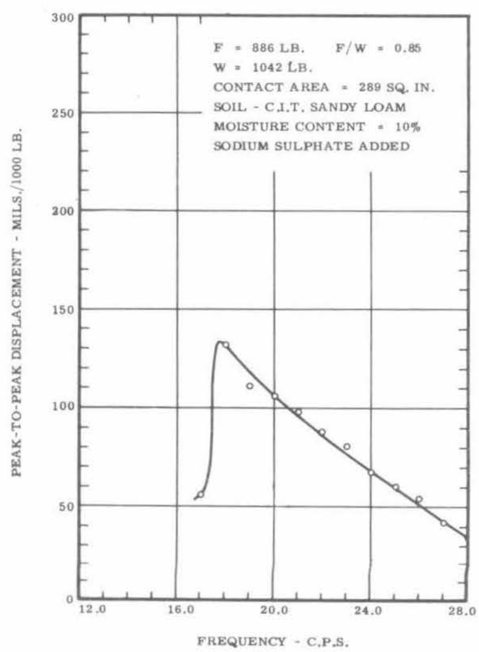


Fig. 2.44 TEST L2-8 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

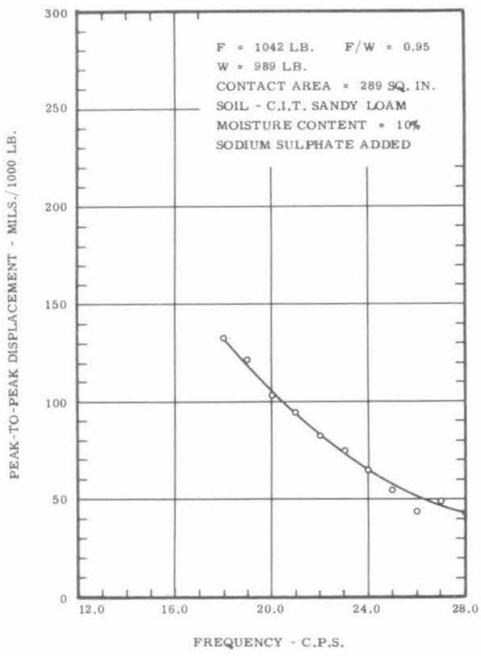


Fig. 2.45 TEST L2-9 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

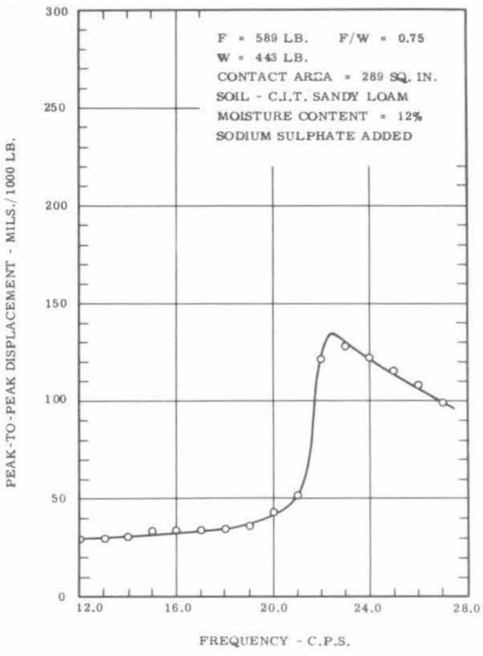


Fig. 2.46 TEST L2-10 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

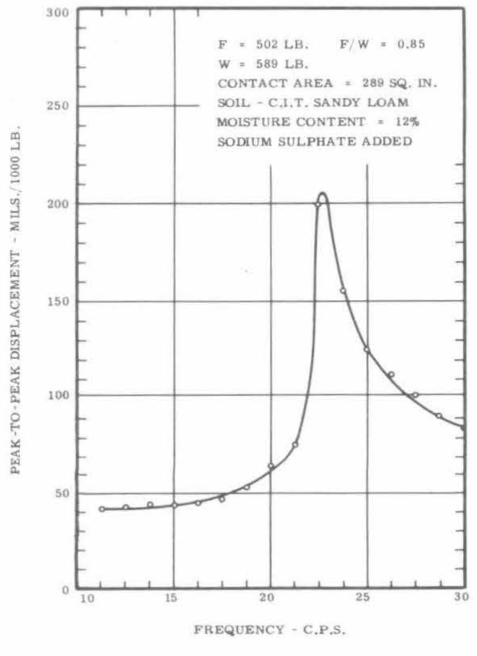


Fig. 2.47 TEST L2-11 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

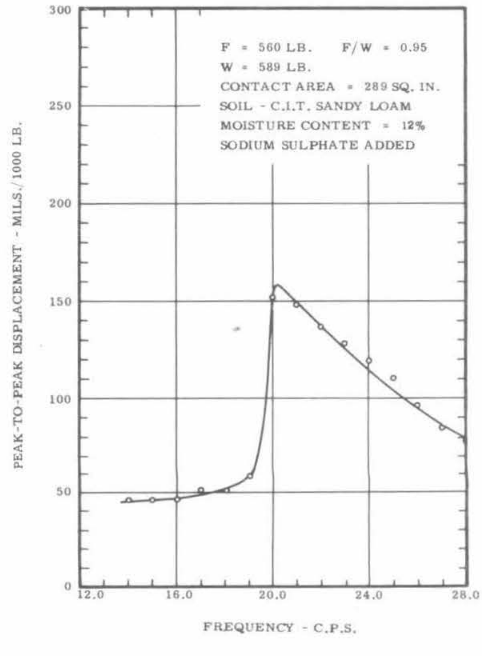


Fig. 2.48 TEST L2-12 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

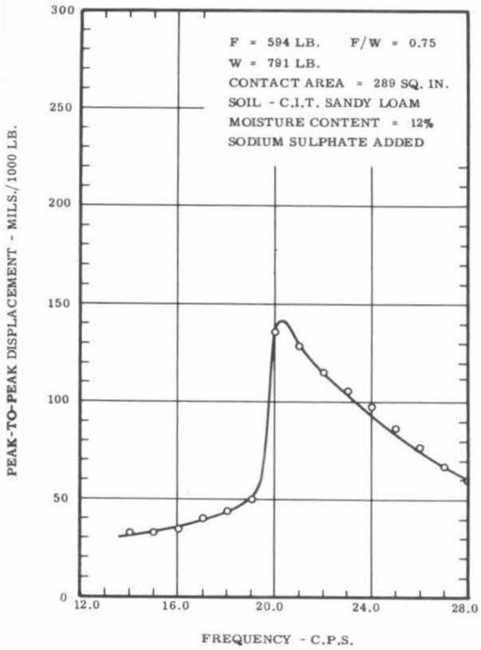


Fig. 2.49 TEST L2-13 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

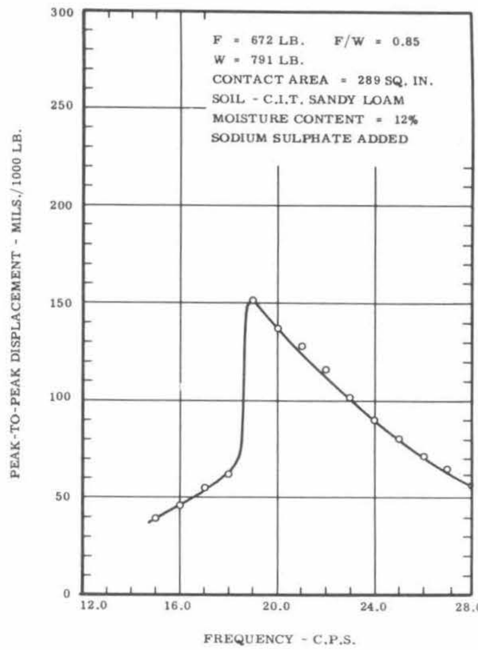


Fig. 2.50 TEST L2-14 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

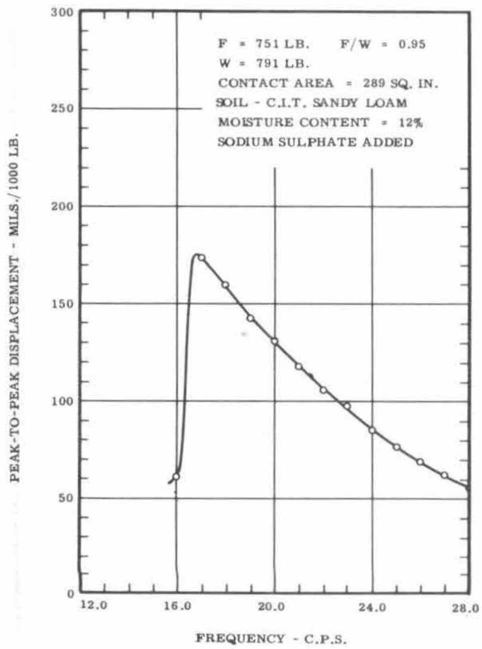


Fig. 2.51 TEST L2-15 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

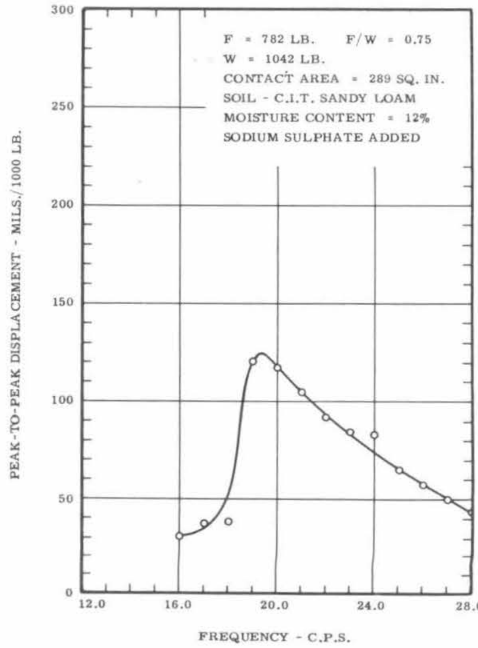


Fig. 2.52 TEST L2-16 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

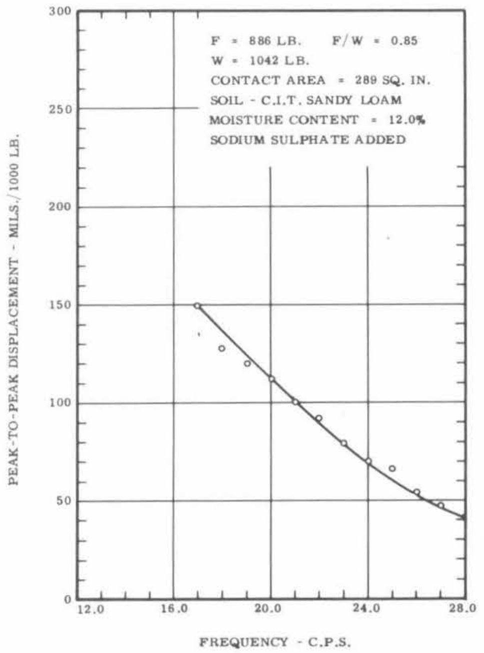


Fig. 2.53 TEST L2-17 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

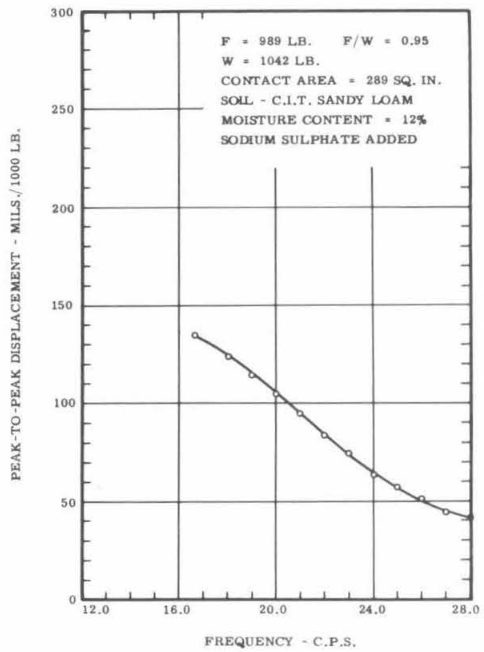


Fig. 2.54 TEST L2-18 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

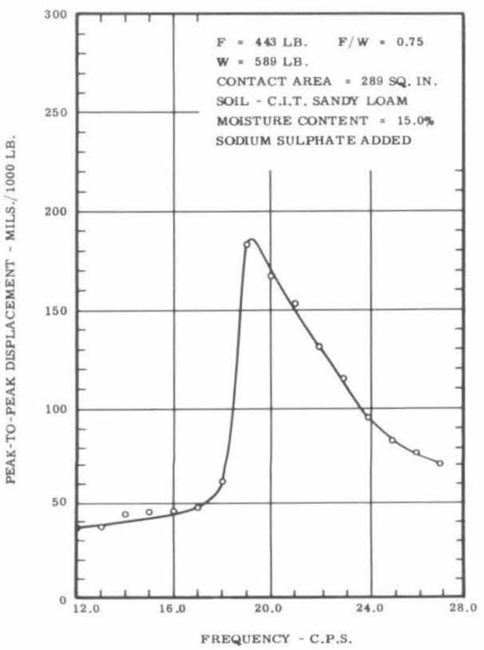


Fig. 2.55 TEST L2-19 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

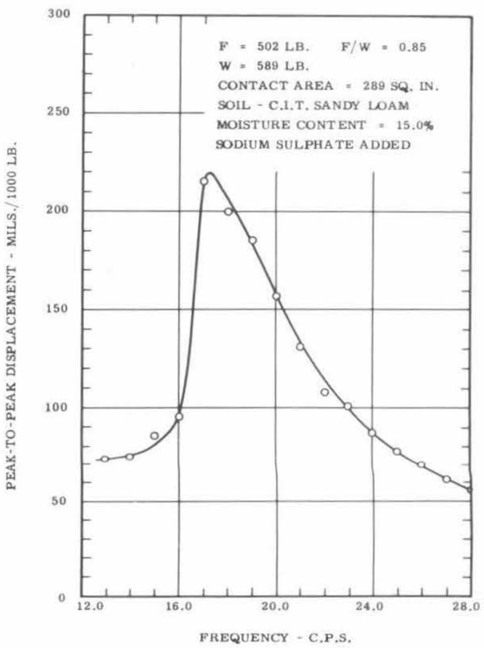


Fig. 2.56 TEST L2-20 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

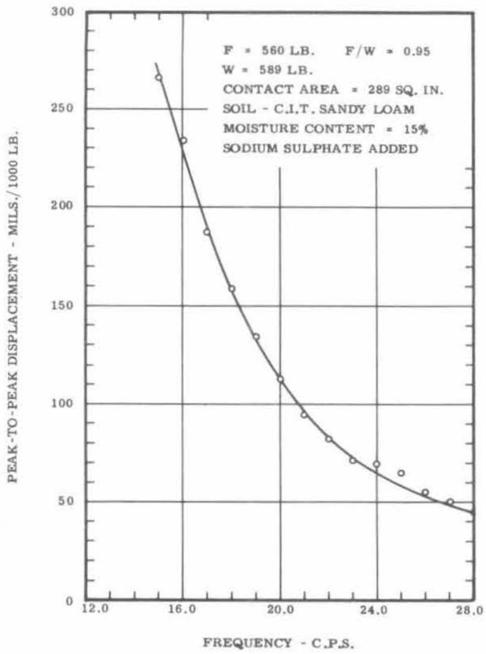


Fig. 2.57 TEST L2-21 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

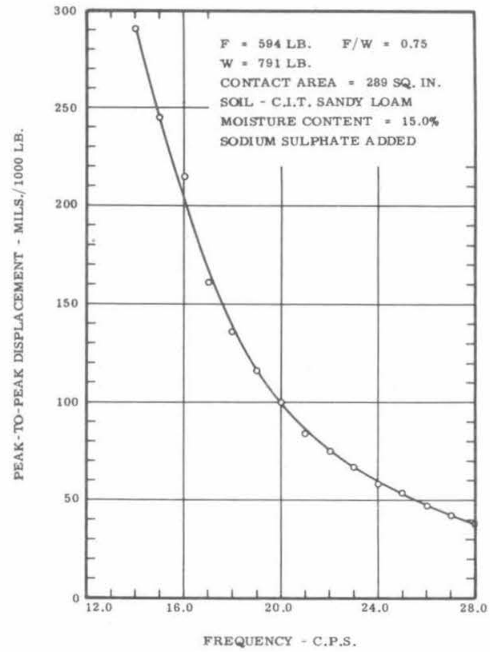


Fig. 2.58 TEST L2-22 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

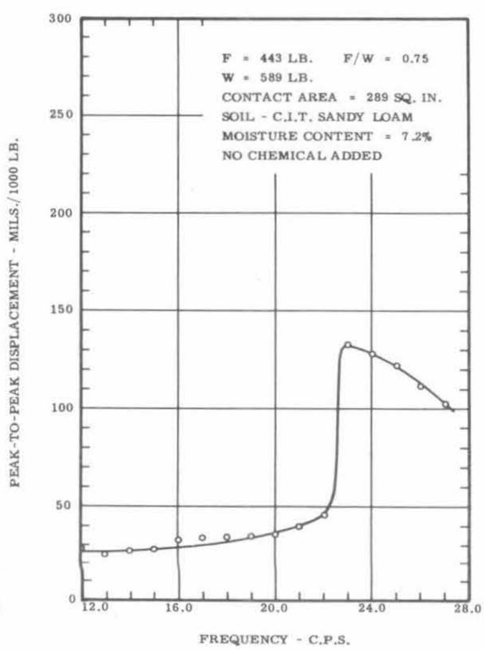


Fig. 2.59 TEST L3-1 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

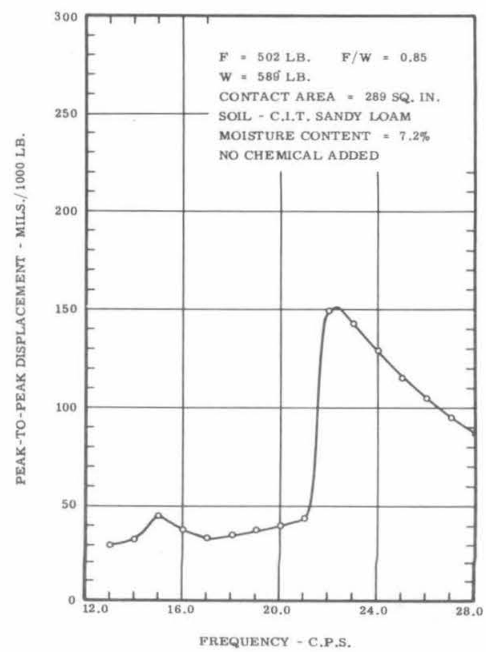


Fig. 2.60 TEST L3-2 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

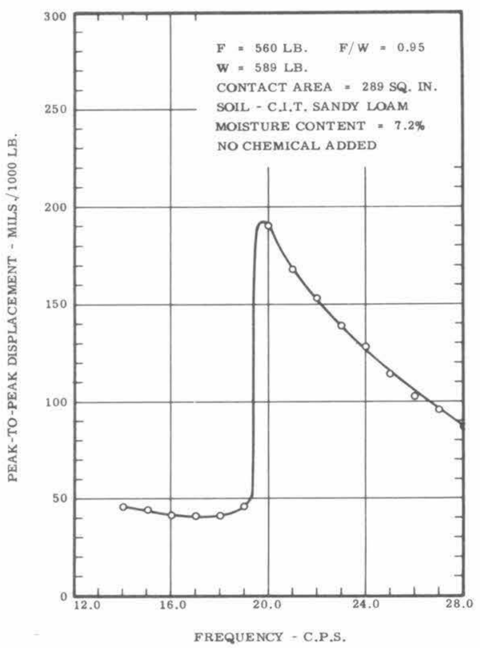


Fig. 2.61 TEST L3-3 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

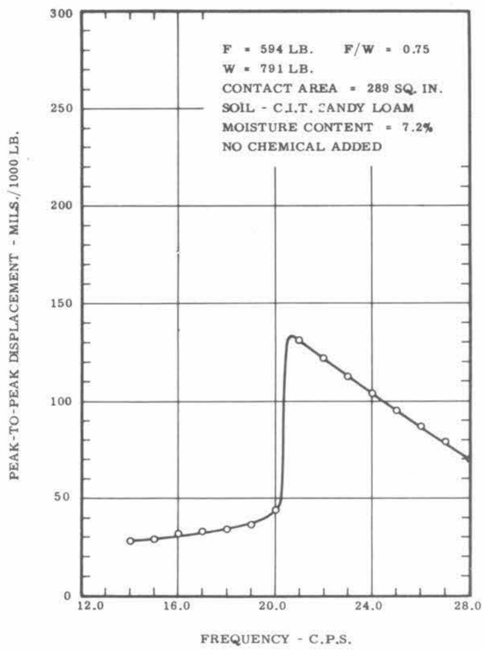


Fig. 2.62 TEST L3-4 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

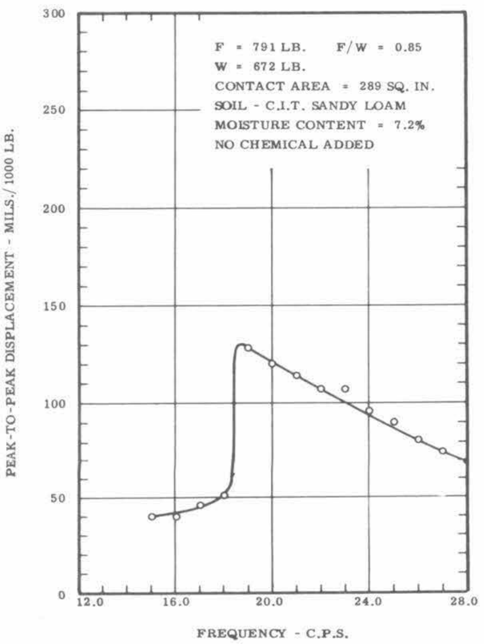


Fig. 2.63 TEST L3-3 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

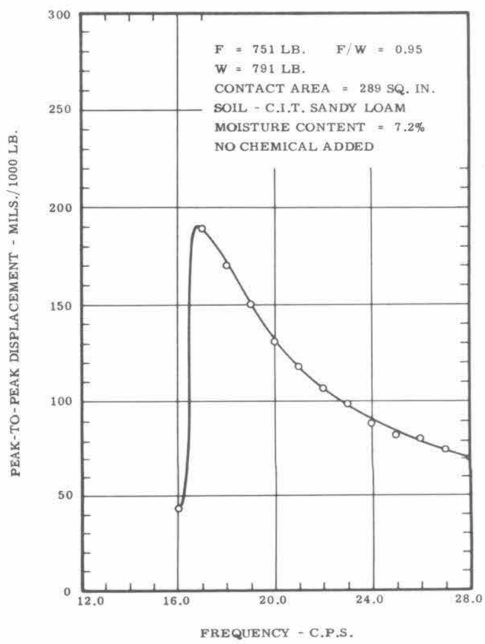


Fig. 2.64 TEST L3-6 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

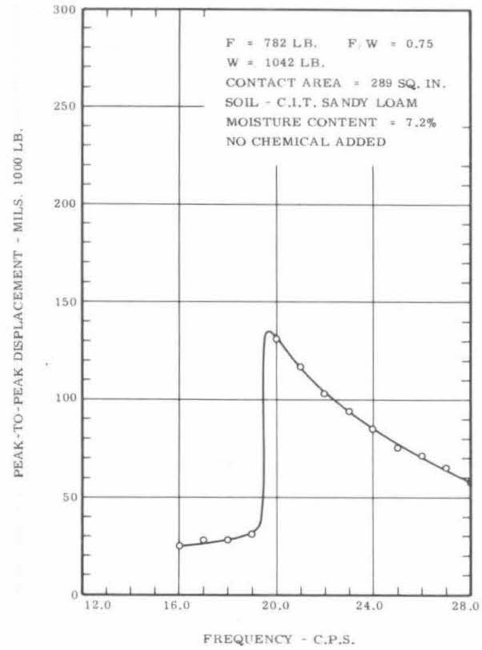


Fig. 2.65 TEST L3-7 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

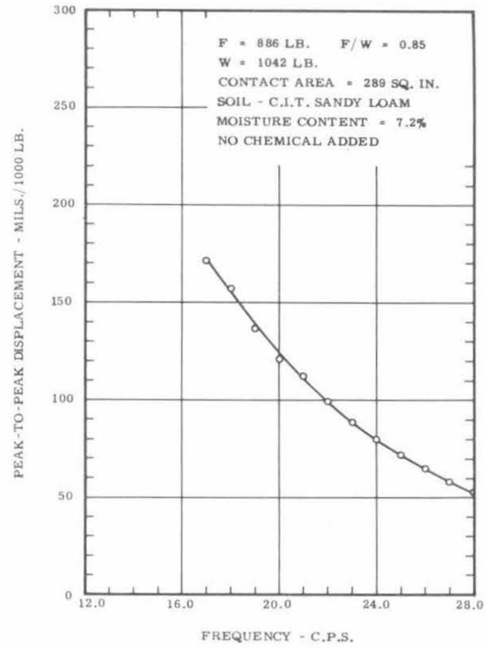


Fig. 2.66 TEST L3-8 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

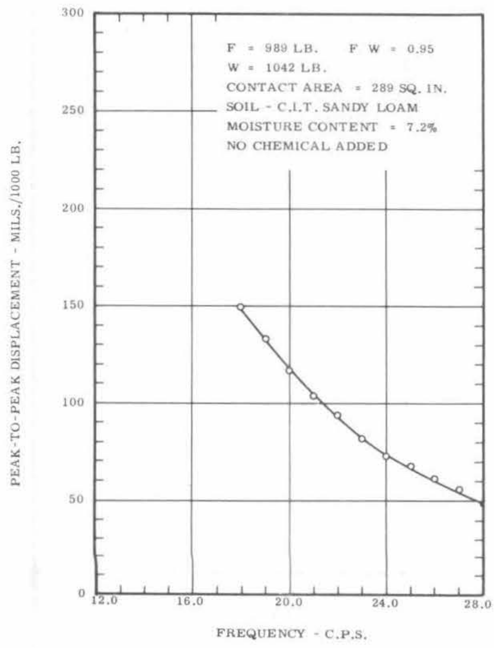


Fig. 2.67 TEST L3-9 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

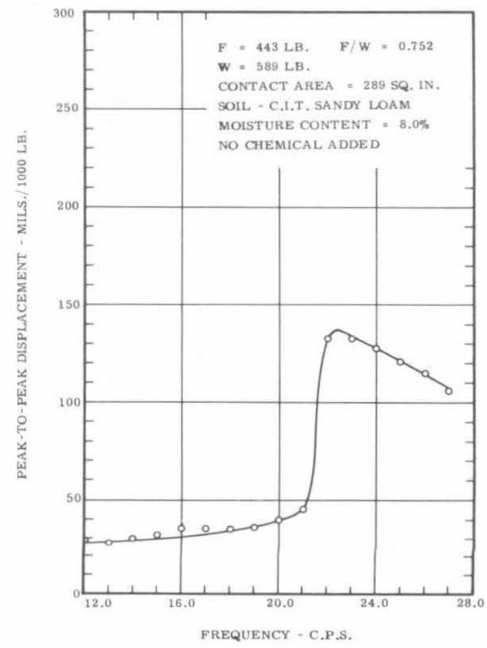


Fig. 2.68 TEST L3-10 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

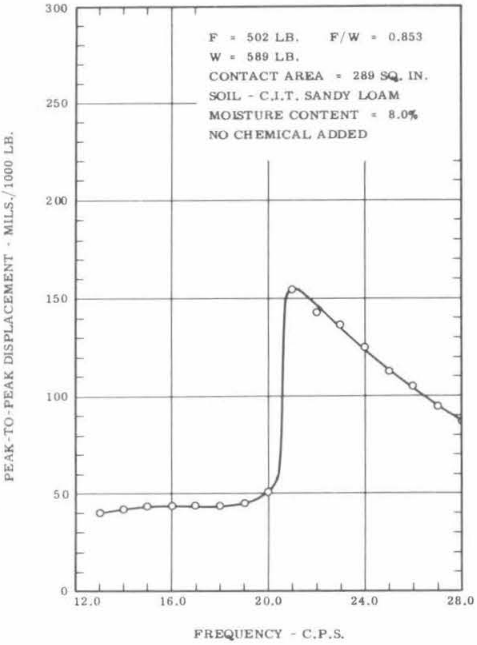


Fig. 2.69 TEST L3-11 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

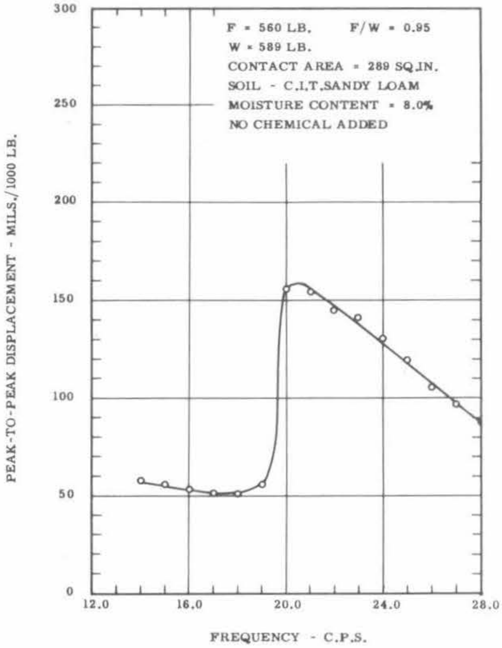


Fig. 2.70 TEST L3-12 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

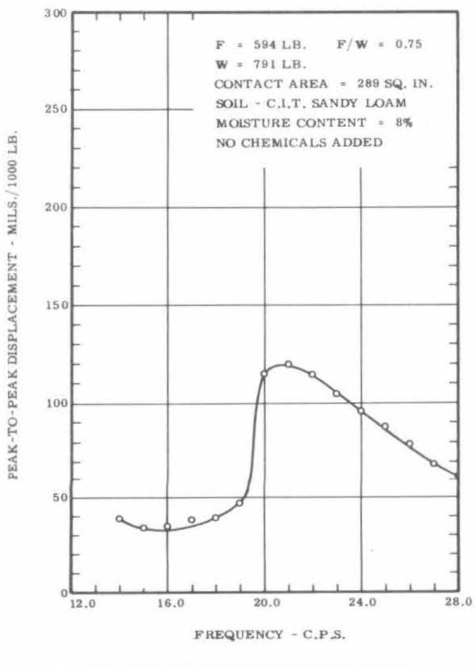


Fig. 2.71 TEST L3-13 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

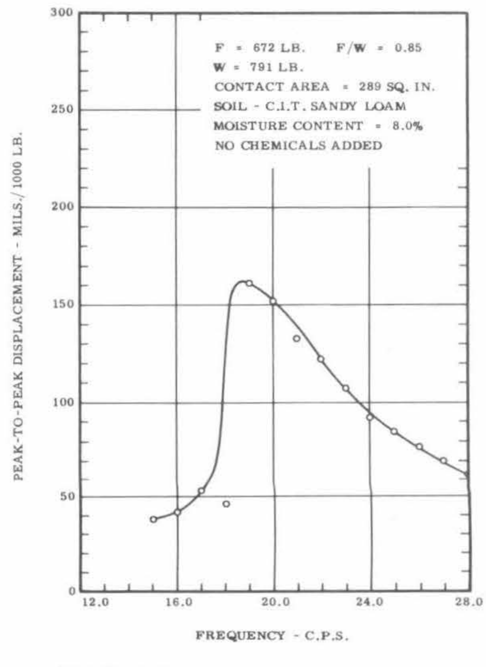


Fig. 2.72 TEST L3-14 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

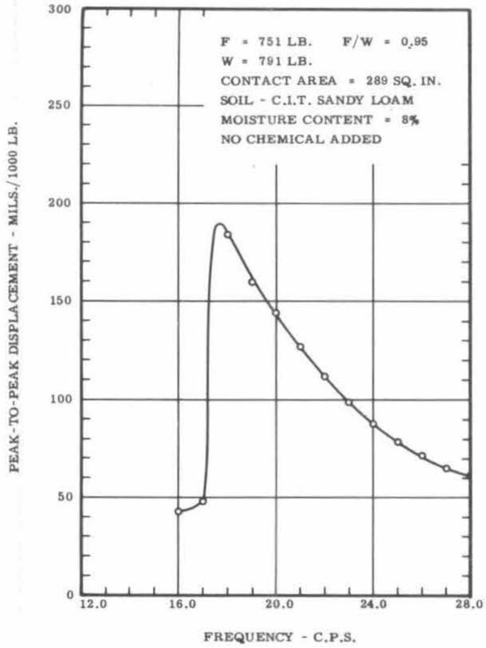


Fig. 2.73 TEST L3-15 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

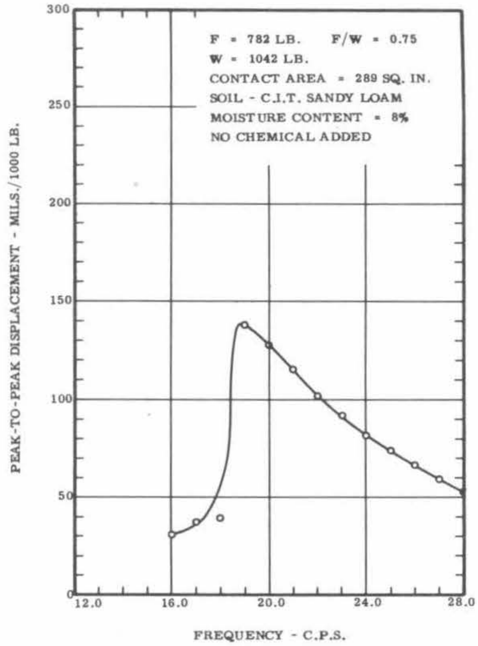


Fig. 2.74 TEST L3-16 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

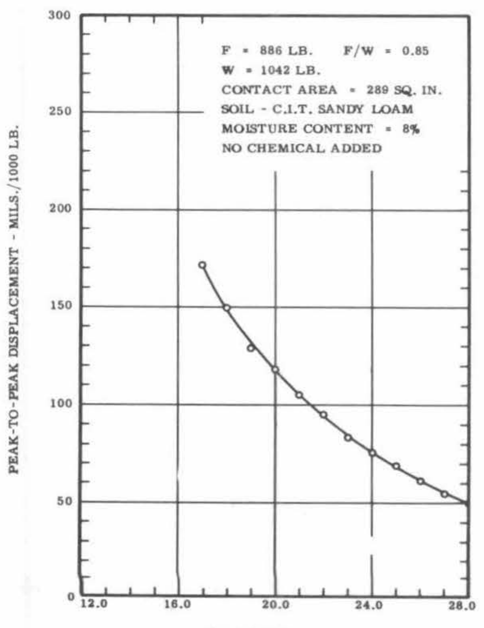


Fig. 2.75 TEST L3-17 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

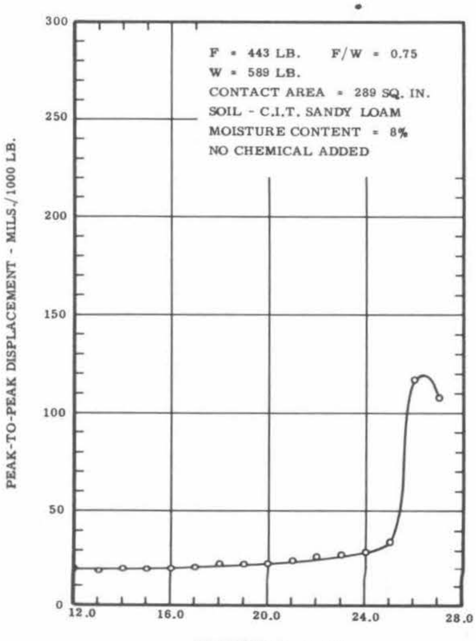


Fig. 2.76 TEST L3-18 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

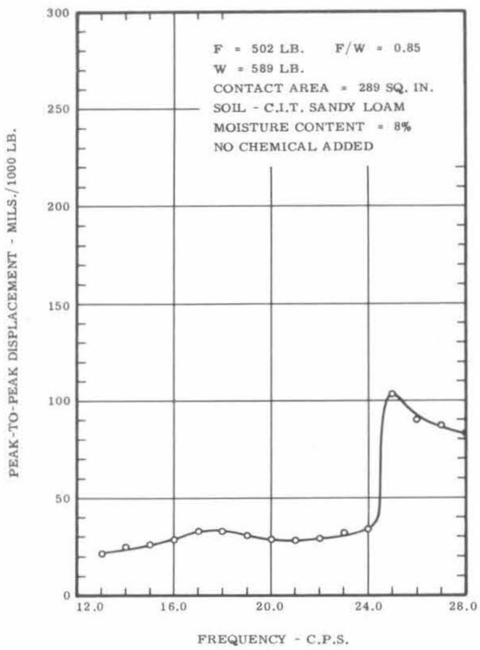


Fig. 2.77 TEST L3-19 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

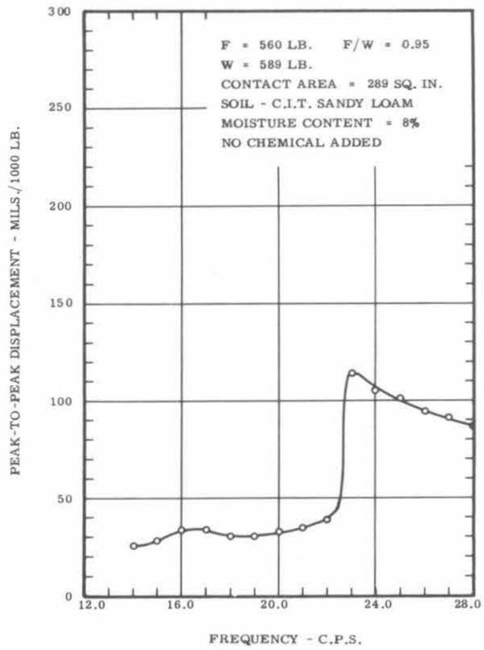


Fig. 2.78 TEST L3-20 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

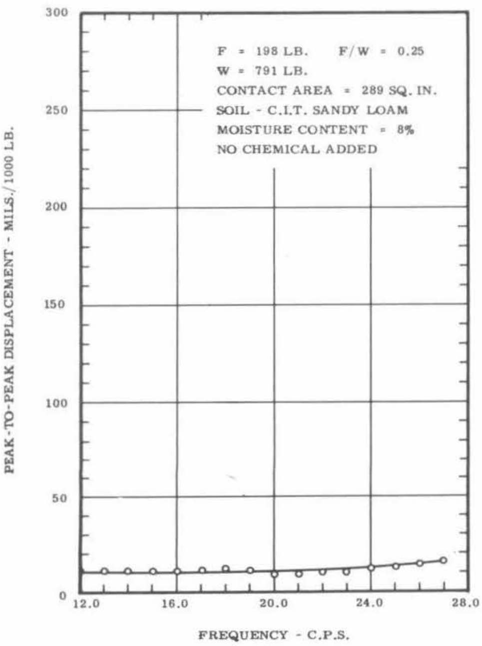


Fig. 2.79 TEST L3-21 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

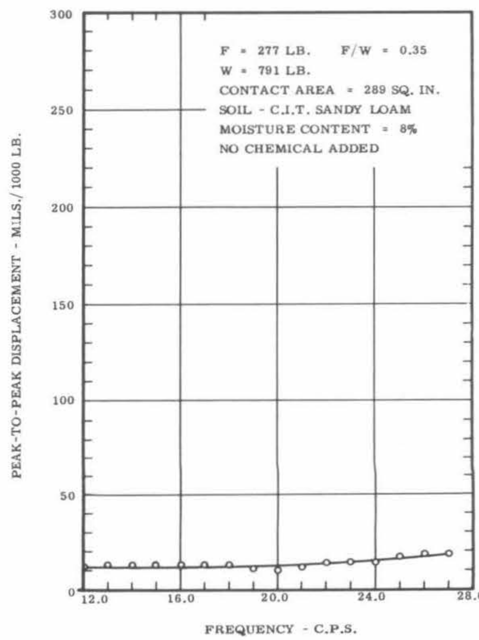


Fig. 2.80 TEST L3-22 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

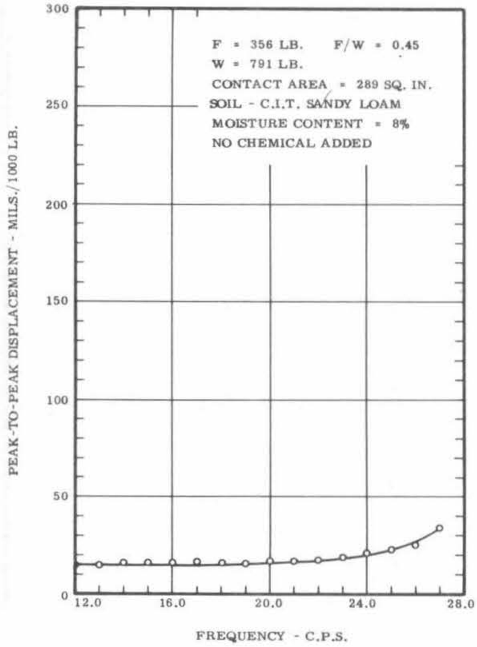


Fig. 2.81 TEST L3-23 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

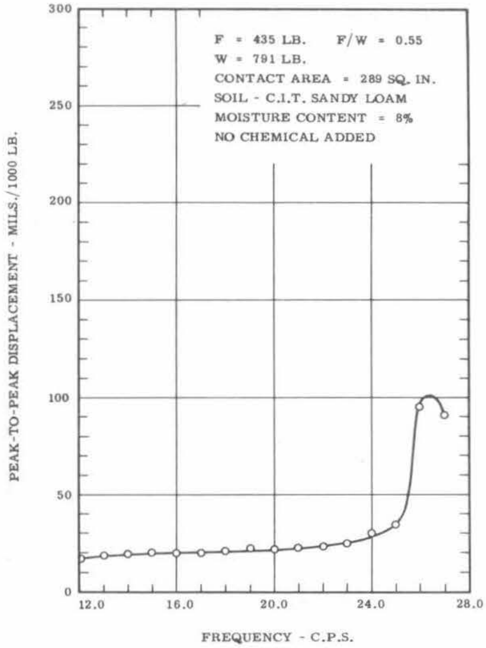


Fig. 2.82 TEST L3-24 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

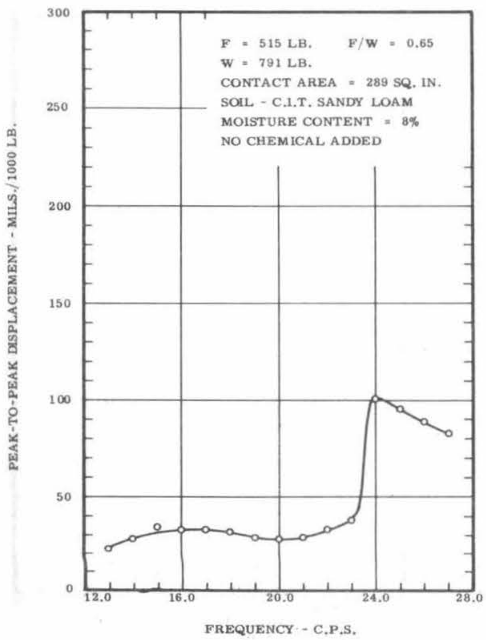


Fig. 2.83 TEST L3-25 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

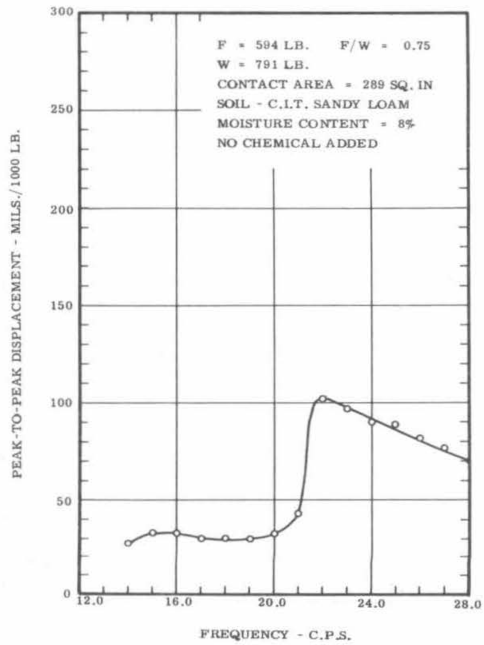


Fig. 2.84 TEST L3-26 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

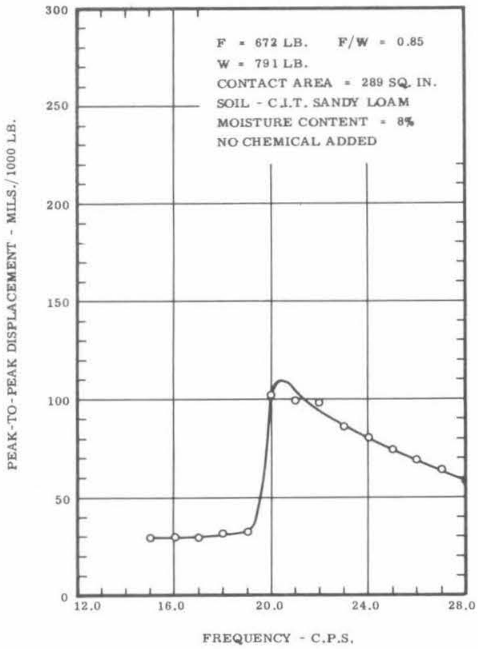


Fig. 2.85 TEST L3-27 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

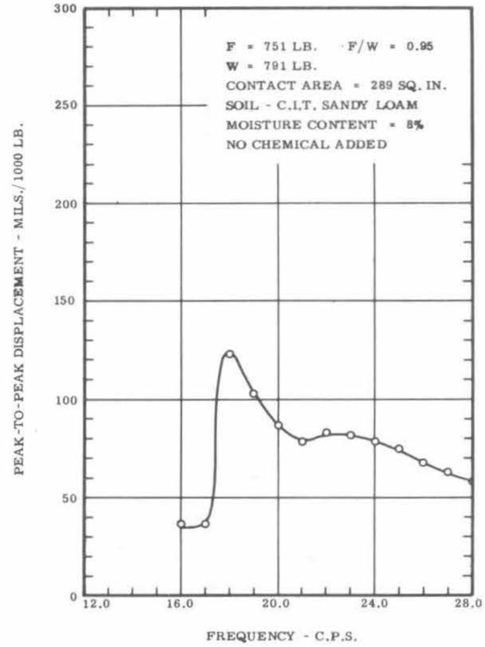


Fig. 2.86 TEST L3-28 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

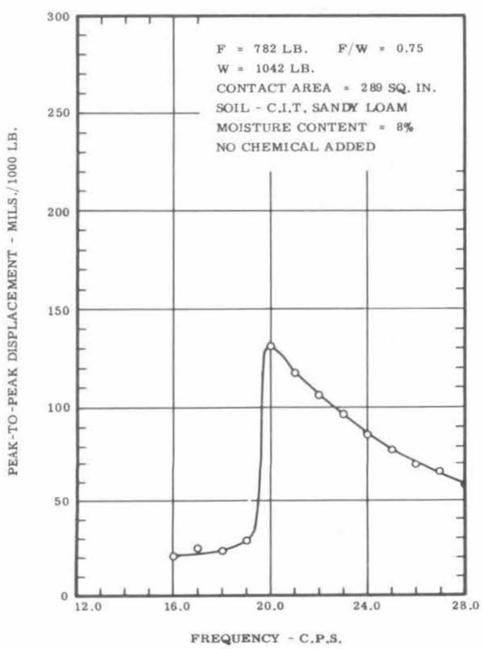


Fig. 2.87 TEST L3-29 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

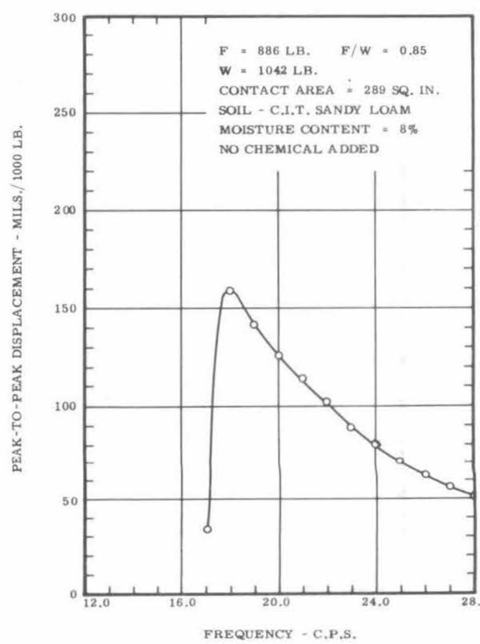


Fig. 2.88 TEST L3-30 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

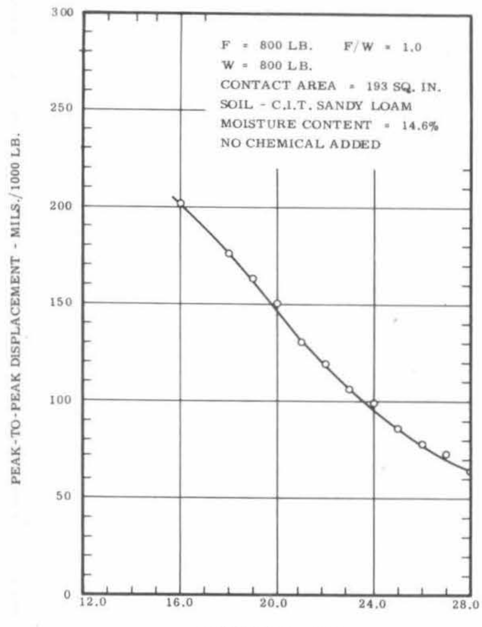


Fig. 2.89 TEST L8-1 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

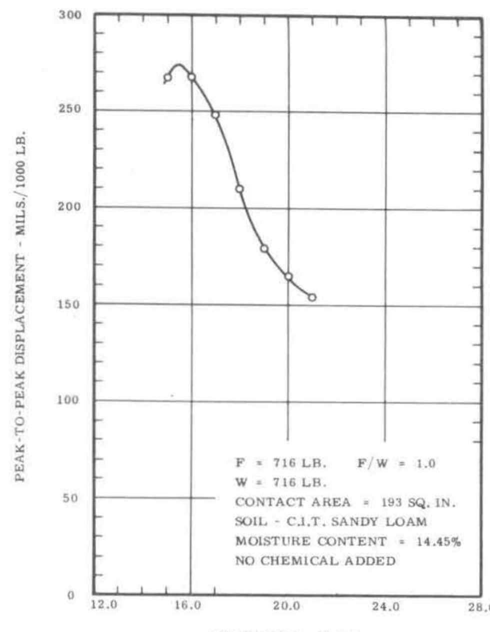


Fig. 2.90 TEST L8-2 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

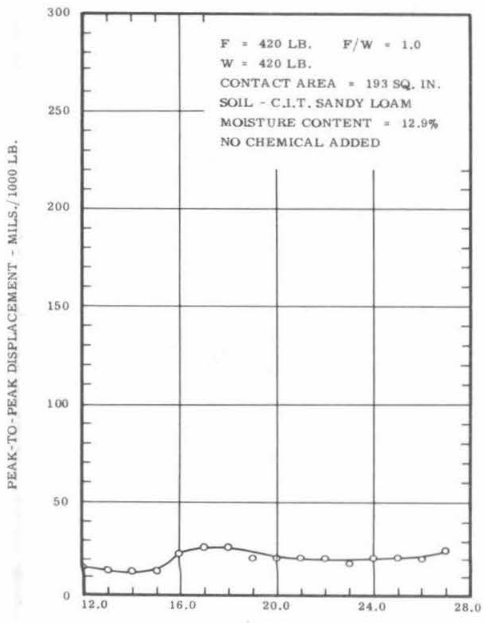


Fig. 2.91 TEST L8-3 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

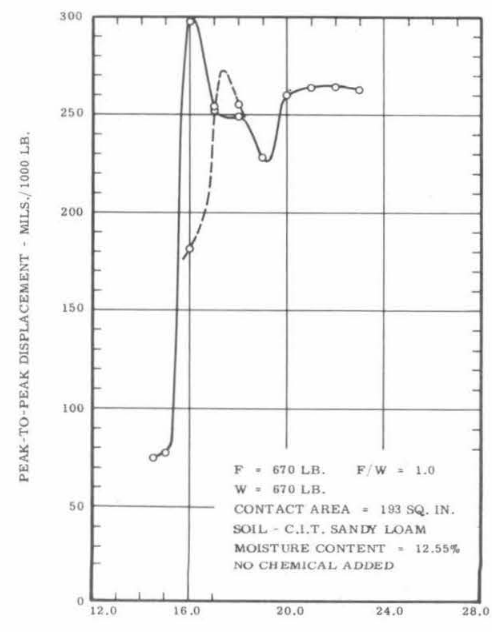


Fig. 2.92 TEST L8-4 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

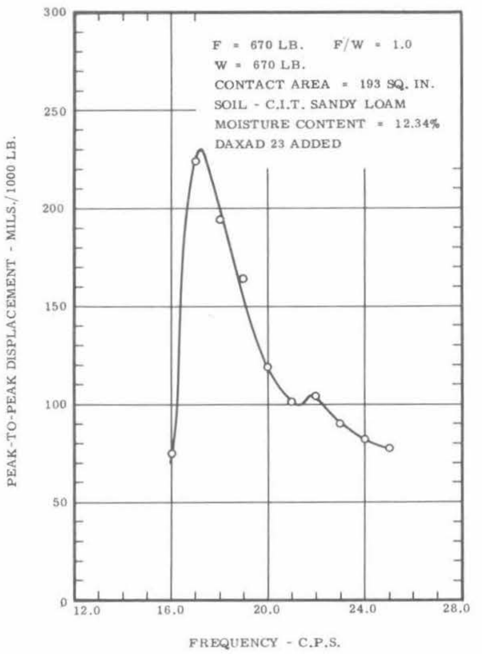


Fig. 2.93 TEST L9-1 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

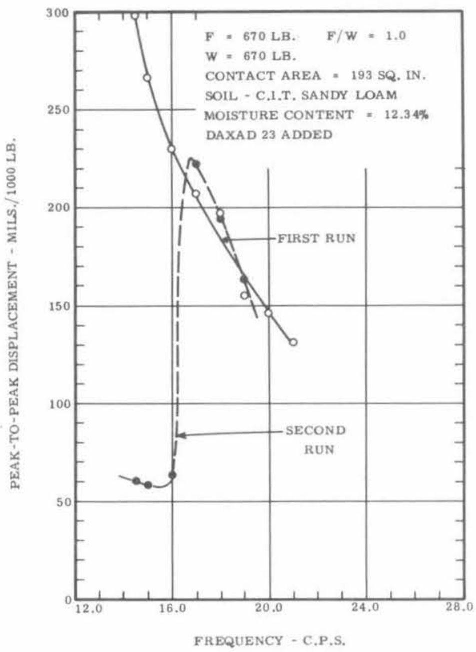


Fig. 2.94 TEST L9-2 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

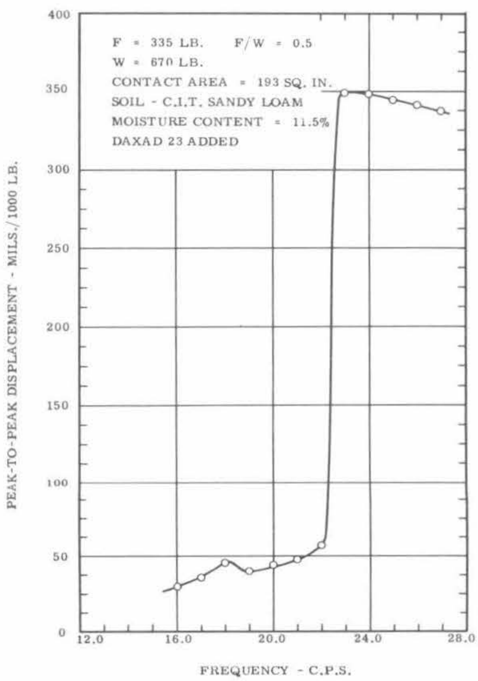


Fig. 2.95 TEST L9-4 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

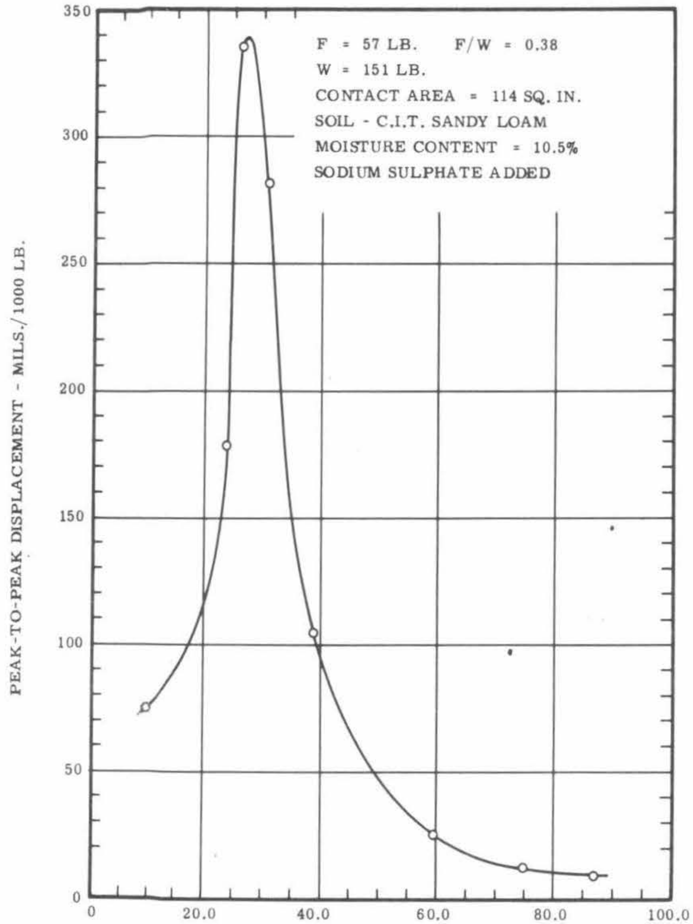


Fig. 2.96 TEST HF3-1 WITH VIBER VIBRATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

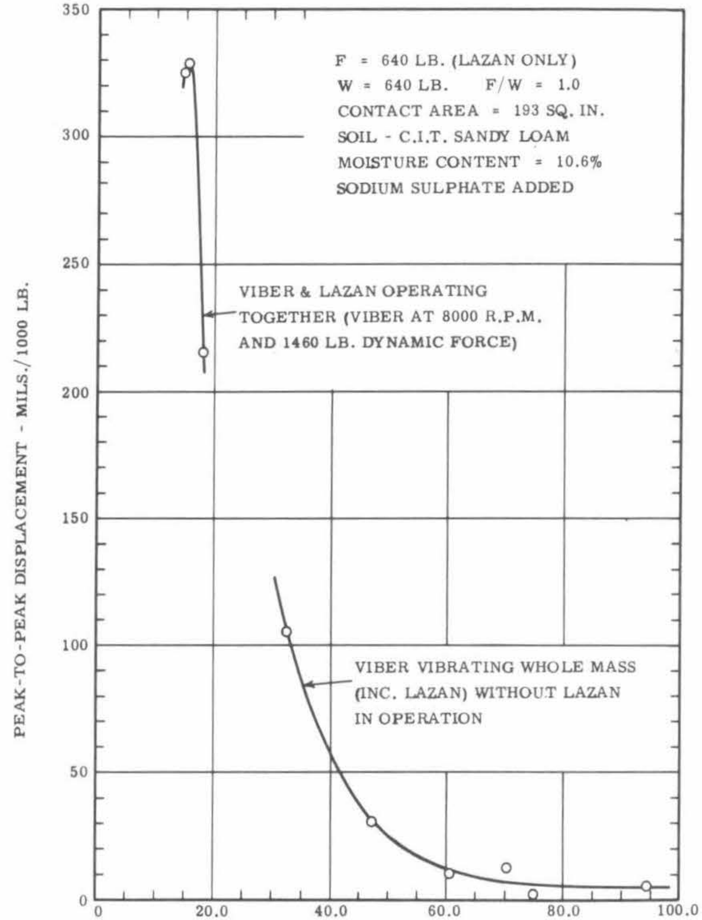


Fig. 2.97 TEST HF3-2 WITH LAZAN OSCILLATOR AND VIBER VIBRATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

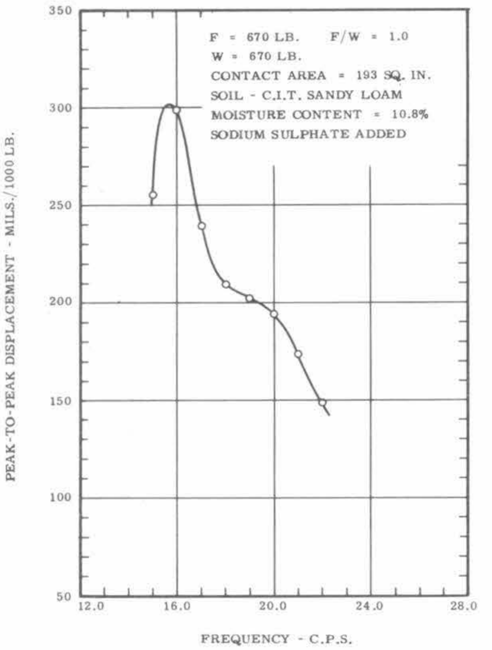


Fig. 2.98 TEST L10-1 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

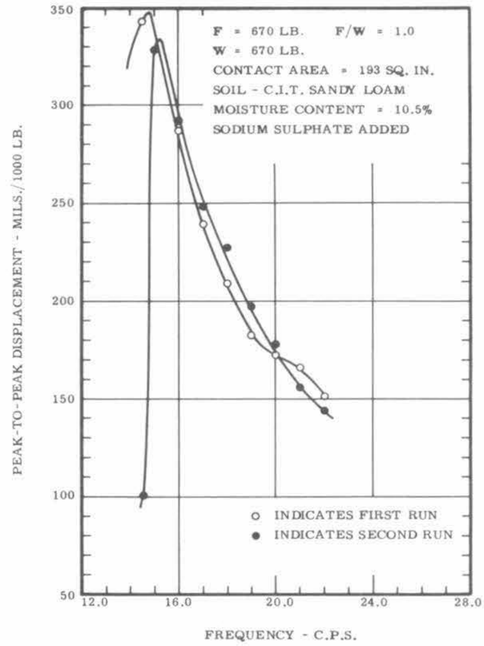


Fig. 2.99 TEST L10-2 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

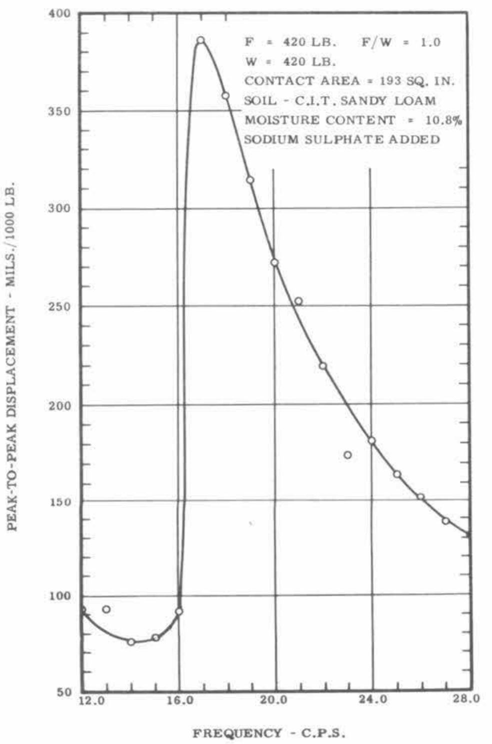


Fig. 2.100 TEST L10-3 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

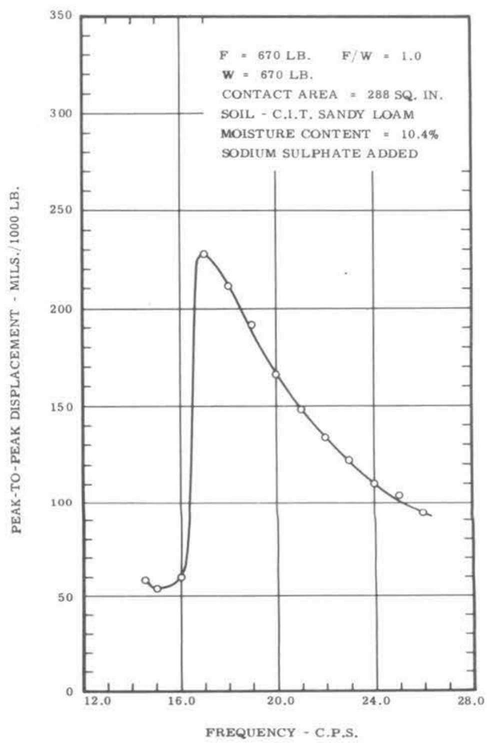


Fig. 2.101 TEST L10-4 WITH LAZAN OSCILLATOR.
DISPLACEMENT vs FREQUENCY RELATIONSHIP.

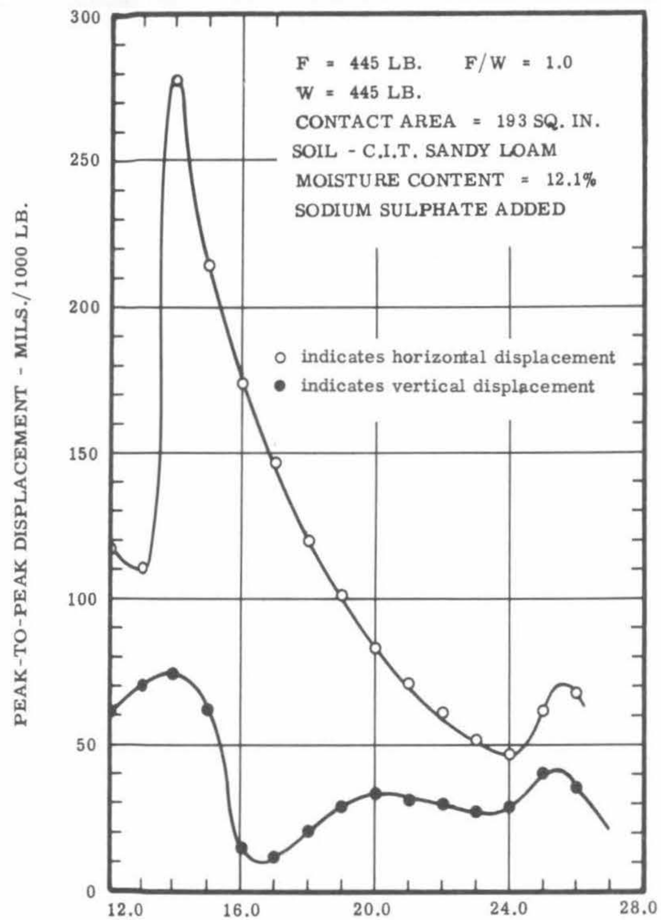


Fig. 2.102 TEST L10-5 WITH LAZAN OSCILLATOR OPERATING IN A HORIZONTAL PLANE. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

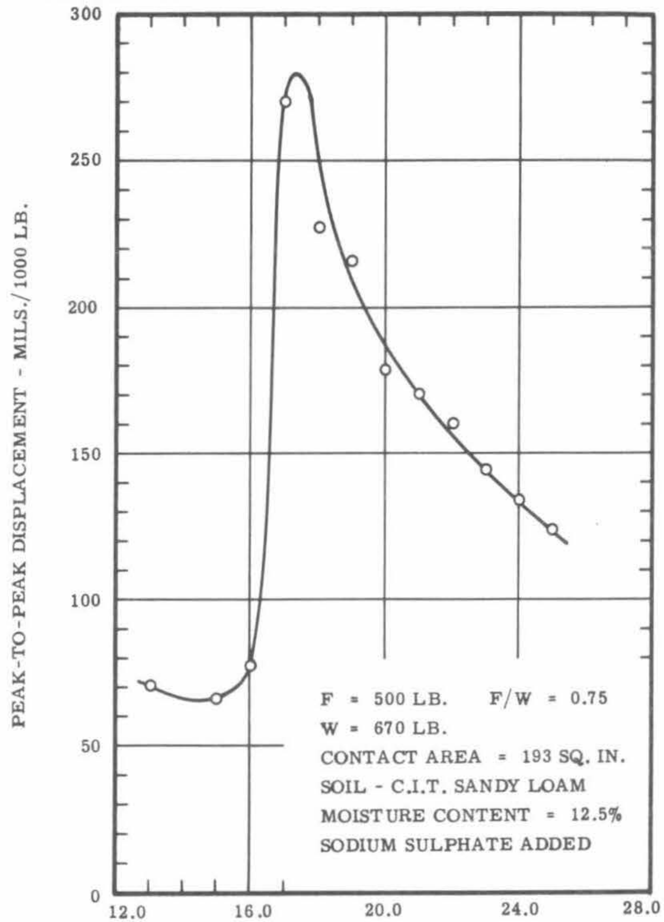


Fig. 2.103 TEST L10-6 WITH LAZAN OSCILLATOR. DISPLACEMENT vs FREQUENCY RELATIONSHIP.

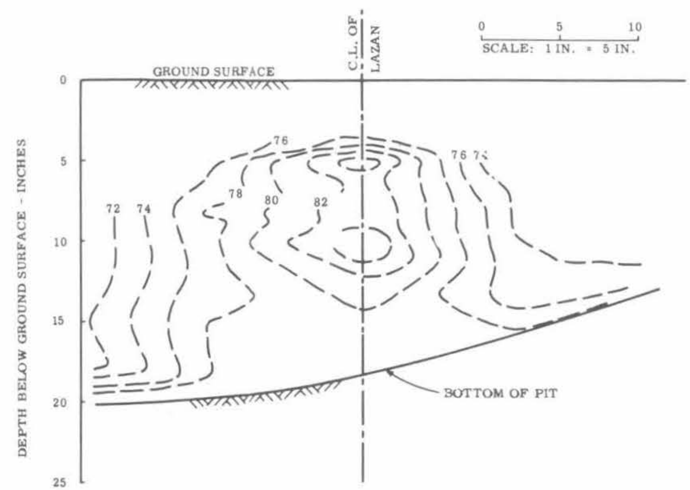


Fig. 2.104 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L8-1. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.), FOR C.I.T. SANDY LOAM, NO CHEMICAL ADDED.

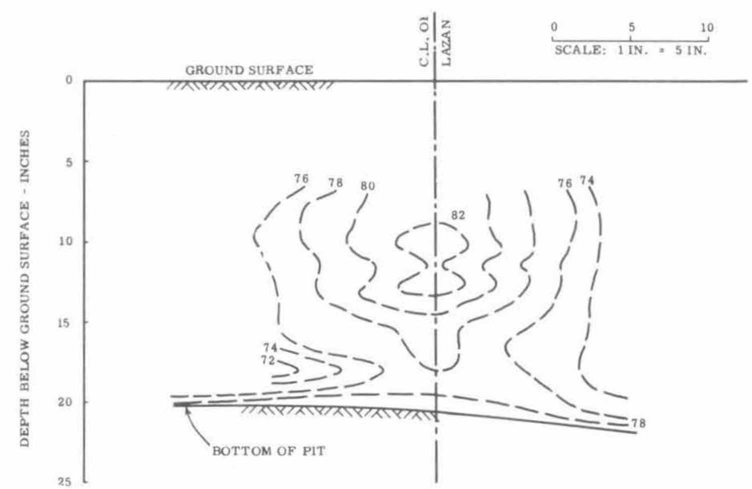


Fig. 2.105 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L8-2. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.), FOR C.I.T. SANDY LOAM, NO CHEMICAL ADDED.

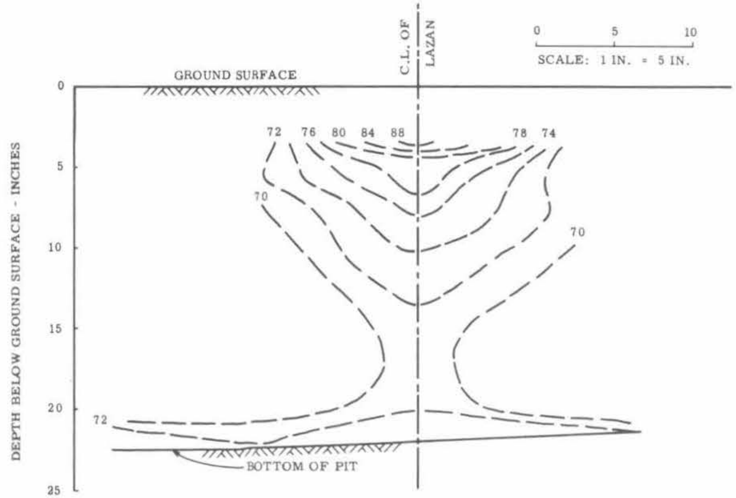


Fig. 2.106 DENSITY CONTOURS ALONG EAST-WEST CENTER LINE OF PIT AFTER TEST L8-3. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.), FOR C.I.T. SANDY LOAM, NO CHEMICAL ADDED.

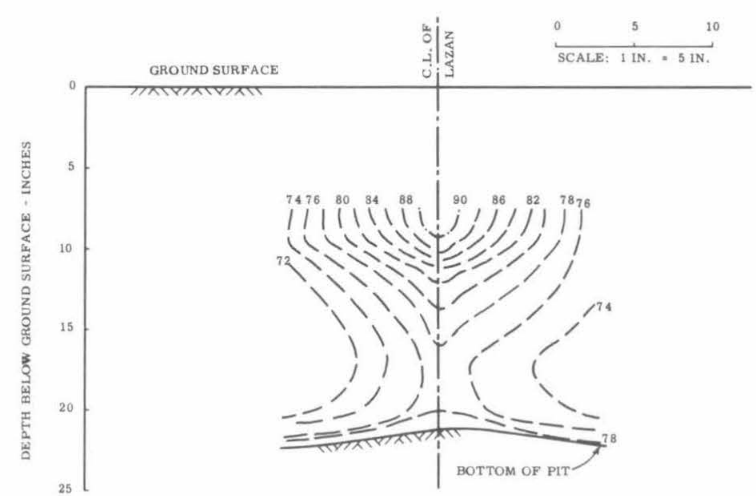


Fig. 2.107 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L8-4. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.), FOR C.I.T. SANDY LOAM, NO CHEMICAL ADDED.

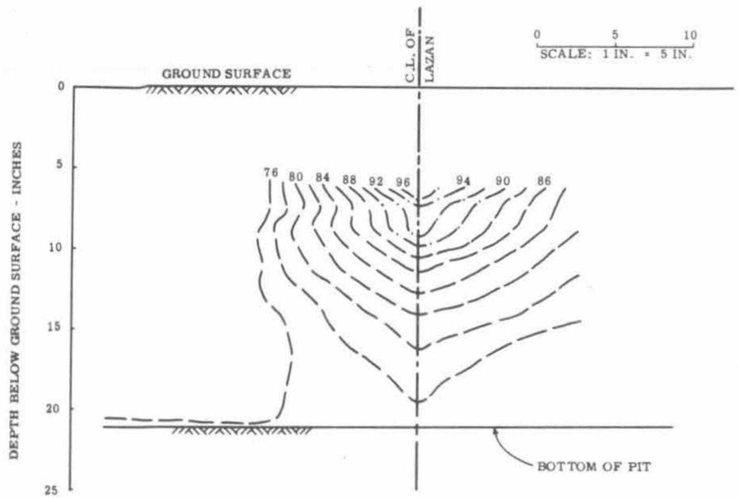


Fig. 2.108 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L9-1. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.), FOR C.I.T. SANDY LOAM. DAXAD 23 ADDED.

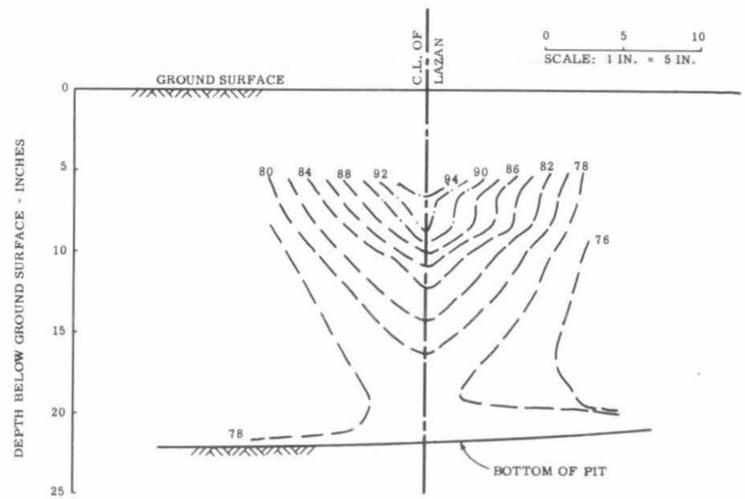


Fig. 2.109 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L9-2. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.) FOR C.I.T. SANDY LOAM. DAXAD 23 ADDED.

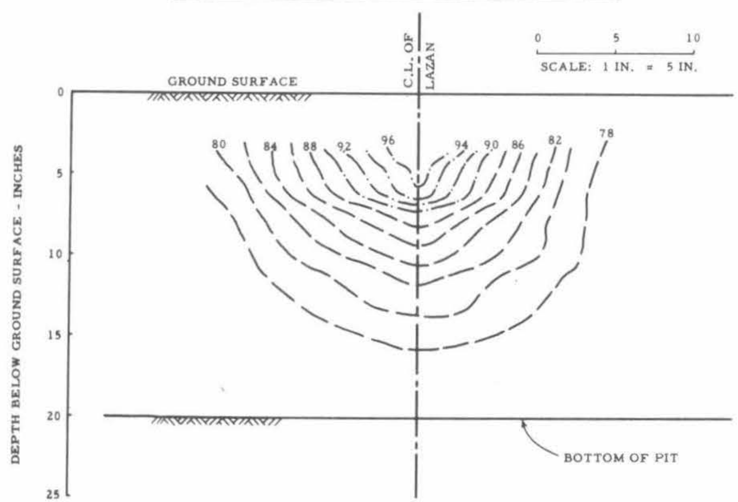


Fig. 2.110 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L9-3. CURVES OF EQUAL DRY DENSITY EXPRESSED AS A PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.) FOR C.I.T. SANDY LOAM, DAXAD 23 ADDED.

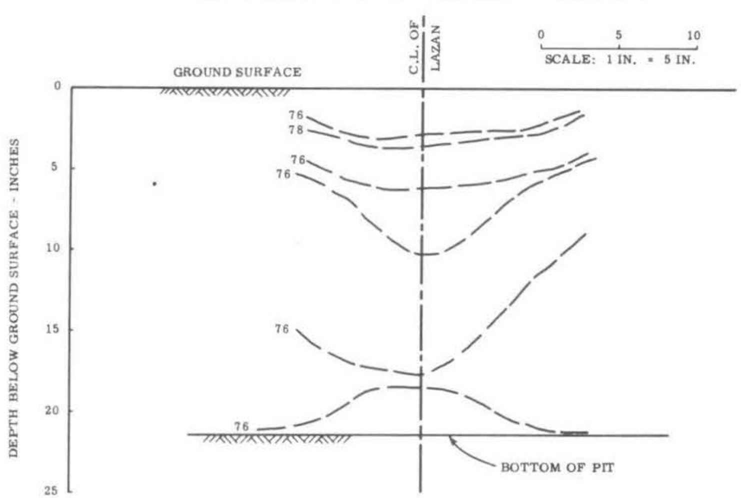


Fig. 2.111 DENSITY CONTOURS ALONG EAST-WEST CENTER-LINE OF PIT AFTER TEST L9-4. CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.) FOR C.I.T. SANDY LOAM. DAXAD 23 ADDED.

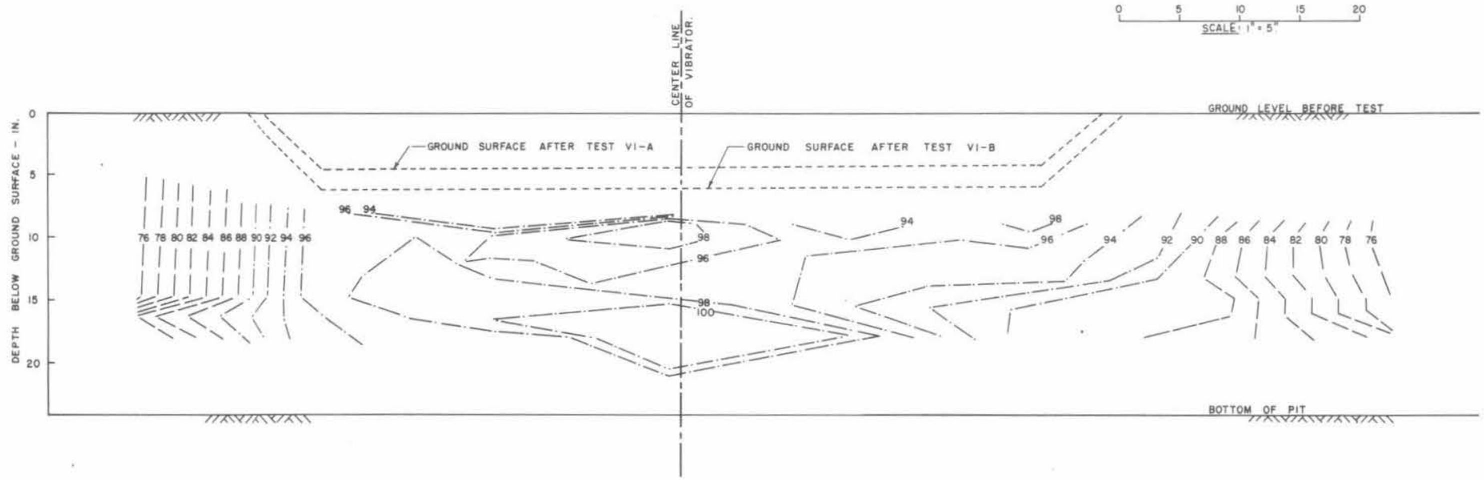


Fig. 2.112 DENSITY CONTOURS ALONG EAST–WEST CENTER LINE OF PIT AFTER TESTS VI-A AND VI-B.
CURVES OF EQUAL DRY DENSITY EXPRESSED AS PERCENTAGE OF OPTIMUM (MOD. A.A.S.H.O.) FOR C.I.T. SANDY LOAM.

APPENDIX E

LIST OF CHEMICALS

USED IN

VIBRATION-TABLE TESTS

Reference No.	Name.	Description.
1	AEROSOL AY	Di-amyl-sodium-sulphosuccinate Anionic Wetting Agent
2	AEROSOL IB	Di-iso-butyl-sodium sulphosuccinate Anionic Wetting Agent
3	AEROSOL MA	Di-hexyl sodium sulphosuccinate Anionic Wetting Agent
4	AEROSOL OS	Iso-propyl-napthalene sodium sulphosuccinate Anionic Wetting Agent
5	AEROSOL OT	Di-octyl sodium sulphosuccinate Anionic Wetting Agent
6	AEROSOL OT-B	85% Aerosol Ot and 15% Sodium Benzoate Anionic Wetting Agent
7	ALFRACAL	Mixture inc. 48% Aryl alkyl sulphonate Anionic Wetting Agent
8	ALUMINUM SULPHATE	
9	AMINE 220	$C_{22}OH_{42}N_2$, Cationic Wetting Agent
10	AMMONIUM CHLORIDE	
11	AMMONIUM NITRATE	
12	AMMONIUM SULPHATE	
13	DIBASIC AMMONIUM PHOSPHATE	
14	AMYL ALCOHOL	$C_5H_{11}OH$, Organic Wetting Agent
15	ARMOFOS	$Na_5P_3O_{10}$, Sodium Tripolyphosphate
16	ARYL ALKYL SULPHONATE	55% active, Wetting Agent
17	"BREEZE"	
18	CALCIUM CARBONATE	
19	CALCIUM CHLORIDE	
20	CALCIUM HYDROXIDE	
21	CARBON	
22	CAROXY METHYL CELLULOSE	60-65% active, organic compound
23	CATIONIC #179	Glyoxalidine salt, 25% active Cationic Wetting Agent

Reference No.	Name.	Description.
24	CUPRIC SULPHATE	
25	DARVAN #1	Polymerized sodium salt of alkyl napthalene sulphonic acid
26	DARVAN #2	Polymerized sodium salt of substituted benzoid alkyl sulphonic acids
27	DAXAD 23	
28	DETERGENT D-40-FG	Sodium aryl alkyl sulphonate base, (40.5%) Anionic Wetting Agent
29	DETERGENT D-40-FG	
30	DETERGENT D-60	Sodium aryl alkyl sulphonate base, (58.0%) Anionic Wetting Agent
31	DISPERSANT NIW	Non-Ionic Wetting Agent
32	ERTRANE C	
33	ETHYL SILICATE, CONDENSATE	
34	FERRIC CHLORIDE	
35	FERRIC SULPHATE	
36	FERROUS AMMONIUM SULPHATE	
37	FERROUS SULPHATE	
38	"KRILIUM"	
39	MAGNESIUM CHLORIDE	
40	MAGNESIUM SULPHATE	
41	METSO 55	
42	METSO 230	
43	METSO 545	
44	MONO-METHYL ACID ORTHO-PHOSPHATE B-7556	
45	NACCONOL NR	
46	NACCONOL NRSF	
47	NEUTRONYX 600	Nonionic Wetting Agent
48	NITRENE C	Coconut fatty amide condensate, 100% active, Nonionic Wetting Agent

Reference No.	Name	Description.
49	NOPCO 1067A	Anionic Wetting Agent
50	NOPCO 1525	Nonionic Wetting Agent
51	NOPCO 2173-B	Cationic Wetting Agent
52	NOPCO 2272R	Anionic Wetting Agent
53	PORTLAND CEMENT	
54	POTASSIUM CHLORATE	
55	POTASSIUM PHOSPHATE	
56	POTASSIUM SULPHATE	
57	R-88	Organic Phosphate, 88% active, Anionic Wetting Agent
58	SANTOMERSE #3	Aryl alkyl sodium sulphonate, 100% active Anionic Wetting Agent
59	SANTOMERSE #80	Aryl alkyl sulphonate and sodium sulphite 100% active, Anionic Wetting Agent
60	SC-50	Sodium methyl siliconate
61	SOAP POWDER	
62	SOAP SOLUTION	
63	SODIUM CARBONATE	
64	SODIUM CHLORIDE	
65	SODIUM FLUORIDE	
66	SODIUM NITRATE	
67	SODIUM ACID PYROPHOSPHATE	Sodium acid pyrophosphate, $Na_2H_2P_2O_7$
68	SODIUM BUTYL PHOSPHATE B-7337	
69	TETRASODIUM PYROPHOSPHATE	Tetrasodium pyrophosphate, $Na_4P_2O_7$
70	SODIUM TRIPOLPHOSPHATE	
71	SODIUM TRIPOLPHOSPHATE 8689	
72	SODIUM SULPHATE	
73	SULFRAMIN AB	Aryl alkyl sulphonate, 88% active, Anionic Wetting Agent
74	"SURF"	

Reference No.	Name	Description.
75	SURFACE ACTIVE AGENT BPE	
76	SURFACE ACTIVE AGENT TR	
77	TAMOL 731	
78	TAMOL N	
79	TEA	
80	TERGITOL DISPERSANT TMN	
81	"TIDE"	
82	TIMSEN	
83	TREND	Aryl alkyl sulphonate base, approx. 25% active, Anionic Wetting Agent
84	TREND A	Identical to TREND, but of more recent manufacture
85	TREND 40	Aryl alkyl sulphonate, 40% active Anionic Wetting Agent
86	TRIETHANOLAMINE	
87	TRITON X-100	Aryl alkyl polyether alcohol Non-ionic Wetting Agent
88	UNR 50	Alkyl dimethyl benzol ammonium chloride Cationic Wetting Agent
		OR
89	VICTAMINE D	$C_{18}H_{37}NH-P$ O $ONH_3C_{18}H_{37}$ Cationic Wetting Agent
		OR
90	VICTAMUL NO. 24C	
91	VICTAWET 14	Medium chain alkyl group-O-P = O
		OR
		(R=water-solubilizing group) Non-ionic Wetting Agent
92	VICTAWET 35B	$Na_5R_5(P_3O_{10})_2$ (R is 2-ethylhexyl) Anionic Wetting Agent

Reference No.	Name.	Description.
93	WETSIT G.S.	Mixture of alkyl aryl sulphonate and diethanol amides of cocoanut fatty amides
94	WETTING AGENT S	Aryl alkyl sulphonate base (48.0%) Anionic Wetting Agent
95	X-10	