R73-12 Soils Publication No. 317

Optimum Seismic Protection
and
Building Damage Statistics

Report No. 6

THE SHEAR WAVE VELOCITY OF BOSTON BLUE CLAY

by

Paul Joseph Trudeau

Supervised by

Robert V. Whitman

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February, 1973

Sponsored by National Science Foundation

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ABSTRACT

The purpose of this report is to provide a best estimate of the shear wave velocity of Boston Blue Clay to be used in soil amplification studies in the design of structures in the Boston area against earthquakes. The in situ shear wave velocities determined using the cross-hole method by Weston Geophysical Research, Inc. are compared with values obtained using MIT's Hardin Oscillator and also empirical correlations proposed by Hardin and Black. Modifications to the laboratory values and the empirical results indicated herein agree favorably with the in situ shear wave velocities of 850 to 900 feet per second.

PREFACE

This is the sixth report prepared under National Science Foundation grants GK-27955 and GI-29936. This report is identical with a thesis written by Paul J. Trudeau in partial fulfillment of the requirements for the degree Master of Science. The research was supervised by Robert V. Whitman and John T. Christian, professors of Civil Engineering. Acknowledgement and thanks are due to Mr. Charles Guild of the American Drilling and Boring Company who generously contributed the borings, to Mr. Vincent Murphy of Weston Geophysical Research, Inc. who generously contributed the in situ wave velocity measurements, and to Prof. Kenneth H. Stokoe of the University of Massachusetts who gave valuable advice concerning the conduct of the resonant column tests.

A list of previous reports appears on the next sheet.

LIST OF PREVIOUS REPORTS

1. Whitman, R. V., Cornell, C. A., Vanmarcke, E. H., and Reed, J. W.: "Methodology and Initial Damage Statistics," Department of Civil Engineering Research Report R72-17, M. I. T., March, 1972.

2. Leslie, S. K., and Biggs, J. M., "Earthquake Code Evolution and the Effect of Seismic Design on the Cost of Buildings," Department of Civil Engineering Research Report R72-20, M. I. T., May, 1972.

3. Anagnostopoulos, S. A., "Non-Linear Dynamic Response and Ductility Requirements of Building Structures Subjected to Earthquakes," Department of Civil Engineering Research Report R72-54, M. I. T., September, 1972.

4. Biggs, J. M., and Grace, P. H., "Seismic Response of Buildings Designed by Code for Different Earthquake Intensities," Department of Civil Engineering Research Report R73-7, January, 1973.

5. Czarnecki, R. M., "Earthquake Damage to Tall Buildings," Department of Civil Engineering Research Report R73-8, M. I. T., January, 1973.

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CHAPTER I

INTRODUCTION

The objective of this study is to provide a best estimate of the shear wave velocity of Boston Blue Clay (BBC) to be used in soil amplification studies (Seed and Idriss, 1969) in the design of structures in the Boston area against earthquakes. The significance of the shear wave velocity to the analysis of small amplitude soil vibration problems has been discussed by Hardin and Black (1968) and the application of this parameter to the design and analysis of foundation vibrations has been presented by Whitman and Richart (1967). Presented herein will be the work leading up to and including the determination of the shear wave velocity of Boston Blue Clay.

This clay was transported by preglacial streams and deposited in the quiet marine waters of the Boston Basin during the Boston substage of the Wisconsin Glacier (approximately 20,000 years ago. Chute, 1959). To indicate the extent of the clay layer an investigation was undertaken. This soil survey was initiated by collecting and analyzing the extensive data that is available for the Massachusetts Institute of Technology (MIT) campus. Another source was the numerous projects that MIT personnel have been involved with: for example, Interstate 95 in Saugus, Green Shoe Factory in the South Boston area, and the University of Massachusetts site at Columbia Point. This starting point gave a good picture of the types of profiles which are to be expected in the Boston Basin area. A later interview with Clifford Kaye of the United States Geological Survey (USGS) in Boston generally confirmed these data.

The profiles are somewhat similar and are differentiated mainly by the thickness of the clay layer. They are, in general, starting from bedrock (which is the Cambridge Argillite in the Basin area) and working up: bedrock, glacial till, outwash sands and gravels, clay (less than 60 feet to a maximum of about 180 feet), outwash sands and gravels, peat and/or organic silt, and heterogeneous man-placed fills. This general scheme is shown in Figure 1.1.

Discussion with other members of the Geotechnical Division at MIT yielded five typical profiles of which three were clay profiles of the type in Figure 1.1 with only the thickness of clay varying:

Case 3 - Up to 60 feet of clay

Case 4 - 60 to 120 feet of clay

Case 5 - 120 to 180 feet of clay.

Case 1 was to be up to 30 feet of fill or silt on firm soil (i.e. till) or rock and Case 2 was Case 1 located above 10 to 30 feet of outwash sands and gravels on rock.

These profiles were then located on a USGS Boston and Vicinity topographic map. Additional subsurface data was obtained from the 1961 Boston Society of Civil Engineers' collection of boring data in the Boston area. This map, shown in Figure 1.2, not only located these profiles but also showed that a good portion of the area could be described by these profiles indicating that they were representative of the area. In Figure 1.2 note that the clay profiles (Cases 3, 4, and 5) are located on the harbor side of the black boundary.

Having determined typical profiles for the Boston area, the next endeavor was to determine the dynamic properties to be used in the soil amplification studies. Therefore, during the month of February, 1972, American Drilling and Boring Company installed four borings in the parking lot between the Joyce Chen Restaurant on Memorial Drive and Westgate II on the MIT campus for the Boston Quake Study Project (see Figure 1.3 for location plan). There were several reasons for making these borings. One was to provide open holes in which Weston Geophysical Research, Inc. could conduct seismic tests to measure in situ the shear wave velocity of the Boston Blue Clay. Another reason was to provide the Boston Quake Study Project with high quality undisturbed samples of the Boston Blue Clay for laboratory testing to determine the shear wave velocity using MIT's Hardin Oscillator and compare the results with those obtained in the field. A further purpose of these borings was to ascertain which typical profile was located at this site. The results of the borings and a comparison with other borings in the area are indicated in Figure 1.4.

These 6 inch diameter wash borings were made using a truck-mounted rotary rig. Due to caving of the layer of sand and gravel between the depths of 15 and 45 feet, 6 inch diameter steel casing was installed for the first 50 feet of these holes. The holes were extended through the clay and clayey sand from 50 feet to 175 feet using drilling mud to keep them open. At a depth of 175 feet a very dense (120 blows/ 4 inches) fine sand layer was encountered and the holes were discontinued. At the completion of each hole, 4 inch O. D. plastic (PVC) pipe in 20 foot lengths connected with 4-3/4 inch O. D. couplings was installed in the holes. This plastic casing was lowered open-ended inside the 6 inch steel casing. At a depth of about 100 feet this casing required pushing --- first by hand, and then the last two sections with the use of the hydraulic jack on the truck-mounted rotary rig. After the plastic casing was installed, the drillers then lowered A-rods with which they washed out the material that had collected inside the plastic casing. The 6 inch steel casing was then removed using the conventional "bumping out" procedure.

In Hole B-1, undisturbed samples (3 inch Shelby tubes) were taken continuously through the clay layer (from a depth of about 50 feet to about 110 feet). These samples were taken with a fixed piston type sampler. Laboratory testing was performed on these samples to obtain values of the shear wave velocity and also to obtain the parameters necessary for use in empirical relationships.

This thesis presents the results of the Hardin Oscillator tests on these undisturbed samples. The shear wave velocities determined by Weston Geophysical Research, Inc. in situ are compared with these results and also with the results of empirical correlations using soil parameters obtained from the laboratory testing of the undisturbed samples. Finally, a conclusion regarding the best estimate of the shear wave velocity of Boston Blue Clay is drawn.

CHAPTER II

IN SITU SHEAR WAVE VELOCITIES

The in situ shear wave velocities were determined by Weston Geophysical Research, Inc. in May, 1972 at the site on the MIT campus. The testing program, utilizing the four boreholes which were described in Chapter I, consisted of the cross-hole method. For a detailed description of this method of seismic testing see Stokoe (1972). Basically, the cross-hole method measures the time it takes for a shear wave to travel a known distance. The shear waves are generated at a certain depth in one borehole while sensors at the same level in the other borehole(s) await their arrival. Knowing the time it takes for the shear waves to travel through the soil and the distance between the boreholes, one can compute the shear wave velocity of the soil.

In this testing program blasting caps (1 or sometimes 2) were detonated in one of the boreholes as the source of the shear waves. The sensors consisted of three velocity transducers, one horizontal, and the other two vertical, which were lowered to the same depth in the other three boreholes. Nothing was done to insure that the jugs containing the velocity transducers were well-coupled to the soil, for it was assumed that they would rest against the inside of the casing. The testing was begun at the bottom of the casing and then measurements were taken at ten foot intervals coming up the profile. The testing was done in this manner because the blasting caps destroyed the plastic pipe thus preventing the lowering of subsequent charges to greater depths. For this reason, Weston took many readings at the same elevation before moving up the hole, to insure acceptable results.

The location plan of these borings is shown in Figure 1.3. The results of the in situ shear wave velocity determinations are shown in Chapter IV. Figure 4.7 indicates that the value of the shear wave velocity of the Boston Blue Clay as obtained in the field is approximately 850 feet per second.

CHAPTER III

LABORATORY TEST PROGRAM

3.1 Introduction

The laboratory test program included Hardin Oscillator tests and index property tests on several samples from different depths in the clay layer. The Hardin Oscillator tests were performed on solid, cylindrical samples of Boston Blue Clay. These standard triaxial specimens, with diameters of approximately 3.5 centimeters and lengths of about 8.0 centimeters, were trimmed from the 3 inch diameter undisturbed Shelby tube samples. Index property tests were performed on the trimmings resulting from the preparation of test specimens for the Hardin test. The index property tests included the determination of Atterberg Limits, specific gravity, natural water contents, and total unit weights. Additional information concerning in situ effective stresses and maximum past pressures was obtained from Ladd and Luscher (1965) and the relationship of K_o vs. Log OCR for Boston Blue Clay from Ladd (1965). This data was used to calculate the dynamic shear modulus, G, using an empirical equation proposed by Hardin and Black (1968) for cohesive soils,

$$G = 1230 \frac{(2.973 - e)^2}{1 + e} \quad OCR^K \quad \bar{\sigma}_o^{1/2} \qquad \text{EQUATION 3.1}$$

in which G is the dynamic shear modulus in PSI, e is the void ratio, OCR is the overconsolidation ratio, and $\bar{\sigma}_o$ is the mean principal effective stress in PSI. The value of K depends on the plasticity index, PI, of the soil as shown in Figure 3.1.

3.2 Determination of Index Properties

The index property tests were performed according to the procedures in Lambe (1951), except for the total unit weights, which were determined by measuring and weighing the Hardin test specimens immediately after trimming. The results of the Atterberg Limits indicated a PI of about 30 leading to K=0.24. Calculations indicated that the in situ void ratio was approximately equal to 1.0 which is typical for Boston Blue Clay. Using the appropriate parameters in Equation 3.1, values of G were obtained for each test specimen. The shear wave velocity was then calculated using

$$C_s = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{G \times g}{\gamma_{WT}}} \qquad \text{EQUATION 3.2}$$

in which C_s is the shear wave velocity, G is the shear modulus, and ρ is the mass density which equals γ_{WT}, the total unit weight divided by g, the acceleration of gravity. These results are shown in Figure 4.7 and the results of the index property tests are shown in Figure 3.2.

3.3 Apparatus and Procedure for the Hardin Oscillator Test

The Hardin Oscillator test is used to determine the dynamic shear modulus of a sample by the resonant column method. The resonant column method is described by Richart, Hall, and Woods (1970) and Hardin and Mossbarger (1966). The Hardin Oscillator test --- apparatus, procedure, and theory --- is described by Hardin and Music (1965) and also by Hardin (1970).

The Hardin apparatus applies a torsional vibration to one end of a specimen within a triaxial cell. A load cell is included within the apparatus (see Figure 3. 3) so that anisotropic states of stress similar to estimated in situ stresses can be applied to the specimen within the triaxial cell during the dynamic test. Figure 3. 4 shows the apparatus in position for testing. Figure 3. 3 shows the oscillator portion of the Hardin device. The electromagnets in Figure 3. 3 are excited by an AC current from the audio oscillator (Hewlett Packard Model 200-AB) producing a sinusoidally varying torque at the top of the specimen. The base of the specimen rests upon a rigid pedestal which has sufficient inertia to make the motion of the attached end of the specimen essentially zero during vibration of the specimen (Hardin, 1970). An accelerometer (see Figure 3. 3) is attached to the oscillator to monitor the movement of the top of the specimen. The frequency of oscillation is varied until the maximum output of the accelerometer is obtained. This output is monitored with an oscilloscope or can be measured with an AC voltmeter. The resonant frequency of the system and specimen occurs when the maximum output of the accelerometer is achieved. Knowing the calibration of the accelerometer, the test can be run at a certain level of shear strain by varying the input voltage and the frequency such that the desired output of the accelerometer is obtained. The theory presented by Hardin and Music (1965) or Hardin (1970) uses this resonant frequency to determine the dynamic shear modulus. Equation 3.2 is then used to obtain the shear wave velocity.

Calibration Factors: During the course of the testing program there was some question as to what were the appropriate calibration factors. It was found that these factors, as discussed in Hardin (1970) or Hardin and Music (1965), changed with different strain levels for the MIT Hardin Oscillator. These changes led to improper trends in the results, i.e. C_s was greater for higher shear strains, which is not true (Hardin and Black, 1968). Telephone conversations with Dr. Hardin at the University of Kentucky and with Dr. Stokoe at the University of Massachusetts at Amherst both proved fruitless, for this phenomenon did not exist with the equipment they had used. It was concluded that the calibration factors corresponding to low strain amplitudes be used and thus, only the low strain amplitude data from these tests is included in this report. Note that Hardin (1970) recommends using an average shear strain of about 2.5×10^{-5} in/in and the shear strains in these tests are close to this value (see Table 4.1).

3.4 Tests on Boston Blue Clay

Procedure for Controlling Strain: There were two different procedures used for these tests. The earlier tests were run at three different levels of input voltage of the Hewlett Packard audio oscillator corresponding to a maximum shear strain, at any point of the specimen, of about 2.5, 5, and 10×10^{-5} in/in. This strain refers to the maximum movement at the circumference of the solid sample. For solid samples Hardin and Drnevich (1972) define average shear strain as equal to $\int_{area}$ (strain) dA/AREA which leads to an average shear strain equal to 2/3 of the maximum shear strain. The later tests were run at a maximum shear strain of approximately 1×10^{-5} in/in (An illustration of how the shear strain is controlled during the test is shown on page of Appendix A.) in an attempt to obtain the maximum value of the dynamic shear modulus. Note that the dynamic shear modulus decreases with increasing strain and 1×10^{-5} in/in is the lowest strain at which satisfactory measurements can be made due to random AC noise in the cathode follower used to couple the output of the accelerometer to the measuring devices.

The initial tests included T-3, T-4, S-1, and S-2. (See Appendix B for a general description of each test.) These tests were isotropically consolidated to the estimated in situ horizontal effective stress, at which point the resonant frequency was determined. Subsequently, the cell pressure was increased to the estimated in situ vertical effective stress, again the specimen was consolidated, and the dynamic test run. The specimens were then consolidated to higher cell pressures in approximately 20 PSI increments, up to a maximum cell pressure of 100 PSI. After running the dynamic test at the maximum confining pressure, the samples were unloaded in 20 - 40 PSI steps, allowed time to rebound, and then the dynamic tests were run again.

Chamber Fluid: A major problem results due to the electrical connections within the triaxial cell. The connections must not be submerged in a fluid that conducts electricity; therefore, the cell can only be filled with a cell fluid up to the base of the oscillator. The cell pressure is then applied by air pressure acting on the fluid within the cell. The use of water or silicone oil as a cell fluid does not provide adequate protection from the diffusion of air, especially under pressures greater than 30 PSI, through the cell fluid and the membrane where, at atmospheric pressure, it comes out of solution thus interfering with volume change readings. An attempt was made to use silicone oil as a cell fluid completely filling the cell and using mercury pots to apply the cell pressure. However, the 5 centistoke silicone oil was too viscous and it was thought that the movement of the magnets induced motion in a certain mass of the oil thus interfering with the resonant frequency of the sample. Therefore, the tests were run with silicone oil only covering the sample and air pressure was applied to the top of the cell. (Silicone oil was used instead of water because water attacks the air pistons and aluminum of the support device for the Hardin apparatus during the set-up of the test.) Volume change readings were not made after the results of the first few tests indicated that they were no good. During subsequent tests an attempt was made to keep the sample wet by flushing water through the porous stone at the base of the specimen. It was found that consistent results of the resonant frequency could be obtained by this procedure as long as the filter strips surrounding the specimen were kept wet. It was found that flushing water through the porous stone once a day was sufficient to remove the air bubbles and to keep the filter strips wet. Consolidation was obtained by allowing approximately 24 hours to pass before running the dynamic test. From past testing on Boston Blue Clay (Edgers, 1967) this was considered more than adequate time for primary consolidation.

Mercury as Chamber Fluid: At the time of this writing, Marcuson and Wahls (1972) have published results of a series of tests on two different clays. They used a Hardin Oscillator, performing tests in essentially the same manner as the earlier tests described above, i. e. they ran the dynamic tests at three different strain levels and different cell pressures up to 100 PSI. However, they used mercury as the cell fluid surrounding the sample because air does not diffuse readily into mercury at the cell pressures involved. They ran tests on the same sample surrounded by mercury and then with the mercury drained out. They found little variation in G (about 7 % at 10 PSI with $\theta = 0.0006$ radians) and no definite trend as the cell pressure was increased. Thus, they conclude that the use of mercury is an effective means of eliminating the problem of air diffusion during long term tests. However, they caution that the mercury results in a pressure differential of about 1.5 PSI from the top to the bottom of the specimen and therefore, may have a significant effect at low confining pressures. Surrounding the sample with mercury allows volume changes to be measured with a burette. Furthermore, a backpressure can be used to maintain a completely saturated specimen.

The later tests, C-1, C-2, G-1, L-1, and AA-1, were run at a maximum shear strain of about 1×10^{-5} in/in. These tests were loaded isotropically to the in situ effective octahedral stress as computed using the values of vertical effective stress and maximum past pressure as shown in Figure 3.2, and the values of K_o as determined by Ladd (1965) for Boston Blue Clay. The dynamic test was run at a maximum shear strain of 1×10^{-5} in/in obtaining values of the resonant frequency with time in a manner similar to a consolidation test. The value of the shear wave velocity, C_s, was then plotted vs. the logarithm of time as shown in Figures 4.5 and 4.6. To compare these results with the initial tests, Tests C-1 and L-1 were then loaded isotropically to higher cell pressures, and again C_s was determined vs. the logarithm of time. Running the test in this manner allowed observation of the results while the specimen was consolidating under each increment of load. Note in Figures 4.5 and 4.6 that after primary consolidation had been completed there was an increase in C_s vs. time. This increase is linear on the C_s vs. Log time plot and has been noted in tests on cohesive soils by Stokoe (1972) and Hardin and Black (1968). This increase equalled approximately 40 feet per second per log cycle of time and was used to extrapolate the laboratory values of C_s to those expected in the field as will be shown in Chapter IV.

3.5 Conclusion

Reported herein are the results of the index property tests. Also included is the description of the Hardin Oscillator test --- apparatus, procedure, and the problems encountered during this study. Important variables during the performance of the Hardin test include the level of the shear strain, consolidation and drainage of the specimen during long term tests, effects of chamber fluid, and the effects of time on the results. Using water (or silicone oil) as the cell fluid upon which air pressure was applied resulted in problems due to air diffusion into the specimen. If this air was removed by flushing water through the porous stone daily, no change in the measured shear wave velocity occurred. Thus it is concluded that if water is to be used as the cell fluid, adequate provisions must be made to insure that the specimen is kept wet.

CHAPTER IV

ESTIMATES OF SHEAR WAVE VELOCITY

4.1 Introduction

The results of this investigation are presented in this chapter. These results include the in situ shear wave velocities measured by Weston Geophysical Research, Inc., the Hardin Oscillator test results, and the results from Hardin and Black's empirical correlation (Eq. 3.1) for cohesive soils. Also included are the corrections to the laboratory results based on strain levels and secondary time effects.

4.2 In Situ Results

The in situ results by Weston Geophysical Research, Inc. are presented in Table 4.1 and are also plotted in Figure 4.7. Weston reported that some of their records were good and some were poor. The majority of the records were average, or at least acceptable. There was some problem due to coupling of the output velocity transducers to the ground due to a possible gap between the plastic casing and the soil. However, as will be shown later, these results agree well with the laboratory results. Weston reports that these values of C_s are good to within 10%.

4.3 Laboratory Results

The results of the laboratory tests that were run at different confining pressures are presented as plots of the shear wave velocity vs. the square root of the effective isotropic confining pressure in Figure 4.1 to 4.4. The shear wave velocities that are plotted are the values of C_s that were measured and corrected for strain amplitude. Also shown on these plots are the values of C_s as predicted by Hardin-Black (Equation 3.1). The values of C_s at the effective in situ octahedral stresses were then interpolated from these results to obtain the values of C_s in column 4 on Table 4.1. Tests G-1 and AA-1 were only run at the in situ effective octahedral stress and, therefore, the results are presented as plots of C_s vs. Log time in Figures 4.5 and 4.6. The reader is cautioned that due to problems during the running of these tests some of the results are suspect. These problems, mainly due to air diffusion into the specimen, are inumerated in Appendix B.

Table 4.1 is a summary of the results of this study for the specimens at the in situ effective octahedral stresses. The values of $C_{s_{meas}}$ (column 1) is the value of the shear wave velocity measured using the Hardin Oscillator at an elapsed time of approximately 1000 minutes and a cell pressure equal to the estimated in situ effective octahedral stresses. These values were obtained at the maximum shear strains indicated in column 2. In order to compare results for the same strains, the $C_{s_{meas}}$ are divided by column 3, the equivalent $C_s/C_{s_{max}}$. The values of equivalent $C_s/C_{s_{max}}$ equal the square root of the values given by Hardin and Drnevich (1972) for G/G_{max}.

Note that there is little difference in $C_{s_{max}}$ (column 4) and $C_{s_{meas}}$ (column 1) using the Hardin apparatus. The computed values of C_s using the Hardin-Black equation (Eq. 3.1) are indicated in column 5. These results are greater than the measured values by more than 200 feet per second, but recall that the measured values of C_s were at an elapsed time of 1000 minutes. Plots of C_s vs. Log time, as in Figures 4.5 and 4.6, show that there exists a linear increase in C_s vs. Log time during secondary consolidation. This increase has been noted by others for cohesive soils (Hardin and Black, 1968; Stokoe, 1972; and Marcuson and Wahls, 1972), and in these tests was approximately 40 feet per second per log cycle. Considering this increase to continue for a length of time equal to the age of the clay (about 20,000 years) yields the values of $C_{s_{20,000}}$ in column 7. These values are surprisingly close to the results obtained in situ by Weston Geophysical Research, Inc.

Hardin and Black (1968) analysed this effect and also the effect of different load increments. They conclude that there is a secondary increase of the vibration shear modulus with time at a constant effective stress that is not accounted for by changes in void ratio. Furthermore, this increase can be destroyed by changes in effective stress and consequently, this effect may be quite important with soils in situ, for the laboratory values of shear modulus (shear wave velocity) depend on the loading scheme used. Tests which Hardin and Black ran on a

Kaolin clay using small load increments (about 1 PSI) yielded values of shear modulus which were about 20% greater than those predicted by their equation. These data fit the equation

$$G = 1630 \frac{(2.973 - e)^2}{1 + e} \quad OCR^K \quad \bar{\sigma}_o^{1/2} \qquad \text{EQUATION 4.1}$$

more closely. Using Equation 4.1 together with the parameters for the specimens of Boston Blue Clay tested yields the values in column 8 of Table 4.1. Note that these values are in better agreement with the laboratory values which were extrapolated to 20,000 years and also the in situ values by Weston Geophysical Research, Inc. than those calculated using Equation 3.1. This agreement is more easily seen in Figure 4.7 which is a plot of the data in Table 4.1.

Insofar as the laboratory test results and the calculated results using Equation 4.1 agree reasonably with the in situ results, the author concludes that the in situ results are probably the best estimate of the shear wave velocity of Boston Blue Clay. Figure 4.7 indicates that C_s varies between 850 and 900 feet per second depending on the depth. This value corresponds to a shear modulus of approximately 19,000 PSI for the Boston Blue Clay.

CHAPTER V

SUMMARY AND CONCLUSIONS

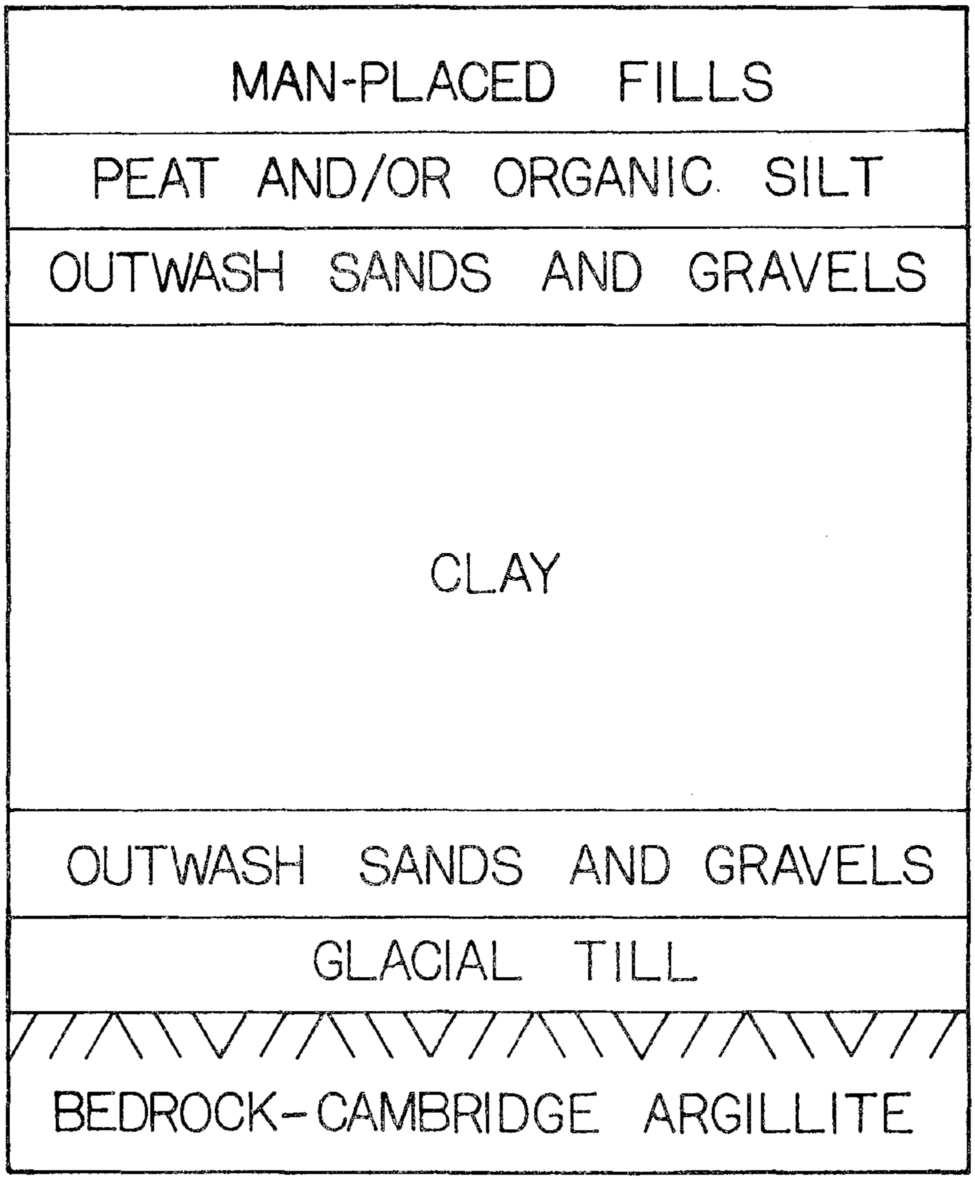
Based on the results of this study, it is the conclusion of the author that the best estimate of the dynamic shear wave velocity of Boston Blue Clay is between 850 and 900 feet per second. This range is substantiated by the in situ measurements made by Weston Geophysical Research, Inc. utilizing a seismic technique known as the cross-hole method. Close agreement with these values is found by extrapolating the laboratory values of the shear wave velocity measured using MIT's Hardin Oscillator to a time equivalent to the age of the clay. The laboratory values were found to increase linearly with the logarithm of time during secondary consolidation as was the case with numerous researchers (Hardin and Black, 1968; Stokoe, 1972; Marcuson and Wahls, 1972). This linear trend in the shear wave velocity vs. the logarithm of time, coupled with the close agreement of the extrapolated results to the in situ results, was considered justification for the extrapolation of the laboratory data. Utilizing parameters of the samples of Boston Blue Clay tested, an empirical correlation, when modified according to results presented by Hardin and Black (1968) to account for the increase in dynamic modulus with time during secondary consolidation and the effects of small load increments, also yielded values of shear wave velocity close to this range. Therefore, based on in situ results and modified laboratory and empirical results, it is concluded that the best estimate of the shear wave velocity of Boston Blue Clay is 850 to 900 feet per second.

TABLE 4.1

SHEAR WAVE VELOCITY DATA FOR BBC

	1	2	3	4	5	6	7	8	9
TEST	$C_{S_{MEAS.}}$	$\gamma_{\theta z\ MAX.}$	EQUIV. C_S / C_S AT 10^{-6} IN/IN	C_S AT 10^{-6} IN/IN	$C_{S_{HAR-BLK}}$	SECONDARY TIME EFFECTS	C_S 20,000 YRS.	C_S MODIFIED HA-BLK	C_S WESTON
	FPS	$\times 10^{-5}$ IN/IN		FPS	FPS	FPS / LOG CYCLE t	FPS	FPS	FPS
T-3&4	535	2.5—5	0.99	540	772	40*	945	1022	840
C-1&2	564	1	0.99	570	747	41	984	990	840
G-1	490	1	0.99	495	701	44	940	930	850
L-1	510	1	0.99	515	675	39	910	895	875
S-2	480	7—10	0.97	495	732	40*	908	970	900
S-1	436	5—7	0.98	445	732	40*	855	970	900
AA-1	508	1	0.99	513	717	44	958	951	850

* ASSUMED



A TYPICAL PROFILE FOR BOSTON BASIN AREA

FIGURE 1.1

GENERAL LOCATION OF PROFILES IN THE BOSTON BASIN AREA

FIGURE 1.2

BOSTON QUAKE
BORING LOCATIONS

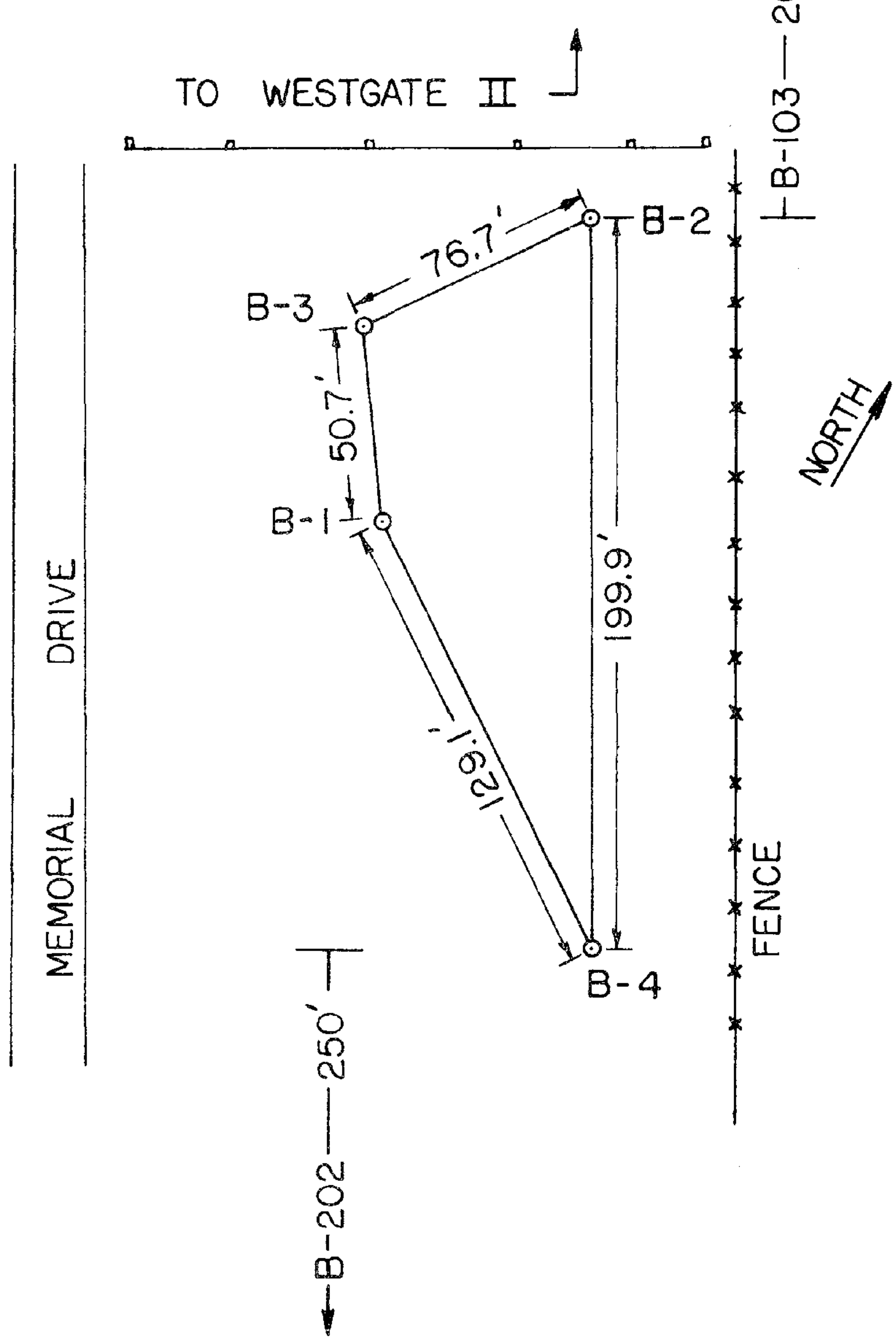


FIGURE 1.3

BOSTON QUAKE PROFILE

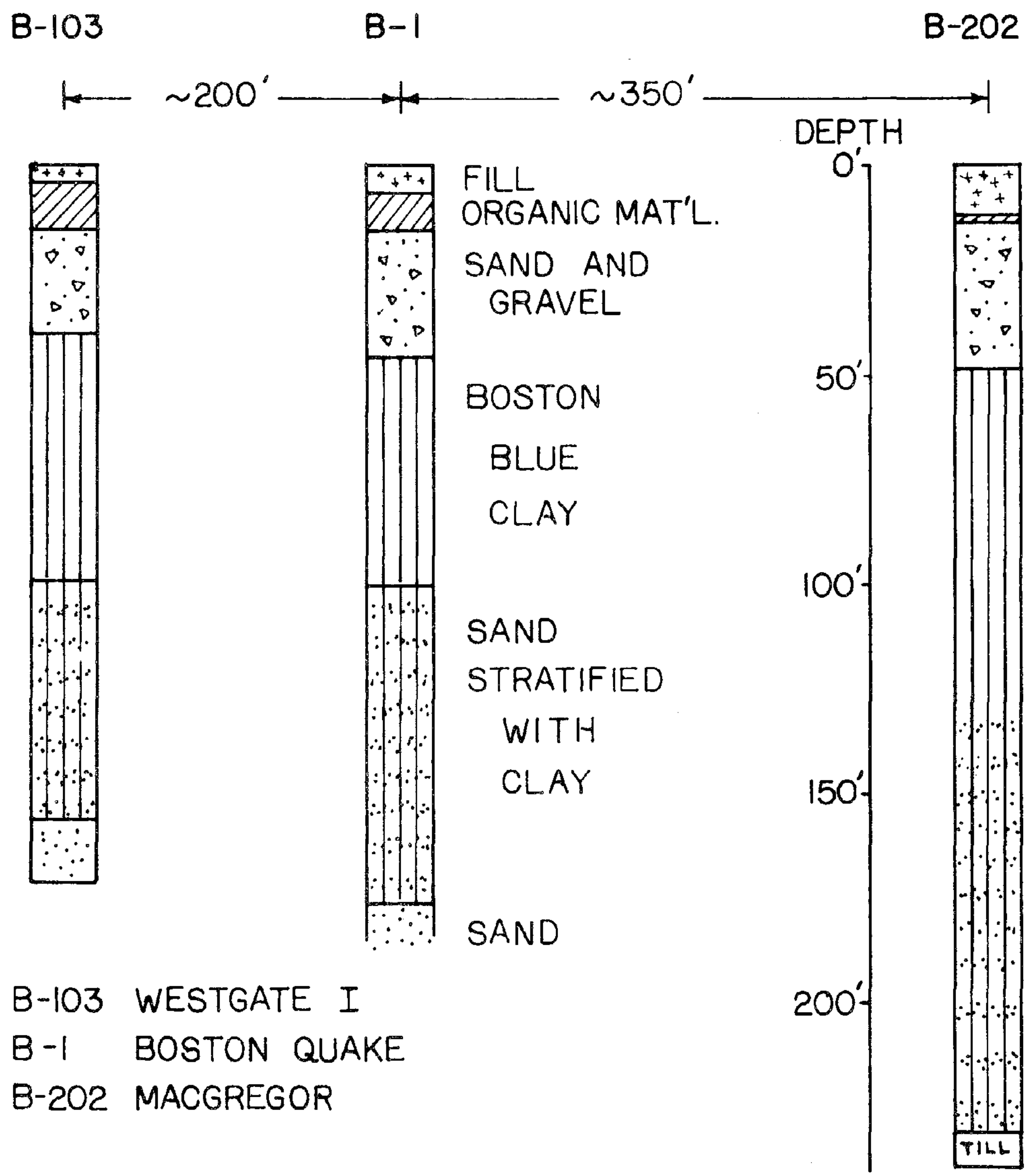


FIGURE 1.4

K VS. P I

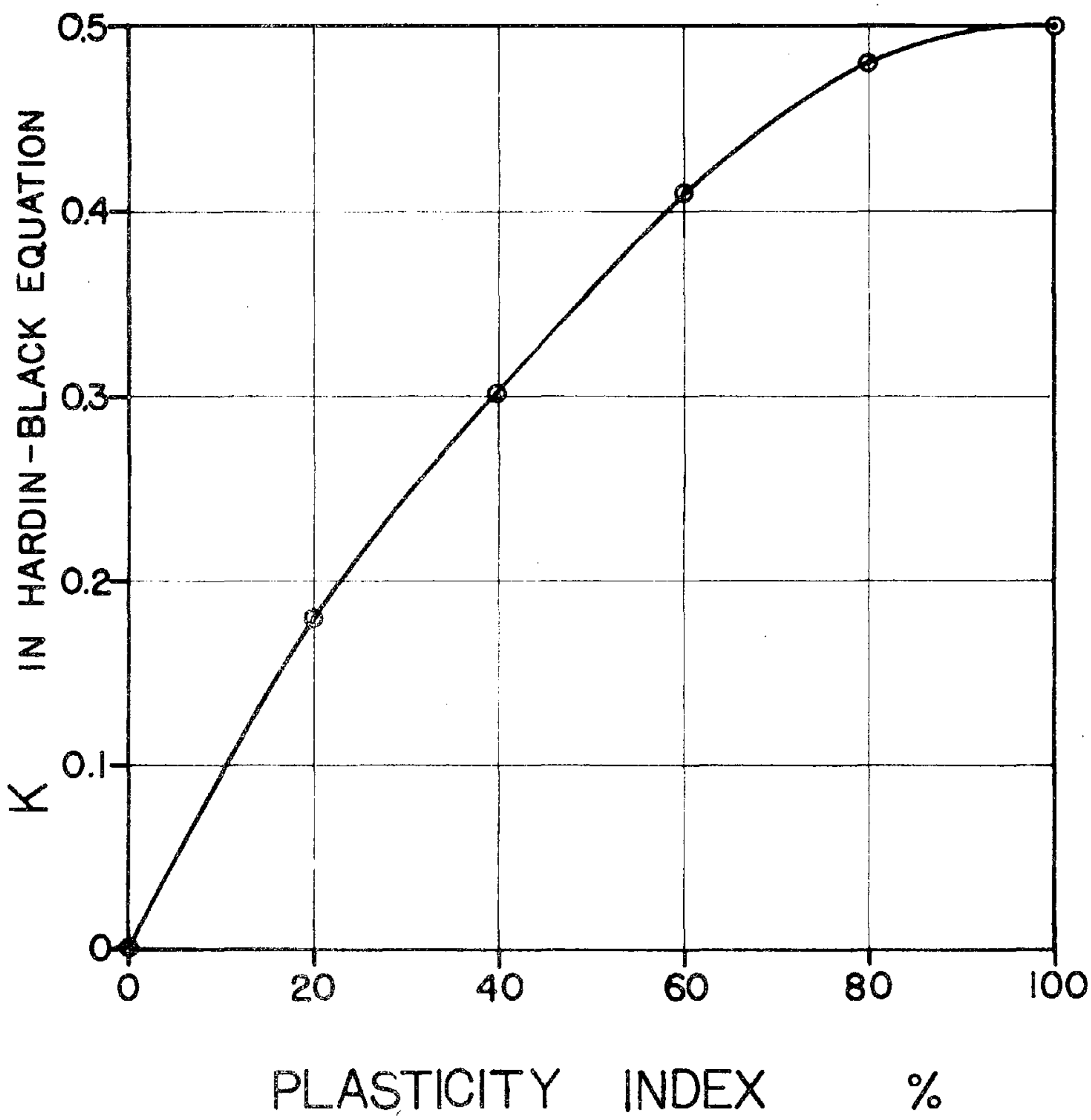


FIGURE 3.1

INDEX PROPERTIES vs. DEPTH

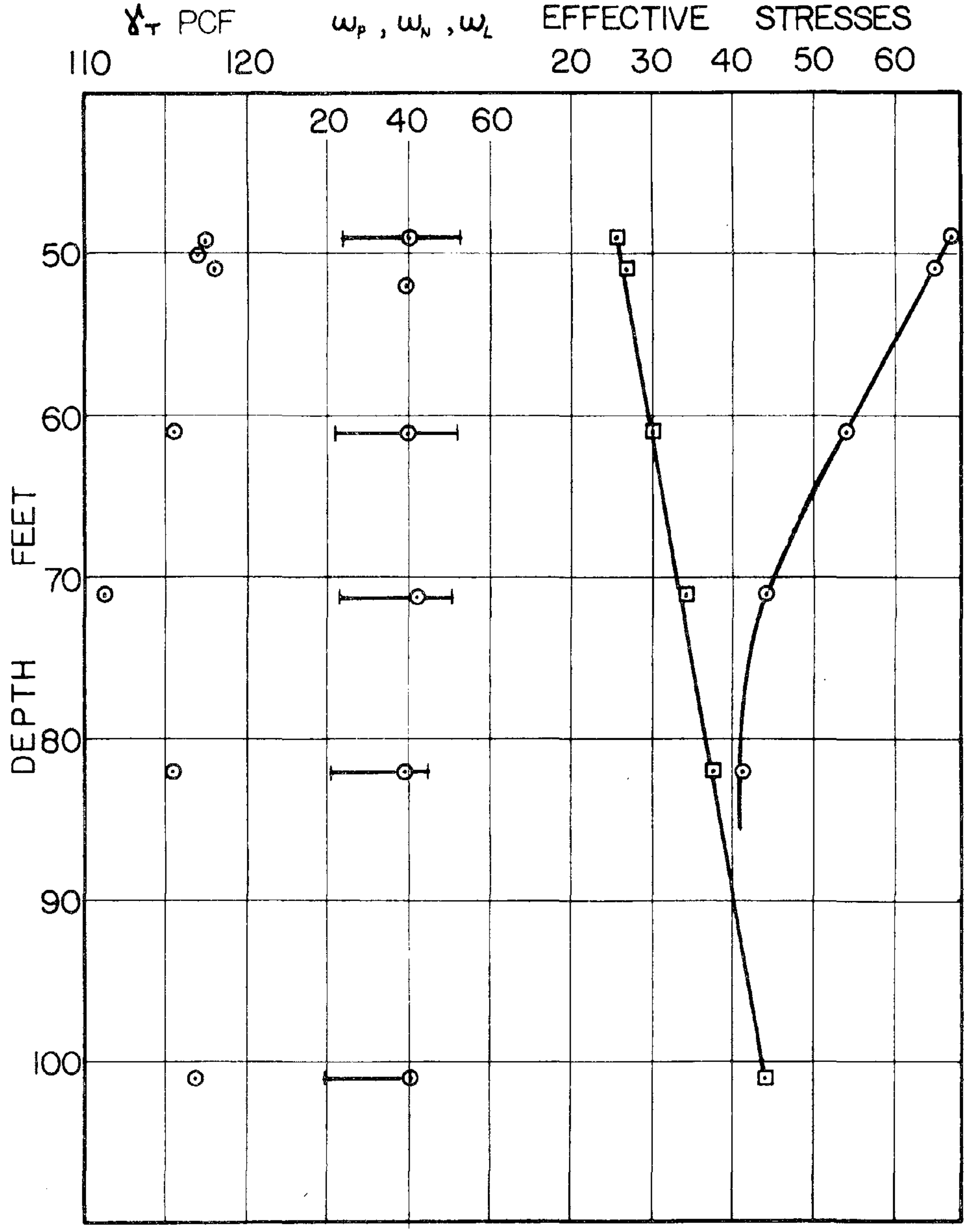
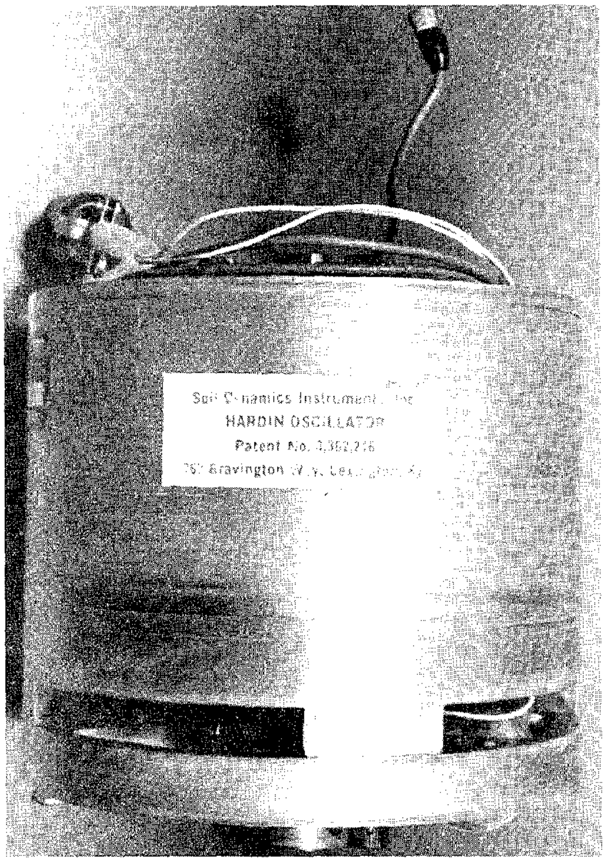
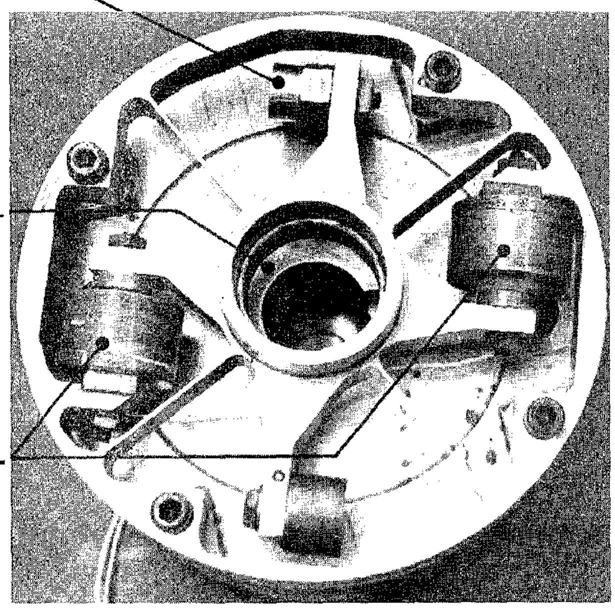


FIGURE 3.2

DRIVING UNIT OF HARDIN OSCILLATOR



SIDE VIEW



BOTTOM VIEW

FIGURE 3.3

HARDIN OSCILLATOR SET-UP IN TRIAXIAL CELL

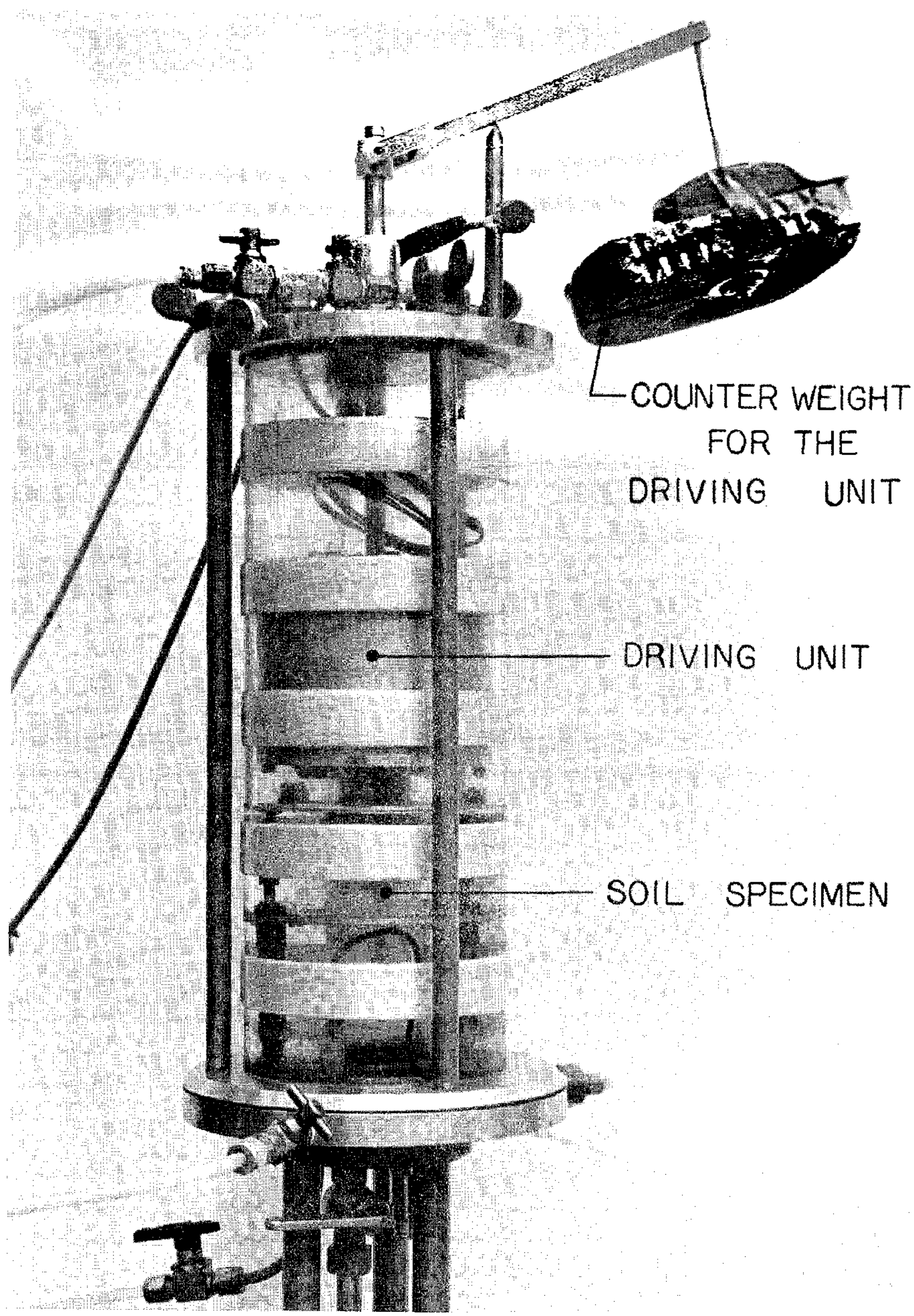


FIGURE 3.4

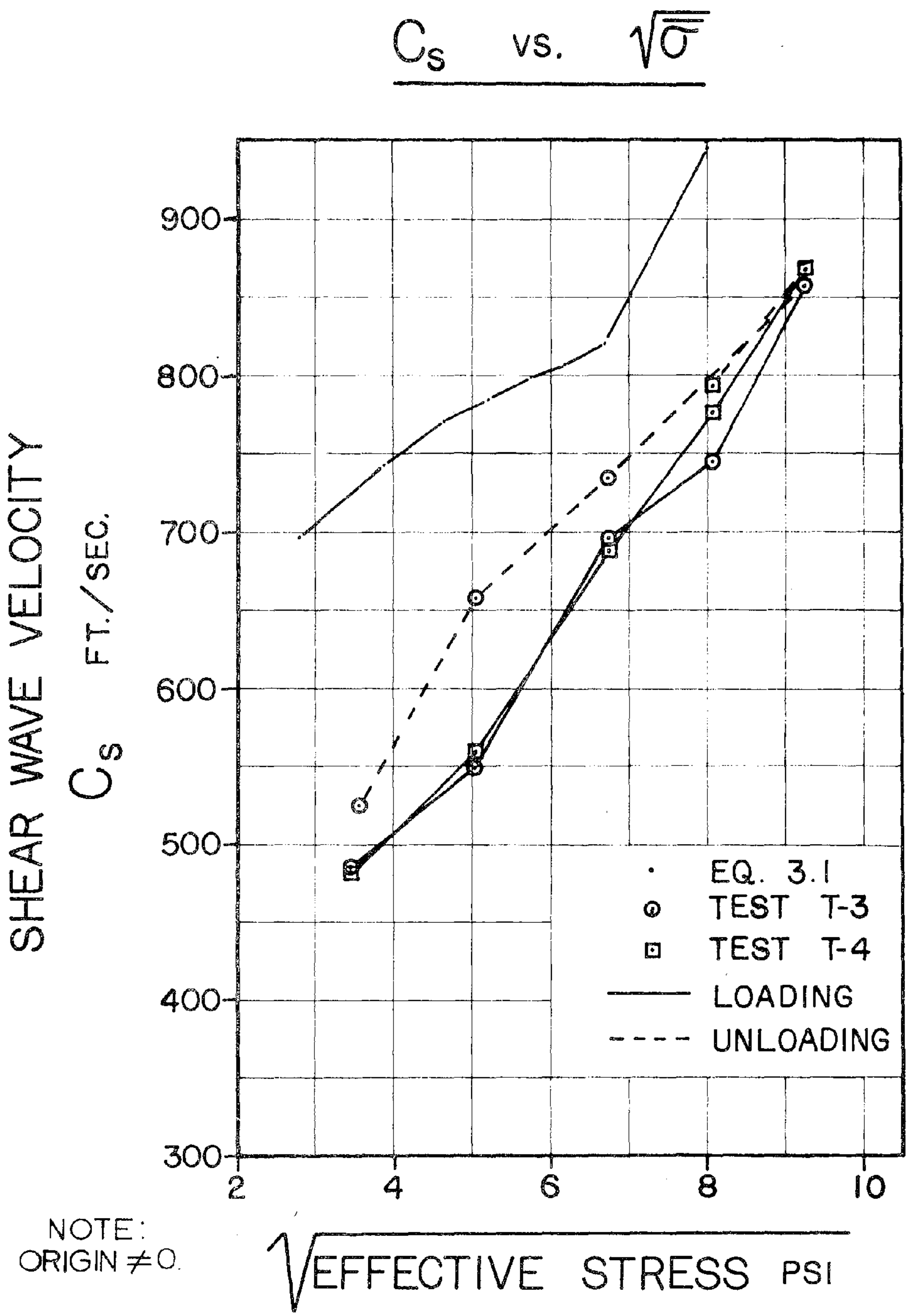
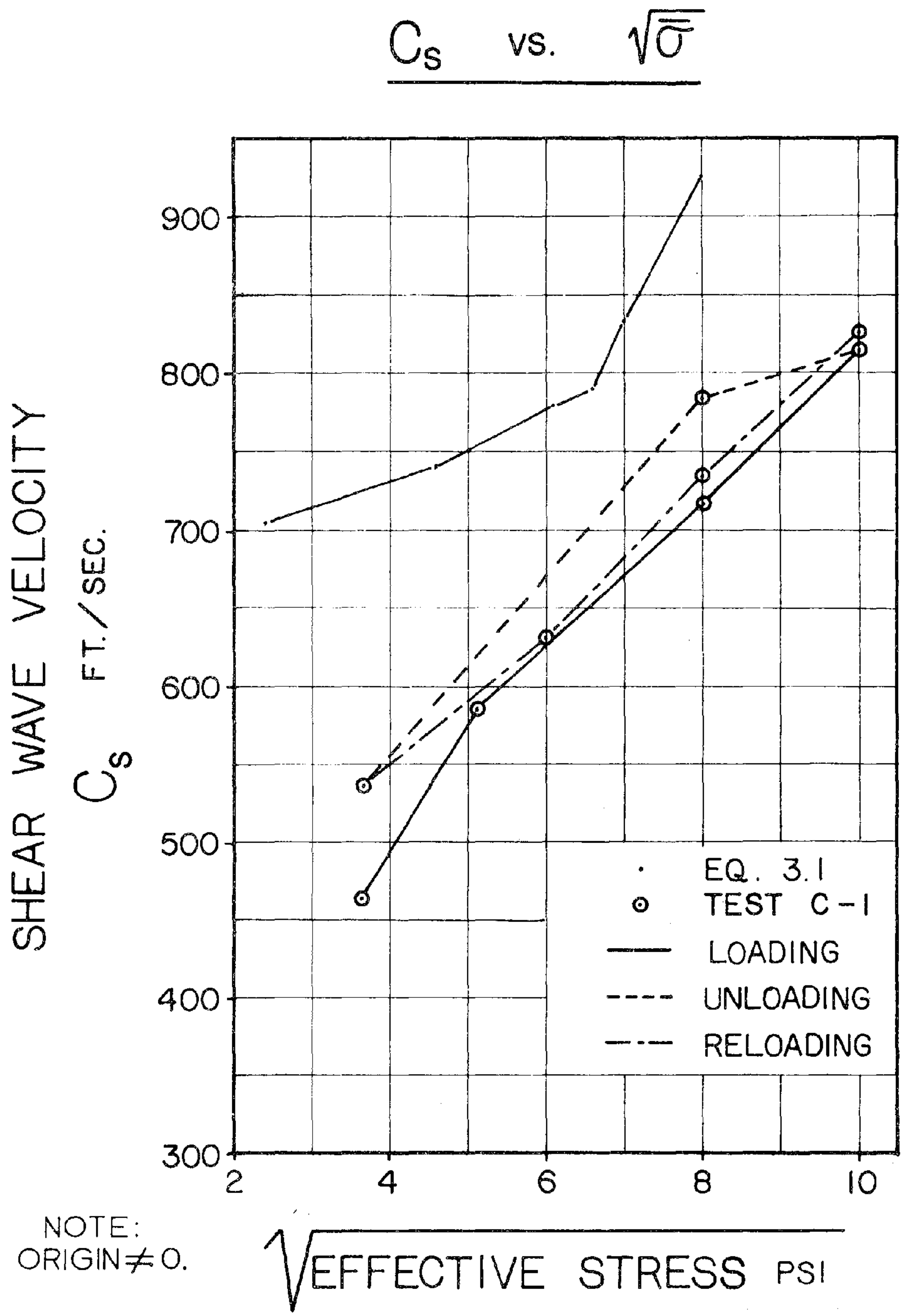

C_S VS. $\sqrt{\sigma}$
SHEAR WAVE VELOCITY
C_S FT./SEC.
900
800
700
600
500
400
300
2
4
6
8
10
EQ. 3.1
TEST T-3
TEST T-4
LOADING
UNLOADING
NOTE:
ORIGIN ≠ 0.
$\sqrt{\text{EFFECTIVE STRESS}}$ PSI

FIGURE 4.1



Cs VS. √σ
SHEAR WAVE VELOCITY
Cs FT./SEC.
900
800
700
600
500
400
300
2
4
6
8
10
EQ. 3.1
TEST C-1
LOADING
UNLOADING
RELOADING
NOTE:
ORIGIN≠0.
√EFFECTIVE STRESS PSI

FIGURE 4.2

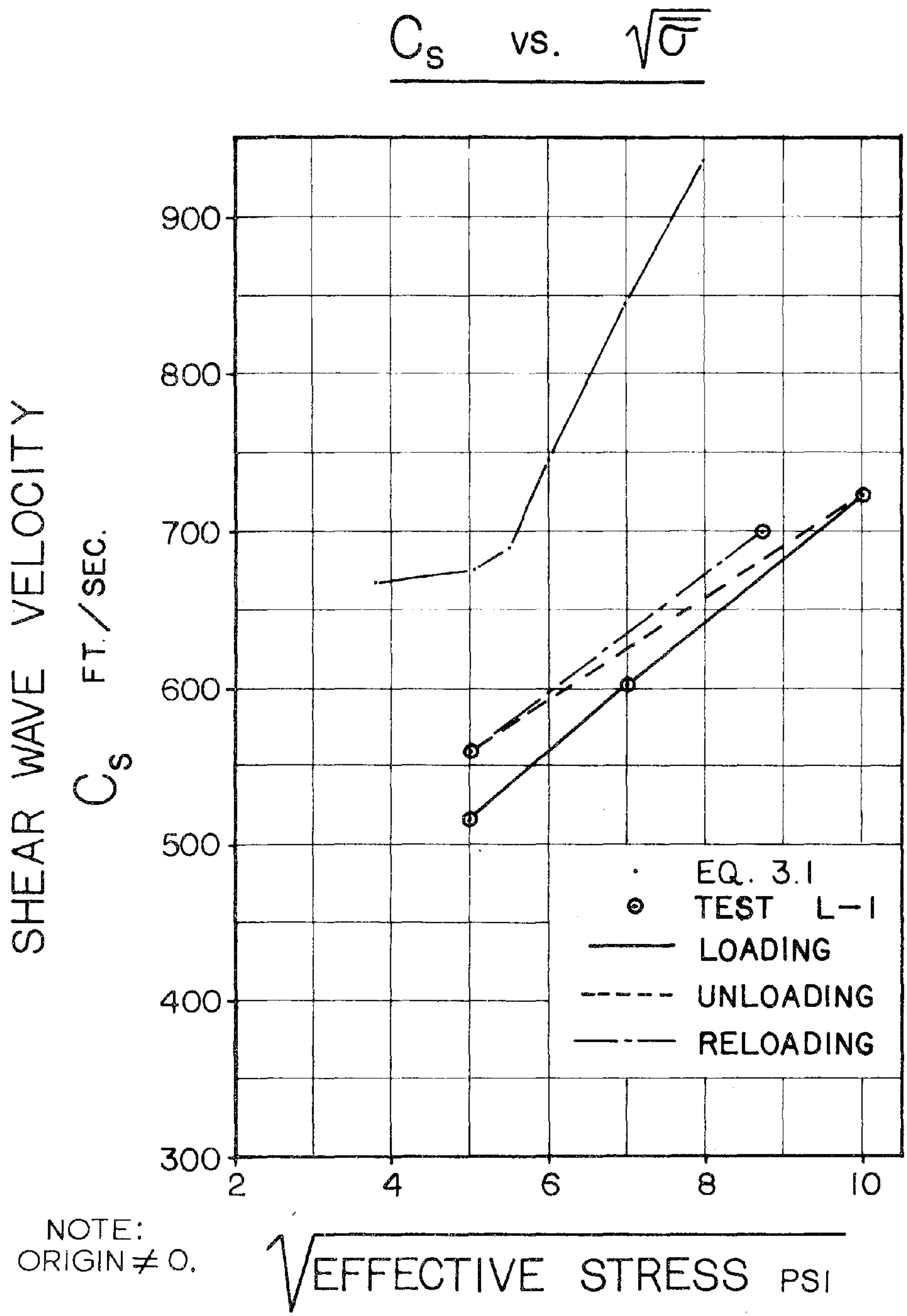
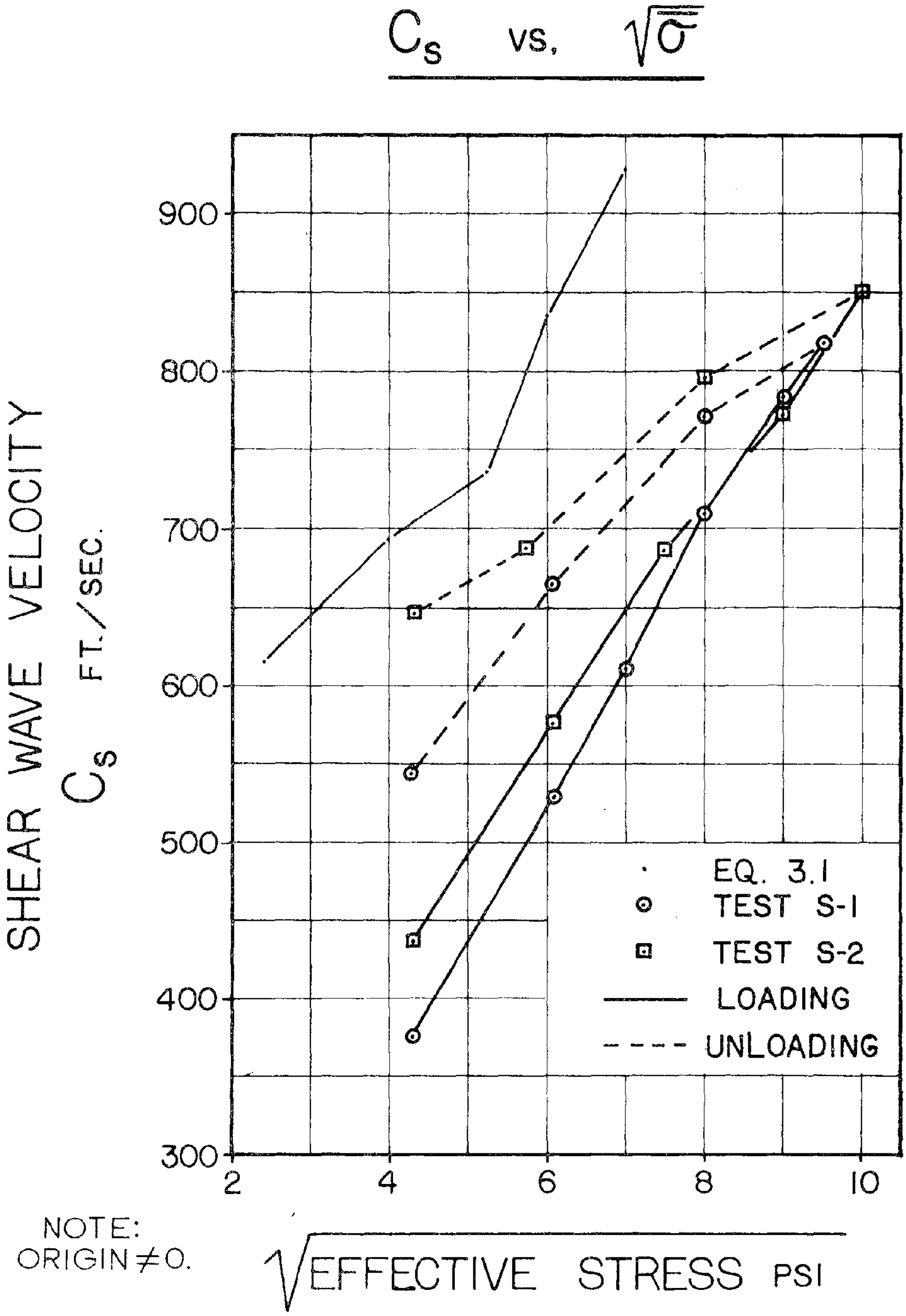

Cs vs. √σ̄
SHEAR WAVE VELOCITY
Cs FT./SEC.
900
800
700
600
500
400
300
2
4
6
8
10
EQ. 3.1
TEST L-1
LOADING
UNLOADING
RELOADING
NOTE:
ORIGIN ≠ O.
√EFFECTIVE STRESS PSI

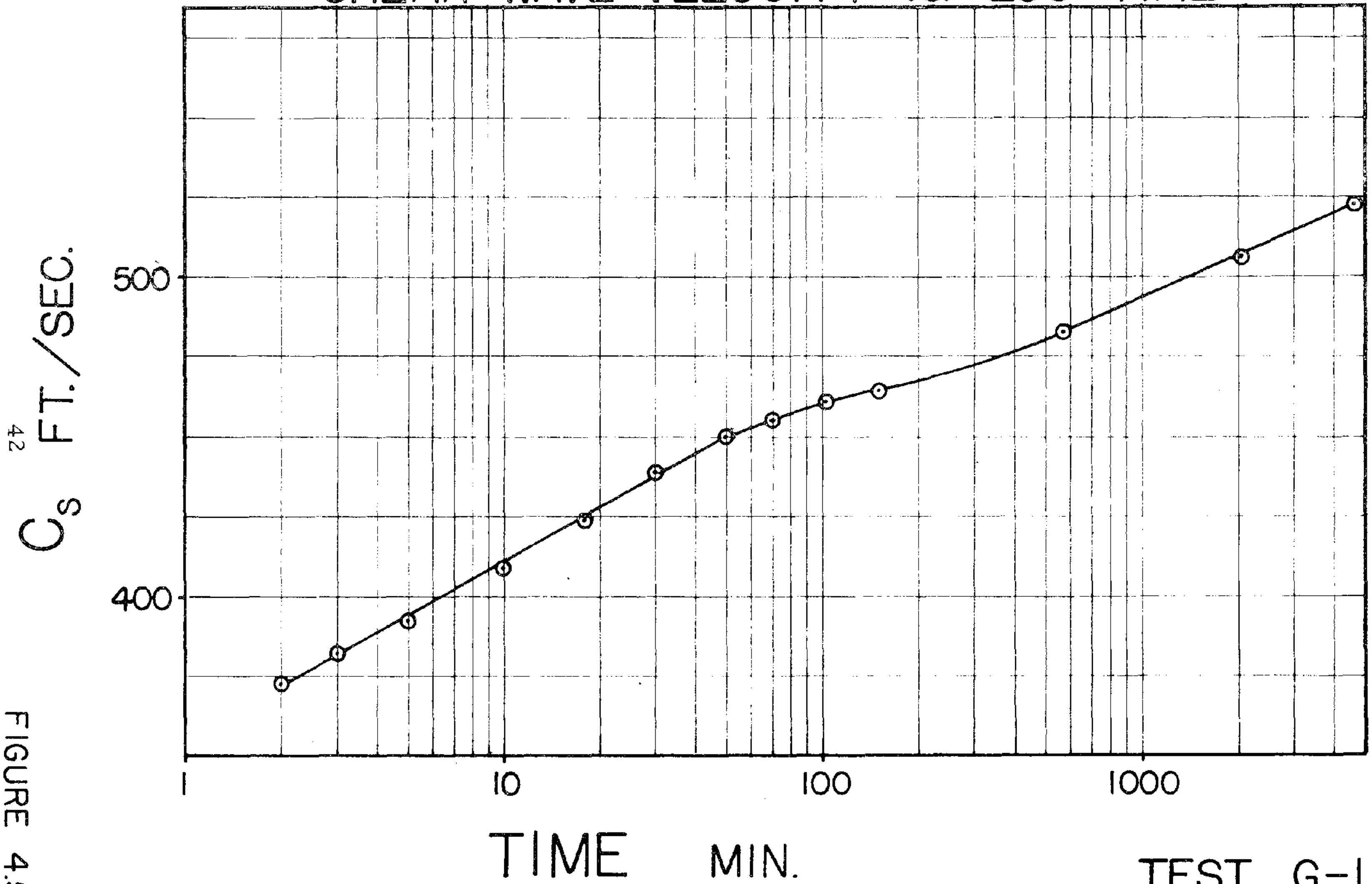
FIGURE 4.3



C_s VS. $\sqrt{\sigma}$
SHEAR WAVE VELOCITY
C_s FT./SEC.
900
800
700
600
500
400
300
2
4
6
8
10
EQ. 3.1
TEST S-1
TEST S-2
LOADING
UNLOADING
NOTE: ORIGIN ≠ 0.
$\sqrt{\text{EFFECTIVE STRESS}}$ PSI

FIGURE 4.4

SHEAR WAVE VELOCITY vs. LOG TIME



TEST G-1

FIGURE 4.5

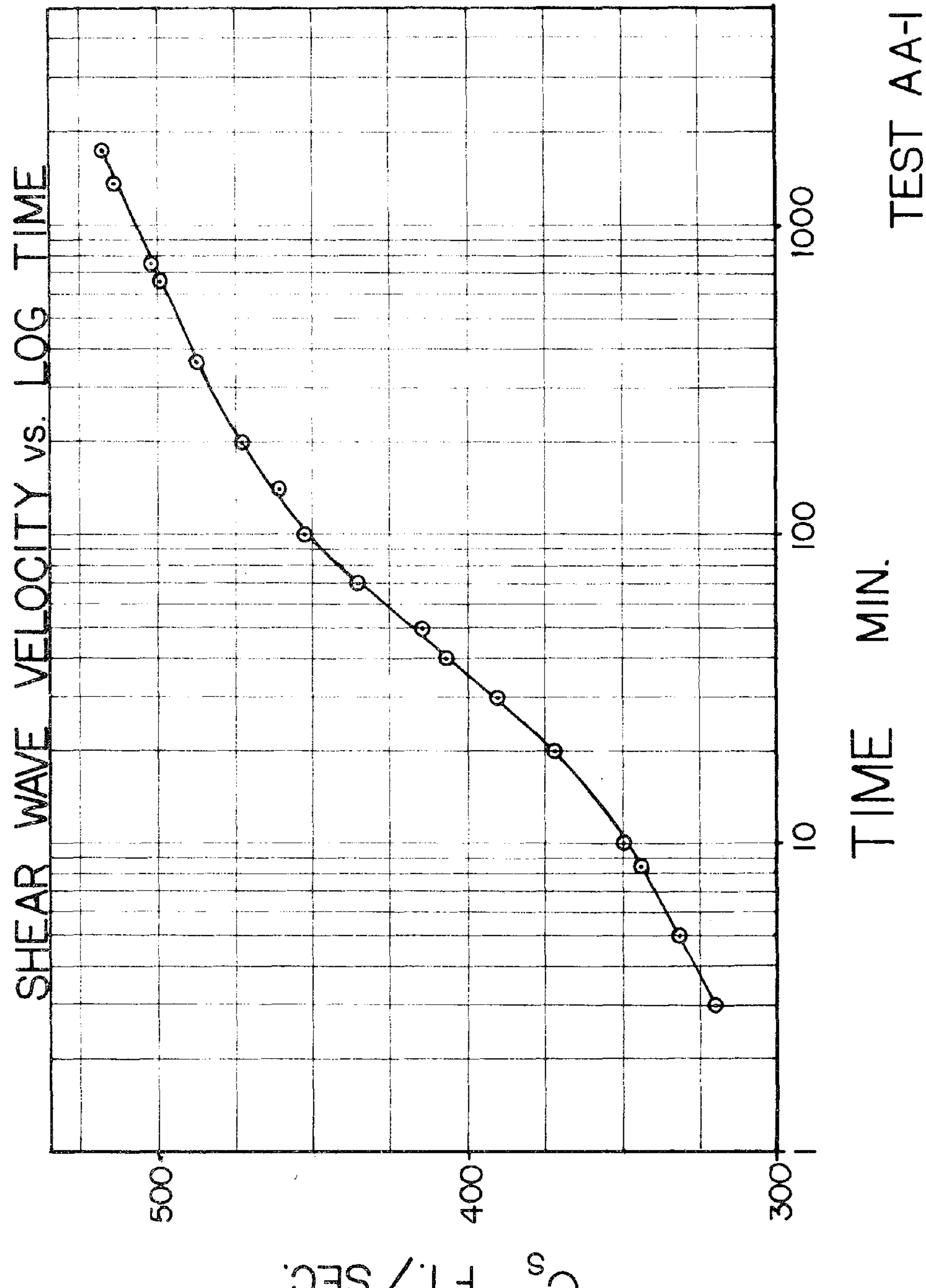
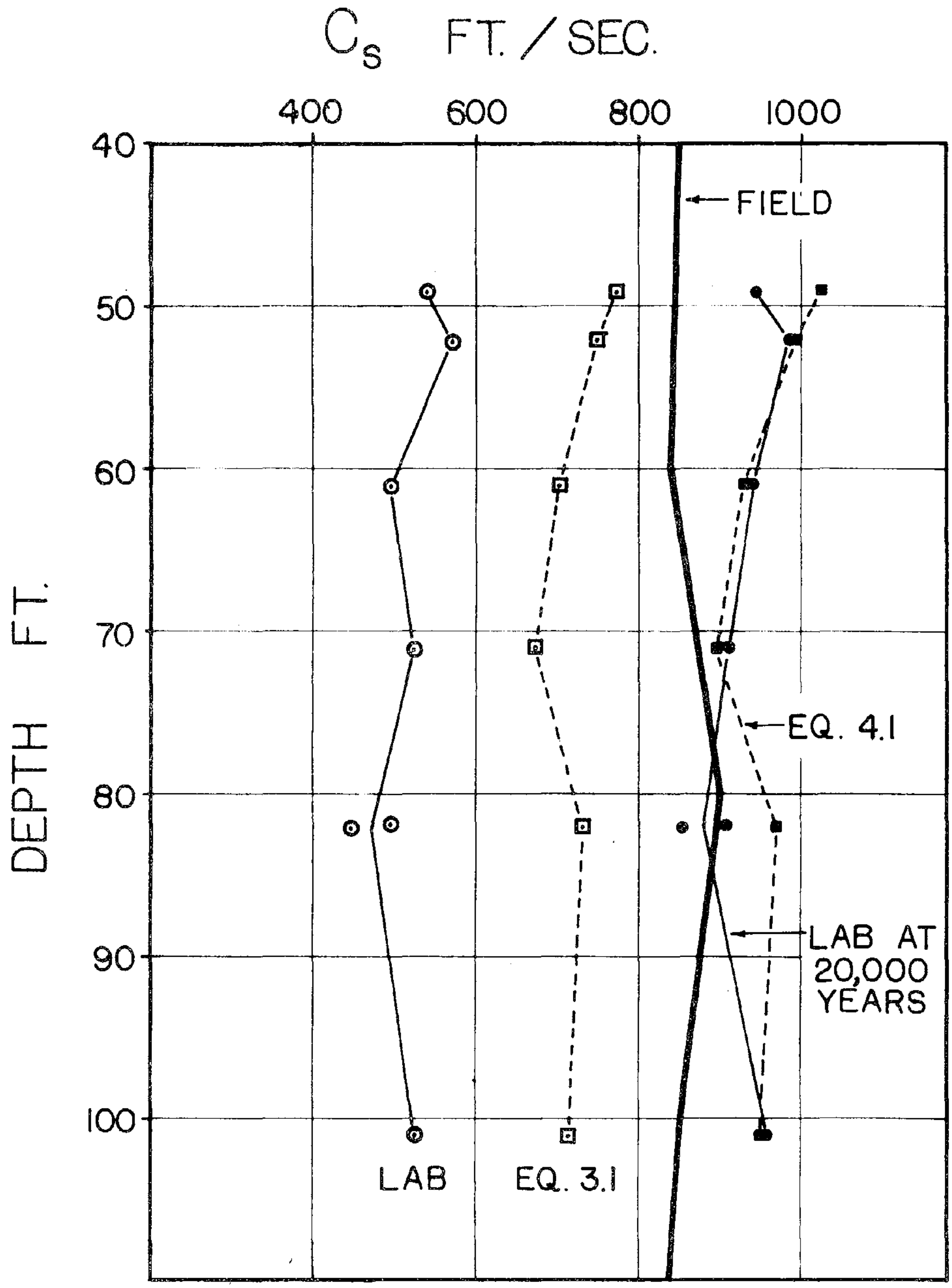

SHEAR WAVE VELOCITY vs. LOG TIME
TEST AA-I
TIME MIN.
10
100
1000
300
400
500
C_s FT./SEC.

FIGURE 4.6



SHEAR WAVE VELOCITY vs. DEPTH

FIGURE 4.7

APPENDIX A

This appendix contains the complete set of test results including data and calculations for Test G-1 as an example of the procedure used for the Hardin Oscillator tests. A detailed description of each page follows.

Page 47 includes such information about the test as date, time, boring and sample location, and a description of the sample. It also includes parameters particular to that individual test: specimen dimensions, weight, estimated in situ effective octahedral stress, total unit weight, and polar moment of inertia. These values will all be used in the calculations to come.

Page 48 is a determination of the natural water contents and Atterberg Limits of the specimen. The water content is important for determining void ratios and the Atterberg Limits are necessary to find K in the Hardin-Black equation (Equation 3.1).

Page 49 uses the Hardin-Black equation to calculate the shear wave velocity, C_s. Note the calculation of void ratio, e, for the cell pressure equal to 23.6 PSI. This calculation uses a recompression ratio, C_r, equal to 0.027 (Ladd and Luscher, 1965). If the air could be kept from diffusing into the volume change devices this change in void ratio could be measured, but in these tests air diffusion was a definite problem; therefore, average values of C_r and C_c for Boston

Blue Clay were used.

Page 50 uses the theory presented by Hardin (1970) or Hardin and Music (1965). Using the parameters for the specimen and the calibration constants for the apparatus, the value of C_s is found for different values of T_n, the resonant period. A_{T_n}, the accelerometer output for a certain shear strain (equal to 1×10^{-5} in/in) is also found. The input voltage (from the Hewlett Packard audio oscillator) is adjusted such that the output of the accelerometer at resonance, as measured with the AC voltmeter, equals the desired A_{T_n}.

On page 51 (Figure A-1) the values of C_s and A_{T_n} are plotted vs. period. This plot allows the operator to run the dynamic test at the desired strain level and also allows immediate determination of the shear wave velocity without any tedious calculations.

Page 52 is a data sheet for running the test to determine C_s vs. Log time. The values of C_s are plotted during the test on page 53 (Figure A-2). Page 54 (Figure A-3) shows the value of F as a function of Z as given in the theory presented by Hardin (1970) or Hardin and Music (1965).

16 DEC. 72 SAMPLE U-7 BORING B-1

TIME 09:47 DEPTH 60.5'-62.5'

TEST G-1 BOSTON QUAKE

DESCRIPTION: MEDIUM BBC, VERY LITTLE SILT

SPECIMEN DEPTH: 61.'

ESTIMATED $\bar{\sigma}_v = 30.$ $\bar{\sigma}_{vm} = 53.8$ PSI

OCR = 1.79 $\longrightarrow$ $K_o = 0.68$ (LADD, 1965)

$$\text{IN SITU } \bar{\sigma}_{OCT} = \frac{30. + 2\,(0.68)\,(30.)}{3} = 23.6 \text{ PSI}$$

BEFORE TEST:

WEIGHT = 148.62 GRAMS

LENGTH = 8.00 CM.

RADIUS = 1.79 CM.

$$\longrightarrow \quad \gamma = 1.846 \text{ G./CM.}^3$$

$$I = \frac{\pi}{2}\,\gamma\, L R^4 = 238.10 \text{ G-CM}^2$$

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

SOIL SAMPLE MEDIUM BBC

LOCATION BOSTON QUAKE

BORING NO. B-1 SAMPLE DEPTH 61'

SAMPLE NO. U-7

SPECIFIC GRAVITY, G_s, 2.78

TEST NO. G-1

DATE 15 DEC 72

TESTED BY P. J. T.

PLASTIC LIMIT

DETERMINATION NO.	1	2	3
CONTAINER NO.	P 6	P 3	
WT. CONTAINER + WET SOIL IN g	4.38	4.55	
WT. CONTAINER + DRY SOIL IN g	4.19	4.35	
WT. WATER, W_w, IN g	0.19	0.20	
WT. CONTAINER IN g	3.33	3.47	
WT. DRY SOIL, W_s, IN g	0.86	0.88	
WATER CONTENT w, IN %	22.1	22.7	

NATURAL WATER CONTENT

DETERMINATION NO.	1	2	3
CONTAINER NO.	11	12	13
WT. CONTAINER + WET SOIL IN g	18.87	16.43	19.78
WT. CONTAINER + DRY SOIL IN g	14.61	12.14	15.27
WT. WATER, W_w, IN g	4.26	4.29	4.51
WT. CONTAINER IN g	3.73	1.53	3.68
WT. DRY SOIL, W_s, IN g	10.88	10.61	11.59
WATER CONTENT w, IN %	39.2	40.4	38.9

LIQUID LIMIT

DETERMINATION NO.	1	2	3	4	5
NO. OF BLOWS	21	26	32	36	
CONTAINER NO.	18	N7	X3	1	
WT. CONTAINER + WET SOIL IN g	18.68	16.98	19.68	12.65	
WT. CONTAINER + DRY SOIL IN g	16.38	14.60	17.03	10.63	
WT. WATER, W_w, IN g	2.30	2.38	2.65	2.02	
WT. CONTAINER IN g	12.10	10.01	11.87	6.63	
WT. DRY SOIL, W_s, IN g	4.28	4.59	5.16	4.00	
WATER CONTENT w, IN %	53.7	51.9	51.4	50.5	

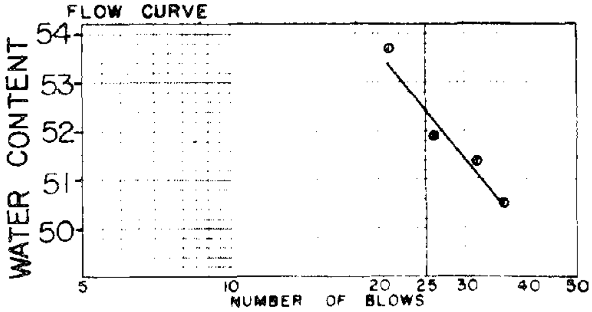
WATER–PLASTICITY RATIO, $B = \frac{w_n - w_p}{w_l - w_p}$

SHRINKAGE LIMIT

DETERMINATION NO.	1	2
UNDISTURBED OR REMOLDED SOIL PAT		
WT. DRY SOIL PAT, W_s, IN g		
WT. CONTAINER + HG. IN g		
WT. CONTAINER IN g		
WT HG. IN g		
VOL. SOIL PAT, V, IN cc		
SHRINKAGE LIMIT, w_s, IN %		

$w_s = \frac{\gamma_w V}{W_s} - \frac{G_T}{G_s}$

FLOW CURVE



RESULT SUMMARY

PLASTIC LIMIT	NATURAL WATER CONTENT	LIQUID LIMIT	SHRINKAGE LIMIT	B VALUE	PLASTICITY INDEX	TOUGHNESS INDEX	FLOW INDEX	
22.4	39.5	52.4			30.			

REMARKS

TEST G-1

CALCULATE C_S USING HARDIN-BLACK

$$\omega_N = 39.5 = \frac{W_W}{W_S} \qquad W = W_W + W_S = 148.62 \text{ G}$$

$$\longrightarrow \quad W_S = \frac{148.62}{1.395} = 106.4 \text{ G} \qquad W_W = 42.2 \text{ G}$$

ASSUME $S = 100\ \%$ $\quad \therefore\ V_V = 42.2\ \text{CM}^3$

$$V_S = \frac{106.4}{\underset{G_s}{(2.78)}\underset{\gamma_w}{(1)}} = 38.3 \quad \text{CM}^3$$

$$\longrightarrow \quad e_o = \frac{42.2}{38.3} = 1.10$$

NOTE: $\bar{\sigma}_o = 23.6$ PSI $\quad$ OCR $= 1.79$ $\quad$ PI $= 30 \longrightarrow K = 0.24$

$$\Delta e_{23.6} = 0.027 \times \text{LOG}\ \frac{23.6}{0.1} = 0.04 \qquad \longrightarrow e_{23.6} = 1.06$$

$$G_{MAX} = 1230.\ \frac{(2.973 - 1.06)^2}{1 + 1.06}\ 1.79^{0.24}\ 23.6^{1/2} \qquad \text{PSI}$$

$$= 12{,}212.\ \text{PSI}$$

$$C_S = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{12{,}212 \times 144 \times 32.2}{1.846 \times 62.4}}$$

$$C_S = 701 \quad \text{FT./SEC.}$$

TEST G-1

SHEAR WAVE VELOCITY VS. PERIOD

$$\frac{I_o}{I} = \frac{2439.}{238.1} = 10.245$$

$$\frac{K_o T_n^2}{4\pi^2 I} = \frac{3.389 \times 10^9}{(39.48)(238.1)} T_n^2 = 0.3605\ T_n^2$$

(T_n IN MSEC)

$$Z = 10.245 - 0.3605\ T_n^2$$

F = FUNCTION (Z) SEE FIGURE A-3

$$C_s = \frac{3.281\ \ 2\pi L}{T_n\ F\ 10^2} = \frac{1649.}{T_n F}$$

T_n mSec

$$A_T = 5.49 \frac{L}{R} \frac{1}{T_n^2} \gamma_{\theta z}$$

M SEC T_N	RMS MV A_T	Z	F	FPS C_S
5.1	9.43	0.867	0.907	356
5.0	9.81	1.231	0.792	416
4.9	10.21	1.588	0.721	467
4.8	10.64	1.938	0.663	518
4.7	11.10	2.280	0.620	566

C_S AND A_T VS. PERIOD

FOR TEST G-1

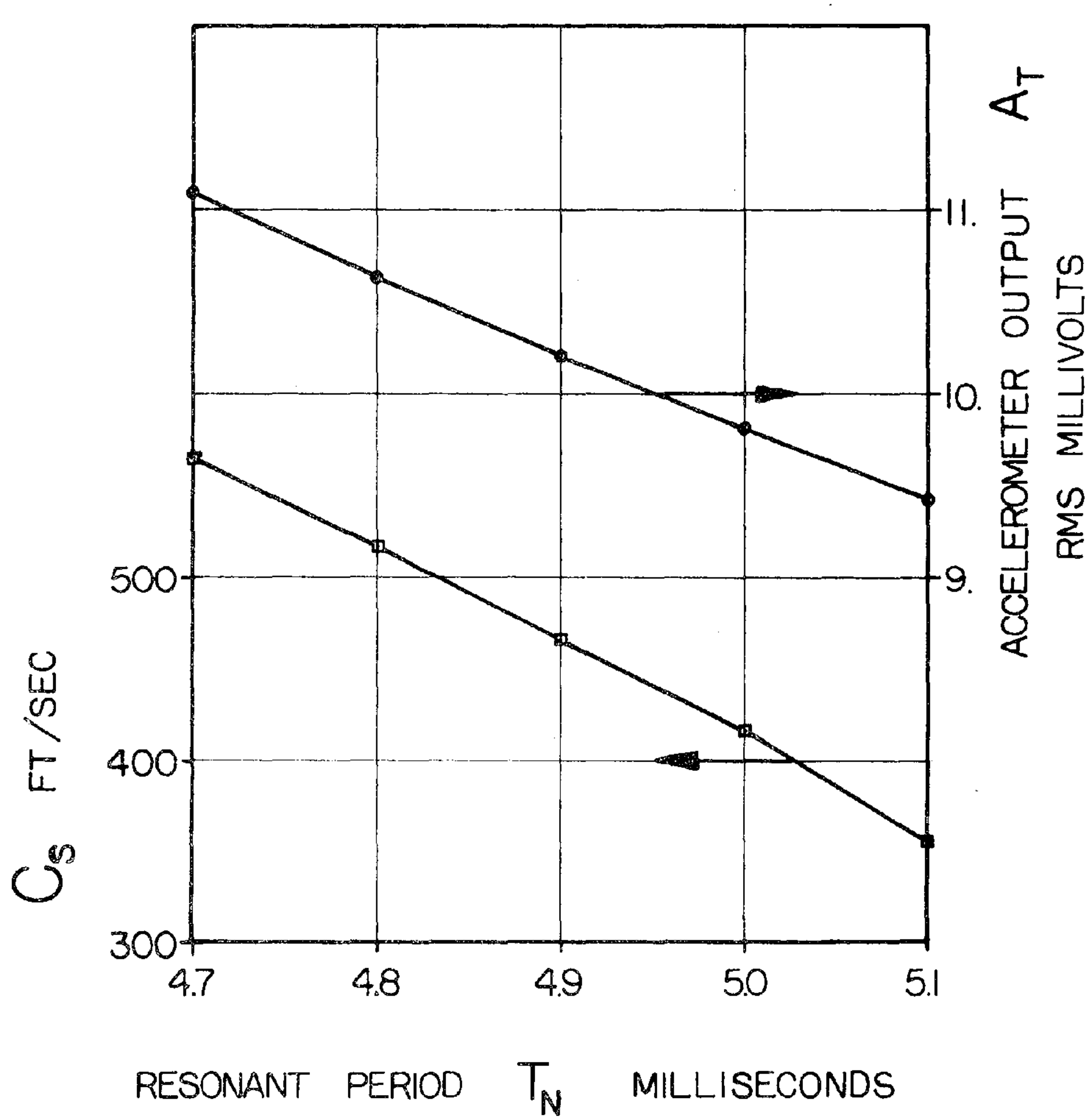


FIGURE A-1

SAMPLE DATA SHEET

TEST G-1 $\gamma_{\theta z} = 1 \times 10^{-5} \frac{IN}{IN}$

SAMPLE U-7

DEPTH 61. FT. AC NOISE < 0.1 RMS mV

DATE	TIME	Δ TIME MIN	$\overline{\sigma}_o$ PSI	T_N mS	A_{T_N} rms mV	C_S FPS
16 DEC	10:34	0	23.6	—	—	—
	10:36	2	23.6	5.071	9.4	373
	10:37	3	23.6	5.057	9.4	382
	10:39	5	23.6	5.040	9.4	392
	10:44	10	23.6	5.011	9.9	409
	10:52	18	23.6	4.984	9.9	424
	11:04	30	23.6	4.954	10.1	439
	11:24	50	23.6	4.932	10.2	450
	11:44	70	23.6	4.923	10.2	455
	12:17	103	23.6	4.913	10.2	461
	13:04	150	23.6	4.906	10.3	464
	20:00	566	23.6	4.867	10.4	483
17 DEC	20:42	2048	23.6	4.823	10.6	506
19 DEC	16:55	4660	23.6	4.790	10.8	523
20 DEC	10:08	5696	23.6	4.763	10.9	536

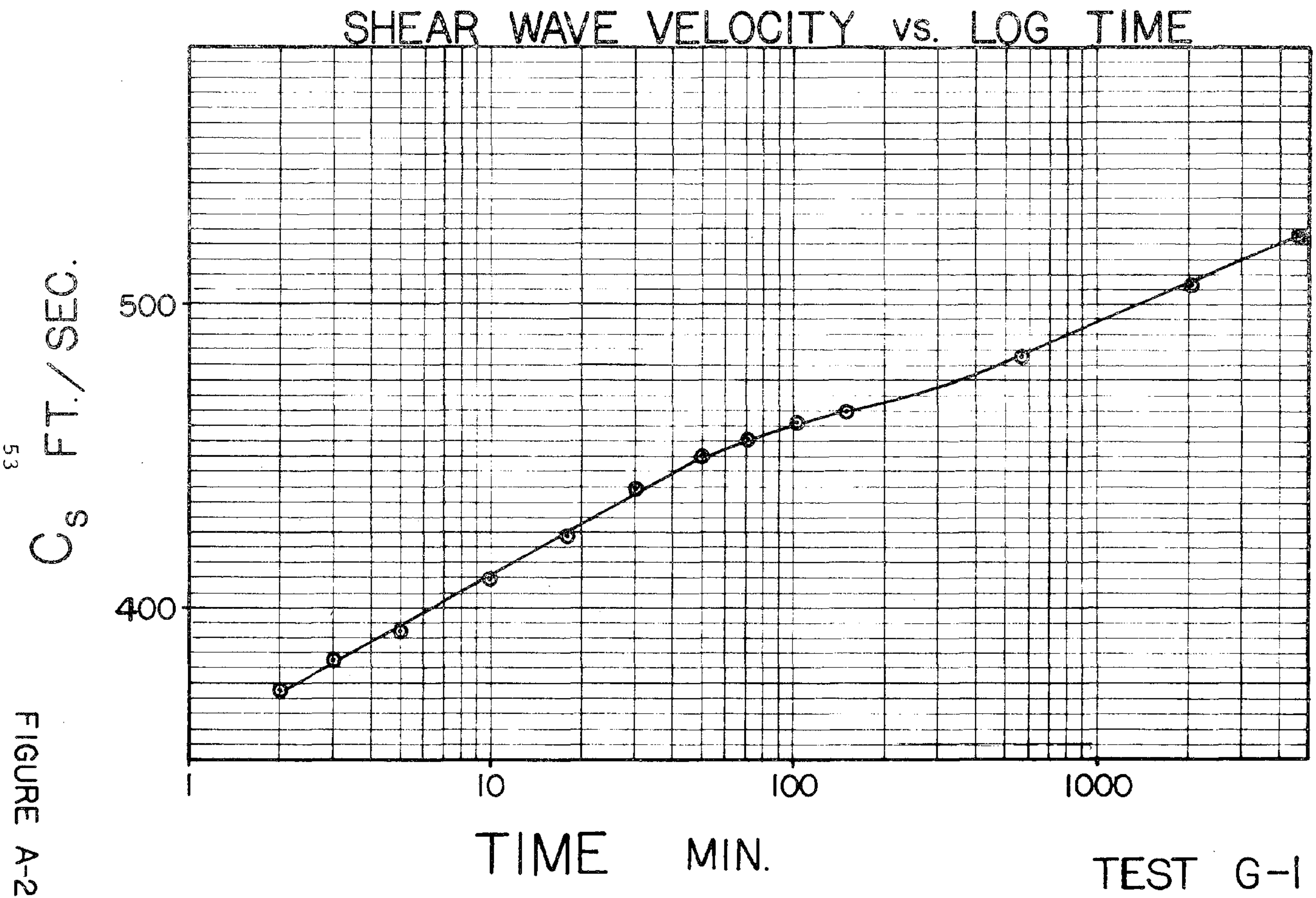

SHEAR WAVE VELOCITY vs. LOG TIME
C_s FT./SEC.
500
400
1
10
100
1000
TIME MIN.
TEST G-1

FIGURE A-2

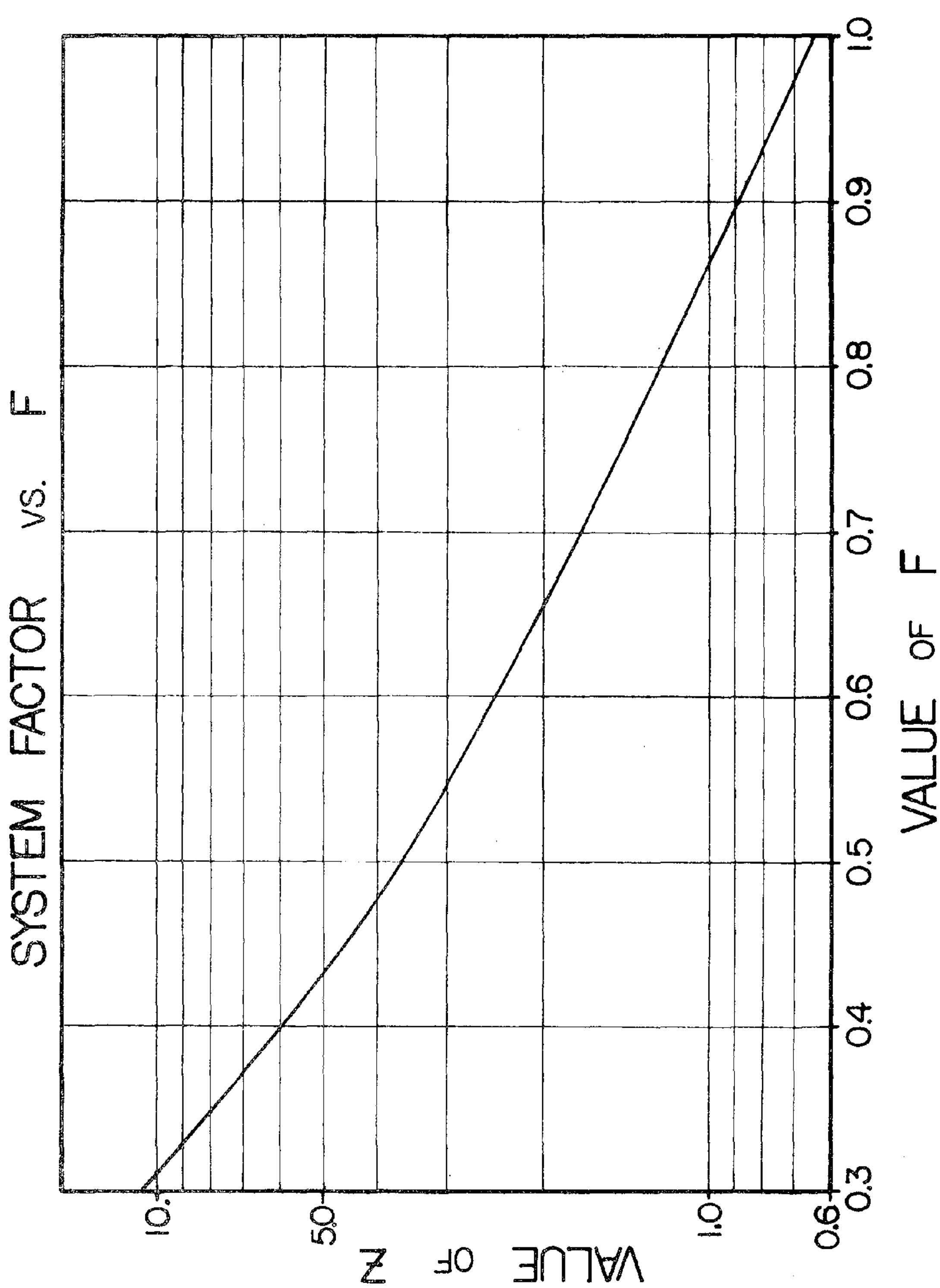

SYSTEM FACTOR vs. F
VALUE OF F
VALUE OF Z
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
10.
5.0
1.0
0.6

FIGURE A-3

APPENDIX B

This appendix delineates the history of the Hardin tests presented in Chapters III and IV. It includes a description of each test with pertinent information concerning dates, testing procedures used, problems encountered, and actions taken to lessen these problems, in an attempt to provide the reader some means of interpreting the test results.

There are two dates of particular note. First, on November 8, 1972, Dr. Kenneth Stokoe, who had worked with the Hardin apparatus at the University of Michigan, visited MIT and presented constructive comment concerning the procedure that was being used at MIT at that time. He suggested performing the test at a strain level of 1×10^{-5} in/in and finding values of C_s with time to prepare plots similar to Figures 4.5 and 4.6. Consequently, later tests were performed in this manner yielding results which were consistent with the general trends mentioned by Stokoe. The second important dates are November 27 through December 5, 1972, during which the electronic equipment was repaired and re-calibrated. Attempting to perform the dynamic test at a shear strain of 1×10^{-5} in/in as suggested by Stokoe indicated interference due to AC noise. Investigation concluded that the cathode follower was at fault, and thus, was repaired. Although this did not affect the values of resonant frequency (and consequently, the shear wave velocity,) it did indicate that the values of shear strain obtained prior to this date were incorrect. An attempt was made to made to develop best estimates of the shear strains in prior tests by comparing values of the input voltage required to obtain a certain strain level with the repaired equipment to recorded values obtained during the prior tests. Since this input voltage data was available, the estimated values of the shear strains are probably close to the actual values.

On the following pages will be found a description of each test --- specimen, procedure, and problems. Note the calendar of events on the next page.

CALENDER OF EVENTS

TEST	FROM	TO
T-3	23 Sept.	2 Oct.
T-4	11 Oct.	16 Oct.
S-1	20 Oct.	1 Nov.
S-2	1 Nov.	12 Nov.

Visit by Dr. Kenneth Stokoe on November 8, 1972

TEST	FROM	TO
C-1	12 Nov.	22 Nov.
C-2	22 Nov.	5 Dec.

Repair and re-calibrate electronic equipment from November 27 to December 5, 1972

TEST	FROM	TO
L-1	5 Dec.	13 Dec.
AA-1	13 Dec.	15 Dec.
G-1	15 Dec.	26 Dec.

TEST DESCRIPTIONS

TEST T-3: The test specimen was silty, medium BBC. An attempt to measure volume changes using a burette and a backpressure equal to 10 PSI did not work. At effective cell pressures greater than 25 PSI the consolidation data appeared as though there was a leak in the membrane due to air diffusion (see Chapter III). The dynamic test was run at the estimated in situ horizontal effective stress, then the vertical, and then in 20 PSI increments up to 95 PSI. The specimen was then unloaded in several steps, performing the dynamic test at each step. Note that water was sucked into the sample during rebound and this, combined with air diffusion, caused problems during rebound. The rebound values of C_s reported in Figure 4.1 were taken after flushing water through the porous stone.

TEST T-4: This test was on a specimen from the same tube as Test T-3. The purpose of this test was to determine the reproducibility of C_s performing the test in this manner; therefore, this test was performed in the same manner as Test T-3, with essentially the same results. Note however, that during the test, higher (than expected) resonant frequencies were observed, but when the apparatus was allowed to vibrate for a few minutes, these frequencies would diminish to the expected values. Upon dismantling the apparatus at the end of the test a magnet was found to be dislodged from its proper place. With further investigation, the higher frequencies could be reproduced by shifting the magnet to and fro such that when it was in the proper place the correct frequency was obtained, but when it was not, the higher frequency was found. The magnet was then reglued to its proper position and the calibration of the equipment checked.

TESTS S-1 and S-2: These tests were performed on specimens of medium BBC with many silt lenses using the same testing procedure as the T tests. Once again there was the problem of air diffusion, which appeared to be leakage on the consolidation time plots. This is evident in the rebound (unloading) curve in Figure 4.4 in which there is some question as to whether the appropriate effective stress is plotted.

TESTS C-1 and C-2: These tests were run taking C_s data with time to produce C_s vs. Log time plots. This was suggested by Stokoe and utilized to give a feel for what happens during consolidation. An attempt was made to perform these tests at a shear strain of 1×10^{-5} in/in , but the AC noise in the electronic equipment interfered with the accelerometer output. Therefore, the electronic equipment was repaired and re-calibrated. Test C-1 was run at different cell pressures (Figure 4.2) to try to correlate these results with the results of the previous tests. Test C-2 was run only at the in situ effective octahedral stress due to a time limitation.

TEST L-1: This test was run at a small shear strain obtaining C_s vs. time for several consolidation pressures. The test consistently showed the linear trend of C_s vs. Log time in secondary consolidation. It also showed that consistent results of C_s (resonant frequency) could be obtained if the specimen was kept wet by flushing water through the porous stone daily. If the filter strips were allowed to dry due to air diffusion, then this procedure resulted in a decrease in C_s. Therefore, the author suggests flushing water through the porous stone daily. An attempt was made to supply a reservoir of water at atmospheric pressure, but this did not work. The air had to be forced out of the stone by pumping water through.

TEST AA-1: This test was performed only at the in situ effective octahedral stress on a specimen of soft, normally consolidated BBC with a one inch sand layer in the middle. This sand layer contained particles up to 1/4 inch. The results of C_s vs. Log time are shown in Figure 4.6.

TEST G-1: This test was performed in essentially the same manner as Test AA-1, except a backpressure of 30 PSI was used. The specimen was medium BBC with little silt. The results of the first day of testing are illustrated in Figure 4.5. An attempt was made to determine long term (time) effects, but air diffusion (see Chapter III) gave rise to erratic results after the fourth day of testing.

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