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SOIL BEHAVIOR UNDER EARTHQUAKE LOADING CONDITIONS

IN SITU IMPULSE TEST FOR DETERMINATION OF SHEAR MODULUS FOR SEISMIC RESPONSE ANALYSES PROGRESS REPORT

Prepared Under
Contract No. AT (04-3)-954

U.S. ATOMIC ENERGY COMMISSION
DIVISION OF REACTOR SAFETY RESEARCH
WASHINGTON, D.C.

JUNE 1974

SW-AA

MASTER

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FOREWORD

This report presents the results of combined experimental and analytical studies undertaken to continue development of an in situ impulse test for the determination of the soil shear modulus. Emphasis of the field work was directed toward making the field measurements at frequent depth intervals and at shear strains in the strong motion earthquake range. Emphasis of the analytical work was aimed toward supporting the field effort through processing and evaluation of the experimental test results combined with additional calculations required to gain insight into data interpretation and the in situ test setup itself. This work represents a part of continuing studies to evaluate free field soil behavior under earthquake loading conditions. This and the other related studies each provide important steps in the overall project for determination of the best methods of evaluation and prediction of soil behavior of potential nuclear power plant sites under seismic loading conditions.

This is the fourth progress report prepared by the joint venture of Shannon & Wilson, Inc., (SW), and Agbabian Associates, (AA), on the in situ test development project. The current work was conducted under Contract No. AT(04-3)-954 between the joint venture (SW-AA) and the United States Atomic Energy Commission.

A number of contributors from both members of the joint venture assisted in the preparation of this report. For Shannon & Wilson, Inc. the primary contributors were Mr. S. D. Wilson, Mr. F. R. Brown and Mr. J. H. Troncoso. Dr. R. P. Miller was the Project Engineer for Shannon & Wilson and is also the Project Manager for the joint venture. For Agbabian Associates the primary contributors were Mr. S. D. Werner,

Project Engineer, assisted by Mr. L. Enochson, Mr. D. Van Dillen, and Mr. K. Boisen. Dr. Lydik S. Jacobsen, special consultant to the joint venture on this aspect of the project, also provided valuable guidance in these studies.

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SYMBOLS

<u>SYMBOL</u>	<u>REPRESENTS</u>
A	Surface area of the three plates on the anchor
a	Constant acceleration
$a_{\max}$	Maximum acceleration of anchor
c_1, c_2	Dilatational and shear wave velocities (in Appendix D)
$D(r)$	Envelope of peak time history response
d	Displacement
E	Modulus of Elasticity
$F(t)$	Force applied to the anchor with time
$F_d(t)$	Dynamic inertia force of the anchor with time
$F_{d\max}$	Maximum dynamic inertia force of the anchor
$F_s(t)$	Shear force at the soil anchor interface with time
G	Shear modulus
G_s	Specific gravity of solids
J_0, Y_0, J_1, Y_1	Bessel functions
K	Constant for normalizing the modulus for sands
k_c	Spring constant (Belleville)
LL	Liquid Limit
m	Mass of anchor
M.A.	Mechanical analysis

SYMBOLS (Contd)

PI	Plasticity index
PL	Plastic limit
Q	Unconsolidated undrained triaxial compression test
Q_u	Unconfined compression test
$\bar{R}$	Consolidated undrained triaxial compression test
$\bar{R}_{cy}$	Consolidated undrained cyclic triaxial test
$R(\tau)$	Cross-correlation function
r	Radial distance from energy source
$S(t)$	Time varying shear stress
T, t	Time
U.S.C.	Unified Soil Classification System
u_r	Radial component of displacement
u_z	Axial component of displacement
V_s	Shear wave velocity
V_p	Particle velocity amplitude
v_{pa}	Particle velocity at crest of wave
W_A	Weight of anchor
W_H	Weight of hammer
W.C.	Water Content
$W(r)$	Displacement profile function at time t
$\dot{y}_n$	Particle velocity signal at sensor n

Greek Letters

α	Shear deformation
γ	Shear strain
γ_d	Dry unit weight
Δt	Delay time
ϵ_f	Axial strain at failure
μ	Shear modulus of the medium (in Appendix D)
μ	Poisson's ratio
ρ	Mass density of the medium
σ	Axial increment of stress
σ_1	Total major principal stress
σ_3	Total minor principal stress
σ_d	Deviator stress
$\sigma_{rr}, \sigma_{\theta\theta},$ σ_{zz}, σ_{rz}	Components of stress
$\bar{\sigma}_0$	Octahedral normal stress
$\bar{\sigma}_1, \bar{\sigma}_2, \bar{\sigma}_3$	Effective major, intermediate, and minor principal stresses
τ	Shear stress
τ	Time shift by cross correlation procedure
ω_{nm}	An eigenvalue - the natural frequency of a given free vibration mode of the medium

IN SITU IMPULSE TEST
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FOR SEISMIC RESPONSE ANALYSES
PROGRESS REPORT

I. INTRODUCTION

1.1 PURPOSE AND OBJECTIVE

Because of the increasing number of nuclear power plants required in the coming years to meet power requirements, public concern that they be designed and operated safely has likewise increased. The Atomic Energy Commission (AEC), responsible for regulating all uses of nuclear energy, must therefore keep abreast of the latest engineering technology from the design and construction stages through actual plant operation. Where data and techniques are seemingly lacking or are not sufficiently developed, the AEC is continuing to support through research the development of this new technology. As a result, considerable effort has in the past and continues to be directed toward developing more rational techniques for earthquake resistant structure design, soil deposit response analysis and evaluation of dynamic soil-structure interaction effects during seismic loading. As more and more sophisticated analytical techniques are developed and refined through the availability of high speed computers, more accurate input parameters must likewise be obtained.

Analytical techniques now used to predict the local ground response during earthquakes require the input of well defined dynamic properties of the underlying soil. One of these properties is the dynamic shear modulus (or an equivalent elastic modulus). Since the shear modulus is both non-

linear and strain dependent, definition of an appropriate value or values during earthquake shaking (covering a wide range of applied strains) is not easily specified. This modulus is presently obtained either in the laboratory or in situ while applying dynamic loads to an element or a mass of soil. The procedures for these existing tests together with the limitations of each test are described in a previous report (SW-AJA, 1972). The tests have a common limitation: modulus determinations are not made at strain levels of interest in the strong motion region.

In situ techniques presently in use induce strains significantly smaller than those produced by earthquakes in the strong motion region, while conventional laboratory tests are performed at strains either higher or lower than those produced by strong motion earthquake shaking. Laboratory test results are further complicated by: 1) specimen disturbance that occurs during field sampling operations, and 2) the inability to reapply the natural in situ stress conditions to the test specimen. Also, effects of soil characteristics such as previous stress history and cementation can rarely be accounted for in the laboratory.

Regardless of the test method used, existing tests require extrapolation from the measured moduli data to values corresponding to the strains of interest (10^{-3} to 10^{-1} percent). To assure that extrapolation techniques of existing field test data are at least reasonably correct and that measured laboratory values of moduli are consistent with those occurring in the field during earthquakes, this new in situ test was developed. The objective of the test is to obtain moduli data in situ at frequent depth intervals and at strains in the strong motion earthquake range.

This objective was achieved by utilizing a modified

cross-hole wave propagation procedure with velocity sensors located in closely spaced adjacent borings. A schematic drawing of the basic system is shown in Fig. 1-1. The signal generating system consists of an in-hole anchor and hammer assembly which imparts an impulse shearing energy to the soil. The shape of the generated pulse is controlled by a spring located between the anchor system and the hammer, and the magnitude by the mass of the hammer and the height of drop.

1.2 SCOPE

From previous studies (SW-AA, 1973), the mechanics of this impulse test were first investigated to establish the feasibility of the approach. Equipment was developed and preliminary tests conducted in shallow borings. The results of this study, supported by analytical calculations, indicated that a clear and repeatable shear stress pulse could be generated and traced horizontally as it passed recording stations located in adjacent borings at predetermined distances. Even though very clear records were obtained from this study, the recording equipment did not have adequate time resolution to enable an appropriate evaluation to be made to establish an exact procedure for identifying the arrival time at each recording station. Following these previously reported efforts, the next steps were to: 1) obtain improved recording and prototype equipment, 2) perform special tests in shallow borings to check the workability of the new equipment and study important test variables and measuring procedures, and then 3) perform actual tests at depth at three sites each with different soil conditions. This report presents the results of these efforts.

The report is organized into six main sections with appendices. Following this introductory section describing the purpose, objectives, and scope of the work accomplished,

FIGURE 1-1 SCHEMATIC OF IN SITU TEST

is Section II discussing the basic procedures and fundamental principles of the test. The remainder of Section II includes a description of the test equipment and discussion and illustration of field measuring and data reduction procedures.

The results of special field studies conducted in shallow borings to investigate test variables and measuring procedures are included in Section III. Test variables include:

- 1) Repeatability
- 2) Prior load history effects
- 3) Jacking pressure effects
- 4) Effects of variations in hammer weight and height of drop
- 5) Asymmetrics and shielding effects

Field velocity and modulus test results obtained from three sites are treated in Section IV, which also describes the soils tested and compares the results obtained from the new test with moduli values obtained by conventional field and laboratory procedures.

Section V is devoted to analytical studies of the system. This work includes finite element calculations used to assist in the evaluation of various procedures considered in determining the best method for identifying the arrival time of the shear pulse at each recording station. The limitations of each procedure are also discussed. The last part of this section describes the results of efforts directed toward developing a closed form solution of an idealized mathematical model of the test set up.

The final section, Section VI, discusses general conclusions derived from this research effort. Also included

is our evaluation of where modifications and refinements in the test must be made in planned future work on this project to obtain more refined results and a more simplified test.

Four appendices accompany the main text. Appendix A presents the results of laboratory tests performed on specimens of soil from each of the three test sites. Emphasis in this laboratory work was directed toward determination of values of moduli on samples from different depth intervals. Also basic classification and mechanical testing was accomplished.

Appendix B provides for the mathematical development for one method considered in efforts aimed at defining the best procedure for identifying the time of arrival of the wave as it passed the various recording stations. The derivation of the basic equations for this method are included as Appendix C.

The final appendix (D) includes the analytical representation of the closed form solution technique described in Section V.

II. IN SITU SHEAR WAVE TEST

2.1 FUNDAMENTAL CONSIDERATIONS

The in situ test may be idealized schematically as an axisymmetric case of shear deformation propagating radially from a vertical axis in an extended medium, as shown in Fig. 2-1. If the medium were elastic the wave equation would be:

$$\frac{\partial^2 (r\alpha)}{\partial t^2} = V_s^2 \frac{\partial^2 (r\alpha)}{\partial r^2} \quad (2-1)$$

where:

α is the shear deformation

V_s is the shear wave velocity of the medium

r is the radial distance, and

t is the time

This equation shows that there is a radial dissipation of the deformation as the wave travels outward from the origin.

The general solution of this equation is:

$$\alpha r = f(r - V_s t) + f_1(r + V_s t) \quad (2-2)$$

This ideal representation enables the shear modulus, G , of a material and the strain level, γ , to be determined at a certain distance from the energy source, if the particle velocity and the shear wave velocity are determined at each point. These parameters are related as follows:

$$G = \rho V_s^2 \quad (2-3)$$

FIGURE 2-1 SCHEMATIC OF SHEAR WAVE PROPAGATION

$$\gamma = \frac{\tau}{G} \approx \frac{V_p}{V_s} \quad (2-4)$$

where V_p and τ are the particle velocity and the shear stress, respectively, at each point under consideration. Equation 2-4 is valid only at a significant distance from the energy source.

Because soils are inelastic materials with hysteretic stress-strain relationships that are strain dependent, the above equations provided by the theories of elasticity are to be considered as approximations to guide the interpretation of the results of the tests.

2.2 DESCRIPTION OF THE TEST

This field test comprises the generation of a clear, repeatable shear wave impulse motion in one boring and the registration of the resulting motion in adjacent borings at different points in the soil mass away from the source motion. This shear wave motion is imparted to the soil by striking with a down hole hammer, an anchor assembly firmly coupled with the walls of a borehole. This impact generates shear stresses at the anchor/soil interface, which subsequently are propagated radially through the soil mass. The shape of the impulse motion is controlled by a spring positioned between the hammer and the anchor assembly. A schematic drawing and a simplified mechanical model of this anchor and hammer assembly and adjacent sensors are shown in Fig. 2-2.

The motion generated is recorded by vertical velocity sensors located in three adjacent borings. These borings are positioned on a radial pattern at different horizontal distances from the central boring containing the anchor and hammer assembly (Fig. 1-1). The recording sensors placed in these

FIGURE 2-2 SCHEMATIC AND SIMPLIFIED MODEL OF ANCHOR AND HAMMER SYSTEM AND ADJACENT SENSORS

borings are lowered to the same depth at a point where a horizontal plane passes through the center of the anchor. The motion of the anchor is also recorded with the same type of velocity sensor. For each pulse generated, the time history of the velocity measured by the four sensors are recorded simultaneously and stored in the memory unit of a portable high speed waveform recorder. This data acquisition unit is capable of producing the output data in either analog or digital form. In analog form, the data is first displayed on the screen of an oscilloscope for a quick visual study and assessment in the field. If the visual data is assessed to be acceptable, it is taken from the recorder as a permanent record in the field by means of a high speed printer which provides a paper print out of the information in digital form.

From precise measurements of travel times and distances from the anchor to each sensor as well as particle velocity amplitudes at each sensor, accurate determinations of the shear wave velocity and corresponding strain levels are possible. The precise travel times and particle velocity amplitudes are determined from the recorded time histories of velocity, while distances are determined from precise surveys of each borehole. These surveys are accomplished by measuring at the ground surface the precise distances between the holes. These holes are then surveyed with a conventional inclinometer and the ground surface distances adjusted at each test depth to account for the inclination of each hole.

The following sections present a brief description of test equipment and field measurement procedures, as well as more detailed procedures for the processing and interpretation of the recorded data.

2.3 TESTING EQUIPMENT

The major development efforts of this prototype equipment were accomplished in previous work, and the results presented in a previous progress report, SW-AA, 1973. However, this prototype equipment was improved and modified slightly, with emphasis on adapting in-hole equipment for testing at depth, and obtaining more flexible recording equipment capable of achieving both greater resolution and a more convenient means for processing bulk data.

Since the details of this equipment have been described previously, only the main features are summarized below:

2.3.1 Anchor and Hammer Assembly

A four foot long 195 pound anchor, formed by three aluminum curved plate segments, is used to transmit the applied impact energy to the soil. The details of this anchor are shown in Fig. 2-3. The vertically oriented piston shown in this photograph is a 25-ton capacity double acting hydraulic ram. This ram, when expanded, extends through a system of arms, forcing the curved plates outward and pressing them tightly against the walls of the borehole. A hand operated pump and gage is located at the ground surface to control and maintain the hydraulic pressure in this ram.

On top of the anchor plate segments, a striking plate and two Belleville springs in series form the cushion. This cushion receives and transmits the impact of the hammer to the anchor assembly. Standard NW drill rod extends through the center of the striking plate and is fastened to the anchor by a yoke. This rod maintains alignment of the striking plate in the center of the anchor and also is used to raise and lower

FIGURE 2-3 ANCHOR AND HAMMER ASSEMBLY

the entire assembly in the borehole.

During testing, the diameter of the expanded anchor can be determined at the ground surface by measuring, with a special key system on the bottom end of the NW rod, the stroke of the piston. If the hydraulic pressure in the ram and the diameter of the anchor are determined, by appropriate calibration, the radial stress at the soil/anchor interface also can be determined.

The last component in the system is the striking mass itself, referred to as the hammer. These hammers consisted of different size steel cylindrical weights, tapered slightly on the lower end. These weights are controlled from the ground surface by a thin steel hoisting cable. The hammers each have a 2-inch diameter hole through their center to be guided and centered in the hole by the NW drill rod. For this current testing, only 58 and 150 pound hammers were used. With these weights, the wide variation in energies was achieved by varying the height of drop.

The entire heavy anchor and hammer assembly is lowered in the borehole by the drill rig used to advance the borings. Fig. 2-4 is a photograph showing the test equipment prepared for lowering in the borehole.

2.3.2 Sensors

The sensor units for this test consisted of vertically oriented (Mark Products L-15 B) velocity transducers. These particular sensors have a natural frequency of 4.5 cycles per second. For these tests, each velocity transducer was placed in water tight casings and fixed to a sensor holder. These holders consisted of a strong, 1 1/2-inch diameter

FIGURE 2-4 ANCHOR AND HAMMER ASSEMBLY PREPARED FOR
INSTALLATION IN A BOREHOLE

inflatable, rubber, cylindrical balloon with two short aluminum heads fastened in each end. To make the holder act as a rigid element vertically, a thin solid steel rod extended through the rubber unit and was fixed to the two aluminum heads. The balloons were pressurized by compressed air and could be expanded to about 6 inches in diameter, if required, to achieve adequate coupling of the sensors with the walls of the boreholes. Orientation of the sensors was maintained at the ground surface through interconnecting 1/2-inch diameter, 10-foot long steel rods fixed to the aluminum head of the balloons. Details of the sensor and sensor holder are shown in Fig. 2-5. The steel rod extending from a sensor hole is shown in Fig. 2-4.

2.3.3 Recording Equipment

The four components of equipment used in the field to record the velocity time history from each sensor are shown in Fig. 2-6. The waveform recorder (Biomation Model 1015 Waveform Recorder) was the primary central data acquisition unit. This unit is capable of recording simultaneously and storing in memory, the velocity time histories independently from four sources. This data could be withdrawn from the recorder in either analog or digital form.

In analog form, the entire signal from each of the four channels was displayed simultaneously on the screen of the oscilloscope (Fig. 2-6). Because the information displayed on the oscilloscope is being taken from a memory bank, adjustments in time and amplitude of the signals are possible after the event. This feature enables small or critical data parts on any channel to be studied separately in detail or compared with critical parts on other channels. The oscilloscope was equipped with a camera mounted in front of the screen. Photographs, taken of the continuous display, provided not only a

FIGURE 2-5 SENSOR AND SENSOR HOLDER

FIGURE 2-6 RECORDING EQUIPMENT

back up recording system but also valuable additional graphical records used for determining visually which tests were the best and should undergo further study and processing.

In digital form, the data was withdrawn at 20 lines per second from the high speed digital printer (Fig. 2-6) on standard adding machine paper. Each of the four channels was printed in sequence as 1024 words or 4096 words in total. Each word was printed with a time and amplitude resolution of 0.05 milliseconds and one part in 1024, respectively. The recorder is capable of achieving a minimum and maximum time resolution ranging between 0.01 milliseconds and 10 seconds per word. The interface unit provided the necessary electrical circuitry for transferring the digital data from the waveform recorder to the printer.

A pretrigger and delay mechanism within the waveform recorder permitted precise control of the total record interval. This unique feature enabled the full test event to be recorded to maximum time resolution without losing any significant part of the test data.

2.4 FIELD EXPLORATIONS AND TESTING

The test program consisted of performing special tests in shallow borings to first establish the workability of the equipment and to evaluate certain test variables. This work was followed by a more extensive test program of performing tests at depth at three different sites. This latter testing was conducted to systematically obtain "real" values of moduli in different types of soil.

Since the special tests were conducted with no well established field exploration and testing procedures and were intended only to simulate "real" testing procedures, detailed

discussions of field procedures for the special tests is not warranted. Therefore, the following paragraphs describe in sequence only the field exploration and test procedures conducted at each of the three "real" test sites.

2.4.1 Field Exploration

At each site four borings were drilled and tests performed to depths on the order of 100 feet. Three of the borings, spaced horizontally as shown in Fig. 1-1, were advanced at each site using rotary drilling methods. Each hole was started by setting 5 feet of 5-inch diameter pipe as surface casing. The remainder of each hole was advanced, without sampling, using a 3 7/8-inch diameter tri-cone bit with drilling fluid to remove cuttings. Either bentonite, revert and/or water were used as the drilling fluid.

In advancing these holes, extreme care must be taken to assure reasonable verticality of the holes especially the two nearest the anchor. Excessive drift in the alignment can result in two holes converging to a common distance from the anchor. This, in effect, causes two potential data acquisition stations to act as one.

The fourth boring was advanced at each site using a 3 3/8-inch I.D. hollow stem continuous flight power auger. This auger produces a hole about 9 inches in diameter. For an adequate description of the physical and engineering properties of the subsurface materials at each site, sampling was accomplished generally at a depth interval of about 5 feet. In cases where the material types appeared to be changing significantly, the depth interval was decreased.

Disturbed sampling of the soil for identification

purposes consisted of advancing a 2-inch O.D. split-spoon sampler 18 inches into the undisturbed soil at the bottom of the boring, with blows of a 140-lb. hammer dropping 30 inches. The number of blows required to produce the final 12 inches of an 18-inch penetration of the sampler, defined as the Standard Penetration Resistance, was recorded for each sample. When very dense or hard soils were encountered, the sampler often could not be driven the full 18 inches, or in some cases even 12 inches. Where a full 18 inches penetration was not achieved, blows per 6-inch interval were recorded. These disturbed samples were classified in the field and then placed in air-tight jars and shipped to our laboratory for detailed examination, classification and testing as necessary.

Undisturbed sampling of clay and sands consisted of utilizing the hydraulic ram on the drill rig to push 3-inch diameter, thin wall, steel Shelby tubes into the soil at the bottom of the hole as drilling progressed. These samples were sealed at the ends with a micro-crystalline wax and delivered to our laboratory for extrusion, detailed classification and testing.

2.4.2 Field Testing

As each boring was completed, 2 3/4-inch O.D. plastic casing was lowered into the hole through the drilling fluid for the full depth and rigidly fixed at the surface. The hole was then surveyed by lowering an inclinometer inside the casing and recording the precise slope of the casing at 2-foot intervals. Vertically aligned slots in the casing, to accommodate the spring-loaded wheels on the instrument, enabled the instrument to be raised and lowered in the casing without rotating. Each borehole was surveyed

on perpendicular axes. The plastic casing was then removed from the hole shortly after the survey was completed.

For the auger boring, the same casing was installed in the 3 3/8-inch hollow stem of the continuous flight auger. The hole was then surveyed and the casing removed prior to pulling the auger. As the auger was removed, drilling fluid was pumped through the hollow stem and out the bottom displacing the auger. This minimized the possibility of caving of the resulting large diameter hole. By having precise alignment surveys of each boring and knowing the distance between successive holes at the ground surface, the distance between the anchor hole and the three sensor holes at any test depth can be computed.

Once all four borings were completed, in situ tests were conducted, usually at 5-foot depth intervals. Testing usually was started at the bottom of the boreholes and progressed upward with 3 to 5 records being taken at each depth. The heavy anchor and hammer assembly was raised and lowered on NW drill rod to the prescribed test depth by the drill rig (Fig. 2-7). The sensors and holder were light weight and could be raised and lowered manually with inter-connecting 1/2-inch diameter, 10-foot long, thin steel rods.

The coupling of the anchor was accomplished by gradually increasing the hydraulic pressure in the 25-ton ram until a stable value was reached. To monitor the diameter of the anchor and the resulting radial stress being applied to the soil, measurements of the vertical stroke of the ram were checked periodically during the jacking process and usually after each test.

Coupling of each sensor was achieved by inflating

FIGURE 2-7 DRILLING RIG PREPARED TO LOWER ANCHOR FOR IN SITU TESTING

the rubber balloon sensor holders with compressed nitrogen. Firm coupling was verified by pulling or pushing manually on the interconnecting rods. Also to assure continued coupling during testing, the rods were left free standing above the ground surface. Thus their weight was supported entirely by the sensor holder providing a simple monitor of the coupling effectiveness. If the coupling became loose, the rods would drop several inches and catch a horizontal clamp on the top of the surface casing (see Fig. 2-4).

At a given test depth, records usually for 3 to 5 impacts were taken. Different amplitude signals were obtained by varying the height of drop of the hammer. Usually the drop was varied from 2 feet to less than one inch.

For each test, a velocity time history was obtained first from all four sensors and displayed on the screen of the oscilloscope. This display was then inspected to determine if all equipment was operating properly (i.e. proper coupling, clear impact and proper scales on recording equipment). The impact was then repeated and other records obtained the same way until four clear properly scaled time histories were obtained from one impact. Two or three tentative trials were usually performed before a good test was obtained. This last test was made a permanent record by taking photographs of the display on the oscilloscope and then printing out in digital form each time history of velocity on paper tape.

In addition to obtaining these permanent records, a permanent data sheet containing a number of scale factors and other information about the test was maintained. A typical sheet with this information is included as Fig. 2-8.

Upon completion of the in situ testing, conventional downhole geophysical tests were performed as the last part of

INSITU IMPULSE TEST – FIELD DATA SHEET

SITE _____
 TEST NO. _____
 DATE _____
 DEPTH _____ FT.
 ENERGY DATA
 WEIGHT _____ LBS.
 DROP _____ FT.
 SPRING _____
 ANCHOR DATA
 RAM PRESSURE _____ PSI. RADIAL STRESS _____ PSI.
 RAM STROKE _____ IN. ANCHOR DIA. _____ IN.
 SENSOR DATA
 AIR PRESSURE _____ PSI.
 WAVEFORM RECORDER DATA

CHANNEL	HOLE NO.	SCALE, VOLTS	ZERO READING	TRANSDUCER NO.	CONVERSION FACTOR
1	_____	_____	_____	_____	_____
2	_____	_____	_____	_____	_____
3	_____	_____	_____	_____	_____
4	_____	_____	_____	_____	_____

 PRETRIGGER SETTING _____
 TRIGGER CHANNEL _____
 SAMPLE RATE _____ MSEC/SAMPLE
 OSCILLOSCOPE DATA
 VERTICAL SCALE _____ V/CM. _____ V/CM.
 TIME SCALE _____ MSEC/CM. _____ MSEC/CM.
 PHOTOGRAPH NO. _____
 REMARKS:

FIGURE 2-8 IN SITU IMPULSE TEST FIELD DATA SHEET

the field effort. For this test two, horizontal axis, velocity sensors with holders were fixed on the ends of a 5-foot long pipe and together lowered on the end of the 10-foot rods used for the in situ tests. A schematic of the downhole test is shown in Fig. 2-9. The input energy was applied near the top of the borehole by striking with a 12-lb. sledge hammer a large steel post attached to two steel plates buried just below ground surface. The striking plates weigh several hundred pounds and have a surface area of about 2 square feet. Coupling between the soil media and the plates was achieved by compacting granular soil tightly around the plates. Rubber pads were sometimes used as a cushion between the hammer and the striking post.

The test was performed by first striking the plate in one direction (Fig. 2-9) and recording the resulting complex pattern of waves. The direction of the impact was then reversed and another measurement obtained. By reversing the direction of the energy blow, the shear wave pattern is recorded in the reverse direction, while the compressional wave pattern is essentially unchanged. This allows a distinction to be made between the compressional and shear wave patterns. The time of arrival of the shear wave at each of the two sensors is thus established by picking the most definitive common points corresponding usually to first arrivals, peaks or crossings.

2.4.3 Laboratory Testing

Laboratory tests were performed on selected soil samples from the anchor boring at each of the three sites. The laboratory testing program was formulated with special emphasis on the determination of the detailed physical properties of the materials and their shear modulus characteristics under cyclic loading conditions. The strength and consolidation characteristics of the cohesive materials were also determined. Results

FIGURE 2-9 SCHEMATIC OF DOWNHOLE GEOPHYSICAL TEST

of classification testing including Atterberg limit tests and moisture determinations are summarized on the logs of each boring presented in subsequent sections. Results of gradation tests and mechanical tests to establish the soils' engineering properties are presented in Appendix A, together with a brief description of each test.

2.5 DATA PROCESSING

Once the field data are obtained, usually at five-foot depth intervals, they must then be reduced in order to determine the in situ shear modulus variation with strain. The pertinent parameters that are obtained from the data are:

- 1) the time of peak strain at each sensor location (two peaks),
- 2) the time of peak stress in the soil at the soil anchor interface (two peaks),
- 3) the time of first arrival of the shear wave at the two distant sensor locations,
- 4) the maximum particle velocity amplitude at each sensor (two peaks),
- 5) the precise horizontal distance between the anchor and adjacent sensors.

The first four measurements are obtained from the velocity time histories, and the final parameter is obtained by the surveying techniques described previously.

The times of arrival of the peak strain for each sensor is plotted first graphically with distance as shown in Fig. 2-10. A smooth curve is then drawn connecting the four data points and passing beyond the distant sensor at a slope equal to or less than the low strain shear velocity. This low strain velocity is determined by dividing the difference of the distances between

FIGURE 2-10 TIME OF ARRIVAL AT PEAK STRESS VS DISTANCE FROM ANCHOR

the anchor and the two distant sensors by the time of the first arrival of the shear wave.

The curve construction process continues by drawing straight lines tangent to the slope of the curve at each sensor location. The slope of this line corresponds to the shear velocity at that point. This shear velocity combined with the corresponding peak particle velocity is inserted in Equations 2-3 and 2-4 to determine four values of shear modulus at four corresponding strain levels. As discussed subsequently, semi-empirical modifications from this simple relationship to determine the shear strains at the anchor are required.

The resulting four values are plotted on semi logarithmic paper and a curve constructed to show the resultant variation in shear modulus or shear velocity with strain for that test depth, (Fig. 2-11). This process is repeated for all test depths. The data can then be summarized and generalized next to the soil log as a family of curves of modulus or shear velocity change with depth for strains which are each an order of magnitude apart. An idealized example of typical results is shown in Fig. 2-12. The results for the three test sites presented in Section 4 were processed and presented in this manner.

2.6 DATA INTERPRETATION

While the above construction procedures are straightforward (even though perhaps oversimplified), the entire process of wave propagation and data interpretation is considerably more complex when one attempts to define rationally from each time history of velocity, the precise times of peak stress or strain at the anchor and each adjacent (or free-field) sensor station. This condition exists because of the complex behavior of soils. For example, the soils at the anchor exhibit large inelastic hysteretic characteristics, while the soils at more distant sensors

FIGURE 2-11 EXAMPLE PLOT OF MODULUS AND VELOCITY VS STRAIN

FIGURE 2-12 SCHEMATIC OF TYPICAL IN SITU TEST RESULTS

behave as a near linear system. Theoretical and empirical approaches, ranging from considerations of the entire waveform signature to consideration of common single points, have been investigated to define the most rational method for solution of the entire problem. These various methods and their general limitations are discussed in Section 5. Basic to the problem of data interpretation is the fact that the generation of a shear wave by means of a force applied to an anchor with a mass means that the time history recorded at the anchor must be treated differently from the records at the other sensors. The remaining three stations are treated together as a near linear elastic problem taking into account the non-linear soil behavior within the time history. For this case, the fundamental linear equations provide good approximations of the shear velocity and strains at these locations. The following paragraphs discuss the two procedures used to establish the times of arrival for this test.

2.6.1 Free-Field Recording Stations

As described previously, a sensor is located in each of three borings adjacent to the anchor (Fig. 1-1). Each is of low mass and is firmly coupled directly against the soil wall at a point closest to the anchor and, therefore, can be reasonably assumed to respond as the surrounding free field soil. This is especially true for the two distant sensors. The closest sensor is likewise believed to be sufficiently far away from the anchor (4 feet) so that the free field motion is not likely to be significantly influenced by the forcing function. This conclusion is based on the fact that the distance between the anchor and first recording sensor (4 feet) is generally greater than one-half the wave length of the input motion.

Typical velocity signatures obtained at each sensor

location are shown in Fig. 2-13. From this record, the primary signal of interest consists of a large single displacement pulse having two well defined velocity peaks and three well defined velocity crossings. Since the particle velocity is approximately proportioned to the shear strain (Equation 2-4) for a linear elastic system, the particle velocity time history for these sensors can be thought of as representing the general shape of a shear strain time history, the peak values corresponding to peak strain. Thus, for an assumed linear elastic system, the stress and strain, as shown in the idealized load curve in Fig. 2-14a would be completely in phase at all points in time during loading and unloading. Therefore, any common peak (1 and 3) or crossing point (2 and 4) should produce a time interval between successive stations corresponding to the shear modulus. The real idealized loading curve for soils, however, has a hysteretic shape as shown in Fig. 2-14b. From this curve, it can be seen that in most cases during cyclic loading, the stress value (for example, Point 2') at zero amplitude leads the strain value (Point 2'') in time by some finite value (i.e., zero stress is reached on the loading curve at some point before zero strain). Thus, at points of zero strain (i.e., zero crossings on the velocity time history) the stress is out of phase and, therefore, does not provide an identifiable time point for computing shear velocities, except possibly for much smaller strains. Although the stress time history at the sensors has not been measured, the hypothetical shape based on the hysteretic relationship is probably as shown in the idealized case, Fig. 2-14c. From Fig. 2-14b or c, it can also be seen that at the points of peak stress and strain (Points 1 and 3), both are in phase and, therefore, either peak point provides a good indication of the arrival time of the shear wave at peak stress or strain. These points correspond to the two peak particle velocities assuming that the anchor and hammer assembly is compatible and produces relatively equal positive and negative shear forces. For our tests

FIGURE 2-13 TYPICAL VELOCITY TIME SIGNATURES DISPLAYED
ON THE SCREEN OF THE OSCILLOSCOPE

a) IDEALIZED CYCLIC LOAD CURVE FOR LINEAR ELASTIC MATERIAL

b) IDEALIZED CYCLIC LOAD CURVE FOR NON LINEAR HYSTERETIC MATERIAL

c) IDEALIZED TIME HISTORY OF LOADING FOR NON LINEAR HYSTERETIC MATERIAL

FIGURE 2-14 IDEALIZED STRESS STRAIN RELATIONSHIPS

reported in Section 4, the second peak velocity amplitude was used to compute the shear modulus. This point is easily identified and can be readily obtained directly from the digital tapes. The reason for using this point will be explained in the next paragraphs.

2.6.2 Anchor Recording Station

The velocity sensor at the anchor is located on the inside of one of the three aluminum plates (Fig. 2-3). The velocity recorded at this station, therefore, represents the velocity of the anchor and, if there is no anchor slippage, also represents the velocity of the soil at the soil/anchor interface. However, the anchor and thus this sensor is not moving as a free field motion of the soil but as a part of a body acting to generate a shear wave within the soil (i.e., as a forcing function on the soil). Because the soil velocity close to the anchor is being generated and is not free field, Equation 2-4 which relates shear strain with particle velocity is not valid. Therefore, at the anchor it is not possible to determine a time of peak strain and instead the time of peak stress must be identified. Since peak stress and peak strain occur at the same time at the more distant sensors, a time corresponding to peak stress at the anchor provides sufficient time data to combine the anchor measurements with the other data.

One approach to this problem is to measure the load-time history of the applied load. Then the anchor can be isolated and treated as a rigid/free body, as shown in the diagram in Fig. 2-15. For an equilibrium condition, the following equation is derived:

$$F_s(t) = F(t) - F_d(t) \quad (2-5)$$

WHERE

$F(t)$ = STRIKING FORCE WITH TIME

$F_d(t)$ = DYNAMIC FORCE WITH TIME

= MASS OF ANCHOR X ACCELERATION

$F_s(t)$ = RESULTANT SHEAR FORCE ON SOIL WITH TIME

$$\Sigma \text{ FORCES} = 0$$

$$F(t) - F_d(t) = F_s(t)$$

FIGURE 2-15 FREE-BODY SCHEMATIC OF ANCHOR MASS

where:

$F_s(t)$ is the shear force in the
soil at the anchor soil interface
 $F(t)$ is the force applied to the anchor
 $F_d(t)$ is the dynamic inertia force of the anchor

As a part of the special test program described in Section 3, $F(t)$ was measured simultaneously with the velocity of the anchor by placing a load cell on top of the anchor. Unfortunately, this load cell system was designed primarily for a low frequency steady state system and thus the resulting load data were in error, both as to magnitude and to time. A simulated experimental test was set up in the laboratory in an attempt to establish approximately what effect a higher frequency pulse would have on the output of the load cell. In general, the output of the pulse was reduced by a factor of 2 to 3, but the time history could not be approximated.

The striking force, which is not now known because it was not properly measured, is resisted partly by the dynamic response of the anchor and partly by the shear stress between the anchor and the soil. Although it has not yet been verified experimentally, we believe that for any specific test the shear stress is proportional to the dynamic response of the anchor. In other words, the shear stress at the anchor is proportional to the acceleration of the anchor and thus is 90 degrees out of phase with the shear strain at the free field sensor where shear strain is proportional to velocity. Therefore, the time at which the anchor acceleration is a maximum corresponds to the time of maximum shear stress. Figs. 2-16a and 2-16b show the velocity and acceleration time histories of the anchor. The Points 1 and 3 shown in Fig. 2-16b correspond with Points 1 and 3 of the velocity sensors, Fig. 2-14c. However, the shearing force cannot be computed

FIGURE 2-16 - MOTION OF THE ANCHOR

for the loading pulse since the ratio of shear force to dynamic response is not known.

After the striker has rebounded from the anchor, the shear force must equal the dynamic response of the anchor.

The dynamic inertia force exerted on the soil, $F_d(t)$, can be computed by knowing the weight of the anchor, (195 lbs.), and the acceleration of the anchor, (the slope of the velocity time history recorded at the anchor).

For lack of more refined measurements of $F(t)$, the time at the second peak point of stress rather than the first was used in the data reduction process. This peak point is, we believe, controlled entirely by the maximum acceleration, which usually occurs from zero to 0.6 milliseconds before the second point of zero velocity is reached on the velocity time history. For data reduction processing, this point was obtained empirically by taking the second point of zero velocity less 0.3 milliseconds. This average approximating method for obtaining the time for this peak, in our opinion, is justified at this time until more exact measurements of $F(t)$ are obtained and the exact influence of this forcing function on the second peak is evaluated in future work. Also in future data evaluation, the actual time of maximum acceleration will be determined. The shear strain, γ , at the anchor can then be approximated by the following equations:

$$F_{d_{\max}} = m a_{\max} \quad (2-6)$$

$$\tau = \frac{F_{d_{\max}}}{A} \quad (2-7)$$

$$\gamma = \frac{\tau}{G} \quad (2-8)$$

where:

m = mass of anchor

a_{max} = maximum acceleration of anchor

A = contact area of the three plates on the anchor (6 ft²)

G = shear modulus at the anchor

From the reduction of data for a number of tests, it was found that this strain value, in general, corresponds to approximately 1/3 of the maximum shear strain computed from Equation 2-4. Since this above method is only an approximate estimate of the shear strain, the more simplified criteria of using the empirically determined 1/3 value was applied in the reduction and presentation of test data. Since modulus values are determined in this test over a very wide range of strains, sensitivity to variations in strain of the order of 2 is not great. In future work efforts, a better, more refined method of defining peak strain will be studied, including the possibility of adding a load cell to the anchor so that F_{d_{max}} can be measured for all the tests.

III. EVALUATION OF SPECIAL TEST RESULTS

3.1 PURPOSE

When an experimental procedure is developed, it is necessary to evaluate the results for sensitivity to variations in the significant test parameters. In particular, for the in situ test method reported herein it was judged necessary to conduct a series of special tests to evaluate the following:

- 1) The degree of repeatability of the measurements
- 2) Prior soil load history effects
- 3) Jacking pressure effects
- 4) Effects of variations in hammer weight and height of drop for a given energy level
- 5) Asymmetries and shielding effects in the propagating wave pattern

These tests, conducted at a site containing hard clays (Site No. 1) were accomplished in shallow borings to 20 feet in depth. The layout of the various sensor holes relative to the anchor hole is shown in Fig. 3-1. These borings were advanced with the same equipment used later for testing at the three "real" sites. This equipment is described in Section 2.4.1.

The purpose of this section is to provide and evaluate the results from each of the above special tests. The results are based primarily on statistical comparisons of the delay time of the waves propagating through the soil and the peak particle velocity, since these are the two parameters used directly to determine the in situ soil properties. In addition, the characteristics of the entire measured velocity signal are compared and evaluated in terms of the stated test parameters.

FIGURE 3-1. HORIZONTAL CROSS SECTION OF SPECIAL TEST SETUP (PLAN VIEW)

3.2 REPEATABILITY AND PRIOR LOAD HISTORY EFFECTS

The purpose of this sequence of tests was first to evaluate the degree of experimental (i.e., uncontrolled) variability that is inherent in the in situ test method. A second purpose was to determine the degree to which the initial dynamic loading of soil materials may affect their subsequent measured shear moduli and strains.

To make these specific evaluations 45 tests were carried out in three groups of 15 tests each. The first group (Cases 33 through 47) corresponded to a 58-lb. hammer dropped 2 ft. onto the anchor. The next group (Cases 48 through 62) corresponded to a higher input energy level, in which a 150-lb. hammer was dropped 2 ft. The final group (Cases 63 through 77) represented conditions identical to those used for the first group, with a 58-lb. hammer and a 2 ft. drop. In these tests, velocity time measurements at the anchor and at Holes 1, 2 and 3 were obtained (Fig. 3-1).

To provide a meaningful basis for evaluating repeatability and prior load history effects, statistical parameters were computed for the various groups of test results indicated above. These parameters, obtained for both the delay times between sensors and the peak particle velocity at a given sensor, are:

- a. The mean value
- b. The variability, computed as the ratio of the standard deviation to the mean
- c. The 90 percent confidence range of the mean value
- d. The range within which 0.9 of the total population of these values will fall, with a 90 percent confidence

Procedures for computing these parameters are provided in

textbooks on statistics (Cramer, 1954 and Crow, et al).

Statistical properties of the delay times measured between the various sensors are provided in Figs. 3-2 to 3-4 for the initial low-energy loading, the high-energy loading, and the final low-energy loading, respectively. These figures provide delay times determined from the following procedures.

a. One procedure consists of the use of a cross-correlation function, $R(\tau)$, computed as

$$R(\tau) = \frac{1}{T} \int_0^T \dot{y}_1(t) \dot{y}_2(t + \tau) dt \quad (3-1)$$

in which $\dot{y}_1$ and $\dot{y}_2$ are velocity signals recorded at two sensors in the field, T is the duration of each signal, and τ is a time shift. In this $\dot{y}_2(t)$ is shifted by an amount τ relative to $\dot{y}_1(t)$ and the product is computed, integrated and averaged. This process is repeated for different values of τ . When $\dot{y}_2(t + \tau)$ "best matches" $\dot{y}_1(t)$, then $R(\tau)$ is a maximum. Visual inspection of a plot of $R(\tau)$ versus τ therefore serves to determine the value of τ for which $R(\tau)$ is a maximum; this value corresponds to one estimate of the delay time of the wave as it travels between the two sensors.

b. A simpler approach for estimating the delay time consists of identifying a single point on the velocity signal measured at two sensors. The difference in the time of occurrence of the point at the two sensors corresponds to the delay time. Two easily identifiable points used in this approach are the time of the first (positive) peak of the velocity signal (herein termed "first peak"), and the first point beyond this peak at which the velocity changes sign (herein termed "zero crossing").

(a) ANCHOR TO HOLE 1 (DISTANCE = 2.10 FT)

(b) HOLE 1 TO HOLE 2 (DISTANCE = 2.73 FT)

(c) HOLE 2 TO HOLE 3 (DISTANCE = 4.49 FT)

FIGURE 3-2. TIME DELAY DATA - INITIAL LOW ENERGY LOADING (CASES 33-47)

FIGURE 3-3. TIME DELAY DATA - HIGH-ENERGY LOADING (CASES 48-62)

(a) ANCHOR TO HOLE 1 (DISTANCE = 2.10 FT)

(b) HOLE 1 TO HOLE 2 (DISTANCE = 2.73 FT)

(c) HOLE 2 TO HOLE 3 (DISTANCE = 4.49 FT)

FIGURE 3-4. TIME DELAY DATA--FINAL LOW ENERGY LOADING (CASES 63-77)

The results shown in Figs. 3-2 to 3-4 show a number of interesting trends. First, rather consistent mean values of the time delay between any two sensors are obtained, regardless of whether these delays are obtained from the cross-correlation function, first peak, or zero crossing. Second, the scatter in the results, as indicated by the variability computation as well as by the various 90 percent confidence bounds, is quite small. Variabilities in which the standard deviation is less than 7 percent of the mean are nearly always obtained. The greatest variations in delay time appear to occur between the Anchor and Hole 1. No consistent trends are observed regarding the degree of variation in delay time between the various groups of tests.

The effects of the prior load history and the energy level on the mean values of the delay time are indicated in Fig. 3-5. An evaluation of prior load history effects on the delay time can be obtained by comparing the results from the initial low-energy loading (Cases 33 through 47) and the final low-energy loading (Cases 63 through 77). This indicates the degree to which prior high-energy loads might modify soil properties obtained from the low-energy input load tests. These comparisons show very little difference between mean values of the various delay times obtained from the two groups of low-energy tests. This trend holds regardless of the method used to determine the delay time.

Comparisons between mean delay times obtained from the high-energy and low-energy tests are also shown in Fig. 3-5. These comparisons indicate that between the Anchor and Hole 1, delay times from the high-energy tests are slightly lower than those from the low-energy tests, whereas between Holes 1 and 2, the opposite trend occurs. Between Holes 2 and 3, there is little difference between delay times from the high-energy

(a) BASED ON CROSS CORRELATION

(b) BASED ON ZERO CROSSING

(c) BASED ON FIRST PEAK

FIGURE 3-5. EFFECT OF ENERGY LEVEL ON MEAN VALUE OF DELAY TIME

and low-energy cases, suggesting that the soil in this region is in an elastic state regardless of which input energy level is used.

The lack of consistency of the delay times in the Anchor/Hole 1 and Hole 1/Hole 2 regions warrants further explanation. It is clear that an increase in the input energy level should result in larger strains, which correspond to a lower shear modulus; this in turn corresponds to a decreased shear wave velocity and an increased delay time. Therefore, the effects of energy level on the delay times in the Hole 1/Hole 2 region appear reasonable, whereas those trends in the Anchor/Hole 1 region seem open to question. The results in the latter region as discussed in Section 2 are affected by the fact that the soil particle velocity close to the anchor is being forced and is not free field. This difference alters the seismic waves generated in the immediate vicinity of the anchor and, therefore, comparison of the data as presented is not possible. Further study of this near-anchor soil response is planned in future work.

Repeatability and prior load history effects regarding the peak vertical velocity at the anchor and sensor holes are shown in Fig. 3-6. This figure indicates a greater degree of scatter in the peak velocities than previously indicated in the delay times. Variabilities (ratio of standard deviation to mean) are typically about 15 to 25 percent, and range from 8 percent to over 40 percent. This increased scatter is also indicated by observing greater differences between upper and lower 90 percent confidence bounds for the peak velocities than for the delay times.

Fig. 3-6 also indicates the effect of input energy level on the peak velocities. As expected, the higher-energy loading

(a) AT ANCHOR

(b) AT HOLE 1

(c) AT HOLE 2

(d) AT HOLE 3

FIGURE 3-6. REPEATABILITY AND PRIOR LOAD HISTORY DATA--PEAK VELOCITIES

results in larger velocities than the lower energy input loads. (This suggests larger strains based on the strain-versus-velocity relationship discussed in Section 2). Furthermore, it is noted that the initial low-energy tests (Cases 33 through 47) produce peak velocities at the anchor that are about 50 percent greater than those for the final low-energy tests (Cases 63 through 77). However, at Holes 1, 2 and 3 a reverse trend is observed, in which peak velocities from the final low-energy tests are greater. The differences in sensor-hole peak velocities between the initial and final low-energy tests are about 30 percent at Hole 1 and are quite small at Holes 2 and 3. These differences in trends regarding peak velocities at the anchor relative to the sensor holes appear to be related to the degree of coupling achieved at the soil anchor interface. As the local soil at the interface becomes disturbed due to repeated testing, it tends to compress and/or squeeze between the anchor plates, causing the diameter of the anchor to slowly increase. If the diameter of the anchor increases slightly with each test while the same hydraulic pressure is maintained in the ram, as in these tests, the radial stress on the soil is also slowly increased. This gradual increase in radial stress achieves better or a more rigid coupling between the soil and anchor and, therefore, for a given constant impact energy, results in smaller motions (particle velocities) of the anchor. Also as indicated from the measurements, better coupling results in a more efficient transmission of energy to the soil. Thus higher particle velocities are achieved at the adjacent sensors with increased testing.

The above indicated trends regarding repeatability and prior load history effects are further supported by plots of the measured velocity time histories shown in Figs. 3-7 through 3-10 for the anchor and three sensor holes respectively. The figures indicate that within a group of tests corresponding

(a) INITIAL LOW ENERGY TESTS

(b) FINAL LOW ENERGY TESTS

(c) HIGH ENERGY TESTS

FIGURE 3-7. REPEATABILITY AND PRIOR LOAD HISTORY EFFECTS ON VELOCITY TIME HISTORY MEASURED AT ANCHOR

(a) INITIAL LOW ENERGY TESTS

(b) FINAL LOW ENERGY TESTS

(c) HIGH ENERGY TESTS

FIGURE 3-8. REPEATABILITY AND PRIOR LOAD HISTORY EFFECTS ON VELOCITY TIME HISTORY MEASURED AT HOLE 1

(a) INITIAL LOW ENERGY TESTS

(b) FINAL LOW ENERGY TESTS

(c) HIGH ENERGY TESTS

FIGURE 3-9. REPEATABILITY AND PRIOR LOAD HISTORY EFFECTS ON VELOCITY TIME HISTORY MEASURED AT HOLE 2

(a) INITIAL LOW ENERGY TESTS

(b) FINAL LOW ENERGY TESTS

(c) HIGH ENERGY TESTS

FIGURE 3-10. REPEATABILITY AND PRIOR LOAD HISTORY EFFECTS ON VELOCITY TIME HISTORY MEASURED AT HOLE 3

to low-energy or high-energy input, the shapes of the signals are remarkably consistent. This, in turn, leads to the close correlations in delay times from the various groups of tests as discussed previously. However, the amplitudes of the peak velocities are seen to be quite variable, which results in the relatively large scatter shown by the statistical correlations in Fig. 3-6.

3.3 EFFECTS OF JACKING PRESSURE AND VARIATIONS IN HAMMER WEIGHT AND HEIGHT OF DROP

To effectively seat the anchor against the sides of the borehole, large radial jacking pressures must be applied. This subjects the soil surrounding the anchor to a static state of stress that could affect the dynamic properties measured by the in situ test. Therefore, to evaluate potential effects of these jacking pressures, velocity time measurements were obtained and compared at two different input energy levels, each corresponding to jack pressures of 2000, 4000, and 6000 psi. Results of these comparisons are presented in this section.

A second variable that requires consideration is the details of the manner in which the input energy levels are applied to the anchor. It is apparent that a given energy level can result from any number of combinations of hammer weight and dropped height. Furthermore, different hammer weights can dynamically interact differently with the anchor, even though the input energy levels corresponding to these different weights are identical. This, in turn, could conceivably affect the response of the anchor and the resulting dynamic characteristics of the waves transmitted to the soil. Therefore, for each input energy level, effects of variations in the hammer weight and height of drop on the measured velocity signals in the soil have been evaluated. These evaluations are based on tests at the three different jacking pressures indicated above,

at which a single energy level (116 ft.-lb.) is represented by two different combinations of hammer weights and drop heights (58 lb. x 2 ft. and 150 lb. x 0.77 ft.).

Effects of jacking pressure and the details of the input loading are shown in Fig. 3-11. This figure indicates that the greatest variation in delay time as a function of input loading occurs when the jacking pressure is 2000 psi, the lowest pressure used. This variation may be due, in part, to anchor slippage. For this low jacking pressure, the actual values of the delay times for a given loading at a given location appear to strongly depend on whether first peaks or zero crossings are used as the bases for the delay time computation. No consistent trends regarding the effects of input loadings on the delay time can be identified from these low jack pressure tests.

At higher jacking pressures, slightly more consistent trends are observed. First, delay times computed from the zero crossing at a 4000 psi jacking pressure appear generally similar to delay times computed from the zero crossing at a 6000 psi jacking pressure. Similarly, delay times computed from first peaks at 4000 and 6000 psi jacking pressures are similar. However, there are marked differences between delay times in the Anchor/Hole 1 regions as computed from zero crossings and first peaks at the same pressure levels (4000 or 6000 psi). These differences are reduced in the Hole 1/Hole 2 region and are quite small in the Hole 2/Hole 3 area. Potential reasons for these trends are discussed subsequently.

The effects of different hammer weights and drop heights for the same (116 ft.-lb.) energy level generally appear significant at the low (2000 psi) jacking pressure, but are relatively minor at the higher jacking pressures. The application of a higher energy level does not appear to result in consistent

FIGURE 3-11. EFFECT OF JACKING PRESSURE, DROPPED WEIGHT, AND HEIGHT OF DROP ON DELAY TIME

comparisons of the delay times with the low-energy results. This may be due, in part, to the fact that only a single test was used to represent each jacking pressure-input load condition combination in this series. Therefore, any trends regarding the input load effects may be masked by statistical scatter. It is recalled that, when numerous samples were evaluated statistically in the prior load history tests (Sec. 3.2 and Fig. 3-5), variations in the input energy level resulted only in minor effects on the mean delay time.

Effects of jacking pressure and input load conditions on the peak particle velocity are shown in Fig. 3-12. This figure indicates that the higher energy level (300 ft.-lb.) resulted in consistently higher peak velocities than did either of the 116 ft.-lb. input conditions. Furthermore, the differences between the high-energy and low-energy results are greatest when the low jacking pressure is applied. The effects of variations in the hammer weight and drop height (for the 116 ft.-lb. input energy level) on the peak particle velocity appear relatively small for the higher jack pressure (4000 and 6000 psi) conditions. At the low jacking pressure (2000 psi) the 58 lb. x 2 ft. input condition resulted in noticeably higher peak velocities at the anchor, whereas, at the three sensor holes, the 150 lb. x 0.77 ft. input provided the higher velocities.

For a given input condition, the variations in jacking pressures produced some interesting effects on the peak velocity, as shown in Fig. 3-12. At the anchor, the peak velocity consistently decreased with increasing jacking pressure, (i.e., better coupling was being achieved at the anchor). At the three sensor holes, an increase in peak velocity with increasing jacking pressure is observed (better coupling of the anchor with the soil results in a more efficient transmission of the imparted input force). These effects on peak

FIGURE 3-12. EFFECT OF JACKING PRESSURE, DROPPED WEIGHT, AND HEIGHT OF DROP ON PEAK VERTICAL VELOCITY

velocity are relatively small when the jacking pressure is increased from 4000 to 6000 psi and are more noticeable when the pressure is changed from 2000 to 4000 psi.

For the 58 lb. x 2 ft. input condition, Fig. 3-13 illustrates the effects of jacking pressure on the velocity time histories measured at the anchor and three sensor holes. From this, it is seen that at all sensors the velocity signal corresponding to the 2000 psi jacking pressure deviates substantially from those at 4000 and 6000 psi. At these higher jack pressure levels, the velocity signals are quite similar, at least through the first peak and zero crossing. Furthermore, close examination of the velocity time histories at the anchor (Fig. 3-13a) provides some insight into possible differences between first peak and zero crossing delay times in the Anchor/Hole 1 region at higher jack pressure levels, as discussed previously and indicated in Fig. 3-11. Fig. 3-13a indicates that the first peaks of the velocity signals corresponding to the 4000 and 6000 psi jack pressure levels are not sharply defined; rather, these peaks are either flat-topped or a series of high-frequency pulses. As a result, definition of an arrival time of the first peak from data of this type becomes uncertain.

3.4 ASYMMETRY AND SHIELDING EFFECTS

The in situ tests measure velocity signals at relatively closely spaced sensor holes, in order to obtain a high shear strain. However, because of the uncertainties involved in drilling closely spaced sensor holes in the field, these holes will typically be staggered, rather than in a single radial line. This type of spacing avoids the shielding problem (i.e., distortion of the signal at a farther sensor due to the presence of a closer sensor along the same radial line); however, it presumes that the signal generated at the anchor and propagated

(a) AT ANCHOR

(b) AT HOLE 1

(c) AT HOLE 2

(d) AT HOLE 3

FIGURE 3-13. EFFECT OF JACKING PRESSURE ON VELOCITY TIME HISTORY MEASUREMENTS CORRESPONDING TO 58 LB HAMMER DROPPED 2 FT

through the soil is symmetrical with respect to the centerline of the anchor. To verify this assumption, tests at the high and low-energy levels were carried out in which velocity signals were measured at circumferentially spaced sensors located at approximately equal radial distances from the anchor. These sensors correspond to Holes 2, 4 and 5 (Fig. 3-1), in which Hole 5 is in a direct radial line with Hole 1, and is therefore shielded from the anchor source by that hole. Therefore, a comparison of measurements at Hole 5 to those at Holes 2 and 4 also gives information regarding potential shielding effects on the measured velocity signal.

Results of these tests are provided in terms of wave arrival times and peak vertical velocities. As shown in Fig. 3-14a for both the high and low-energy levels, the arrival time of the wave is not strongly dependent on the circumferential placement of the sensor holes. Only about a 10 percent difference exists in the arrival times at Holes 2, 4 and 5, whether by first peak or zero crossing. Differences in these arrival times appear more closely related to minor differences in the radial positions of Holes 2, 4 and 5 than to any asymmetry or shielding effects.

A somewhat larger difference exists in the peak vertical velocities than in the arrival times. Fig. 3-14b indicates similar peak velocities at Holes 2 and 5; but at Hole 4, the peak velocities are 20 to 30 percent lower. The differences between Hole 4 and Holes 2 and 5 may be at least partially due to statistical scatter, since only a single test was processed at each energy level. As noted previously, the statistical variability in the peak velocities is much greater than in the delay times.

(a) ARRIVAL TIME OF WAVE

(b) PEAK VERTICAL VELOCITY

FIGURE 3-14. EFFECTS OF ASYMMETRY AND SHIELDING ON WAVE ARRIVAL TIME AND PEAK VERTICAL VELOCITY

3.5 CONCLUSIONS

From the preceding statistical evaluations of the special test results, the following general conclusions were reached.

1) Repeatability - Statistically the data indicate that the variation in delay times between respective holes are small while the scatter in peak velocity amplitudes are much larger (typically 15 to 25 percent). This variation in the data is considered small in view of the fact that the components which apply the impulse are submerged in drilling mud, that they are manually controlled from the ground surface by means of a hoisting cable, and that they are subject to variables such as the friction between the hammer and the borehole wall and the drill rod. Also, the described gradual increase in coupling between the anchor and soil wall with repeated testing accounts for a large part of the scatter in the results. Therefore, for the hard clays, this data together with the visual observation of the shapes of superimposed signatures for a number of cases, (Figs. 3-7 through 3-10), leads to the general conclusion that the test is indeed repeatable. Furthermore, testing at the three sites in different soil types supports this conclusion by consistently producing velocity-time history signatures of the same general shape as obtained in this special testing program.

2) Prior Load History Effects - Random scatter of the test values of delay time and velocity amplitudes between successive tests make it difficult to evaluate, quantitatively, the effect of prior load history. The stress distribution of applied shear and radial stresses, however, decreases so fast with radial distance that the natural state of stress should not be significantly changed beyond a distance

of one or two diameters from the anchor. Comparisons of delay times for the close-in sensors, where presumably the effect is likely to be most significant, also support this conclusion. Qualitatively, therefore, we believe that the effect of prior load history is small for these soils and probably, if it were evaluated close in, would only be noticeable in the vicinity within about one to two diameters of the anchor hole.

3) Jacking Pressure Effects - In situ low jacking pressure tests showed no consistent delay time trends, when conducted at the same energy level using different hammers. Also at the low jacking pressures, more complex (distorted) signal shapes were recorded at all stations. This tends to indicate that positive coupling and the desirability of keeping anchor slippage to a minimum is extremely important. At higher jacking pressures, changes in delay time at constant energy levels were relatively small. This comparison leads to the general conclusion that the radial jacking stress applied to these soils does not significantly alter the characteristics of these soils.

Comparisons of particle velocity amplitudes for these tests at increasing jack pressures show a gradual decrease in the particle velocity amplitude at the anchor, with a corresponding increase in particle velocity amplitude at the free-field sensors. This trend indicates that better coupling (higher jacking pressures) of the anchor achieves higher shear strains in the soils and less slippage at the anchor. Therefore, from these tests, we conclude that reasonably high jack pressures should be used in this test. These higher pressures also lead to less complex, more uniform, velocity time signatures at all recording stations.

4) Effects of Variations in Hammer Weight and Height of Drop - For the two higher jacking pressures, the tests performed at a constant energy level indicate that the heavier weight (150 pounds) and smaller drop (.77 feet) results in slightly higher particle velocities (strains) than does the light weight (58 pound) and higher drop (2 feet). The data also show that as the height of drop is increased by 2.5 times the resulting particle velocity is not increased proportionally, but only by a factor of .3 to 1.2. Both of these relationships indicate that for this test heavier weights and lower heights of drop are more desirable. Also, with the heavier weights, the hammer falls more freely through the heavy drilling mud producing more repeatable test results.

5) Asymmetry and Shielding Effects - In these special tests, differences in the arrival times were found to be small (about 10 percent). This small amount of scatter appears more related to minor differences in the radial distance between the recording sensor and the anchor. While the particle velocity amplitudes show a much greater difference for the two cases considered, they are well within the statistical scatter of the data evaluated in the repeatability tests (45 cases). Therefore, for these radial distances, the effects of both asymmetry and shielding are considered unimportant in this test.

IV. TESTING AT THREE SITES

Field testing was accomplished at three sites to obtain actual moduli data in different soil types and to check the adequacy of the equipment to perform at depth. Site 1 was located on the west side of Queen Anne Hill in Seattle. This site was selected because the subsurface materials consisted of hard clays that, when drilled, would stand unsupported for long periods of time. Site 2 was located on the Fort Lawton Military Reservation also in Seattle. This site was selected because the subsurface materials consisted of dense sand. Sands, in general, when drilled tend to cave more readily and, therefore, experience gained in testing in such materials where casing may have been required was considered desirable. The third site was located at an Air Force Radar Station in Birch Bay, Washington. The subsurface materials at this site consisted of medium to stiff marine clays. This site was selected because it was desirable to check the adaptability of the equipment to work in soils of different consistency. Also, the measured shear wave velocities in similar materials with a vastly different stiffness could be compared.

The following sections discuss each site, the subsurface conditions and the moduli results obtained from the in situ test. Also included is a final section comparing the shear velocity results of all sites with values obtained by conventional field and laboratory tests.

4.1 TESTING AT SITE 1

4.1.1 Site Description and Geology

Site 1 is located at the base of a 25-foot high steep cut on the west side of Queen Anne Hill, Seattle,

Washington. Locally the area contains small industrial buildings with occasional older residential dwellings nearby.

The soil exposed in the adjacent steep cut is very dense, silty, gravelly SAND (Vashon Till). The soil unit at this site is thought to be a non-glacial unit formed in late Pleistocene times during the transition from late interglacial time to Vashon glacial time (Mullineaux, et al, 1965). Geologic evidence indicate that this deposit was formed in a non-glacial environment of flood plains and shallow lakes. This condition is characterized by layers of silty clay and clayey silt interbedded with layers of sand that are locally crossbedded. Wood fragments, predominantly spruce and pine, are scattered throughout the unit suggesting that at the time of deposition, the climate was cooler and moister than at present.

The subsequent advance of the Puget glacial lobe into the Seattle area has heavily overconsolidated most of the soils under the weight of many feet of glacial ice. The soils at this site have been compressed by this ice and are, therefore, generally hard materials.

4.1.2 Subsurface Conditions

The subsurface soils at Site 1 are generalized by the soil boring log presented as Fig. 4-1. This material, in general, consists of hard, gray, silty CLAY for the full depth of this boring, although locally numerous sand, silt and gravel zones of various thicknesses were encountered. The hard consistency of the clay is reflected by a high penetration resistance in most cases greater than 30 blows per foot. The penetration resistance versus depth is plotted in Fig. 4-1.

FIGURE 4-1 LOG OF BORING AT SITE NO. 1

The average water content is about 22 percent for the full depth. This average value in most cases is equal to or less than the plastic limit indicating the overconsolidated character of these soils.

Four unconfined compression tests gave shear strength values ranging between 1.2 and 3.9 tsf or an average value of about 2.5 tsf. Three triaxial unconsolidated, undrained tests performed at the approximate effective overburden stress gave shear strength values of 4.8, 4.5 and 3.1 tsf. These shear strengths are based on one-half the unconfined compression strength or one-half the deviator stress for the triaxial tests. The detailed results of these and other tests are presented in Appendix A.

The groundwater level was not measured in the borings due to the use of heavy drilling mud, but based on surface evidence it is expected to be relatively high. Groundwater seepage from the face of the cut and a wet ground surface during the winter indicate that most of the year the water table is at the ground surface while during the summer it probably drops several feet. The explorations and testing at this site were accomplished in March and April, 1974.

4.1.3 In Situ Test Results

The results of the in situ testing at this site are presented and summarized in Figs. 4-2 through 4-5. The decrease in shear velocity with strain between the general range of 10^{-3} to 10^{-1} percent is illustrated by the curves shown in Figs. 4-2 and 4-3. The change in shear velocity for these hard clays occurs at a rate of about 2.5 to 3 times faster at strains between 10^{-3} and 10^{-2} than at strains

FIGURE 4-2 SHEAR WAVE VELOCITY VS STRAIN AT SITE NO. 1
DEPTH 0-50 FEET

FIGURE 4-3 SHEAR WAVE VELOCITY VS STRAIN AT SITE NO. 1
DEPTH 50-90 FEET

FIGURE 4-4 IN SITU TEST RESULTS AT SITE NO. 1

FIGURE 4-5 SHEAR MODULI OF HARD CLAY AT SITE NO. 1

one order of magnitude higher. Since the modulus varies as the square of the shear velocity, the large variation in modulus in this lower strain range is even more pronounced (Fig. 4-5). The family of curves in Figs. 4-2 and 4-3 also show a gradual increase in shear velocity with depth. Data for all test depths were not shown on this plot for reasons of clarity. The variations of shear wave velocity with depth are, however, indicated in summary form in Fig. 4-4.

Because of the large variations in the silt content within this deposit and the presence of numerous thin sand lenses throughout, this material and the test results are considered representative for a site containing multi-layered and variable soils in contrast to a uniform deposit. As a result, this data was not normalized, but is summarized in terms of shear modulus as shown in Fig. 4-5. Evaluation of these results and their comparison with other conventional tests is presented in Section 4.4.

The in situ testing program for this first site in general went very smoothly. The use of heavy bentonite drilling mud in anticipation of caving of the deeper sandy and gravelly deposits, however, prevented the lighter hammers from falling freely the full drop distance. As a result, most tests at this site were performed using the 150-lb. hammer with drops ranging from 2 feet to less than 1 inch. This hammer worked so well for this site, that the procedures used at the other two sites were simplified by also adopting this one hammer for all tests.

It was also observed that the anchor hole had an average diameter of about 9 inches near the bottom and gradually increased to 12 inches near the top. Spiraling of the auger and eventual erosion of the walls of the borehole

was the probable cause. This condition provided an advance indication of the possibility of obtaining too large a hole diameter at shallow depths at other sites where the soil walls are less stable, such as Site 2. The existing anchor assembly achieved adequate coupling in borings with a diameter ranging between 8 and 12.5 inches.

4.2 TESTING AT SITE 2

4.2.1 Site Description and Geology

Site 2 is located about 2.3 miles northwest of Site 1 on the Fort Lawton Military Reservation also in Seattle, Washington. The surface topography of the Reservation consists primarily of grass-covered rolling hills with changes in elevation of several hundred feet. The actual site is located in the southwest part of the Reservation, about 400 feet from Magnolia Bluff, which comprises steep vertical sea cliffs overlooking Puget Sound.

The sand encountered at this site, and exposed in the nearby sea cliffs, is the Esperance sand member of the Vashon Drift, a conformable late Pleistocene sequence of interglacial and glacial sediments (Mullineaux et al, 1965). This sand member appears to be chiefly a preglacial fluvial and lacustrine deposit that was laid down after the lake was mostly filled with silt and clay. The streams that carried the sand are regarded as having come principally from the advancing Puget lobe (Crandell, et al, 1965). Locally to the east this deposit is directly overlain by Vashon till.

Radio carbon dating of samples indicate that this sand unit was deposited during a short geological period (several hundred years) approximately 15,000 years ago, just

before the ice overrode the locality (Mullineaux et al, 1965). Like Site 1, the high density of this sand was achieved by compaction under the heavy weight of past overlying glaciers.

4.2.2 Subsurface Conditions

The subsurface soils encountered at the test site are generalized by the soil boring log presented as Fig. 4-6. These soils consist largely of silty fine SAND with a silt content varying between 8 and 18 percent. It also is mostly gray in color and very dense, especially below a depth of about 15 feet. The results of Standard Penetration Tests are presented on the boring log. This test provides an estimate of the relative density or compactness of granular soils. A penetration resistance greater than 50 blows per foot indicates a very dense material.

Eleven gradation tests, performed on samples obtained throughout the 100-foot test depth, indicate that this deposit is of very uniform grain size. The average water content is estimated to be about 9 percent.

The groundwater level was measured at this site several weeks following drilling and testing, and was found to be about 85 feet below the ground surface. This reading was made in one uncased hole left open for this purpose. Explorations and testing at this site were accomplished in May, 1974.

4.2.3 In Situ Test Results

The in situ test results for Site 2 are presented in Figs. 4-7 through 4-10. The relationships of shear wave velocity to strain for test depths of 0 to 50 feet and 50 to

FIGURE 4-6 LOG OF BORING AT SITE NO. 2

FIGURE 4-7 SHEAR WAVE VELOCITY VS STRAIN AT SITE NO.2
DEPTH 0 - 50 FEET

FIGURE 4-8 SHEAR WAVE VELOCITY VS STRAIN AT SITE NO. 2
DEPTH 50 -100 FEET

FIGURE 4-9 IN SITU TEST RESULTS AT SITE NO. 2

FIGURE 4-10 SHEAR MODULI OF VERY DENSE SAND AT SITE NO 2

100 feet are summarized in Figs. 4-7 and 4-8, respectively. Average data in general indicate, as for hard clays, a more rapid change in the velocity (approximately 2 to 2.5 times), for strains between 10^{-3} to 10^{-2} percent, as compared with strains of 10^{-2} to 10^{-1} percent.

This same shear wave velocity data for strains of 10^{-3} , 10^{-2} , and 10^{-1} percent are shown in Fig. 4-9 plotted with depth beside a description of the corresponding soils tested. This log and the log in Fig. 4-6 supported by laboratory and field penetration tests indicate that these soils have reasonably uniform physical and engineering properties. While these data do not show a large change in velocity with depth, when converted to shear modulus and normalized to the mean effective principal stress, the results fall within a relatively narrow band shown in Fig. 4-10. Thus, one simplified method for obtaining the average change in modulus with strain at this site would be to use the average curve shown in Fig. 4-10. A comparison of this data with data obtained from the more conventional tests is presented in Section 4.4.

The testing at this site was completed without any major testing difficulties. In anticipation of obtaining an enlarged auger hole in the upper 50 feet of depth (as obtained at Site 1), this upper zone was drilled and tested first prior to advancing the auger boring and subsequent testing below this depth. This procedure maintained a sufficiently small hole in this highly erodible soil so that firm coupling between the anchor and soil face was achieved. Also, it minimized the disturbance of the local soil in the upper zone prior to testing.

In an attempt to obtain a smoother drop of the

hammer on the anchor assembly, Revert drilling mud was tried rather than the standard bentonite mud used at Site 1. This mud not only maintained a stable borehole, but also provided a more viscous fluid in the hole during testing; which minimized friction between the hammer and the drill rod, and also caused the hammer to fall more freely achieving a larger impact force on the anchor assembly, for any given drop distance.

4.3 TESTING AT SITE 3

4.3.1 Site Description and Geology

Site 3 is located near Birch Bay, Washington, within the confines of the Blaine Air Force Station. Locally within the station, the test area was positioned in the southwest corner, about 100 feet behind the engineering building. The surrounding terrain is flat to gently rolling, with the immediate site being relatively level. The elevation of the site is approximately 67 feet above sea level. The surface cover at the site is grass. The grass cover extends to adjacent terrain, which contains localized wooded areas.

The subsurface soils in this area were believed to be deposited as glacio-marine drift, i.e., by floating ice masses. During the retreat of the last major glaciation in this area, the sea level was much higher than it is today. Large floating masses of glacial ice melted and dropped a mixture of silt and clay with minor amounts of sand and gravel and an occasional boulder onto the sea bottom. Through a process of rebound of the earth's crust, following the removal of the glacial ice and possibly the lowering of the sea level, or both, these sediments became exposed. In general, this soil deposit is normally consolidated.

It has a stiff top layer, approximately 8 to 12 feet thick (overconsolidated by dissiccation) and softer clay layers below it.

4.3.2 Subsurface Conditions

The subsurface soils at Site 3 are generalized by the soil boring log presented as Fig. 4-11. This boring at the center of the test area encountered clay from the surface to the bottom of the hole at 120 feet. A few thin sand seams and occasional gravel particles are dispersed throughout the deposit. The upper portion of the clay to a depth of 8 to 12 feet is sandy in places, gray-brown in color, and is considerably stiffer than the underlying material. This zone apparently has been subjected to seasonal wetting and drying, achieving its stiffness and compactness by dissiccation. The ground water level was not measured in the borings because of the heavy mud used in the drilling process. A small pit, however, dug beside one boring indicated that during the period of testing (May, 1974) the water table was probably about three feet below the ground surface. The lower limit of the seasonal ground water is probably at the base of the stiff surface layer.

The zone below the upper layer is gray in color and generally medium stiff for the next 70 feet of the boring. Near a depth of 80 feet the soil deposit gradually changes from medium stiff to stiff. The consistency of this material as indicated by the Standard Penetration Test is shown on the boring log.

The average water contents of the upper and lower zones are about 18 and 23 percent, respectively. Atterberg limit tests at select depths show that the average water content of the medium stiff zone is slightly less than

FIGURE 4-11 LOG OF BORING AT SITE NO. 3

midway between the liquid and plastic limits. The actual values in general indicate that the clay has a relatively low plasticity. Near the surface the natural water content is below the plastic limit.

Three unconfined compression tests gave shear strength values ranging between 0.1 and 0.5 tsf or an average value of about 0.3 tsf. Three triaxial unconsolidated, undrained tests performed at the approximate effective overburden stress gave shear strength values of 0.7, 1.0 and 1.6 tsf. The detailed results of these tests are presented in Appendix A.

Three consolidation tests also shown in Appendix A, indicate that the lower medium stiff clay deposit is slightly overconsolidated to a depth of about 70 feet. Below this depth, it is probably normally consolidated to slightly overconsolidated.

4.3.3 In Situ Test Results

The results of in situ testing for Site 3 are presented in Figs. 4-12 through 4-14. The change in shear velocity with strain, summarized in Fig. 4-12, illustrates two factors. First, if the data for depths 7 and 12 feet are ignored (the zone with stiff soils), it is evident that there is only a slight increase in velocity with depth. In many cases, any one curve for any depth overlaps other curves at depths both less than or greater than the depth in question. This in general indicates that the medium stiff clay has near identical modulus properties and is not strongly influenced by a change in confining pressure. Secondly, these same curves show about an equal change in shear velocity between the strains of 10^{-3} to 10^{-2} percent and between 10^{-2}

FIGURE 4-12 SHEAR WAVE VELOCITY VS STRAIN AT SITE NO.3

FIGURE 4-13 IN SITU TEST RESULTS AT SITE NO. 3

4.14 SHEAR MODULI OF MEDIUM STIFF CLAY AT SITE NO. 3

and 10^{-1} percent. This is considerably different from the results obtained for Sites 1 and 2 (i.e. sites with more competent subsurface conditions) and indicates the importance of recognizing that the rate of change in shear velocity with strain is different for different soils.

These same shear velocity data for this site are summarized with depth in Fig. 4-13. The change in shear velocity between the upper, more compact zone, and the lower zone is most evident in this figure.

The results are presented in terms of real values of moduli in Fig. 4-14. For clarity and because the soils from 10 to 70 feet have about the same moduli values only three average curves are shown. One curve is representative of the upper, more competent zone, while the other two are indicative of the lower zone grouped into two major depth zones.

As indicated in Fig. 4-13, tests at this site were conducted to a depth of 107 feet. The data and results for tests conducted below 72 feet are not shown because accurate construction of time distance plots (Fig. 2-10) was not possible. Following the completion of testing, it was determined that the alignment of holes containing Sensors No. 1 and 2 was such that each station was about the same horizontal distance from the anchor. Therefore, both sensors measured about the same results producing in effect only one meaningful data point. With only three resultant data points for each test at these depths, construction of a smooth curve is at best only approximate.

This difficulty points out the importance of making every effort to keep near vertical alignment of the test holes. As a matter of routine, it also points out that

survey data should be reduced immediately after it is taken such that adjustment in the location of the borings can be accomplished if necessary. For example, if one of two holes drilled is not well aligned, the misaligned hole can be designated the most distant hole from the anchor. Misalignment of the most distant hole usually has less influence on the resulting data.

One other difficulty encountered at Site 3 involved the drilling of the anchor hole. In anticipation of possible squeezing of these softer soils, the hole advanced for the anchor assembly was accomplished using a slightly larger size auger bit. When squeezing did not occur, a larger hole than desired was obtained. As a result, firm coupling between the anchor and the soil wall could not always be achieved. When this occurred, excessive slippage resulted producing more distorted records than at the first two sites. Usually, when slippage was excessive, the first major peak on each particle velocity time history was well defined on all records, while the second peak was only roughly defined and in some cases completely obliterated. For tests in these softer soils, it is therefore desirable to have a well defined load time history at the anchor (see Section 2.6.1) such that the first point of peak stress can be utilized rather than the second. Also, slippage in such cases where coupling is a problem can be minimized by using a lighter weight hammer.

4.4 COMPARISON OF IN SITU TEST WITH CONVENTIONAL TESTS

In order to compare the results of this new test with traditional field and laboratory tests, appropriate additional testing had to be accomplished at each of the three sites. In the field low strain level downhole geophysical measurements

described in Section 2.4, were accomplished at each site in one of the existing uncased boreholes following the in situ test program. The results of measurements accomplished at five foot depth intervals are shown for each site on Figs. 4-15 through 4-17. Based on particle velocity amplitude measurements accomplished at Site 3, the average strain level for these tests, as computed using Equation 2-4, is approximately 10^{-5} percent. At Site 1, the downhole tests were only conducted to a depth of 60 feet, because the hole caved before deeper measurements were obtained.

As noted in these figures not all data points are shown. Difficulties encountered in picking the precise arrival time of the shear wave from the complex signature caused the results obtained to be subjective and in many cases subject to considerable interpretation. Where such data was questionable, it has been omitted from the plots. In general, the downhole data at Site 3 was considered the best for the three sites.

From the general trends indicated on the plots, it may be observed that downhole geophysical values of shear velocity in general are slightly less than values obtained by the in situ test at strains of 10^{-3} percent. The downhole shear velocities are also less than values determined by the crosshole method (i.e. the low strain velocity obtained from the time of first arrival of the shear wave recorded in the in situ test - Fig. 2-10). These low strain crosshole values would correspond in most cases to a value which is equal to or slightly greater than the in situ value at the strain level indicated as 10^{-3} percent. Therefore, it can be concluded from these measurements that for these soil deposits, shear waves propagating in the vertical direction travel at a slightly lower velocity than those traveling in a horizontal direction.

FIGURE 4-15 SUMMARY OF SHEAR WAVE VELOCITIES AT SITE NO. 1

FIGURE 4-16 SUMMARY OF SHEAR WAVE VELOCITIES AT SITE NO. 2

One possible explanation of this result lies in the geologic formation of these deposits. Each of these deposits for the three sites were initially formed in flat lying areas and, therefore, are layered to some degree. Layered materials in the vertical direction generally have a larger variation in gradation and physical characteristics than in the horizontal direction. The wave velocity is more influenced by the presence of softer layers when it is traveling normal to them. Therefore, a wave traveling vertically will do so at a lower velocity, than when traveling horizontally, for the sites under consideration.

While the above data does show a variation in velocities at low strains by the two methods, we consider the presented explanation as preliminary in light of the fact that these measurements are difficult to evaluate. Further measurements and similar comparisons are, however, needed to substantiate this conclusion.

In the laboratory, cyclic triaxial tests were performed on three to six selected specimen from each site to determine modulus values considered representative at high strain levels. The cohesive materials at Sites 1 and 3 were tested in the undisturbed state, while the granular material at Site 2 was tested in a remolded state compacted to a very high density. The detailed results of the tests are presented in Appendix A. A summary of the cyclic triaxial test results converted to shear velocity for each site for a strain level of 10^{-1} percent are presented with depth on Figs. 4-15 through 4-17 along with data obtained by the other methods.

Comparison of the laboratory results with the in situ results at a strain level of 10^{-1} percent indicates various differences in shear velocity for each site. For Site 3 the agreement is considered good. The cyclic triaxial tests

give approximately the same or slightly higher (converted) shear velocities than the in situ tests. Therefore, for these slightly overconsolidated to normally consolidated soils, disturbance and other inherent test limitations do not apparently affect significantly the moduli results obtained by this laboratory procedure.

For the hard, overconsolidated clays of Site 1, the cyclic triaxial tests give consistently smaller values of shear velocities (or moduli) than the in situ test. The shear velocities in general by the laboratory procedure are lower than the in situ values by a factor of about 1.5 to 2. This means that the shear modulus measured in the laboratory by this procedure is correspondingly low by a factor of 2 to 4. These results indicate the importance of the effects of changing the state of stress and other sources of disturbance caused by sampling of this overconsolidated clay soil.

Sandy soils, as those encountered at Site 2, are even more difficult to test in the laboratory in an undisturbed state. It is known that for these materials, the stiffness (modulus) of the structure is very much controlled by the relative density and effective stresses (SW-AJA, 1972) plus the cementing effect of the active content of fine grained soils and the moisture. This latter bonding effect is lost and cannot be reproduced or evaluated in the laboratory by testing remolded specimens. Therefore, lower values of shear velocity or modulus should be expected from the laboratory tests when compared with in situ values. For these soils, the shear velocities from the laboratory tests were lower than the in situ test values by a factor of about 1.5 to 2. This magnitude is about the same as for the overconsolidated clays (Site 1). The advantages of making in situ tests are also evident in this case.

In summary, the in situ impact test provides measurements of shear modulus in the range of strains between 10^{-1} and 10^{-3} percent corresponding to strong earthquake motions. Laboratory cyclic triaxial tests and geophysical downhole tests provide measurements at strain levels above and below this range, respectively. The results of triaxial tests are very much dependent upon the disturbances inherent in all processes of sampling. The disturbance of the structure of the soils also affects the determination of shear modulus in the region adjacent to the anchor in the in situ test. This region close to the anchor will be further studied in continuing efforts; however, from the in situ results obtained at the three sites, and from their comparison with the results of triaxial tests, it seems that this disturbance is limited to a small concentric zone probably not larger than one diameter of the anchor.

Therefore, for earthquake design, the in situ test has the important advantages of providing a consistent set of measurements in the strain range of interest. Also, these measurements are obtained from a mass of soil essentially undisturbed and under its natural state of stress.

V. ANALYTICAL STUDIES

Analytical studies were carried out to provide additional insight into the phenomenology and results of the in situ test method. These studies included finite element calculations, for purposes of evaluating delay time procedures, and development of a closed form solution of an idealized mathematical model of the test setup. The results of these studies are described in this section.

5.1 FINITE ELEMENT RESULTS

5.1.1 Description of Problem

In the early data obtained from the test program, shear wave travel times were obtained using the cross correlation and first peak or crossing approaches described in Section 3. Both of these approaches employ the assumption that corresponding points on the velocity time histories recorded at two sensors are identical points (points of constant phase) on the propagating wave. However, when a propagating wave is decaying in amplitude, this assumption is not exactly true. Fig. 5-1 shows a propagating shear wave, attenuating according to an arbitrarily assumed envelope of peak response. Since no part of the wave can ever exceed the response envelope, the peak response recorded at any sensor location must correspond to that point of the propagating wave (point A) at which the slope of the wave equals the slope of the response envelope. This can never be the peak of the wave form (point B), since the slope of the wave at point B is zero. Note further that during the time interval from t_1 to t_2 , point A moves closer to point B because the slope of the response envelope decreases with increasing radial distance, r . Thus the points of peak response of the time history records propagate more slowly than

FIGURE 5-1. SPATIAL DISTRIBUTION OF RESPONSE NEAR THE PEAK OF A PROPAGATING, DECAYING SHEAR WAVE, AT TIMES t_1 AND t_2

the shear wave itself, although this effect is primarily significant near the anchor, where the decay rate is appreciable.

As a result of this observation, S. D. Wilson devised a graphical procedure to account for the differences between arrivals of the wave crest and the peak response measured at the sensors. This section contains the results of an analytical evaluation of this graphical method (herein termed the Wilson method) as well as the cross correlation and first peak techniques. This analytical evaluation is based on a linear elastic finite element analysis of an anchor-soil system whose geometry and material properties are generally representative of field conditions. An anchor velocity-time history taken from field measurements was applied directly to the soil grid, and soil velocity time histories were computed at selected node points within the grid. The Wilson method, as well as first peak and cross correlation techniques, was applied to the computed velocity time histories to obtain shear wave velocities. A mathematical development of the Wilson method which involves several simplifying assumptions was used in this evaluation rather than the graphical procedure. This mathematical development is provided in Appendices B and C.

There are several advantages of using a linear elastic soil model to evaluate the various techniques for obtaining shear wave velocities. First, in a linear soil representation, the "true" value of the shear wave velocity is known from the input elastic properties of the medium. Therefore, a definite basis for evaluating various methods for determining shear wave velocities from velocity time histories is established. Second, the problem toward which this investigation is directed shows up in the linear elastic system in the form of geometric attenuation. Hence, an elastic finite element analysis provides a simple approach for evaluating the methods for computing delay times without introducing

the soil nonlinearities and other effects that might tend to complicate the results.

5.1.2 Scope of Calculations

A finite element calculation was performed to provide a basis for evaluating analytically the Wilson method for obtaining shear wave velocities from the field data. This calculation was performed using a finite element grid consisting of elastic, axisymmetric quadrangular elements, as shown in Fig. 5-2. The locations of the anchor and the three sensor holes are shown in this figure. The elements in the vicinity of the anchor and the sensor holes are refined to a sufficiently small size to allow undistorted transmission of signals with a frequency content of up to 200 to 400 hertz, and to provide a sufficient number of data points to determine the attenuation of shear wave forms with increased radial distance. Away from the test area, where accurate transmission of the signal is less important, larger elements are used.

Boundary conditions for this grid consist of energy-absorbing boundaries along the bottom and right side and free surfaces along the upper and left side. Anchor motion is applied to the soil by prescribing the velocity of the nodal points along the anchor to be the anchor/soil motions recorded during Case 48 of the in situ special tests, as shown in Fig. 5-3. The soil is represented as an elastic medium, with a shear wave velocity of 800 fps, a Poisson's ratio of 0.40, and a unit weight of 100 pcf.

As indicated above, the first peak and cross correlation methods for computing time delays are included in addition to the Wilson method in this study. These techniques are discussed in Section 3 and therefore only a summary is presented here. Briefly, the first peak method involves noting

FIGURE 5-2. FINITE ELEMENT GRID

FIGURE 5-3. INPUT ANCHOR MOTION FOR FINITE ELEMENT CALCULATIONS

the arrival time of the first peak of the velocity reading at each of two adjacent sensors, and dividing the separation of the sensors by the time difference. The cross correlation method consists of artificially time shifting the entire velocity time history signal from one sensor relative to the other, computing the product of the shifted signals, and integrating and averaging the product over the time duration of the signal. This computation of the cross correlation function is repeated for different time shifts, and that particular shift resulting in the largest value of this function is considered to be the delay time between the two signals.

5.1.3 Results

Displacement and velocity time histories computed at nodal points 601, 606 and 610, of the finite element grid of Fig. 5-2 are presented in Fig. 5-4. These nodal points correspond, respectively, to the sensor locations in the Anchor Hole ($r = 4.5$ in.), Hole 1 ($r = 34.5$ in.), and Hole 2 ($r = 59.5$ in.), where r is measured from the axis of symmetry. The records shown in Fig. 5-4 are typical of the free-field data that would be recovered from a field test. Methods for determining the shear wave velocity from field data are designed to use data primarily of the type shown in Fig. 5-4.

As mentioned in Section 5.1.1, the Wilson method recognizes a distinction between the peak of the time history record and the peak of the propagating wave. The ability to deduce the arrival time of the peak of the propagating displacement wave is of prime importance in applying the Wilson method. In order to determine, from the finite element data, the location of the peak of the propagating wave at various instants in time, several instantaneous displacement profiles were plotted with radial distance at several discrete times.

(a) ANCHOR DISPLACEMENT RECORD

(b) ANCHOR VELOCITY RECORD

FIGURE 5-4. DISPLACEMENT AND VELOCITY RECORDS
AT THE SENSOR LOCATIONS

(c) HOLE 1 DISPLACEMENT RECORD

(d) HOLE 1 VELOCITY RECORD

FIGURE 5-4. (CONTINUED)

(e) HOLE 2 DISPLACEMENT RECORD

(f) HOLE 2 VELOCITY RECORD

FIGURE 5-4. (CONCLUDED)

Fig. 5-5 contains profiles for times of 5.0, 5.5, 6.0, 6.5, 7.0, 8.5, 9.0, and 11.5 milliseconds.

Fig. 5-6 provides the envelope of peak response of the displacement wave. As explained in Appendix B, the slope of this peak response curve at each sensor location must be known in order to apply the mathematical development of the Wilson method. Obtaining the peak response envelope poses a problem in the application of this method to field test data, because of the coarse spacing of the sensor locations at which the soil motions are obtained. Because of the fine mesh size of the finite element calculation, however, there is ample data for constructing the response envelope in the present case. The slopes of the response envelope at three sensor locations are given in Table 5-1.

TABLE 5-1. SLOPE OF RESPONSE ENVELOPE

Sensor	Location (In. from axis of symmetry)	Slope, in/in
Anchor	4.5	0.00210
Hole 1	34.5	0.000206
Hole 2	59.5	0.000102

As explained in Section 5.2.1, application of the first peak method requires only a knowledge of the distance between the anchor and the sensors, and the time of peak velocity recorded at each sensor. Application of this method is performed in Table 5-2, using the data of Fig. 5-4. It should be kept in mind when evaluating the results of these methods that the shear wave velocity is a known input quantity. Its value is 9600 inches per second.

Cross correlation curves, obtained as described in Section 5.1.2, between the velocity histories recorded by the

FIGURE 5-5 INSTANTANEOUS DISPLACEMENT PROFILES

FIGURE 5-6. ENVELOPE OF PEAK DISPLACEMENTS

TABLE 5-2. APPLICATION OF FIRST PEAK METHOD

Inner Sensor	Outer Sensor	Separation (inches)	T* Inner Sensor	T* Outer Sensor	Travel Time (msec)	Computed Shear Wave Velocity (ips)	Ratio of Computed to Actual Wave Velocity
Anchor	Hole 1	30	3.5	7.0	3.5	8,571	0.89
Hole 1	Hole 2	25	7.0	9.5	2.5	10,000	1.04

T* is the time at which the peak velocity reading is recorded at a specified sensor.

TABLE 5-3. APPLICATION OF CROSS CORRELATION METHOD

Inner Sensor	Outer Sensor	Separation (inches)	Travel Time (msec)	Computed Shear Wave Velocity (ips)	Ratio of Computed to Actual Wave Velocity
Anchor	Hole 1	30	3.5	8,571	0.89
Hole 1	Hole 2	25	3.0	8,333	0.87

sensors at the Anchor and Hole 1, and between those at Hole 1 and Hole 2, are shown in Fig. 5-7. The point of maximum correlation determines the travel time of the signal between sensors. The results of the application of the time series method are presented in Table 5-3.

It should be noted that the results of all time determination procedures are adversely affected by the relatively large time interval (0.5 msec.) used between data points in the finite element solution. In contrast to this, field data measurements have been recorded at a time interval of 0.05 milliseconds. Had the time interval used in the finite element calculations been reduced to more nearly correspond to the field data resolution, the cross correlation and first peak methods would undoubtedly have produced a closer comparison with the true value of the shear wave velocity.

The iterative procedure described in Appendix B was next applied to the finite element data. The values of the slope of the response envelope tabulated in Table 5-1 were used in Equation B-6 of Appendix B. The execution of the iterative procedure is presented in Table 5-4.

It can be seen from Table 5-4 that the determination of the arrival time of the wave crest at the anchor failed to converge. As a result, the Wilson method could not be applied to the near-anchor regions for this particular example. This may be due in part to the complexities of the waves at the anchor-soil interface where, as indicated in Section 2, special data processing techniques are required. In addition, the Wilson method involves certain mathematical approximations in defining the response envelope and measured signals near the anchor (i.e., as truncated Taylor series), as described in Appendix B. These approximations are poor near the anchor

FIGURE 5-7. CROSS CORRELATION OF THE VELOCITY HISTORIES

TABLE 5-4. COMPUTATION OF SHEAR WAVE VELOCITIES USING WILSON METHOD

Location of Velocity Measurements			Iteration No.	Trial Wave Velocity (ips)	V _{pa} (ips)		Wave Crest Arrival Time (msec)		Travel Time msec	$\frac{\text{Wave Velocity (ips)}}{\text{Travel Time}} = \frac{\text{Distance}}{\text{Travel Time}}$
Station 1	Station 2	Separation Distance (in.)			Station 1	Station 2	Station 1	Station 2		
Anchor	Hole 1	30	1	0	0	0	4.812	8.419	3.607	8317
			2	8317	17.466	1.713	Velocity of anchor never reaches V _{pa}	--	--	Did not converge at this location in grid
Hole 1	Hole 2	25	1	0	0	0	8.419	11.170	2.751	9087
			2	9087	1.874	0.927	8.745	11.399	2.654	9420
			3	9420	1.941	0.961	8.757	11.408	2.651	9420

but appear more reasonable with increasing distance from it. In fact, the Wilson method provides an excellent estimate of the shear wave velocity between Holes 1 and 2, as shown in Table 5-4. Also good agreement is obtained by this method even closer in if a simplified graphical procedure is conducted using the instantaneous plots shown in Fig. 5-5. For example, the crest for the 7 and 11.5 millisecond plots occur 43.5 inches apart. This corresponds to a shear velocity of 9700 inches per second.

To gain insight into near-anchor wave characteristics the arrival of the wave crest at the anchor was also estimated from the displacement profile shown in Fig. 5-5 to be about 6.5 milliseconds. From Table 5-4, it is seen that the arrival time of the wave crest at Hole 1 is 8.8 milliseconds. Therefore, the time for the wave crest to travel from the anchor to Hole 1 is

$$\Delta t_{A-1} = 8.8 - 6.5 = 2.3 \text{ msec}$$

and the computed value of the average shear wave velocity in this region is

$$V_{s_{A-1}} = \frac{\text{distance between Anchor-Hole 1}}{\Delta t_{A-1}} = \frac{30 \text{ inches}}{2.3 \text{ msec}} = 13000 \text{ ips}$$

This is considerably in excess of the true shear wave velocity of 9600 ips for the soil medium in this model. This variation may be in part due to the relatively large time step used in the analyses. However, from this, it appears that near the anchor, the speed of propagation of the wave crest may not be a good indicator of the shear wave velocity.

5.1.4 Conclusions

Based on the finite element calculations described herein, conclusions are provided regarding the propagation of waves through the soil medium and the use of various methods for computing time delay:

1) Consistent points on a velocity signal measured at different sensor locations (i.e., first peaks, etc.) do not always correspond to consistent points on the wave. This effect is important near the anchor and becomes less important with increasing distance away from the anchor.

2) The Wilson method of computing time delays provides a rational procedure for determining the shear wave velocities at locations away from the anchor but, in its present form, does not appear applicable near the anchor. The difficulty in the vicinity of the anchor could possibly be related to the complexity of the waves generated in that region, as discussed in Section 2. Furthermore, the method has a number of uncertainties when applied to test data. First, as indicated in Appendix B, the method is based on a truncated Taylor series expansion of the response envelope and measured signal. This truncation appears reasonable for points further from the anchor but becomes questionable near the anchor. Second, the method requires a definition of a wave response envelope and its first spatial derivative, taking account of possible effects of geometric attenuation, dispersion, and hysteretic damping in the soil. In the finite element analysis, the closely spaced node points near the anchor permitted an adequately assumed definition of the response envelope; however, in the field, the response envelope and especially its spatial derivatives cannot be well defined by the relatively coarsely spaced sensors. Sensitivity studies could, however, be performed

together with limited testing to establish this envelope empirically.

3) Both the first peak and cross correlation methods gave reasonable overall correlations with the actual shear wave velocity, although in the region beyond the first hole, the Wilson method appears to provide improved results. The application of the time determination methods for these calculations was limited by the relatively large time step (0.5 msec) used between data points in the finite element calculation, as compared to that used in the field data (0.05 msec).

4) In view of the simplicity and reasonable accuracy associated with the use of first peaks for estimating delay times, this method based on this analytical study appears preferable to the cross correlation and Wilson methods. However, regardless of which method is selected, they all, for the non-linear case, apply to the region away from the anchor. With only data from three stations in the field, construction of a time distance curve (Fig. 2-10) through these points (excluding the anchor record) is highly suspect and subject to some interpretation.

5.2 CLOSED FORM SOLUTION

5.2.1 Description of Problem

During evaluation of in situ test data as well as finite element representations of results from this testing procedure, a number of questions arose. For example, the front of the waves in the soil indicated the presence of precursors whose origin could not be clearly defined by evaluation of the tests and the finite element results themselves. In addition, it became desirable to obtain additional information regarding the manner by which the amplitudes of the signals

decayed with distance from the anchor. A closed form solution offered promise as a useful tool in considering such questions. It was judged that this solution would help evaluate basic characteristics of the in situ soil behavior that might otherwise be masked by the complexities inherent in the field data or the finite element analyses.

In this section, the formulation of a closed form solution to help answer the above questions is provided and results obtained to date are presented. In this, much of the work on the closed form solution consisted of the formulation and coding of the method and the evaluation of the closed form solution through a comparison with finite element results. Further refinements of the closed form solution may be implemented in future work and computations carried out, as required.

5.2.2 Description of Approach

The configuration considered in the closed form solution is compared to the actual in situ test setup in Fig. 5-8. This figure indicates the closed form solution to be based on the following idealizations of the actual in situ test conditions.

- a. Waves propagating from the anchor are axisymmetric with respect to the centerline of the anchor hole.
- b. The depth of the anchor below the ground surface is sufficient such that boundary effects of the free ground surface can be ignored.
- c. The axisymmetric section (Fig. 5-8b) representing the region of soil under consideration is fixed along the right side boundary, and rollered along the top and bottom boundaries. The left side boundary represents the anchor hole wall, which

NOTES:

- (1) VALUES OF H , R , AND b CAN BE SELECTED ARBITRARILY
- (2) TIME VARIATION OF $S(t)$ IS ARBITRARY
- (3) μ = SHEAR MODULUS
 ν = POISSON'S RATIO
 ρ = MASS DENSITY

FIGURE 5-8. COMPARISON OF *IN SITU* TEST SETUP AND CONFIGURATION CONSIDERED IN CLOSED FORM SOLUTION

is stress free above and below the anchor.

d. The effects of the anchor upon the soil medium are applied by means of an arbitrarily time varying shear stress distributed uniformly over the length of the anchor. The effect of jacking pressure has not been included in the analysis.

e. The soil is represented as a linear, homogeneous, elastic medium.

The implications of the above assumptions warrant some discussion. First, the boundary conditions imposed in assumption c were selected such that a relatively simple form of solution would result. Applications of the closed form solution will cover, primarily, situations in which the soil medium is of infinite extent. The boundaries of Fig. 5-8b must, therefore, be placed far enough from the anchor that, within a reasonably large neighborhood of the anchor, the region appears infinite. The particular choice of boundary restraint imposed along these remote boundaries is, therefore, of little consequence.

As a second point, the decision to exclude jacking pressure (assumption d) is reasonable in view of assumption e. For an elastic system, the principle of superposition is valid. The anchor loading can therefore be separated into a static normal stress resulting from the jacking pressure and a dynamic shear stress resulting from the anchor motion. It is the response due to the latter component of load which is determined from the closed form solution. To obtain the response due to the total load, one must add a static stress state to the dynamic results.

Perhaps the most important assumption associated with the closed form solution is the use of a linear elastic

representation of the soil material. When evaluating this assumption, a number of points should be made. First, the purpose of the closed form solution is to provide a simple, inexpensive approach for investigating basic trends regarding the in situ soil behavior, and not to perform a detailed evaluation of actual test data. It is judged that a linear elastic soil model is satisfactory for this purpose. Furthermore, past practice in many soil dynamics applications have often relied heavily on the use of closed form solutions with linear elastic soil models (i.e., the use of the theory of a rigid disk on an elastic half space for soil-structure interaction evaluations). The use of the closed form solution described in this section is consistent with past applications of this type.

5.2.3 Solution Technique

The solution of the boundary value problem of Fig. 5-8b is obtained using a technique similar to a Green's function analysis. Rather than solving the boundary value problem illustrated in the figure, the homogeneous problem is considered. This latter problem is identical to the problem of Fig. 5-8b except that: (1) there is no input shear stress ($S(t) = 0$); and (2) the initial state of motion of the region consists of a velocity field that vanishes everywhere except at a single, arbitrary point (delta function distribution of initial velocity). The solution to the problem posed in Fig. 5-8b is obtained from this homogeneous solution by use of a reciprocal relation, such as the "general reciprocal relation for elastokinetics" (Fung, 1965).

Functions that satisfy the equations of motion in the interior of the region, and satisfy homogeneous boundary conditions, exist only for special values of vibration frequency known as the eigenvalues of the system, or the natural frequencies

of free vibration. The solution of the homogeneous problem is a linear combination of these functions, known as eigenfunctions. The coefficient of each eigenfunction (known as the Fourier coefficient) in the homogeneous solution, is determined from the initial conditions, making use of the orthogonality relationship between eigenfunctions.

The solution obtained by this method is included in Appendix D. This solution, as represented by Equations D-1 and D-2 of Appendix D, is valid for any point in the interior of the region of Fig. 5-8b. However, to use this solution in a computer code, the doubly infinite summation involved in these equations must be truncated. The quality of the computer output, therefore, depends on the speed of convergence of the solution at the point in question, and on the number of terms used in the summation.

A difficulty exists for points in the vicinity of the anchor. The condition to be satisfied at the anchor is that the shear stress must correspond to a finite, nonzero value defined by the input stress time history, $S(t)$. However, the eigenfunctions from which the solution is constructed employ the condition that the shear stress must vanish at the anchor. Thus the solution attempts to satisfy an inhomogeneous boundary condition in terms of homogeneous eigenfunctions. Fourier analysis techniques permit such a situation; for example, as illustrated in Fig. 5-9 (Wayland, 1957) a square pulse can be represented by a truncated sine series. In this, the value of the square pulse at the end points of the interval is 1, while each term of the sine series is zero at the end points. However, as can be seen in Fig. 5-9, speed of convergence at points near the boundary is slow by comparison with that of the interior points.

Two courses of action are available with which to deal

FIGURE 5-9. TRUNCATED FOURIER SINE SERIES REPRESENTATION OF SQUARE PULSE

with this reduced rate of convergence at the anchor-soil interface. The first is to apply a method of satisfying time-dependent boundary conditions first described by Mindlin and Goodman, 1950. Using this method, a transformation is made whereby all inhomogeneous boundary conditions are replaced by a time varying body force field distributed throughout the medium. It is the transformed stress state having homogeneous boundary conditions, rather than the original stress state, which is represented as a summation of eigenfunctions. Good convergence can, therefore, be expected throughout the region, including points near the anchor.

A problem arises in applying the above method due to the discontinuity in boundary conditions shown in Fig. 5-8b. The shear stress along the anchor is nonzero, whereas above and below the anchor, the shear stress vanishes. The shear stress distribution expected in the medium in the vicinity of one of these discontinuities is illustrated in Fig. 5-10. The point of discontinuity in the figure is seen to represent a singularity of the shear stress distribution, since the value of shear stress at that point, computed as the limiting value as the discontinuity is approached from some point in the region, depends on the direction of approach. When this inhomogeneous boundary condition is exchanged for a body force field, following the procedure of Mindlin and Goodman, the resulting body force field is singular. This singular body force field must be integrated over the region for each eigenfunction considered in the summation. Since numerical integration procedures for singular integrals are complicated and time consuming, use of this method is not considered attractive unless the integrals can be evaluated in closed form. Although this latter question has not been investigated as yet, it may be considered in future refinements of this closed form solution technique.

FIGURE 5-10. EXPECTED SHEAR STRESS DISTRIBUTION IN THE SOIL MEDIUM AT THE BOTTOM OF THE ANCHOR

The second course of action by which the slow convergence near the anchor can be dealt with is to use more terms in the truncated summation. The implication of this method is that there will always be a region of insufficient accuracy surrounding the anchor. The accuracy of the truncated summation will be adequate outside this region, and the extent of the region can be reduced by considering more terms in the summation. This approach has been used thus far in carrying out computations using the closed form solution.

5.2.4 Comparison of Closed Form Solution with Finite Element Results

To provide a basis for evaluating the closed form solution, results from the finite element analysis described in Section 5.1 were compared to results from the closed form approach. In this, it is noted that the configuration considered in the finite element analysis was identical to that of the closed form solution (Fig. 5-8b), except that (1) the top boundary was free rather than rolled, and (2) the finite element analysis utilized energy absorbing techniques along the right and bottom boundaries of the grid to simulate a soil medium of infinite extent. Material properties and geometrical data used in the closed form solution are provided in Table 5-5.

As noted in Section 5.1, the finite element analysis utilized velocity time history input at the anchor-soil interface whereas the closed form solution is formulated in terms of shear stress input. Thus, in order to carry out this comparison, the input velocity time history used in the finite element analysis had to be converted to an equivalent shear stress history for use as input to the closed form solution. This was carried out by first utilizing 450 terms of Equation D-1 (Appendix D) to express the velocity at a point on the anchor in terms of shear stress. Fourier transform

TABLE 5-5. MATERIAL PROPERTIES AND GEOMETRICAL
DATA USED IN CLOSED FORM SOLUTION

Shear Modulus (psi)	13800
Poisson's Ratio	0.40
Mass Density (lb-sec ² /in. ⁴)	1.5×10^{-4}
P-Wave Speed (ips)	23500
S-Wave Speed (ips)	9600
Inner Radius (in.)	4.5
Outer Radius (in.)	360
Half Height of Region (in.)	360
Half Height of Anchor (in.)	24

techniques were then used to invert Equation D-1, to provide the shear stress history that corresponds to the desired anchor velocity history. This was then used as input to the closed form solution.

Comparisons of the finite element and closed form results are shown in Figs. 5-11 and 5-12. Fig. 5-11 shows a comparison of the particle velocity at two sensor locations (Hole 1 and Hole 2) as computed by the two methods. The behavior of both solutions is strikingly similar in magnitude, frequency, time of arrival, and the presence of precursors. The major difference between the solutions is in the magnitude of the precursor.

Fig. 5-12a shows a comparison of peak velocities as a function of radial distance. Agreement between the methods appears excellent out to a range of about 50 in., where the difference in boundary restraint between the two calculations begins to have an effect. The problem of convergence near the anchor is clearly identified in Fig. 5-12b, which plots the peak shear stress as a function of range. The peak shear stress computed by the closed form solution, vanishes at the anchor, and begins to approach a reasonable solution only beyond a radius of about 15 in.. This situation can be improved (but not eliminated) by considering more terms; however, this is costly. At larger ranges, beyond about 50 in., a gradual divergence of the two solutions occurs because of the difference in boundary conditions.

5.2.5 Conclusions

A closed form solution has been developed which represents a promising technique for providing insights into the in situ test results. In regions where its results have not been distorted by boundary effects or slow convergence at

(a) HOLE 1

(b) HOLE 2

FIGURE 5-11. A COMPARISON OF PARTICLE VELOCITIES COMPUTED FROM FINITE ELEMENT AND CLOSED FORM SOLUTIONS

(a) PEAK VELOCITIES

(b) PEAK SHEAR STRESS

FIGURE 5-12. COMPARISON OF PEAK VELOCITIES AND SHEAR STRESSES COMPUTED BY FINITE ELEMENT AND CLOSED FORM ANALYSES

the anchor, the closed form solution has indicated excellent correlations with finite element results. However, these undistorted regions are presently too limited to be of practical value. Reasons for these limited regions of application of the solution technique are related to the number of Fourier series terms which can be considered in the summation process. This number of terms is limited not only by computation time, but by peripheral storage limitations, access of peripheral storage, and in the manipulations involved in merely adding up the terms. Therefore, it is necessary that a simpler, faster algorithm be developed.

In view of the desirability of defining the behavior of the soil medium at the anchor-soil interface, a simplified model of the in situ test configuration appears worthy of consideration, at least until the problem of convergence near the anchor in the present model has been solved. This simplified model, for which a solution can be obtained, would consider the anchor shear stress to be distributed along the entire length of the hole. The response of the medium as computed from this model would provide a reasonable representation of the behavior of the soil near the anchor where the applied shear stresses along that portion of the anchor hole beyond the anchor itself would have a negligible effect.

VI. CONCLUSIONS AND FUTURE WORK

The results obtained by performing special tests at one site and actual tests at three sites indicate that this new test is well adaptable for testing at depth, as well as in different soil types and produces shear moduli results especially suited for solving earthquake engineering problems. Moduli values obtained from this test are achieved not only by in situ methods but also at the strains of interest (i.e., in the strong motion earthquake region). While good field data was obtained at all three sites, experiences gained from this testing lead to the conclusion that additional work is required to simplify both test procedures and the evaluation of data. Careful planning, careful testing techniques, patience and flexibility of equipment are required for performing this test. However, with additional simplification of the technique, this test, in our opinion, can be standardized and will produce more positive predictable field data than existing field and laboratory techniques. Based on our experience with this test, it also has two not so obvious advantages over existing field geophysical methods:

- a. the amplitude level for the in situ test is so large, that background noise is not present, and
- b. the well coupled, controlled input source for this test produces a predictable signature which is similar for most soils. Therefore, identification of the shear wave from a complex pattern of waves, as well as its time of first arrival is not a problem.

From our studies of the test results, it was established that modifications in the data acquisition system, the anchor system measuring procedures, and in data interpretation

are required in future work.

While the velocity data obtained from this work was of sufficient precision, processing of paper tapes proved cumbersome (i.e., approximately 68 feet of paper tape per test). The data acquisition system can, therefore, be greatly improved by placing future data on digital magnetic tape in the field. This tape can then be placed directly into the computer and analyzed or plotted as required. Also the large anchor system was found to have a practical depth limitation in many soils of about 100 feet. Below this depth, it is generally difficult to drill and maintain a 9-inch diameter uncased borehole over long periods of time while heavy equipment (about 350 lbs.) is raised and lowered constantly. A smaller more compact anchor unit capable of going through about a 4-inch diameter borehole either cased or uncased is planned for consideration in future work. This smaller unit would enable testing to be conducted at depths even greater than 100 feet.

As discussed briefly in Section 2, modifications in measuring procedures will consist of adding an appropriately designed load cell to the anchor assembly. The load cell will be designed to have a flat response at high frequencies and will be adapted to record the load applied by the hammer as a function of time, $F(t)$. This measurement will provide added and possibly more precise information regarding the determination of the times at peak stress at the anchor. It may also lead to a better understanding or data interpretation of the anchor system, possibly to the extent that hysteretic damping can be approximated in situ.

Once these improvements are accomplished, future work is planned to consist of testing at four new sites. The sites will be located at accelograph stations, where strong motion earthquake records have been obtained. Appropriate laboratory testing

to determine the physical and engineering properties of the soils at these sites is also planned. This added information will serve a 3-fold purpose. First, it will define the soil conditions at those sites, whose earthquake records are available and presently used in the design of nuclear plants. Also the moduli values obtained at these strong motion stations will provide added research information for checking presently used response analysis techniques. Finally, experience will be gained in testing with the new equipment to presumably enable further standardization such that the test can be simplified for performance as a routine test at all potential nuclear power plant sites.

Additional analytical work is planned that is associated with the evaluation of field test data and the implementation of analyses to provide further insights into the basic soil behavior during the in situ tests. Of particular importance is the development of improved and simplified procedures for interpreting the data near the anchor-soil interface, since it is in this region that soil strain levels representative of strong earthquakes are typically generated. In addition, processing of the data for limited additional special tests is planned. Finally, considerable data processing effort will be associated with future testing.

As discussed previously, techniques presently used in the in situ tests for determination of shear moduli and strains from the field data are based partly on one-dimensional linear elastic theory. Therefore, it is of prime importance to evaluate analytically the adequacy of these simplified methods when applied to nonlinear soil materials and an axisymmetric configuration that simulates the in situ test setup. To carry out this evaluation, it is planned to perform nonlinear finite element analyses of the in situ field configuration. In these analyses, shear moduli and strains determined from the

nonlinear material model would be compared to the moduli and strains determined from the methods used to process the field data. In addition, strain levels in the vicinity of the applied loads would be evaluated to determine the adequacy of the level of the applied loads as well as the load application procedures.

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APPENDIX A

LABORATORY TEST PROCEDURES AND RESULTS

APPENDIX A

LABORATORY TEST PROCEDURES AND RESULTS

Index, mechanical and dynamic properties of the soils at each site were measured by laboratory tests performed on selected samples obtained from the field explorations. Since the laboratory procedures for testing the soil samples at each site are in general the same, they are described together in Sections A-1, 2 and 3. These descriptions of procedures are then followed by the detailed results, each test grouped with one of the three corresponding sites. The results included as Sections 4, 5 and 6 for Sites 1, 2 and 3 respectively are presented largely as tables and figures.

At the beginning of the sections containing the results are tables (Tables A-I-1, A-I-2 and A-I-3) summarizing the test program at each site. Each of these tables is then followed by figures presenting in graphical form the test results.

A-1 IDENTIFICATION AND INDEX PROPERTIES

All samples from each site were classified visually in the laboratory, and the field classifications were amplified or modified as necessary. From this information, the boring logs in Section 4 were prepared on which are shown a generalized description of each strata encountered. The natural water content, defined as the weight of water expressed as a percentage of the weight of the dry soil, was also measured for all disturbed and undisturbed samples. The results of these visual classifications and moisture determinations are included in Tables A-I, as well as on the boring logs in Section 4 adjacent to the Standard Penetration Test results.

The grain size distribution was measured by both sieve and hydrometer analyses on select samples from each site. These tests were conducted in accordance with ASTM Designation D 422-63. Results of these tests are presented in Figs. A-2, A-15 through A-17, and A-21.

Atterberg limit tests were performed on numerous samples from Sites 1 and 3 to determine the plasticity characteristics of these cohesive soils. These tests were conducted in accordance with ASTM Designation D 424-59. The results are included in Tables A-I and plotted in Figs. A-3 and A-22. The latter two plots indicate that the soils at both Sites 1 and 3 are of low plasticity.

The unit weight and specific gravity of the soils was measured on several representative specimens from undisturbed 3-inch diameter Shelby tube samples. The specific gravity of the solids was measured in accordance with ASTM Designation D 854. The results of these tests are included in Tables A-I.

A-2 MECHANICAL PROPERTIES

a. Shear Strength

The shear strength characteristics of the clay soils at Sites 1 and 3 were determined by several triaxial compression tests, and unconfined compression tests and numerous torvane tests described below. The results of the tests are presented in Tables A-II.

Triaxial compression tests were performed on cylindrical undisturbed specimens of soil approximately 2.8 inches in diameter and 6 inches high. Each sample was extruded from a 3-inch thin wall Shelby tube, carefully trimmed at the ends, enclosed in a rubber membrane, and placed in the test chamber.

The triaxial equipment consists of a 6-inch O.D. aluminum chamber having a wall thickness of 0.25 inches. The loading piston is stainless steel and slides in a pair of ball bushings soaked with medium weight oil to minimize the friction and to provide simultaneously an impervious seal.

The specimen, mounted in the chamber, is supported at top and bottom by caps which have porous stones hydraulically connected to allow water to circulate through the specimen. This enabled the specimen to be saturated under a back pressure, if necessary. The specimens of soil from Sites 1 and 3 were saturated while the specimens from Site 2 were not.

Following sample preparation and placement in the test chamber, the desired stress and drainage conditions are set and the specimen failed under axial loading conditions.

Most of these samples were first subjected to low strain cyclic tests to determine their dynamic properties and then failed to determine the soils shear strength. Procedures for cyclic loading are presented in Section A-3.

The triaxial test has the advantage over other shear strength tests that the strength can be measured under different boundary conditions which can be selected to better represent the actual field stresses. In this project, both consolidated undrained and unconsolidated-undrained tests were performed on the clays of Sites 1 and 3. These tests are referred to as $\bar{R}$ and Q tests, respectively. In the $\bar{R}$ tests, the specimens were first consolidated at an all-around confining pressure equal to the effective overburden stress and then gradually loaded axially, without permitting any further change in volume. The pore water pressure was measured for each increment of load. In the Q tests the specimens were not consolidated under the all-around confining pressure but failed in a quick loading mode, without

drainage.

The axial load was measured by a calibrated proving ring, the axial strain by a .001-inch precision dial gage and the pore water pressure by a pressure transducer.

The results of these tests are presented in the form of stress-strain curves and Mohr stress circles in Figs. A-4 through A-7, and A-23, together with the pertinent data for each specimen and test condition.

Unconfined compression tests were performed on undisturbed specimens from Sites 1 and 3. This test followed standard procedures and employed a strain-controlled apparatus. Each specimen was tested at a height-to-diameter ratio of approximately two. Measurements of axial load and deformation are achieved using a calibrated proving ring and dial gage, respectively. The results of these tests are presented in Tables A-II.

Portable torvane tests were performed in the bottom of many of the undisturbed samples of cohesive soils in the field, and/or in the laboratory in the top and bottom, after extrusion from the thin wall Shelby tubes. The portable Torvane is a small hand operated vane shear device. The test consists of the measurement of the torque applied through a calibrated spring to a set of small radial vanes pressed to their full depth into the soil sample. A quick reasonably reliable value of the shear strength, or apparent cohesion for clay soils, is obtained by measuring the rotational shear resistance of the soil to failure. The results of these tests are not shown, but were used together with field penetration tests to establish the consistency or stiffness of the soils.

b. Compressibility

The compressibility characteristics of the clays were investigated by means of several consolidation tests performed on samples from various depths in the borings at Sites 1 and 3. The time rates of deformation and the curves of reduction of void ratio (or percent settlement) versus applied stress are measured in floating ring consolidometers, with drainage permitted from the top and base of the specimen.

The results of these tests are presented in Figs. A-8 and A-24 as curves of percent settlement versus logarithm of stress for loading and unloading. As indicated in the same figures, for the harder soils smaller diameter specimens were tested in order to achieve loads sufficiently large enough to define the virgin loading part of the curves. Saturation of each specimen was accomplished at the stress level corresponding to the approximate overburden pressure. Prior to this stress level, it was maintained in a moist environment.

A-3 DYNAMIC PROPERTIES

Dynamic properties of the soils were determined in the laboratory by means of cyclic triaxial tests. These tests were performed on undisturbed specimens of clays from Sites 1 and 3, and recompacted composite specimens of sands from Site 2. These specimens were generally the same as tested to failure under static loading conditions using previously described triaxial compression test procedures. This dynamic testing, however, was completed prior to failing these specimens.

The equipment used in the cyclic triaxial test is basically the same described for the static triaxial compression test with the following modifications.

The testing cell is made of lucite with a special stainless steel top cover equipped with a double-acting bellofram air cylinder connected to the loading piston. The piston is fixed to the cap of the specimen in order to permit the application of upward and downward axial forces to the specimen.

The specimens are cyclically loaded as shown schematically in Fig. A-1. The reversal of shear stresses is obtained by maintaining the confining stress constant while adding and subtracting an axial increment of stress σ_d . In this manner, the stress conditions on a 45-degree plane are a normal stress $\sigma = \sigma_3 = \text{constant}$ and a shear stress, which cyclically reverses between $\tau = + \frac{\sigma_d}{2}$ and $- \frac{\sigma_d}{2}$. The normal stress is always kept positive.

The specimens of Sites 1 and 3 were cyclically loaded by manual operation of the air supply valve, connected to the double-acting bellofram cylinder, at a rate variable between 1 cycle per 8 seconds and 1 cycle per 14 seconds, depending on the strain level. For the recompacted specimens of soil from Site 2 the manual operation was replaced by an air operated single solenoid valve working at a rate of 1 cycle per second.

The axial strains were measured with a linear variable differential transducer, (LVDT), the loads by a pressure transducer connected to the air cylinder and the pore water pressure by a pressure transducer. These three measuring instruments were connected to two Sanborn 321 dual-channel carrier amplifier-recorders.

The preparation of the clay specimen is the same as in the static triaxial tests, including saturation under back pressure and consolidation under an all-around confining stress σ_3 .

The remolded sand specimens were compacted in eight equal

STAGES a) CONSOLIDATION

b) COMPRESSION AND SHEAR

c) TENSION AND SHEAR

FIGURE A-1 CYCLIC TRIAXIAL TEST - $\bar{R}_{cy}$ — LOADING CONDITIONS

layers to a water content of about 10 percent and a wet unit weight of 102.5 to 103.7, pcf. These values were established to simulate the in situ degrees of saturation and very dense compaction of the natural soils.

The results of the tests, as well as the test conditions and specimen data, are presented in the Figs. A-9 through A-14, A-18 through A-20, and A-25 through A-28.

A-4 TEST RESULTS FOR SITE NO. 1

The results of laboratory testing on soil specimen obtained from Site No. 1 are presented in the following tables and figures. These soils, as indicated from the boring log (Fig. 4-1), are chiefly silty clays with a general consistency of hard. Locally sand and gravel layers and seams were present throughout the deposit. Since cohesive soils can, in general, be adequately sampled using undisturbed procedures, most of the detailed physical testing was accomplished on these undisturbed specimens.

Table A-I-1 summarizes in detail that classification and physical testing accomplished on each of the 27 samples obtained at this site. Emphasis of this program was directed toward establishing primarily the soils' index and dynamic properties. For completeness, however, limited testing to investigate the soils' mechanical properties was also accomplished.

TABLE A-1-1 SUMMARY OF LABORATORY TESTS FOR SITE NO. 1
BORING A-1

SAMPLE NUMBER	DEPTH FEET	TYPE OF SAMPLE	NATURAL WATER CONTENT PERCENT	ATTERBERG LIMITS			OTHER TESTS	SOIL CLASSIFICATIONS
				LL	PL	PI		
S-1	8.0-10.0	Shelby	18.9					Medium to dense, light gray, silty SAND.
S-2	13.0-15.0	Shelby	24.0	47	32	15	Q_u , $\bar{R}_{cy}$, $\bar{R}$	Hard, gray-brown to gray SILT.
S-3	18.0-19.5	Shelby	21.1 O.D.	41 35	24 23	17 12	Consol.	Hard, gray-brown to gray, organic, silty CLAY.
S-4	23.0-24.5	Shelby	24.2				M.A.: 96.3% < #200	Hard, gray CLAY.
S-5	24.5-26.0	Split spoon	28.8				Q_u	Hard, light gray, silty CLAY.
S-6	28.0-30.2	Shelby	28.9	42	26	16		Hard, gray, silty CLAY.
S-7	30.2-31.7	Split spoon	30.0	47	31	16		Hard, gray SILT.
S-8	33.0-35.2	Shelby	22.3	31	22	9	$\bar{R}_{cy}$, Q	Hard, gray, silty CLAY.
S-9	35.7-36.7	Split spoon	16.9				M.A.: 68.7% < #200	Same as S-8.
S-10	38.0-39.3	Shelby	30.0	44	27	17	M.A.: 99.1% < #200	Same as S-8.
S-11	43.0-44.8	Shelby	28.2	38	36	2	$\bar{R}_{cy}$, Q	Hard, gray, clayey SILT.
S-12	48.0-49.2	Shelby	10.3-20.3				M.A.: 58.5% < #200 $G_s=2.66$	Layered, dense, gray SAND.
S-13	53.0-54.0	Shelby	15.1-22.6	31	20	11	M.A.: 58.5% < #200	Hard, sandy, silty CLAY.
S-14	54.0-55.5	Split spoon	25.5	43	22	21	Comb. Anal.	Hard, gray, silty CLAY.
S-15	58.0-59.0	Shelby	15.1-27.1				$G_s=2.68$	Dense, gray, sandy SILT to silty, fine to medium SAND.
S-16	59.0-60.5	Split spoon	18.3	36	21	15		Hard, gray, silty CLAY.
S-17	63.0-64.5	Shelby	25.1	33 38	21 22	12 16	$\gamma_d=97$ pcf $G_s=2.57$ $\bar{R}_{cy}$	Same as S-17.
S-18	64.5-66.0	Split spoon	21.3				M.A.: 5.7% < #200	Medium to very dense, gray, gravelly SAND.
S-19	68.0-69.5	Shelby	16.8-23.1					Same as S-18.
S-20	69.5-70.5	Split spoon	14.1				M.A.: 10.5% < #200	Same as S-18.
S-21	73.0-74.5	Split spoon	6.8				M.A.: 8.9% < #200	Same as S-18.
S-22	78.0-79.5	Split spoon	27.1	35	22	13		Hard, gray, silty CLAY.

TABLE A-1-1 (CONTINUED)

<u>SAMPLE NUMBER</u>	<u>DEPTH FEET</u>	<u>TYPE OF SAMPLE</u>	<u>NATURAL WATER CONTENT PERCENT</u>	<u>ATTERBERG LIMITS</u>			<u>OTHER TESTS</u>	<u>SOIL CLASSIFICATIONS</u>
				<u>LL</u>	<u>PL</u>	<u>PI</u>		
S-23	83.0-84.0	Shelby	24.9				$\bar{R}_{cy}$	Same as S-22.
S-24	88.0-90.0	Shelby	26.8	64	31	33	Q_u	Same as S-22.
S-25	93.0-93.8	Shelby	24					Same as S-22.
S-26	98.0-98.8	Shelby	24.6	52	27	25	Consol.	Same as S-22.
S-27	103.0-104.0	Shelby	24.5	53	25	28	M.A.: 98% < #200 $Q_u = 2.69,$ $G_s, \bar{R}_{cy}$	Same as S-22.

SAMPLES:

● S-2	□ S-8	◆ S-14	◐ S-24
▲ S-3	■ S-10	⊗ S-16	◑ S-26
× S-6	+ S-11	▲ S-17	◒ S-27
○ S-7	◇ S-13	△ S-22	

SITE 1
PLASTICITY CHART
BORING: A-1

TABLE A-11-1 SUMMARY OF SHEAR STRENGTHS FOR SITE NO. 1
BORING A-1

<u>SAMPLE NUMBER</u>	<u>DEPTH FEET</u>	$\frac{1}{2}(\sigma_1 - \sigma_3)_f$ <u>TSF</u>	c_f <u>PERCENT</u>	<u>TEST</u>
S-2	13.0-15.0	6.2 1.7	11.2 2.3	$\bar{R}$ Q_u
S-5	24.5-26.0	1.2	13.7	Q_u
S-8	33.0-35.2	4.8	3.1	Q
S-11	43.0-44.8	4.5	6.9	Q
S-24	88.0-90.0	3.3	1.9	Q_u
S-27	103.0-104.0	3.1 3.8	5.4 4	Q Q_u

A-14

FIGURE A-4

STRESS IN TONS/SQ. FT.

SHEAR STRESS IN TONS/SQ. FT.

TYPE OF TEST

☐ Q ☒ R ☐ S ☒ UNDISTURBED ☐ COMPACTED
☒ PRESATURATED ☐ BACK PRESSURE

TEST NO.	R-1			
BORING NO.	B-A1			
SAMPLE NO.	S-2			
CHAMBER PRESSURE, tsf	0.4			
HEIGHT, inches	5.96			
DIAMETER, inches	2.83			
UNIT WEIGHT, pcf, WET	128.4			
% COMPACTION	----			
W.C.%, BEFORE TEST	25.5			
W.C.%, AFTER TEST	26.0			
DEGREE OF SATURATION	100			
AVG. % STRAIN/min.	0.064			
BACK PRESSURE, PSI	40			

☒ STRAIN-CONTROLLED ☐ STRESS-CONTROLLED

COMPACTIVE EFFORT

—— lb. hammer —— inch drop
 —— layers —— blows/layer
 —— % compaction @ ——

VISUAL CLASSIFICATION

GRAY, CLAYEY SILTY, TRACE OF SAND

LL = 47, PL = 32, PI = 15

REMARKS:

CYCLIC LOAD TEST ALSO PERFORMED ON THIS SPECIMEN

SITE 1
 TRIAXIAL COMPRESSION TEST
 SUMMARY

TYPE OF TEST

☒ Q ☐ R ☐ S ☐ PRESATURATED ☐ BACKPRESSURE _____ psi

REMARKS: SATURATED, CYCLIC LOAD TEST ALSO
PERFORMED ON THIS SPECIMEN

σ_3 = CHAMBER PRESSURE = 1.08 tons/sq. ft.

SPECIMEN

CLASSIFICATION GRAY, SILTY CLAY, TRACE OF SAND

☐ COMPACTED ☒ UNDISTURBED

COMPACTIVE EFFORT:

SITE 1

TRIAXIAL COMPRESSION TEST

HEIGHT 6.24 inches

DIAMETER 2.85 inches

WET UNIT WEIGHT 127.8 pcf

WATER CONTENT:

BEFORE TEST 21.7 %

AFTER TEST 21.7 %

BORING: A-1

SAMPLE: S-8

DEPTH 33.0 - 35.2 FEET

TYPE OF TEST

☒ ☐ ☐ ☐ PRESATURATED ☐ BACKPRESSURE _____ psi

REMARKS: SATURATED, CYCLIC LOAD TEST ALSO
 PERFORMED ON THIS SPECIMEN

σ_3 = CHAMBER PRESSURE = 1.29 tons/sq. ft.

SPECIMEN

CLASSIFICATION GRAY, SLIGHTLY CLAYEY SILT, TRACE
 OF SAND

☐ COMPACTED ☒ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT 5.94 inches

DIAMETER 2.83 inches

UNIT WEIGHT 125.6 pcf

WATER CONTENT:

BEFORE TEST 27.5 %

AFTER TEST 27.5 %

FAILURE SKETCH

SITE 1

TRIAXIAL COMPRESSION TEST

BORING: A-1

SAMPLE: S-11

DEPTH 43.0 - 44.8 FEET

TYPE OF TEST

☒ Q ☐ R ☐ S ☐ PRESATURATED ☐ BACKPRESSURE ____ psi

REMARKS: SATURATED CYCLIC LOAD TEST ALSO PERFORMED ON THIS SPECIMEN

σ_3 = CHAMBER PRESSURE = 3.0 tons/sq. ft.

SPECIMEN

CLASSIFICATION GRAY, SILTY CLAY

☐ COMPACTED ☒ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT 6.69 inches

DIAMETER 2.87 inches

UNIT WEIGHT 127.3 pcf

WATER CONTENT:

BEFORE TEST 23.4 %

AFTER TEST 23.4 %

SITE 1

TRIAXIAL COMPRESSION TEST

BORING: A-1

SAMPLE S-27

DEPTH 103.0-103.9 FEET

BORING	SAMPLE	DEPTH	WATER CONTENT, %		HEIGHT	DIAMETER	CLASSIFICATION
		feet	Before	After	inches	inches	
A-1	S-3	18.0-19.0	19.8	19.5	0.950	2.77	GRAY, SILTY CLAY, WITH POCKETS OF FINE SAND
A-1	S-26	98.0-98.10'	25.2		0.45	1.77	GRAY, SILTY CLAY, WITH POCKETS OF FINE SAND

SITE 1

CONSOLIDATION TEST

SUMMARY

BORING A-1

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	11,760	3,920	.047
□TEST 2	5,260	1,750	.12
○TEST 3	4,170	1,390	.25
●TEST 4	2,940	980	.47
△TEST 5	2,380	790	.69

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	50,000	16,700	.011
□TEST 2	28,600	9,530	.028
○TEST 3	15,380	5,130	.105
●TEST 4	10,000	3,330	.23
△TEST 5	7,140	2,380	.46

▲TEST 1 50,000
 □TEST 2 28,600
 ○TEST 3 15,380
 ●TEST 4 10,000
 △TEST 5 7,140

SHEAR
MODULUS, PSI
 SHEAR
STRAIN %

AXIAL STRESS
PSI

20

15

10

5

0

-5

-10

-15

-20

A-20

AXIAL STRAIN %

-3

-2

-1

0

1

2

3

SPECIMEN DATA

HT 6.36 IN
 DIA 2.85 IN
 UNIT WT-WET 130.1 PCF

TEST DATA

CONFINING PRESSURE 15 PSI
 BACK PRESSURE 50 PSI
 SPECIMEN SATURATED

SITE 1
 CYCLIC LOAD TEST

SUMMARY

BORING A-1
 SAMPLE S-8
 DEPTH 33.0 - 35.3 FEET

FIGURE A-10

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	29,500	9,830	.0124
□TEST 2	15,500	5,167	.063
○TEST 3	11,360	3,786	.124
●TEST 4	6,670	2,223	.24
△TEST 5	4,020	1,340	.615

▲TEST 1 29,500 9,830 .0124
 □TEST 2 15,500 5,167 .063
 ○TEST 3 11,360 3,786 .124
 ●TEST 4 6,670 2,223 .24
 △TEST 5 4,020 1,340 .615

AXIAL STRESS
PSI

AXIAL STRAIN %

SPECIMEN DATA

HT 5.94 IN
 DIA 2.83 IN
 UNIT WT-WET 125.6 PCF

TEST DATA

CONFINING PRESSURE 18 PSI
 BACK PRESSURE 85 PSI
 SPECIMEN SATURATED

SITE 1 CYCLIC LOAD TEST

SUMMARY

BORING A-1
 SAMPLE S-11
 DEPTH 43.0 - 44.8 FEET

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	33,300	11,100	.011
□TEST 2	11,800	3,930	.057
○TEST 3	7,810	2,600	.117
●TEST 4	4,540	1,510	.24
△TEST 5	2,600	867	.48

▲TEST 1 33,300 11,100 .011
 □TEST 2 11,800 3,930 .057
 ○TEST 3 7,810 2,600 .117
 ●TEST 4 4,540 1,510 .24
 △TEST 5 2,600 867 .48

AXIAL STRESS
PSI

20

15

10

5

-5

-10

-15

-20

A-22

AXIAL STRAIN %

SPECIMEN DATA

HT 6.02 IN
 DIA 2.85 IN
 UNIT WT-WET 126.2 PCF

TEST DATA

CONFINING PRESSURE 26 PSI
 BACK PRESSURE 34 PSI
 SPECIMEN SATURATED

SITE 1
 CYCLIC LOAD TEST

SUMMARY

BORING A-1
 SAMPLE S-17
 DEPTH 63.0 - 64.5 FEET

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲ TEST 1	43,500	14,500	.015
□ TEST 2	39,000	13,000	.058
○ TEST 3	18,600	6,200	.116
● TEST 4	13,300	4,430	.232
△ TEST 5	11,300	3,770	.284

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲ TEST 1	44,600	14,900	.012
□ TEST 2	32,800	10,900	.033
○ TEST 3	23,400	7,800	.105
● TEST 4	16,500	5,500	.214

▲ TEST 1 44,600
 □ TEST 2 32,800
 ○ TEST 3 23,400
 ● TEST 4 16,500

PSI

AXIAL STRAIN %

SPECIMEN DATA

HT 6.69 IN
 DIA 2.87 IN
 UNIT WT-WET 127.3 PCF

TEST DATA

CONFINING PRESSURE 42PSI
 BACK PRESSURE 20PSI
 SPECIMEN SATURATED

SITE 1
 CYCLIC LOAD TEST

SUMMARY

BORING A-1
 SAMPLE S-27
 DEPTH 103.0 - 103.9 FEET

A-24

FIGURE A-14

A-5 TEST RESULTS FOR SITE NO. 2

The results of laboratory testing on soil specimen obtained from Site No. 2 are presented in the following tables and figures. The deposit at this site, as indicated from the boring log (Fig. 4-6) consists of a very dense fine sand for most of the 100-foot test depth. Locally, however, a few thin silt seams were encountered. Because very dense granular soils tend to loosen when sampled using undisturbed methods, good undisturbed samples were not obtained. Therefore, only limited physical testing was accomplished with emphasis on determining the dynamic shear modulus. For this testing, specimens were prepared by recompacting a composite sample from this site to a density and moisture content considered representative of the in situ condition.

Table A-I-2 summarizes in detail the testing accomplished on the 23 samples recovered at this site. In general classification testing consisted of performing mechanical analyses (gradation tests) on many of the samples. The results of these tests and the three cyclic triaxial tests are presented in the following figures (Figs. A-15 through A-20).

TABLE A-1-2 SUMMARY OF LABORATORY TESTS FOR SITE NO. 2
BORING A-2

SAMPLE NUMBER	DEPTH FEET	NATURAL WATER CONTENT PERCENT	ATTERBERG LIMITS			OTHER TESTS	SOIL CLASSIFICATION
			LL	PL	PI		
S-1	3.0-4.5	11.8	Non-Plastic			M.A.	Loose, brown, silty, fine to medium SAND.
S-2	8.0-9.5	12.3				M.A.	Medium dense, gray-brown, silty, fine to medium SAND, trace of fine gravel.
S-3	13.0-14.5	9.0				M.A.	Dense, gray-brown, slightly silty, fine SAND.
S-4	18.0-19.5						Same as S-3, except very dense.
S-5	23.0-24.5	7.2				$\gamma_d=96.2$ pcf, M.A., R_{cy}	Same as S-4.
S-6	28.0-29.5	9.0				M.A.	Very dense, gray-brown, silty, fine SAND.
S-7	33.0-34.5						Same as S-6, except with trace of fine gravel.
S-8	38.0-39.5						Same as S-6, except medium dense.
S-9	43.0-44.5					M.A.	Same as S-6.
S-10	48.0-48.5	7.0				$\gamma_d=96.9$ pcf, M.A., R_{cy}	Same as S-6.
S-11	48.5-50.0	9.0				M.A.	Same as S-6.
S-12	53.0-54.5	9.8				M.A., $G_s=2.64$	Same as S-6.
S-13	58.0-59.5						Same as S-6.
S-14	60.0-60.8	6.4				$\gamma_d=96.5$ pcf, M.A., R_{cy}	Same as S-6.
S-15	63.0-64.0	9.4				M.A.	Same as S-6.
S-16	68.0-68.8						Same as S-6.
S-17	73.0-74.5	19.9					Same as S-6.
S-18	78.5-79.2	9.4				M.A.	Same as S-6.
S-19	83.0-83.7						Same as S-6.
S-20	88.0-89.5	23.9				M.A.	Dense, gray, silty, fine SAND.
S-21	93.0-94.5	22.8				M.A.	Same as S-20, except very dense.
S-22	98.0-98.4						Same as S-21.
S-23	108.0-108.5						Same as S-21.

Note: R_{cy} tests were performed in Composite Specimens S-5, S-10 and S-14.

Note: R_{cy} samples 5, 10, and 14 taken with Shelby tubes; all others with Split Spoon.

SAMPLE NO	DEPTH-FT	U S.C.	CLASSIFICATION	NAT W C %	LL	PL	PI	SITE 2 GRAIN SIZE CLASSIFICATION BORING A-2
S-18	78.5-79.2	SM	Gray-brown, silty SAND	9.4				
S-20	88.0-89.5	SM	Gray, silty SAND	23.9				
S-21	93.0-94.0	SM	Gray, silty SAND	22.8				

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	20,900	8,040	.03
□TEST 2	16,600	6,390	.05
○TEST 3	10,800	4,150	.1

SPECIMEN DATA

HT 6.89 IN
DIA 2.98 IN
UNIT WT-DRY 103.7 PCF
SPECIMEN TESTED @ NAT.
W.C. 10%

TEST DATA

CONFINING PRESSURE 10 PSI
BACK PRESSURE, AIR 65 PSI

SITE 2
CYCLIC LOAD TEST

SUMMARY

BORING A-2
COMPOSITE SAMPLE OF S-5,
S-10, and S-14

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲ TEST 1	29,400	11,300	.02
□ TEST 2	16,500	6,350	.06
○ TEST 3	14,000	5,390	.12
● TEST 4	13,300	5,120	.18

▲ TEST 1 29,400 11,300 .02
 □ TEST 2 16,500 6,350 .06
 ○ TEST 3 14,000 5,390 .12
 ● TEST 4 13,300 5,120 .18

AXIAL STRESS
PSI

AXIAL STRAIN, %

-0.15 -0.10 -0.05 0.05 0.10 0.15

SPECIMEN DATA

HT 6.93 IN
 DIA 2.99 IN
 UNIT WT-DRY 102.5 PCF
 SPECIMEN TESTED AT NAT.
 W.C. 10%

TEST DATA

CONFINING PRESSURE 25 PSI
 BACK PRESSURE, AIR 50 PSI

SITE 2
 CYCLIC LOAD TEST

SUMMARY

BORING A-2
 COMPOSITE SAMPLE OF S-5,
 S-10 and S-14

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲TEST 1	36,600	14,090	.032
□TEST 2	20,400	7,850	.052
○TEST 3	18,300	7,040	.11

A-6 TEST RESULTS FOR SITE NO. 3

The results of laboratory testing on soil specimen obtained from Site No. 3 are presented in the following tables and figures. The boring sampled at this site (Fig. 4-11) encountered a gray medium stiff clay. In the upper 8 to 12 feet, however, this material was very stiff probably due to dissication. Because undisturbed samples of this material could be readily obtained, more extensive physical testing was possible at this site than at Site 2.

Table A-I-3 summarizes in detail that classification and physical testing accomplished on each of the 25 samples recovered at this site. This laboratory program was directed toward obtaining a good definition of the index and dynamic properties of the clays at this site. For completeness, limited testing to investigate the strength and consolidation characteristics of these materials was also accomplished.

TABLE A-1-3 SUMMARY OF LABORATORY TESTS FOR SITE NO. 3
BORING A-3

SAMPLE NUMBER	DEPTH FEET	TYPE OF SAMPLE	NATURAL WATER CONTENT PERCENT	ATTERBERG LIMITS			OTHER TESTS	SOIL CLASSIFICATIONS
				LL	PL	PI		
S-1	3.0-4.5	Split spoon	16.0				Comb. Grain Size Anal.	Hard, gray-brown, sandy, silty CLAY, trace of gravel.
S-2	5.0-6.5	Split spoon	18.2	38	21	17	Comb. Anal.	Same as S-1, except very stiff.
S-3	8.0-10.0	Shelby	18.5				$Q, \bar{R}_{cy}$ $\gamma_d = 113.7$ pcf	Same as S-2.
S-4	13.0-14.5	Split spoon	21.4	35	20	15		Same as S-2.
S-5	18.0-20.0	Shelby	17.6					Medium stiff, gray, fine sandy, silty CLAY, trace of medium to coarse sand and fine gravel.
S-6	23.0-24.5	Split spoon	17.8					Same as S-5.
S-7	28.0-30.0	Shelby	20.2				Consol.	Same as S-5.
S-8	33.0-34.5	Split spoon	21.4	33	17	16	Comb. Anal.	Same as S-5.
S-9	38.0-40.0	Shelby	23.9				$Q_u,$ $\gamma_d = 111.1$ pcf	Same as S-5.
S-10	43.0-44.5	Split spoon	24.1					Medium stiff, gray, silty CLAY, trace of fine to coarse SAND.
S-11	48.0-50.0	Shelby	21.8	37	18	19	$Q, \bar{R}_{cy}$ $\gamma_d = 106.8$ pcf	Same as S-10.
S-12	53.0-54.5	Split spoon	23.1					Same as S-10.
S-13	58.0-60.0	Shelby	23.8				Consol.	Same as S-10, except with lenses of silty, fine to medium sand.
S-14	63.0-64.5	Split spoon						Same as S-10, except layer of coarse sand and gravel at top.
S-15	68.0-70.0	Shelby	20.8				$Q, Q_u,$ $\bar{R}_{cy}$ $\gamma_d = 106.9$ pcf	Same as S-10.
S-16	73.0-74.5	Split spoon	22.8	48	17	31	Comb. Anal.	Same as S-10.
S-17	78.0-80.0	Shelby	23.2				Consol.	Same as S-10.
S-18	83.0-84.5	Split spoon	22.1					Same as S-10.
S-19	88.0-90.0	Shelby	22.5					Same as S-10.

TABLE A-1-3 (CONTINUED)

SAMPLE NUMBER	DEPTH FEET	TYPE OF SAMPLE	NATURAL WATER CONTENT PERCENT	ATTERBERG LIMITS			OTHER TESTS	SOIL CLASSIFICATIONS
				LL	PL	PI		
S-20	93.0-94.5	Split spoon	21.7					Same as S-10, except very stiff.
S-21	98.0-100.0	Shelby	23.4	36	18	18	$Q_u, \bar{R}_{cy}$ $\gamma_d=111\text{pcf}$	Same as S-10.
S-22	103.0-104.5	Split spoon	21.4					Same as S-10, except stiff.
S-23	108.0-109.5	Split spoon	23.3					Same as S-22.
S-24	113.0-114.0	Split spoon	21.7	39	16	23	Comb. Anal.	Same as S-22, except with trace of fine gravel.
S-25	118.0-119.5	Split spoon	21.2					Same as S-22.

SAMPLE NO	DEPTH-FT	U S C	CLASSIFICATION	NAT W C %	LL	PL	PI	SITE 3 GRAIN SIZE CLASSIFICATION BORING A-3
S-1	3.0-4.5		Gray-brown, sandy, silty CLAY, trace of gravel	16.0				
S-2	5.0-6.5	CL	Gray-brown, sandy, silty CLAY, trace of gravel	18.2	38	21	17	
S-8	33.0-34.5	CL	Gray, sandy, silty CLAY	21.4	33	17	16	
S-16	73.0-74.5	CL	Gray, sandy, silty CLAY, trace of gravel	22.8	48	17	31	
S-24	113-114		Gray, sandy, silty CLAY, trace of gravel	21.7

SITE 3
PLASTICITY CHART
BORING A-3

TABLE A-11-3 SUMMARY OF SHEAR STRENGTHS FOR SITE NO. 3
BORING A-3

<u>SAMPLE NUMBER</u>	<u>DEPTH FEET</u>	$\frac{1}{2}(\sigma_1 - \sigma_3)$ <u>TSF</u>	c_f <u>PERCENT</u>	<u>TEST</u>
S-3	8.0-10.0	1.6	12.1	Q
S-9	38.0-40.0	0.3	15.3	Q _u
S-11	48.0-50.0	1.0	4.8	Q
S-15	68.0-70.0	0.7	13.6	Q
		0.5	7.6	Q _u
S-21	98.0-100.0	0.1	10.4	Q _u

TYPE OF TEST

☒ Q ☐ R ☐ S ☒ UNDISTURBED ☐ COMPACTED
☐ PRESATURATED ☐ BACK PRESSURE

TEST NO.			
BORING NO.	A-3	A-3	A-3
SAMPLE NO.	S-3	S-11	S-15
CHAMBER PRESSURE, tsf	0.72	1.87	1.29
HEIGHT, inches	6.61	6.54	6.61
DIAMETER, inches	2.85	2.83	2.83
UNIT WEIGHT, pcf	135.1	129.6	134.2
% COMPACTION	---	---	---
W.C., BEFORE TEST	18.8	21.3	19.9
W.C., AFTER TEST	20.1	20.5	19.9
DEGREE OF SATURATION	---	---	---
AVG. % STRAIN/min.	0.72	0.72	0.72
BACK PRESSURE	---	---	---

☒ STRAIN-CONTROLLED ☐ STRESS-CONTROLLED

COMPACTIVE EFFORT

_____ lb. hammer _____ inch drop
 _____ layers _____ blows/layer
 _____ % compaction @ _____

VISUAL CLASSIFICATION

S-3, 8.0' - 10.0' : GRAY-BROWN, SANDY, SILTY CLAY

S-11, 48.0' - 50.0' : GRAY, SANDY, SILTY CLAY

S-15, 68.0' - 70.0' : GRAY, SANDY, SILTY CLAY

REMARKS:

SPECIMENS FROM CYCLIC LOAD TEST

SITE 3
 TRIAXIAL COMPRESSION TESTS
 SUMMARY

BORING	SAMPLE	DEPTH	WATER CONTENT, %		HEIGHT	DIAMETER	CLASSIFICATION
		feet	Before	After	inches	inches	
A-3	S-7	28.0-30.0	19.6	15.0	0.972	2.77	GRAY, CLAYEY FINE SANDY SILT
A-3	S-13	58.0-60.0	21.5	17.8	0.950	2.77	GRAY, SANDY, SILTY CLAY
A-3	S-17	78.0-80.0	23.2	18.2	0.954	2.77	GRAY, SANDY, SILTY CLAY

SITE 3

CONSOLIDATION TEST

SUMMARY

BORING A-3

	ELASTIC MODULUS, PSI	SHEAR MODULUS, PSI	SHEAR STRAIN %
▲ TEST 1	20,600	6,870	.0225
□ TEST 2	12,800	4,270	.054
○ TEST 3	8,200	2,730	.143
● TEST 4	6,000	2,000	.278

▲ TEST 1 20,600 6,870 .0225
 □ TEST 2 12,800 4,270 .054
 ○ TEST 3 8,200 2,730 .143
 ● TEST 4 6,000 2,000 .278

AXIAL STRESS
PSI

AXIAL STRAIN %

SPECIMEN DATA

HT 6.61 IN
 DIA 2.85 IN
 UNIT WT-WET 135.0 PCI

TEST DATA

CONFINING PRESSURE 10 PSI
 BACK PRESSURE 70 PSI
 SPECIMEN SATURATED

SITE 3
 CYCLIC LOAD TEST

SUMMARY

BORING A-3
 SAMPLE S-3
 DEPTH 8.0 - 10.0 FEET

AXIAL STRESS
PSI

ELASTIC
MODULUS, PSI

SHEAR
MODULUS, PSI

SHEAR
STRAIN, %

▲ TEST 1	26,200	8,730	.045
□ TEST 2	10,500	3,500	.168
○ TEST 3	6,200	2,070	.323

20

15

10

5

0

-5

-10

-15

-20

AXIAL STRAIN %

-.3

-.2

-.1

.1

.2

.3

SPECIMEN DATA

HT	6.54 IN
DIA	2.84 IN
UNIT WT-WET	129.6 PCF

TEST DATA

CONFINING PRESSURE	26 PSI
BACK PRESSURE	50 PSI
SPECIMEN SATURATED	

SITE 3
CYCLIC LOAD TEST

SUMMARY

BORING	A-3
SAMPLE	S-11
DEPTH	48.0 - 50.0 FEET

FIGURE A-27

APPENDIX B

MATHEMATICAL DEVELOPMENT OF WILSON METHOD

APPENDIX B

MATHEMATICAL DEVELOPMENT OF WILSON METHOD

The Wilson method considers a single point on the wave as it propagates through the soil and away from the anchor. This is different from considering a single point on the measured time history at a sensor, since that point may not correspond to the same point on the wave as it propagates from sensor to sensor. As will be discussed, this is primarily due to attenuation of the propagating wave.

Travel time of the wave, in this method, is based on the first peak displacement or crest of the wave. The advantages of selecting this point are that (1) it is discernable, and (2) it has some physical significance as it corresponds to a point of zero shear stress (slope of the spatial variation of the waveform is zero). However, in theory, the method could be based on any point on the wave, if any other point can be identified.

To provide a mathematical development of the Wilson method, it is assumed that the displacement wave attenuates such that the envelope of peak time history responses is $D(r)$, where r is the distance from the axis of symmetry (see Fig. B-1). The propagating wave is shown in Fig. B-1 at two instants of time. The first is at time t^* , when the peak reading of the displacement time history is reached at the sensor location, r^* . At this time the waveform is tangent to the response envelope at point r^* . The second instant is $t^* + \Delta t$. At this point in time, the displacement wave has propagated such that the crest of the wave coincides with the sensor location. It is the goal of the Wilson method to determine Δt .

Vector $\vec{EC}$ of Fig. B-1 is of length $V_s \Delta t$, where V_s is the

FIGURE B-1. PROPAGATING DISPLACEMENT WAVE

shear wave velocity. During the time interval Δt , the sensor displacement changes from point B to point C. It is known that the particle velocity is zero at time t^* , since this is the maximum displacement of the sensor. Assuming the sensor to move with constant acceleration, a , over the time interval, Δt , the distance traveled in moving from B to C is $\ddagger^\dagger$

$$|\vec{BC}| = \frac{1}{2} a (\Delta t)^2 \quad (B-1)$$

Assuming that the shape of the wave can be approximated by a parabola, and knowing that the slope of the wave at r^* must equal the slope of the response envelope at r^* , one finds that the length of $\vec{AF}$ is[†]

$$|\vec{AF}| = \frac{1}{2} \frac{dD(r)}{dr} V_s \Delta t \quad (B-2)$$

Then, from trigonometry,

$$|\vec{AE}| = |\vec{BC}| + |\vec{AF}| = |\vec{EC}| \tan \theta \quad (B-3)$$

[‡] It is noted that this assumption of constant acceleration over time Δt is valid if Δt is small, i.e., if the lag of the peak wave displacement behind the peak measured displacement is small. This is reasonable for points far from the anchor but becomes questionable for points near the anchor.

[†] The derivation of Equations B-1 and B-2 is discussed in Appendix C.

This becomes

$$\frac{1}{2} a(\Delta t)^2 + \frac{1}{2} \frac{dD(r)}{dr} V_s \Delta t = \frac{dD(r)}{dr} V_s \Delta t \quad (B-4)$$

$$a\Delta t = \frac{dD(r)}{dr} V_s \quad (B-5)$$

The left hand side of Equation B-5 is the estimate of the particle velocity at the sensor when the crest of the wave arrives. Let this velocity be called v_{pa} , i.e.:

$$v_{pa} = \frac{dD(r)}{dr} V_s \quad (B-6)$$

From the velocity time history record of the sensor, one can see at what time the sensor achieves velocity v_{pa} . This determines the arrival time of the peak of the displacement wave, $t^* + \Delta t$.

An iterative procedure for determining the shear wave velocity, using Equation B-6, can be applied. The slope of the response envelope must be measured for two sensors. A trial value of V_s is used in Equation B-6 to determine v_{pa} for each of the two sensor locations. The velocity time records of the sensors are next consulted to determine the time, following the peak displacement, at which the particle velocity equals v_{pa} . This is the arrival time of the peak of the wave at the sensor

locations. The difference between these times is the travel time. Knowing the separation of the sensors, one can compute the shear wave velocity by dividing by the travel time. If the value of V_s thus obtained differs from the trial value, it can be used as a new trial value in the next iteration.

In the special case when

$$D(r) = \frac{k}{r}, \quad (B-7)$$

the slope becomes

$$\frac{dD(r)}{dr} = - \frac{D(r)}{r} \quad (B-8)$$

Thus Equation B-6 becomes

$$v_{pa} = - \frac{V_s}{r} D(r) \quad (B-9)$$

APPENDIX C

DERIVATION OF EQUATIONS B-1 AND B-2 OF APPENDIX B

APPENDIX C

DERIVATION OF EQUATIONS B-1 AND B-2 OF APPENDIX B

Equation B-1 of Appendix B is the result of the assumption that the sensor moves with constant acceleration over the time interval Δt . A more rigorous method of deriving Equation B-1 is to say that the displacement of the sensor, $d(t)$, as a function of time, can be expressed as a Taylor series expanded about time t^* (the time of peak measured displacement at the sensor):

$$d(t) = \sum_{k=0}^{\infty} c_k (t - t^*)^k \quad (C-1)$$

The coefficients can be expressed in terms of the derivatives of $d(t)$ at t^* . Thus, Equation C-1 becomes

$$d(t) = \sum_{k=0}^{\infty} \frac{1}{k!} \frac{d^k(t^*)}{dt^k} (t - t^*)^k \quad (C-2)$$

Since t^* is the time of peak displacement of the time history record,

$$\begin{aligned} d(t^*) &= d_B \\ \frac{d}{dt} d(t^*) &= 0 \\ \frac{d^2}{dt^2} d(t^*) &= a \end{aligned} \quad (C-3)$$

where d_B is the displacement of point B of Fig. B-1, and a is the acceleration of the sensor at time t^* . Using Equation C-3, Equation C-2 can be rewritten as

$$d(t) = d_B + \frac{1}{2} a(t - t^*)^2 + \sum_{k=3}^{\infty} \frac{1}{k!} \frac{d^k(t^*)}{dt^k} (t - t^*)^k \quad (C-4)$$

At time $t^* + \Delta t$, the sensor is located at point C of Fig. B-1. Thus,

$$d_c = d(t^* + \Delta t) = d_B + \frac{1}{2} a(\Delta t)^2 + 0(\Delta t^3) \quad (C-5)$$

The length of line $\vec{BC}$ of Fig. B-1 can be determined by knowing the displacements of B and C.

$$|\vec{BC}| = d_c - d_B = \frac{1}{2} a(\Delta t)^2 + 0(\Delta t^3) \quad (C-6)$$

The assumption that Δt is short enough[†] that terms of third order and higher in Δt can be neglected, reduces Equation C-6 to Equation B-1 of Appendix B.

[†]The condition that must be satisfied is

$$\Delta t \ll \frac{3 \frac{d^2 d(t^*)}{dt^2}}{\frac{d^3 d(t^*)}{dt^3}}$$

Equation B-2 of Appendix B is the result of the assumption that the shape of the propagating displacement wave is parabolic along the segment from A to B of Fig. B-1. This equation can also be derived in a more rigorous manner. Let the displacement profile at time t^* be described by the function $w(r)$ (see Fig. B-1). This function can be expressed as a Taylor series expanded about point A of Fig. B-1,

$$w(r) = \sum_{k=0}^{\infty} b_k (r - r_A)^k \quad (C-7)$$

where, since the peak of the wave travels from r_A to r^* during the time interval Δt at the velocity V_s ,

$$r_A = r^* - V_s \Delta t \quad (C-8)$$

where r^* , V_s , and Δt have been defined in Appendix B. Since point A is the crest of the wave,

$$\begin{aligned} b_0 &= w(r_A) = d_A \\ b_1 &= \frac{dw(r_A)}{dr} = 0 \end{aligned} \quad (C-9)$$

From Equations C-7, C-8, and C-9 we get

$$w(r) = d_A + \sum_{k=2}^{\infty} b_k (r - r^* + V_s \Delta t)^k \quad (C-10)$$

If the length $V_s \Delta t$ is small enough,[†] Equation C-10 can be truncated to the form

$$w(r) = d_A + b_2 (r - r^* + V_s \Delta t)^2 \quad (C-11)$$

while point B remains within the interval of convergence of Equation C-11. From Fig. B-1, it can be seen that at point B ($r = r^*$) the slope of $w(r)$ equals the slope of the response envelope $D(r)$.

$$\begin{aligned} \frac{dw(r)}{dr} &= 2b_2 (r - r^* + V_s \Delta t) \\ \frac{dw(r^*)}{dr} &= 2b_2 V_s \Delta t = \frac{dD(r^*)}{dr} \\ b_2 &= \frac{1}{2V_s \Delta t} \frac{dD(r^*)}{dr} \end{aligned} \quad (C-12)$$

From Equation C-12, Equation C-11 becomes

$$w(r) = d_A + \frac{1}{2V_s \Delta t} \frac{dD(r^*)}{dr} (r - r^* + V_s \Delta t)^2 \quad (C-13)$$

[†]The condition that must be satisfied is

$$V_s \Delta t \ll \frac{3 \frac{d^2 w(r_A)}{dr^2}}{\frac{d^3 w(r_A)}{dr^3}}$$

It is not possible to test this condition based on field measurements, since the details of the spatial variation of the wave cannot be well defined from the sensors in their present locations.

The length of line $\vec{AF}$ in Fig. B-1 can be determined from the displacements of points A and F. It can be seen from the figure that the displacement of F, d_F , is equal to the displacement of B, d_B .

$$|\vec{AF}| = d_F - d_A = d_B - d_A = w(r^*) - d_A$$

$$|\vec{AF}| = \frac{1}{2V_s \Delta t} \frac{dD(r^*)}{dr} (V_s \Delta t)^2 \quad (C-14)$$

$$|\vec{AF}| = \frac{1}{2} \frac{dD(r^*)}{dr} V_s \Delta t$$

This corresponds to Equation B-2 of Appendix B.

APPENDIX D

ANALYTICAL REPRESENTATION OF CLOSED FORM SOLUTION

APPENDIX D

ANALYTICAL REPRESENTATION OF CLOSED FORM SOLUTION

The solution of the boundary value problem illustrated in Fig. 5-2 can be expressed as the following doubly infinite eigenfunction expansion:

$$u_i(r, z, t) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} u_{inm}(r, z) F_{nm}(t) \quad (D-1)$$

(i = r, z)

$$\sigma_{ij}(r, z, t) = \sum_{n=1}^{\infty} \sum_{m=1}^{\infty} \sigma_{ijnm}(r, z) F_{nm}(t) \quad (D-2)$$

(ij = rr, \theta\theta, zz, rz)

where u_r and u_z are the radial and axial components of displacement, respectively, and σ_{rr} , $\sigma_{\theta\theta}$, σ_{zz} , σ_{rz} are the components of stress. The time function, $F_{nm}(t)$, of Equations D-1, D-2 is

$$F_{nm}(t) = -\frac{a}{\rho} \frac{\left\{ \int_{\eta=-b}^b u_{znm}(a, \eta) d\eta \right\} \left\{ \int_{\tau=0}^t S(t-\tau) \sin \omega_{nm} \tau d\tau \right\}}{\omega_{nm} \int_{\xi=a}^R \int_{\eta=-H}^H \left\{ [u_{rnm}(\xi, \eta)]^2 + [u_{znm}(\xi, \eta)]^2 \right\} \xi d\eta d\xi} \quad (D-3)$$

where a and R are the inner and outer radii of the region of Fig. 5-8b, $2H$ is the height of the region, $2b$ is the height of the anchor, ρ is the mass density of the medium, and $S(t)$ is the arbitrary, time varying shear stress applied by the anchor to the medium. The parameter ω_{nm} is an eigenvalue, i.e., the natural frequency of a given free vibration mode of the medium. The calculation of the eigenvalues will be discussed below.

The eigenfunctions of Equations D-1 and D-2, which also appear in Equation D-3, are defined as follows:

$$u_{rnm}(r,z) = \frac{\kappa}{2\mu} \cos [\kappa(z + H)] [\alpha_1 \lambda_1 J_1(\lambda_1 r) + \beta_1 \lambda_1 Y_1(\lambda_1 r) + \alpha_2 \lambda_2 J_1(\lambda_2 r) + \beta_2 \lambda_2 Y_1(\lambda_2 r)]$$

$$u_{znm}(r,z) = \frac{1}{2\mu} \sin [\kappa(z + H)] \left\{ \kappa^2 [\alpha_1 J_0(\lambda_1 r) + \beta_1 Y_0(\lambda_1 r)] + \left[\kappa^2 - 2(1 - \nu) \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) \right] [\alpha_2 J_0(\lambda_2 r) + \beta_2 Y_0(\lambda_2 r)] \right\}$$

$$\sigma_{rrnm}(r,z) = \kappa \cos [\kappa(z + H)] \left\{ \nu \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) [\alpha_1 J_0(\lambda_1 r) + \beta_1 Y_0(\lambda_1 r)] - \alpha_1 \left[\frac{\lambda_1}{r} J_1(\lambda_1 r) - \lambda_1^2 J_0(\lambda_1 r) \right] - \beta_1 \left[\frac{\lambda_1}{r} Y_1(\lambda_1 r) - \lambda_1^2 Y_0(\lambda_1 r) \right] - \alpha_2 \left[\frac{\lambda_2}{r} J_1(\lambda_2 r) - \lambda_2^2 J_0(\lambda_2 r) \right] - \beta_2 \left[\frac{\lambda_2}{r} Y_1(\lambda_2 r) - \lambda_2^2 Y_0(\lambda_2 r) \right] \right\}$$

$$\sigma_{\theta\theta nm}(r,z) = \kappa \cos [\kappa(z + H)] \left\{ \nu \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) [\alpha_1 J_0(\lambda_1 r) + \beta_1 Y_0(\lambda_1 r)] + \alpha_1 \frac{\lambda_1}{r} J_1(\lambda_1 r) + \beta_1 \frac{\lambda_1}{r} Y_1(\lambda_1 r) + \alpha_2 \frac{\lambda_2}{r} J_1(\lambda_2 r) + \beta_2 \frac{\lambda_2}{r} Y_1(\lambda_2 r) \right\}$$

$$\sigma_{zznm}(r,z) = \kappa \cos [\kappa(z + H)] \left\{ \nu \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) [\alpha_1 J_0(\lambda_1 r) + \beta_1 Y_0(\lambda_1 r)] + \kappa^2 [\alpha_1 J_0(\lambda_1 r) + \beta_1 Y_0(\lambda_1 r)] + \left[-2(1 - \nu) \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) + \kappa^2 \right] [\alpha_2 J_0(\lambda_2 r) + \beta_2 Y_0(\lambda_2 r)] \right\}$$

$$\sigma_{rznm}(r,z) = -\sin[\kappa(z+H)] \left\{ \kappa^2 \lambda_1 [\alpha_1 J_1(\lambda_1 r) + \beta_1 Y_1(\lambda_1 r)] + \lambda_2 \left[-(1-\nu) \left(\frac{\omega^2}{c_2^2} - \frac{\omega^2}{c_1^2} \right) + \kappa^2 \right] [\alpha_2 J_1(\lambda_2 r) + \beta_2 Y_1(\lambda_2 r)] \right\} \quad (D-4)$$

where μ is the shear modulus of the medium, ν is Poisson's ratio, and c_1 and c_2 are the dilatational and shear wave velocities, respectively, of the medium. The functions J_0 , Y_0 , J_1 , Y_1 are Bessel functions of order 0 and 1. The parameters κ , ω , λ_1 , and λ_2 appearing in Equation D-4 are defined as follows:

$$\begin{aligned} \kappa &= \kappa_n = \frac{n\pi}{2H} & \lambda_1 &= \sqrt{\frac{\omega_{nm}^2}{c_1^2} - \kappa_n^2} \\ \omega &= \omega_{nm} & \lambda_2 &= \sqrt{\frac{\omega_{nm}^2}{c_2^2} - \kappa_n^2} \end{aligned} \quad (D-5)$$

The eigenvalues, ω_{nm} , and the coefficients α_1 , β_1 , α_2 , β_2 which appear in Equation D-4 are obtained through satisfying the boundary conditions along the left and right sides of the region of Fig. 5-8b. This operation takes the form of the matrix equation

$$[B] \begin{Bmatrix} \alpha_1 \\ \beta_1 \\ \alpha_2 \\ \beta_2 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \quad (D-6)$$

where the matrix [B] is defined as follows:

$$[B] = \begin{Bmatrix} \left[\omega \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) + \lambda_1^2 \right] J_0(\lambda_1 a) - \frac{\lambda_1}{a} J_1(\lambda_1 a) & \left[\omega \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) + \lambda_1^2 \right] Y_0(\lambda_1 a) - \frac{\lambda_1}{a} Y_1(\lambda_1 a) & \lambda_{20}^2 J_0(\lambda_2 a) - \frac{\lambda_2}{a} J_1(\lambda_2 a) & \lambda_{20}^2 Y_0(\lambda_2 a) - \frac{\lambda_2}{a} Y_1(\lambda_2 a) \\ -\lambda_1 \kappa^2 J_1(\lambda_1 a) & -\lambda_1 \kappa^2 Y_1(\lambda_1 a) & \left[(1-\nu) \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) - \kappa^2 \right] \lambda_{21} J_1(\lambda_2 a) & \left[(1-\nu) \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) - \kappa^2 \right] \lambda_{21} Y_1(\lambda_2 a) \\ \lambda_{11} J_1(\lambda_1 R) & \lambda_{11} Y_1(\lambda_1 R) & \lambda_{21} J_1(\lambda_2 R) & \lambda_{21} Y_1(\lambda_2 R) \\ \kappa^2 J_0(\lambda_1 R) & \kappa^2 Y_0(\lambda_1 R) & \left[-2(1-\nu) \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) + \kappa^2 \right] J_0(\lambda_2 R) & \left[-2(1-\nu) \omega^2 \left(\frac{1}{c_2^2} - \frac{1}{c_1^2} \right) + \kappa^2 \right] Y_0(\lambda_2 R) \end{Bmatrix}$$

The eigenvalues, ω_{nm} , are computed as singular points of the matrix [B]; i.e., points at which

$$\det |B| = 0 \quad (D-8)$$

The coefficients $\alpha_1, \beta_1, \alpha_2, \beta_2$ are then determined, for each eigenvalue, according to the relations

$$\begin{aligned} \alpha_1 &= -\det |B_{41}| \\ \beta_1 &= \det |B_{42}| \\ \alpha_2 &= -\det |B_{43}| \\ \beta_2 &= \det |B_{44}| \end{aligned} \quad (D-9)$$

where the notation $[B_{ij}]$ stands for matrix [B] with row "i" and column "j" deleted.