Prepared for
National Science Foundation

Evaluation of Techniques for Predicting Soil Liquefaction and Verification of Field Techniques to Predict In Situ Shear Wave Velocity

July 1981

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/CEE-81080	2.	3. PB82-168444

4. Title and Subtitle
Evaluation of Techniques for Predicting Soil Liquefaction and Verification of Field Techniques to Predict In Situ Shear Wave Velocity

5. Report Date July 1981

6. 063227

7. Author(s)
J.S. Chak, C.M. McRae, K. Arulanandan, A.D. Bailey

8. Performing Organization Rept. No. 78-22

9. Performing Organization Name and Address
J.H. Kleinfelder & Associates
1901 Olympic Blvd., Suite 300
Walnut Creek, CA 94596

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C) PFR7918559
(G)

12. Sponsoring Organization Name and Address
W. Hakala, CEE
Directorate for Engineering (ENG)
National Science Foundation
Washington, DC 20550

13. Type of Report & Period Covered

14.

15. Supplementary Notes
Submitted by: Communications Program (OPRM)
National Science Foundation
Washington, DC 20550

16. Abstract (Limit: 200 words)
The liquefaction potential for a site near the San Andreas fault was evaluated by the standard penetration test and analytical, strain potential, and electrical methods. Each method examined influences on the liquefaction characteristics of a saturated sand deposit including density, grain structure, and length of time the sand was subjected to sustained pressure. The standard penetration test procedure indicated that a potential for liquefaction existed in the upper 20 feet of the soil strata; below 20 feet, the site was less likely to liquefy. Results derived by the other three methods indicated a high potential for liquefaction for the total depth explored. Although the degree of liquefaction varied according to the method used, overall results indicated that the sight will liquefy during an earthquake similar to the 1906 San Francisco earthquake. The report also examines the relationship between seismic shear wave velocity and structure index, one of the parameters derived from the electrical probe measurements.

17. Document Analysis a. Descriptors
California
Earthquakes
Secondary waves
Soil structure
Field tests
Soils
Liquefaction

b. Identifiers/Open-Ended Terms
Environmental effects
Ground motion
San Andreas fault
Shear waves

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS	20. Security Class (This Page)	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77)
(Formerly NTIS-35)
Department of Commerce

Evaluation of Techniques for Predicting Soil Liquefaction and Verification of Field Techniques to Predict In Situ Shear Wave Velocity

for

National Science Foundation
Solicitation No. 78-22
Grant No. PFR 7918559

by

Javed S. Chak (1)
Cyril M. McRae (2)
K. Arulanandan (3)
Allen D. Bailey (4)

(1) Senior Engineer - J. H. Kleinfelder & Associates
(2) Principal Engineer - J. H. Kleinfelder & Associates
(3) Professor of Civil Engineering - Univ. of California Davis
(4) President - Bailey Scientific

ABSTRACT

The liquefaction potential of saturated sands has become a critical item to be evaluated for all project sites within seismic areas. Although methods are available for estimating and predicting liquefaction potential for specific sites, these methods lack a quantitative procedure of taking into account the effects of field soil structure on liquefaction potential. The need for applied research to verify and compare new and promising prediction methods, with established methods under actual field conditions, has prompted this study.

This report describes the evaluation of liquefaction potential for a specific site by four methods. The evaluation is conducted on a site near the San Andreas fault, where evidence was observed indicating liquefaction during the 1906 San Francisco earthquake. The study uses various methods to obtain field data, Standard Penetration Tests, electrical probe and seismic surveys.

These studies provide an opportunity to check the validity of liquefaction prediction methods against reported observation of liquefaction during the 1906 earthquake.

The results of the liquefaction evaluation by the four methods indicate the site will liquefy during an earthquake similar to the 1906 event. The degree of liquefaction and consequent level of damage which might be expected appears to vary considerably as noted in the report.

A second part of the study examines the relationship between seismic shear wave velocity and Structure Index, one of the parameters derived from the electrical probe measurements.

Relationships developed by this study between seismic shear wave velocity and electrical probe measurements are encouraging. However, additional data should be collected from other sites to verify the basic correlations obtained.

ACKNOWLEDGEMENTS

Appreciation is gratefully extended to the many associates and colleagues who have assisted in the preparation of this report through informal discussion and reviews. Special appreciation is extended to the National Science Foundation and to J. H. Kleinfelder & Associates who provided financial support for the effort.

We wish to also acknowledge the extended effort of Dr. Kandiah Arulanandan and Mr. Allen D. Bailey, consultants to the project, who provided advice and guidance in their specialies, and in the preparation of the text.

TABLE OF CONTENTS

TABLE OF CONTENTS
(cont'd)

1.0 Introduction

1.1 General

Since the catastrophic failures due to soil liquefaction in Alaska (1964) and Nigata (1964), the liquefaction potential of saturated granular soils has become an important item to be evaluated for all project sites within seismically active areas. Landslides (Seed 1968), lateral movements of bridge supports (Ross, Seed and Migliaccio 1969), settling and tilting of bridges (Ohsaki 1969), and failure of waterfront structures have resulted from liquefaction of supporting soils. The amount of catastrophic failure attributed to liquefaction has prompted considerable effort by researchers to develop methods of evaluating the liquefaction potential of saturated granular soil deposits.

Liquefaction may be described as a phenomenon by which earthquake induced cyclic stresses generate pore pressures in a cohesionless soil deposit to such a level that the soil loses its effective shear strength and undergoes large deformations. These cyclic stresses are believed to be primarily due to upward propagation of shear waves in soil deposits. The application of cyclic stress causes the structure of cohesionless soils to reduce in volume which results in transfer of stresses from grains to pore water. As a result, pore pressure increases and effective stress reduces. The mechanism can be further explained in terms of pore pressure increase in sands due to any given sequence of stress application such that the pore pressure becomes equal to the confining pressure causing total loss of soil strength. Once the cyclic stress application is ceased, a residual pore pressure equal to the overburden pressure may develop. The onset of this condition is commonly associated with surface features such as sand boils (Seed 1976).

Four methods are currently used in the analysis of liquefaction potential of level sites underlain by saturated granular soils. These methods are used as a basis for the studies described in this report. The methods are:

a. <u>Standard Penetration Test Method</u> - This method is based on observations of the performance of sand deposits in previous earthquakes. The method uses an established relationship between stress ratio induced by an earthquake and normalized blow counts from standard penetration tests (Seed 1976). A chart developed by Seed (1976) separates liquefiable and non-liquefiable zones on the basis of peak ground acceleration at a site. The liquefaction potential is determined by the use of this chart.

b. <u>Analytical Method</u> - This method is based on a comparison of the available strength in a soil specimen determined by cyclic triaxial tests to the seismically induced shear stress in a soil deposit using a simplified procedure (Seed 1976) or ground response analyses.

c. Strain Potential Method - This method is based on a threshold strain concept. The threshold strain for liquefaction, as defined by Dobrey (1980) sets the basis for predicting liquefaction potential of a soil deposit. The evaluation is based on comparing a threshold peak ground surface acceleration [$(Ap)_t$] needed to develop excess pore pressure, to the design earthquake acceleration. If the design acceleration exceeds the threshold acceleration, liquefaction potential exists.

d. Electrical Method - Uses empirical correlation between cyclic stress ratio required to cause liquefaction in the laboratory as determined by cyclic triaxial tests and electrical parameters developed in the field by in situ measurements of an electrical probe (Arulanandan and Kutter 1978).

Although these methods are currently being used for predicting the liquefaction potential for specific sites, there is a need to develop an understanding of the influence of soil fabric on the liquefaction potential results. There is also a need to compare the results of liquefaction potential studies using current prediction methods with actual field behavior to further establish their validity. This study addresses both the need to understand the relationship between soil fabric and liquefaction potential and to compare the results with field performance. The results also provide data for future reference on a specific site in an area of high seismicity which is underlain by deposits suspected of being subject to liquefaction.

1.2 Objectives

The primary objectives of this study are:

a. To evaluate and compare the liquefaction potential of a site by established, as well as newly developed procedures. The results are compared to historic observed behavior during past earthquakes.

b. To develop a laboratory correlation between shear wave velocity and soil structure parameters measured by the electrical probe.

c. To compare field measured shear wave velocity soil structure relationships to those established in the laboratory.

This report describes studies which have been conducted to meet these objectives. Descriptions of the procedures used to obtain data and the relevance of the data are presented. Conclusions regarding the liquefaction potential of the site investigated by the various methods are provided. Correlation between shear wave velocity and soil structure parameters derived from the electrical probe and the significance of such data are presented and discussed.

1.3 Background

The most common method of field soil characterization for liquefaction evaluation uses penetration resistance measurements such as the Standard Penetration Test (Seed 1976). Penetration resistance values rely on the insitu measurements of a single mechanical property that has complex boundary conditions. The measured mechanical property, penetration resistance, is related to the cyclic stress ratio required to cause liquefaction.

Recently, a new method of field soil characterization which considers the anisotropic property of soil structure has been proposed (Arulanandan and Kutter 1978). This method uses an electrical probe to measure in situ soil fabric properties which can be used as an indicator of liquefaction potential. This study examines the validity of the electrical probe as a tool in liquefaction evaluations and compares its results to the more established method of using the Standard Penetration Test for soil characterization.

Liquefaction evaluations are also performed using the two other analytical methods described. Results are compared to observed site performance during the 1906 San Francisco earthquake.

1.4 Site Description

The site chosen for study--Lawson's Landing--is located at the mouth of Tomales Bay in Marin County, California, approximately 40 miles north of San Francisco. A location map is provided as Plate 1. The site is an alluvial deposit in a marine environment. Surface deposits include sand dunes and beach sands. The site surface is relatively flat and about 5 feet above high tide level. The underlying bedrock complex is Franciscan melange of Jurassic age.

The San Andreas fault zone extends through the area as indicated on Plate 2. Several feet of lateral strike-slip movement were noted (Lawson 1908) along the fault during the 1906 San Francisco earthquake. Descriptions of "crater-like depressions" and other liquefaction oriented phenomena were noted in the Lawson report. Additional descriptions of the site area seismicity are provided in the Appendix A.

2.0 Data Acquisition

Data was developed both in the field and in the laboratory.

2.1 Field Data

Field data was obtained within a limited area of the total site. Samples were collected from a 200 foot by 150 foot area within the level portion of the site. A total of eighteen holes were drilled ranging in depth from 30 to 50 feet. The schematic locations of these holes are shown on Plate 3.

The field investigation consisted of Standard Penetration Tests (SPT), seismic surveys and electrical probe tests to determine penetration resistance, shear wave velocity and electrical properties, respectively.

A CME 55 drill rig with hollow stem augers was used to advance borings at the site. Hollow augers were used in 5 foot sections and SPT values were obtained at the bottom of each section by ASTM standard procedures. Details of the field procedures are described in Appendix A. A summary of SPT blow counts versus depth is presented on Plate 4.

Seismic shear wave velocities were measured by a downhole procedure which is described in Appendix A. Six holes were cased with PVC pipe and geophones installed to receive seismic wave signals. The wave triggering source was a hand-operated hammer situated approximately 5 to 6 feet away from the borehole. Shear wave velocities were measured in each of the cased holes at 5 to

10 foot intervals. The shear wave velocity versus depth are presented on Plate 5.

Electrical properties for the soil profile were obtained by the use of a GE100 in situ soil probe manufactured by Geoelectronics-KA Associates, Davis, California. Dr. K. Arulanandan, developer of the probe, served as a consultant to the study team during the data collection. The electrical data were generated by inserting the probe inside the hollow auger and then advancing it 8 to 12 inches beyond the bottom of the auger into the undisturbed soil. The details of this procedure are presented in Appendix A. The electrical properties were measured in four test holes at 5 foot intervals below the ground surface. Plate 6 presents the basic electrical data developed in the field.

2.2 Laboratory Tests

Laboratory tests were performed to assess physical properties, cyclic shear strength and shear wave velocity of site soils. Soil gradation, void ratio, and maximum and minimum density tests were carried out in accordance with ASTM procedures. The soil gradation data are presented on Plate 7. Void ratios, porosities, and maximum and minimum densities are presented in Table 1 on Plate 8. Laboratory tests using the electrical probe were performed on remolded samples of the site soils. Electrical soil parameters are summarized in Table 2 on Plate 8. Since a large amount of data is available for Ottawa sand, additional probe tests were performed on Ottawa 'C-109' sand for comparison purposes. Laboratory test results for this sand are presented on Tables 3 and 4 on Plate 9.

Cyclic shear tests were performed on remolded site material using an MTS cyclic loading device. The procedures used in performing these tests are in accordance with ASTM standards. A summary of cyclic shear test results is presented on Plate 10.

The procedure used for evaluating electrical parameters are described by Arulanandan and Kutter (1978). Electrical parameters, F_v, F_h and $\overline{F}$, vertical, horizontal and average formation factor respectively, are presented in Plate 11. The procedure used for obtaining shear wave velocity in the laboratory is presented in Appendix B. The laboratory shear wave velocity data is presented in Table 5 on Plate 12.

3.0 Site Characterization

Data obtained from the field investigation was used to characterize the site in terms of subsurface material composition, relative density and shear modulus.

3.1 Subsurface Profile

Soil conditions investigated in the selected area were found to be somewhat uniform. The material in the upper 10 to 20 feet is medium dense sand (20 to 35 blows/ft SPT). Below 20 feet to a depth of 40 feet, medium dense to dense sands (30 to 50 blows/ft SPT) with occasional loose pockets were encountered. Materials in the upper 40 feet are uniform fine to medium sands. Coarse sands were commonly encountered at 45 to 50 feet below the ground surface. The sands at the 45 to 50 foot depth are very dense (average SPT blow count exceeding 50). A gradation curve indicating the grain size distribution of the sand within the upper 40 feet is noted on Plate 7. Thin silt lenses and occasional pebbles were encountered in a few borings. These discontinuities are not of significance to the evaluations performed. The water table was within 2 feet of the ground surface with minimal tidal effect.

3.2 Standard Penetration Test

Penetration test data obtained from the bore holes was accumulated to describe a profile of penetration resistance versus depth as shown on Plate 4. Average values were used in the analyses.

Standard Penetration Test data are widely used in empirical and analytical interpretations to obtain estimates of soil strength, density, shear wave velocity and stress ratio required to cause liquefaction (Mitchell et al, 1978, Seed 1976, Ohsaki-Iwasaki 1973). These interpretations rely heavily on empirical correlations. A commonly used correlation converts SPT blow counts into relative density. Work by Gibbs & Holtz (1957) initially provided such a relationship. A family of curves relating penetration resistance to relative density for various overburden pressures was then developed by Bazaraa (1967), and recently Marcuson et. al. (1977) has presented relationships between relative density, overburden pressure and SPT (N) values. Bazaraa's correlation was used in this study and is included as Plate 14. An average penetration value for all field tests was used to obtain relative density of the site material using Bazaraa's correlations. The relative density profile as established from the average field SPT results is summarized on Plate 14.

3.3 Electrical Method

The response of sands and silts when subjected to alternating current has been found to be independent of the frequency of the alternating current (Arulanandan & Kutter 1978). It is possible, therefore, to characterize sands and silts by the electromagnetic theory of Maxwell (1892). A number of researchers have explored the use of electrical measurements to characterize those materials (Archie, 1942, Arulanandan & Kutter 1978).

Fabric properties routinely used to characterize sands are porosity shape, size distribution and orientation of particles. A related parameter has been established termed the formation factor ($\bar{F}$) which can be measured by electrical methods. (Archie

1942, Arulanandan & Kutter 1978). Arulanandan has also developed a probe designed to measure parameters in the field from which the formation factor may be obtained. The probe is described in Appendix A and uses electric currents to produce response measured in terms of conductivity and dielectric constant. The formation factor ($\bar{F}$) is the ratio of the conductivity of the pore solution to the conductivity of the particulate system. By measuring ($\bar{F}$) in the horizontal and vertical directions, a measure of particle size, shape and orientation can be obtained.

$$\bar{F} = (F_v F_h^2)^{1/3}$$

(Kutter, Arulanandan, Dafalias 1979)

where $\bar{F}$ = average formation factor

F_v = vertical formation factor

F_h = horizontal formation factor

Laboratory correlations between formation factor and porosity have been performed for numerous sands with varying methods of placement (Arulanandan 1978). Relative density can be computed from laboratory data based on e_{min} and e_{max} obtained from field measured formation factors.

Formation factor data and relative densities obtained from the site by the electrical probe are presented on Plate 6.

3.4 Seismic Survey

The seismic method in engineering commonly utilizes response of body waves through a solid media. A major assumption used in this method is that the waves travel through the interior of an elastic media. Two types of body waves are generated through the solid media, shear or S waves and compressional or P waves. S waves represent the propagation of shear strains which result in a linear particle motion perpendicular to the direction of wave travel. The P waves transmit compressional or dilatational strains resulting in linear particle motion in the direction of wave travel. The velocities of these two types reflect the elastic properties of the medium. For example, the shear wave velocity V_s is expressed as:

$$V_s = \left(\frac{G}{D}\right)^{1/2}$$

where G and D represent shear modulus and mass density of the material respectively. Shear modulus measured by a field seismic survey is a low strain (10^{-4} percent) modulus. It is presently used for determining soil stiffness in all low amplitude dynamic analyses. The following empircal correlation for the shear modulus of sands (Seed and Idriss 1970) was used in this study.

$$G_{max} = 1{,}000\ K_{2_{max}}\ (\sigma'_m)^{1/2}$$

where G_{max} = the maximum shear modulus obtained from field shear wave velocity measurements.

K_{2max} = a relative density dependent parameter.

σ'_m = mean effective overburden stress.

4.0 Discussion

4.1 SPT

Standard Penetration Tests (SPT) have been widely used for correlating various properties of granular soils including liquefaction potential. It has also been widely recognized that the SPT in practice is far from standard. Yet with all its weaknesses, the SPT provides readily obtainable data which, when carefully collected, can be used to estimate granular soil properties with considerable success.

Careful attention was paid to the collection of SPT data, yet as noted on Plate 4, considerable scatter was obtained between bore holes in a relatively small area. The data reflects the inherent variations in properties resulting from deposition in a marine tidal environment. It also illustrates the practical difficulty of evaluating liquefaction potential of a natural sand deposit.

The Standard Penetration Test values are influenced by properties of the sand deposit, including grain size, grain shape, and orientation, density, and stress history. The magnitude of the contribution of each of these factors which make up the soil fabric is unknown. The disturbance effect of the SPT method is also unknown. The relative density of sand deposits obtained from correlations with SPT data are dependent upon individual soil structure factors, but only the combined effect of these various factors can be identified.

4.2 Electronic Probe

The electronic probe is an instrument which measures soil formation factors in the horizontal and vertical direction. The average formation factor derived from the electrical data is a measure of soil anisotropy.

Correlations between average formation factor and porosity have been developed in the laboratory by Arulanandan and Kutter (1978), which take into account size, shape and orientation of particles as well as the density of the soil mass. This relationship is provided on Plate 11. Relative density can then be computed using laboratory determinations of e_{min} and e_{max} and the porosity values.

Research (by Mulilis 1974, Mitchell et. al., 1976) has provided sufficient evidence that sands with different orientation of particles at the same relative density exhibit substantially different liquefaction characteristics. For this reason, a parameter which provides a measure of soil anisotropy which can be related to liquefaction potential should be very valuable.

The average formation factor, $\bar{F}$, measured in the field by the electronic probe is also a measure of relative packing of sand. This recently defined index, relative packing (P_r), (Arumuli 1980), has been correlated to relative density. This correlation was derived from a number of laboratory tests by determining $\bar{F}$ along with $\bar{F}_{min}$ and $\bar{F}_{max}$ (formation factors for the loosest and densest state of sand).

Formation factor has also been correlated with such properties as relative density, friction angle, stress ratio required to cause initial liquefaction, compressibility, and permeability. (University of California, Davis 1980).

4.3 Seismic Data

The low strain ($<10^{-4}$) shear modulus, G_{max}, is commonly developed by geophysical methods. In the high amplitude analyses G_{max} is used as a reference value. In other situations, empirical equations or laboratory testing methods are used to determine G_{max}.

Empirical equations available for sands are:

$$G_{max} = 1230 \frac{(2.97-e)^2}{1 + e} \sigma'_m{}^{0.5} \qquad \text{Hardin and Black (1)}$$

$$G_{max} = 1000 K_{2_{max}} \sigma'_m \qquad \text{Seed and Idriss (2)}$$

where e = void ratio

= mean effective confining pressure
in psi (Hardin and Black)
in psf (Seed and Idriss)

$K_{2_{max}}$ = a relative density dependent parameter

Equations (1) and (2) give similar results. Seed's and Idriss' procedure for the verification of Equation (2), used in part the data developed by Hardin and Black. Equation (2) has been used for the laboratory correlation in this study.

Investigations have shown that moduli values for sands at low confining pressures, are strongly influenced by the confining pressure, the strain amplitude, the void ratio, and the angularity of the particles. They are, however, not significantly affected by grain size characteristics. Limited data is available to indicate the influence of anisotropy on G_{max}. The parameter K_{2max} is a function of porosity, shape and orientation of particles while G_{max} is a direction dependent property. K_{2max} also depends on cementation of particles as pointed out by Seed and Idriss (1970). Factors which characterize a sand deposit include porosity, shape, and orientation of particles, anisotropy and cementation. A method by which K_{2max} values can be correlated with these characteristics is needed. Such a correlation can be used to predict shear wave velocities. The correlation between K_{2max} and packing index P_r provides such a tool. Data developed at the site is plotted on Plate 15.

The results obtained by Hardin and Richart for Ottawa sand at e values of .56 and .62 with e_{max} of .71 and e_{min} of .495 and measured shear wave velocities of 800 feet per second and 700 feet per second, respectively, are plotted on the Plate 15.

The close agreement between the predicted and measured values of K_{2max} or G_{max} in this case suggest that the electrical approach has the potential to predict in situ shear wave velocity. Further studies should be undertaken to substantiate these results.

5.0 Liquefaction Potential

Primary factors which significantly influence the liquefaction characateristics of a saturated sand deposit, as described by Seed (1976) are summarized below:

- the density or relative density
- the grain structure
- the length of time the sand is subjected to sustained pressure

- the value of K_o (coefficient of lateral earth pressure)
- prior seismic or shear strain history

This study considers the influence of these important factors in evaluating liquefaction potential for the site. Four methods of evaluating liquefaction potential were applied to the site. These methods are discussed below.

5.1 SPT Method

This method is based on observations of the performance of sand deposits in previous earthquakes. First such data was compiled by the Japanese after the 1964 Nigata earthquake (Kishida 1966, Koisumi, 1966). Several areas were studied by these authors to differentiate between liquefiable and non-liquefiable conditions. Subsequently, a more comprehensive collection of site conditions at various locations where liquefaction or no liquefaction was known to have occurred was reported by Seed and Peacock (1971). These data were then used to determine the relationship between computed field values of cyclic stress ratio required to cause liquefaction and standard penetration resistance. The values of stress ratio, which are known to be associated with liquefaction and no liquefaction in the field were plotted as a function of corrected average penetration resistance N_1 (Seed 1976).

The corrected penetration resistance, as reported by Seed (1976), is obtained from the relationship:

where $N_1 = CN(N)$

$CN = 1 - 1.25 \log \frac{\sigma'_o}{\sigma'_1}$

σ'_o = Effective overburden pressure

σ'_1 = One ton per square foot

Cyclic stress ratio at any depth below the ground surface can be determined with the simplified formula (Seed et al 1971), as shown below:

$$\frac{\tau}{\sigma'_o} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} \cdot r_d$$

where a_{max} = maximum ground surface acceleration

σ'_o = effective overburden pressure

σ_o = total overburden pressure

r_d = a stress reduction factor

Seed (1976) has presented charts summarizing the field data of liquefied and non-liquefied sites versus the corrected blow count N_1. The relationships are shown on Plate 16.

The stress ratio induced in the field at the project site was calculated on the basis of a postulated 1906 San Francisco earthquake with peak ground acceleration of 0.5g. The average blow counts obtained for the site were corrected for one (1) ton per square foot overburden pressure (Seed 1976). The results ($\frac{\tau}{\sigma_o'}$ vs. N_1) are plotted on Plate 16.

These results indicated that the site will experience liquefaction in the upper 20 feet of soil deposit and marginal to no liquefaction below that depth. This conclusion is not in full agreement with the results of other methods used as discussed in the Summary and Conclusions section of this report.

5.2 Electrical Method

The electrical method is a procedure based on an empirical correlation between the cyclic stress ratio required to cause

liquefaction in the laboratory (cyclic triaxial test) and electrical parameters:

where $\bar{F}$ = the average formation factor which characterizes the packing

A = square root of $\frac{F_V}{F_H}$, an anistropy index,

F^V = vertical formation factor

F^H = horizontal formation factor,

$\bar{f}^m$ = the shape factor characterizing shape of the particles.

The value of $\bar{f}^m$ is derived from the average value of the slope of the porosity versus $\bar{F}$ relationship obtained at the maximum and minimum void ratios in the laboratory.

The probe has the basic ability to characterize soils in situ. Because of the minimal disturbance achieved in the collection of electrical data, it is considered that the method accounts for the effects of soil age and prestraining or seismic history. The curves derived during this study by laboratory triaxial tests were used to determine liquefaction potential for the site.

The curves shown on Plate 17 separate liquefiable from non-liquefiable soils. These curves were based upon the results of laboratory work which correlated electrical characteristics of the soils to the stress ratio required to cause liquefaction. Liquefaction potential evaluation for the project site by the electrical method indicate that the site has a high potential for liquefaction.

Although the method shows promise for considering significant factors that contribute to liquefaction potential, there are still some questions to be resolved. For example, it is not certain at this time if pushing of the probe causes significant disturbance to significantly affect the results. A sufficient number of field and laboratory tests are necessary to resolve this question. Currently, a laboratory program is underway at the University of California at Berkeley to determine the influence of soil disturbance due to advancement of the probe.

There is also a need to develop more field data with the probe to compare liquefaction potential derived by the probe to actual field results rather than laboratory data.

5.3 Strain Potential Method

This approach, as presented by Dobrey et al (1980), proposes a stiffness method for evaluating liquefaction potential of saturated level sand deposits. The stiffness of a sand layer is defined by a low strain shear modulus (G_{max}) which is obtained in the field by the geophysical methods. This method is based on threshold strain concept which, based on experimental evidence, shows that sands possess a threshold strain of 10^{-2}%. Cyclic strains below this value do not induce build up of pore pressure in saturated sands. The threshold strain concept as used in this approach is described below.

This method proposes a thershold acceleration, $(A_p)_t$, which is used as a basis to determine liquefaction potential of a site. The threshold acceleration is evaluated from the following formula:

$$(A_p)_t = 1.154 \times 10^{-4} \quad \frac{G_{max}}{\sigma_v \; rd}$$

where

G^{max} = Low strain shear modulus

v = Vertical stress in the soil

r^d = Flexibility coefficient defined by Seed & Idriss (1971)

The threshold acceleration is defined as the level of acceleration which develops just enough pore pressure to start liquefaction of a soil deposit. Therefore, if the design acceleration for the site exceeds the threshold acceleration, the site will be susceptible for liquefaction. Table 10 presented on Plate 18 shows that the design site acceleration (0.5g) is sufficiently above the threshold acceleration which shows that the site is susceptable to liquefaction.

5.4 Analytical Method

The analytical procedure for the evaluation of liquefaction potential was proposed originally by Seed and Idriss (1967). This method involves the following steps:

- An evaluation of the cyclic stress induced at various levels in the soil deposit by an earthquake.

- A laboratory investigation to determine cyclic strength of soil (cyclic stress ratio at a predefined failure level).

The liquefaction potential is then evaluated by comparing the cyclic stresses induced in the field to the stresses required to cause liquefaction in the laboratory sample.

The cyclic stresses induced in the field due to seismic activity can be computed by a ground response analysis or by a simplified procedure on the basis of maximum ground acceleration (Seed and Idriss 1971). The simplified procedure was used in this investigation. The induced stresses were calculated on the basis of an 8+ magnitude event with peak ground surface acceleration of 0.5g. The results are summarized on Plate 18.

Various types of laboratory test equipment and procedures have been used to investigate the cyclic stress conditions required to cause liquefaction. These include cyclic simple shear, cyclic torsional shear, shaking table, and cyclic triaxial tests.

Equipment for conducting the simple shear test is somewhat complicated, therefore, as a practical and convenient alternative, the cyclic loading triaxial test is commonly used (Seed and Lee 1966). The triaxial test does not reproduce the correct initial stress conditions in the ground as does the simple shear test. It must be performed with an initial ambient pressure condition to represent level ground conditions. The stress ratio used to express the results ($\frac{\sigma_{dc}}{2\sigma_3}$) is the raio of the maximum shear stress to the ambient pressure, rather than the ratio of shear stress on the horizontal plane to the effective overburden pressure (τ / σ'_0), as used in the cyclic simple shear test. For these reasons, the stress ratios causing liquefaction in the two types of tests will necessarily be different. Field stresses are usually related to triaxial data by the expression (Seed and Peacock 1971):

$$(\tau \ \sigma'_0)_{\text{field}} = \frac{\sigma_{dc}}{2\sigma_3} \quad \text{triaxial}$$

Values of C^r ranging from 0.57 to 0.72 have been reported (Seed, Peacock 1971, Finn et al, 1970, Castro, 1975).

The cyclic stress ratio for this study τ / σ'_0 was obtained in the laboratory at an initial liquefaction (pore pressure equal to confining pressure) or at 5 percent double amplitude strain, whichever occurred first. The laboratory test results were adjusted for field conditions by the factor $C^r = 0.60$ to account for sample disturbance, three dimensional shaking, age of deposit and other factors discussed above. The laboratory test results are presented in Table 11 on Plate 18. The factors of safety, which are the ratios of soil strength to induced stresses, are well below 1, indicating a high potential for liquefaction.

The results by this method are in general agreement with the electrical probe and strain potential methods. These methods reach the same general conclusion that under the influence of an

8+ magnitude event, the site has a high potential for liquefaction. The results also agree with the SPT method in the upper 20 feet.

6.0 Summary and Conclusions

This study was directed toward the evaluation of in situ methods as a means of characterizing the soil profile and predicting the liquefaction potential of a granular soil deposit. These methods included SPT, electrical and seismic survey. The electrical method was used to develop a correlation between packing index as derived from electrical measurements, and shear modulus of sands.

A table summarizing the results of liquefaction potential analyses by the methods used in this study is provided on Plate 19. All analyses were performed using an 8+ magnitude earthquake adjacent to the site which would produce a 0.5g acceleration at the site.

The SPT procedure indicates that a potential for liquefaction exists in the upper 20 feet of the soil strata. Below 20 feet, the site is less likely to liquefy. This conclusion differs somewhat from the results derived by the other three methods which indicate a high potential for liquefaction for the total depth explored.

One of the difficulties associated with the SPT method is the determination of representative standard penetration values. Another problem lies in the reliability of the lower bound curve used to determine liquefaction potential which is based on limited available data. Data is particularly scarce at high values of (τ / σ_0'), the range in which this study was performed. Additional data are needed to better define the liquefaction potential curves in this range. Furthermore, this method of analysis takes no account of important factors such as duration of shaking and details of pore pressure buildup and dissipation.

The electrical method indicates that the site is highly susceptible to liquefaction. The results agree well with analytical and strain potential methods. The electrical method minimizes the problems associated with sampling and sample disturbance. Important factors relating to fabric of the soil deposit are taken into account. However, there are still some difficulties to be resolved. It is not fully understood at this time if the pushing of the probe causes sufficient disturbance of the in situ material to change its basic characteristics. Also, most of the basic correlations developed by the probe are drawn from laboratory similation of field conditions. In many cases, laboratory determinations underestimate the field values of stress ratio required to cause liquefaction and provide answers that are too conservative (Peck 1979). Hence, as in the SPT approach, a large amount of field data are necessary to establish a demarcation between liquefiable and non-liquefiable areas.

The results of the strain potential method indicate that the peak ground surface acceleration from the selected ground motion are sufficiently above the threshold peak ground surface acceleration to suggest that the site is susceptible to liquefaction. The strain potential determined from the values of G_{max} obtained from the seismic survey takes into account the significant soil fabric properties which influence liquefaction characteristics.

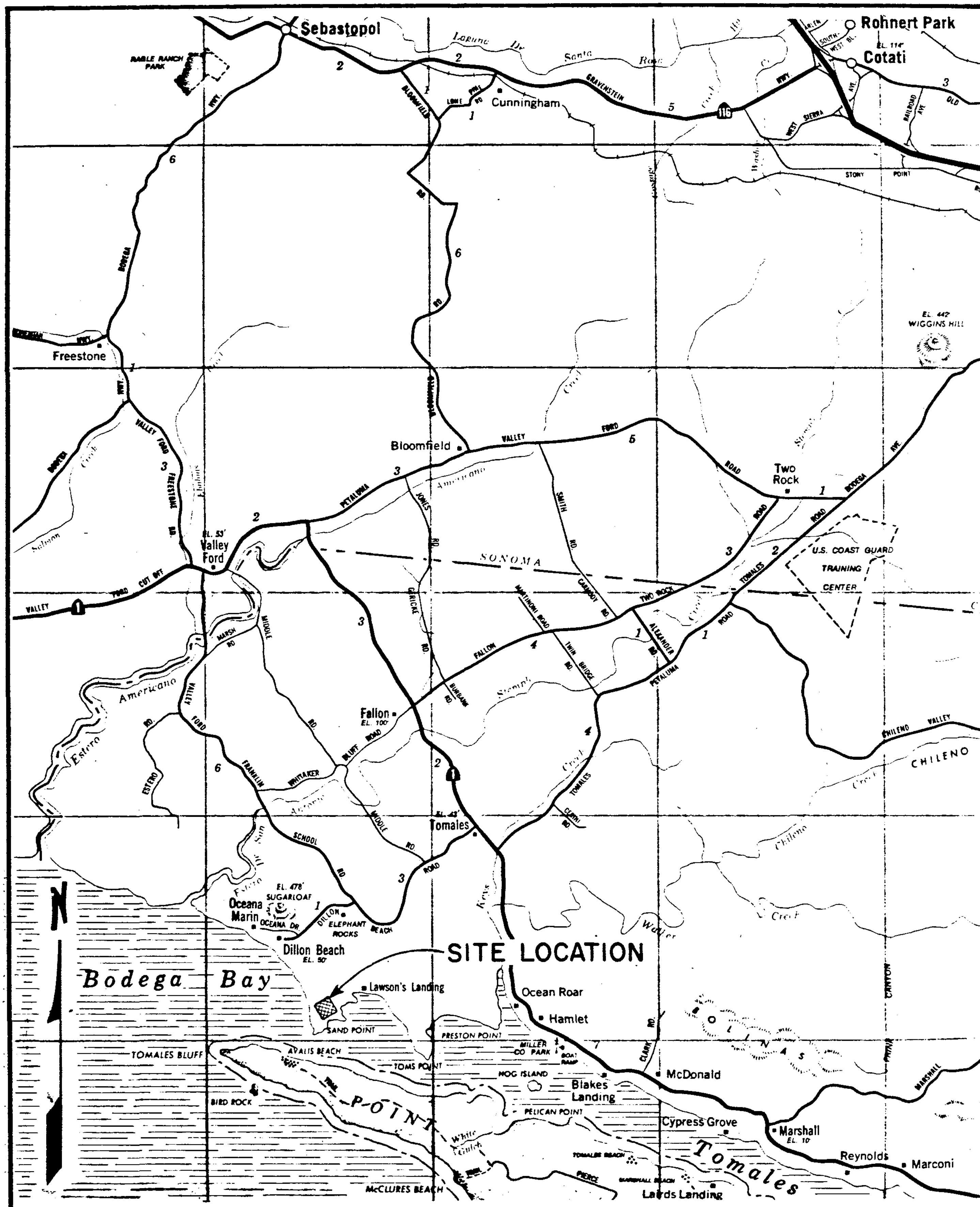
The analytical method indicates that the site is susceptible to liquefaction. This method provides an insight into the mechanism of liquefaction phenomena, as the performance of the soil deposit during an earthquake can be modeled in detail. Primary problems associated with this method stem from the difficulties and inaccuracies of laboratory testing. Many types of laboratory equipment have been used to evaluate the cyclic stress conditions required to cause liquefaction. It has been well recognized that all of these tests are subject to some degree of error due to method or equipment limitations. Another limitation of this method arises from the fact that sampling and testing procedures currently practiced cause soil disturbance and changes important soil fabric characteristics. Selection of the proper empirically derived correction factor (C_r) used to account for various field conditions to a great extent represents the engineer's best judgment and may not fully compensate for actual field conditions. The liquefaction potential expressed in the laboratory in terms of limited strain potential does not represent actual liquefaction conditions but only a limited deformation of the material. Attempts to go beyond the limited strains in the laboratory sample causes other more serious problems in the results.

The four methods discussed above provide a basis for the evaluation and prediction of liquefaction potential for level soil deposits. For this evaluation, it was found that three of the four methods indicated a high potential for liquefaction during an earthquake producing a 0.5g ground surface acceleration at the site. The fourth method SPT indicates there is only a marginal potential for liquefaction. The historic record indicates that there was significant evidence of liquefaction in the vicinity of the site during the 1906 San Francisco earthquake (magnitude 8.3). This evidence was in the form of sand boils and craters rather than flow slides, lateral spreading or other phenomena commonly associated with catastrophic liquefaction.

It is our opinion that the historic evidence would indicate that liquefaction, ranging somewhat between the marginal case indicated by the SPT method and the more disastrous case indicated by the other three methods, actually occurred. It is recognized that the present condition of the sand deposit may be considerably different than was present prior to 1906. Having collected detailed data on this site, further insight into the validity of these methods will be gained during the next major event near the site.

Additional studies of this type should be performed in areas of high seismicity to gather pre-earthquake data which can be used to further verify the methods currently being used for liquefaction evaluation. Until more data can be collected to verify the various methods, it is in order to suggest that in the case of large or sensitive projects, liquefaction potential be evaluated by two or more of these methods.

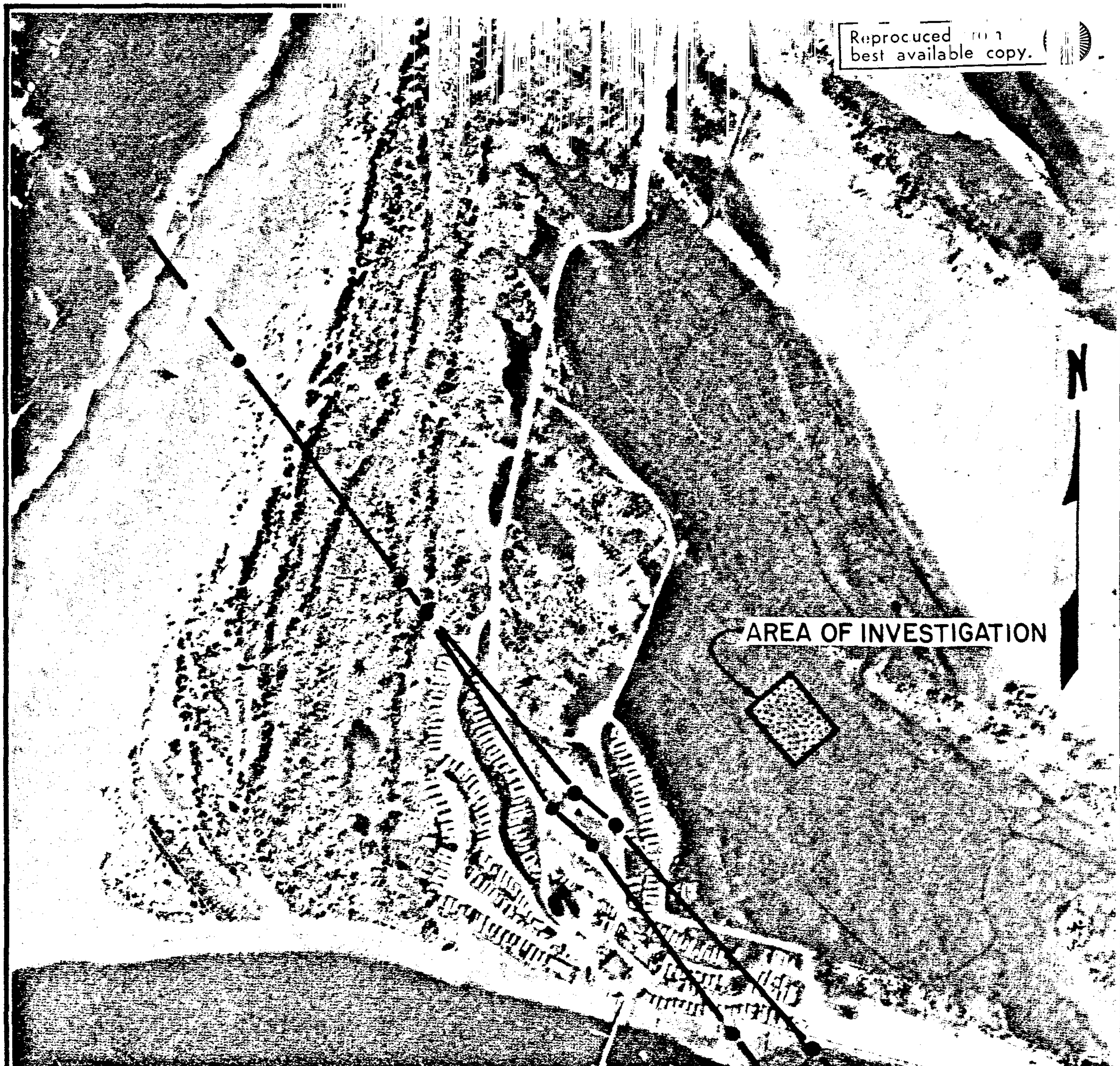
The electrical probe shows promise in providing another effective tool for in situ site evaluation. Continued development of field data using this method is suggested. The use of the electrical probe to obtain data correlatable with field shear wave velocity appears very promising. Reasonable correlations were obtained in both the field and laboratory which would justify continued studies.



SITE LOCATION PLAN

LAWSON'S LANDING

PLATE 1



EXPLANATION

San Andreas Fault

BASE: Orthophoto from
Marin County Department
of Public Works

0 200 400 800

SCALE: 1"=400'

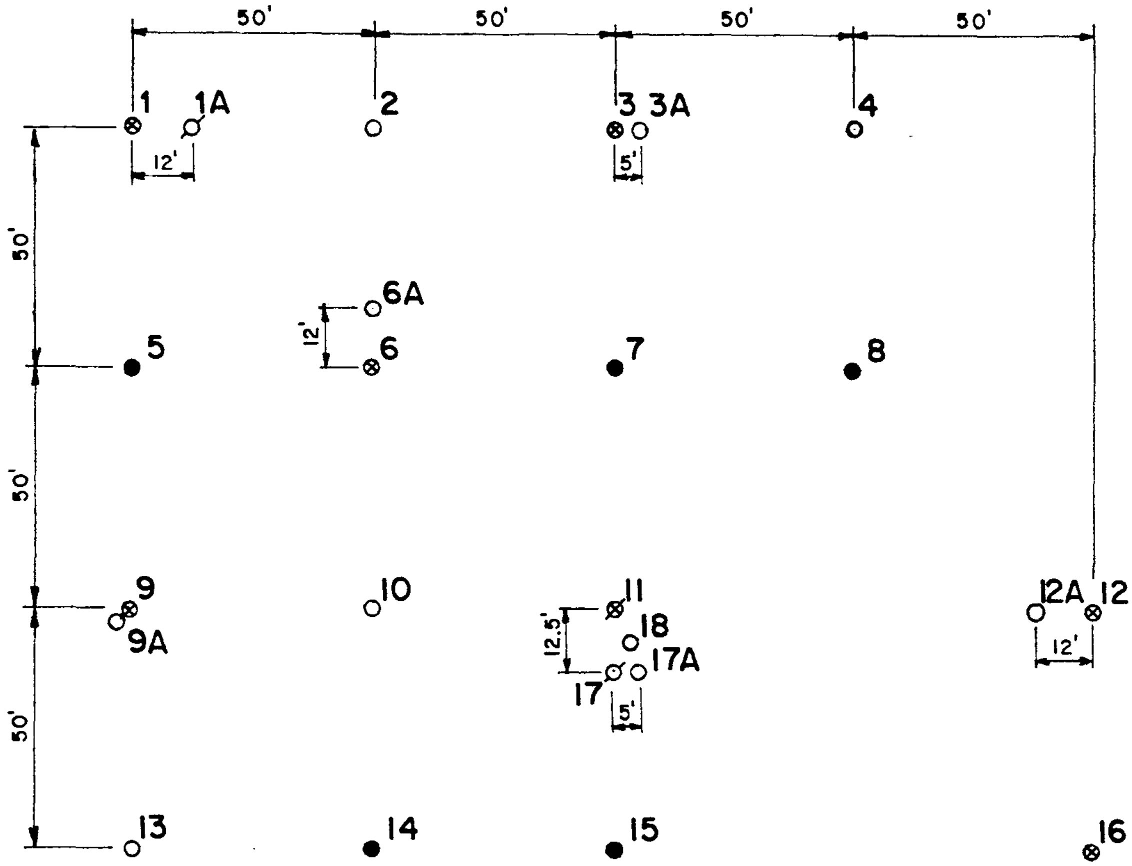
SITE PLAN

Reproduced from best available copy.

LAWSON'S LANDING

PLATE

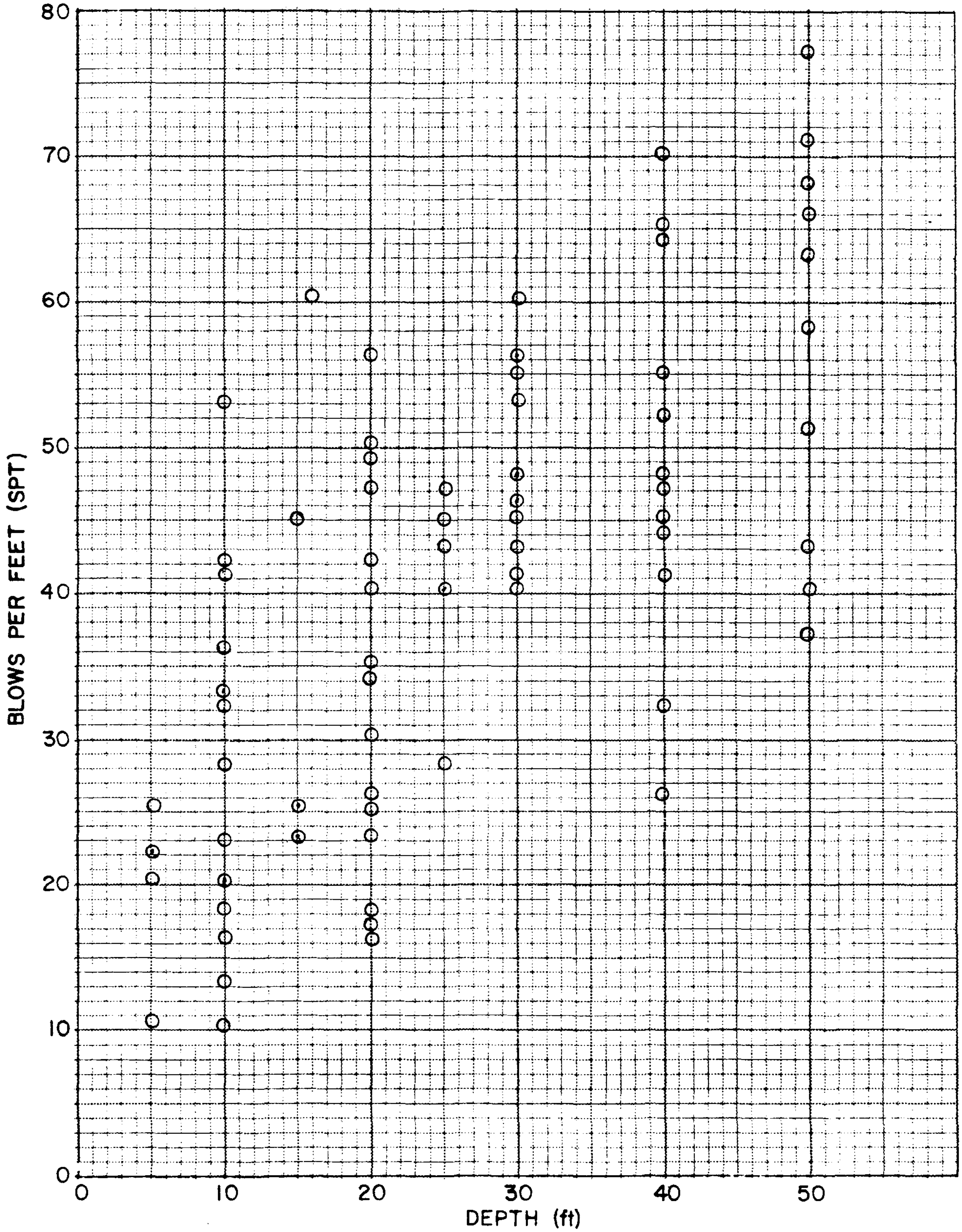
2



LEGEND

- ⊗ CASED HOLES FOR SEISMIC SURVEY
- ● NOT DRILLED
- ∅ ⊗ ELECTRICAL PROBE TESTS
- ○ SPT DATA ONLY

SCHEMATIC BORING LOCATION LAWSON'S LANDING	PLATE 3

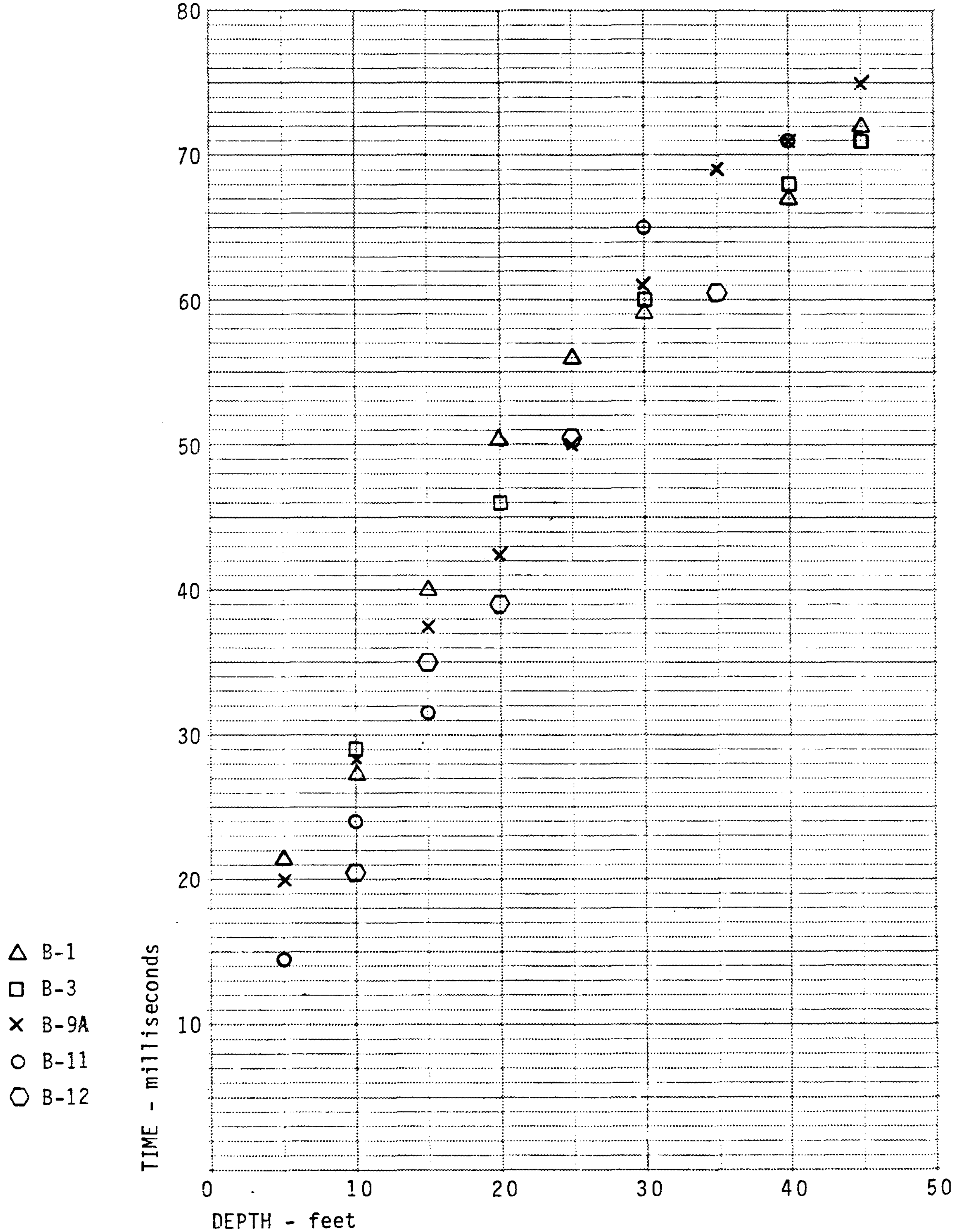


BLOW COUNT VS DEPTH

LAWSON'S LANDING

PLATE

4



PLATE

5

DOWN-HOLE SHEAR WAVE VELOCITY

LAWSON'S LANDING

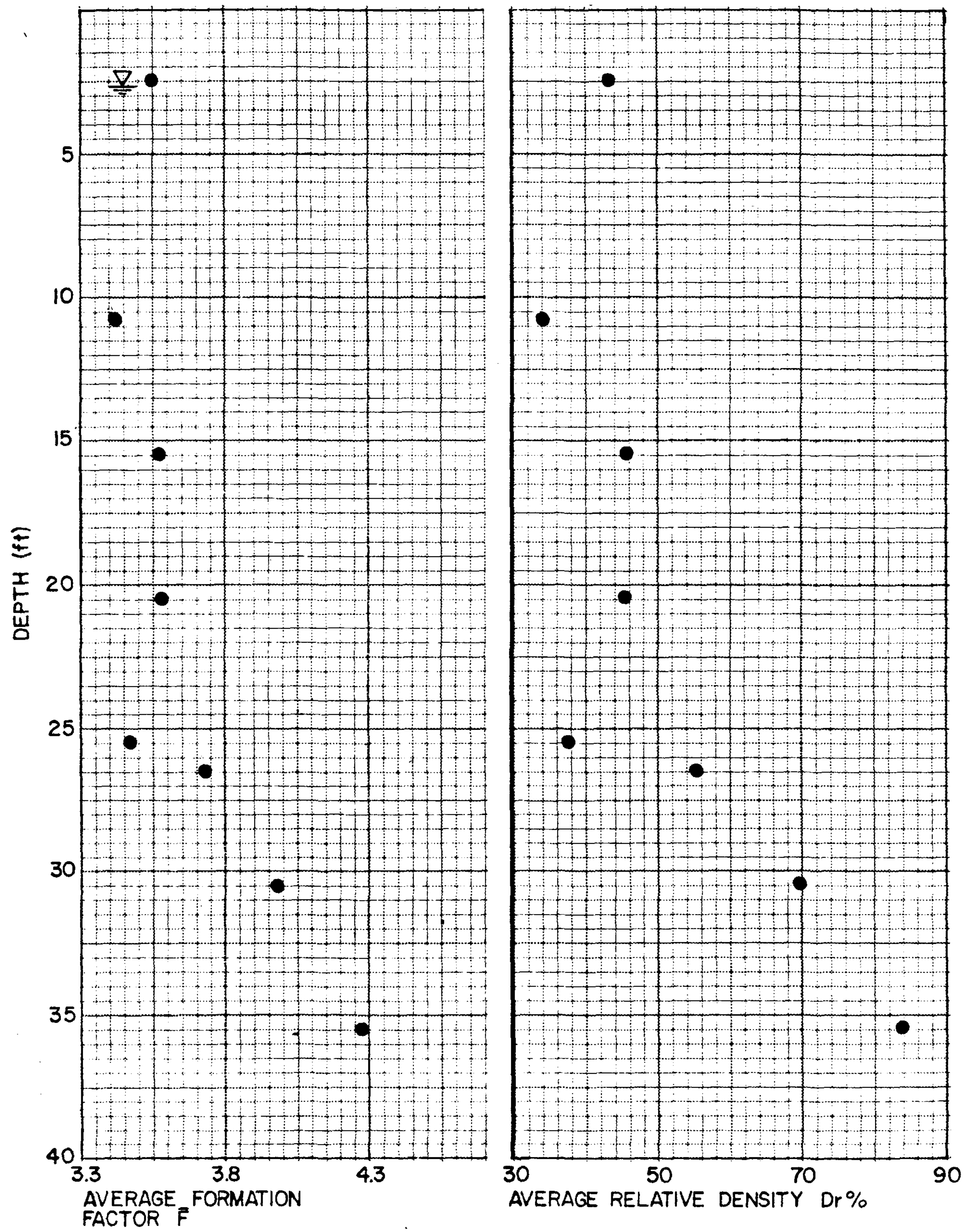
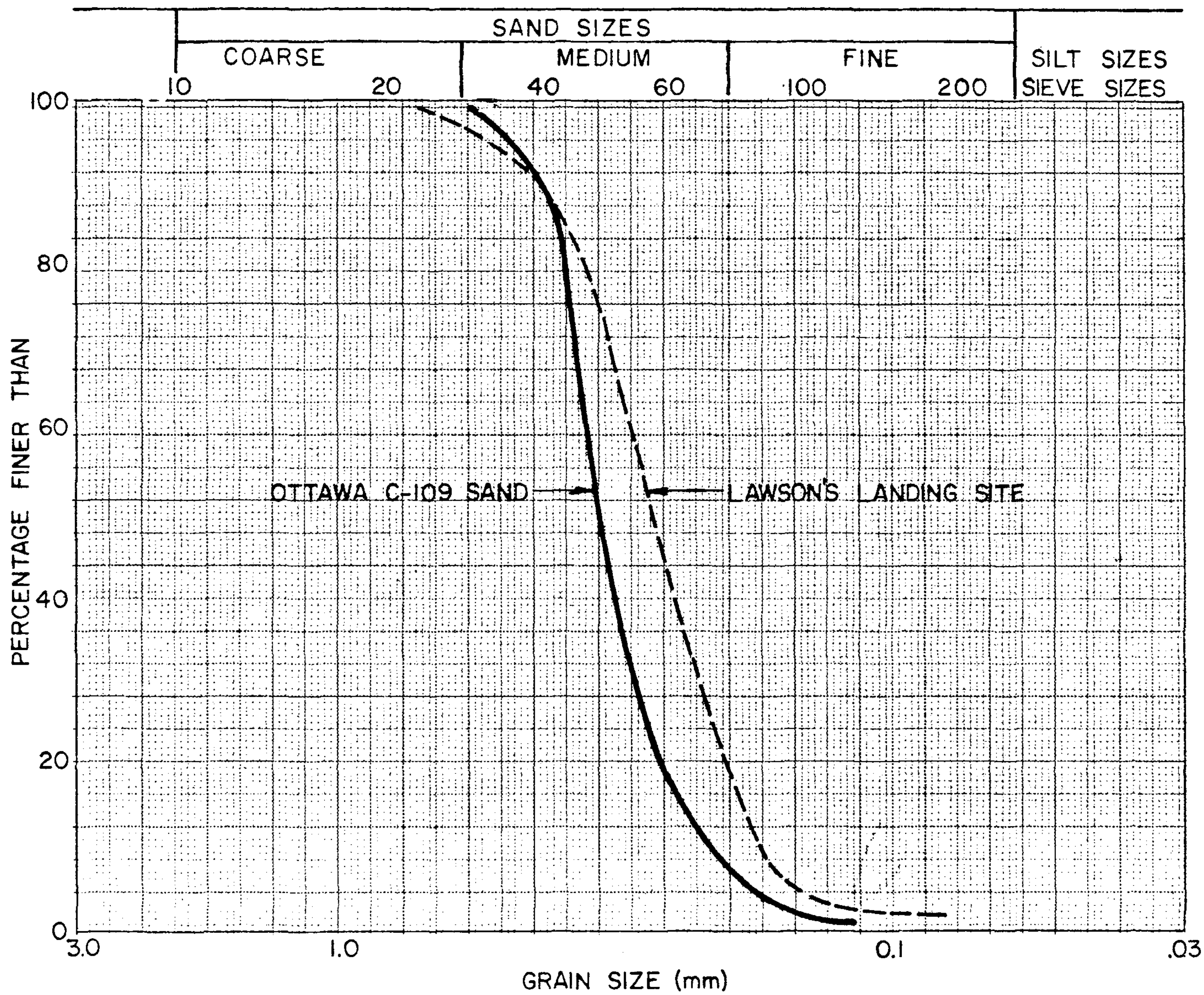


PLATE 6

DEPTH VS FORMATION FACTOR, DEPTH VS RELATIVE DENSITY

LAWSON'S LANDING



GRADATION CURVES FOR OTTAWA C-109 SAND

LAWSON'S LANDING

PLATE

7

LAWSON'S LANDING SITE SAND

Specific gravity G_s = 2.66

$D_{10} \simeq 0.2$ mm

$D_{60} \simeq 0.3$ mm

Uniformity coefficient $C_u \simeq 1.5$

TABLE 1

Maximum-minimum porosities, void ratios and densities

	Density lb/ft^3	Void Ratio e	Porosity η
Maximum	106.9	0.555	0.357
Minimum	86.8	0.916	0.478

TABLE 2

ELECTRICAL PARAMETERS

	Average Formation Factor $\overline{F}$	Average form Factor $\overline{f}$
Maximum	4.70	1.511
Minimum	3.03	1.502

Mean value of the Average form factor $\overline{f} = \dfrac{1.511 + 1.502}{2} = 1.506$

LABORATORY TEST RESULTS

LAWSON'S LANDING

PLATE 8

OTTAWA 'C-109' SAND

Specific Gravity G_s = 2.65

Gradation: chart attached

$D_{10} \simeq 0.36$ mm

$D_{60} \simeq 0.47$ mm

Uniformity coefficient $C_u \simeq 1.3$

TABLE 3

Maximum-minimum porosities, void ratios and densities

	Density lb/ft^3	Void Ratio e	Porosity η
Maximum	112.1	0.486	0.327
Minimum	88.8	0.866	0.464

TABLE 4

ELECTRICAL PARAMETERS

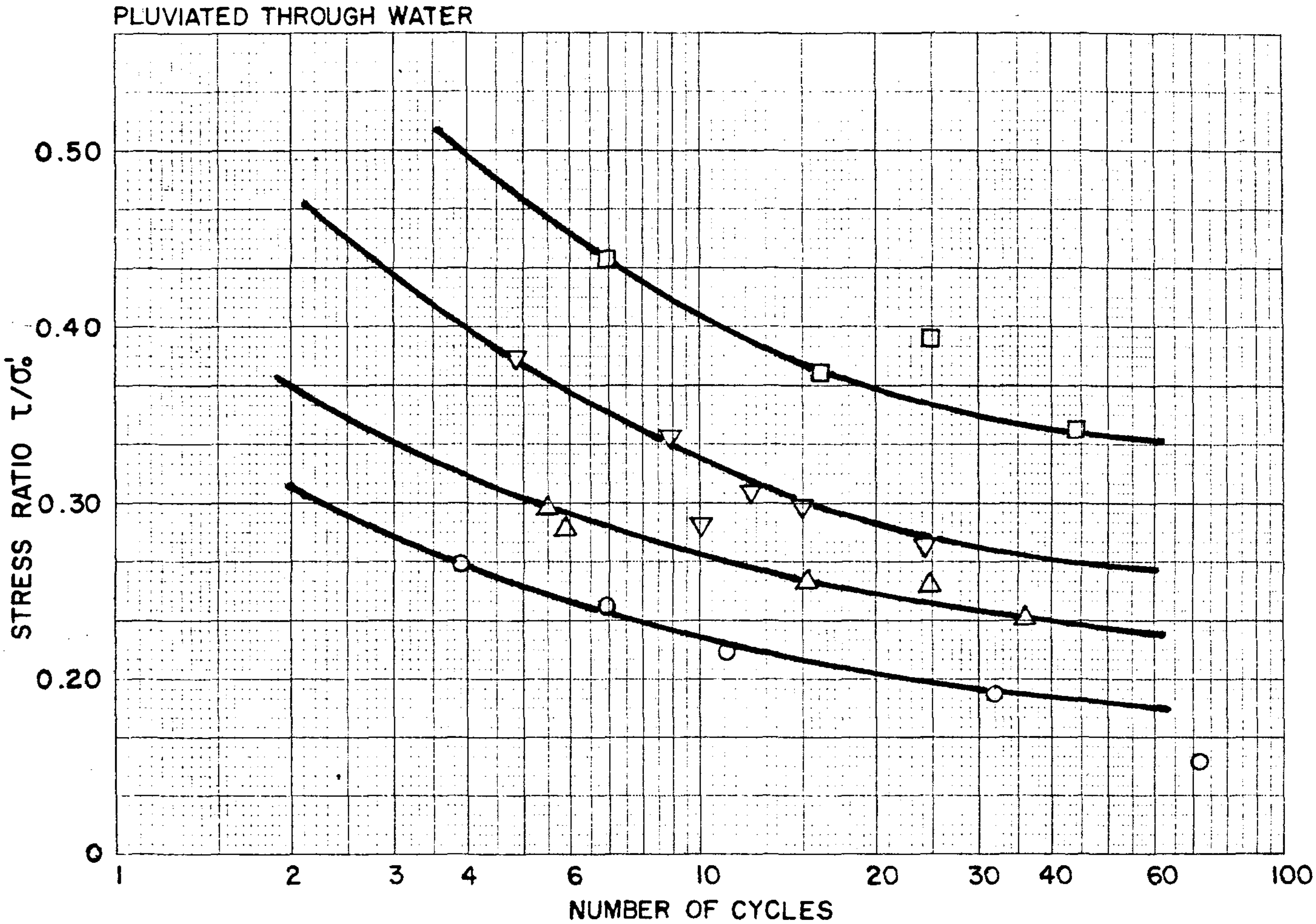
	Average Formation Factor $\overline{F}$	Average form Factor $\overline{f}$
Maximum	4.50	1.346
Minimum	2.95	1.409

Mean value of $\overline{f} = \dfrac{1.346 + 1.409}{2} = 1.378$

LABORATORY TEST RESULTS

LAWSON'S LANDING

PLATE 9



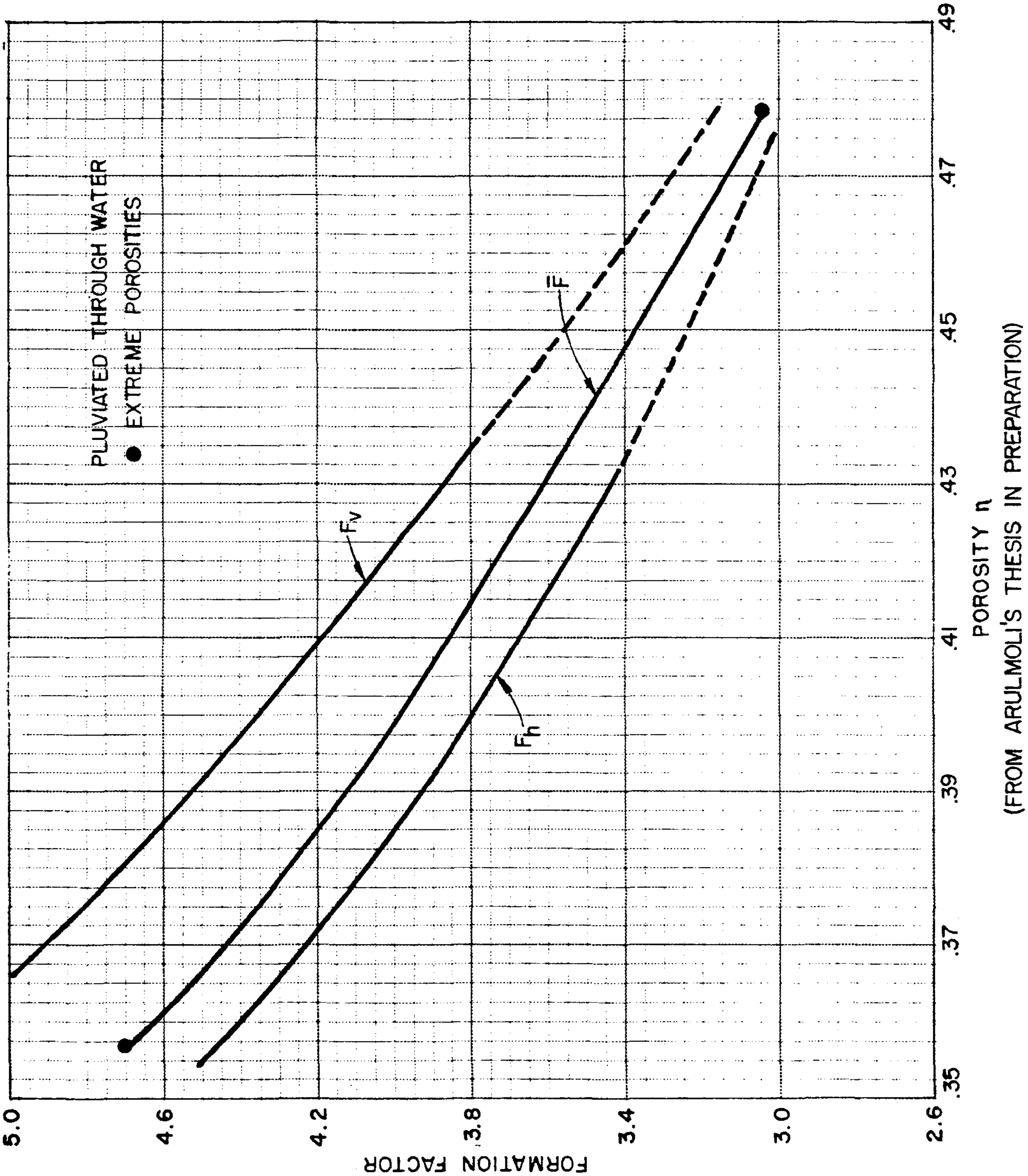
LEGEND

○ — D_r = 46%
△ — D_r = 54%
▽ — D_r = 60%
□ — D_r = 74%

STRESS RATIO VS NUMBER OF CYCLES

LAWSON'S LANDING

PLATE 10



FORMATION FACTOR VS POROSITY

LAWSON'S LANDING

PLATE 11

T A B L E 5

LABORATORY SHEAR WAVE VELOCITY TEST RESULTS:

Soil Type	Porosity n	Relative Density Dr%	Relative Packing Pr%	Shear Wave Velocity V_s(ft/s)	K_{2max}
Ottawa 'C-109' Sand	0.39 0.34	50 92	45 86	260 310	52 75
Lawson's Landing Site Sand	0.40 0.36	59 90	45 82	260 310	47 66

T A B L E 6

FIELD SHEAR WAVE VELOCITY RESULTS: (Down Hole Data)

Depth (ft)	V_s (ft/s)	K_{2max}	Relative Packing Pr%
10	400	28	22
15	480	34	33
20	560	41	33
30	720	56	55
35	840	71	75

T A B L E 7

Results from Seed & Idriss (1970)

K_{2max}	34	40	43	52	61	70
Pr%	22	27	31	45	62	83

LABORATORY TEST RESULTS

LAWSON'S LANDING

PLATE 12

TABLE 8

LABORATORY CYCLIC TEST AND ELECTRICAL TEST RESULTS:

Porosity n	Relative Density Dr%	Formation Factor $\underline{F}$	(Anisotropy Index) $A^2=F_v/F_n$	$\left(\frac{A}{F}\right)^2 \cdot \frac{1}{fm}$	0.6x Cyclic Stress Ratio to cause liquefaction	
					10 Cycles	30 Cycles
0.429	46	3.62	1.063	.0539	.135	.114
0.419	54	3.74	1.066	.0506	.162	.143
0.412	60	3.82	1.068	.0486	.195	.165
0.394	74	4.07	1.074	.0431	.243	.210

TABLE 9

Depth ft-in	Average Formation Factor $\underline{F}$	Relative Packing Pr%	Relative Density Dr%
2'-6"	3.54	31.0	43.0
10'-9"	3.41	23.0	34.0
15'-6"	3.56	32.0	45.0
20'-6"	3.56	32.0	45.0
25'-6"	3.45	25.0	37.0
26'-6"	3.71	41.0	55.0
30'-6"	3.96	56.0	69.0
35'-6"	4.26	74.0	83.0

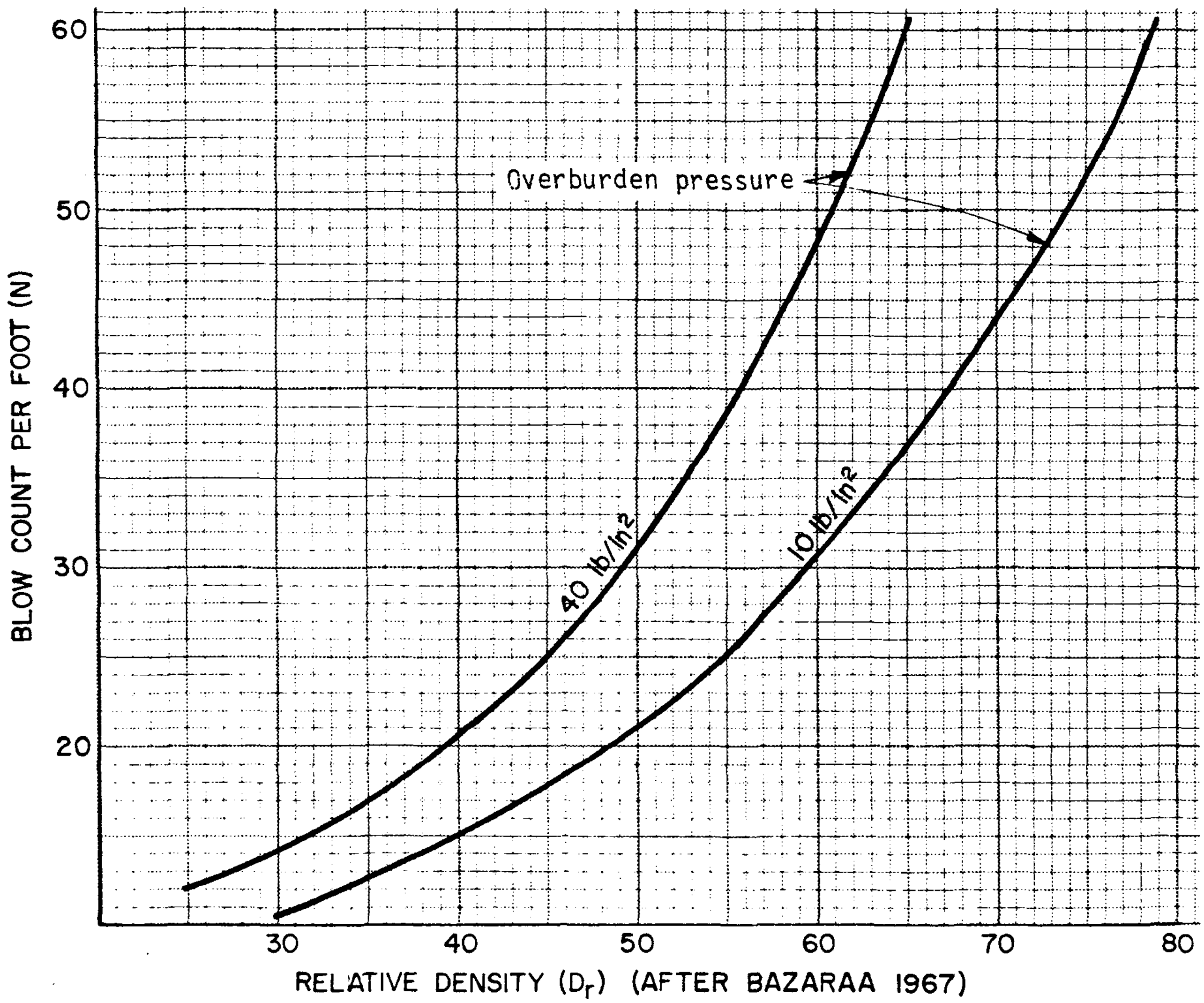
LABORATORY TEST RESULTS

LAWSON'S LANDING

PLATE 13

Depth (ft)	SPT N Values	N_1	Relative Density SPT[1]	Relative Density Electrical Probe[2]
5	20	34	52	40
10	28	45	59	35
15	31	43	62	45
20	34	41	63	45
25	40	44	67	50
30	47	47	71	70
40	51	46	72	84
50	57	46	73	--

(1) From Plate 14 (Bazarra)
(2) From Plate 6

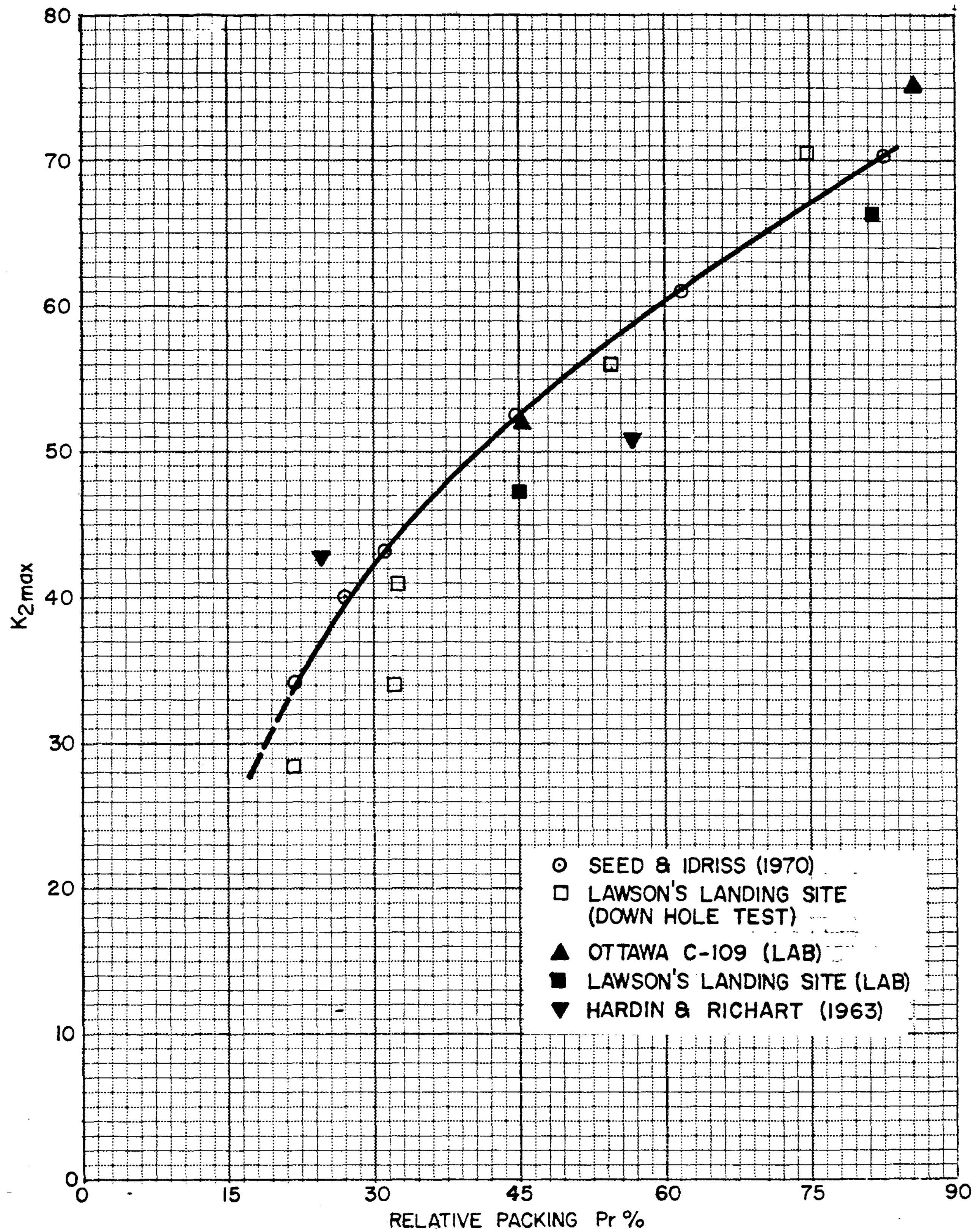


RELATIVE DENSITY VS BLOW COUNT

LAWSON'S LANDING

PLATE

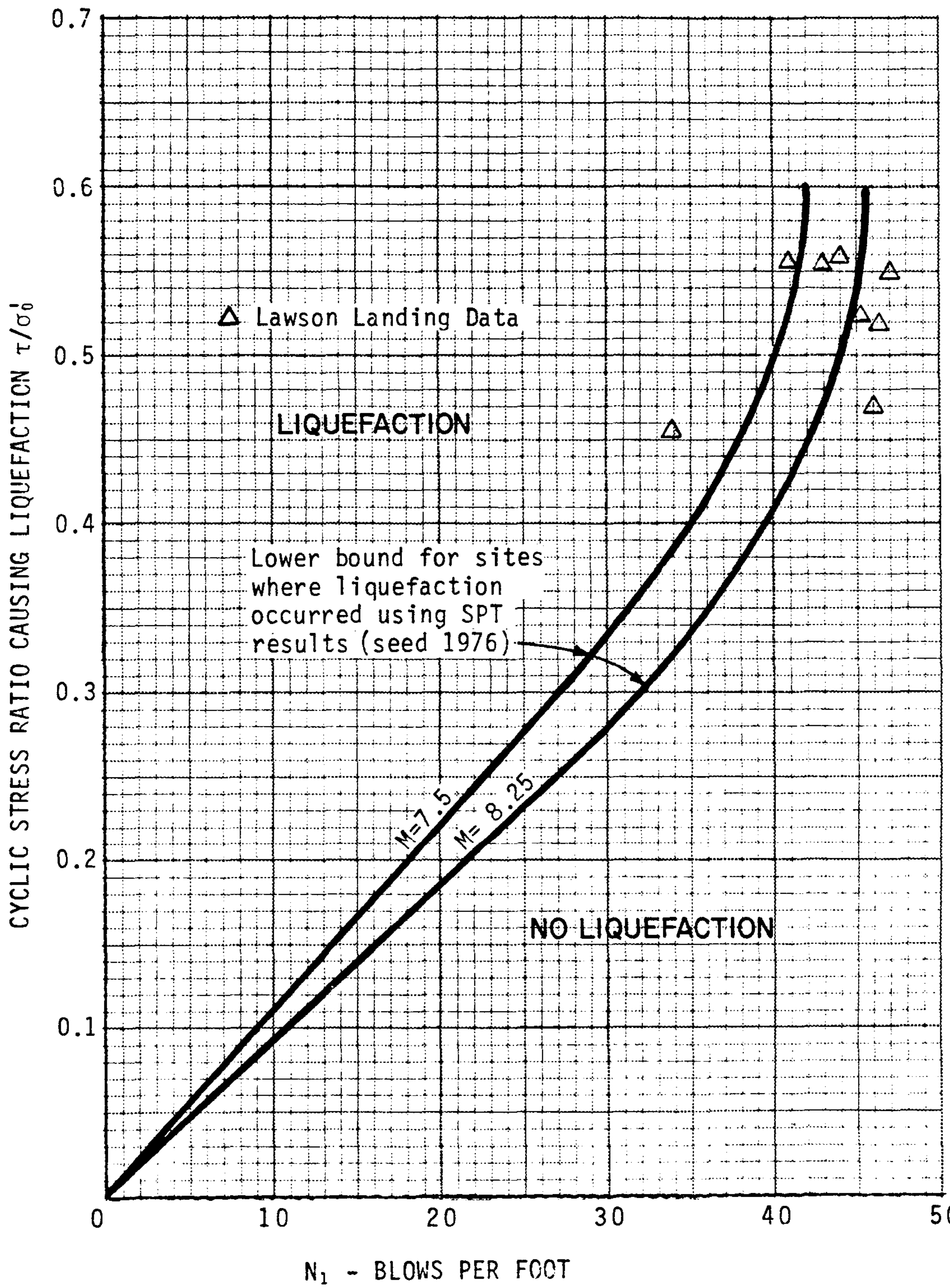
14



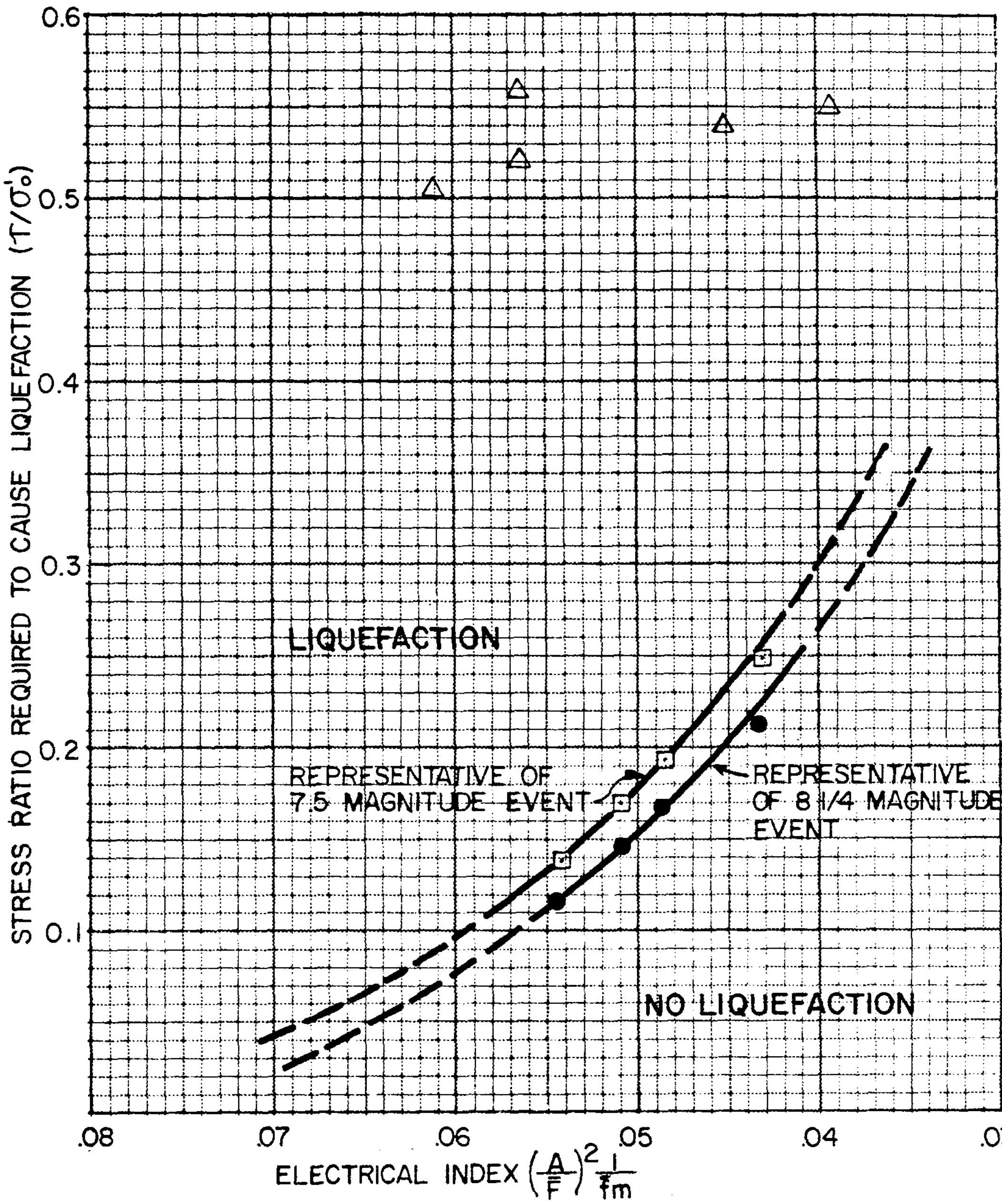
RELATIVE PACKING RELATIONSHIP FOR THREE SANDS

LAWSON'S LANDING

PLATE 15



LIQUEFACTION POTENTIAL VS CORRECTED SPT BLOW COUNT LAWSON'S LANDING	PLATE 16



LEGEND

- ⊡ CYCLIC STRENGTH (10 CYCLES)
- ● CYCLIC STRENGTH (30 CYCLES) (FIELD ADJUSTED)
- △ STRESS RATIO GENERATED DUE TO 8 1/4 M EQ (AT VARIOUS DEPTHS, LAWSON'S LANDING)

LIQUEFACTION POTENTIAL AND ELECTRICAL PARAMETERS

LAWSON'S LANDING

PLATE 17

TABLE 10

Strain Potential Method

Depth (ft)	σ_v (KSF)	G_{max} (KSF)	$(A_p)_t$
10	1.2	600	0.32
15	1.8	860	0.30
20	2.4	1170	0.31
30	3.6	1930	0.34
35	4.2	2630	0.40

TABLE 11

Analytical Method Results

Depth (ft)	Shear Stress Ratio (τ/σ_0') Induced by 8+ magnitude	Stress Ratio Required To Cause Liquefaction: Analytical Method
10	0.51	0.23
15	0.52	0.23
20	0.56	0.25
30	0.55	0.30
35	0.55	0.33

RESULTS OF LIQUEFACTION ANALYSIS

LAWSON'S LANDING

PLATE 18

TABLE 12

Comparison of results based on different procedures to predict liquefaction

Depth (ft)	Corrected Blow Count N_1	Average Formation Factor $\bar{F}$	Electrical Parameter $\left(\frac{A}{\bar{F}}\right)^2 \frac{1}{\bar{f}m}$	Shear Stress Ratio (τ/σ_0') Induced by 8+ magnitude	Stress Ratio Required To Cause Liquefaction		
					SPT Method	Electrical Method	Analytical Method
10	45	3.41	0.061	0.51	0.43	0.08	0.23
15	43	3.56	0.056	0.52	0.43	0.12	0.23
20	41	3.56	0.056	0.56	0.55	0.12	0.25
30	47	3.96	0.045	0.55	0.55	0.21	0.30
35	46	4.26	0.039	0.55	$\geq$0.8	0.28	0.33

RESULTS OF LIQUEFACTION ANALYSIS

LAWSON'S LANDING

PLATE

19

REFERENCES

1. Archie, G. E., "The Electrical Resistivity Log as an Aid in Determining Some Reservoir Characteristics", A.I.M.E., Trans. Vol. 146, pp 54-61, 1942.

2. Arulanandan, K., "Methods and Appartus for Measuring In Situ Density and Fabric of Soils", U.S. Patent, 1979- all 52 claims approved Canada Patent, 1979, pending; Japanese Patent, 1979, pending.

3. Arulanandan, K., Basu, R. and Schardin, R. J., "Significance of the Magnitude of Cielectric Dispersion in Soil Technology Highway Reserach Record, No. 426, pp. 23-32, 1973.

4. Arulanandan, K., and Dafalias, Y. F., "Significance of Formation Factor in Sand Structure Characterization", Lett. Appl. Engr. Sci., 17, 109-112, 1979.

5. Arulanandan, K., and Kutter, B., " A Directional Structure Index Related to Sand Liquefaction", Proc. ASCE, Geotechnical Engr. Div., Specialty Conf., Earthquake Engr.

6. Arulanandan, K. and Smitt, S. S., "Electrical Dispersion in Relation to Soil Structure", Proceeding of the American Society of Civil Engineers, J. of Soil Mechanics and Foundation Div., 99, No. SM12, 1113-1133, Dec. 1973.

7. Arulmoli, K., "Sand Structure Characterization for In Situ Testing", M. S. Thesis, University of California, Davis, 1980.

8. Bazaraa, A.R.S.S., "Use of the Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand", Ph.D. Thesis, University of Illinois, Urbana, 1967.

9. Castro, G., "Liquefaction of Sands", Harvard Soil Mechanics Series, No. 81, 1969.

10. Castro, G., "Liquefaction and Cyclic Mobility of Saturated Sands", Journal of the Geotechnical Engineering Div., ASCE, Vol. 101, No.. GT6, pp. 551-569.

11. Dobrey, R., Ladd, R. S., "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes and Liquefaction Potential: Science vs. Practice", Journal of Geotechnical Engineering Div., ASCE, Vol. 106, No. GT6, June, 1980, pp. 720-724.

12. Finn, W. D. L., Bransby, P. L., and Pickering, D. J., "Effect of Strain History on Liquefaction of Sands", Journal of the Soil Mechanics and Foundation Div., ASCE, Vol. 96, No. SM6, Proc. Paper, 1970.

13. Fletcher, G. F. A., "Standard Penetration Test: It's Uses and Abuses", Journal of the Soil Mechanics and Foundations Div., ASCE, Vol. 91, No. SM4, Proc. Paper 4395, July, pp 67-75.

14. Gibbs, H. J., and Holtz, W. G., "Research on Determining the Density of Sands by Spoon Penetration Testing", Proceeding Fourth International Conference on Soil Mechanics and Foundation Engineering, London, England, Vol. 1, 1957.

15. Hardin, B. O., and Black, W. L., "Vibration Modulus of Normally Consolidated Clay", ISMFD, ASCE, 94, No. SM2, pp. 353-369, March, 1968.

16. Housner, G. W., "Intensity of Earthquake Ground Shaking Near the Causative Fault", Third World Conference on Earthquake Engineering, New Zealand, pp. 94-111, 1965.

17. Kolbuszewski, J. J., "An Experimental Study of the Maximum and Minimum Porosities of Sand", Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1948.

18. Kovacs, W. D., Evans, J. C., and Griffith, A. H., "Towards a More Standardized SPT", Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 269-276, 1977.

19. Kutter, B. L., Arulanandan, K., and Dafalias, Y. F., "A Comparison of Electrical and Penetration Methods of Site Investigation", Proceedings of the Eleventh Annual Offshore Technological Conference, Houston, Texas OTC 3488, 11 pages, April 30 through May 3, 1980.

20. Lawson, A. C. et. al., "The California Earthquake of April 18, 1906, "Report of the State Earthquake Investigation Commission, Vol. 1", 1980.

21. Lee, K. L., and Chan, K., "Number of Equivalent Significant Cycles in Strong Motion Earthquakes", Proc. International Conference on Microzonation, Seattle, Washington, Vol. II, pp. 609-627, Oct., 1972.

22. Marcuson, W. F., Bieganousky, W. A., "Laboratory Standard Penetration Tests on Fine Sands", Journal of Geotechnical Engineering Division, ASCE, Vol. 103, June 1977.

22. Maxwell, J. C., "A Treatise on Electricity and Magnetism", Oxford Univ. Press, London, England, 1892.

23. Mitchell, J. K., and Lunne, T., "Cone Resistance as a Measure of Sand Strength", Journal of the Geotechnical Engineering Div., ASCE, July 1978.

24. McJunkin, R. D., and Turpen, C. D., "Strong Motion Records Recovered from the Trinidad Off-Shore, California Earthquake of Nov. 19, 1980", Division of Mines and Geology, Office of Strong Motion Studies, Sacramento, California.

25. Mitchell, J. K., Chatoian, J. M., and Carpenter, G. C., "The Influence of Fabric in the Liquefaction Behavior of Sand", Report to the U.S. Army of Engineering Waterways Experiment Station, Vicksburg, Berkeley, 1976.

26. Mulilis, J. P., Seed, H. B., Chan, C. K., Mitchell, J. K. and Arulanandan, K., "Effects of Sample Preparation of Sand Liquefaction", Journal of Geotechnical Engineering Div., Proc. of the American Society of Civil Engineers, 103, No. G52, 91-108, Feb. 1977.

27. Oda, M., "Initial Fabrics and Their Relations to Mechanical Properties of Granular Materials", Soils and Foundations, 12, 1, pp. 17-37, 1972.

28. Oda, M., "The Mechanisms of Fabric Changes During Compression Deformation of Sands", Soils and Foundations, 12, 2, pp. 2-18, 1972.

29. Ohsaki, Y. and Iwasaki, R., "On Dynamic Shear Moduli and Poisson's Ratio of Soil Deposits", Soils and Foundations, (Japan), Vol. 13, No. 4.

30. Peck, R. B., "Liquefaction Potential: Science v. Practice", Journal of the Geotechnical Engineering Div., Vol. 105, pp. 393-398, March, 1979.

31. Richart, F. E., Jr., Hall, J. R., Jr., and Woods, R. D., "Vibrations of Soils and Foundations", Prentice-Hall, Inc., Englewood Office, New Jersey, p. 414, 1970.

32. Schnabel, P. and Seed, H.B., "Accelerations in Rock for Earthquakes in the Western United States", Bull. Seism. Soc. America, 63, pp. 501-516.

33. Seed, H. B., "Evaluation of Soil Liquefaction Effects on Level Ground During Earthquakes", State-of-the-Art paper for the National Convention on Liquefaction Problems in Geotechnical Engineering, ASCE, 1-104, Sept. 27 through Oct. 1, 1976.

34. Seed, H. B. and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analysis", EERC-70-10, Earthquake Engr. Research Center, University of California, Berkeley, 1970.

35. Seed, H. B., Arango, I., Chan, C. K., Gomez-Masso, A., and Ascoli, R. G., "Earthquake-Induced Liquefaction Near Lake Amatitlan, Guatemala.

36. Seed, H. B., and Idriss, I. M., "Analysis of Soil Liquefaction: Niigata Earthquake", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM3, pp. 83-108, 1967.

37. Seed, H. B. and Idriss, I. M., "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM9, pp. 1249-1273, 1971.

38. Seed, H. B., and Lee, K. L., "Liquefaction of Saturated Sands During Cyclic Loading", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 92, No. SM6, pp. 105-134, 1966.

39. Seed, H. B., Lee, K. L., and Idriss, I. M., "An Analysis of the Sheffield Dam Failure", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM6, pp. 1453-1490, 1969.

40. Seed, H. B., and Peacock, W. H., "Test Procedure for Measuring Soil Liquefaction Characteristics", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM8, pp. 1088-1119, 1971.

41. Seed, H. B., Idriss, I. M., Makdisi, F., and Banerjee, N., "Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses", Report No. EERC 75-29, Earthquake Engineering Research Center, University of California, Berkeley, 1975.

42. Smith, S. S. and Arulanandan, K., "Relationship of Soil Structure to Engineering Properties", Proc. of ASCE, Journal of the Geotechnical Engineering Division, 1980 (in press).

43. Stokoe, K. H., and Hoar, R. J., "Variables Affecting In Situ Seismic Measurements", Proceedings of the Earthquake Engineering and Soil Dynamics Conf., ASCE, June 19-21, 1978.

44. Tavenas, F. A, Discussion, "The Standard Penetration Test", by Victor de Mello, Proceedings of the Fourth Pan-American Conference on Soil Mechanics and Foundation Engineering, Vol. 111, 1971.

45. Trifunac, M. D., and Brady, A. G., "On The Correlation of Seismic Intensity Scales with the Peak of Recorded Strong Ground Motion", Bull. Seism. Soc. Am. Vol. 65, Feb., 1975.

46. Woods, Richard, "Measurement of Dynamic Soil Properties", Proceedings of the Earthquake Engineering and Soil Dynamics Conf., (State-of-the-Art Paper), ASCE, Vol. 1, pp. 91-178, June, 1978.

47. Youd, T. L., and Hosoe, S. N., "Historic Ground Failures in Northern California Triggered by Earthquakes", Geologic Survey Professional Paper 993, 1978.

APPENDIX A

APPENDIX A

Plates

Preceding page blank

A-1 Site Description

The site chosen for the field portion of this study is located at the mouth of Tomales Bay in Marin County, California, approximately 40 miles north of San Francisco.

The property is presently occupied by Lawson's Landing, a recreational facility for camping, fishing, and recreational vehicles. The property is bounded on the west by the Pacific Ocean, on the south by Tomales Bay, and on the north and east by uplands. The portion of the site investigated is near the center of approximately 40 acres of relatively level land. Sand dunes extend along the ocean beach to the west and along the base of the hills to the east. The relatively level area between the dunes is about 5 feet above high tide. Plates 1 and 2 show the site location.

A-1.1 Geologic Setting

The San Andreas fault zone lies within Tomales Bay and separates granitic and gneissic basement rocks on the Point Reyes Peninsula southwest of the site from a basement complex of Franciscan melange northeast of the site. The fault location with respect to the site is noted on Plate 2. A geologically long period of strike-slip faulting has taken place along the fault zone. Rocks older than Pliocene age, differ considerably on opposite sides of the fault zone due to the long period of repeated movements.

In the Lawson's Landing area, the Franciscan assemblage east of the fault zone is overlain by Pliocene through Holocene sediments which are partially consolidated to unconsolidated. Plio-Pleistocene Merced formation consisting of siltstone and sandstone occurs north and east of the Lawson's Landing property. These materials are the source of much of the sediments deposited on the site. The only sediments exposed on the site are Holocene sand deposits which form the beach dune and meadow areas. The meadow is underlain by sands deposited in a surf or tidal environment.

A-1.2 Historic Faulting

The strongest historic earthquake that has affected the site was the April 18, 1906, San Francisco earthquake. Its intensity was not recorded at the site, but according to the Carnegie Institue report on the earthquake (Lawson, 1908), the area was observed by Professor R. S. Holway on June 11, 1906. The fault line was "near the base of the spit" and on both sides of the line were "crater-like depressions". Another belt of craters occurred within a zone "about 70 feet wide" southwest of the fault.

From Tom's Point, one and a half miles south of Lawson's Landing, and with sufficient elevation for a good view of the fault (which had 2 traces at Tom's Point), it could be visibly traced northwest at low tide toward Lawson's Landing through the intervening mud flats. At Tom's Point, horizontal displacement was estimated to be about 8 feet, (Lawson, page 65). G. J.

Gilbert also inspected the Tomales Bay region. He reported (Lawson, page 68), the trace was N35W, and that it "nowhere departs more than a few hundred feet from the straight line connecting its extreme points". Gilbert described three modes of disruption: straight single line; divided, with traces separating and reuniting; and en echelon, unconnected. According to Gilbert, the fault trace across sloping areas indicated it is approximately vertical. Observations by others, reported on page 84 of the Lawson report (1908), indicated no vertical displacement, only horizontal displacement was noted in the Tomales Bay area.

A-1.3 Ground Motion

For engineering analysis, important parameters by which a ground motion can be described are its magnitude, acceleration, duration and frequency content. Since the details of future earthquakes are unknown, determining these parameters is a statisticl problem. Analysis can be based on ground motion that are in some sense averages of those measured in the past, or on particular past earthquakes which are assumed to be typical or can be scaled to represent certain limiting conditions.

Based on the historic event of 1906 and the potential of the San Andreas fault, an earthquake of 8.3 Richter magnitude was selected for the site. On the basis of attenuation relationships (Housner 1965, Schnabel & Seed 1972), the maximum ground surface acceleration for the site was determined to be between 0.5g to 0.6g. Based on the damage reported in Lawson (1908), the maximum intensity was correlated to a maximum ground acceleration of about 0.5g.

For liquefaction analysis, one of the most important parameters characterizing ground motion is acceleration. The duration of the ground motion is represented by an euqivalent number of uniform cycles (Lee et. al. 1972, Seed et. al. 1975). Therefore, in the analysis for induced stresses, a ground motion with peak acceleration of 0.5g and 30 equivalent cycles of shaking was used (Seed et. al. 1975).

A-2 Field Investigations

A-2.1 Drilling

The field drilling program was carried out using a CME 55 truck-mounted rig. Due to the caving of sands, the entire drilling program was conducted using hollow stem augers. The augers had an 8-inch outside diameter, and a 3-7/8-inch inside diameter. The length of each section was 5 feet.

After advancing each section of the hollow stem, sampling was attempted by Modified Porter, Shelby tube and SPT samplers. The most successful sampling was achieved by the SPT sampler. Each boring was logged in the field through the observation of SPT samples and drill cuttings.

A-2.2 SPT Procedures

A standard SPT sampler driven with a 140-pound hammer falling 30 inches was used to obtain blow counts and samples. Initially, 2 turns of a rope around the pulley were used to drop the hammer and drive the sampler 18 inches into the soil below the bottom of the hollow stem. Blow count obtained in the last 12 inches were used in this study. To investigate the sensitivity of friction between the rope and the surface of the pulley, 1-1/2 to 2-1/2 turns of rope around the pulley were tried. No significant differences were noted in the results. The Standard Penetration Test method as carried out in this study was in accordance with ASTM Specification D-1586-67.

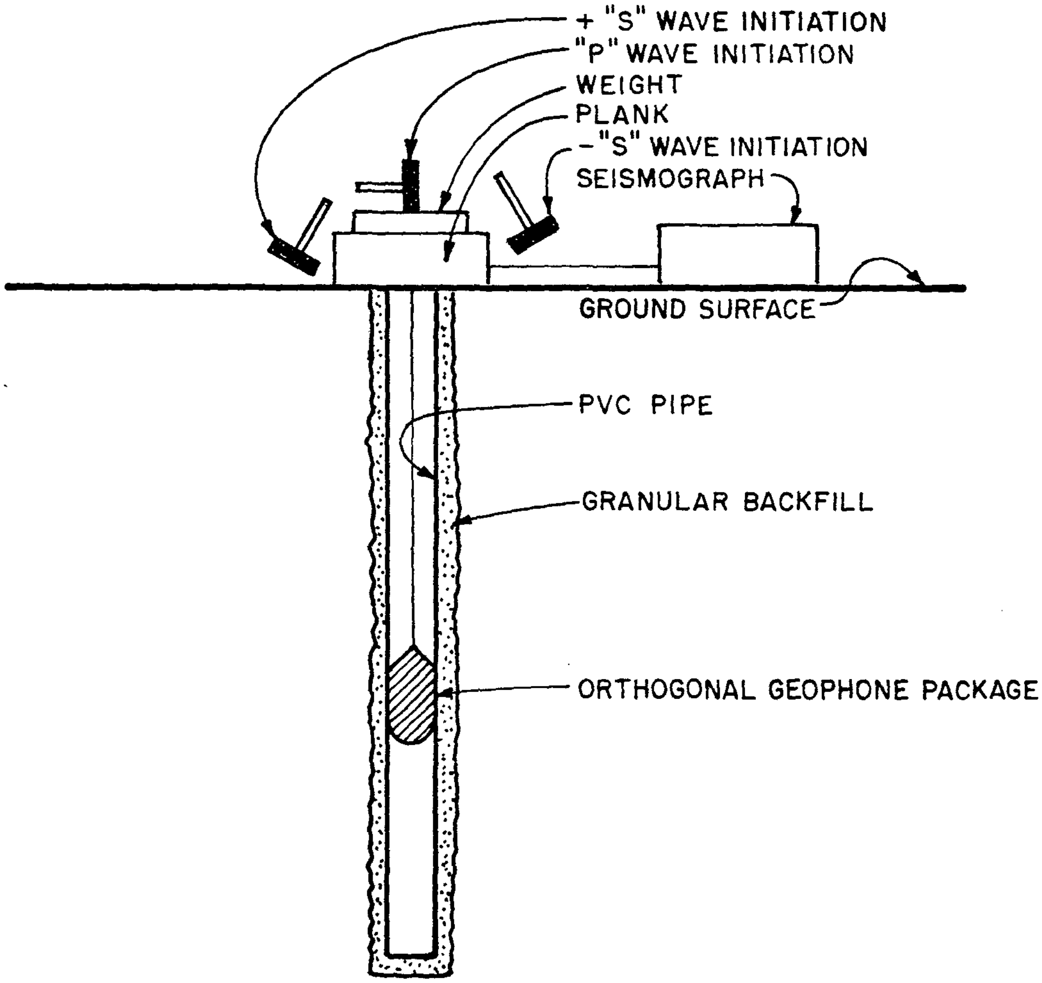
A.2-3 Electrical Probe

The electrical probe was used to measure soil properties within the boreholes. Measurements were taken within four borings. After advancing the drill hole 5 feet, the probe was inserted inside the hollow stem auger and pushed 8 to 12 inches into the undisturbed soil. Measurements of horizontal conductivity, vertical conductivity and pore fluid conductivity were obtained at each testing level. The probe was then withdrawn from the hole and drilling continued. This procedure was continued to a depth of 35 feet. The detailed methodology for using the electrical probe presented by Arulanandan and Kutter (1975) was used in this study.

A-2.4 Seismic Survey

Borings were drilled to fifty feet in depth and cased with 2-inch diameter PVC casing. The ends of the casing were sealed and slots cut along the casing length to facilitate placement in the water-filled boring. Pea gravel was poured in the annular space between the casing and the boring wall to provide a contact between the two surfaces. Due to sloughing of the boring wall, some granular materials accumulated at the bottom of the boring which prevented the casing from being installed to the total fifty foot depth. In most cases, this sloughing amounted to only a few feet of accumulated material.

Downhole measurements are made by measuring the time for a shear wave to travel from the ground surface to the geophone package located in the boring casing. The technique employed is that developed by Kobayashi in 1959 at the Earthquake Research Institute in Japan. This technique, the "Horizontal Traction Method", is illustrated in Figure A-1. A plank is placed near the cased boring and a heavy object is placed on it to provide sufficient bonding to the ground surface. Impacts are made on one end of the plank and the shear wave enhanced for ready identification. The opposite end of the wooden plank is then struck and a reversed breaking shear wave arriving at the same time as the previous arrival is observed for confirmation. For compression wave velocities, the plank is struck on top in a vertical direction. Because of the saturated "clean" granular soils at the site, the only compression velocity observed was that of the intersticial water or 5,000 ft/sec. This velocity was faster than the wave path through the individual sand grains and therefore obscured the later arriving sand wave. Consequently, only the compression velocity of the water was observed.



"P" & "S" WAVE VELOCITY DETERMINATION

DOWN HOLE SEISMIC METHOD

LAWSON'S LANDING

PLATE

A-1

APPENDIX B

B-1 - Laboratory Test

Procedures for Shear Wave Velocity Measurements

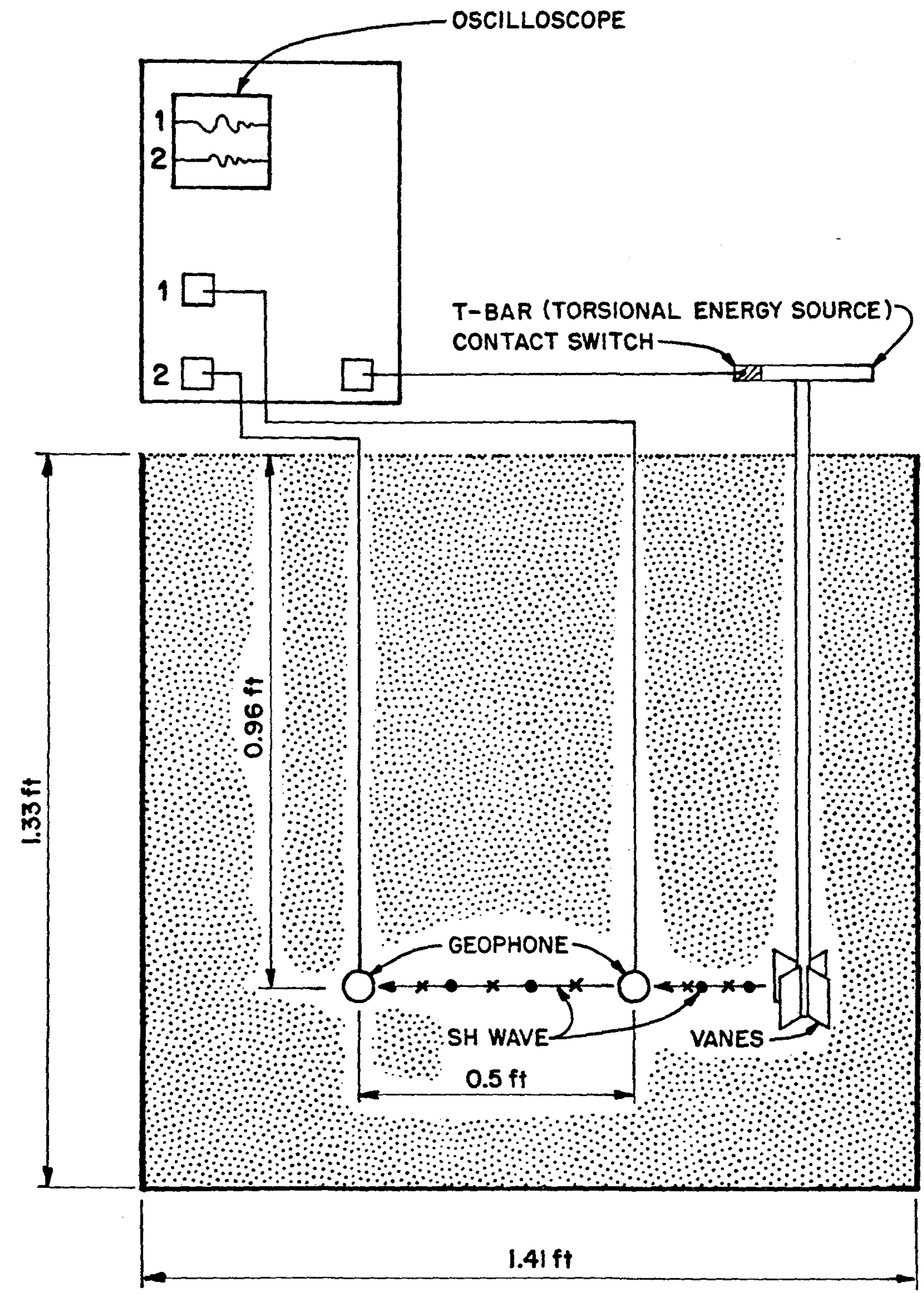
B-1 Laboratory Shear Wave Testing

The shear wave velocity of both Ottawa C-109 and sand from the site were measured. The shear wave velocities of these materials were measured in both a loose and densified state in the horizontal direction in a waterproof box capable of being vibrated. The sands were weighed and poured in the partially water-filled box until the box was filled. Since the volume of the box and the sand's dry weight were known, the density was calculated. After the shear wave velocity was determined, the sand was densified by vibrating the box. The new sand density was calculated and the shear wave measurements repeated.

Attempts were made to measure: (1) a horizontally polarized shear wave from the top of the sand to near the bottom and, (2) a vertically polarized shear wave within the body of the sand. Both these methods produced compression waves which interferred with the later arriving shear waves as well as producing interfering refractions off the sides of the box. Successful shear waves were generated through the use of a "T" bar with two mutually perpendicular vanes welded to its long end. The "T" bar was buried with the vanes next to the horizontal geophone (7 Hz). Another horizontal geophone was positioned in line and 0.50 feet away. Both geophones were oriented with their axes parallel to the shear wave's oscillations (perpendicular to the direction of propagation). Details of the laboratory test procedure are shown on Plate B-1.

To time and observe the wave forms generated by the geophones, a Tektronics 5000 Series dual trace storage oscilloscope was used with an external contact switch for triggering. The contact switch was attached to the cross bar portion of the "T" to facilitate striking it when generating the shear wave.

By striking the cross bar, the energy was transferred via torsion to the vanes at the bottom of the "T". The shear wave generated was horizontally polarized. As in the field procedure, reversal of the energy source produced a reversal of the shear wave, thus confirming its presence. Two geophones were used in testing so that any error produced by the contact switch would be eliminated since only the interval time between the two geophones was measured.



NOT TO SCALE

SCHEMATIC OF CROSS-HOLE LABORATORY TEST

LAWSON'S LANDING

PLATE B-1