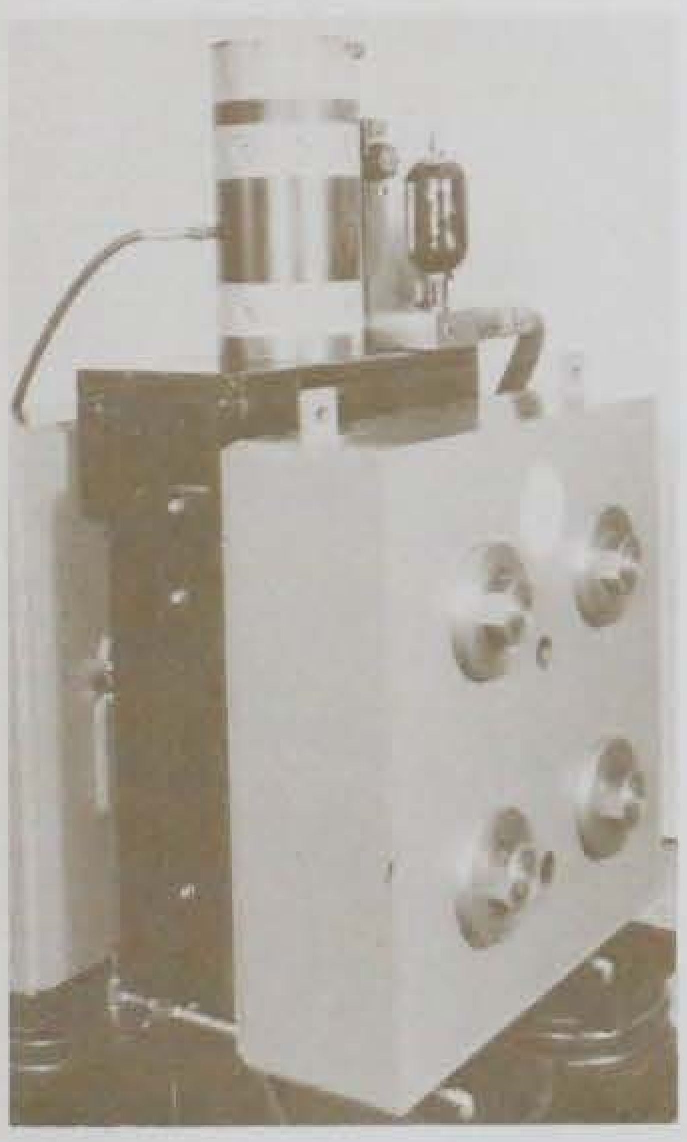
my Corps
gineers



GEOTECHNICAL LABORATORY

MISCELLANEOUS PAPER GL-87-26

CREATION OF A DATA BASE OF SEISMIC SHEAR WAVE VELOCITIES FOR CORRELATION ANALYSIS

by

David W. Sykora

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

September 1987

Final Report

Approved For Public Release, Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
Assistant Secretary of the Army (R&D)
Washington, DC 20314-1000

Under Project No. 4A161101A91D

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY 2b. DECLASSIFICATION / DOWNGRADING SCHEDULE	3. DISTRIBUTION / AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Miscellaneous Paper GL-87-26	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES Geotechnical Laboratory — 6b. OFFICE SYMBOL (If applicable) CEWESGH-R	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS (City, State, and ZIP Code) PO Box 631 Vicksburg, MS 39180-0631	7b. ADDRESS (City, State, and ZIP Code)
8a. NAME OF FUNDING / SPONSORING ORGANIZATION US Army Corps of Engineeers — 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER DACA39-86-M-0571
8c. ADDRESS (City, State, and ZIP Code) Washington, DC 20314-1000	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. — PROJECT NO. See reverse — TASK NO. — WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)

Creation of a Data Base of Seismic Shear Wave Velocities for Correlation Analysis

12. PERSONAL AUTHOR(S)
Sykora, David W.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM ______ TO ______	September 1987	116

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Crosshole seismic — Shear modulus
Data bases — Shear wave velocities
In situ

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

This report summarizes the compilation of a sizable data base of seismic velocities measured in situ with high-quality crosshole methods. The objective of developing the data base is to determine, eventually, the most accurate means of estimating dynamic shear modulus or shear wave velocity in soil given any number of known parameters. This data base may be useful in a number of Corps of Engineers applications including prediction of ground motions from nuclear or high-explosions, deployment of passive seismic arrays, employment of active seismic systems, and evaluation of dynamic response of facilities.

Data were collected from several sources in private industry and state and Federal government agencies at sites located across the United States. Different types of projects, soils, geologic backgrounds, seismic zones, and ground-water conditions were used. Over 2,000 data points were collected in two distinct data bases that were developed for different approaches to correlations.

(Continued)

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code) — 22c. OFFICE SYMBOL

DD Form 1473, JUN 86 — Previous editions are obsolete. — SECURITY CLASSIFICATION OF THIS PAGE

Unclassified

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

10. PROJECT NO. (Continued).

4A161101A91D

19. ABSTRACT (Continued).

Factors known to affect shear modulus or shear wave velocity and parameters used as independent correlative variables are discussed and presented in this report. Results of numerous previous correlative studies are used. Criteria used to select data are presented and justified.

The distribution of data in the two data bases and ranges in shear wave velocity are summarized to characterize the nature of the data. Visual observations of preliminary correlations are made.

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

PREFACE

This study was conducted by the US Army Engineer Waterways Experiment Station (WES) for the Assistant Secretary of the Army (R&D), Project Number 4A161101A91D, as an In-House Laboratory Independent Research (ILIR) program during FY 86 and FY 87. Initial appropriations were received in January 1986. The title of the overall study was "Evaluation of Dynamic Soil Stiffness Based on Correlations with Other Geotechnical Parameters."

This ILIR study was proposed and performed by Mr. David W. Sykora of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), WES. The report was prepared by Mr. Sykora. It is intended to be the second of three reports published under the overall ILIR study topic. The first report was published as a WES Miscellaneous Paper.

Some information contained herein was used by Mr. Sykora in a thesis presented to the University of Texas at Austin in partial fulfillment of the degree of Master of Science in Engineering. That work was performed under the direction of Professor Kenneth H. Stokoe II, Department of Civil Engineering, and published as an engineering report. Dr. Stokoe also provided input to the methodology of the study reported herein through purchase order DACA 39-86-M-0571.

The author is very grateful to the many people who have offered their time and assistance in making data and information available for this study. The complete list of participants is extensive--too long to list here. However, those persons who deserve special recognition are:

Mr. Don Babbitt, Mr. Cliff Nomura, and Mr. Les Harder	California Department of Water Resources
Mr. Francke Walberg and Mr. Robert Dimmit	US Army Engineer District, Kansas City
Mr. Abbas Roodsari	US Army Engineer District, Los Angeles
Mr. Gil Avila	US Army Engineer District, Sacramento
Mr. Larry Cave, Jr.	US Army Engineer Division, Lower Mississippi Valley
Mr. Robert Ballard, Jr. and Mr. Joseph Curro, Jr.	US Army Corps of Engineers, WES
Mr. Howard Spellman	Converse Consultants
Mr. Reid McLamore	ERTEC, Inc.
Dr. Gonzalo Castro and Mr. Michael Paster	Geotechnical Engineers, Inc.

Mr. William Weiler	Haley and Addrich, Inc.
Mr. Jerry Nelson	Harding-Lawson, Assoc.
Mr. Eric Strassburger	Harlan-Miller-Tait and Assoc.
Mr. Robert White	Law Environmental Services
Mr. John Barneich, Dr. I. M. Idriss, and Dr. Arthur Dvinoff	Woodward-Clyde Consultants
Dr. Roy J. Greenfield	Professor of Geophysics, Pennsylvania State University
Dr. Kenneth H. Stokoe II	Professor of Civil Engineering, University of Texas at Austin

Messrs. Donald Douglas, Harley Alderson, and Melvin Seid input data into the computer and manipulated data base files. Mr. William Hanks, Soil Mechanics Division, drafted figures. The report was edited by Mrs. Joyce H. Walker, Information Products Division, Information Technology Laboratory. Mr. Joseph P. Koester, EEGD, provided technical assistance.

Supervision at WES was provided by Dr. A. G. Franklin, Chief, EEGD. The project was conducted under the general supervision of Dr. William F. Marcuson III, Chief, GL.

COL Dwayne G. Lee, CE, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (Continued)

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
kips (mass) per square foot	4,882.428	kilograms per square metre
pounds (force)	4.448222	newtons
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds per cubic foot	16.01846	kilograms per cubic metre
tons (force) per square foot	95.76052	kilopascals

CREATION OF A DATA BASE OF SEISMIC SHEAR WAVE VELOCITIES FOR CORRELATION ANALYSIS

PART I: INTRODUCTION

1. The dynamic response of a soil mass subjected to seismic excitation is the focus of much attention among engineers both in research studies and in application of state-of-the-art technology to practical problems. A key property necessary to properly evaluate dynamic response of soil is dynamic shear modulus (modulus of rigidity) G . Shear modulus is necessary to evaluate geotechnical engineering problems both quantitatively and qualitatively, including earthen structures (e.g., Makdisi and Seed 1977), foundations for superstructures (e.g., Franklin 1979), deep foundation systems (e.g., Randolph 1980), dynamic soil-structure interaction (e.g., Lysmer et al. 1975), machine foundations (e.g., Richart, Hall, and Woods 1970), and free-field response (e.g., Chen, Lysmer, and Seed 1981; Schnabel, Lysmer, and Seed 1972). Shear modulus is also used to evaluate susceptibility of soils to liquefaction (Dobry et al. 1981) and to predict the ground surface and subsurface motions from outrunning ground shock produced by the detonation of high or nuclear explosives (Hadala 1973). In general, determination of G has been adopted by the engineering community as an integral part of geotechnical engineering studies for all major facilities and smaller facilities in areas where significant earthquake or synthetic vibration hazard exists.

2. Values of G are determined either by measurement in the laboratory on "undisturbed" soil samples or by calculations using shear wave velocity V_s measured in situ and the mass density of the soil. Mass density ρ may be determined using "undisturbed" soil samples or in situ density tests. Shear modulus measured at small shear strain (less than 10^{-5} in./in.*), referred to as G_{max} , ultimately is the fundamental design parameter (Hardin and Drnevich 1972). Using elastic theory which is approximately valid at these small strains, G_{max} is calculated from V_s using the following equation:

$$G = \rho V_s^2 \qquad (1)$$

---

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 6.

3. In situ measurement of V_s provides the most accurate means to determine G_{max} (i.e., from V_s) (Woods 1986). Shear modulus measured in the laboratory by methods such as the resonant column test are subject to empirical corrections and rely heavily on the assumption that samples are undisturbed (in particular, have not undergone alterations in void ratio, fabric, or cementation) and are representative even if restored to the in situ stress state. Anderson, Espana, and McLamore (1978) and Arango, Moriwaki, and Brown (1978) independently used the results of field and laboratory test measurements to determine that laboratory-derived values of G_{max} were as low as 50 percent of in-situ-derived values, even after empirical corrections were included.

4. Values of G or V_s have been estimated for various projects based on reported empirical correlations which were available and seemed appropriate at the time. Use of correlations typically results from economic concerns over the expense of field and laboratory measurements to determine G , especially within a reasonable degree of statistical reliability. Creditable field correlations would be very useful in supplementing in situ seismic investigations so that a larger percentage of soil effectively could be sampled at a lower cost. This study suggests using correlations to estimate V_s or G in all aspects of a geotechnical engineering study.

5. This report summarizes the strategy used to create a seismic data base. It represents the second phase of an overall study which attempts to compile the most accurate, comprehensive, and broad-range investigation possible to evaluate empirical relationships between dynamic soil stiffness (G or V_s) and parameters typically obtained in geotechnical investigations. It was intended to include as many pertinent parameters as possible in logical groupings. Results would be presented in a manner such that any quantity or quality of appropriate information available could be used to estimate G or V_s within a stated confidence limit. This methodology would apply to numerous in situ conditions with varying degrees of accuracy.

6. This report begins in the next chapter with the formulation of pertinent parameters and continues in the following chapter with the system used. In the following chapters, the process of data accumulation and reduction are described. Then, a general presentation of information collected is provided with comments.

7. Subsequent reports will document various analyses conducted using

the data base to gain a better understanding of how correlations should be applied to engineering studies. These studies will focus on actual correlations performed and comparisons with previously reported studies. Follow-on studies are expected to examine specific questions of interest such as V_s measurements in gravel and rockfill materials, influence of various drilling techniques, and comparison of V_s as measured using downhole and crosshole techniques. Studies involving compression wave velocity V_p will also be considered.

PART II: SELECTION OF DATA BASE PARAMETERS

8. The methodology adopted to conduct a study of this nature is an important aspect. This chapter provides insight into the logic and justification used to select and utilize data base parameters. More details with regard to reduction of variables are contained in a later part (Part IV).

Approach

9. Estimation of G or V_s may be desirable for any combination of field conditions. Therefore, an approach was used in this study to optimize any amount of reliable and pertinent information available. Data and information were partitioned into different groups of similar importance for use in analyses. Although this approach is warranted to accomplish the stated purpose of this study, care was exercised to always consider the variables in light of their mechanical influence and interdependence on dynamic soil stiffness.

10. The use of correlations to estimate V_s or G can be made in at least three stages of a geotechnical investigation program. An example of this division is provided in Table 1. At each stage, a different set of known parameters is available, becoming progressively more detailed and numerous. Therefore, at any particular stage, a certain set of parameters can be expected and should be incorporated to estimate V_s. Generally, the more information available, the more accurate the estimate of V_s should be (Ohta and Goto 1978). Estimates of V_s may be very useful to plan an overall seismic exploration program, optimize velocity measurement locations to detect low velocity zones, and select design parameters for analysis. Therefore, V_s correlations provide a means to optimize actual measurements and support and supplement measured values.

11. The above discussion applies to actual application of V_s correlations to engineering problems henceforth referred to as functional analysis. Also of interest to this study is the applicability of information in the data base to sensitivity analyses. Sensitivity analyses implies that different variables are examined individually to determine the effect on correlations and possible associations with variables known to affect V_s.

Table 1

Example of Correspondence Between Stages of Construction and Use of Velocity Correlations

Stage	Geotechnical Engineering Program	Parameters Typically Available	Use of Velocity Correlations
I	Reconnaissance	Geologic description Geographic location Seismic history Overburden (stress) condition	Plan all aspects of a seismic exploration program
II	Exploration	SPT* N-values Measured V_s Phreatic surface General soil types	Develop site-specific correlations with N-values to optimize V_s measurements of suspected low velocity zones
III	Analysis	All types of geotechnical engineering data (laboratory and field)	Develop site-specific correlations with stress to compare with measured and estimated values of V_s . Identify and define critical subsurface zones. Integrate all data to make best estimate of design values

* SPT = Standard Penetration Test.

Both types of approach are important and the data base created is conducive to use for each.

12. The selection of appropriate data base parameters is contingent on the availability of sufficient research studies which examine factors affecting V_s or G . These studies are used to define variable groupings thought to affect correlative parameters and various other factors which may be used to better define parameters or material properties. A comprehensive study of correlations between V_s and G_{max} reported by various authors was conducted by Sykora (1987). Information contained in that report is a basis for selections made in this study. Excerpts from Sykora (1987) are contained herein. Factors that affect correlative parameters and V_s (or G_{max}) also are compared.

Physical Factors Affecting Shear Wave Propagation

13. Investigators have been attempting to develop reliable correlations between V_s or G and various measurements in soil or soil properties for about the last two decades. Correlations have been examined using measurements made both in the field and in the laboratory, although the accuracy and applicability of such correlations developed in these two environments differ considerably.

14. Precise and detailed analyses of factors affecting V_s have been performed under controlled laboratory conditions. Laboratory studies using constructed samples have been very useful in determining soil properties and test conditions upon which G and V_s are most dependent. However, G_{max} for field samples is difficult to determine in the laboratory because of adverse effects of sample disturbance. Similarly, reconstructed samples which offer greater consistency to the investigator cannot be conditioned to simulated age and cementation effects which occur after tens of thousands of years in situ. These effects are known to affect the magnitude of V_s significantly.

15. Conversely, correlations involving V_s and field-derived parameters have been crude, with considerable scatter of the data. Scatter is associated with the variable nature of some measurement techniques (e.g., SPT), variable nature of soil deposits, and inability of field measurements to completely reflect parameters known from laboratory studies to affect V_s . Despite the large number of field correlations reported in the literature, only a few general conclusions can be made. As yet, field correlations have proven to be functional to only a limited extent in geotechnical engineering practice.

Laboratory correlations

16. Correlations made from laboratory testing programs offer this study a definite basis from which to begin. Sykora (1987) summarized laboratory studies of V_s and G_{max} correlations. Lee and Stokoe (1986) have examined, in detail, the effects of stress anisotropy on estimation of V_s . Results of their study are also used.

17. Hardin and Drnevich (1972) may have best summarized the influence of various geotechnical engineering parameters on G . They presented a table of factors with proposed relative influences which is shown in Table 2.

Table 2

Factors Affecting the Shear Modulus and Damping of Soil as Determined by Laboratory Tests (from Hardin and Drnevich 1972)

	Importance To*			
	Modulus		Damping	
	Clean Sands	Cohesive Soils	Clean Sands	Cohesive Soils
Strain amplitude	V	V	V	V
Effective mean principal stress	V	V	V	V
Void ratio	V	V	V	V
Number of cycles of loading	R**	R	V	V
Degree of saturation	R	V	L	U
Overconsolidation ratio	R	L	R	L
Effective strength envelope	L	L	L	L
Octahedral shear stress	L	L	L	L
Frequency of loading (above 0.1 cycle/sec)	R	R	R	L
Other time effects (Thixotropy)	R	L	R	L
Grain characteristics, size, shape, gradation, mineralogy	R	R	R	R
Soil structure	R	R	R	R
Volume change due to shear strain (for strains less than 0.5 percent)	U	R	U	R

* V = very important, L = less important, and R = relatively unimportant except that it may affect another parameter; U = relative importance is not clearly known at this time.

** Except for saturated clean sand where the number of cycles of loading is a less important parameter.

Hardin and Drnevich (1972) suggested the relation:

$$
\begin{aligned}
G_{max} &= f\left(\bar{\sigma}_m^{\,0.5}\right) \\
&= f\ \frac{(2.973 - e)^2}{1 + e} \\
&= f(OCR^k) \qquad (2)
\end{aligned}
$$

where

$\bar{\sigma}_m$ = mean effective stress

e = void ratio

OCR = overconsolidation ratio

k = dimensionless quantity which is a function of plasticity index (ranges from 0 for sands to 0.50 for a soil with a plasticity index greater than 100 percent)

18. Using Table 2, Hardin and Drnevich (1972) suggest that besides the very important factors shown in Equation 2, the degree of saturation in cohesive soils is very important. Effective strength envelope and octahedral shear stress are less important factors for sands and clays. The number of cycles of loading, degree of saturation, frequency of loading, thixotropy, grain characteristics, and soil structure are relatively unimportant to measurement of G in clean sands. The number of cycles and frequency of loading, grain characteristics, soil structure, and volume change are relatively unimportant to measurement of G in cohesive soils.

19. The results of several later studies can be used to supplement these findings. Marcuson and Wahls (1972) used results of numerous tests to determine that G measured in the laboratory varies with time of confinement. Based on this study, and other similar studies, time of confinement must be considered when applying results from laboratory-prepared samples to field conditions. More importantly to this study was the noted increase in G with time above and beyond that associated with a decrease in void ratio, even for sands. This implies that factors such as soil fabric contribute to increases in G with time, even for the relatively short periods feasible for laboratory testing.

20. Knox, Stokoe, and Kopperman (1982) and Lee and Stokoe (1986) examined the effect of effective stress states on V_s for anisotropic conditions.

They determined that V_s is controlled by only two principal stresses--those in the directions of particle motion and wave propagation--not $\bar{\sigma}_m$. Therefore, Equation 2 should be modified to be:

$$G_{max} = f\left(\bar{\sigma}_A{}^{0.5}\right) = f(e) = f(OCR^k) \qquad (3)$$

where

G_A = shear modulus in principal stress plane a-b
$\bar{\sigma}_A$ = cross-anisotropic effective stress = $(\bar{\sigma}_a \cdot \bar{\sigma}_b)^{0.5}$
$\bar{\sigma}_a$ = effective stress in direction of shear wave propagation
$\bar{\sigma}_b$ = effective stress in direction of particle motion

21. In summary, the various parameters that affect V_s or G measured in the laboratory are well defined. However, determination of the most important parameters which influence V_s (i.e., void ratio and $\bar{\sigma}_a \cdot \bar{\sigma}_b$) for field studies is difficult and expensive. This condition may explain the low correlation coefficients typically associated with field correlations. The parameters which most influence V_s are not available for field correlations. Given the assumed reliability of laboratory-determined influences, it is necessary for field studies to reproduce these factors using other measurements typically obtained in field studies.

Field studies

22. Numerous field studies have been conducted to examine correlations between various geotechnical parameters and V_s. An overwhelming number of these correlations involve SPT N-value or depth as the independent variable. Other factors, namely soil type and geologic age, have been used to further examine and enhance correlations.

23. Most investigators conclude that SPT N-value is an adequate to good indicator of V_s and useful in correlations. Although the SPT is a large-strain technique as compared with a small-strain measurement of V_s, soil-dependent relationships between N-value, effective stress, and void ratio have been derived (Marcuson and Bieganousky 1977). Effective stress and void ratio have been shown in the laboratory to affect V_s. Therefore, the same primary parameters, namely void ratio and effective stress, affect both N and V_s,

and in the same general manner. The SPT is a commonly used measurement in geotechnical engineering field studies which makes N-values readily available for correlation.

24. Comparisons in this study of the different relationships involving SPT N-value proposed by Japanese and US investigators were normalized for the effect of assumed energy delivered to the sampler. A recent study by Seed et al. (1985) that compared energy efficiencies and techniques of typical Japanese SPT equipment and procedures with US equipment and procedures indicated that a one-to-one correspondence of N-values between countries is incorrect. Given that techniques for measurement of dynamic properties are equivalent between countries, comparisons between N versus V_s correlations from Japan and the United States must be put on an equivalent basis by adjusting N-values to account for differences. Relations plotted in this part have been adjusted to account for differences in energy. Japanese N-values were assumed to correspond to an efficiency of 67 percent of free-fall energy (N_{67}) and were adjusted to an assumed US efficiency of 60 percent (N_{60}) which is applicable to a safety hammer operated with a rope and cathead, used on many drill rigs operated in the United States (Seed et al. 1985).

25. Three selected correlative relations involving N and V_s are presented in Figure 1 for comparison. These three relations were selected based on quantity of data available and quality of analysis performed (Sykora 1987). These three relations are very similar at N-values less than 30 blows/ft. At this range, the dependence of V_s on N is very high. Above an N-value of 30, the dependence of V_s on N decreases and the relationships differ more. The relationship developed from Ohsaki and Iwasaki (1973) deviates considerably at large N-values. At an N-value of 100 blows/ft, the maximum difference between calculated values of V_s using the three relations in Figure 1 is 435 fps.

26. A number of the studies available developed for granular soils are compared in Figure 2. Best-fit relations presented are very different at N-values greater than about 25 blows/ft. At N-value of 100 blows/ft, the maximum difference between calculated values of V_s for different relationships is 635 fps.

27. Four selected correlations between G and N are plotted in Figure 3. Distinctions between applicable soil types are included. These four best-fit relations are quite similar below N-values of 25 blows/ft. At

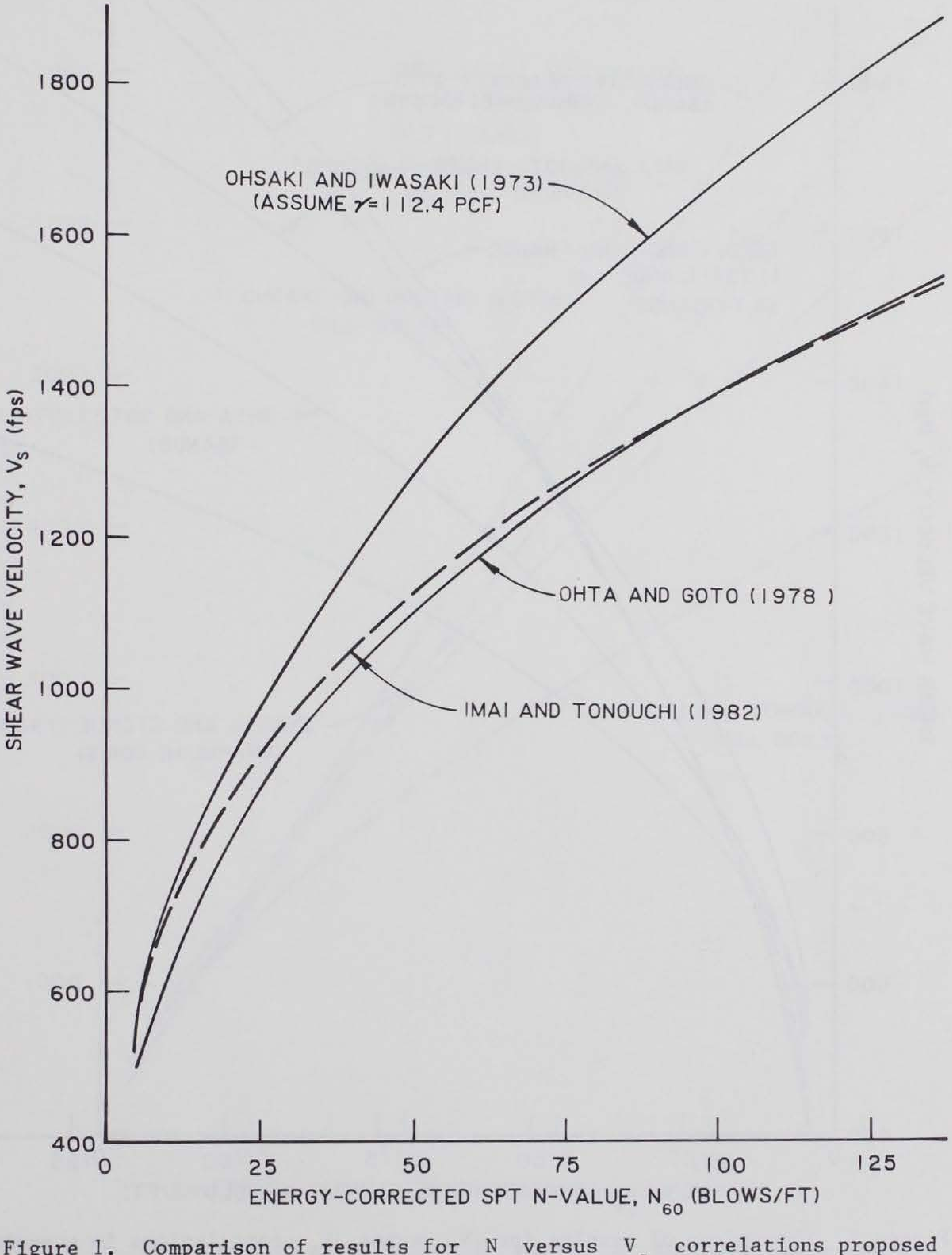


Figure 1. Comparison of results for N versus V_s correlations proposed by selected studies (from Sykora 1987)

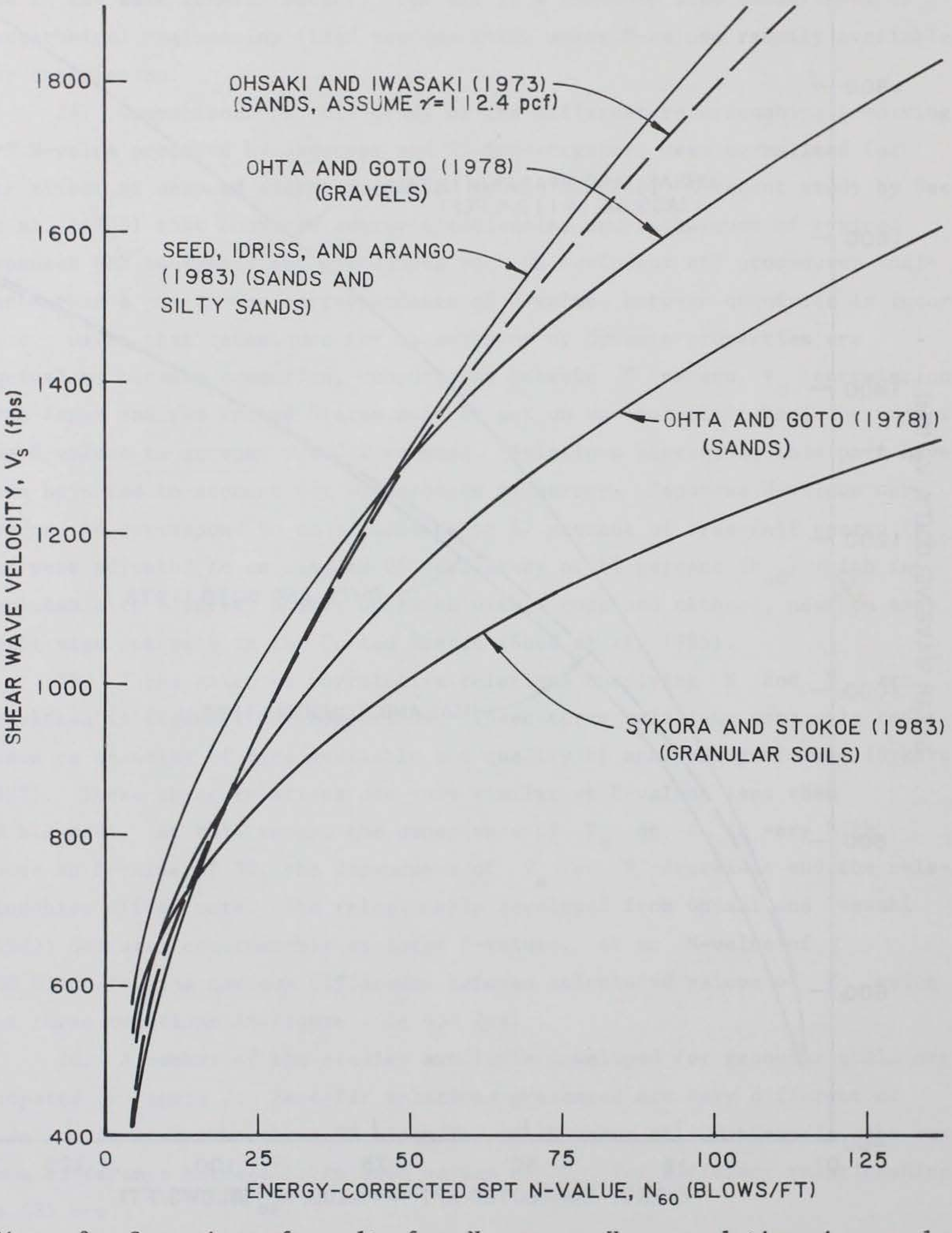


Figure 2. Comparison of results for N versus V_s correlations in granular soils proposed by selected studies (from Sykora 1987)

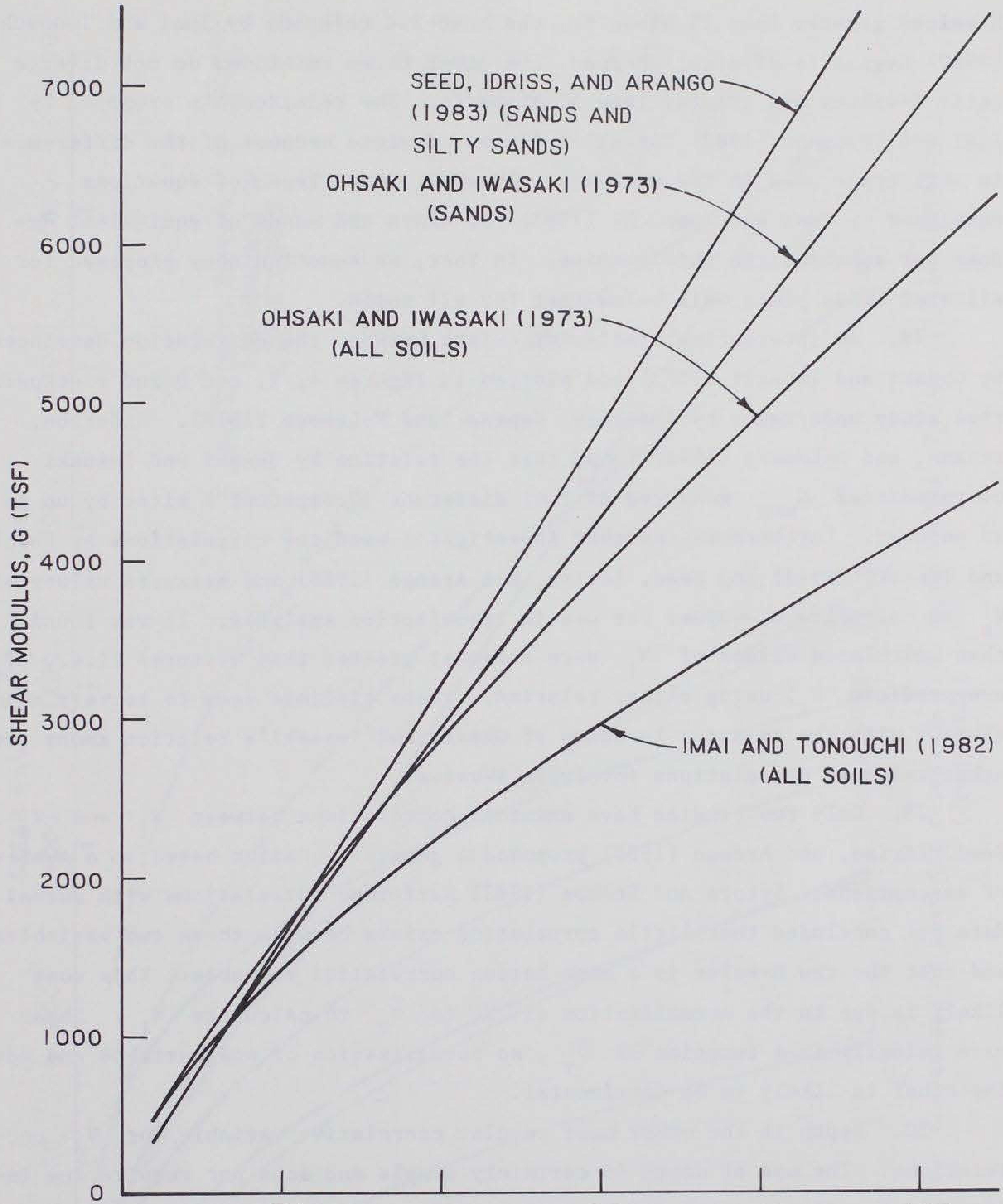


Figure 3. Comparison of results for N versus G correlations proposed by selected studies (from Sykora 1987)

N-values greater than 25 blows/ft, the best-fit relation by Imai and Tonouchi (1982) begins to diverge; whereas, the other three relations do not diverge until N-values are greater than 50 blows/ft. The relationship proposed by Imai and Tonouchi (1982) for all soils may deviate because of the difference in soil types used in the analyses. However, comparisons of equations developed by Imai and Tonouchi (1982) for clays and sands of equivalent age does not substantiate this premise. In fact, an equation they proposed for alluvial sands plots well below that for all soils.

28. An interesting similarity exists between the correlation developed by Ohsaki and Iwasaki (1973) and plotted in Figures 1, 2, and 3 and a comparative study undertaken by Anderson, Espana, and McLamore (1978). Anderson, Espana, and McLamore (1978) found that the relation by Ohsaki and Iwasaki overpredicted G_{max} measured at four different ("competent") sites by up to 25 percent. Furthermore, another investigator used the correlations by Ohsaki and Iwasaki (1973) and Seed, Idriss, and Arango (1983) and measured values of V_s to calculate N_1-values for use in liquefaction analyses. It was found that calculated values of N_1 were somewhat greater than measured (i.e., V_s overpredicts N_1) using either relation. These findings seem to be very consistent with the relative location of Ohsaki and Iwasaki's relation among the other selected correlations involving N-value.

29. Only two studies have examined correlations between N_1 and V_s. Seed, Idriss, and Arango (1983) proposed a general equation based on a number of assumptions. Sykora and Stokoe (1983) performed correlations with actual data but concluded that little correlation exists between these two variables and that the raw N-value is a much better correlative variable. This most likely is due to the normalization of N to $\bar{\sigma}_v$ to calculate N_1. Shear wave velocity is a function of $\bar{\sigma}_v$, so normalization of one variable and not the other is likely to be detrimental.

30. Depth is the other most popular correlative variable for V_s correlations. The use of depth is certainly simple and does not require any information on field or laboratory test results. Not surprisingly, the accuracy of these correlations typically is poor. A comparison of various relations is made in Figure 4. Studies corresponding to the different relations are summarized in Table 3.

31. Major differences exist between the equations plotted in Figure 4. The range in V_s produced by the different relations is tremendous and

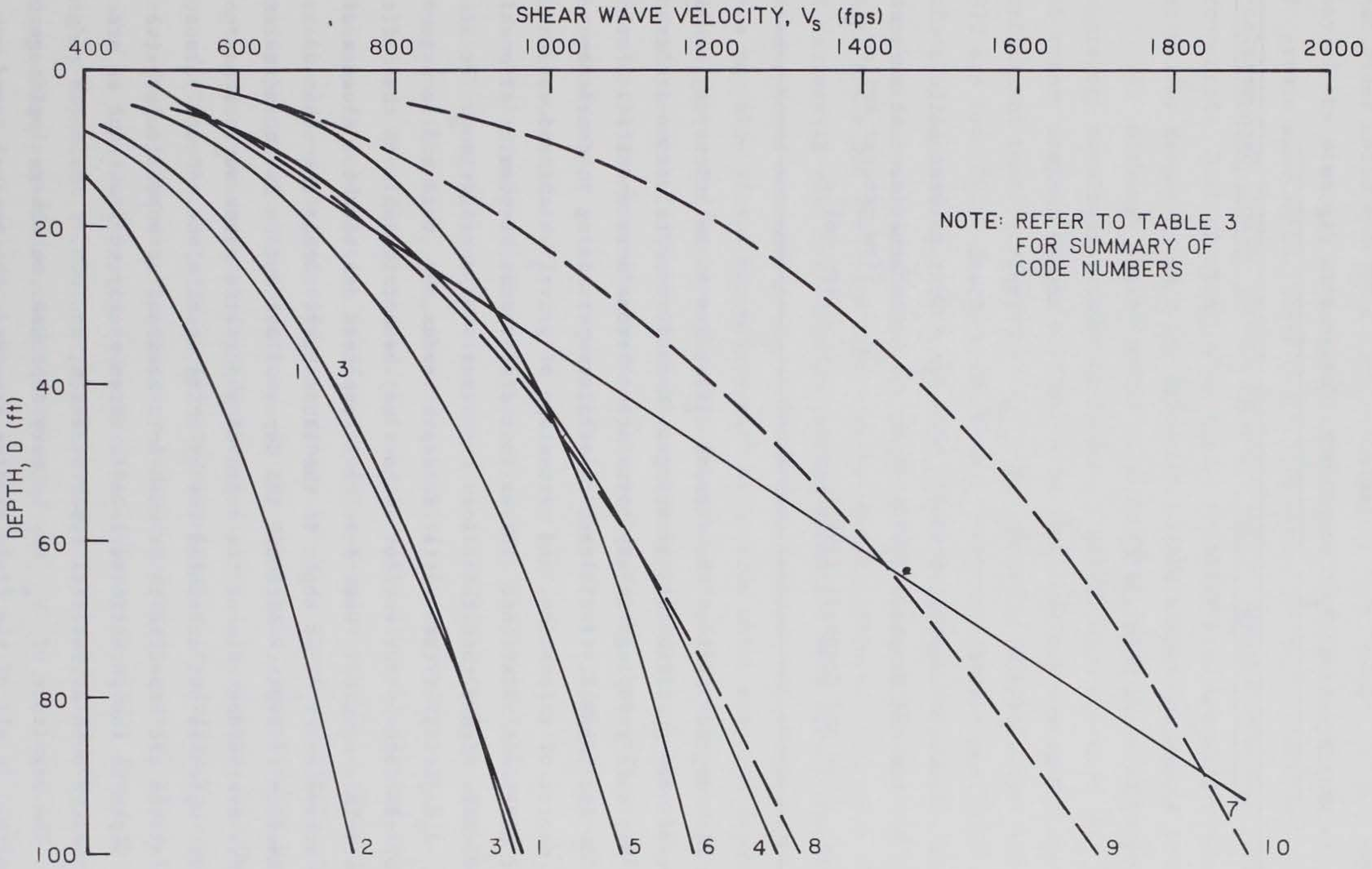


Figure 4. Comparison of best-fit relations from depth versus V_s correlative studies (from Sykora 1987)

Table 3

Depth Versus V_s Correlations Compared in Figure 4

No.	Study	Soil Category
1	Ohta and Goto (1978)	All
2	Ohta and Goto (1978)	Clays
3	Ohta and Goto (1978)	Sands
4	Ohta and Goto (1978)	Gravels
5	Hamilton (1976)	Marine sands
6	Fumal (1978)	Sands
7	Fumal (1978)	Clays
8	Lew and Campbell (1985)	Soft natural soils
9	Lew and Campbell (1985)	Intermediate and saturated firm natural soils
10	Lew and Campbell (1985)	Firm soils

increases with depth. Part of the extreme difference in relations may be due to the use of very specific soil groupings. More diverse relations are expected as the soil groupings in any correlation become more specific. For example, Lew and Campbell (1985) used constraints pertaining to consistency of the soil, degree of saturation, and percentage of gravel which produced a variety of different relations. The maximum differences between relationships are considerably higher than those for N versus V_s correlations.

32. A majority of the scatter in depth versus V_s data and incongruity between correlations by the various authors may be attributable to the difference in materials sampled. Some scatter is expected due to the influence of effective stress on V_s, as shown in the laboratory, rather than that of a simpler function of depth. Although the depth of overburden is an indicator of effective overburden stress, the depth of a phreatic water surface has a major effect which is not accounted for by using depth alone. Depth versus V_s correlations are expected to be much more accurate at a particular site where the depth to the phreatic surface is often nearly constant and so are other parameters such as material type, strength, and OCR.

33. The magnitude of V_s may be very dependent on the geologic age of soil deposits. In all of the field studies examined, the general trend was

that older soils exhibited higher velocities than younger soils. However, in many of the studies, even though there was a distinction between ranges of V_s of certain aged soils, there was little effect on the correlation between N-value and V_s. The effects on depth versus V_s correlations were more pronounced. Most of the studies showed well-defined, though not mutually exclusive, ranges in V_s for Holocene- and Pleistocene-age soil divisions.

34. Although the influence of geologic age cannot be reproduced in laboratory samples, a number of factors found to have an effect on V_s in laboratory samples may be applicable to field correlations. Increased time of confinement tends to increase G_{max} in laboratory samples (Marcuson and Wahls 1972 and Tono 1971) due to a decrease in void ratio and other factors not specifically identified. Ohta and Goto (1978) suggest that geologic age divisions sufficiently represent the degree of cementation in cohesive soils. Older soils are also more likely to be overconsolidated, which has the effect of decreasing void ratio and increasing effective mean normal confining stress, hence increasing V_s. These same factors that change with geologic time may also affect parameters such as N-value which are used in correlations. Divisions between the V_s in soils from different geologic epochs should also take into consideration the relative depths of the soil deposits. Older soils are more likely to be at greater depths than younger soils. Therefore, older soils are more likely to exist at a higher state of stress thereby increasing V_s.

35. Similarly, but to a lesser extent, soil type influences the magnitude of V_s. Soils with wide ranges of grain sizes tend to have smaller average void ratios, and, therefore, exhibit larger values of V_s.

36. Most authors use data from all soils (types) measured to develop a relation for correlative studies involving N-value and V_s. Even though different soils exhibited different ranges in V_s, there was little effect of soil type on best-fit relations. On the other hand, correlations based on depth and V_s were more dependent on soil type. As suggested by Ohta and Goto (1978), the use of soil type in correlations involving V_s, particularly using depth, improves the accuracy because a certain general range in void ratio is inherently specified. Hardin and Drnevich (1972) found that G is highly dependent on void ratio and hardly affected by grain characteristics, size, shape, gradation, and mineralogy.

Summary

37. Previous field correlations between V_s or G and field parameters have not been refined to a level such that they can be confidently used to estimate V_s accurately. A majority of previous correlations examined have investigated relationships between V_s and N-value or depth, or both, with some authors making further distinctions with regard to geologic age, soil type, effective stress, relative firmness, and degree of saturation. When analyzed individually, previous correlations involving only V_s and N-value are more accurate than previous correlations involving only V_s and depth. However, results of some statistical analyses suggest using as many variables as are known to improve the accuracy of V_s correlations.

38. The results of laboratory tests are corroborated by both direct and indirect field measurements, indicating that void ratio and effective stress states are the most important functional variables of V_s and G, especially for granular soils. In addition, it can be concluded that "other" factors related to the geologic age of a soil deposit affect V_s and G to a greater extent than the effects from changes in void ratio and effective stress. These factors most likely include cementation and soil fabric.

39. Given parameters that are known to affect V_s or G from laboratory studies field correlations may be substantiated in terms of these parameters. SPT N-value is known to be influenced by several in situ conditions, especially void ratio and effective stress states (same as V_s). Therefore, N-value is a readily available parameter that can be used to estimate V_s. Other correlations rely on effective stress to correlate with V_s and use of factors such as geologic age and soil type to define potential ranges in void ratio. Correlations that use depth without a parameter such as N are not very reliable, or even justified, except on site-specific bases. Use of geologic age and soil type improves their usefulness.

40. Variables found to be most influential on previous correlations involving V_s and N are geologic age and depth. Division of data among different geologic age groups significantly improved the accuracy of the correlations. In general, V_s increases with increasing N-value, depth, and geologic age. Soil type was found to have varied effects on the different correlations and its influence is unknown.

41. Previous correlations involving V_s and depth were greatly influenced by the inclusion of SPT N-value, geologic age, and soil type.

Correlative equations were quite different with much improved accuracy when geologic age and soil type differentiations were made. In general, V_s increases with increasing depth, geologic age, and relative grain size.

42. Many inconsistencies exist between studies reviewed in this report, especially field studies. The nature of correlations and characteristics of data from previous studies could and should have significant effects on the results of correlations, especially in the absence of a large data base. These differences are difficult to quantify. However, some discussion has been provided in this report to assist the practitioner in using available correlations in an appropriate manner.

Development of Correlative Variables

43. Selection of correlative (independent) variables for this study was based on two primary criteria--allow flexibility to develop correlations for any combination of field conditions and use results of previous studies to interpret influential parameters. Most variables adopted are considered in light of being mechanical and physical indices of known influential parameters. One field measurement, cone penetration resistance, was not considered because another investigator is examining this topic.

Standard penetration resistance

44. The SPT (as described in ASTM D 1586 (1987a)) is one of the most widely used tests for subsurface exploration. It is a useful tool in foundation engineering and liquefaction hazard analysis to identify and characterize subsurface strata. As will be discussed herein, many of the same factors which affect V_s affect N-value, and in a similar manner. Therefore, it seems justified, to correlate V_s with N.

45. The response of the SPT in sands is much more defined than is the SPT response in clays. Few investigators have examined factors that affect N-values in cohesive soils. Typically, N-values in cohesive soils are used to categorize soils according to consistency and correlate with undrained shear strength. Therefore, a dichotomy exists in analysis available for the two types of materials.

46. Although the SPT has been used by engineers for many years, many inconsistencies and nonuniformities still exist in equipment, operators, and test results. The important parameters other than equipment and technique

affecting N-values have been examined extensively. A brief summary of some studies that examined the effects of soil properties and site conditions on SPT N-values follows.

47. Gibbs and Holtz (1957) concluded from tests in sands conducted in the laboratory that effective stress has a significant effect on N . N-value was also found to be related to relative density D_r (a function of void ratio) so that N increased as $\bar{\sigma}_v$ or D_r , or both, increased and N decreased as $\bar{\sigma}_v$ or D_r , or both, decreased.

48. Gibbs and Holtz also found N-values decreased slightly when a soil became saturated. Schultze and Menzenbach (1961) defined this reduction in N to be an average of 15 percent for fine sands when tested below the water table, the effect being more pronounced in looser soils. Melzer (1971) also supports the observation that N is lower below the water table than above based on both field and laboratory results. It cannot be confirmed whether the effect of soil suction above the water table was included in these conclusions. Wu, Gray, and Richart (1984) have shown that the magnitude and influence of soil suction can be very significant for fine sands and silts.

49. De Mello (1971), in a state-of-the-art review of the SPT, examined many soil parameters and factors which affect N , and analyzed the relation in sands between N , D_r , $\bar{\sigma}_v$, and angle of internal friction, ϕ . He noted that N-values are subject to large variations for sands at very shallow depths. Overconsolidation of sands affects the N-value significantly because of large lateral stresses in the soil. De Mello concluded that the SPT is affected more by the mean effective state of stress in the soil $\bar{\sigma}_m$ than the vertical effective stress $\bar{\sigma}_v$.

50. Marcuson and Bieganousky (1977) reported the results of SPT's performance in the laboratory at Waterways Experiment Station (WES). They found N-values to be affected by changes in density, effective vertical stress, and OCR. N-values were found to be repeatable in homogeneous deposits and not repeatable in inhomogeneous deposits. Another important factor found to affect N was the lateral stress conditions in the soil. An increase in lateral stress significantly increased the N-value. N-values were also reported to be sensitive to soil placement method. Marcuson and Bieganousky (1977) concluded that geologic deposition and previous overburden (OCR) are some of the parameters affecting SPT N-values in the field.

51. Marcuson and Bieganousky (1977) note that there is a variation in

relationships between N and D_r for different soils. This evolved from combined SPT testing in four different sands. Best-fit relations between $\bar{\sigma}_v$, N , and D_r for two soils (Reid Bedford and Ottawa sands) tested are given in Figure 5. Marcuson and Bieganousky concluded that curves relating

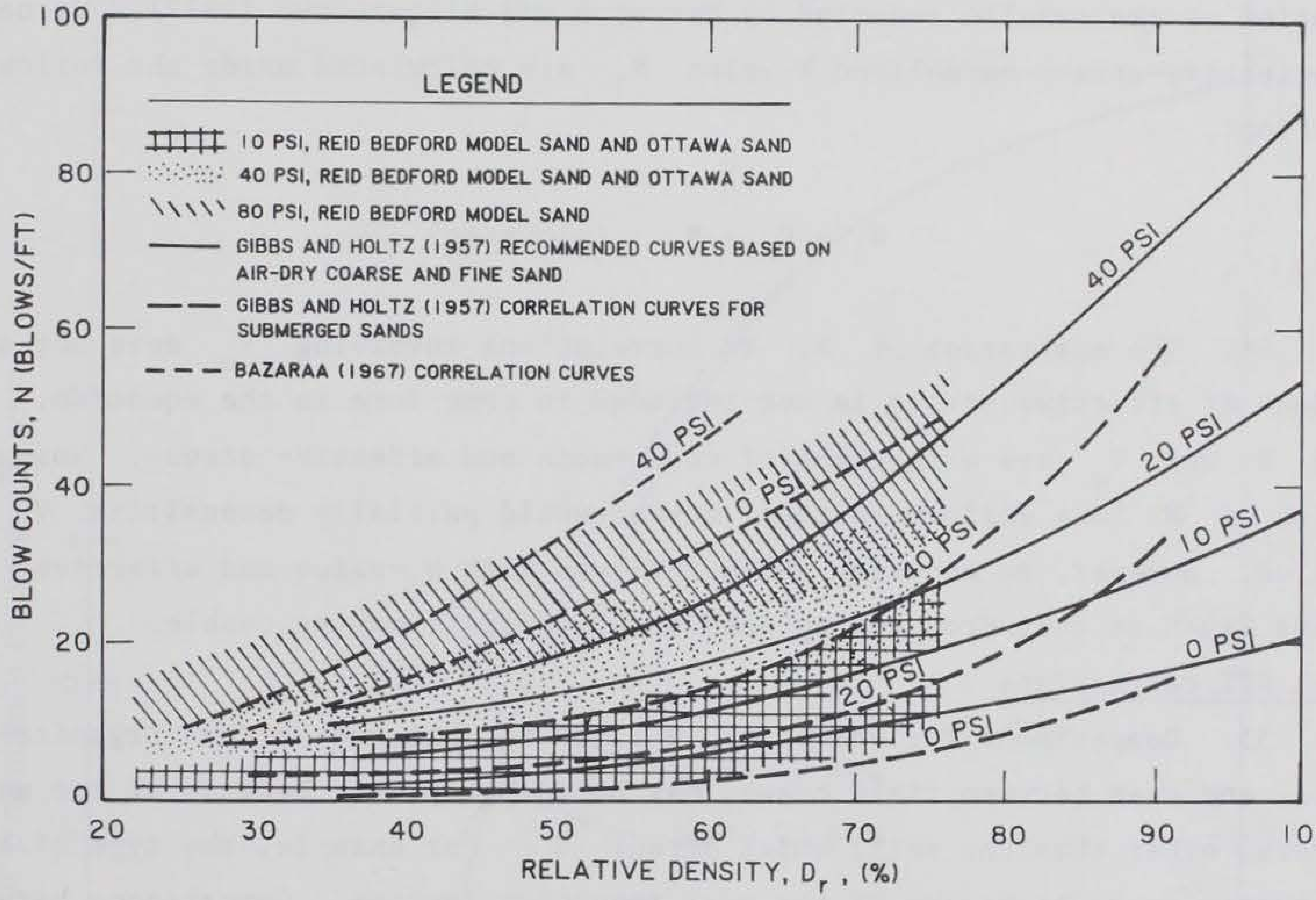


Figure 5. Relationships between SPT N-value, relative density, and effective overburden stress for four sands, presented by Marcuson and Bieganousky (1977)

$\bar{\sigma}_v$, N , and D_r in cohesionless soils are only applicable to that specific soil or a similar soil.

52. As anticipated, many of the same variables that affect V_s affect SPT resistance, and in a similar manner. The more prominent variables are effective stress and void ratio (or relative density). A very important point to be considered is that there are individual N-$\bar{\sigma}_v$-D_r relations for each sand or group of similar sands. This finding may have a major impact on V_s correlations involving N-value when comparing relations from different soils.

<u>Depth-corrected N-values</u>

53. Normalization of SPT N-values to account for the effect of effective vertical stress has become popular in several applications of

geotechnical engineering practice. Applicable correction factors have been proposed by Peck, Hanson, and Thornburn (1974) and Seed and Idriss (1981). The correction factor C_N adopted for this study to normalize N-values in sand to an effective vertical stress of 1.0 tsf was proposed by Seed and Idriss (1981) as shown in Figure 6. These relations resulted from a simplification of the results reported by Marcuson and Bieganousky (1977). Values of effective-stress-normalized N-value N_1 are calculated using the following equation:

$$N_1 = C_N \cdot N \quad \text{(blows/ft)} \tag{4}$$

54. The adaptation of N_1 to correlations involving V_s does not seem prudent if effective stress is not included in some form in the equation. Both N and V_s are a function of void ratio and effective stress. Normalization of N to a uniform vertical stress would partially desensitize V_s to N-value. However, an equation which involves both N_1-value and effective stress (such as that proposed by Seed et al. (1984)) may be usable.

Other SPT corrections

55. Comparisons for values of N and N_1 between various organizations, and even between field crews, may be inappropriate because of the many factors, other than the soil, which affect N . For example, the type of hammer system seems to be one of the most important factors. Comparisons between various triggering systems have indicated variations of approximately 40 percent (Frydman 1970; Kovacs, Evans, and Griffith 1977; Douglas, Olsen, and Martin 1981). Other factors, in addition to these, are discussed further in papers by McLean, Franklin, and Dohlstrand (1975) and Kovacs, Evans, and Griffith (1977).

56. A study by Seed et al. (1985) which summarized the effects of energy efficiency for various hammer/trigger systems and drill rigs was adopted for this study. Seed et al. (1985) proposed a system of normalization of N to an efficiency ratio of 60 percent of "free-fall" energy. Both uncorrected and energy-corrected values of N and N_1 (N_{60} and $N_{1,60}$, respectively) were used in this study.

57. Another correction suggested by Seed et al. (1985) involves the use of different split-spoon samplers. Internal frictional resistance for each type of sampler is different, producing N-values which differ by amounts that

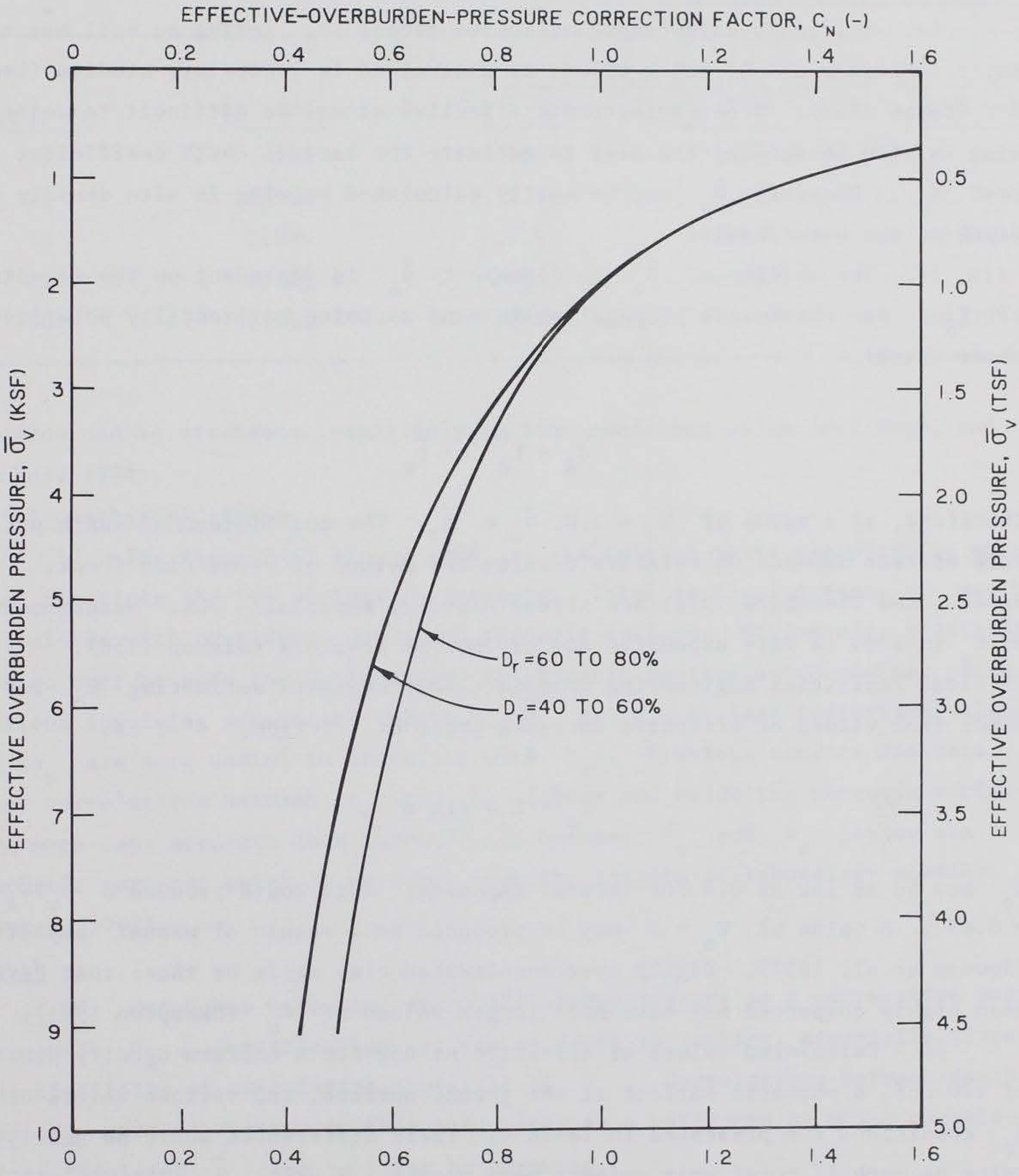


Figure 6. Relationships for correction factors proposed by Seed and Idriss (1981) to normalize N-values to 1.0 tsf

approach 30 percent. Therefore, the type of sampler used is important and included in this study.

Effective overburden stress

58. The cross-anisotropic effective stress $\bar{\sigma}_A$ acting on soil has a major influence on G and N-value, as determined in laboratory studies (Lee and Stokoe 1986). Cross-anisotropic effective stress is difficult to determine in situ because of the need to estimate the lateral earth coefficient at rest K_o . However, $\bar{\sigma}_v$ can be easily calculated knowing in situ density and depth of the water table.

59. The ability of $\bar{\sigma}_v$ to represent $\bar{\sigma}_A$ is dependent on the magnitude of K_o . For shear wave propagation in sand assuming horizontally polarized shear waves:

$$\bar{\sigma}_A = K_o^{0.5} \cdot \bar{\sigma}_v \tag{5}$$

Therefore, at a value of $K_o = 1.0$, $\bar{\sigma}_v = \bar{\sigma}_A$. The coefficient of earth pressure at rest depends on relative density and method of deposition (Peck, Hanson, and Thornburn 1974) and stress history, especially OCR. Measurement of K_o in situ is very expensive and may not be possible (Bishop 1958). US Naval Facilities Engineering Command (1982) suggests estimating K_o for sands from values of effective internal angle of friction, ϕ' , as:

$$K_o = 1 - \sin \phi' \tag{6}$$

K_o may be as low as 0.4 for natural deposits. This would produce a $\bar{\sigma}_v/\bar{\sigma}_A$ = 0.63 . A value of $K_o = 1$ may be produced as a result of manual compaction (Sowers et al. 1957). Highly overconsolidated clay soils or those that have been highly compacted may have much larger values of K_o (Skempton 1961).

60. Calculated values of effective stress for a uniform density profile of 120 pcf, a phreatic surface at the ground surface, and various values of K_o conditions are presented in Table 4. These differences would be nearly twice as much if total unit weights were used.

61. Sykora and Stokoe (1983) concluded that correlations between $\bar{\sigma}_v$ and V_s can be just as accurate as correlations between N and V_s . Unfortunately, $\bar{\sigma}_v$ can only be determined accurately below the phreatic surface. Values calculated above the phreatic surface without the inclusion of soil

Table 4

Comparison of Effective Vertical and Cross-Anisotropic Stresses for an Example Profile

Depth, ft	$\bar{\sigma}_v$, psf	$\bar{\sigma}_A$, psf $K_o = 0.4$	$K_o = 1.0$	$K_o = 1.5$
10	576	364	576	705
25	1,440	911	1,440	1,764
50	2,880	1,821	2,880	3,527
100	5,760	3,643	5,760	7,055
200	11,520	7,286	11,520	14,109

suction can be erroneous, especially in fine sands and silts (Wu, Gray, and Richart 1984).

Total overburden stress

62. The inaccuracy of $\bar{\sigma}_v$ and $\bar{\sigma}_A$ calculated above the phreatic surface restricts the use of these parameters. Total vertical stress σ_v may be calculated with confidence above the phreatic surface. Ultimately, a determination must be made for soils above the phreatic surface as to whether correlations involving inaccurate values of $\bar{\sigma}_v$ or $\bar{\sigma}_A$ or less indicative values of σ_v are more useful to correlate with V_s . Previous studies indicate that correlations between σ_v and V_s (above and below the phreatic surface) are much less accurate than correlations between $\bar{\sigma}_v$ and V_s (below the phreatic surface) which is expected from the results of laboratory studies (Sykora 1987).

Depth

63. Depth has been used by several investigators as a correlative variable with V_s . Justification for use of depth is lacking, especially given the simplicity of calculating a profile of σ_v . Correlations between depth and V_s were considered in this study only as a reference by which correlations involving $\bar{\sigma}_v$ and σ_v may be compared.

Void ratio

64. Void ratio e is the other primary variable that affects V_s (Hardin and Drnevich 1972). Unfortunately, void ratio is cumbersome and rather expensive to determine. It is also subject to errors resulting from

sampling disturbance due to handling and extraction. Some field studies indicate that strong correlations exist between e and V_s with a very high dependence of V_s on e at low void ratios. However, few values of void ratio are measured and reported in geotechnical engineering reports. Therefore, field correlations involving void ratio are not considered feasible for most applications. Rather, field parameters such as SPT N-value are used to represent differences in void ratio.

Relative density

65. Relative density D_r is a parameter that is applicable to cohesionless soils and is calculated using void ratios:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \cdot 100 \text{ (percent)} \tag{7}$$

where

e_{max} = maximum index void ratio
e = void ratio of test sample
e_{min} = minimum index void ratio

A close correlation between D_r and V_s is expected based on the reported correlations between e and V_s and the association of D_r to e.

66. The accuracy of D_r is a function of the accuracy of three measurements of void ratio and consistency of two test procedures (to determine e_{min} and e_{max}). Therefore, it could be concluded that correlations might be more advantageous and more accurate by using void ratio directly. It may be feasible to use measured values of V_s to estimate D_r.

Development of Variable Groupings

67. Once actual correlative variables were selected, it was deemed necessary to develop categories of other important data corresponding to different field conditions. Use of additional variables is expected to enhance correlative accuracy. Development of variable groupings facilitated sensitivity analyses to be conducted to determine the influence of various parameters on measured values of V_s. Categories were divided into two main groups--primary and secondary--a function of importance and proposed use in

correlation analysis. The selected groups are listed in Table 5 and discussed below.

Primary data categories

68. Six primary variables were selected as listed in Table 5. These variables were considered primary because of the proven performance in previous correlations or the combination of common availability and suspected dependency of the variable on a known influential variable. Each of these variables were required to be known for data used in this study and are discussed in more detail herein.

69. Geologic origin. The engineering behavior of a soil deposit is, in part, a function of the depositional or formational process. Different engineering behavior (e.g., V_s) is expected of soils with all physical properties being similar but originating by a different depositional process. These different behaviors are thought to be dependent on inherent differences in void ratio and soil fabric associated with each depositional process. An example of the influence of geologic origin was noted by Sykora and Stokoe (1982). Within a lateral distance of 670 ft, three alluvial deposits (point bar, channel fill, and levee) were identified and tested. Each deposit exhibited a unique profile of V_s, N-value and cone penetration resistance. Similar differences are apparent and can be inferred from subsequent laboratory testing (Kuo and Stokoe 1982) even though significant sample disturbance may have occurred.

70. At a minimum, the use of divisions according to geologic origin appears to allow dividing V_s into ranges. This parameter should be useful for estimating V_s as part of a reconnaissance-type field study.

71. Geologic age. The magnitude of V_s is highly dependent on age although the geologic age of the soil did not have much effect on previous correlations between N-value and V_s (Sykora 1987). Geologic epochs tended to have near-exclusive ranges in V_s associated with them. In general, younger soils had smaller values of V_s and more scatter in V_s correlations as compared with older soils. Geologic age did have a significant effect on V_s versus depth correlations.

72. The primary effect of increasing geologic age is the associated decrease in void ratio. Data used by Tono (1971) indicate that this increase is independent of effective stress. Decreasing void ratio increases both V_s and N-value. Another effect of increasing age is the increase in cementation

Table 5

Categories Used for Data Base Variables

Correlative (Independent) Variables	Primary Variable Categories	Secondary Variable Categories
SPT N-values*	Geologic origin	Atterberg limits**
Effective-stress-corrected N-values*	Geologic age	Coefficient of uniformity†
Effective overburden stress	Soil type	Particle size at 50 percent passing
Total overburden stress	Seismic zonation	Particle size at 10 percent passing
Depth	Overburden (stress) conditions	Percentage of gravel (larger than No. 4 sieve)
Void ratio††	Relative location of phreatic surface	Percentage of sand (smaller than No. 4 sieve, larger than No. 200 sieve (0.075 mm))
Relative density††		Percentage of fines (≤0.075 mm)
		Percentage of clay-sized fines (≤0.005 mm)
		Dry unit weight
		Degree of saturation
		Unconfined strength**
		Angle of internal friction
		Cohesion
		Overconsolidation ratio

* With and without energy normalization (normalized to 60 percent theoretical).
** Cohesive soils only.
† Granular soils only.
†† Quantity of data so restricted that little emphasis placed on correlations with these variables.

between particle grains and change in soil fabric. Cementation increases the stiffness of the soil which increases V_s. Changes in soil fabric with time also tend to increase V_s of the soil. Marcuson and Wahls (1972) determined that increases in G with age resulted from decreases in void ratio and other factors (soil fabric).

73. Comparisons concerning ranges in V_s with respect to geologic age divisions should include the influence of the depth of the soil layer. Larger values of V_s associated with older deposits may result from older deposits being at greater depths, confined at higher effective stresses and therefore exhibiting higher values of V_s. Conversely, young deposits are usually near the surface, confined at low effective stresses, and therefore exhibit lower values of V_s.

74. Soil type. Intuition would suggest that different soil types, under similar conditions, would exhibit different values of V_s. This could be attributed largely to differences in grain-size distribution and the consequent effect on soil particle contacts. Nearly all of the authors who researched field correlations involving V_s differentiated between soil types when performing their analyses. A consensus of the studies suggests that although individual soil types seem to have characteristic ranges in V_s, soil type has little effect on the results of V_s versus N correlations. Soil type did have a more important role in $\bar{\sigma}_v$ and depth versus V_s correlations. Soil type has not generally been incorporated as a variable in equations calculating V_s.

75. Laboratory studies previously discussed have associated changes in V_s directly with changes in void ratio, independent of soil type. However, void ratio is difficult to determine in situ and can only be estimated from D_r when using site-specific N versus $\bar{\sigma}_v$ correlations. Since, in general, increasing the grain size leads to a decreasing void ratio, a premise that increasing grain size increases V_s is applicable. This premise was generally confirmed in previous field correlations. Using this premise and given the ease with which soil type can be determined from disturbed split-spoon samples, soil type was used for correlations in this work. By specifying soil type, then, it was anticipated that a specific range in void ratio was depicted.

76. Seismic zonation. Soils in highly active seismic regions of the United States may exhibit different behavior, in particular, values of V_s,

than soils in aseismic regions under similar in situ conditions. For example, various studies have documented the effect of previous seismic straining on susceptibility of soils to liquefaction. Finn, Bransby, and Pickering (1970) used the results of triaxial and simple shear test to determine that significant increases in resistance to liquefaction occur when samples are subjected to small strains. Conversely, samples were noted to decrease in resistance to liquefaction at very high strains. It was proposed that soil fabric was the intrinsic parameter which affected the difference.

77. Seed, Mori, and Chan (1977) used the results of shake-table tests to confirm conclusions by Finn, Bransby, and Pickering (1970). It was determined that a sample with significant prior straining at a relative density of 54 percent behaved like a sample at 80 percent relative density. Although these studies apply to liquefaction analysis, this type of analysis is a dynamic study which is dependent on void ratio, continuing pressure, and strain amplitude, all of which are factors that affect G.

78. Recent tests conducted by WES both before and after a high-explosive event in a dry, clayey sand alluvial deposit indicate that large shear strains can reduce the V_s of a material significantly. The reduction is greatest near the ground surface and negligible at greater depths.

79. The influence of shear strain amplitude on measured values of G has been identified (Hardin and Drnevich (1972) and Seed, Idriss, and Arango (1983)). This study will consider only geophysical measurements made at low strain levels (less than 10^{-5} in./in.) thereby nullifying this effect. Since shear strain amplitude is an important factor to determine the effect of prior straining on measured values of V_s it was necessary to estimate the magnitude of shear strains produced by earthquakes. Sugimura (1977) estimated the magnitude of shear strain induced by two earthquakes--El Centro, Calif., in 1940 and Taft, Calif., in 1952. The El Centro earthquake had a Richter magnitude of 7.1 and produced estimated shear strains at the ground surface 7.3 miles away on the order of 10^{-3} percent. The Taft earthquake had a Richter magnitude of 7.7 and produced estimated shear strains at the ground surface 25 miles away on the order of 10^{-4} percent. These two examples suggest that most areas within a highly active seismic zone but not close to a fault fall into the category of small strain and, therefore, soils in these areas are expected to increase in V_s or G with repeated loadings.

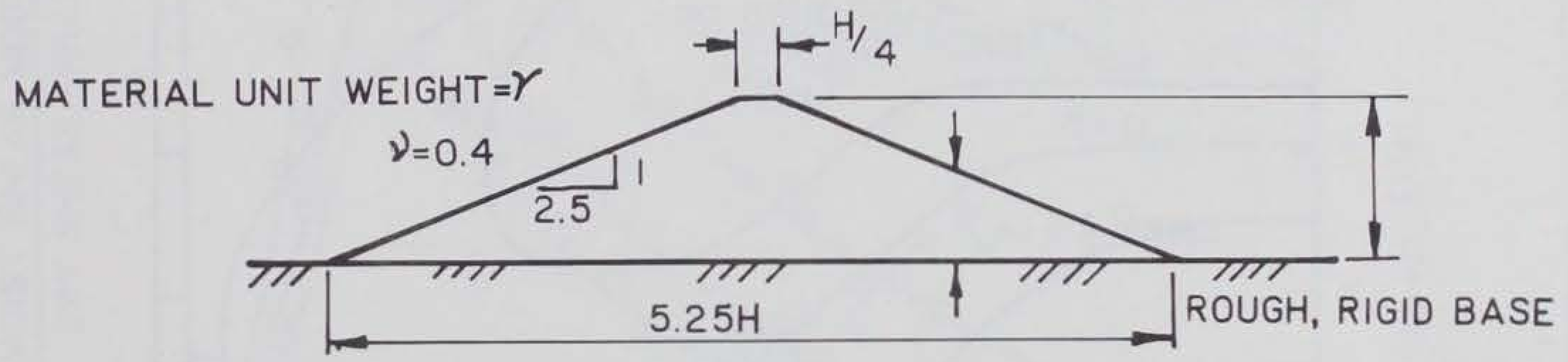
80. In most cases, prior straining resulting from seismic vibrations

should increase values of V_s . Prior straining also is expected to increase values of N and N_1 . Some qualitative factor needs to be established to assimilate the extent of prior straining. The usefulness of seismic zonation is more apparent in correlations involving stress or depth.

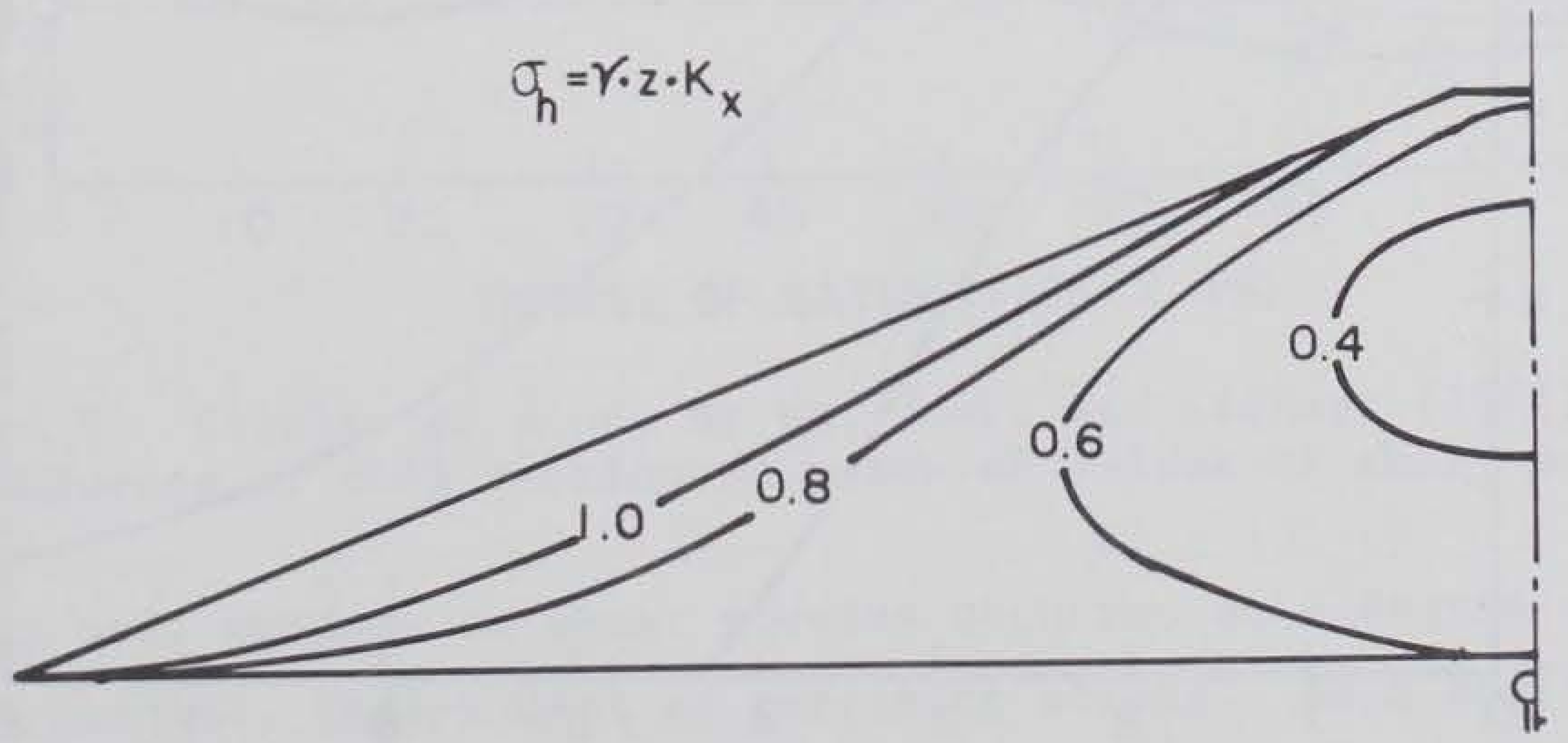
81. Overburden (stress) conditions. Previous discussions have emphasized the importance of stress states on V_s . Also mentioned has been the difficulty in determining $\bar{\sigma}_m$ because of the dependence on K_o . Estimates of K_o are dependent on effective angle of internal friction.

82. Values of V_s are measured in many different stress fields resulting from nonlevel overburden conditions. Numerical solutions using elastic theory allow approximation of vertical and horizontal stress for various overburden conditions. In this study, nonlevel overburden conditions were observed for measurements made in dams.

83. Clough and Woodward (1967) and Perloff, Baladi, and Harr (1967) provide solutions for stresses involving dams. Clough and Woodward (1967) calculated vertical and horizontal stresses for an embankment on a rigid base. Contours corresponding to a constant ratio of $\bar{\sigma}_h/\bar{\sigma}_v$ are shown in Figure 7.



a. Assumed conditions



b. Contours of constant horizontal stress factor

Figure 7. Distribution of horizontal stresses in an embankment on a rigid, rough base, as proposed by Clough and Woodward (1967)

The ratio of $\bar{\sigma}_h/\bar{\sigma}_v$ (= K_x) ranges from less than 0.4 near the center line to greater than 1.0 at the face of the slope. Therefore, zones near the center line are in a state of stress similar to natural normally consolidated materials. These values of K_x do not reflect the influence of residual horizontal stresses produced from compaction (Sowers et al. 1957).

84. Horizontal stresses produced in foundation materials were estimated by Perloff, Baladi, and Harr (1967) based on numerical solutions using the theory of elasticity. A sample of the distribution of horizontal stresses is presented in Figure 8. A zone of primary influence exists directly beneath the embankment. A zone of secondary influence appears to coincide approximately to the slope of the embankment extended beneath the ground surface.

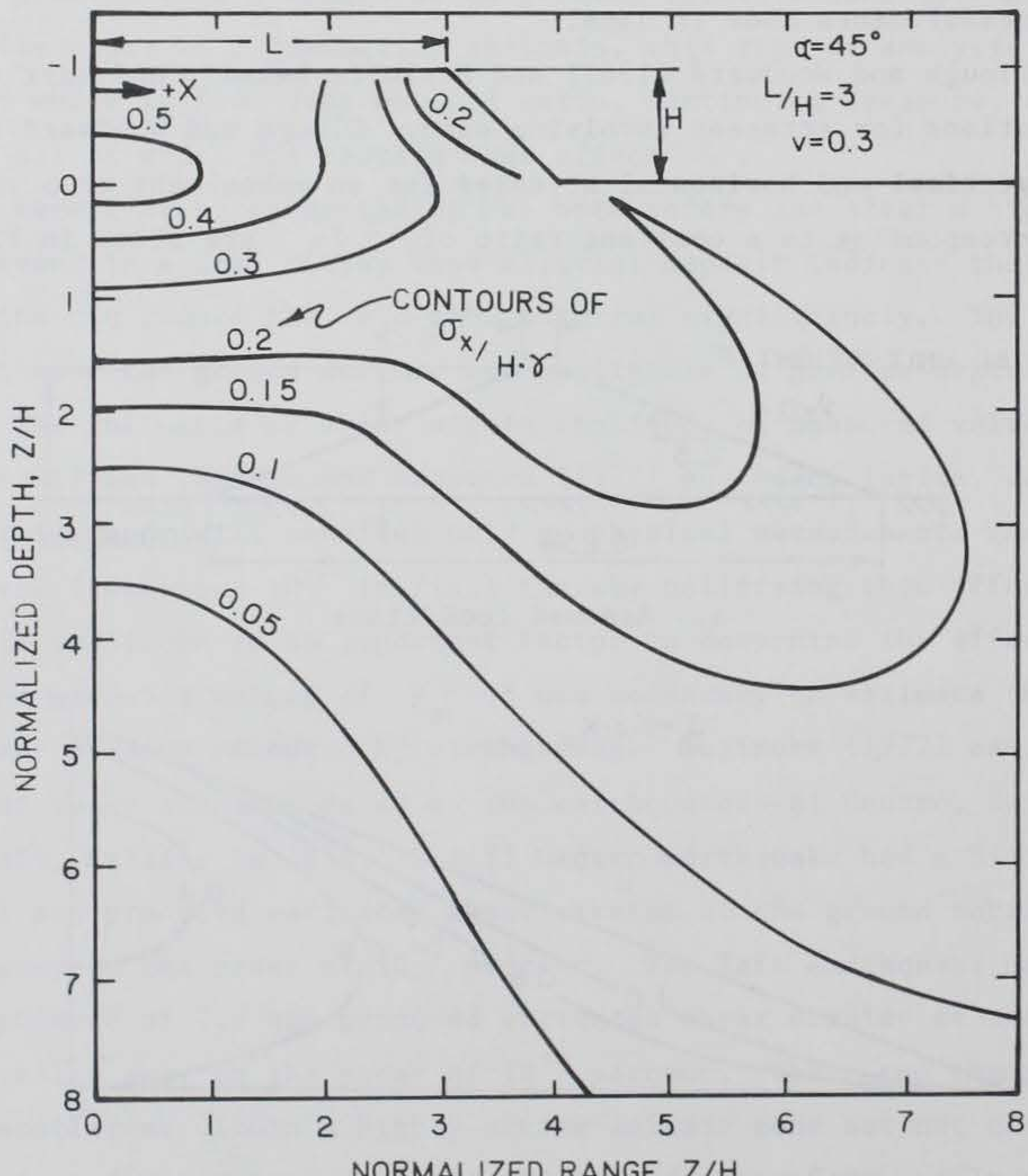


Figure 8. Distribution of horizontal stresses in the foundation produced by an embankment, as proposed by Perloff, Baladi, and Harr (1967)

85. Zones of near-equal horizontal stresses may be selected from a geometric profile based on elastic solutions. This allows estimation of $\bar{\sigma}_h$ (or K_o) to estimate $\bar{\sigma}_A$ either qualitatively or quantitatively.

86. Relative location of phreatic surface. Relative location of the phreatic surface is important for two reasons--primarily because of different major effects on effective stress (and consequently on V_s) and secondarily because of minor influences of free water on SPT N-values. The relative location of the phreatic surface is also important to define a zone of capillarity in which high soil suction is present.

87. Wu, Gray, and Richart (1984) have recognized an extreme influence of soil suction represented as a function of degree of saturation S_r for a particular soil on measured values of G. A normalized relation between G and S_r for one particular silty soil is shown in Figure 9. The maximum

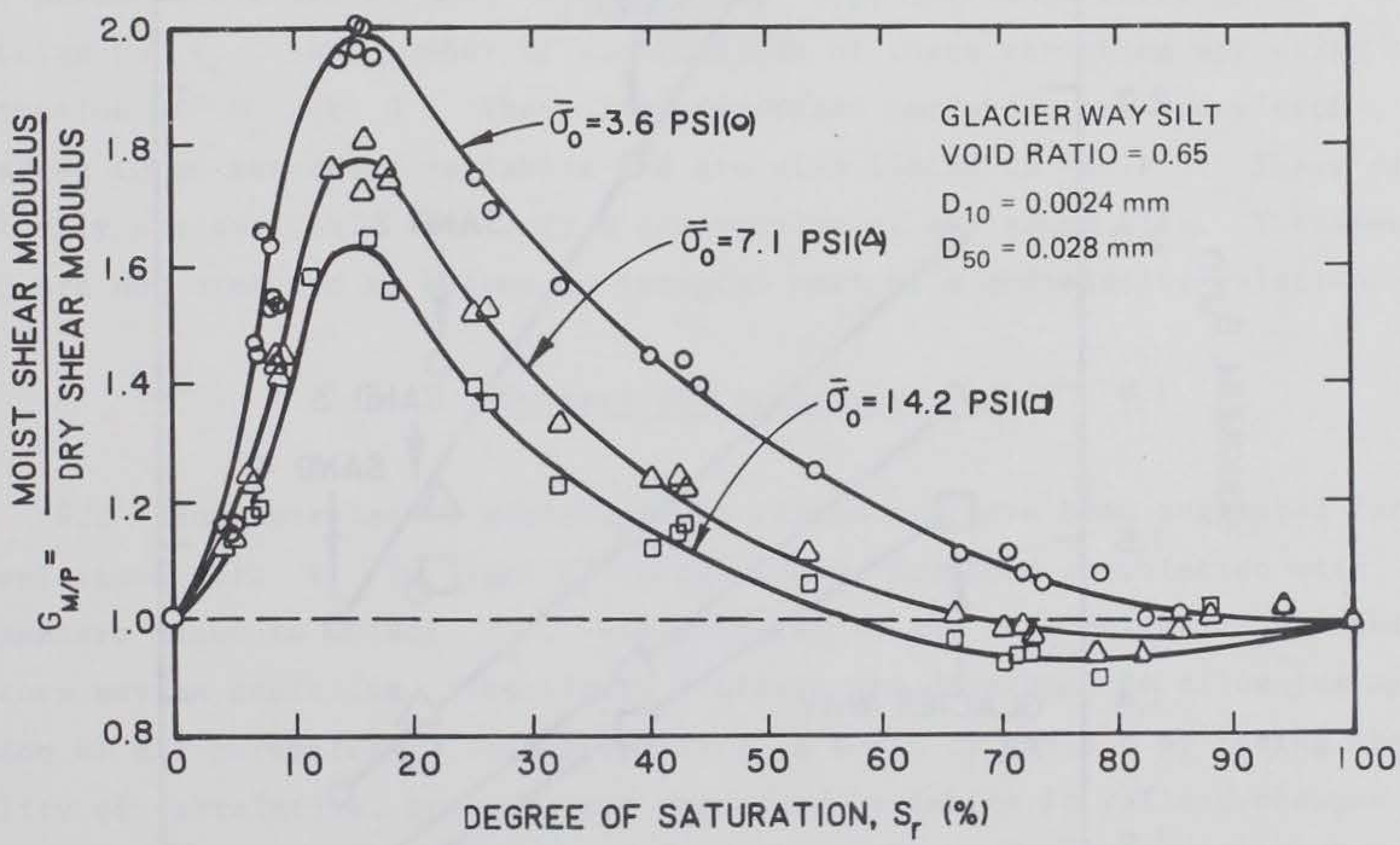


Figure 9. Results of study by Wu, Gray, and Richart (1984) showing influence of soil suction on measured values of shear modulus

influence of soil suction on shear modulus occurred at a degree of saturation of about 15 percent, independent of confining stress. At a confining stress of 3.6 psi, the ratio of G_{moist} to G_{dry} is about 2.0. This ratio is very significant.

88. The sensitivity of the influence of soil suction to soil type (as a

function of percentage of particles at 10 percent passing D_{10}) is shown in Figure 10. A total of five soils were tested by Wu, Gray, and Richart (1984) (Glacier Way Silt and four different sands). The respective void ratios or degrees of saturation at peak G_{M10} for the sands were not reported. However, preliminary relations were proposed per confining stress to relate G_{M10} to D_{10} independent of soil type. Data shown in Figure 10 indicate that the G_{M10} increases as confining stress decreases.

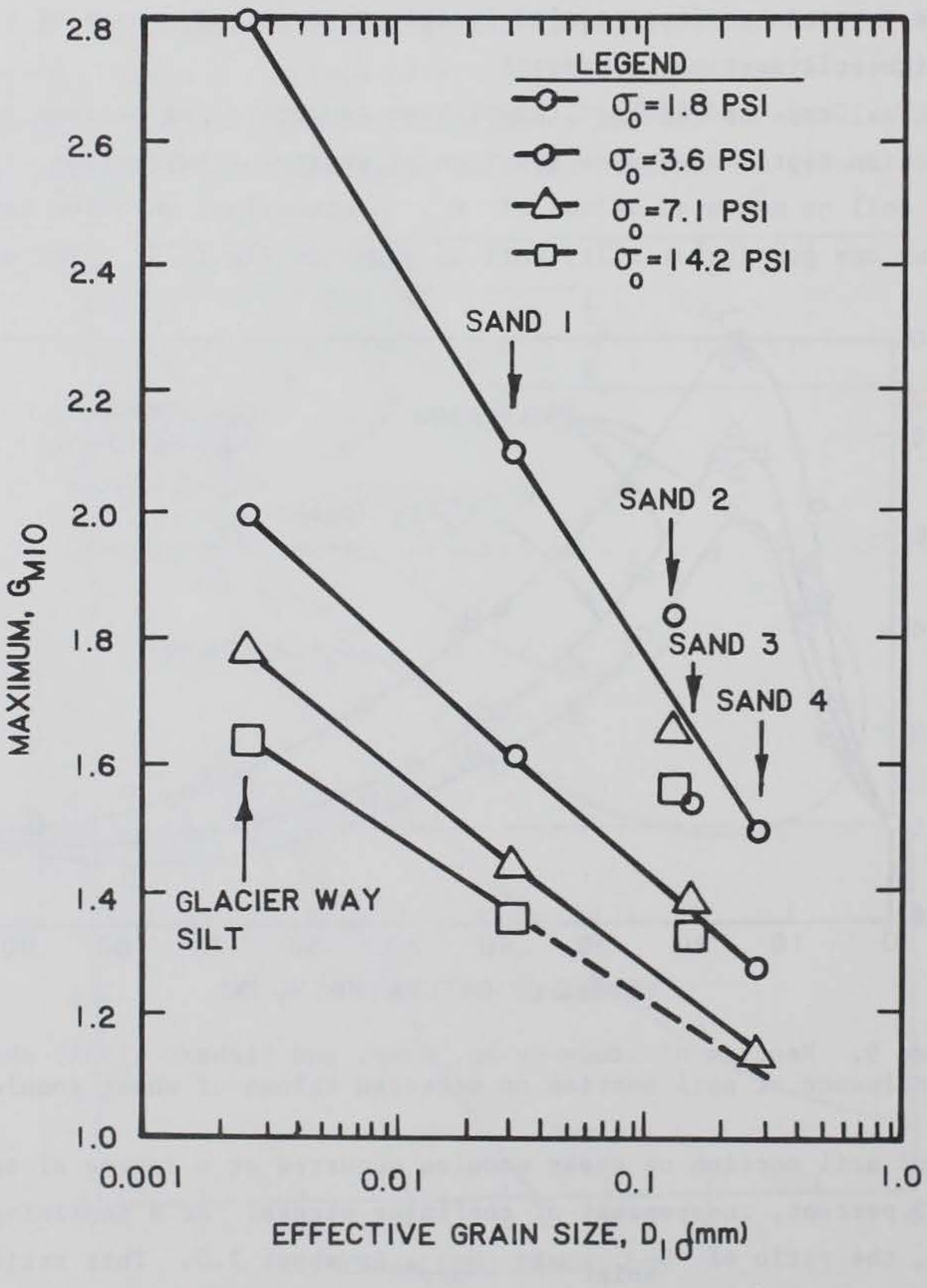


Figure 10. Variation of measured shear modulus as a function of D_{10} (Wu, Gray, and Richart 1984)

89. Based on these results, values of effective stress calculated above the water table and estimated values of V_s based on parameters such as depth or σ_v which do not reflect this elevated-stress state in the capillary zone are subject to large errors. Use of relative depth of the phreatic surface and D_{10} may allow correlations to account for this serious condition.

90. The influence of free water on SPT N-value has been discussed in a previous section. The use of relative depth of the phreatic surface in correlations involving N and V_s is not expected to be very noticeable.

Secondary data categories

91. Many other variables other than those mentioned heretofore are expected to affect correlation analysis even though they may not relate directly to V_s . A number of these variables pertain to physical and engineering properties of soils as determined in the laboratory. Physical property parameters could be used to define soil type more accurately as it pertains to V_s . Any number of combinations of these variables may exist for estimation of V_s or G . These less-important variables are henceforth referred to as secondary variables and are also listed in Table 5. These data typically are available from only a few samples at any given site. Therefore, they are not expected to become an integral part of a correlative relation.

Summary of Parameters

92. Many correlative variables and parameters have been suggested for correlations with V_s in light of recognized or proposed association with parameters known to affect V_s . The integration and interaction of all these factors may be confusing. Therefore, a matrix was developed to allow juxtaposition of all parameters. This comparison is shown in Table 6 by rating the ability of correlative, primary, and secondary variables to reflect changes or differences in parameters known to affect V_s . Each of the parameters known to affect V_s and variables suggested for correlations have been discussed previously in this report.

93. Most correlative variables have very limited correlation with factors which affect V_s . This finding justifies the need to include other types of distinctions to better qualify the analysis. Only SPT N-values tend to reflect changes in a number of factors. N-values normalized to effective vertical stress are thought to better reflect most less-influential parameters

because N_1 and $N_{1,60}$ are less sensitive to $\bar{\sigma}_v$ and, thereby, more sensitive to other factors. Based on this qualitative comparison, SPT N-values appear to be the best single variable to reflect overall changes in parameters which affect V_s. However, effective vertical stress may be better than N_1 or $N_{1,60}$.

94. Primary variables appear to contribute significantly to the estimation of V_s. As summarized in Table 6, geologic age, geologic origin, soil types, and relative location of the phreatic surface all reflect changes, to varying degrees, in a number of parameters which affect V_s. The inclusion of primary variables in correlations appears necessary to improve accuracy.

95. Secondary variables are much less important than primary variables to reflect changes in parameters which affect V_s, as indicated in Table 6. In many cases, secondary variables only slightly enhance one or two particular parameters. The use of secondary variables should be restricted to those specific cases as a way to enhance primary variables.

Table 6

Comparison of Parameters Used in This Study and Parameters Known to Affect V_s

Parameters, B	Parameters, A								
	σ_m*	Void Ratio	S_r**	OCR**	Soil Suction	Cementation†	Soil Fabric	Particle Shape†	Prior Straining††
Correlative Variables									
N, N_{60}	3	4**; 3†	3	3	4	3	4	4	4
N_1, $N_{1,60}$	5	3**; 2†	2	2	5	2	3	4	4
Depth	2	4	5	5	5	5	5	5	5
Total vertical stress	2	4	5	5	5	5	5	5	5
Effective vertical stress	1	4	5	5	5	5	5	5	5
Void ratio	5	--	5	4	4	5	5	5	4
Relative density	5	NA	5	4	4	5	5	5	4
Primary Variables									
Geologic origin	4	3	4	3	5	3	3	3	4
Geologic age	4	2	5	4	5	2	2	3	4
Soil type	5	2	5	5	2	4	4	4	5
Relative location of phreatic surface	2	4	1	5	1	4	5	5	5
Overburden conditions	3	5	5	4	5	5	4	5	5
Seismic zonation	5	5	5	5	5	5	5	5	3
Secondary Variables									
Atterburg limits	5	5	5	5	4	5	5	5	5
Gradation parameters	5	3	5	5	1	4	4	5	5
Dry unit weight	5	3	5	5	5	5	5	5	4
Degree of saturation	5	5	--	5	1	5	5	5	5
Unconfined strength	5	3	2	3	3	3	4	4	4
Confined strength	5	2	2	3	3	3	4	4	4
Overconsolidation ratio	4	4	5	--	5	5	3	5	5

Note: 1 = Parameter B closely reflects changes/differences in parameter A.
2 = Parameter B moderately reflects changes/differences in parameter A.
3 = Parameter B somewhat reflects changes/differences in parameter A.
4 = Parameter B may reflect changes/differences in parameter A.
5 = No correlation or uncertain.
* Not including soil suction (typically not measured).
** Cohesive soils only.
† Granular soils only.
†† Not of a level to cause changes in void ratio.

PART III: DATA BASE SYSTEM

96. The basic framework used to construct, manage, and manipulate the data base is described herein. This part is divided into two sections: Data Base System and Data Accumulation. The former is used to describe computer hardware and software. The latter section summarizes the screening processes for selection of data used in this study.

Data Base System

Computer hardware

97. An International Business Machines (IBM) Personal Computer (PC) AT® was used to conduct this study. Internal configuration consisted of an Intel 80287 coprocessor chip, internal clock, 1.2 megabyte (mbyte) floppy drive, 30 Mbyte hard disk, 1,152 kilobyte (kbyte) of Random Access Memory (RAM) and a 64 kbyte internal print buffer.

Computer software

98. IBM version 3.10 of the Disk Operating System (DOS) was installed on the PC/AT for general operations and initiation of other software packages. The software package "dBase III® Plus" from Ashton-Tate was used for data base management and manipulation. The software package SuperCalc® 4 was used to plot data.

99. A program named STRESS.F was written by the author for this study to calculate total and effective stresses at various depths and associated effective stress correction factors. This program greatly reduced the amount of time required for data reduction. STRESS.F was written in the Fortran programming language. More details of assumptions and algorithms used in this program are described and source codes are provided in Appendix B. The program STRESS.F was compiled using IBM Professional Fortran (Ryan-McFarland Corporation), which is also operated using IBM DOS.

Data Accumulation

100. The prospect for success with this study hinged on the availability and incorporation of an unprecedented quantity of reliable data. Therefore, a great deal of time and effort was expended to accumulate high-quality

information for the seismic data base. This was accomplished by personal visits to various organizations and contributions of personal contacts to sift through project files and copy desired information.

101. The process of selecting data for this study is described in this section. It begins with a review of criteria imposed that is intended to maintain a certain degree of reliability. The sources of information then are disclosed along with a compendium of occurrence. Actual project names and association of data with sources will remain anonymous throughout this study at the request of contributors.

Selective criteria

102. The quality of data used in correlation analysis was of utmost importance to this study. As methodology for determination of dynamic soil stiffness in situ has evolved to a standard (ASTM D 4428 (1987b)) over the years (e.g., Ballard 1976, Auld 1977, Hoar and Stokoe 1977, Stokoe 1980, and Woods 1986), several inaccurate or less-desirable techniques and practices have been abandoned. Obviously, data are available throughout this period and may involve varying degrees of reliability. Therefore, criteria were selected to discriminate data which, in the opinion of the investigator, would detract from the validity, reliability, and benefits of the study. Criteria were selected based on objective thought.

103. Particular criteria used to select data for this study are summarized below:

- a. Seismic arrival times measured using the crosshole method in general compliance with state-of-the-art practice.
- b. Mechanical (nonexplosive) downhole seismic sources with low shear-strain amplitude.
- c. Crosshole arrays in embankments oriented parallel to the axis of the dam.
- d. Hole-to-hole spacings of less than 25 ft.
- e. Known ground-water conditions.
- f. Defined geologic history (origin and age of deposits).
- g. Known overburden conditions.
- h. Known soil types (general or specific).
- i. Geographic location (seismic zonation).

Criteria e through i are primary variables defined in Table 5. Justification for each of these criteria including deviations from standards are provided in the following paragraphs.

104. Crosshole method. The crosshole seismic method provides, unequivocally, the most accurate profile of V_s to determine G with depth (Woods 1986). The preponderance of shear wave velocity data has been measured using other methods, namely surface seismic refraction (surface-to-surface propagation) or downhole seismic (surface to subsurface propagation). Despite a considerable amount of data available from surface refraction tests, the inherent lack of accuracy due to V_s averaging associated with this technique was not considered to lend itself appropriately to V_s correlations. In fact, the accuracy (more specifically, the accurate variation in V_s with depth) desired for V_s correlations is questionable in downhole measurements incorporating direct travel time measurements. (See Stokoe (1980) or Patel (1981) for a description of the various travel time measurements used with downhole methods.) A significant amount of V_s averaging is inherent when using direct travel times with the downhole test. Of late, engineers have presented V_s derived from both direct and pseudointerval downhole travel times. Pseudointerval travel times generally reflect more accurately the variations in V_s with depth than direct downhole travel times. Recent methods developed at The University of Texas at Austin (Stokoe 1987*) indicate that very accurate profiles may be produced using downhole techniques.

105. Actually, differences between results of downhole and crosshole techniques are more fundamental. As shown in Figure 11, Stokoe (1980) indicates primary differences exist in particle motion for the two techniques. Crosshole methods incorporating downhole mechanical sources (used primarily to date) incorporate vertically polarized shear waves which would be governed by the effective stresses in the vertical and in-line horizontal directions (Knox, Stokoe, and Kopperman 1982). In theory, the state of stress along the entire path in a crosshole array (at equal depth) is equal for level-ground conditions. Downhole methods incorporate horizontally polarized shear waves traveling in a near-vertical direction. The effective vertical confining stress i.e., $\bar{\sigma}_v$ increases with depth. Furthermore, the inclination of the near-vertical propagation significantly affects the stress state. This is more critical at shallow measurement depths. Therefore, it can be concluded that exclusive use of data collected using crosshole seismic methods would

---

* Personal Communication, 1987, K. Stokoe, Professor of Civil Engineering, University of Texas at Austin, Austin, Tex.

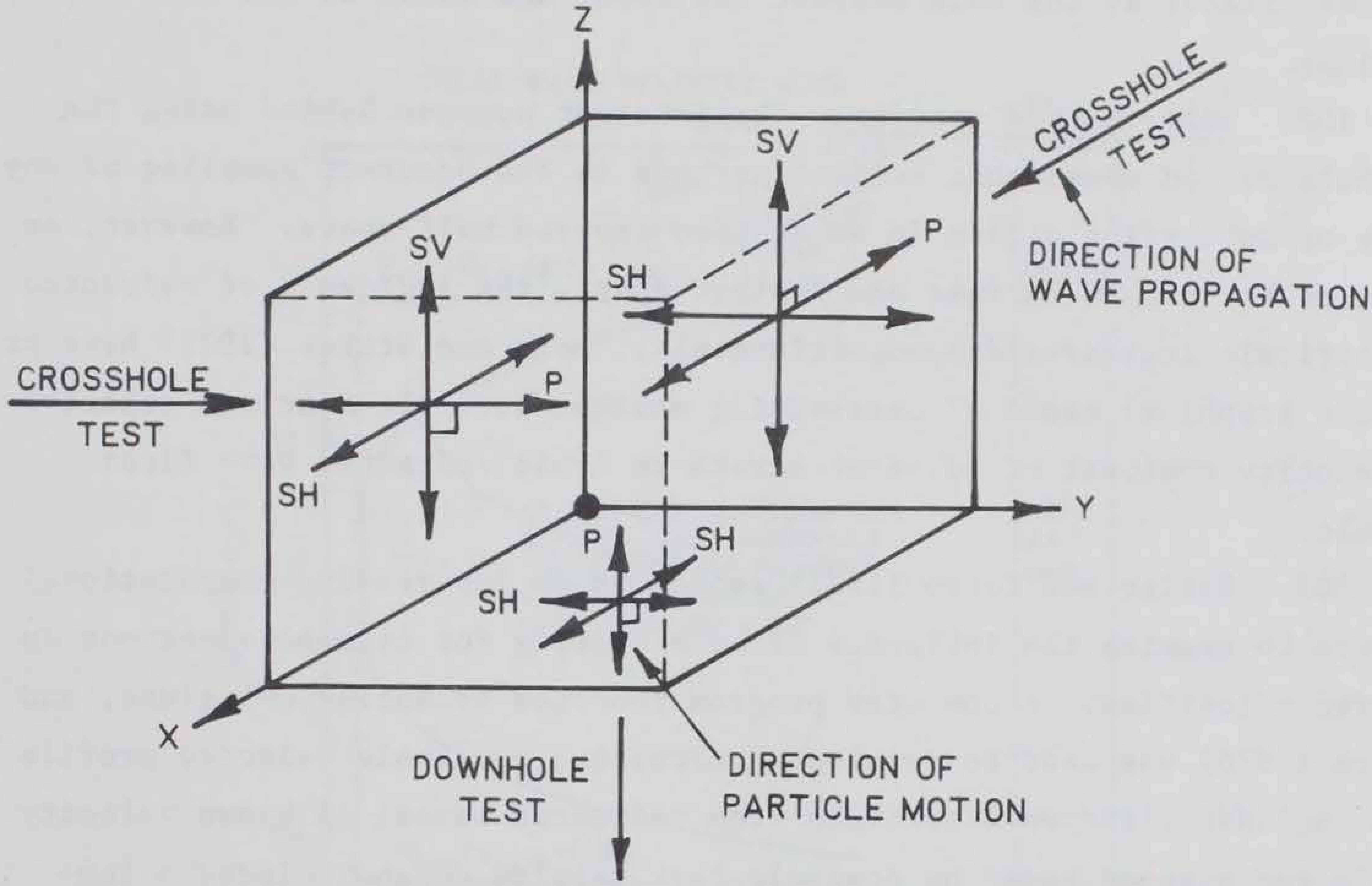


Figure 11. Wave propagation directions and particle motion directions possible using crosshole and downhole techniques (from Stokoe 1980)

benefit correlations made for this study by providing a more detailed and consistent profile of V_s .

106. Mechanical sources. Data were available from sites at which explosive downhole sources were used to create seismic waves. Several studies have concluded that explosive sources produce essentially the same values of V_s in situ as mechanical (vertically polarized) sources (e.g., Woods 1978). These studies are convincing. However, explosive-source crosshole measurements were not included in this study initially. This decision was made because explosive sources are not expected to produce a true vertically polarized shear wave. Rather, a multidimensional wave is produced. Representation of $\bar{\sigma}_A$ for shear waves polarized at angles oblique to vertical is difficult, more so than assuming that $\bar{\sigma}_v = \bar{\sigma}_A$. The use of explosive source measurements may be added at a later date.

107. Parallel arrays. Crosshole arrays in an embankment should be linear and parallel to the dam axis. This is necessary to ensure near-uniform stress states along the wave path at a uniform depth. If the array were situated perpendicular to the axis of the dam, the vertical (horizontal) stresses

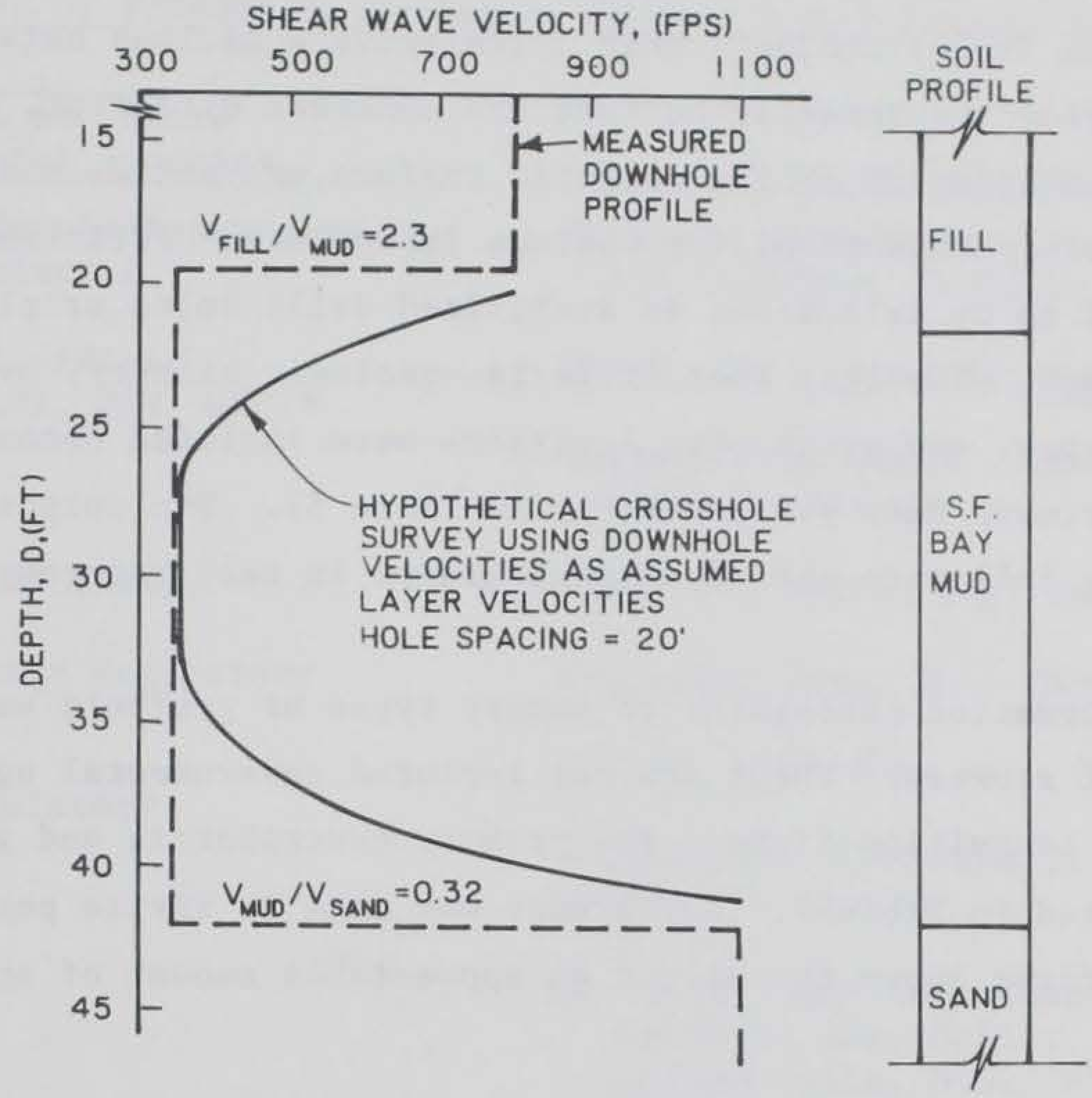
would be greater at the hole nearest the crest and least at the hole furthest downslope.

108. Hole-to-hole spacing. The inherent purpose behind using the crosshole method over other seismic methods is the accurate sampling of any number of subsurface strata in an assumed layered half-space. However, as the boreholes are placed further and further apart, the incidence of refracted wave arrivals increases disproportionately. Hoar and Stokoe (1977) have presented a graphical means of determining maximum borehole spacings relative to the velocity contrast of adjacent strata to avoid refracted wave first arrivals.

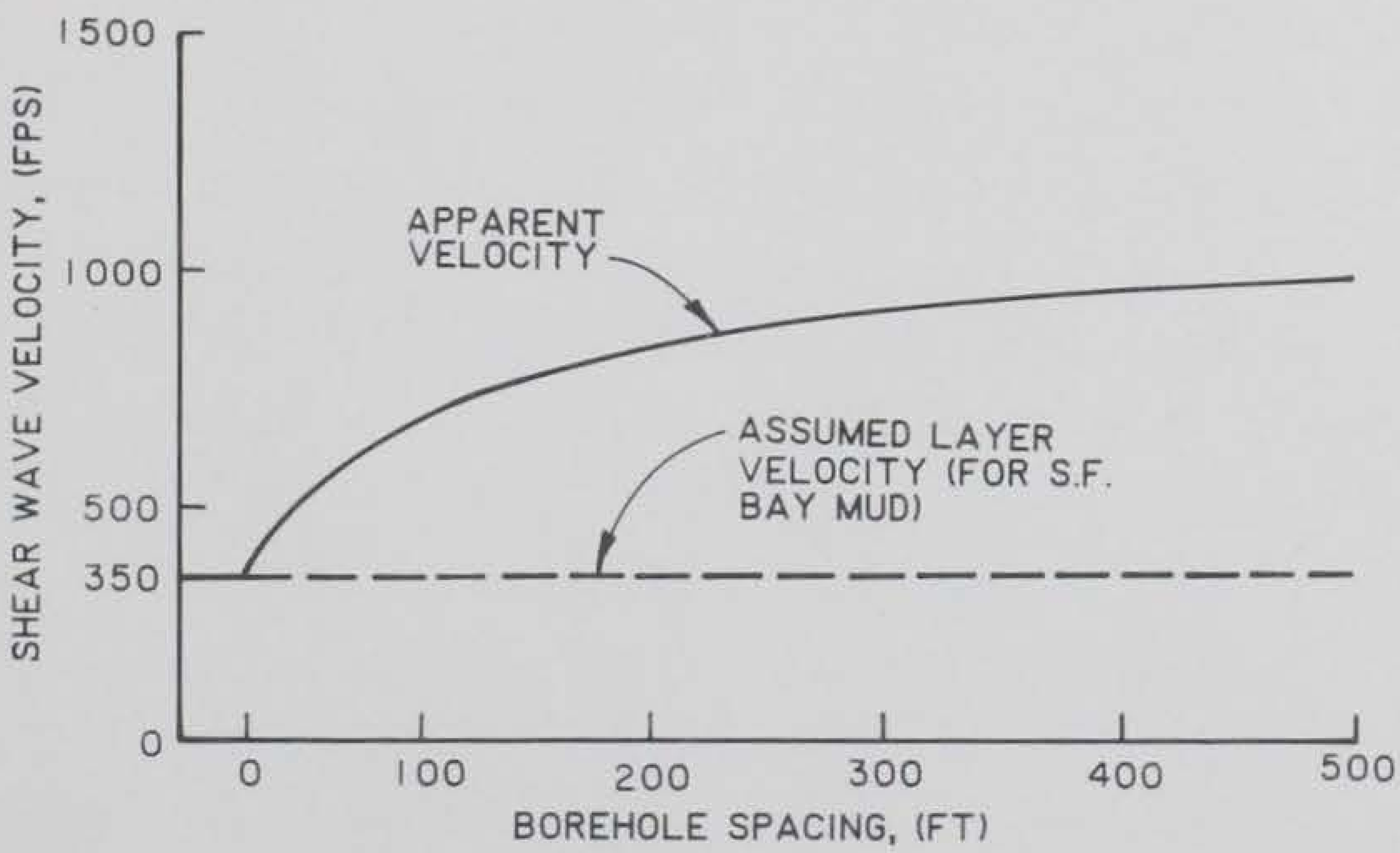
109. Butler and Curro (1981) conducted an interesting computational analysis to examine the influence of hole spacing for crosshole methods on measured velocities. A computer program reported by Butler, Skoglund, and Landers (1978) was used to develop a calculated crosshole velocity profile which includes first wave arrivals from refracted waves. A known velocity profile was assumed based on downhole test results which included a low-velocity layer located between two higher-velocity layers. The results of the computations are shown in Figure 12a indicating that measured values of velocity only represent the true velocity over about 40 percent of the layer thickness. This inaccuracy is expected when the hole spacing is too large (in this case, 20 ft) with respect to the minimum velocity expected, thickness of low velocity zones, and relative velocities of adjacent layers. Butler and Curro (1981) also offer a relationship between measured V_s and borehole spacing, shown in Figure 12b, to indicate the magnitude of potential error and rapid decay of accuracy with increase in spacing.

110. A maximum threshold distance of 25 ft was adopted for this study. Further qualitative examination of hole separation and ratio of velocities in adjacent layers was conducted for each individual study to determine if hole spacings were too great to ensure adequate accuracy of measured values. The program reported by Butler, Skoglund, and Landers (1978) could not be incorporated because raw data on hole verticality measurements were rarely available.

111. Ground water. Location of the phreatic surface is very important in several respects. It is requisite to calculate effective stress which, in turn, is used to calculate values of N_1. It has also been shown (Wu, Gray, and Richart 1984) that the effect of capillary rise on G can be tremendous. Capillary rise is measured up from the phreatic surface. Of less importance



a. Hypothetical results compared with profile



b. Sensitivity of velocity to hole spacing

Figure 12. Example of effect of borehole spacing on resulting velocity profile (from Butler and Curro 1981)

is the direct influence of free water on measurement of N and V_s . Several authors reported differences in N-values above and below the phreatic surface. Sykora and Stokoe (1983) indicate that correlative equations between N and V_s above and below the phreatic surface are somewhat different.

112. Known location of the phreatic surface was set as a selective criterion in this study because of its obvious importance. Furthermore, the values of depth had to be determined in stabilized drill holes or piezometers.

113. Others. The last four criteria--geologic history, overburden conditions, soil types, and geographic location--were included because they apply to predefined primary data groups (refer to Table 5). For purposes of this study, data must fall into one of several groups in each category.

Sources

114. Information pertaining to sundry types of projects was acquired from a number of sources. These sources included governmental agencies and several private consulting firms. The primary contributors and general sources are listed in Table 7. An attempt was made to invite participation of all consulting firms known to collect an appreciable amount of applicable V_s data.

Table 7

Listing of Primary Contributors of Data and Information Sources

Federal and State Governmental Agencies	Private Consulting Firms
State of California-Department of Water Resources	Converse Consultants (San Francisco and Pasadena, Calif.)
USACE district offices (SPK, SPL, LMS, NPW, MRK)*	ERTEC, Inc. (Long Beach, Calif.)
USACE WES	Geotechnical Engrs., Inc. (Boston, Mass.)
US Department of Energy	Haley & Aldrich, Inc. (Boston, Mass.)
US Federal Energy Regulatory Commission	Harding-Lawson, Inc. (Novato, Calif.)
US Nuclear Regulatory Commission	Shannon & Wilson, Inc. (Seattle, Wash.)
	Dr. Kenneth H. Stokoe II, P.E. (Austin, Tex.)
	Woodward-Clyde Consultants (Oakland and Santa Ana, Calif., Chicago, Ill., Clifton Hills, N.J., Plymouth Meeting, Pa.)

* SPK = US Army Engineer District, Sacramento.
SPL = US Army Engineer District, Los Angeles.
LMS = US Army Engineer District, St. Louis.
NPW = US Army Engineer District, Walla Walla.
MRK = US Army Engineer District, Kansas City.

PART IV: DATA REDUCTION

115. Once data were acquired from the various sources, the formidable task of associating parameters to measured values of V_s remained. This "data reduction" was contingent somewhat on the approach adopted for the overall study described previously. A high-quality and flexible data base was desired. Therefore, it was deemed necessary to impose criteria to winnow usable data even beyond criteria adopted for project selection described previously. Specific guidelines for data reduction evolved, as necessary, which remained consistent throughout the reduction phase. At the outset of this study, it was determined that two primary methods of interpretation were appropriate for correlations desired. Therefore, two independent data bases were actually created.

116. Variables were separated into three groups (refer to Table 5) for correlation analysis--correlative, primary, and secondary variable categories. Correlative variables are measured discrete random variables or calculated continuous random variables. Typically, little "reduction" is required. However, some guidelines were necessary to associate measured correlative variables with each value of V_s . Primary variables contain exclusive categories for grouping of data. Secondary variables provide additional information thought to be beneficial to correlations. Sections pertaining to other aspects of data reduction follow.

Primary Methods of Interpretation

117. A decision was made to prepare data to correlate various forms of SPT N-value and various forms of stress with V_s . These two sets of independent correlative variables are distinctly different. SPT N-values are measured values representing discrete random variables which may exist at any combination of depths and locations at a particular site. Conversely, stress is a calculated value (continuous random variable) which may be determined at any point given certain known values. It seems intuitively obvious that a different approach to data reduction is appropriate for these discrete and continuous independent variables. Therefore, two primary methods of interpretation were adopted for this study resulting in two individual data bases to provide the best means of analysis possible.

118. One data base, referred to as depth-respective V_s , was created expressly for the purpose of correlating continuous random variables (such as stress) with V_s . A data set was said to correspond to depths at which V_s was measured. Stress was then calculated for these respective depths. This one-to-one correspondence between stress and V_s is unique. All primary variable categories were available for each depth of V_s measurement. Secondary variables were available at other depths and thereby required some guidelines to associate with values of V_s . A summary of all variables and information included in the depth-respective V_s data base is provided in Table 8.

119. The second data base, referred to as SPT-respective V_s , was created expressly for the purpose of correlating the discrete random variables N , N_1 , N_{60} , and $N_{1,60}$ with V_s . A data set was said to correspond to each depth (per borehole) at which an SPT was performed. Shear wave velocity was then interpreted from the overall V_s profile with respect to soil-type layers and interpreted refracted wave arrivals. For example, if values of V_s were measured at 5-ft-depth intervals and an SPT was performed at middepth of a particular depth interval, the value of V_s said to correspond to that N-value was the average of the two adjacent measured values (if wholly contained in a uniform-material layer). If an SPT was performed at the same depth as a V_s measurement, a measured-to-measured correspondence exists. All primary variables categories are available for each depth of SPT measurement. Secondary variables typically are associated with depths of SPT measurement since disturbed samples from the SPT sampler typically are used for physical property tests. A summary of all variables included in the SPT-respective V_s data base is provided in Table 9.

120. Some secondary variables are available at depths and locations not associated with V_s or SPT measurement. These variables typically were made available from undisturbed sampling in the same or adjacent boreholes. In these cases, guidelines were developed to associate these parameters with depth-respective or SPT-respective values of V_s , within reason.

Correlative Variables

121. Guidelines established to create data sets for both depth- and SPT-respective data bases are presented in the following paragraphs. The

Table 8

Data Base Structure for Depth-Respective V_s Correlations

General Information	Correlative Variables	Primary Variables Categories	Secondary Variables Categories
Project code	Depth, ft	Overburden conditions	Atterburg limits
Geophysical method	Effective vertical stress, tsf	Relative location of phreatic surface	Coefficient of uniformity
Measurement quality	Total vertical stress, tsf	Geologic origin	Particle size at 50 percent passing
Year	Shear wave velocity, fps	Geologic age	Particle size at 10 percent passing
Hole deviation	Compression wave velocity, fps	Soil type	Percentage of gravel (larger than No. 4 sieve)
Drill technique	Shear modulus--field, M_{psf}	Seismic zonation	Percentage of sand (smaller than No. 4 sieve, larger than No. 200 sieve (0.075 mm))
Distance V_s to lab data hole	Shear modulus--lab, M_{psf}		Percentage of fines (≤0.075 mm)
	Confining pressure for G_{lab}, psi		Percentage of clay-sized fines (≤0.005 mm)
			Dry unit weight, pcf
			Degree of saturation, percent
			Unconfined strength, ksf
			Angle of internal friction, degrees
			Cohesion, ksf
			Overconsolidation ratio

Table 9

Data Base Structure for SPT N-Value Versus V_s Correlations

General Information	Correlative Variables	Primary Variables Categories	Secondary Variables Categories
Project code	Shear wave velocity, fps	Overburden conditions	Atterburg limits
Geophysical method	Compression wave velocity, fps	Relative location of phreatic surface	Coefficient of uniformity
Measurement quality	Shear modulus--field, M_{psf}	Geologic origin	Particle size at 50 percent passing
Year	Shear modulus--lab, M_{psf}	Geologic age	Particle size at 10 percent passing
Depth	Confining pressure for G_{lab}, psi	Soil type	Percentage of gravel (larger than No. 4 sieve)
Hole deviation	N-value, blows/ft	Seismic zonation	Percentage of sand (smaller than No. 4 sieve, larger than No. 200 sieve (0.075 mm))
Drill rig type	N_1-value, blows/ft		Percentage of fines (≤0.075 mm)
Drill technique	N_{60}-value, blows/ft		Percentage of clay-sized fines (≤0.005 mm)
Distance from SPT to V_s	$N_{1,60}$-value, blows/ft		Dry unit weight, pcf
SPT energy ratio	Effective vertical stress, tsf		Degree of saturation, percent
SPT spoon type	Effective-stress SPT correction factor		Unconfined strength, ksf
SPT spoon correction			Angle of internal friction, degrees
			Cohesion, ksf
			Overconsolidation ratio

development of depth-respective data is somewhat trivial. The development of SPT-respective data required more thought and interpretation.

Standard Penetration Test

122. Use of SPT data involves dealing with inaccuracies due to inconsistencies in techniques, equipment, and procedures. Therefore, guidelines were developed to minimize the effect of those inconsistencies for correlation of N, N_1, N_{60}, and $N_{1,60}$ with V_s as discussed herein. A discussion of criteria used to associate values of N with V_s is also included.

123. Inconsistencies in equipment. Many types and variations of drill equipment exist which influence the SPT. This equipment includes hammer configuration, weight (size) of drill rod, size and weight of rope, size and rotation speed of cathead, hammer trip system, etc. One of the most important differences has been reported to be the type of SPT hammer (and associated release system). A study by Seed et al. (1985) was used to account for the differences produced by hammer types, when that information was available, to calculate N_{60} (or $N_{1,60}$). Drill rig types, although recorded, were not used initially. Likewise, the interior dimensions of split-spoon samplers were recorded, when known. Values of N_{60} (or $N_{1,60}$) were not corrected for differences proposed by Seed et al. (1985) due to significant scatter in data for the proposed correction factor. Corrections can be incorporated in the future very simply.

124. Inconsistencies in techniques. Seed et al. (1985), and others, have suggested specific techniques to conduct the SPT in order to maintain consistency, particularly in liquefaction analysis. These techniques include use of uncased boreholes, bentonite drilling mud, and rate of blows. Although considered important, it was not feasible to discern all conditions for projects used in this study. The general type of drilling technique used was included in the data base. Therefore, division of data among different general drilling techniques such as rotary-mud versus hollow-stem-auger advancement can be made.

125. Inconsistencies in procedures. A criterion was needed for dense cohesionless deposits requiring a large number of hammer blows to drive the sampler. Standards require that the sampler first be driven an initial 6 in. to seat it, then driven another 12 in. while counting the number of hammer blows. In very dense deposits, the sampler may not be driven the full 18 in. with a reasonable number of hammer blows. In such instances, the number of

blows to drive the sampler a particular distance less than 18 in. was usually recorded by field personnel. (Some firms increased the hammer weight for very dense deposits, but only N-values accumulated using the standard 140-lb hammer were used in this study.) The standard for acceptance of SPT's not driven the complete 18 in. was that the sampler was to be driven the initial 6 in., as specified, plus a minimum of 3 in. of the specified final 12 in. The record number of blows was calculated from linear extrapolation of blows for the partial distance beyond 6 in. In this manner, N-values were adjusted for partial driving distances in dense deposits.

126. Association with V_s. SPT N-values were recorded at depths corresponding to the mid-height of the (assumed) final 12-in. drive. If more than one borehole from the crosshole set, or even boreholes within a reasonable distance from the crosshole set, were used to conduct the SPT, all values of N were recorded in the data base. Therefore, more than one value of N (or N_1, N_{60}, or $N_{1,60}$) could be available for any given depth.

127. Determination of a corresponding V_s for SPT-respective correlations involved some interpretation when depths of seismic measurements did not correspond to depths of N-values. This interpretation was complicated when considering different seismic methods of V_s determination, lateral homogeneity of the soil layer, and layer inclination. In crosshole tests, travel times of horizontally propagating shear waves are measured between the source and receivers at equal depths. However, if the soil layer was not horizontal or the SPT was performed at a distance offset from the crosshole array, the two tests may have sampled different soil layers, even at the same testing depth. If this were the case, only N-values and V_s in the same soil layer were used in the analyses.

128. Typical depth intervals for measurement of V_s were 5 and 10 ft. If the soil layer were thick enough such that more than one V_s measurement from the same crosshole array was made in the soil deposit at a different depth, V_s was said to vary linearly with depth between the measured values of V_s. If only one value of V_s were measured within a given layer, N-values within a 1-ft-depth range were correlated with that value of V_s. Thus, given that an N-value was measured at a depth between the depths of two V_s measurements, all in the same soil deposit, a V_s corresponding to that N-value at the same depth could be determined.

129. Some difficulty was encountered when effective-vertical-stress-correction factors C_N for N-value were calculated. As proposed by Seed and Idriss (1981), C_N is dependent on $\bar{\sigma}_v$, and to a certain extent on relative density. Two categories of data were represented--40 to 60 percent and 60 to 80 percent relative density. However, as Marcuson and Bieganousky (1977) concluded, relations between $\bar{\sigma}_v$, D_r , and N-value vary for different sands. Therefore, without extensive laboratory testing for soils at each site, relative densities could not be estimated accurately.

130. Given that the dependence of C_N on D_r is significant at high stresses, it was decided to use the two categories of D_r proposed by Seed and Idriss (1981) to determine C_N for calculation of N_1 and $N_{1,60}$. A relationship between $\bar{\sigma}_v$, and N corresponding to D_r = 60 percent was selected as the means to separate soils above or below 60 percent relative density. This relationship is presented in Figure 13 based on results from simplification of a proposed relationship by Marcuson and Bieganousky (1977) and comparisons with other reported relationships. All granular soils for which N versus $\bar{\sigma}_v$ plotted above the line were said to fall into the 60 to 80 percent D_r category. Although the equation of the line may be representative of only a few sands, it is assumed to be an appropriate general

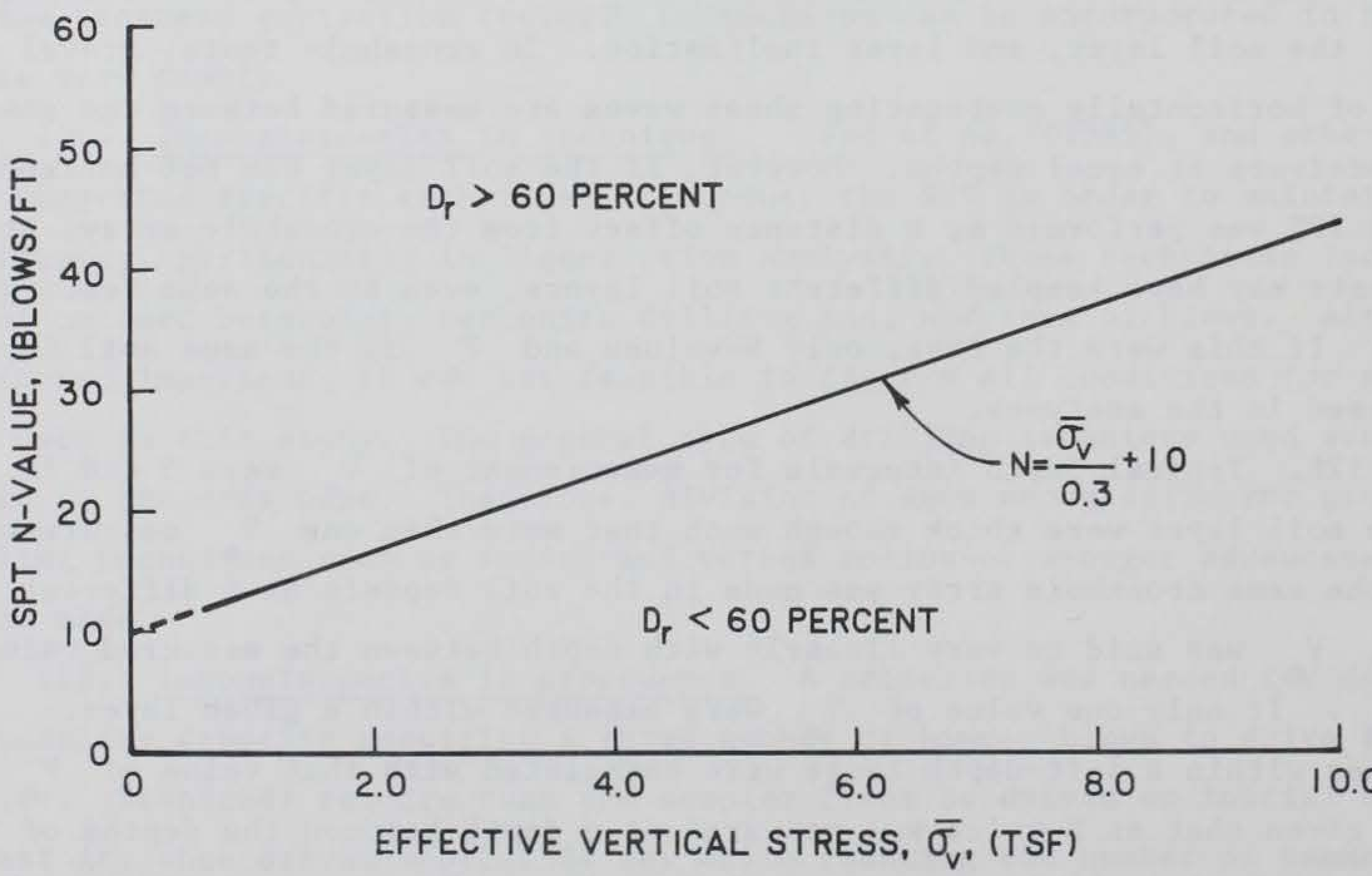


Figure 13. Proposed relationship between effective vertical stress and SPT N-value used to distinguish granular soils above and below a relative density of 60 percent

relationship for purposes of this study. About 14 percent of granular soils for which data were available were found to correspond to less than 60 percent relative density using this criterion.

Vertical stress and depth

131. Vertical stress is trivial to calculate given a density profile and depth of phreatic surface. Depths of measurement are known. The known depth to the phreatic surface is a requirement. The accuracy of stress calculations is not particularly sensitive to accuracy of unit weight.

132. Typically, measured values of density were reported in the geotechnical engineering reports. In a few instances reasonable estimates of density were made in the report based on some qualitative or quantitative assessment. All measured values of moist unit weight within a reasonable distance from the crosshole array were used to develop a density profile. If more than one density value were reported for a depth interval of about 2 ft or less, the values were averaged. If no measured values were available, reported estimated densities were allowed.

Primary Data Categories

Geologic age

133. The geologic age of soil deposits was either available in geotechnical engineering or geology reports, or was requested from the exploration firm expressly for purposes of this study. A geologic time scale common to practitioners was used in this study, as summarized in Table 10. In addition to the standard upper Cenozoic time ranges, a more recent range was defined as 0 to 100 years to include fill soils and historic natural deposits.

Table 10

Geologic Time Scale Used in This Study

Geologic Epoch	Geologic Period	Data Base Symbol	Age years
Recent	--	RC	Historic (0-100)
Holocene	Quaternary	HO	100-10,000
Pleistocene	Quaternary	PL	10,000-2,000,000
Pliocene	Tertiary	TR	>2,000,000

Geologic origin

134. Geologic origin addresses the mode of deposition or other forms of creation of soil deposits. A category was created for all reported origins, as listed in Table 11. Most reports were not very specific about formation of the deposits and referred generally to alluvium or coluvium, etc. These soils were put in general origin groups (e.g., "A_" or "C_", respectively) exclusive of other alluvial and coluvial groups. If more specific references were made, these were included in the data base.

Table 11

Geologic Origin Divisions Used in This Study

Data Base Symbol*	Geologic Origin
A_	Alluvium (general)
AC	Alluvium--channel fill deposits
AF	Alluvium--fan deposits
AL	Alluvium--levee deposits
AO	Alluvium--outwash deposits
AP	Alluvium--point bar deposits
AT	Alluvium--terrace deposits
C_	Coluvium (general)
CT	Coluvium--talus
CS	Coluvium--slope wash deposits
DS	Dune sand
FC	Fill--controlled
FH	Fill--hydraulic
FR	Fill--random
FU	Fill--uncontrolled
FW	Fill--wagon
GT	Glacial till
L_	Lacustrine (general)
M_	Marine sediments
RS	Residual soil
S_	Loess

* Unified Soils Classification System (USCS).

Soil type

135. Soil type is a common parameter specified in geotechnical engineering reports. The USCS (originally proposed by the US Army Engineer, Waterways Experiment Station 1960) is used widely in practice and was adopted for this study. A summary of soil types and symbols used is presented in Table 12. Two additional material types were included for this study--rockfill and random fill. Rockfill typically is very large, angular gravel, cobbles, or broken rock. Random fill may consist of any combination of sand, silt, gravel, and cobbles.

136. The designation of a soil classification was trivial when physical property evidence was supplied. However, these data were not available for every sample. Visual classifications typically were available from all SPT samples. However, the latter type of classification system demands some scrutiny. In general, three different classification systems were defined for this study to develop guidelines for use of soil type classifications in the data base. First, if no supportive evidence was available from a particular profile or site, only general soil descriptions were used in this study (i.e., "S_", "G_", "M_", or "C_") exclusive of other data groups. Second, if only a few physical property data were available, specific classifications in and around the known classification were made specific (if other forms of information such as drill logs were consistent), but the remainder of the profile incorporated general classifications. Third, if a detailed profile of known classifications were substantiated, classifications of samples without supportive data were made specific by juxtaposition of all available information.

Relative location of the phreatic surface

137. The depth of the phreatic surface at the time of geophysical testing was required for this study for several aforementioned reasons. Depth to the phreatic surface was determined by piezometric data or static water levels in boreholes advanced without drilling mud. Typically, piezometric data and pool elevations were acquired from respective water districts or other supervising agency for profiles in embankment dams. Sometimes, interpretation of fluctuations based on pool levels or the like was necessary to determine the depth to the phreatic surface at a particular location during a given time period.

138. Depths interpreted from compression wave velocity profiles were

Table 12

Soil-Type Divisions Used in This Study

Data Base Symbol*	Soil (Type) Description
G_	Gravel (general)
GW	Well-graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
GWGM	Well-graded gravel with silt
GWGC	Well-graded gravel with clay
GPGM	Poorly graded gravel with silt
GPGC	Poorly graded gravel with clay
GCGM	Silty-clayey gravel
S_	Sand (general)
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
SWSM	Well-graded sand with silt
SWSC	Well-graded sand with clay
SPSM	Poorly graded sand with silt
SPSC	Poorly graded sand with clay
SCSM	Silty-clayey sand
M_	Silt (general)
ML	Low-plasticity silt
MH	High-plasticity silt
C_	Clay (general)
CL	Low-plasticity clay
CH	High-plasticity clay
CLML	Silty clay--clayey silt
OL	Organic silt or clay with low plasticity
OH	Organic silt or clay with high plasticity
PT	Peat
RK	Rockfill
RD	Random fill (undifferentiated)

* USCS classification.

not considered. The use of a threshold value of compression wave velocity around 5,000 fps to define the phreatic surface was not adopted for this study because of noted discrepancies between accurate piezometric data and depths determined by this procedure. The magnitude of differences has exceeded 10 ft. This discrepancy has been noted by others (Stokoe 1986*), and reasons for the difference are currently under examination.

Overburden (stress) conditions

139. It seems plausible to develop a qualitative factor to estimate zones of equivalent $\bar{\sigma}_h$ for use with $\bar{\sigma}_v$ to simulate changes in $\bar{\sigma}_A$. This is necessary due to difficulties associated with using a completely quantitative value of stress. The qualitative factor would be intended to account for difference ranges in K_o thereby making correlations between $\bar{\sigma}_v$ and V_s more valid. Using this premise, data were divided among distinct overburden (stress) condition zones.

140. A total of five zones were created to apply to most possible stress conditions assuming that geophysical crosshole measurements were made parallel to the dam center line. These zones are shown in Figure 14. For level sites (slope less than 5 (horizontal) 1 (vertical)) without the influence of an adjacent slope, a "free field" zone (F) is appropriate. In this case, K_o is only a function of relative density and OCR. Typical values of

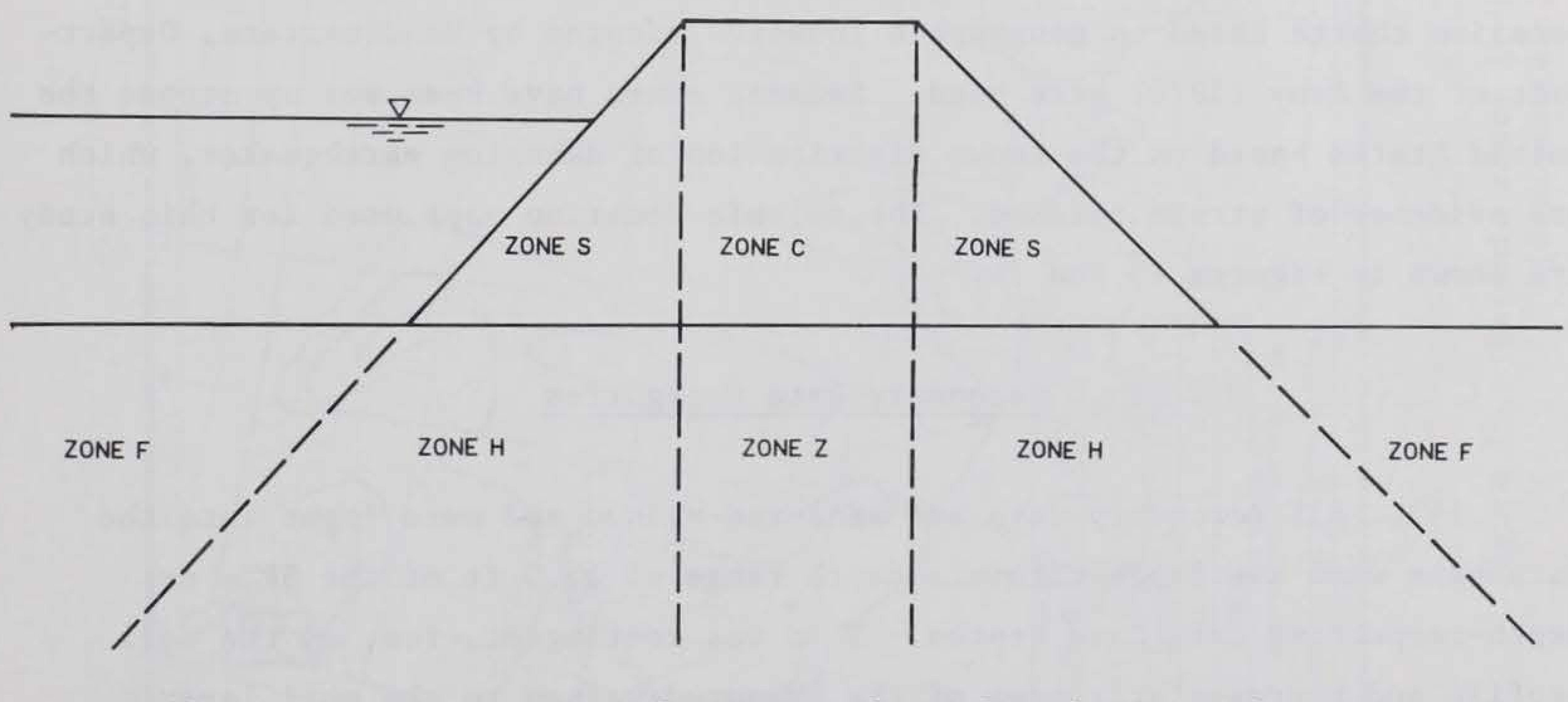


Figure 14. Zones created to define general uniform horizontal stress regimes

* Personal Communication, K. Stokoe, 1986, Professor of Civil Engineering, University of Texas at Austin, Austin, Tex.

K_o might range from 0.4 to 0.8. Two zones were delineated for embankments or slopes. These zones were based on solutions presented by Clough and Woodward (1967). At points near the center line of the dam, horizontal stress factors K_x were in the range of 0.4 to 0.6 (similar to a normally consolidated soil). At points nearer the slopes, K_x approached 1.0, therefore it seemed reasonable to divide the embankment into two zones--one even with the width of the crest throughout the embankment height (zone C) and the other comprising remaining areas upstream and downstream (zone S).

141. The two remaining zones (Z and H) correspond to soils located beneath an embankment and are extensions of zones in the embankment. These two zones were kept distinct from the embankment since they represent natural soils which most likely are normally consolidated (and may yet be in the process of consolidating). Materials in these zones are expected to behave differently than fill soils in the embankment. The demarcation between free field and the field beneath the embankment slopes was selected based on solutions of embankments on elastic foundation by Perloff, Baladi, and Harr (1967) and convenience.

Seismic zonation

142. It is very difficult, if not impossible, to determine the amount of cycling or previous straining a soil layer has undergone in situ. For lack of a better index to use in sorting soil layers by prior straining, seismic zonation charts based on geographic location adopted by Headquarters, Department of the Army (1970) were used. Seismic zones have been set up across the United States based on the known distribution of damaging earthquakes, which are evidence of strain release. The seismic zonation maps used for this study are shown in Figures 15 and 16.

Secondary Data Categories

143. All secondary data are measured values and were input into the data base when available within a depth range of ±1.5 ft of the SPT- or depth-respective data base depths. This was contingent, too, on the soil profile and representativeness of the measured values to the soil layer.

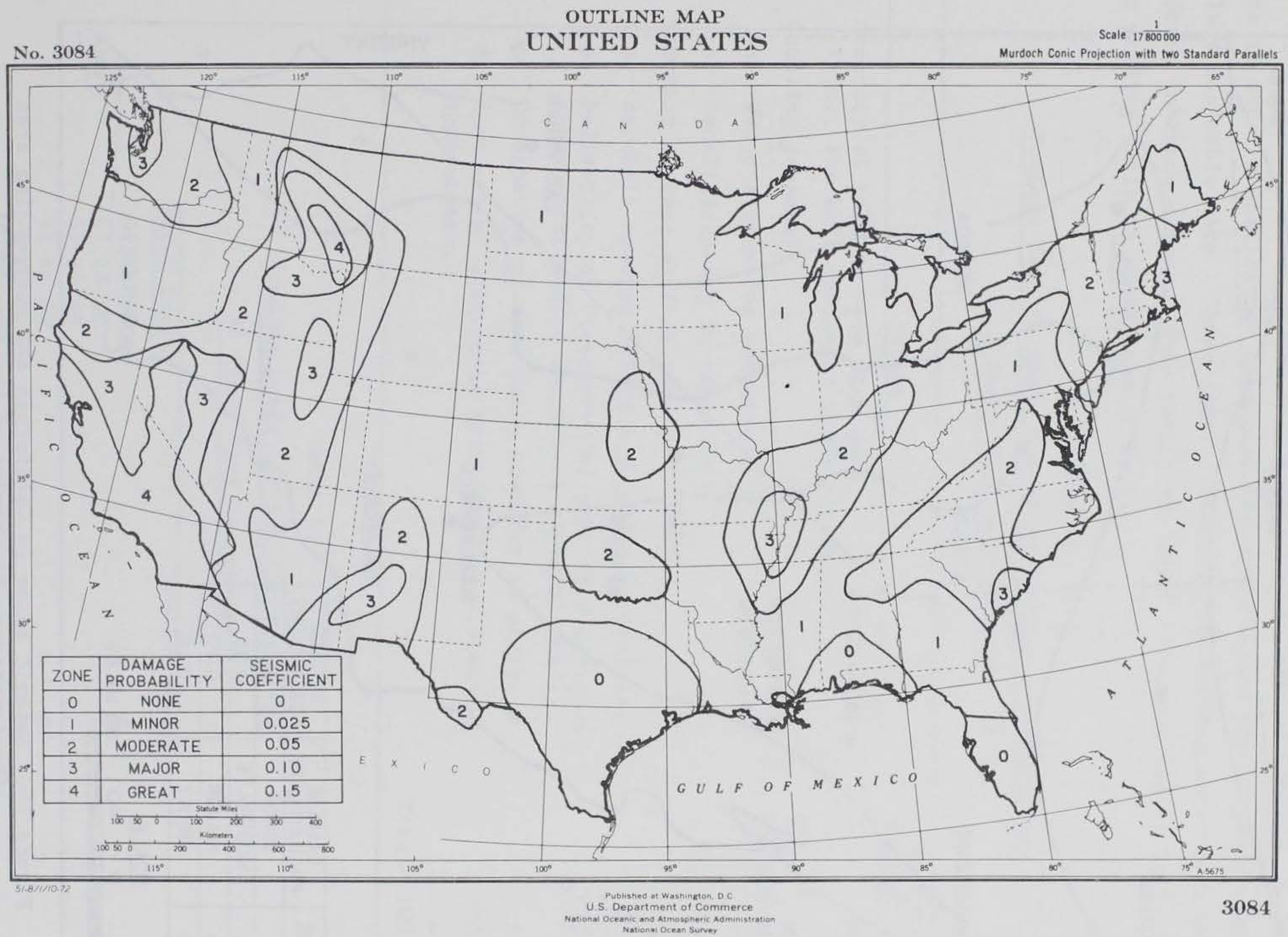
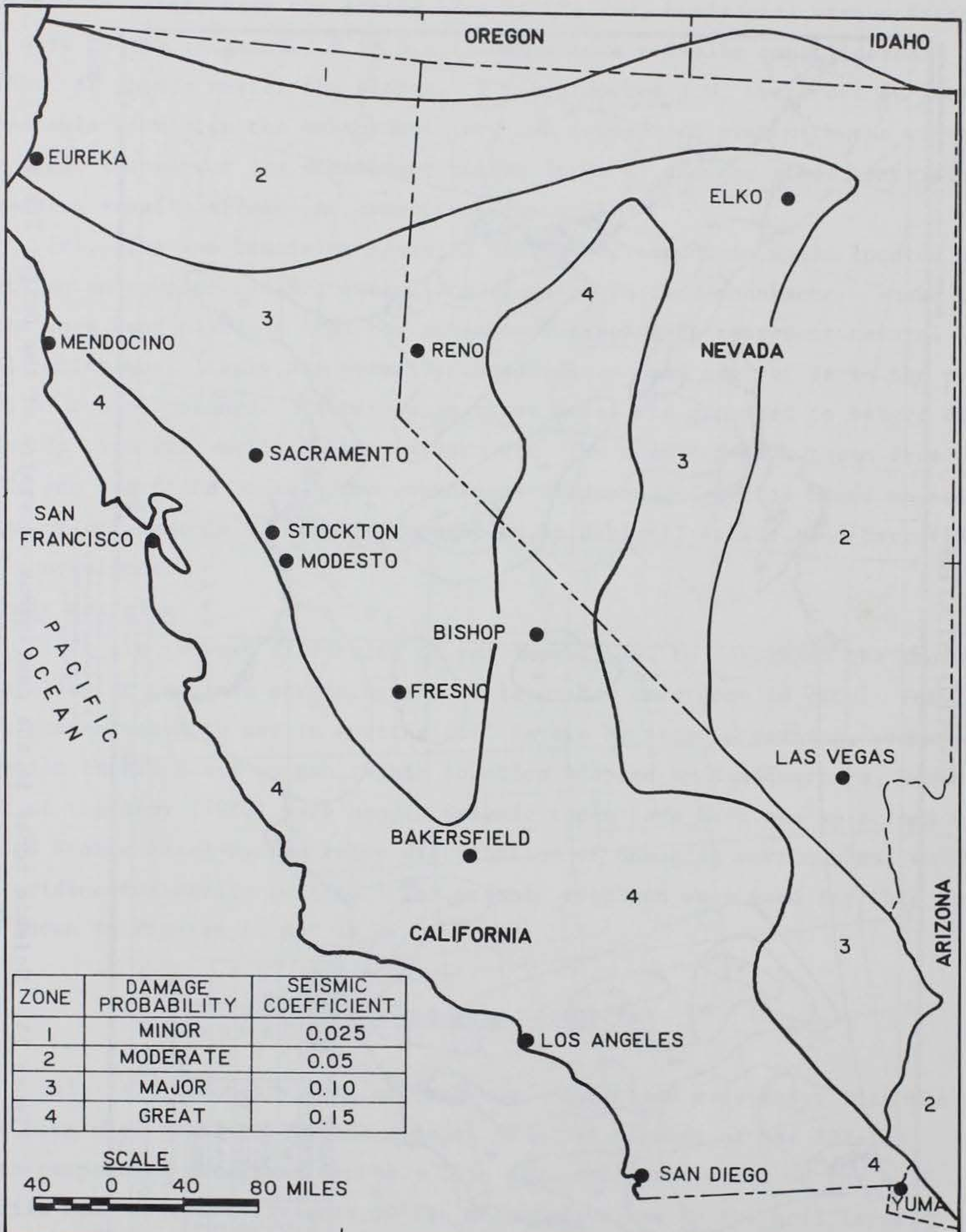


Figure 15. Seismic zones in the contiguous 48 states of the United States (Headquarters, Department of the Army 1970)



ZONE	DAMAGE PROBABILITY	SEISMIC COEFFICIENT
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

Figure 16. Seismic zones in California and surrounding area (Headquarters, Department of the Army 1970)

Prorating Measurement Quality

144. Subjective ratings of the methodology used to measure V_s were made to offer a qualitative aspect to the study beyond the criteria used to select desirable data. Three general factors were adopted which indirectly address inherent errors. This rating procedure is summarized in Table 13. The grading scale is summarized in Table 14.

Table 13

Factors Used to Apply Subjective Rating to Methodology of Shear Wave Measurements

Factor	Approximate Rating, percent
1. Accuracy of hole-to-hole distances (i.e., Was hole deviation study performed?).	30
2. Coupling adequacy.	
a. Wedge system for geophones.	5
b. Backfill/casing installation.	15
3. Accuracy of wave arrival times.	
a. Borehole spacing relative to velocity profile.	15
b. Reliability of first arrival pick (i.e., Signal enhancement and/or shear wave reversal used?).	10
c. Combination of wave paths used for an array.	5
d. Accuracy of electronic measuring and processing apparatuses.	20
Total	100

Table 14

Grading Scale for Subjective Rating of Shear Wave Measurements

Total Rating	Grade
91 to 100	A
81 to 90	B
61 to 80	C
41 to 60	D*
0 to 40	E*

* Data not input into the data base.

145. The most effective use of this proration system has not been fully ascertained. It may be useful to examine variations in V_s as a function of quality of measurement.

Other Considerations

146. Two conditions may exist that also had to be addressed to accomplish the data reduction phase. These include the distance of V_s measurement arrays from other measured geotechnical engineering properties and existence of multiple data from boreholes within the geophysical array.

147. It was preferable to have SPT and other geotechnical data from a borehole also used in the crosshole seismic array. This was not always possible. In these instances, data from nearby boreholes were used (when data were not available from the crosshole boreholes) to correlate with V_s . Obviously, this separation introduces some inherent error being a function of magnitude of separation and homogeneity of the deposit. Discretion was employed for each individual case. Distances between the two boreholes were recorded into the data base for future reference and scrutiny. These distances do not exceed 25 ft, and typically are much lower.

148. In some cases, SPT or laboratory data were available from more than one borehole in a crosshole set. As stated previously, a data point was created for each measured N-value for the SPT-respective V_s data base. Specific soil types and physical property data derived from an SPT sample obviously have a one-to-one correspondence. For the depth-respective data base, if more than one set of physical property data existed at a particular depth, the data were either averaged or data thought to be more representative of the soil layer were selected.

PART V: PRESENTATION OF INFORMATION COLLECTED

149. A brief summary of data available in the data bases is presented in this part to allow examination of data available. This is accomplished in two sections. In the first section, the quantity and distribution of data, including ranges in V_s per specific category are presented. In the second section, visual observations resulting from preliminary correlative analysis are presented.

150. Data bases created at WES will be stored permanently and updated periodically. Correlative analyses will also be performed periodically. Data will reside on a computer hard disk and 5-1/4-in. floppy diskettes. Distribution of the data is open to the public.

Distribution of Data

151. The distribution of data both geographically and among data groups is summarized herein to characterize the data bases and interpret the nature of data available. Uniform distribution of data is important to correlative analyses. Unfortunately, categories of variables available for this study exhibit a near-uniform distribution.

152. The distribution of data among different project sites and geographically, by state, is shown in Tables 15 and 16. A total of 1,257 and 1,026 data are available for depth- and SPT-specific correlations, respectively. Fewer data exist for N_{60} and $N_{1,60}$ than N and N_1 groups since information necessary to determine SPT energy efficiency was not always available. Data from 40 project sites, 5 using 8 crosshole arrays, are stored. A clear majority (about 60 percent) of data for each correlation category is below the phreatic surface. The distribution across the United States is nonuniform. Although data were collected from projects in eight contiguous states, over 70 percent of the sites are located in the state of California.

153. The distribution of data among seismic zones and overburden conditions is represented in Table 17. Several (8) combinations of these two primary variables have no data available. All of these null sets correspond to seismic zones 0 and 1. The "free field" (F) overburden condition is the exception with a nearly uniform distribution of data among seismic zones for

Table 15

General Distribution of Data

Correlative Data Base	Location(s) Relative to Phreatic Surface	Number of Sites	Number of Arrays	Number of Data
Depth-specific V_s	All	40	85	1,257
	Above	38	72	491
	Below	36	76	766
SPT-specific V_s				
N and N_1	All	32	56	1,026
	Above	27	35	393
	Below	28	44	633
N_{60} and $N_{1,60}$	All	27	45	888
	Above	23	32	368
	Below	24	33	520

Table 16

Geographical Distribution of Projects

Project Type	Location by State	Number of Project Sites
Foundation engineering/exploration	Ariz.	3
	Calif.	9
	Ill.	1
	Mass.	1
	Ohio	1
	S.C.	1
	Tex.	1
	Utah	1
Seismic dam stability	Calif.	19
	Kans.	1
	Mass.	1
	S.C.	1

Table 17

Distribution of Data Among Overburden Conditions and Seismic Zones

Overburden Condition*	Seismic Zone	Depth-Respective V_s: No.	Depth-Respective V_s: Range in V_s, fps	SPT N-Value-Respective V_s**: No.	SPT N-Value-Respective V_s**: Range in V_s, fps
F	0	117	590-1,840	77	625-1,800
	1	81	235-1,415	6	995-1,125
	2	156	285-1,890	69	325-1,765
	3	108	375-3,390	74	400-2,880
	4	154	380-1,810	114	400-1,865
		616	235-3,390	340	325-2,880
C	0	1	800	1	870
	1	0	--	0	--
	2	64	440-1,200	186	310-1,145
	3	62	450-1,325	84	455-1,300
	4	144	545-1,560	46	695-1,550
		271	440-1,560	317	310-1,550
Z	0	0	--	0	--
	1	0	--	0	--
	2	12	680-1,240	29	680-1,235
	3	0	--	0	--
	4	30	715-2,000	27	590-1,940
		42	680-2,000	56	590-1,940
S	0	0	--	0	--
	1	0	--	0	--
	2	54	310-1,440	89	535-1,285
	3	36	450-1,750	27	610-1,765
	4	60	580-1,780	44	675-1,285
		150	310-1,780	160	535-1,765
H	0	0	--	0	--
	1	0	--	0	--
	2	30	410-1,160	60	455-1,100
	3	13	1,045-1,340	9	1,055-1,340
	4	123	530-1,530	83	540-1,205
		166	410-1,530	152	455-1,340

* For summary of codes and zones refer to Figure 14 and Figures 15 and 16, respectively.

** N and N_1 data.

depth-respective V_s correlations. The distribution of data solely among overburden conditions is weighted heavily with measurements in the "free field" and through the core of dams (C).

154. The distribution of data among geologic origin and age divisions is summarized in Tables 18 and 19 for depth- and SPT-respective correlations, respectively. Tertiary soils were not included in these tables since only three data existed from that period. Over 90 percent of the data are from either alluvium or fill material--about 50 and 40 percent, respectively. Recent and Pleistocene-age soils have about the same quantity of data; whereas, Holocene-age soils comprise about 20 percent of the data. Therefore, the data base is considered to be unrepresentative of all geologic conditions.

155. The distribution of data among soil type divisions both for depth- and SPT-respective data bases is presented in Table 20. The majority of data represents sands (about 55 percent of each data base). The percentage of data for general soil categories with respect to major divisions and the entire data base is presented in Table 21.

156. The distribution of data among gravel classifications for depth-respective correlations is very poor. Nearly all gravels (67 of 88 data) are classified as poorly graded gravel (GP). The distribution among SPT-respective correlations is better, but few data (29) exist. Gravelly soils represent only 7 and 3 percent of the granular data for depth- and SPT-respective correlations, respectively.

157. Data among sandy soil types are distributed rather well with a few exceptions. For depth-respective correlations, only well-graded, well-graded with clay, and poorly graded with clay sand classifications have less than 25 data. These same classification groups have few data in the SPT-respective data base.

158. A clear majority (greater than 70 percent) of cohesive soils classify with a low plasticity designation. Very few clayey silt-silty clay and peat soils were available. Rockfill and random fill groups also have a limited quantity of data.

159. The quantities of various secondary variables available for use in analyses are summarized in Table 22. A moderate percentage of data (greater than 15 percent) have Atterberg limits, D_{50} , percentage of fines, and dry unit weight values defined. Most other categories have very limited quantities of data which minimized the value of their use.

Table 18

Distribution of Data Among Geologic Origin and Age Divisions for Depth-Respective Correlations

Group*	Recent No.	Recent Range, fps	Holocene No.	Holocene Range, fps	Pleistocene No.	Pleistocene Range, fps
A_	15	560-1,530	270	325-1,725	370	270-3,390
AC	4	415-450	0	--	0	--
AF	0	--	0	--	29	1,000-1,890
AL	0	--	5	420-480	0	--
AO	0	--	0	--	9	550-770
AP	0	--	5	520-600	0	--
AT	0	--	0	--	0	--
All alluvium	19	415-1,530	280	325-1,725	408	270-3,390
C_	0	--	0	--	4	280-620
CT	0	--	0	--	0	--
CS	0	--	0	--	10	1,410-1,940
All coluvium	0		0		14	280-1,940
FC	337	200-1,825		N/A		N/A
FH	60	440-1,090		N/A		N/A
FR	17	580-1,220		N/A		N/A
FU	3	380-415		N/A		N/A
FW	6	830-1,145		N/A		N/A
All fill	423	200-1,825				
GT	0	--	0	--	2	1,110-1,140
L_	0	--	0	--	76	235-1,660
M_	0	--	0	--	0	--
RS	0	--	0	--	2	810-830
S_	0	--	0	--	24	905-1,765

* Tertiary soils excluded because too few data exist. For summary of codes, refer to Table 11.

Table 19

Distribution of Data Among Geologic Origin and Age Divisions for SPT-Respective Correlations*

Group**	Recent		Holocene		Pleistocene	
	No.	Range, fps	No.	Range, fps	No.	Range, fps
A_	0	--	210	325-1,400	283	430-2,880
AC	2	415-440	0	--	0	--
AF	0	--	0	--	0	--
AL	0	--	2	435-460	0	--
AO	0	--	0	--	4	560-750
AP	0	--	2	520-585	0	--
AT	0	--	0	--	0	--
All alluvium	2	415-440	214	325-1,400	287	430-2,880
C_	0	--	0	--	0	--
CT	0	--	0	--	0	--
CS	0	--	0	--	24	1,465-1,940
All coluvium	0		0		24	1,465-1,940
FC	284	455-1,765		N/A		N/A
FH	144	310-1,285		N/A		N/A
FR	36	595-1,220		N/A		N/A
FU	3	400-415		N/A		N/A
FW	11	830-1,145		N/A		N/A
All fill	478	310-1,765				
GT	0	--	0	--	2	1,150
L_	0	--	0	--	0	--
M_	0	--	0	--	0	--
RS	0	--	0	--	0	--
S_	0	--	0	--	14	925-1,765

* N and N_1 data.

** Tertiary soils excluded because too few data exist. For summary of codes, refer to Table 11.

Table 20

Distribution of Data Among Soil-Type Divisions

Soil Type*	Depth-Respective V_s		SPT-Respective V_s **	
	No.	Range, fps	No.	Range, fps
G	0	--	0	--
GW	1	1,550	2	1,140-1,550
GP	67	750-3,390	8	860-2,500
GM	1	935	0	--
GC	11	525-1,525	8	525-1,565
GWGM	1	1,250	1	1,280
GWGC	1	1,300	1	1,300
GPGM	1	1,825	4	1,250-1,765
GPGC	3	1,100-1,300	3	1,100-1,280
GCGM	2	1,400-1,525	2	1,400-1,525
All gravels	88	525-3,390	29	525-2,500
S	79	285-1,650	14	795-1,340
SW	5	800-2,885	2	795-2,880
SP	99	450-1,865	63	468-1,863
SM	148	325-1,725	171	340-1,660
SC	180	545-1,780	150	525-1,940
SWSM	28	590-1,550	39	455-1,525
SWSC	3	1,250-1,500	3	1,100-1,500
SPSM	92	400-1,580	74	345-1,420
SPSC	4	375-465	18	375-485
SCSM	27	550-1,750	23	380-1,700
All sands	665	285-2,885	557	340-2,880
All cohesionless soils	753	285-3,390	586	340-2,880
M	1	1,280	0	--
ML	84	310-1,890	112	310-1,730
MH	2	1,280-1,580	8	815-1,610
All silts	87	310-1,890	120	310-1,730
C	0	--	0	--
CL	239	280-2,105	195	455-1,795
CH	132	235-2,445	83	490-2,450
All clays	371	200-2,445	278	455-2,450
CLML	15	520	9	520-2,175
OL	0	--	0	--
OH	0	--	0	--
All cohesive soils	486	200-2,445	407	310-2,450
PT	0	--	1	390
RK	11	620-1,325	0	--
RD	15	580-1,205	32	595-1,165

* For summary of codes, refer to Table 11.

** N and N_1 data.

Table 21

Proportions of Data Among General Soil Types

General Soil Types	Depth-Respective Percent of Granular	Percent of Cohesive	Percent of Data Base	SPT-Respective* Percent of Granular	Percent of Cohesive	Percent of Data Base
Gravels	12	--	7	5	--	3
Sands	88	--	53	95	--	54
Silts	--	18	7	--	29	12
Clays	--	82	30	--	71	27
Other	--	--	3	--	--	4

* N and N_1 data.

Table 22

Distribution of Data in Secondary Variable Categories

Categories	Number of Data Available: Depth-Respective V_s	SPT-Respective V_s *
Atterberg limits	312	237
Coefficient of uniformity	82	81
Coefficient of curvature	62	55
Particle size at 50 percent passing, D_{50}	194	172
Particle size at 10 percent passing, D_{10}	98	117
Percentage of fines	376	388
Percentage of clay-sized fines	90	89
Void ratio	72	21
Relative density	0	0
Dry unit weight	348	110
Degree of saturation	44	15
Unconfined strength	10	7
Angle of internal friction	22	14
Cohesion	22	14
Overconsolidation ratio	31	3

* N and N_1 data.

Results of Preliminary Correlation Analysis

160. The analysis of correlations between G or V_s and various geotechnical engineering parameters contained in the data base merits a significant amount of time and effort. Such an effort is expected to be conducted at WES in the future. Some preliminary results are presented herein to further examine the nature of the data base.

161. Two primary approaches, described at the outset of this report, are followed in this presentation of data. Namely, a sensitivity-type analysis involving correlative parameters such as SPT N-value and functional-type analysis involving a general form of an equation are used separately.

Sensitivity-type analysis

162. Seven independent variables, summarized in Table 5, were correlated individually with V_s. These variables are SPT N-value, SPT N_{60}-value, SPT N_1-value, SPT $N_{1,60}$-value, $\bar{\sigma}_v$, σ_v, and depth. Plots of data for these correlations for various soil type groupings and all conditions (the one exception is $\bar{\sigma}_v$ versus V_s in Figure 21) are shown in Figures 17 through 23, respectively. The correlation between $\bar{\sigma}_v$ and V_s involves only data from measurements made below the phreatic surface since large errors may be made by estimating $\bar{\sigma}_v$ above the phreatic surface.

163. Different groupings of soil types were necessary to perform these preliminary correlations. Correlations involving N, N_{60}, $\bar{\sigma}_v$, σ_v, and depth used all soil type groups. Correlations involving N_1 and $N_{1,60}$ used only gravels and sands (granular soils) because the correction of N-value for the effect of effective vertical stress is applicable to granular soils only. The quantity of data (n) used for each correlation is provided in each figure for reference.

164. Noticeable differences were identified between correlations involving different variables of similar nature (i.e., N, N_{60}, N_1, and $N_{1,60}$ and $\bar{\sigma}_v$, σ_v, and depth). In the case of forms of SPT N-value, the most significant difference in distribution of data seems to exist between N and N_{60} and N_1 and $N_{1,60}$, respectively. This effect of normalizing N-values to 1.0 tsf produces a wider band of scatter at lower N-values (N_1 < 40 blows/ft). Based on this visual observation, the use of N (or N_{60}) appears to be more appropriate for future analyses. This occurrence is consistent with results reported by Sykora and Stokoe (1983).

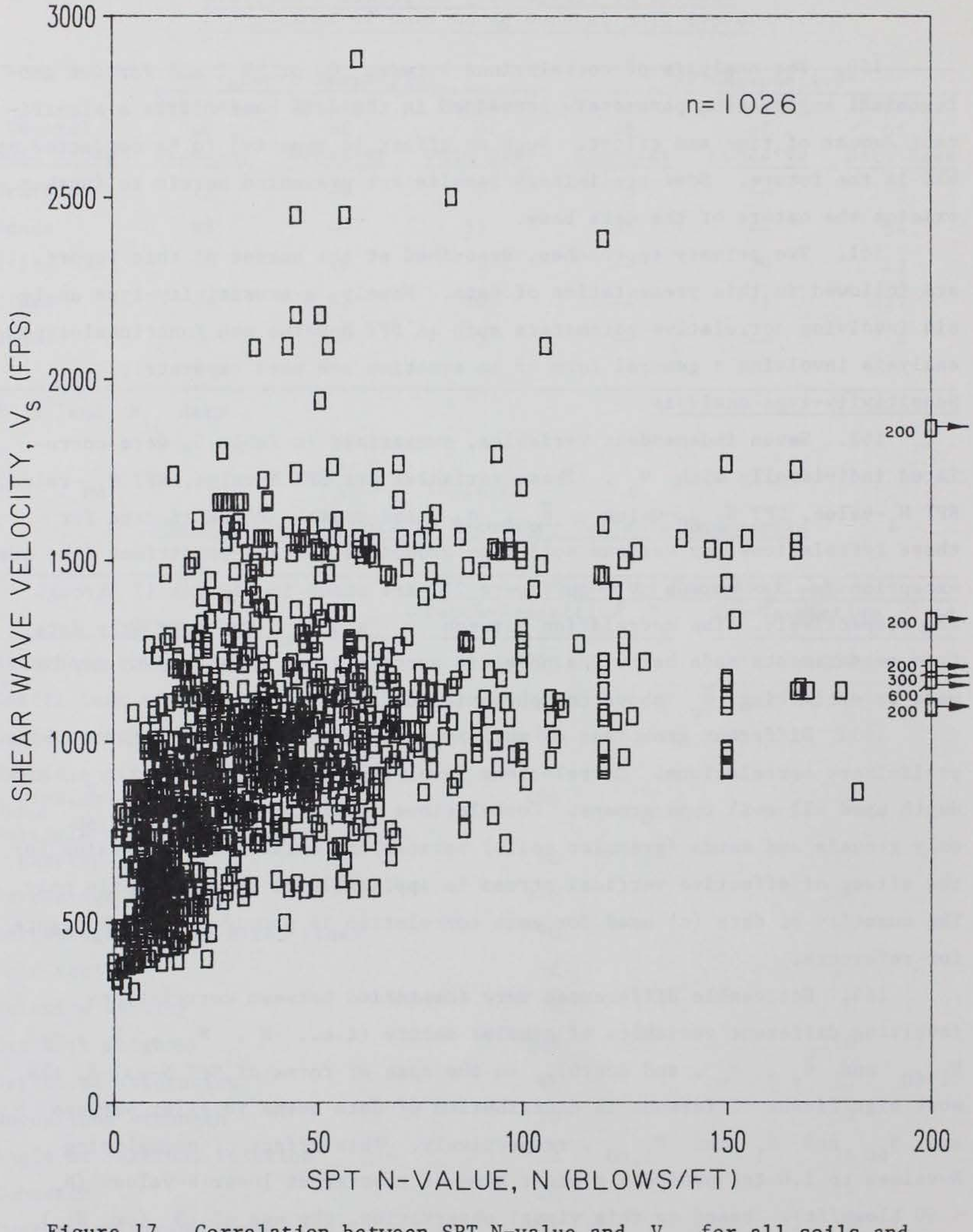


Figure 17. Correlation between SPT N-value and V_s for all soils and conditions

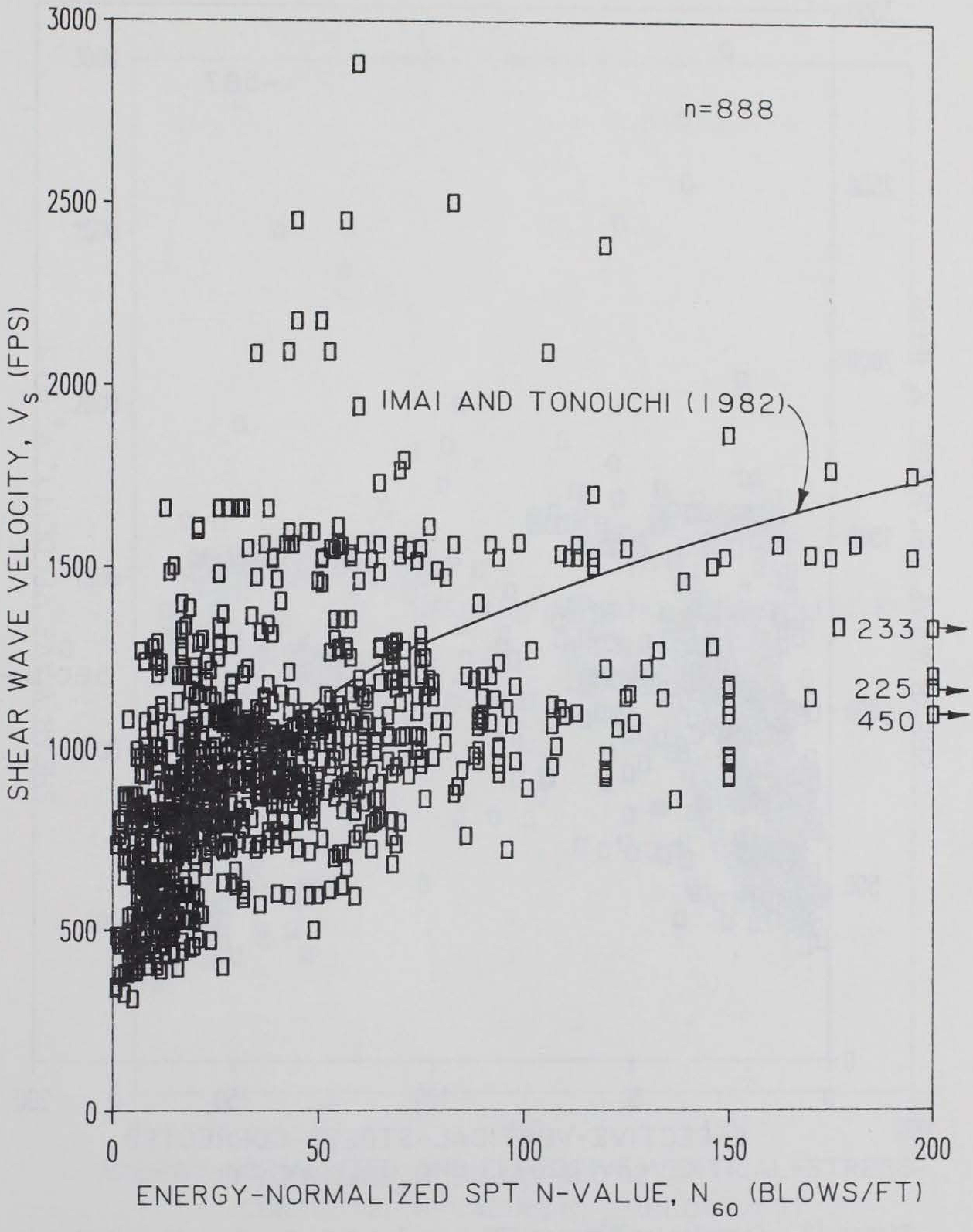


Figure 18. Correlation between SPT N_{60}-value and V_s for all soils and conditions

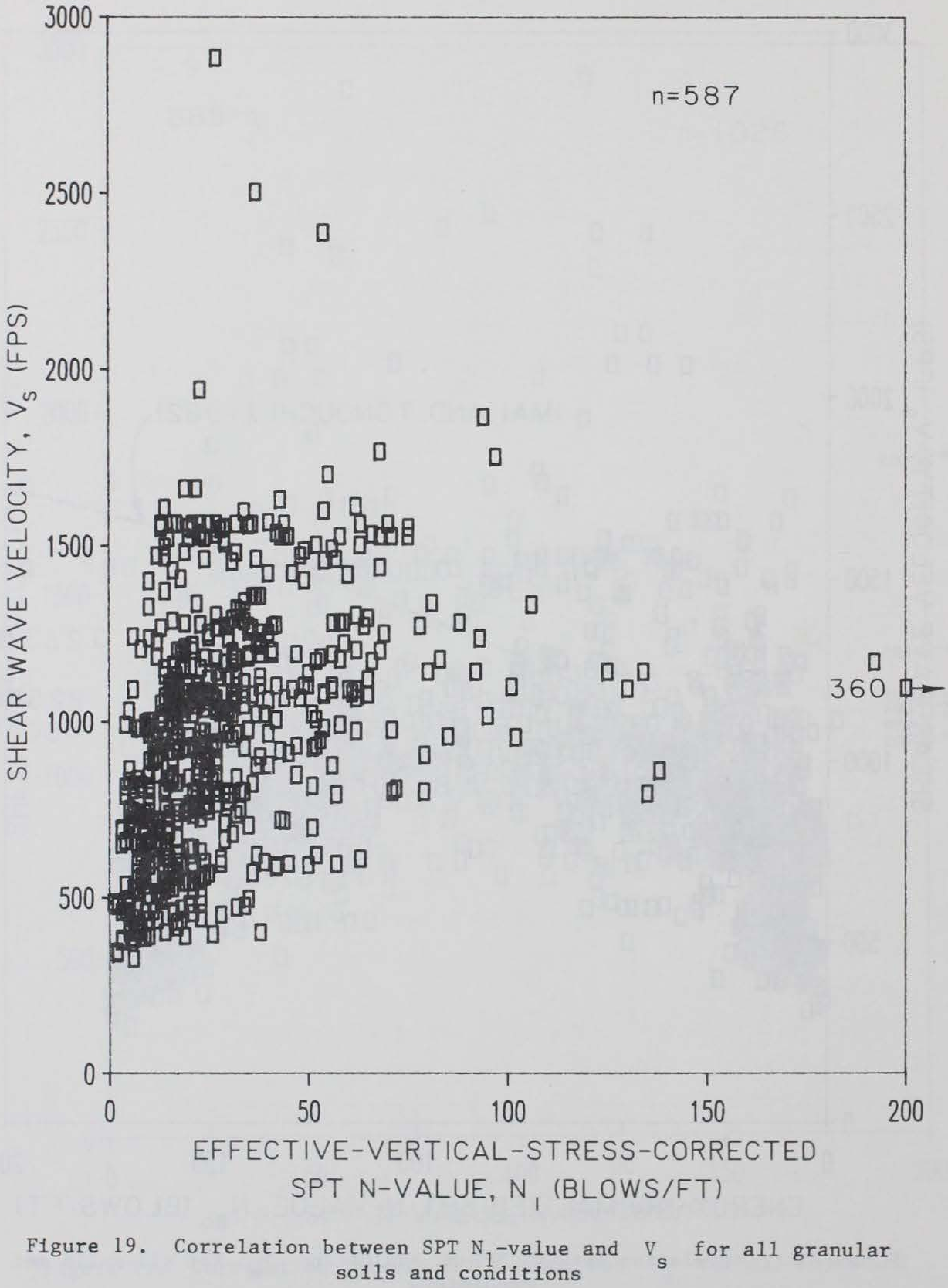


Figure 19. Correlation between SPT N_1-value and V_s for all granular soils and conditions

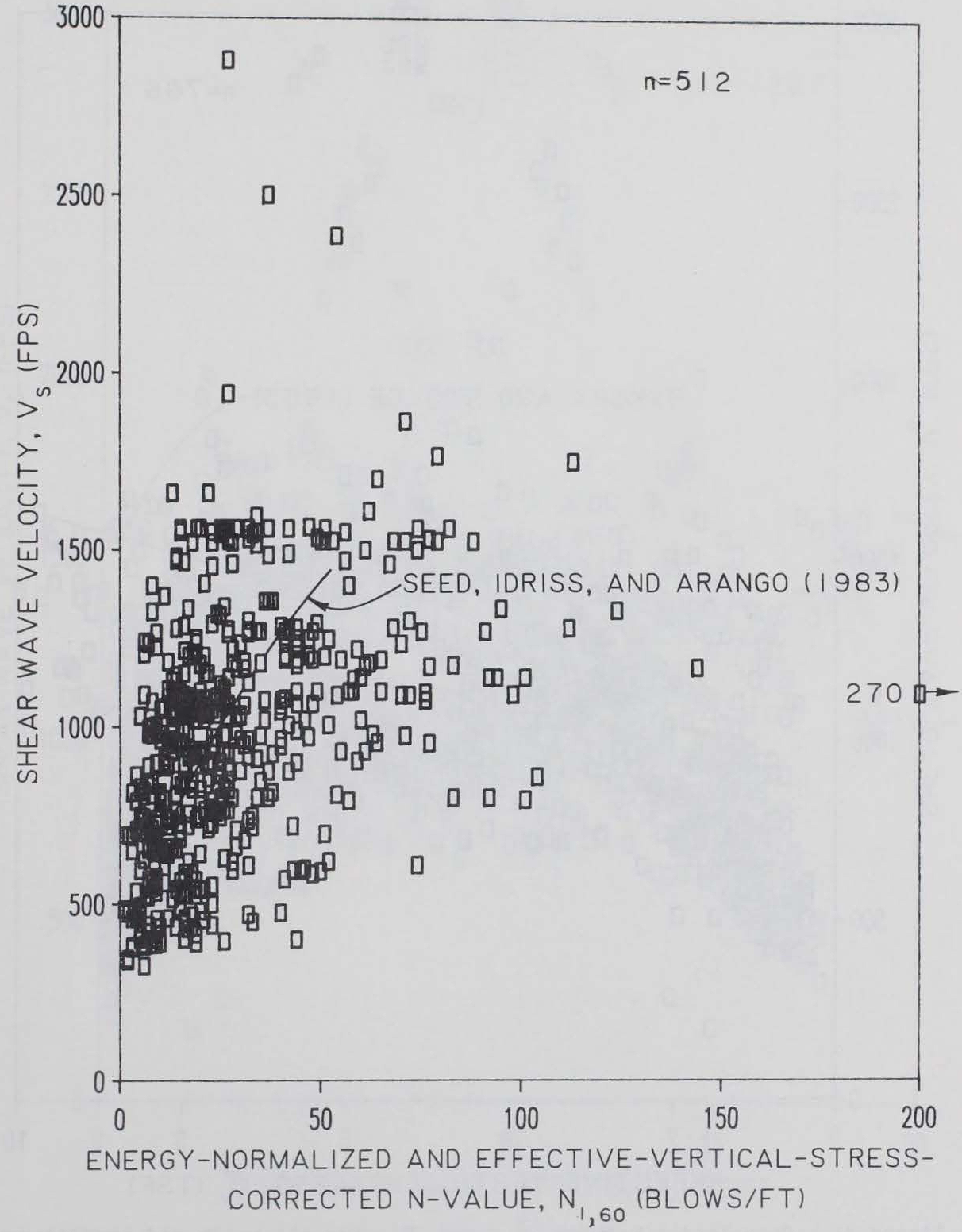


Figure 20. Correlation between SPT $N_{1,60}$-value and V_s for all granular soils and conditions

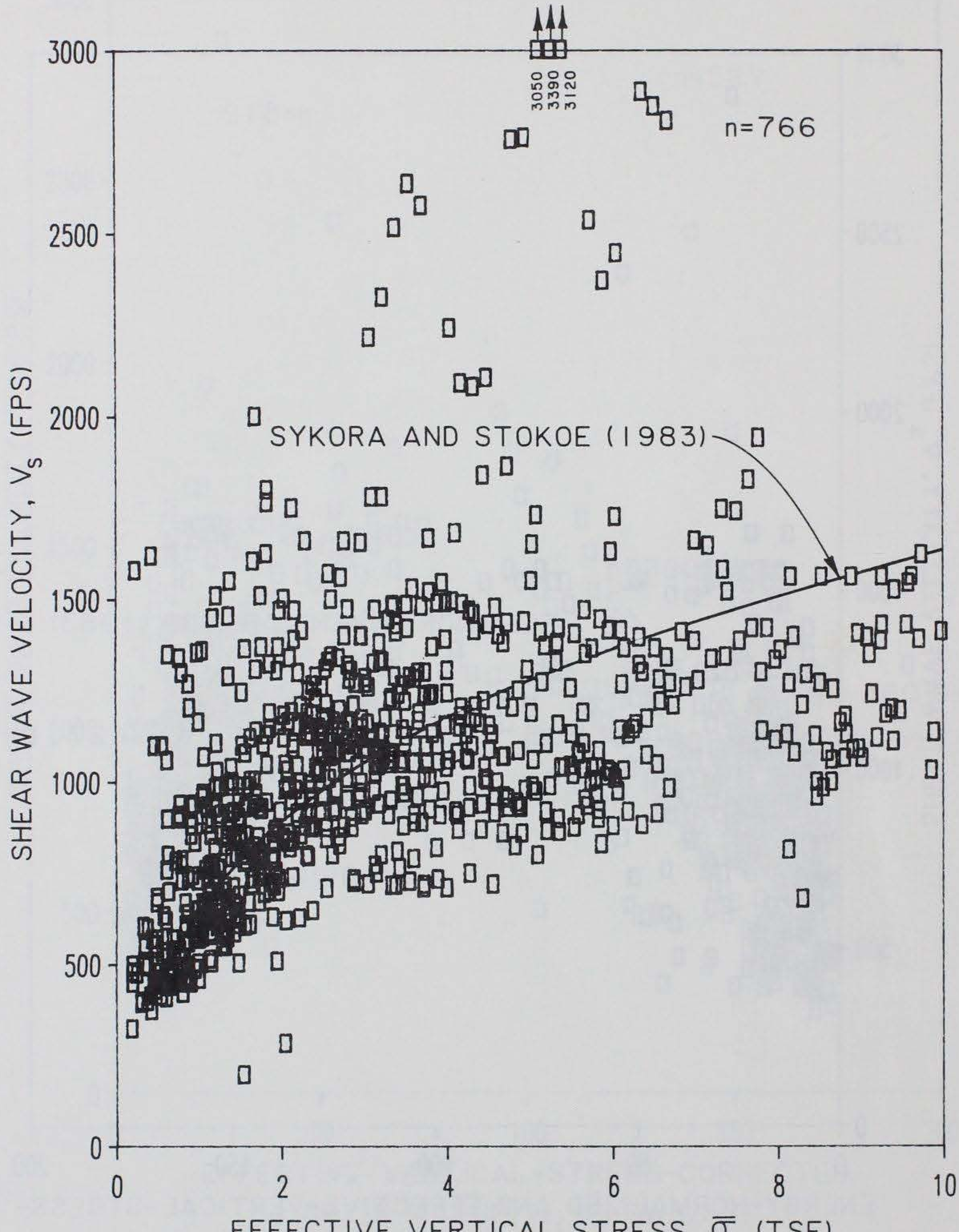


Figure 21. Correlation between $\bar{\sigma}_v$ and V_s for all soils and conditions below a phreatic surface

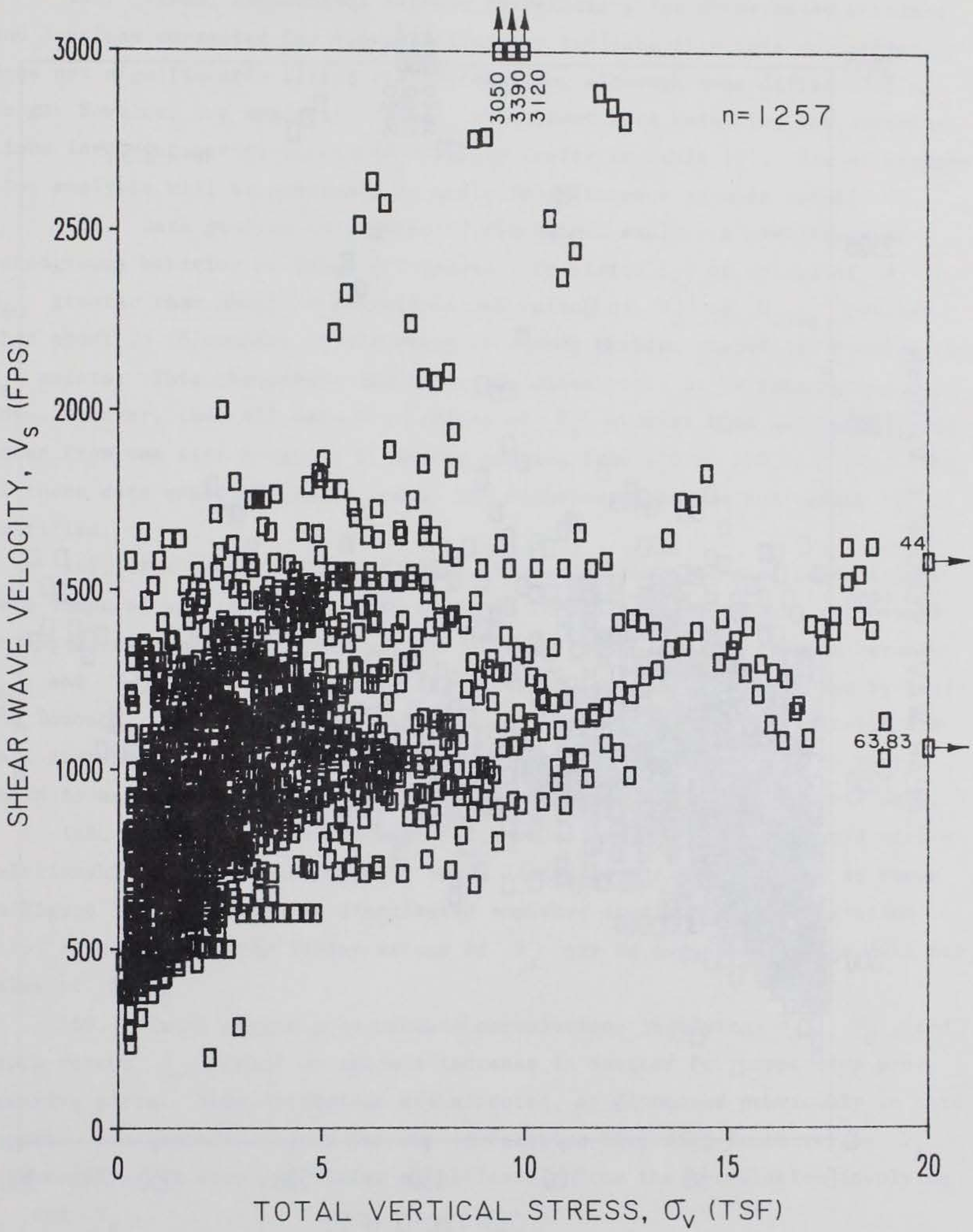


Figure 22. Correlation between σ_v and V_s for all soils and conditions

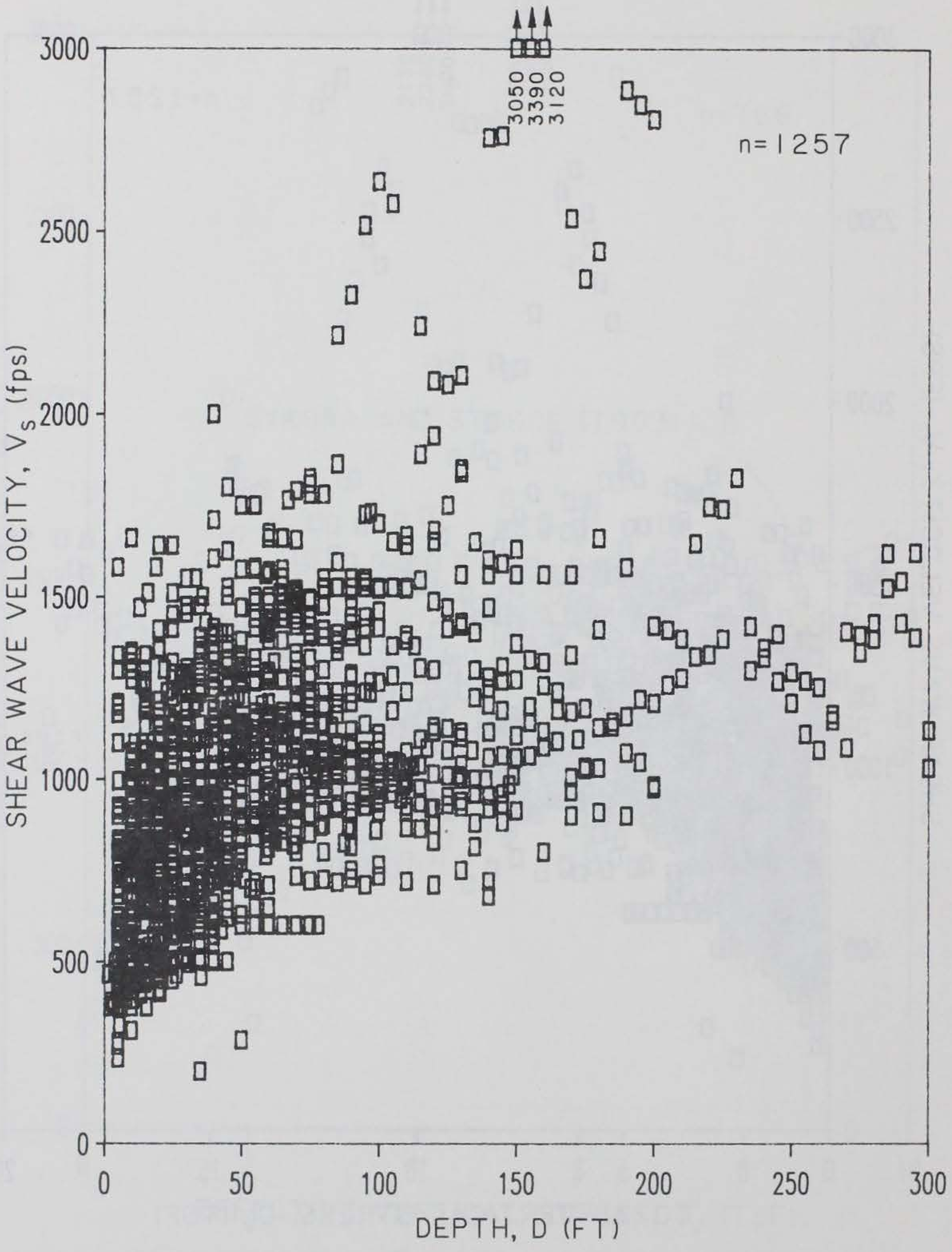


Figure 23. Correlation between depth and V_s for all soils and conditions

165. Visual comparisons between correlations for uncorrected N-values and N-values corrected for energy efficiency indicate that this correction does not significantly affect the correlation, although some differences at larger N-values are apparent. Recall that fewer data exist for the correlations involving energy-corrected N-values (refer to Table 15). Use of regression analysis will be necessary to evaluate difference in more detail.

166. Data plotted in Figures 17 through 20 exhibit a similarity of incongruous behavior at large SPT values. Specifically, at values of N or N_{60} greater than about 50 (blows/ft) and values of N_1 or $N_{1,60}$ greater than about 25 (blows/ft), a dichotomy of trends between respective N-value and V_s exists. This phenomenon cannot be explained yet. It is interesting to note, however, that all data with values of V_s greater than 2,000 fps originated from one site location at depths ranging from 120 to 200 ft. Exclusion of these data would eliminate one of the dichotomous trends but cannot be justified.

167. The distributions of data for some of the presented correlations were compared with a relationship reported in the literature that represents an approximate average (Sykora 1987). Specifically, the correlation between N_{60} and V_s was compared with a relationship for all soils proposed by Imai and Tonouchi (1982), as shown in Figure 18. Data presented in Figure 18 are well distributed about this relationship. A relationship similar to that shown is expected to be produced in future regression analysis of this data.

168. Data correlated between $N_{1,60}$-value and V_s were compared with a relationship for sands proposed by Seed, Idriss, and Arango (1983), as shown in Figure 20. The data are distributed somewhat evenly about the relationship, although slightly larger values of V_s may be suggested by the data per value of $N_{1,60}$.

169. Visual comparisons between correlations involving $\bar{\sigma}_v$, σ_v, and depth versus V_s reveal an obvious increase in scatter for respective progressive plots. This occurrence was expected, as discussed previously in this report. The scatter of data for the correlation involving depth versus V_s is excessive but does not differ significantly from the correlation involving σ_v and V_s.

170. The correlation of data between $\bar{\sigma}_v$ and V_s was compared with a relationship for granular soils proposed by Sykora and Stokoe (1983). The two

are in good agreement, especially if data corresponding to velocities greater that 2,000 fps are excluded.

171. Visual observations offer only a preliminary assessment of the use of geotechnical parameters to estimate V_s. Based on those observations, it appears that N (and N_{60}) and $\bar{\sigma}_v$ are the best suited of the correlative variables considered to estimate V_s. This conclusion is consistent with the findings of Sykora (1987).

Functional-type analysis

172. A general form of an equation was used to perform functional-type correlations involving field parameters. This equation was developed based on results of laboratory and field correlations. More specifically, V_s has been found by numerous field studies related to SPT N-value raised to a power and a linear constant. Laboratory studies indicate that V_s is a function of $\bar{\sigma}_v$ raised to the 0.25 power. Combining these two general findings:

$$V_s = a \cdot N_n^b \cdot \bar{\sigma}_v^{\,c} \tag{8}$$

where

a, b, and c = constants

$N_n = N_1, N_{60},$ or $N_{1,60}$

Preliminary correlations were conducted by setting $c = 0.25$ and modifying Equation 8 to:

$$\frac{V_s}{\bar{\sigma}_v^{\,0.25}} = a \cdot N_n^b \tag{9}$$

Units of measurement used are V_s in fps and $\bar{\sigma}_v$ in tsf.

173. Correlations between $V_s/\bar{\sigma}_v^{\,0.25}$ and N, N_1, and $N_{1,60}$ are plotted in Figures 24 through 27, respectively. Correlations involving N and N_{60} were performed using sands, silts, and clays. Correlations involving N_1 and $N_{1,60}$ were performed using sands since the correction of N-values for the effect of effective vertical stress is applicable for granular soils only. The quantity of data (n) used for each correlation is provided in each figure for reference.

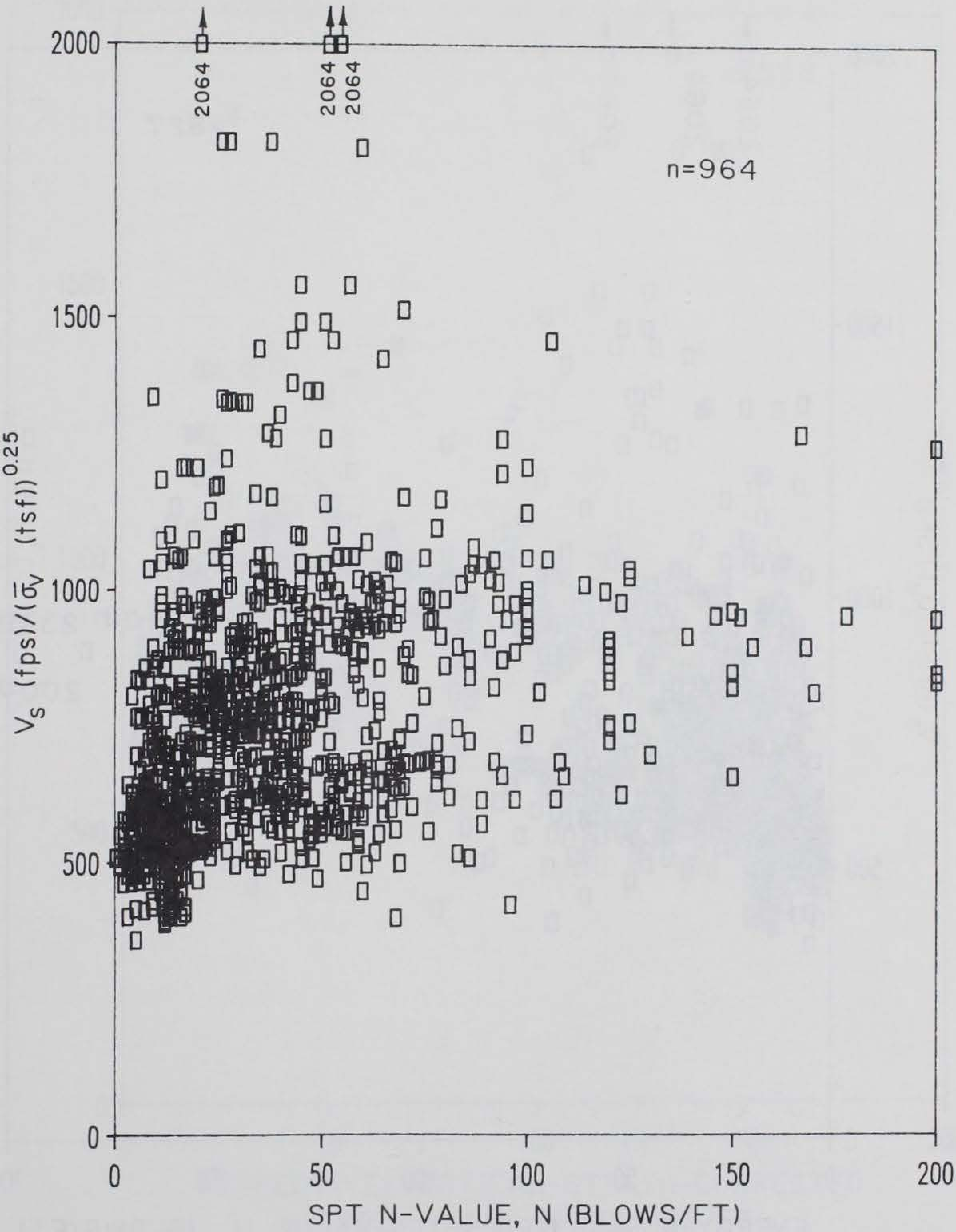


Figure 24. Correlation between SPT N-value and V_s normalized to $\bar{\sigma}_v^{0.25}$ for sands, silts, and clays

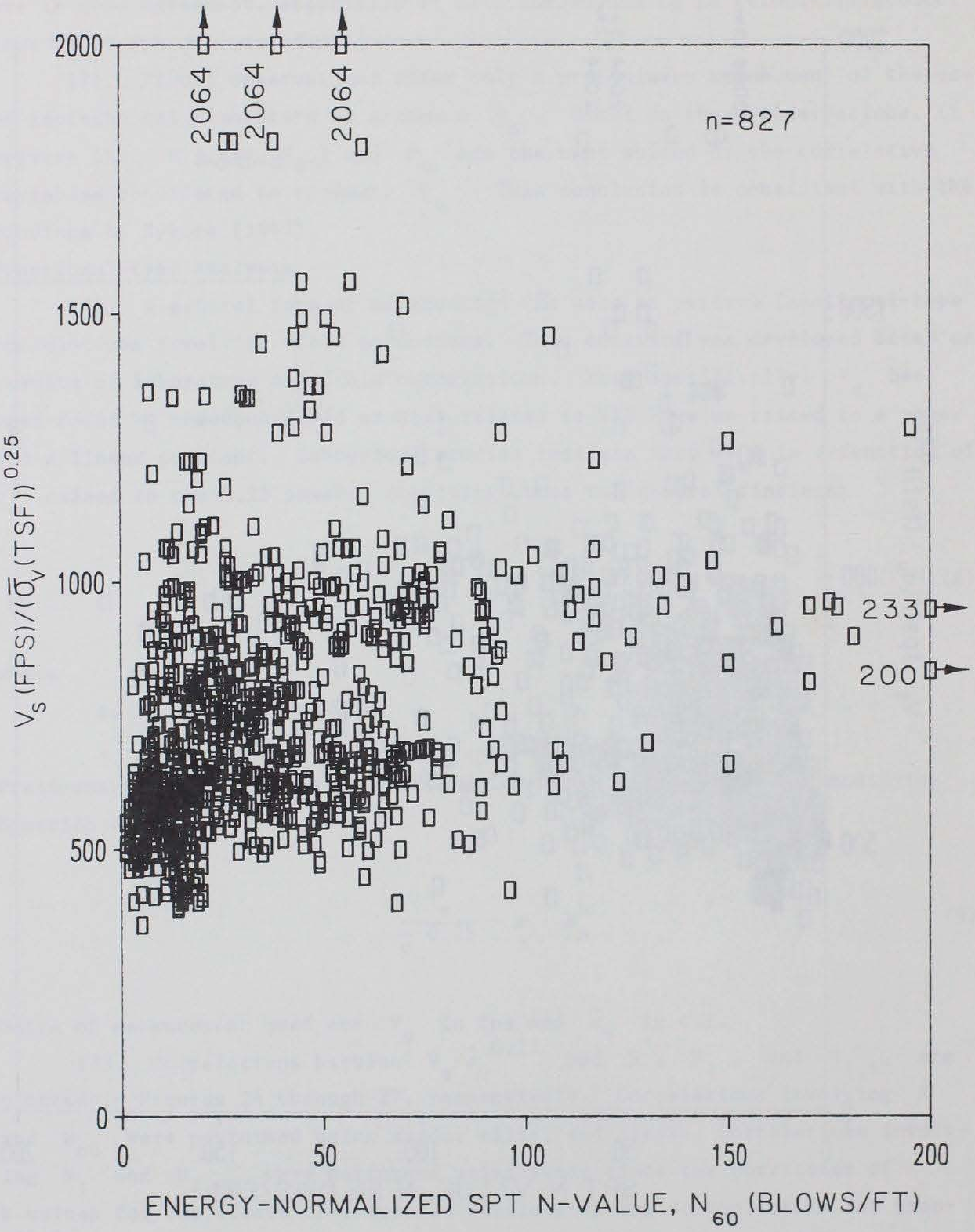


Figure 25. Correlation between SPT N_{60}-value and V_s normalized to $\bar{\sigma}_v^{\ 0.25}$ for sands, silts, and clays

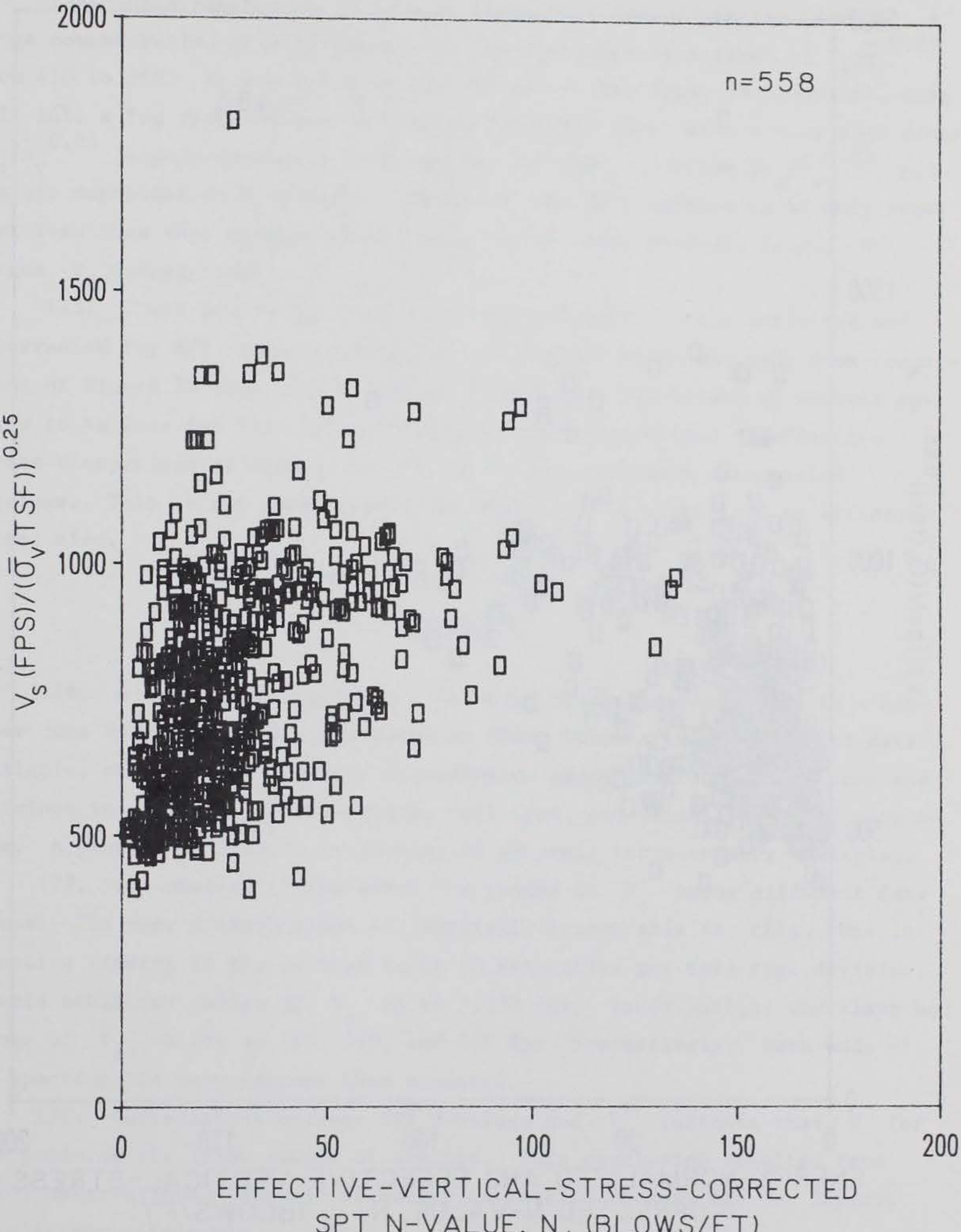


Figure 26. Correlation between SPT N_1-value and V_s normalized to $\bar{\sigma}_v^{\ 0.25}$ for sands

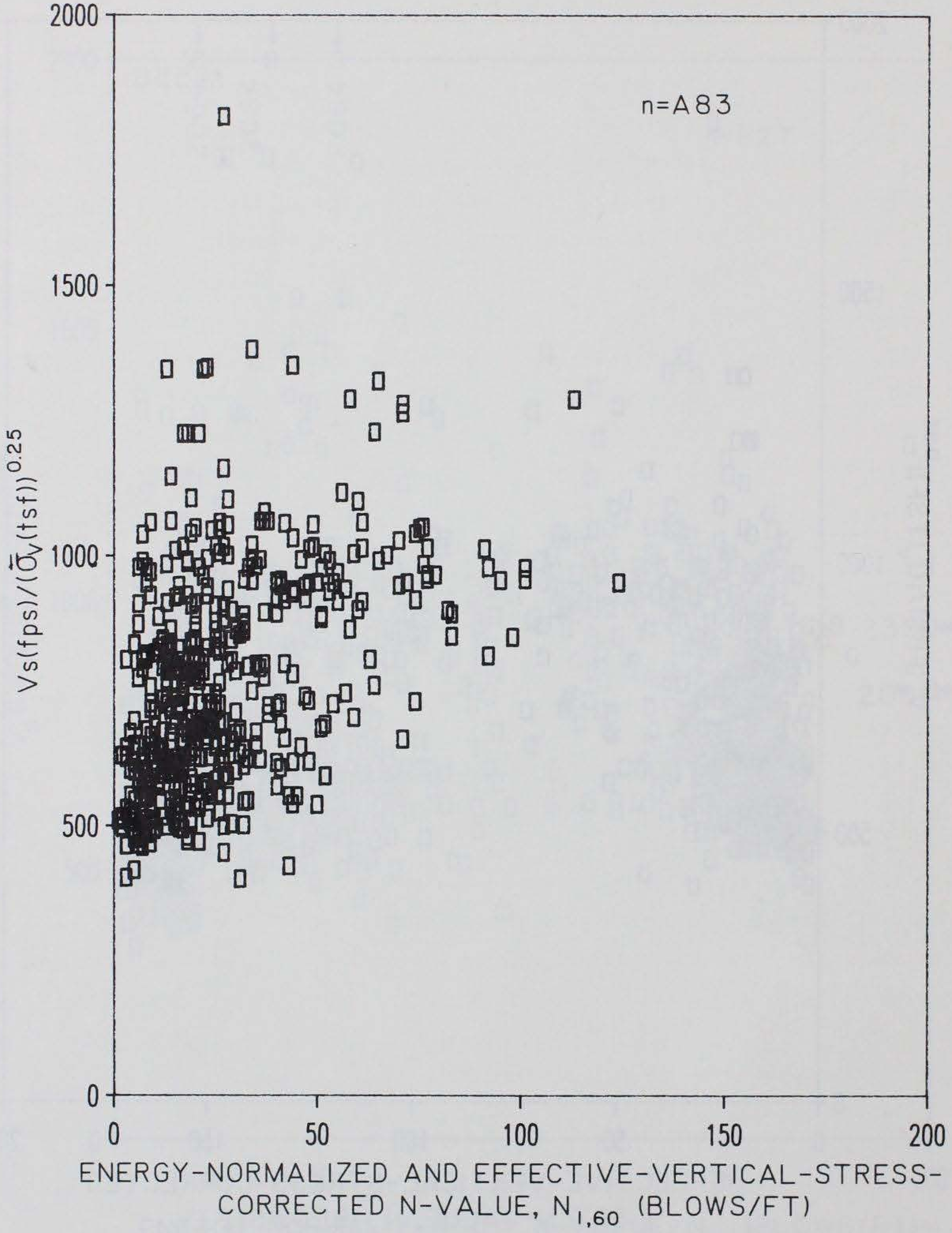


Figure 27. Correlation between SPT $N_{1,60}$-value and V_s normalized to $\bar{\sigma}_v^{0.25}$ for sands

174. Data plotted in Figures 24 through 27 have a similar pattern. A large concentration of data exists for low N_n-values at a range of $V_s/\bar{\sigma}_v^{0.25}$ from 450 to 550. At greater N_n-values (greater than about 10 blows/ft), data fall into a fan distribution bounded by imaginary lines with a very high slope ($V_s/\bar{\sigma}_v^{0.25}$ highly dependent on N_n-value) and moderate slope ($V_s/\bar{\sigma}_v^{0.25}$ moderately dependent on N_n-value). Scatter in the data appears to be only somewhat less than that associated with sensitivity-type analysis (e.g., N versus V_s correlations).

175. There are no noticeable differences between data corrected and uncorrected for SPT energy efficiency. This observation was made from comparisons of Figure 24 with 25 and Figure 26 with 27. The amount of scatter appears to be less for data plotted using N-values normalized to effective stress (comparison of Figures 24 and 26) as compared with uncorrected N-values. This is not unexpected given that V_s is normalized to effective stress also.

Discussion

176. A large quantity of data has been collected and stored in a computer data base conceived and created at WES. Despite the quantity of data available, obvious gaps exist in distribution among geographic locations and divisions in geologic age and origin, soil type, and seismic zonation categories. A paucity of data is available, in general, for secondary variables.

177. Few obvious trends exist for ranges in V_s among different data groups. The poor distributions are partially accountable for this. One interesting finding is the maximum range in velocities per soil type division. Gravels exhibited values of V_s up to 3,390 fps. Sands, silts, and clays had values of V_s as low as 285, 310, and 200 fps, respectively. Both ends of the spectrum are more extreme than expected.

178. Correlations between SPT N-values and V_s indicate that N (or N_{60}) produces the least amount of scatter. This conclusion resulted from visual observations. The use of energy correction factors did not improve correlations noticeably.

179. Correlations between $\bar{\sigma}_v$, σ_v, and depth and V_s indicate that the least amount of scatter is produced using $\bar{\sigma}_v$ as the correlative variable. The correlation involving σ_v had a larger amount of scatter, and

depth correlations had excessive scatter. This was expected. The scatter for $\bar{\sigma}_v$ versus V_s correlations is about the same as that for N versus V_s correlations.

180. Correlations involving values of V_s normalized to $\bar{\sigma}_v^{0.25}$ for functional-type analysis indicate that N_1 (or $N_{1,60}$) is the best correlative variable. The scatter is less for this type of analysis. Unlike correlations between N and V_s, N_1 (or $N_{1,60}$) was found to best correlate with $V_s/\bar{\sigma}_v^{0.25}$.

REFERENCES

American Society for Testing and Materials. 1987a. "Standard Method for Penetration Test and Split-Barrel Sampling of Soils," Designation: D 1586, Philadelphia, Pa.

_________. 1987b. "Standard Test Methods for Crosshole Seismic Testing," Designation: D 4428, Philadelphia, Pa.

Anderson, D., Espana, C., and McLamore, V. 1978. "Estimating In Situ Shear Moduli at Competent Sites," Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, American Society of Civil Engineers, Pasadena, Calif., Vol I, pp 181-197.

Arango, I., Moriwaki, Y., and Brown, F. 1978. "In-Situ and Laboratory Shear Velocity and Modulus," Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, American Society of Civil Engineers, Pasadena, Calif., Vol I, pp 198-212.

Auld, B. 1977. "Cross-hole and Down-hole V_s by Mechanical Impulse," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 103, No. 12, pp 1381-1398.

Ballard, R. F., Jr. 1976. "Method for Crosshole Seismic Testing," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 102, No. 12, pp 1261-1273.

Bazaroa, A. R. S. S. 1967. "Use of the Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sands," thesis in partial fulfillment of requirements for Ph.D., University of Illinois, Urbana, Ill.

Bishop, A. W. 1958. "Test Requirements for Measuring the Coefficient of Earth Pressure at Rest," Proceedings of the Conference on Earth Pressures Problems, Brussels, Belgium, Vol 1, pp 2-14.

Butler, D. K., and Curro, J. R., Jr. 1981. "Crosshole Seismic Testing -- Procedures and Pitfalls," Geophysics, Vol 46, No. 1, pp 23-29.

Butler, D. K., Skoglund, G. R., and Landers, G. B. 1978. "Crosshole: An Interpretive Computer Code for Crosshole Seismic Test Results," Miscellaneous Paper S-78-12, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Campbell, K., Chieruzzi, R., Duke, C., and Lew, M. 1979. "Correlations of Seismic Velocity with Depth in Southern California," School of Engineering and Applied Science, Report ENG-7965, University of California at Los Angeles, Los Angeles, Calif.

Chen, J. C., Lysmer, J., and Seed, H. B. 1981. "Analysis of Local Variations in Free Field Seismic Ground Motion," Earthquake Engineering Research Center, Report No. UCB/EERC-81/03, University of California, Berkeley, Calif.

Clough, R. W., and Woodward, R. J. 1967. "Analysis of Embankment Stresses and Deformations," Journal of the Soil Mechanic and Foundations Division, American Society of Civil Engineers, Vol 93, No. 4, pp 529-549.

De Mello, V. F. 1971. "The Standard Penetration Test," Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, Puerto Rico, Vol 1, pp 1-86.

Dobry, R., Stokoe, K. H., III, Ladd, R. S., and Youd, T. L. 1981. "Liquefaction Susceptibility From S-Wave Velocity," Proceedings of In Situ Tests to Evaluate Liquefaction Susceptibility, American Society of Civil Engineers, St. Louis, Mo.

Douglas, B. J., Olsen, R. S., and Martin, G. R. 1981. "Evaluation of the Cone Penetrometer Test for SPT-Liquefaction Assessment," Proceedings of In Situ Testing to Evaluate Liquefaction Susceptibility, American Society of Civil Engineers, St. Louis, Mo.

Headquarters, Department of the Army. 1970. "Engineering and Design Stability of Earth and Rock-fill Dams," Engineering Manual 1110-2-1902, Washington, DC.

_________. 1979. Geophysical Exploration, Engineering Manual 1110-1-1802, Washington, DC.

Finn, W., Bransby, P., and Pickering, D. 1970. "Effect of Strain History on Liquefaction of Sands," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol 96, No. 6, pp 1917-1934.

Franklin, A. G. 1979. "Proposed Guidelines for Site Investigations for Foundations of Nuclear Power Plants," Miscellaneous Paper GL-79-15, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Frydman, S. 1970. "Discussion," Geotechnique, The Institution of Civil Engineers, London, Vol XX, No. 4, pp 454-455.

Fumal, T. 1978. "Correlations Between Seismic Wave Velocities and Physical Properties of Geologic Material in the Southern San Francisco Bay Region, California," Open-File Report 78-1067, US Geological Survey, Menlo Park, Calif.

Gibbs, H., and Holtz, W. 1957. "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol 1, pp 35-39.

Hadala, P. F. 1973. "Effect of Constitutive Properties of Earth Media on Outrunning Ground Shock from Large Explosions," thesis presented to the faculty of the University of Illinois in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

Hamilton, E. 1976. "Shear Wave Velocity Versus Depth in Marine Sediments: A Review," Geophysics, Fall 1941, No. 5, pp 985-996.

Hardin, B., and Drnevich, V. 1972. "Shear Modulus and Damping in Soils: Design Equations and Curves," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol 98, No. 7, pp 667-691.

Hoar, R. J., and Stokoe, K. H., II. 1977. "Investigation of Variables Affecting In Situ Seismic Wave Velocity Measurements," Contract Report submitted to US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Imai, T., and Tonouchi, K. 1982. "Correlation of N Value with S-wave Velocity and Shear Modulus," Proceedings of the Second European Symposium on Penetration Testing, Amsterdam, The Netherlands, pp 67-72.

Knox, D., Stokoe, K., and Kopperman, S. 1982. "Effect of State of Stress on Shear Wave Velocity in Dry Sand," Geotechnical Engineering Report GR 82-23, University of Texas at Austin, Austin, Tex.

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

Kuo, J. H. C., and Stokoe, K. H., II. 1982. "Laboratory Investigation of Static and Dynamic Soil Properties of Three Harbor Road Sands After October 15, 1979, Imperial Valley Earthquake," Geotechnical Engineering Report No. GR 82-25, University of Texas at Austin, Austin, Tex.

Lee, S. H. H., and Stokoe, K. H., II. 1986. "Investigations of Low-Amplitude Shear Wave Velocity in Anisotropic Material," Geotechnical Engineering Report No. GR 86-6, University of Texas at Austin, Austin, Tex.

Lew, M., and Campbell, K. W. 1985. "Relationships Between Shear Wave Velocity and Depth of Overburden," Proceedings of Measurement and Use of Shear Wave Velocity for Evaluating Dynamic Soil Properties, American Society of Civil Engineers, Denver, Colo., pp 63-76.

Lysmer, J., Udaka, T., Tsai, C. F., and Seed, H. B. 1975. "FLUSH - A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems," Earthquake Engineering Research Center, Report No. UCB/EERC-75/30, University of California, Berkeley, Calif.

Makdisi, F. I., and Seed, H. B. 1977. "A Simplified Procedure for Estimating Dam and Embankment Earthquake - Induced Deformations," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 104, No. 7, pp 849-867.

Marcuson, W., III, and Bieganousky, W. 1977. "Laboratory Standard Penetration Tests on Fine Sands," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 103, No. 6, pp 565-588.

Marcuson, W. F., III, and Wahls, H. E. 1972. "Time Effects on Dynamic Shear Modulus of Clays," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol 98, No. 12, pp 1359-1373.

McLean, F., Franklin, A., and Dahlstrand, T. 1975. "Influence of Mechanical Variables on the SPT," Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, North Carolina State University, Raleigh, N.C., Vol 1, pp 287-318.

Melzer, K. 1971. Discussion of De Mello's paper "The Standard Penetration Test," Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, Puerto Rico, Vol III, pp 80-81.

Ohsaki, Y., and Iwasaki, R. 1973. "On Dynamic Shear Moduli and Poisson's Ratio of Soil Deposits," Soil and Foundations, Vol 13, No. 4, pp 61-73.

Ohta, Y., and Goto, N. 1978. "Physical Background of the Statistically Obtained S-Wave Velocity Equation in Terms of Soil Indexes," Butsuri-Tanko (Geophysical Exploration) (in Japanese, as translated by Y. Yamamoto), Vol 31, No. 1, pp 8-17.

Patel, N. 1981. "Generation and Attenuation of Seismic Waves in Downhole Testing," Geotechnical Engineering Thesis GT 81-1, University of Texas at Austin, Austin, Tex.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. 1974. Foundation Engineering, 2nd ed., Wiley, New York.

Perloff, W. H., Baladi, G. Y., and Harr, M. E. 1967. "Stress Distribution Within and Under Long Elastic Embankments," Highway Research Record, No. 181, p 12.

Randolph, M. F. 1980. "PIGLET: A Computer Program for the Analysis and Design of Pile Groups Under General Loading Conditions," Soil Report TR91, CUED/D, Cambridge University, Cambridge, England.

Richart, F. E., Jr., Hall, J. R., Jr., and Woods, R. D. 1970. Vibrations of Soils and Foundations, edited by N. M. Newmark and W. J. Hall, Prentice-Hall International Series in Theoretical and Applied Mechanics, Englewood Cliffs, N. J.

Schnabel, P. B., Lysmer, J., and Seed, H. B. 1972. "SHAKE - A Computer Program for Earthquake Resource Analysis of Horizontally Layered Sites," Report No. UCB/EERC-72/12, Earthquake Engineering Research Center, University of California, Berkeley, Calif.

Schultze, E., and Menzenbach, E. 1961. "Standard Penetration Test and Compressibility of Soils," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol 1, pp 527-532.

Seed, H. B., and Idriss, I. M. 1970. "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley, Calif.

________. 1981. "Evaluation of Liquefaction Potential of Sand Deposits Based on Observations of Performance in Previous Earthquakes," Proceedings of the Conference on In Situ Testing to Evaluate Liquefaction Susceptibility, American Society of Civil Engineers, St. Louis, Mo.

Seed, H. B., Idriss, I. M., and Arango, I. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 109, No. 3, pp 458-482.

Seed, H., Mori, K., and Chan, C. 1977. "Influence of Seismic History on Liquefaction of Sands," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 103, No. 4, pp 257-270.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. 1985. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 111, No. 12, pp 1425-1445.

Seed, H. B., Wong, R. T., Idriss, I. M., and Tokimatsu, K. 1984. "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Report No. UCB/EERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, Calif.

Skempton, A. W. 1961. "Horizontal Stresses in an Overconsolidated Eocene Clay," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol 1, pp 351-357.

Sowers, G. F., Robb, A. D., Mulilis, C. H., and Glenn, A. J., 1957. "The Residual Lateral Pressures Produced by Compacting Soils," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol II, pp 243-247.

Stokoe, I., II, and Hoar, R. 1978. "Variables Affecting In Situ Seismic Measurements," Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, American Society of Civil Engineers, Pasadena, Calif., Vol II, pp 919-939.

Stokoe, K. H., II. 1980. "Field Measurement of Dynamic Soil Properties," Proceedings of the Conference on Civil Engineering and Nuclear Power, American Society of Civil Engineers, Knoxville, Tenn.

________. 1986. Letter report summarizing work performed under Purchase Order DACA 39-86-M-0571, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Sugimura, Y. 1977. "Seismic Strain Induced in the Ground During Earthquakes," Report No. UCB/EERC-77/14, Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, Calif.

Sykora, D. W. "Examination of Existing Shear Wave Velocity and Shear Modulus Correlations In Soils" (in publication) US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Sykora, D. W., and Stokoe, K. H., II. 1982. "Seismic Investigation of Three Harbor Road Sites after October 15, 1979 Imperical Valley Earthquake," Geotechnical Engineering Report No. GR 82-24, University of Texas at Austin, Austin, Tex.

________. 1983. "Correlations of In Situ Measurements in Sands With Shear Wave Velocity," Geotechnical Engineering Report No. GR 83-33, University of Texas at Austin, Austin, Tex.

Tono, I. 1971. "Continuous Motion from the Viewpoint of Mechanics," Metamorphic Action (in Japanese, translated by H. Umehara, as reported by Ohta and Goto (1978)), Vol 17, pp 95-105.

US Army Engineer Waterways Experiment Station. 1960. "The Unified Classification System," Technical Memorandum No. 3-357, Vicksburg, Miss.

US Naval Facilities Engineering Command. 1982. Foundations and Earth Structures, DM-7.2, Alexandria, Va., p 76.

Woods, R. D. 1978. "Measurement of Dynamic Soil Properties," Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, American Society of Civil Engineers, Pasadena, Calif., Vol I, pp 91-118.

________. 1986. "In Situ Tests for Foundation Vibrations," Proceedings of Specialty Conferences on Use of In Situ Tests in Geotechnical Engineering, American Society of Civil Engineers, Blacksburg, Va., pp 336-375.

Wu, S., Gray, D. H., and Richart, F. E., Jr. 1984. "Capillary Effects on Dynamic Modulus of Sands and Silts," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 110, No. 9, pp 1188-1203.

APPENDIX A: AUTHOR INDEX

APPENDIX B: PROGRAM STRESS.F

1. Program STRESS.F was written expressly for the purpose of calculating values of stress and effective-stress correction factors C_N (used to determine N_1) at desired increments of depth. The program uses input density values and depth to the phreatic surface to calculate effective vertical stress $\bar{\sigma}_v$, total vertical stress σ_v, and C_N for relative density D_r ranges of 40-60 percent and 60-80 percent as per Seed et al. (1985).* The user may then select desired values from a table produced as output.

2. Program STRESS.F was written in the FORTRAN programming language for use on both IBM-compatible computers and a MASSCOMP 5500 computer. It uses interactive screen input requiring the following values:

- Project name.
- Maximum depth of interest.
- Depth of the phreatic surface.
- Density profile.
- Output increments of depth.

3. Total vertical stress is defined as:

$$\sigma_v = \gamma_m \cdot z \tag{B1}$$

where

γ_m = moist unit weight (density)

z = depth

4. Effective vertical stress is defined as:

$$\bar{\sigma}_v = \sigma_v - u \tag{B2}$$

where

u = hydrostatic pressure = $62.4 \cdot z_w$ (psf) (B3)

z_w = depth below phreatic surface (ft) (z_w = 0 above phreatic surface)

5. Values of C_N were calculated using equations derived from Figure 6 (Seed et al. 1981). These equations are:

---

* References cited in this Appendix are included in the References at the end of the main text.

for $\bar{\sigma}_v < 1.0$ tsf:

$$C_N = 1.0 - (1.25 \cdot \log \bar{\sigma}_v) \tag{B4}$$

for $\bar{\sigma}_v < 1.0$ tsf:

$$C_N = 1.0 \tag{B5}$$

for $1.0 < \bar{\sigma}_v < 5.0$ tsf and $D_r < 60\%$:

$$C_N = 1.02 - \bar{\sigma}_v^{-0.577} \tag{B6}$$

for $1.0 < \bar{\sigma}_v < 5.0$ tsf and $D_r \geqq 60\%$:

$$C_N = 0.98 - (0.76 \cdot \log \bar{\sigma}_v) \tag{B7}$$

for $\bar{\sigma}_v \geqq 5.0$ tsf and $D_r < 60\%$:

$$C_N = 0.4 \tag{B8}$$

for $\bar{\sigma}_v \geqq 5.0$ tsf and $D_r \geqq 60\%$:

$$C_N = 0.45 \tag{B9}$$

6. Program STRESS.F uses densities input to select density boundaries. The middepth between two density values is set to be the boundary between the two values. Therefore, for each density input, the computer program creates a layer of uniform density. Density above the first depth of density input and below the last depth of density input are assumed to equal these first and last values, respectively.

7. This method of analysis may not be desirable in certain instances where well-defined layers of known uniform density exist. However, the data input can be manipulated to produce the desired effect. For this case, densities are input at depths such that layer boundaries correspond to those in situ.

8. Two examples of input producing desired results are provided.

Case I uses the program to determine arbitrary boundaries at middepth between density input depths. The phreatic surface is set at 20 ft and four densities are used. A profile summarizing Case I is shown in Figure B1. The output for Case I is found on page B6. Case II is summarized in Figure B2 for a profile of known uniform-density layers and desired boundaries. It was assumed that the phreatic surface was not in the range of interest. The resulting input is shown in Figure B2 with output on page B8. Note that there is more than one method to produce the profile desired for Case II.

9. The Fortran source code for IBM-compatible PC computers is provided on pages B9 to B12.

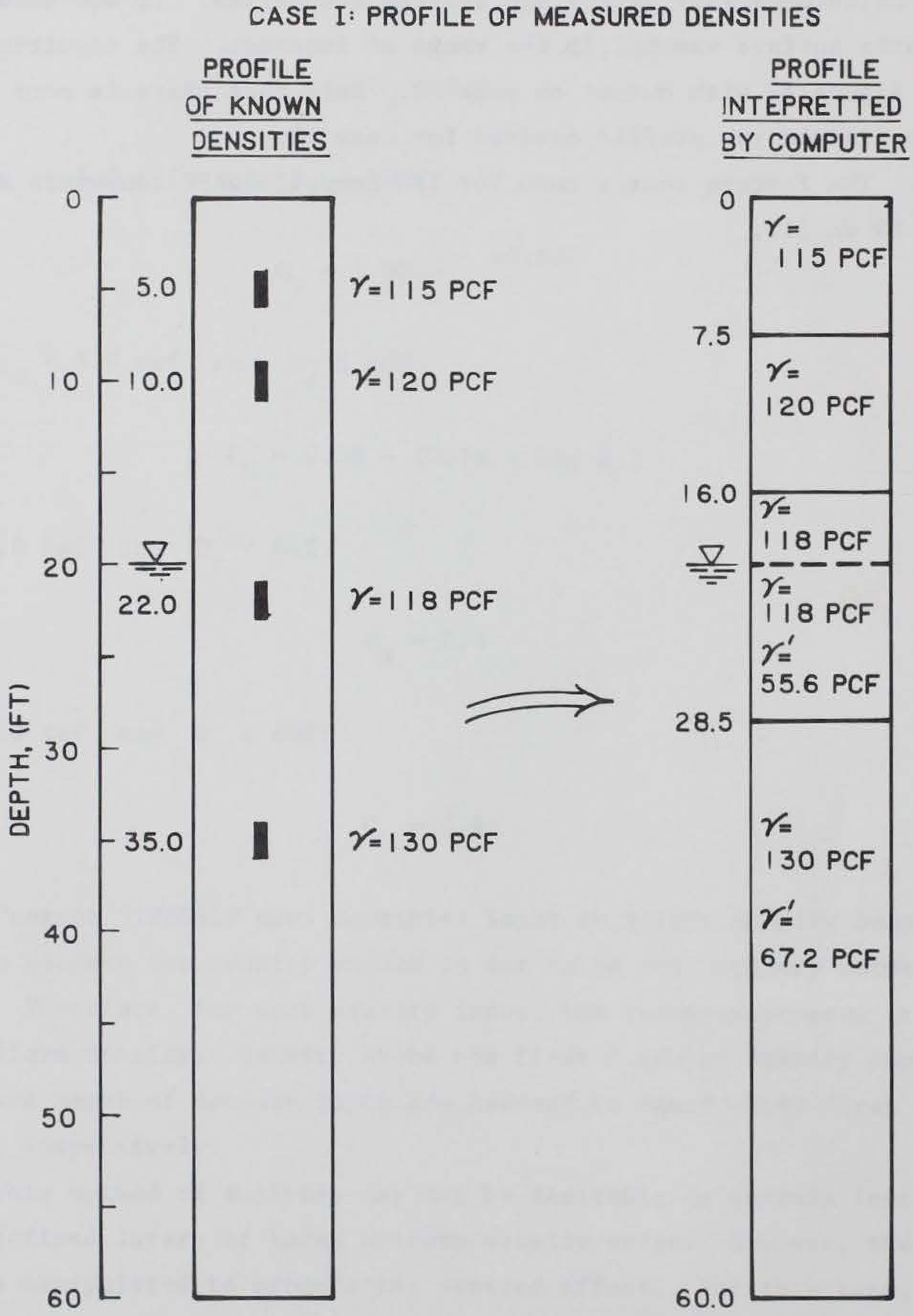


Figure B1. Input and interpreted profiles for Case I

************************PROGRAM STRESS ************************

The name of the project is: CASE I

The maximum depth (ft) of interest is :
60.0

The depth (ft) of the phreatic surface is :
20.0

The input depths and densities are:

DEPTH (ft)	MOIST DENSITY (pcf)
5.0	115.0
10.0	120.0
22.0	118.0
35.0	130.0

Effective stress calculations are considered
inaccurate for depths (ft) less than :
20.0

DEPTH (ft)	MOIST DENSITY (pcf)	TOTAL STRESS (tsf)	EFFECTIVE STRESS (tsf)	N CORRECTION 40-60%	60-80%
2.0	115.0	0.12	0.12	2.17	2.17
4.0	115.0	0.23	0.23	1.80	1.80
6.0	115.0	0.34	0.34	1.58	1.58
8.0	120.0	0.46	0.46	1.42	1.42
10.0	120.0	0.58	0.58	1.29	1.29
12.0	120.0	0.70	0.70	1.19	1.19
14.0	120.0	0.82	0.82	1.11	1.11
16.0	120.0	0.94	0.94	1.03	1.03
18.0	118.0	1.06	1.06	0.99	0.96
20.0	118.0	1.18	1.18	0.93	0.93
22.0	118.0	1.30	1.23	0.90	0.91
24.0	118.0	1.41	1.29	0.88	0.90
26.0	118.0	1.53	1.34	0.86	0.88
28.0	118.0	1.65	1.40	0.84	0.87
30.0	130.0	1.78	1.46	0.82	0.85
32.0	130.0	1.91	1.53	0.80	0.84
34.0	130.0	2.04	1.60	0.78	0.82
36.0	130.0	2.17	1.67	0.76	0.81
38.0	130.0	2.30	1.73	0.74	0.80
40.0	130.0	2.43	1.80	0.73	0.79
42.0	130.0	2.56	1.87	0.71	0.77
44.0	130.0	2.69	1.94	0.70	0.76
46.0	130.0	2.82	2.01	0.68	0.75
48.0	130.0	2.95	2.07	0.67	0.74
50.0	130.0	3.08	2.14	0.66	0.73
52.0	130.0	3.21	2.21	0.65	0.72
54.0	130.0	3.34	2.28	0.63	0.71
56.0	130.0	3.47	2.34	0.62	0.70
58.0	130.0	3.60	2.41	0.61	0.69
60.0	130.0	3.73	2.48	0.60	0.68

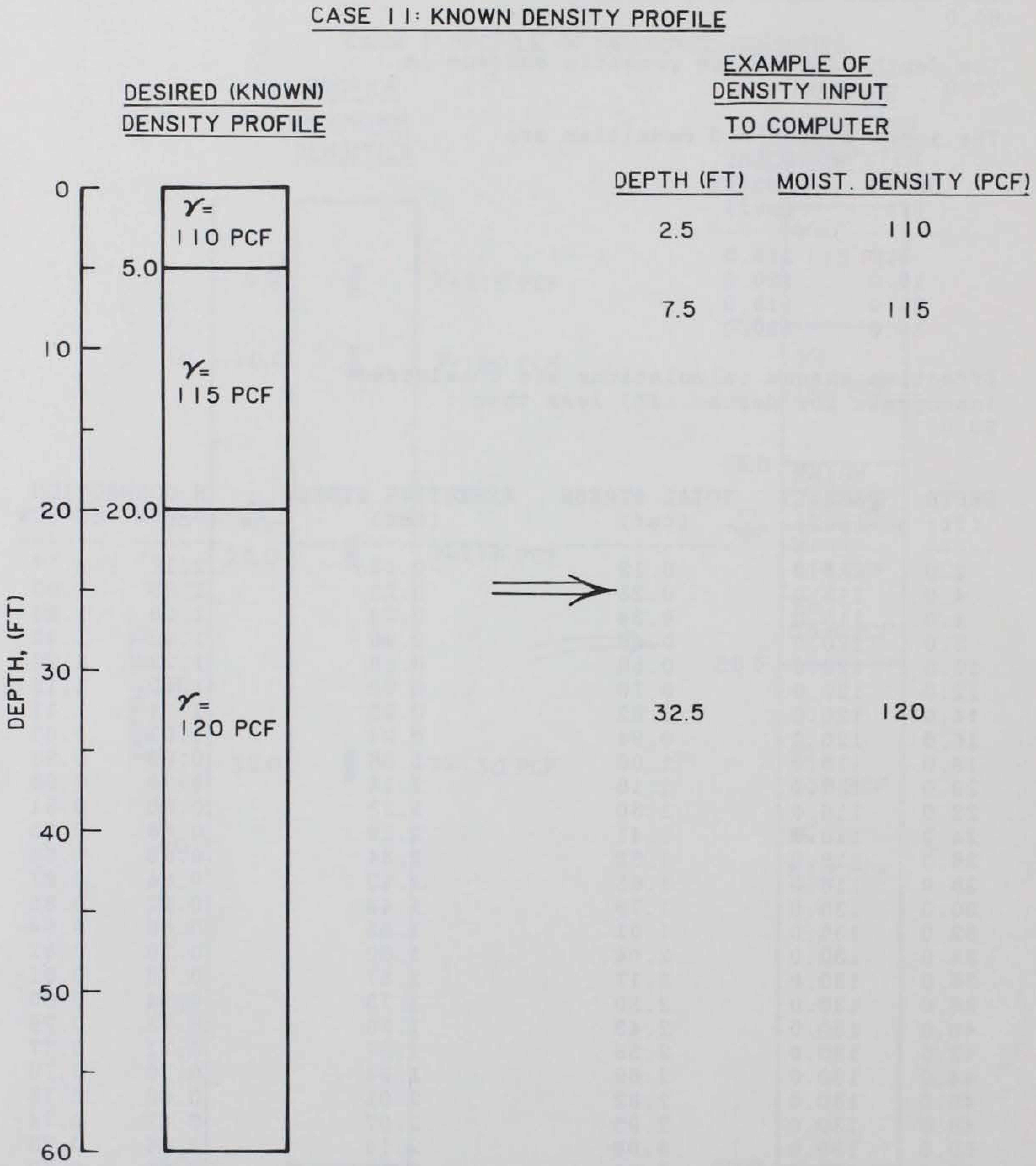


Figure B2. Input and interpreted profiles for Case II

************************PROGRAM STRESS *************************

The name of the project is: CASE II

The maximum depth (ft) of interest is :
60.0

The depth (ft) of the phreatic surface is :
999.0

The input depths and densities are:

DEPTH (ft)	MOIST DENSITY (pcf)
2.5	110.0
7.5	115.0
32.5	120.0

Effective stress calculations are considered inaccurate for depths (ft) less than :
999.0

DEPTH (ft)	MOIST DENSITY (pcf)	TOTAL STRESS (tsf)	EFFECTIVE STRESS (tsf)	N CORRECTION 40-60%	60-80%
2.0	110.0	0.11	0.11	2.20	2.20
4.0	110.0	0.22	0.22	1.82	1.82
6.0	115.0	0.33	0.33	1.60	1.60
8.0	115.0	0.45	0.45	1.44	1.44
10.0	115.0	0.56	0.56	1.31	1.31
12.0	115.0	0.68	0.68	1.21	1.21
14.0	115.0	0.79	0.79	1.13	1.13
16.0	115.0	0.91	0.91	1.05	1.05
18.0	115.0	1.02	1.02	1.01	0.97
20.0	115.0	1.14	1.14	0.95	0.94
22.0	120.0	1.26	1.26	0.89	0.90
24.0	120.0	1.38	1.38	0.85	0.87
26.0	120.0	1.50	1.50	0.81	0.85
28.0	120.0	1.62	1.62	0.77	0.82
30.0	120.0	1.74	1.74	0.74	0.80
32.0	120.0	1.86	1.86	0.71	0.78
34.0	120.0	1.98	1.98	0.69	0.75
36.0	120.0	2.10	2.10	0.67	0.74
38.0	120.0	2.22	2.22	0.64	0.72
40.0	120.0	2.34	2.34	0.62	0.70
42.0	120.0	2.46	2.46	0.61	0.68
44.0	120.0	2.58	2.58	0.59	0.67
46.0	120.0	2.70	2.70	0.58	0.65
48.0	120.0	2.82	2.82	0.56	0.64
50.0	120.0	2.94	2.94	0.55	0.62
52.0	120.0	3.06	3.06	0.54	0.61
54.0	120.0	3.18	3.18	0.52	0.60
56.0	120.0	3.30	3.30	0.51	0.59
58.0	120.0	3.42	3.42	0.50	0.57
60.0	120.0	3.54	3.54	0.49	0.56

```
C      ****************** PROGRAM STRESS ***********************
C
C
C             Program STRESS.F calculates a table of effective and
C      total overburden stresses for a given column of known densities.
C      Effective-vertical-stress correction factors for two relative
C      density ranges (as per Seed, et al. 1981) are also calculated
C      to determine respective values of N1.
C             This program was written primarily for use in a seismic
C      velocity correlation study but also could be used in numerous
C      other applications in geotechnical engineering.
C             The program divides the soil column into zones of equal
C      density based on the distance between known densities.  For ex-
C      ample, if the densities are known at depths of 10 and 20 feet,
C      the computer will assign a uniform density from 0 to 15 feet
C      and another uniform density below 15 feet.  The program may be
C      manipulated to provide a detailed density profile.
C
C      Variable List:
C
C      D = Depth of unit weight (ft)
C      DD = Unit Weight (pcf)
C      Y = Depths of density transition
C      YY = Depths for stress and CN calculations & output
C      DINC = Increment of depth for stress output (ft)
C      CN4 = SPT N-value correction for relative density of 40-60%
C      CN6 = SPT N-value correction for relative density of 60-80%
C
C
       PROGRAM STRESS
C
       CHARACTER * 12 NAME
       INTEGER I,IMAX,IMAX2
       REAL MAXD,D,DD,Y,WT,DINC,CN4,CN6
       DIMENSION D(100),DD(100),Y(100),YY(600)
C
C      Print instructions to the terminal
C
       PRINT *,'     Program STRESS will calcualte the effective and',
     &  ' total vertical stress'
       PRINT *,'at specified increments of depth.  The operator must',
     &  ' input the maximum '
       PRINT *,'depth of interest, all known densities and their',
     &  ' corresponding depths,'
       PRINT *,'and the depth of the water table.'
C
C      Interactive Data Entry
C
       PRINT*
       PRINT*,'Enter a project code in single quotations: '
       READ*,NAME
C
       PRINT*
       PRINT *,'Enter the maximum depth of interest: '
       PRINT*
       READ(*,5) MAXD
C
       PRINT*
       PRINT*,'Enter the depth of the water table: '
       PRINT*
```

```
      READ(*,5) WT
C
5     FORMAT (F5.1)
      PRINT*
      PRINT*,'Enter one set of depth and corresponding',
     & ' moist unit weight data per '
      PRINT*,'line separated by a space (maximum of 100 sets).'
      PRINT*,'Enter the data in order of increasing depth.'
      PRINT*,'Do not enter more than one value of density for any',
     & ' given depth.'
      PRINT*,'Enter 0 0 when data entry is complete'
      PRINT*
C
C     Print input information to output file
C
      WRITE (10,*) '************************PROGRAM STRESS ',
     & '*************************'
      WRITE (10,*)
      WRITE (10,*) 'The name of the project is:  ',NAME
      WRITE (10,*)
      WRITE (10,*) 'The maximum depth (ft) of interest is :'
      WRITE (10,5) MAXD
      WRITE (10,*)
      WRITE (10,*) 'The depth (ft) of the phreatic surface is :'
      WRITE (10,5) WT
      WRITE (10,*)
      WRITE (10,*) 'The input depths and densities are:'
      WRITE (10,*) '                     MOIST '
      WRITE (10,*) '     DEPTH     DENSITY'
      WRITE (10,*) '      (ft)       (pcf)'
      WRITE (10,*) '     -----     -------'
C
C     Read depth and density input from terminal
C
      D(0)=0.0
      I=0
100   DO 140 J=1,100
      I=I+1
      READ*,D(I),DD(I)
      IF (D(I) .EQ. 0.) GOTO 145
      IF (D(I) .LE. D(I-1)) GOTO 135
      IF (D(I) .GT. MAXD) GOTO 130
      WRITE (10,101) D(I) ,DD(I)
101   FORMAT (5X,2(F5.1,5X))
      WRITE(*,102) D(I),DD(I),I
102   FORMAT (5x,F5.1,5x,F5.1,5x,I3)
      PRINT*
      GOTO 140
130   PRINT*
      PRINT*,'The entered value of depth exceeds the maximum profile',
     & 'depth.'
      PRINT*,'It has therefore been rejected!'
      PRINT*
      GOTO 139
135   PRINT*
      PRINT*,'The entered value of depth is not in increasing order.'
      PRINT*,'It has therfore been rejected!'
      PRINT*
139   I=I-1
140   CONTINUE
```

```
145     CONTINUE
        IMAX=I-1
        PRINT *, IMAX,' Pairs of data have been accepted for the profile
        PRINT*
        PRINT*,'Enter the increment of depth (ft) for stress output:
        READ*,DINC
C
        WRITE (10,*)
        WRITE (10,*) 'Effective stress calculations are considered'
        WRITE (10,*) 'inaccurate for depths (ft) less than :'
        WRITE (10,5) WT
        WRITE (10,*)
C
C       Calculate intermediate (density transition) depths
C
        IMAX2=IMAX-1
        DO 150 I=1,IMAX2
        Y(I)=(D(I+1)+D(I))/2.0
150     CONTINUE
        Y(IMAX)=MAXD
C
C       Print Table headings
C
        WRITE (10,160)
        WRITE (10,161)
        WRITE (10,162)
        WRITE (10,163)
160     FORMAT (10x,'MOIST')
161     FORMAT (1X,'DEPTH',3X,'DENSITY',3X,'TOTAL STRESS',3X,
     &  'EFFECTIVE STRESS',5X,'N CORRECTION')
162     FORMAT (2x,'(ft)',4x,'(pcf)',7x,'(tsf)',12x,'(tsf)',10x,
     &  '40-60%',2x,'60-80%')
163     FORMAT (1x,'-----',3x,'-------',3x,'------------',3x,
     &  '----------------',4x,'------',2x,'------')
C
C       Calculate tatal and effective stresses at incremental depths
C
        YY(0)=0.0
        Y(0)=0.0
        DD(0)=DD(1)
        II=0
        TS=0
        ES=0
        DO 300 I=1,IMAX
        PRINT*,I,IMAX
        DO 200 J=1,100
        II=II+1
        YY(II)=YY(II-1)+DINC
        PRINT*,YY(II),Y(I)
        IF (YY(II) .GT. Y(I)) GOTO 170
        GOTO 175
170     TS=TS+(((Y(I)-YY(II-1))*DD(I)+(YY(II)-Y(I))*DD(I+1))/2000.)
        DD(I)=DD(I+1)
        GOTO 180
175     TS=TS+(((YY(II)-YY(II-1))*DD(I))/2000.)
180     IF (YY(II) .GT. WT) THEN
                ES=TS-(((YY(II)-WT)*62.4)/2000.)
        ELSE
                ES=TS
        ENDIF
```

```
      IF (ES .LT. 1.0) GOTO 190
      IF (ES .GT. 5.0) GOTO 191
      CN4=1.02*(ES**(-0.577))
      CN6=0.98-(0.76*(LOG10(ES)))
      GOTO 195
190   CN4=1.0-(1.25*(LOG10(ES)))
      CN6=1.0-(1.25*(LOG10(ES)))
      GOTO 195
191   CN4=0.40
      CN6=0.45
195   CONTINUE
      IF (YY(II) .GT. MAXD) GOTO 400
      WRITE (10,196) YY(II),DD(I),TS,ES,CN4,CN6
196   FORMAT (2(F6.1,3x),3x,F6.2,11x,F6.2,9x,2(F6.2,2x))
      IF (YY(II) .GT. Y(I)) GOTO 300
200   CONTINUE
300   CONTINUE
400   CONTINUE
      CLOSE (10)
      END
```