TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWATX78-1765F		

4. Title and Subtitle	5. Report Date
DESIGN PROCEDURES FOR AXIALLY LOADED DRILLED SHAFTS	December 1977
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Gerardo W. Quiros and Lymon C. Reese	Research Report 176-5F
9. Performing Organization Name and Address	10. Work Unit No.
Center for Highway Research The University of Texas at Austin Austin, Texas 78712	11. Contract or Grant No. Research Study 3-5-72-176
	13. Type of Report and Period Covered
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763	Final
	14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.
Research Study Title: "The Behavior of Drilled Shafts"

16. Abstract

This report represents the culmination of a series of studies that have been conducted on the behavior of axially loaded drilled shafts. Of primary interest are shafts that receive support from the soil along the sides of the shaft.

Design procedures established from extensive research on drilled shafts installed in clays, clay-shales, sand, and cemented fine-grained alluvial fan deposits are evaluated and summarized. Revised correlations between dynamic penetration resistance and undrained shear strength for clays, and between dynamic penetration resistance and load transfer in sand are presented. The developed computer programs SHAFT1 and BSHAFT were developed to shorten the time involved in computing curves giving ultimate capacity versus depth. Check design problems utilizing SHAFT1 and BSHAFT are included in this report.

Uncertainty of in situ soil properties remains a major obstacle in establishing a more generalized design procedure. However, the design procedures presented herein will give satisfactory predictions of shaft ultimate capacities.

17. Key Words	18. Distribution Statement
drilled shafts, axially loaded, design procedures, clays, clay-shales, sand, cemented fine-grained alluvial fan deposits	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	176	

Form DOT F 1700.7 (8-69)

DESIGN PROCEDURES FOR AXIALLY LOADED DRILLED SHAFTS

by

Gerardo W. Quiros
Lymon C. Reese

Research Report 176-5F

The Behavior of Drilled Shafts
Research Project 3-5-72-176

conducted for

Texas
State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

December 1977

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

This report presents procedures for the design of drilled shafts under axial loading and is the final report on Project 3-5-72-176. A comprehensive review was made of existing data, design procedures were formulated, and computer programs were written to facilitate the required analyses.

The authors wish to thank the State Department of Highways and Public Transportation for their sponsorship of the work and to express appreciation for the assistance given by many members of their staff. Appreciation is also expressed to Dr. Kenneth H. Stokoe II who made many helpful suggestions during the preparation of the manuscript, to Mrs. Cathy L. Collins for her patience in the typing of the many drafts, and to Mrs. Joan Cantu for excellent work in the preparation of the figures.

Gerardo W. Quiros

Lymon C. Reese

December 1977

LIST OF REPORTS

Report No. 176-1, "The Behavior of Axially Loaded Drilled Shafts in Sand," by Fadlo T. Touma and Lymon C. Reese, presents the results of an investigation of the behavior of drilled shafts in sand.

Report No. 176-2, "The Behavior of an Axially Loaded Drilled Shaft Under Sustained Loading," by John A. Wooley and Lymon C. Reese, presents the results of an investigation of a drilled shaft under sustained loading.

Report No. 176-3, "Behavior of Three Instrumented Drilled Shafts Under Short Term Axial Loading," by Donald E. Engeling and Lymon C. Reese, is concerned with the analysis of the behavior of drilled shafts under axial loading.

Report No. 176-4, "Behavior of Axially Loaded Drilled Shafts in Clay-Shales," by Ravi P. Aurora and Lymon C. Reese, presents their results of an investigation concerning the behavior of drilled shafts in clay-shales.

Report No. 176-5F, "Design Procedures for Axially Loaded Drilled Shafts," by Gerardo W. Quiros and Lymon C. Reese, summarizes the recommended design procedures for axially loaded drilled shafts in clay, sand, and cemented fine-grained alluvial fan deposits established from the results of recent intensive investigations.

ABSTRACT

This report represents the culmination of a series of studies that have been conducted on the behavior of axially loaded drilled shafts. Of primary interest are shafts that receive support from the soil along the sides of the shaft.

Design procedures established from extensive research on drilled shafts installed in clays, clay-shales, sand, and cemented fine-grained alluvial fan deposits are evaluated and summarized. Revised correlations between dynamic penetration resistance and undrained shear strength for clays, and between dynamic penetration resistance and load transfer in sand are presented. The developed computer programs SHAFT1 and BSHAFT were developed to shorten the time involved in computing curves giving ultimate capacity versus depth. Check design problems utilizing SHAFT1 and BSHAFT are included in this report.

Uncertainty of in situ soil properties remains a major obstacle in establishing a more generalized design procedure. However, the design procedures presented herein will give satisfactory predictions of shaft ultimate capacities.

SUMMARY

This study presents the design procedures for axially loaded drilled shafts thus far developed from extensive research programs. These procedures can be used for the design of shafts installed in clay, clay shale, sand, or alluvial fan deposits that are cemented and fine-grained.

A review of data from previous load tests was undertaken and some revisions in previous design procedures are suggested. A major portion of the revisions involve the correlation between dynamic penetration resistance and undrained shear strength of cohesive soils, and the correlation between dynamic penetration resistance and load transfer in the case of sand.

Two computer programs are presented which can serve as time-saving tolls when making computations of ultimate load capacity. One program is based on the primary design procedure for computation of ultimate load capacity; that is, the computer program requires information on soil properties obtained from an extensive field exploratory investigation. The other computer program is based on the secondary design procedure for computation of ultimate load capacity. In this instance, the computer program only requires results from dynamic penetration tests. A detailed explanation of each program is given, along with example problems.

IMPLEMENTATION STATEMENT

The design procedures in this report are recommended for use in design offices of the Texas Highway Department. The proposed design methods and computer programs can be useful in determining the ultimate capacity for a drilled shaft of a given size. Further, the methods can be used to investigate a number of geometries in order to achieve the most economical design.

The methods presented herein will generally give a conservative design but the designs should be more economical than those made using previously available methods. The engineer should employ caution, however, when utilizing results from dynamic penetration tests because of the limited reliability of these tests.

The methods presented in this report should not be employed for the design of drilled shafts for areas or for soil profiles that have not been investigated by the performance of a load test of an instrumented drilled shaft. However, the methods will be useful in the detailed design of the test shaft.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1. INTRODUCTION

RECENT STUDIES ON DRILLED SHAFTS

Cooperative Program in Texas

Since 1965, the Center for Highway Research at The University of Texas at Austin, in cooperation with the State Department of Highways and Public Transportation, has conducted extensive research on the behavior of axially loaded drilled shafts. A series of reports has been published on the topic, with Report No. 89-1, by Reese and Hudson (1968), laying the foundation for the investigations.

Reese and Hudson (1968) established a plan of research for investigating the load-carrying capacity of such shafts through field tests. The scheme called for developing instrumentation to obtain information on the soil and shaft interaction, performing full-scale load tests of drilled shafts, determining significant soil properties, and using the results from field and laboratory tests to develop theories of drilled shaft behavior. Subsequently, these theories were to be verified with additional load tests and a design procedure was to be established from the results of the studies.

Pursuing the research objectives established by Reese and Hudson (1968), Reese, Brown, and Dalrymple (1968) reported the conclusions of their study involving the design, construction, and testing of instrumentation capable of measuring lateral earth pressures along a drilled shaft. This project produced a pressure cell that measured lateral pressures against a drilled shaft up to 50 psi. The cell was used to measure the pressures under curing as well as under loading conditions. The authors felt that the lateral-pressure measuring gage was adequate for drilled shafts installed in sand or in clay strata.

Sequentially, the Center for Highway Research published the results of an investigation regarding the nature of moisture migration from unset cement mortar to soil (Chuang and Reese, 1969). The study also consisted of an investigation of the interaction between the soil and the cement mortar. Over 200 cement mortar-soil samples were tested to determine the factors affecting moisture migration, and another 70 such samples were tested to study the shear strength reduction factor for soils adjacent to fresh concrete. Information was presented on moisture migration, and shear strength reduction factors (α) were suggested for estimating the skin friction of drilled shafts in homogenous clay.

Because the research project on drilled shafts also included the need for determining soil properties, Ehlers, Reese, and Anagnos (1969) conducted an investigation for the purpose of studying the capabilities, limitations, and problems associated with the nuclear method of moisture determination at depth. Nuclear equipment manufactured by Troxler Electronic Laboratories was used to make measurements of moisture change. Briefly, the results of the investigation demonstrated that the nuclear method was a fast and efficient means of determining the soil moisture content. The accuracy of the method established by using results from gravimetric testing, proved to be satisfactory. Recalibration of the nuclear equipment was found not necessary.

Campbell and Hudson (1969) completed an investigation which involved methods of in situ determination of soil properties. In their report, the authors were concerned with three devices: the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and the University of Texas In Situ shear strength testing device. Each device was described and evaluated in reference to equipment, testing procedure, data analysis and limitations.

A major problem in studying the interaction between a drilled shaft and the supporting soil consists of defining the load distribution along the length of the shaft, or the measurement of load at various points along the length of the shaft. Consequently, there was a need to develop suitable instrumentation for measuring load in drilled shafts as a function of depth. A total of five different instrumentation systems were tested at The University of Texas. One such system was adopted and used principally in the instrumentation of test shafts for the project and is shown in Fig. 1.1. Barker and Reese (1970) discussed the proposed load-measuring system in a report at the Center for Highway Research.

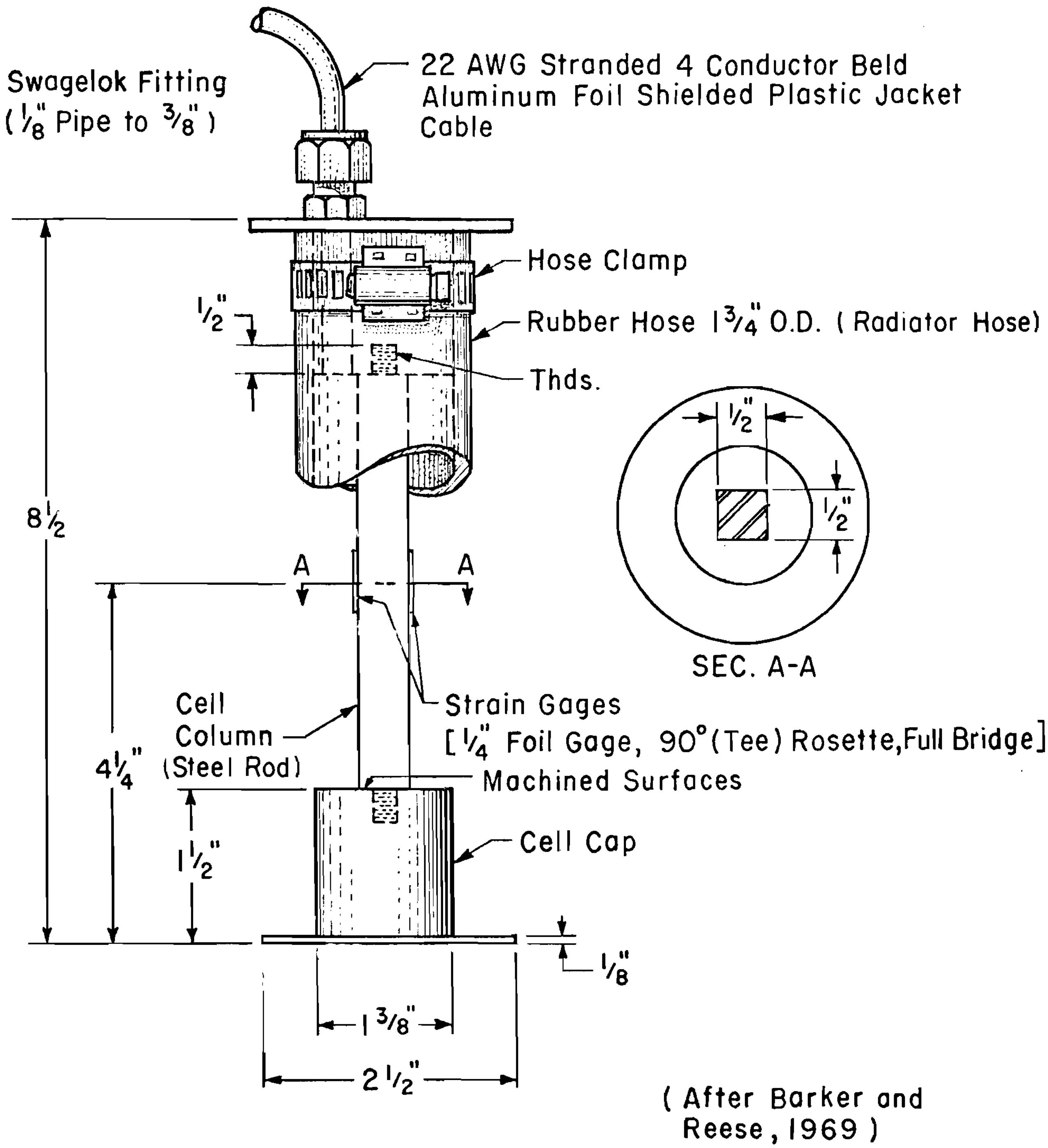


Fig. 1.1 Components of a Mustran cell

From 1968 to 1975, a total of 19 full-scale drilled shafts were tested under axial load by the Center for Highway Research (CFHR). The test shafts were installed at sites having different soil types, such as clay, clay shale, and sand. The majority of the shafts were located in Texas at sites shown in Fig. 1.2. Two drilled shafts were tested in Puerto Rico.

Evaluations of load tests performed primarily in clay were reported by O'Neill and Reese (1970), Barker and Reese (1970), and Engeling and Reese (1974). Vijayvergiya and Reese (1969) and Aurora and Reese (1976) reported the results of testing done in shale. Results of load tests conducted in sand can be found in the reports by Touma and Reese (1972) and by Engeling and Reese (1974). In a majority of these reports, construction procedures are discussed as well as their effects on soil-shaft interaction.

Wooley and Reese (1975) reported a study in which a drilled shaft supporting a bridge was instrumented and the behavior of the shaft under in-service loading was studied. The shaft was installed in a stiff clay in Houston and no significant settlement or change in load distribution was observed over a two-year period.

Another of the objectives of the drilled shaft project was to investigate the behavior of laterally loaded drilled shafts. Welch and Reese (1972) subjected a full-scale, instrumented shaft to repeated lateral loads. From the observations, a procedure was developed for predicting the response of a stiff clay to short-term static loading or to repeated loading.

In a separate investigation, Parker and Reese (1970) performed combined axial and lateral load tests on small-sized piles installed in sand. From the test results the authors generated axial and lateral interaction curves to be used in design.

Arizona Highway Department Studies

During the period from September 1970 to February 1973, the Arizona Highway Department in cooperation with the Federal Highway Administration of the U.S. Department of Transportation sponsored an extensive investigation of the load-carrying capacity of drilled shafts (Beckwith and Bedenkop, 1973). The objective of that study was to determine the support that drilled shafts can derive from very coarse granular deposits and from cemented, fine-grained,

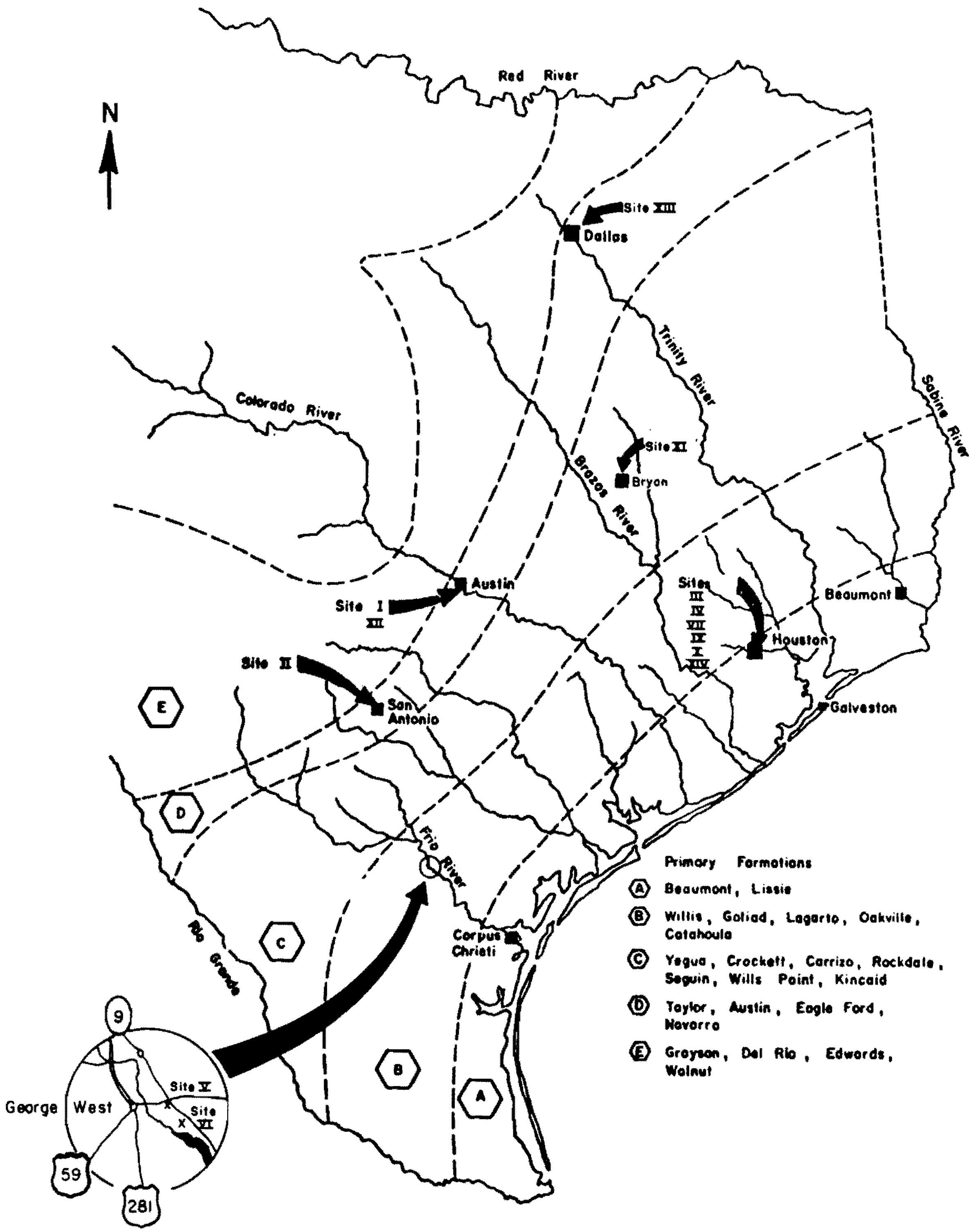


Fig. 1.2 Location of the Different Test Sites in Texas (Adapted After Barker and Reese, 1970)

fan deposits. This type of soils predominates in the heavily populated areas of central and southern Arizona.

A total of 27 drilled shafts were tested during that investigation. Seven load tests were performed in coarse granular soils and 20 tests were conducted in cemented, alluvial fan deposits. The results of that study are summarized in the following chapter.

SCOPE OF THIS REPORT

The purpose of this report is to review existing design procedures for axially loaded drilled shafts and to propose any revisions deemed desirable. In addition, two computer programs were developed to serve as time-saving tools in the design computations of drilled-shaft foundations. These programs, SHAFT1 and BSHAFT, are described in detail in this report.

CHAPTER 2. PREVIOUS PROCEDURES FOR THE DESIGN OF DRILLED SHAFTS

GENERAL

For the convenience of the reader, the relevant procedures that have been suggested for the design of drilled shafts are reviewed in this section. Some discussions are presented of the various aspects of the design methods.

CENTER FOR HIGHWAY RESEARCH PROCEDURE

Drilled Shafts in Clay

Research on drilled shafts conducted by the University of Texas at Austin indicates that the ultimate side resistance and ultimate tip resistance developed by a drilled shaft can be superimposed to obtain the total ultimate capacity.

$$Q_{ult} = (Q_s)_{ult} + (Q_B)_{ult} \tag{2.1}$$

where

Q_{ult} = the total ultimate axial load capacity of the shaft,

$(Q_s)_{ult}$ = the ultimate side resistance, and

$(Q_B)_{ult}$ = the ultimate base capacity.

For clays, the total ultimate capacity, Q_{ult} , corresponds to the "plunging" failure load. This is the load which will cause an increase in settlement with no further increase in load.

The ultimate side resistance, $(Q_s)_{ult}$, can be computed by the following expression:

$$(Q_s)_{ult} = \alpha_{avg}\, s_u\, A_s = \alpha_{avg}(c_Q + \gamma z \tan \phi_Q)\, A_s \tag{2.2}$$

where

α_{avg} = a shear strength reduction factor, which is a correlation between the shear strength of a soil layer and the shear stress mobilized within that layer.

A_s = the effective surface area of the side ot the shaft,

c_Q = the undrained cohesion of the soil,

ϕ_Q = the undrained angle of internal friction of the soil,

γ = the average effective density of the soil above depth z ,

s_u = the undrained shear strength of the soil, and

z = depth from ground surface to center of section considered.

The undrained shear strength, s_u , is obtained from the results of laboratory tests on representative soil samples or from subsurface penetrometer soundings.

The undrained shear strength should be obtained from unconsolidated-undrained triaxial tests at confining pressures equal to the field overburden pressures. However, if only soundings of the Texas State Department of Highways and Public Transportation (SDHPT) dynamic cone penetrometer or of the Standard Penetration Test are available, Touma and Reese (1972) recommended the following correlations:

For the SDHPT Dynamic Penetration Test,

$$s_u = N_{SDHPT}/21 \tag{2.3}$$

where

s_u = the approximate undrained shear strength, tsf,

N_{SDHPT} = the average number of blows/ft the SDHPT cone penetrometer.

For the Standard Penetration Test,

$$s_u = N_{SPT}/15 \tag{2.4}$$

where

N_{SPT} = the average number of blows/ft for the Standard Penetration Test.

The full undrained shear strength is not mobilized in skin friction due to several factors. These include the remolding of the soil during drilling, the opening of fissures or cracks during and after the drilling operation, and the migration of excess water from the concrete to the adjacent soil (O'Neill and Reese, 1970). Other elements that may contribute to the reduction in mobilized shear strength are the mechanical interaction between the soil near the base of the shaft and along the sides, the shrinking of surface soils due to drying, and the use of drilling fluid during construction of the shaft.

In design, one can account for this reduction in shear strength through the use of the factor α_{avg} . This parameter is actually the ratio of the average peak mobilized shear stress to the average shear strength of the soil. The value of α_{avg} will vary, depending on the construction procedure, geometrical shape of the drilled shaft, and the type of soil. Engeling and Reese (1974) reported values of α_{avg} to be used in calculating the ultimate side capacity. These values are listed in Table 2.1.

Results from load tests of drilled shafts have demonstrated that the top 5 feet of soil generally contribute no support against the sides of the shaft. Therefore, it follows that one should regard that portion as contributing no resistance to the ultimate side capacity (O'Neill and Reese, 1970). Furthermore, some test shafts showed little side resistance along the bottom few feet in straight shafts. An underreamed drilled shaft that

TABLE 2.1. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY
(After Engeling and Reese, 1974)

	Parameter	Design Category* A.1	A.2	B.1	B.2	C	D
Side Resistance in Clay	α_{avg}	0.6	0.3[a]	0.3	0.15[c]	0	0
	Limit on Side Shear (tsf)	2.0	0.5[b]	0.5	0.3[d]	0	0
Tip Resistance in Clay	N_c	9	9	9	9	9	9
	p (SPT)	1.6	1.6	1.6	1.6	1.6	1.6
	p (SDHPT penetrometer)	2.8	2.8	2.8	2.8	2.8	2.8

* As described in the text later in this chapter.

[a] May be increased to Category A.1 value for segments of shaft drilled dry

[b] Limiting side shear = 2.0 tsf for segments of shaft drilled dry

[c] May be increased to Category B.1 value for segments of shaft drilled dry

[d] Limiting side shear = 0.5 tsf for segments of shaft drilled dry

was tested, also, demonstrated that as much as 5 feet of the stem immediately above the bell obtained no support from the surrounding soil (Fig 2.1). Yet other tests (Engeling and Reese, 1974) indicated no such phenomenom. For instance, load tests in shales (Vijayvergiya, Hudson, and Reese, 1969; Aurora and Reese, 1976) revealed that a large percentage of the side resistance was acquired over all of the bottom portion of the straight shaft where a shale layer was situated. In such a case, an extremely conservative and uneconomical design would result had one disregarded the bottom 5 feet of soil because it offered no support. Consequently, the amount of lower length of shaft to be ignored in computing the side resistance depends on the founding type of soil and is a matter to be studied as further research is conducted. The recommendations of O'Neill and Reese (1970) and Engeling and Reese (1974) are shown in Fig. 2.1.

To calculate the ultimate base capacity, one can use the following bearing capacity equation:

$$(Q_B)_{ult} = N_c c_Q A_B \tag{2.5}$$

where

c_Q = the average undrained cohesion of the soil for a depth of two base diameters beneath the base. The undrained shear strength, s_u , can be substituted for c_Q, for soils with an undrained angle of internal friction of 10 degrees or less (psf),

N_c = a bearing capacity factor, and

A_B = the area of the base (sq ft).

If unconsolidated-undrained triaxial test results are not available, O'Neill and Reese (1970) recommended the following correlations for dynamic penetration test blow counts:

$$(Q_B)_{ult} = \frac{N}{p} A_B \tag{2.6}$$

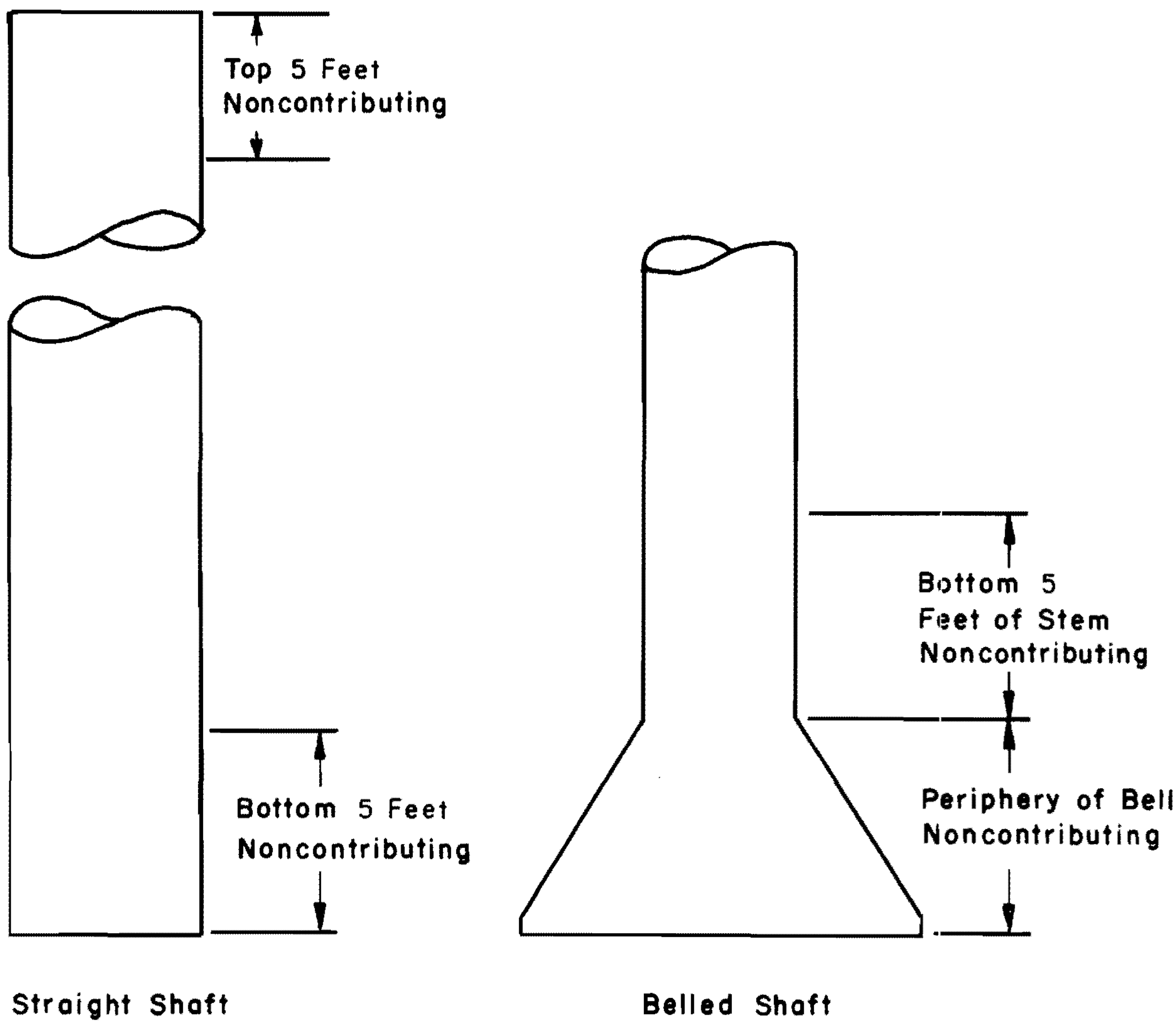


Fig. 2.1 Design Recommendations for Drilled Shafts in Clay
(after Engeling and Reese, 1974)

where

N = the average number of blows/ft from either the SDHPT cone penetration test or the Standard Penetration Test for a distance of two base diameters below the tip,

p = the correlation factor as shown in Table 2.1, and

A_B = the area of the base (sq ft).

This correlation was proposed for clays with soil properties similar to those of Beaumont Clay.

As mentioned earlier in this chapter, the geometry of a drilled shaft as well as its method of construction will invariably affect the load transfer characteristics of the shaft. For this reason, the design criteria have been classified into four major categories according to the shaft geometry and type of founding soil. Two major categories are subdivided into minor groups in order to consider the effects of the construction method of installation (O'Neill and Reese, 1970). The different types of construction methods of shaft installation are discussed in Appendix C.

Category A: Straight-sided shafts in homogeneous or layered soil with no soil of exceptional stiffness relative to soil around the stem, below the base.

Subcategory A.1: Shafts installed dry or by the slurry displacement method.

Subcategory A.2: Shafts installed with drilling mud along some portion of the hole such that entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category B: Underreamed drilled shaft in either homogeneous or layered clay with no soil of exceptional stiffness relative to the soil around the stem, below the base.

Subcategory B.1: Shafts installed dry or by the slurry displacement method.

Subcategory B.2: Shafts installed with drilling mud along some portion of the hole such that entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category C: Straight-sided shafts with base resting on soil significantly stiffer than the soil around the stem. The stiffer soil will not allow the shaft side resistance to be developed.

Category D: Underreamed shaft with base resting on soil significantly stiffer than the soil around the stem. The stiffer soil will not allow the shaft side resistance to be developed.

The design criteria for each design category are found in Table 2.1. Once the ultimate axial capacity of a drilled shaft is determined, one must apply an appropriate factor of safety to the failure load. Reese and O'Neill (1971) suggested that a factor of safety of not less than 2.2 be applied to the total ultimate capacity Q_{ult}, to obtain one working load for highway bridges. Then, to insure against excessive immediate settlement that may occur for large diameter shafts, a factor of safety of 1.0 should be applied to the ultimate side resistance and a factor of safety of 3.0 should be applied to the ultimate tip capacity to obtain a second working load. The smaller of the two working capacities is selected as the design load.

Drilled Shafts in Clay-Shale

Early in 1976, Aurora and Reese published the results of a series of load tests that were performed on drilled shafts installed in clay-shales. The term "clay-shale" used here is in accordance with the classification method suggested by Morgenstern and Eigenbrod (1974). The knowledge obtained from the study permitted the researchers to suggest a rational approach for designing shafts in such soil. However, since all the shafts tested were less than 30 feet long, Aurora and Reese carefully limited the proposed design criteria for drilled shafts of comparable lengths that penetrate approximately 5 feet into a shale stratum.

From the investigation, Aurora and Reese (1976) concluded that shaft construction procedures had a marked effect on the load transfer characteristics of the deep foundation. For drilled shafts installed by the dry method, it was suggested that the shear strength reduction factor, α, could be as high as 0.75 in shale. On the other hand, this value must be reduced to 0.5 for shafts installed by the casing method or by the slurry displacement method.

The shear strength of the clay-shale was investigated by in situ methods and by testing "undisturbed" samples in the laboratory. The undrained shear strength was obtained from laboratory triaxial tests; however, field tests using the static cone gave results reasonable close to those from laboratory testing. Because of the difficulty of sampling clay-shale and because it is strongly anisotropic, the shear strength determinations must be considered to be somewhat uncertain. Therefore, the value of α for the clay-shale must be considered as approximate.

To determine the bearing capacity of drilled shafts in shales, it was suggested that a value of 7.0 be used for the bearing capacity factor, N_c, for shafts built by the slurry displacement method. This value can be increased to 8.0 when shafts are constructed by the casing method or by the dry method.

The research program of shafts in clay-shales also resulted in the establishing of correlations between dynamic-penetration-resistance data and unconsolidated-undrained shear strength of shales, and between dynamic penetration resistance and unit base resistance. For the shear strength correlation, the following equation was suggested:

$$c_Q = \frac{N_{SDHPT}}{75} \tag{2.7}$$

where

c_Q = the unconsolidated-undrained shear strength of the clay-shale, tsf, and

N_{SDHPT} = the average number of blows/ft from the SDHPT cone penetrometer test for a distance of two base diameters below the tip.

For the bearing capacity correlation, the following equation was suggested:

$$q_b = \frac{N_{SDHPT}}{10} \tag{2.8}$$

where

q_b = unit base resistance, tsf.

These correlations are discussed in more detail in the next chapter and are summarized in Table 3.2

Aurora and Reese (1976) proposed that the working load be computed by applying a factor of safety of 2.0 to the ultimate base capacity, $(Q_B)_{ult}$, and a factor of safety of 1.0 to the ultimate side resistance, $(Q_s)_{ult}$. This design recommendation was suggested for shafts with total lengths under 30 feet and penetrating 5 feet into clay-shale. The designer should be aware that such a recommendation results in an overall factor of safety of less than 2.0 with respect to the total ultimate shaft capacity. Consequently, suitable adjustments should be made for variability in scil conditions, and to meet shaft movement requirements.

Drilled Shafts in Sand

From 1970 to 1971, a total of five load tests were performed on drilled shafts installed at sites with soil profiles containing sand. Touma and Reese (1972) reported the results of this research project. The following sections briefly discuss the recommendations which were made for the design of shafts in sand.

To calculate the total ultimate capacity of a drilled shaft in sand, the same equation for drilled shaft in clay is used:

$$Q_{ult} = (Q_s)_{ult} + (Q_B)_{ult} \tag{2.1}$$

However, because the load-settlement characteristics of a drilled shaft in sand differ significantly from those of one in clay, the "plunging" load of a shaft in sand must be defined differently. In the load tests, researchers observed the long, sweeping nature of the load-settlement curves for shafts in sand. Furthermore, it was discovered that large settlements were necessary to mobilize a large fraction of the ultimate axial capacity. The entire side resistance was generally mobilized at displacements of 0.25 to 0.50 inch, whereas a large portion of the tip resistance was not mobilized until the shaft achieved a tip movement of approximately 5.0 percent of the tip diameter. Therefore, Touma and Reese (1972) recommended that the "plunging" load be designated at a shaft displacement of 5.0 percent of the diameter and the "failure" load be defined at a shaft displacement of 1.0 inch. One could then calculate a working load by applying a factor of safety of 2.0 to the "failure" load. This "failure" load is the ultimate axial capacity, Q_{ult}, for drilled shafts installed in sand.

To find the ultimate side resistance supporting a shaft, one can use the following expression:

$$(Q_s)_{ult} = \alpha_{avg} C \int_0^H \bar{p} \tan \bar{\phi} \, dz \tag{2.9}$$

where

C = circumference of the shaft, ft,

H = total depth of embedment of the shaft in sand, ft,

dz = differential element of length, ft,

$\bar{\phi}$ = effective friction angle, degree,

$\bar{p}$ = effective overburden pressure, psf, and

α_{avg} = correlation factor between the shear strength of a soil layer and the shear stress mobilized within that layer.

Results from the load tests indicated that α_{avg} decreased with depth. Thus, Touma and Reese (1972) suggested that a value for α_{avg} of 0.70 be used in sand up to a depth of 25 ft; 0.60 from 25 ft to 40 ft; and 0.50 for depths greater than 40 feet.

It is very difficult to recover sand samples that are representative of field conditions, but the angle of internal friction, $\bar{\phi}$, can be estimated from the results of the SDHPT cone penetration test or the Standard Penetration Test using charts such as those in Fig. 2.2 and Fig. 2.3.

For obtaining the base resistance, $(Q_B)_{ult}$, in sand at a shaft downward displacement of one inch, the following expression is suggested:

$$(Q_B)_{ult} = \frac{\pi D^2}{4k} q_b \tag{2.10}$$

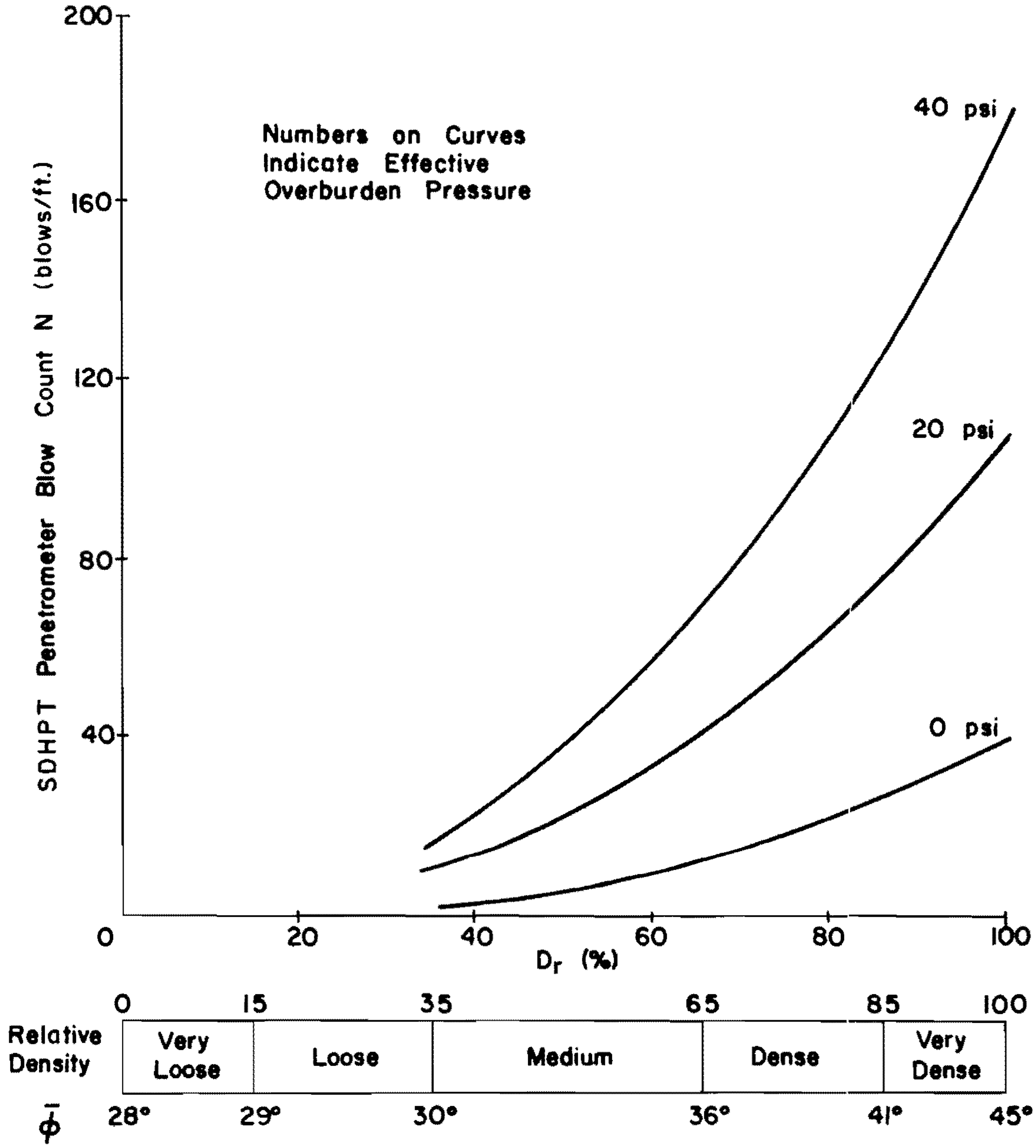


Fig. 2.2 Relation Between SDHPT Penetrometer Blow Count and the Friction Angle (Touma and Reese, 1972)

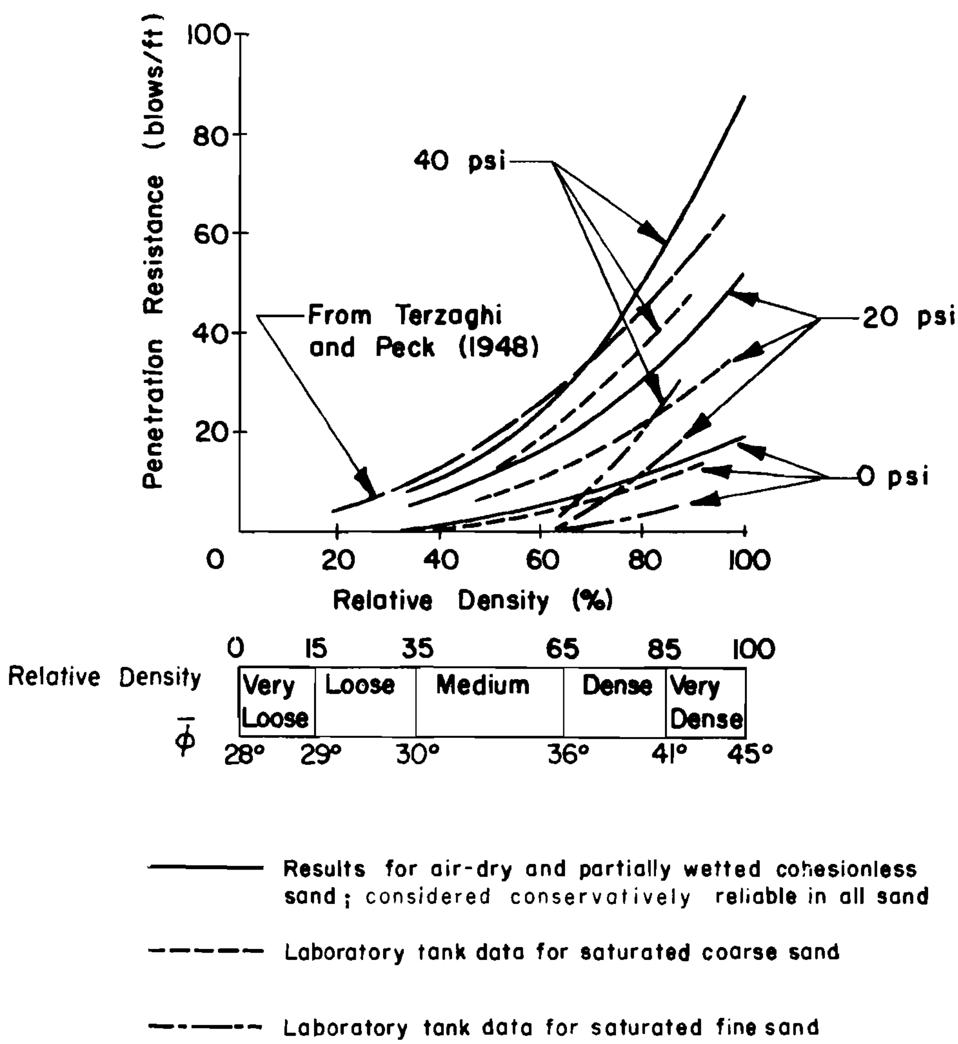


Fig. 2.3 Relationship Between Penetration Resistance and Relative Density for Cohesionless Sand (Gibbs and Holtz, 1957)

where

D	=	base diameter, ft,
k	=	1.0 for D $\leqslant$ 1.67; k = 0.6D for D $\geqslant$ 1.67; and
q_b	=	base capacity at five percent tip movement. For loose sand, q_b is taken as 0 psf; for medium dense sand, as 32,000 psf; for very dense sand, as 80,000 psf. These values are for tips at depths greater than 10 shaft diameters.

The term, k, which is shown in Eq. 2.10, reduces the tip capacity for shafts with a base diameter larger than 20 inches so as to limit the shaft settlement to one inch.

TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

The Texas State Department of Highways and Public Transportation has published a foundation manual which presents several methods of designing drilled shafts. Basically, the different methods of computing the shaft working loads depend on the procedures used in obtaining shear and bearing strength values of the supporting soils. The State Department of Highways and Public Transportation determines the shear and bearing strength of a soil from the results of the transmatic triaxial test (TAT) and the SDHPT cone penetrometer test (SDHPT Manual, 1972). Since the majority of the laboratory testing programs for drilled shaft projects did not involve performance of the TAT test, we can compare CFHR design methods with those of the State Department of Highways and Public Transportation only by making use of the SDHPT cone penetrometer results.

Over the years, the State Department of Highways and Public Transportation has developed empirical relationships between their cone penetrometer values and soil strength. These correlations are presently being used to determine design shear strengths to be used in computing shaft side resistance as well as allowable unit bearing resistance. Figures 2.4 through 2.7 are from the State Department of Highway and Public Transporta-

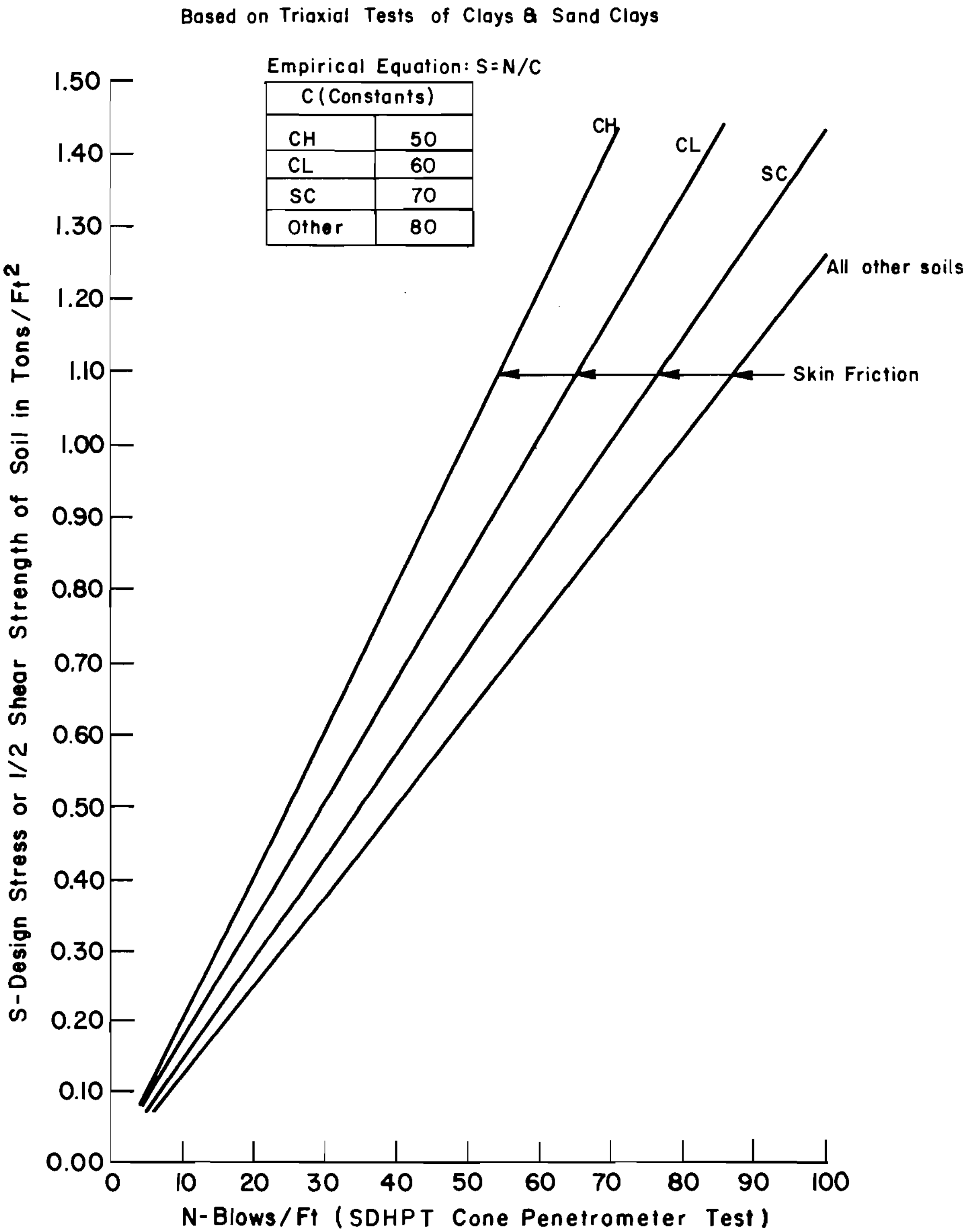


Fig. 2.4 Drilled Shaft Side Resistance Design (Texas SDHPT Foundation Exploration and Design Manual, 1972)

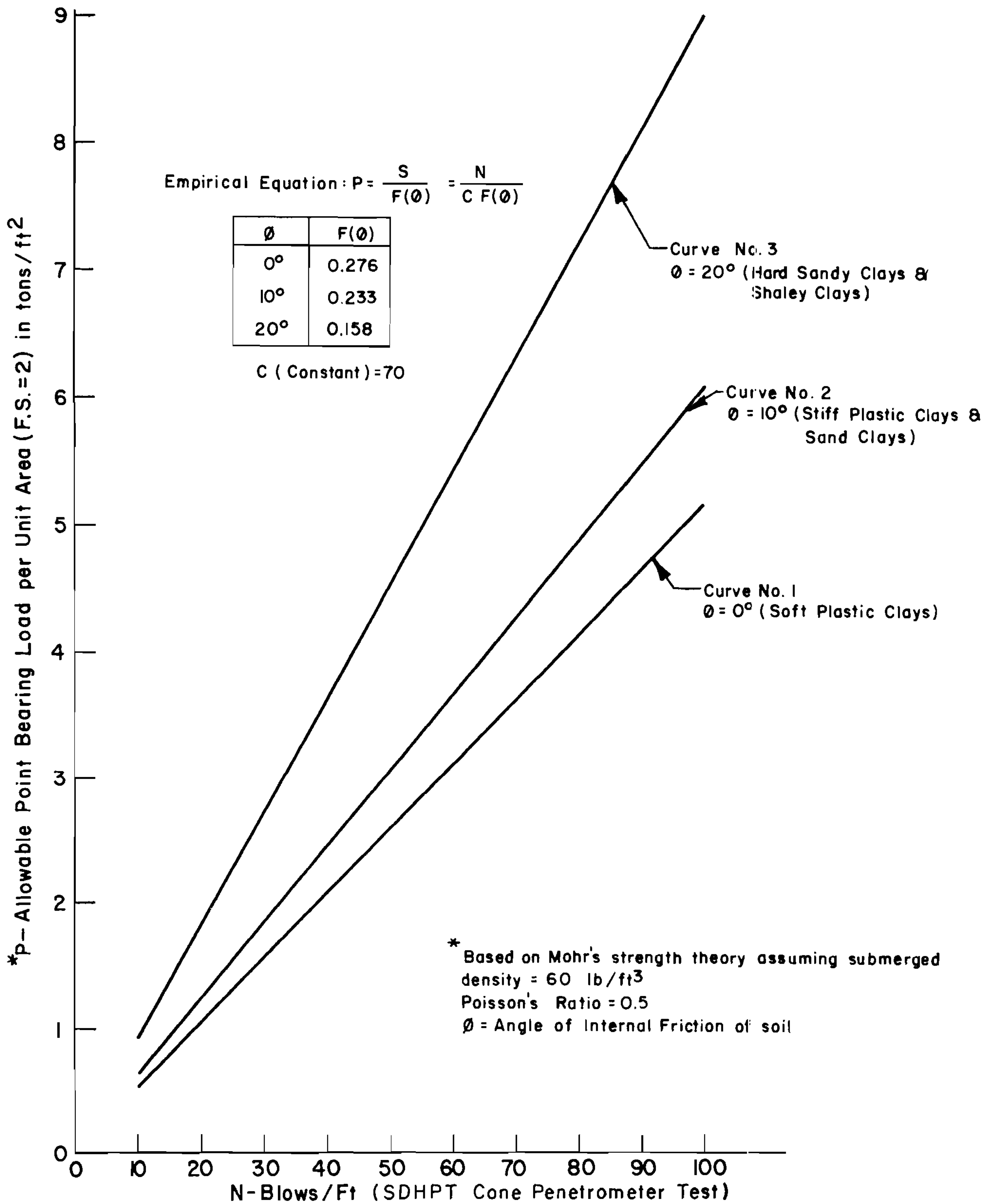


Fig. 2.5 Drilled Shaft Base Resistance Design (Texas SDHPT Foundation Exploration and Design Manual, 1972)

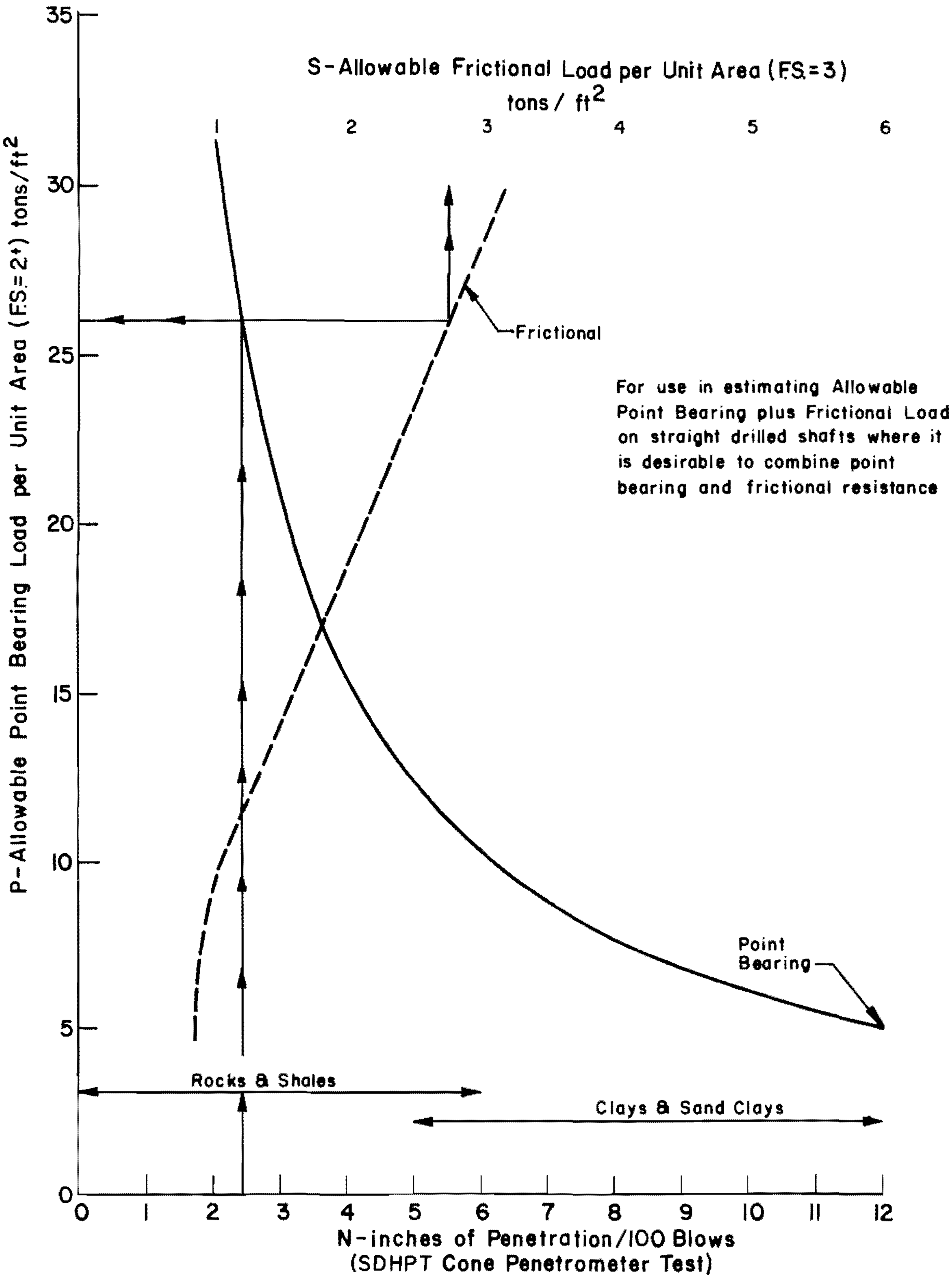


Fig. 2.6 Drilled Shaft Side and Base Resistance Design (Texas SDHPT Foundation Exploration and Design Manual, 1972)

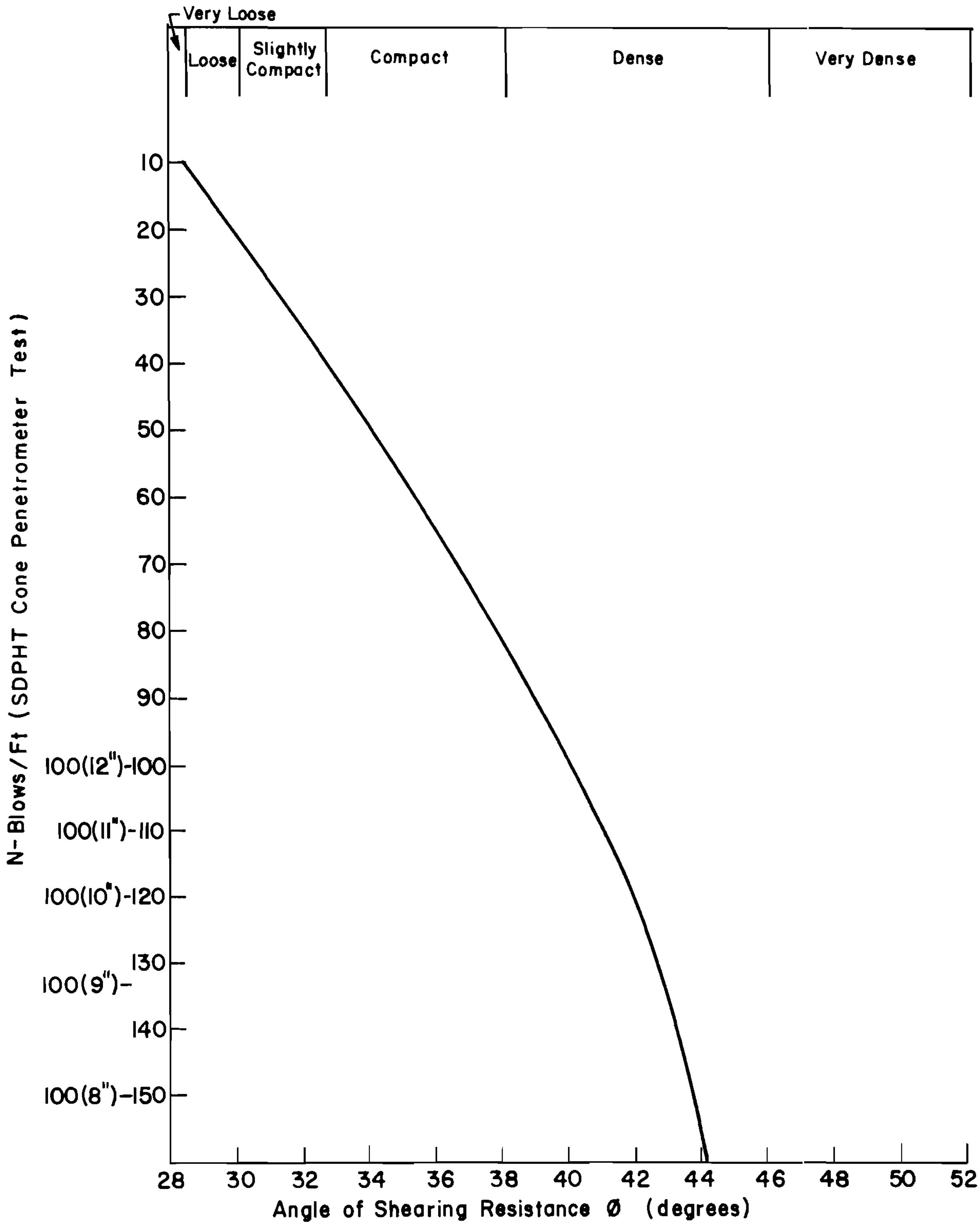


Fig. 2.7 SDHPT Cone Penetrometer Test versus Angle of Internal Friction of Cohesionless Soils (Texas SDHPT Foundation Exploration and Design Manual, 1972)

tion Bridge Division Foundation Exploration and Design Manual (1972). Figures 2.4 and 2.5 are used for soils with N_{SDHPT} values that exceed 100 blows per ft. In order to obtain the angle of internal friction of a cohesionless soil, one can refer to Fig. 2.7 if SDHPT cone penetrometer data are available.

In conjunction with the above figures (and Equations 2.1, 2.2, 2.5, 2.9, 2.10) an engineer is required to employ a strength reduction factor α as follows:

(1) α = 0.6 for shafts that are drilled dry without casing and where the concrete is placed normally.

(2) α = 0.5 for portions of shafts requiring casing with or without drilling mud and where the concrete is placed normally. α = 0.6 can be employed below the casing if the lower portion is drilled dry.

(3) α = 0.5 for shafts where concrete is placed under water or where casing and/or drilling mud is required.

(4) For underreamed drilled shafts, the three preceding criteria also apply. However, side resistance is discounted for a distance of one base diameter above tip.

In Chapter 3, working loads computed by the SDHPT procedure are compared with working loads computed by other procedures.

ARIZONA DESIGN PROCEDURES

The drilled shaft load test project completed for the Arizona Highway Department (Beckwith and Bedenkop, 1973) was a significant contribution to the understanding of the load-carrying characteristics of shafts in soils possessing engineering properties quite different from those which were tested by the Center for Highway Research.

Drilled shafts were installed at three different sites designated as A, B, and C, which are described in the following sections.

The sole purpose for testing shafts at site A was to determine the relationship between width of a loaded area and settlement under a series of bearing pressures. The bearing stratum was located at a depth of approximately 15 feet. This soil, termed SGC, consisted predominately of sandy gravel and cobbles with a small amount of silt and was found to classify as GP in the Unified Soil Classification system. The soil typically contained particles of up to 12 inches and scattered boulders of up to

24 inches. The SGC soil had a high percentage of quartzite, chert, and other hard materials, and it was found to be uncemented. Empirical curves were established for use as design aids.

The load tests that were performed at sites B and C are of primary interest because the majority of the shafts tested at those sites acquired resistance from soil along the sides as well as at the base. Some other shafts were tested for "side shear" only or for "end bearing" only. Moreover, a few shafts were drilled with shear collars belled at various depths to investigate any benefit such a geometry might offer in increasing axial capacity. Only results of the tests B and C will be presented in this report.

The soil profile at site B consisted of weakly cemented, moderately firm, silty clays with a few stratifications of silty sand and clayey sand to a depth of 19 feet. Underlaying these soils were hard, strongly cemented, highly plastic clays, silty clays, and sandy silts which extended to approximately 27 feet. Below this depth there existed moderately to strongly cemented, firm silty clays, sandy silts, and clayey sands. A generalized soil profile is shown in Fig. A.23 of Appendix A.

The soil at site C was similar to that found at site B. The first stratum was a moderately firm, weakly to moderately cemented, silty clay two to three feet thick. Below this stratum, highly stratified clayey sands and sandy clays of medium to high plasticity extended to a depth of 30 feet. Generally, these soils were found to be moderately to strongly cemented with lime and to contain varying amounts of gravel. A generalized soil profile of the site is in Fig. A.25 of Appendix A.

At both sites B and C, there existed great variations in the soil profile in both the vertical and horizontal directions. Furthermore, because of the texture of the soil at these sites, it was difficult to recover undisturbed samples representative of field conditions. Direct shear tests greatly underestimated the actual field strengths. Pressuremeter tests conducted in the field gave a better representation of the actual soil engineering characteristics.

The soils that were tested by the Center for Highway Research for the drilled shaft investigation were entirely different from those found in Arizona. If the CFHR design criteria were applied to the Arizona soils using the measured direct shear strength results, the ultimate axial capacities of the shafts would be greatly underestimated. Therefore, the main procedures suggested by Beckwith and Bedenkop (1973) are presented in the following sections.

Investigated Design Procedures

Due to the fact that undisturbed soil samples could not be recovered, the criteria that were established were basically empirical. In order to determine the best possible method of calculating the total ultimate capacity of a drilled shaft in the Arizona cemented alluvial fan deposits, Beckwith and Bedenkop (1973) considered eight different methods of analysis. The basic approach involves the use of Eq. 2.1,

$$(Q)_{ult} = (Q_s)_{ult} + (Q_B)_{ult} \tag{2.1}$$

Method 1. For the first method of analysis, the direct shear test data are utilized and side shear is computed:

$$(Q_s)_{ult} = A_s\ (\alpha_c c_d + k_o \gamma z \tan \delta) \tag{2.11}$$

where

c_d	=	the cohesion shear strength obtained from direct shear test results
A_s	=	the peripheral surface area,
α_c	=	cohesion reduction factor, assumed equal to 1.0,
δ	=	angle of friction between soil and surface of shaft, assumed equal to ϕ,
k_o	=	earth pressure coefficient, assumed equal to 1.0,
γ	=	average effective density of soil above depth z, and
z	=	depth from ground surface to center of section considered.

End-bearing was determined using the following equation:

$$(Q_B)_{ult} = A_B\ (c_d N_c + D\gamma N_q) \tag{2.12}$$

where

c_d = cohesive shear strength,

D = depth of the shaft,

γ = average effective density of soil above depth D, and

N_c and N_q = dimensionless bearing capacity factors, as proposed by various investigators, which depend on the angle of internal friction, ϕ, the geometry of the drilled shaft, the shape of the failure plane, and the roughness of the base.

For this first method, the average values of strength parameters from the direct shear tests were used in computing the ultimate side resistance. Values of N_c (Terzaghi, 1943) and N_q (Vesic, 1965) were employed in Eq. 2.12 to compute **base resistance.**

Method 2. The second method that was considered involves using the upper range of the shear strength from direct shear tests in estimating the side shear by use of Eqs. 2.11 and 2.12.

Method 3 and Method 4. Both of these methods of analysis use the same equations to define the ultimate load transfer and the ultimate bearing pressure. The ultimate peripheral unit load transfer, q_s, is determined by

$$q_s = s_{avg} \tag{2.13}$$

where

s_{avg} = the average shear strength obtained from the direct shear test (Method 3 and Method 4).

To compute the unit bearing pressure, q_B, an empirical relation is used as shown in Eq. 2.14:

$$q_b = 10s \qquad (2.14)$$

where

$s = s_{avg}$ (Method 3) and

s = upper range of shear strength from direct shear tests (Method 4).

Method 5 and Method 6. These two methods make use of pressuremeter test results to estimate the total ultimate capacity. In Method 5, the ultimate unit side shear, q_s, is defined by s_o, which is the shear strength computed from pressuremeter test; that is,

$$q_s = s_o \qquad (2.15)$$

Therefore, the ultimate side resistance is determined in the following manner:

$$(Q_s)_{ult} = A_s\, q_s \qquad (2.16)$$

End-bearing by Method 5 involves the use of

$$(Q_B)_{ult} = A_B\, q_b \qquad (2.17)$$

where

$q_b = 1.4\, p_L$

p_L = the limit pressure determined from pressuremeter tests.

Method 6 is merely a more conservative version of Method 5. This computation procedure places the following limitations on side shear:

$q_s \leq$ 7.0 ksf for shafts with a rough surface texture (as occurred at site C) and

$q_s \leq$ 5.0 skf for shafts with a relatively smooth surface texture (as occurred at site B).

Maximum end-bearing pressure was limited to the limit pressure, p_L, such that

$$q_b = p_L \text{ (in ksf)} \tag{2.19}$$

<u>Method 7.</u> An effort was made to correlate the Standard Penetration Test blow count, N_{SPT}, with the unit side resistance and the unit base resistance. The study resulted in the following relationships:

$$q_s = N_{SPT}/\ 10 \text{ (in ksf)} \tag{2.20}$$

and

$$q_b = N_{SPT} \text{ (in ksf)} \tag{2.21}$$

where

$q_s \leq$ 7.0 ksf for shafts with a rough surface texture (as occurred at site C),

$q_s \leq$ 5.0 ksf for shafts with a smooth surface texture (as occurred at site B), and

$q_b \leq$ 75.0 ksf.

<u>Method 8.</u> For shaft drilled shafts like some tested at site B where samples could be obtained at shallow depths to run unconfined compression tests, capacity computations were suggested using the following

equations:

$$q_s = q_u/4 \qquad (2.22)$$

and

$$q_b = 4.5\ q_u \qquad (2.23)$$

where

q_u = the unconfined compressive strength

Recommended Design Procedures

Of the methods used in the Arizona study for computing ultimate capacity, four procedures stand out as the most reliable, Methods 3 through 6. Explanations for rejection of the other methods are given in the following paragraphs.

Methods 1 and 2 proved to be highly sensitive to the angle of internal friction, ϕ, which cannot be determined with the required precision from the results of the direct shear test. In these cases, the direct shear test gave values of ϕ, which when used in the analysis, greatly overestimated the ultimate bearing capacity, $(Q_B)_{ult}$.

Method 7, which is based on Standard Penetration Test data, tended to predict capacities very conservatively and, thus, uneconomically.

Method 8 was rejected mainly because of the difficulty in obtaining undisturbed samples from the cemented soil deposits for performing unconfined compression tests. However, should a technique be developed for obtaining samples representative of actual soil conditions, further study of this method would be desirable.

The study by Beckwith and Bedenkop (1973) indicates that the pressuremeter can give reliable estimates of the engineering properties of the soils they tested. Methods 5 and 6 for computing ultimate capacities are based on such pressuremeter testing. On major construction projects where a large number of pressuremeter tests can be performed, pressuremeter test results can be used to calculate the ultimate axial capacity, Q_{ult}. A summary of the equations is given below:

$$(Q_s)_{ult} = q_s A_s \tag{2.16}$$

$$q_s = s_o \tag{2.15}$$

$$(Q_B)_{ult} = q_b A_B \tag{2.17}$$

$$q_b = 1.4 p_L \tag{2.18}$$

$$Q_{ult} = (Q_s)_{ult} + (Q_B)_{ult} \tag{2.1}$$

where

q_s = load transfer along pheriphery of shaft, psf,

s_o = average shear strength computed from the pressuremeter data, psf,

A_s = peripheral area of shaft, sq ft,

q_b = ultimate bearing pressure, psf,

p_L = limit pressure determined from pressuremeter tests, psf, and

A_B = area of the base of the shaft, sq ft.

However, in cases where only a few pressuremeter tests can be performed, the factor of 1.4 is dropped from Eq. 2.18 and load transfer is limited to the following values:

$q_s \leq$ 7.0 ksf for shafts with a rough surfact texture and

$q_s \leq$ 5.0 ksf for shafts with a smooth surfact texture.

When pressuremeter tests are not available, Beckwith and Bedenkop (1973) recommed the use of results from the direct shear tests in the following manner. Combining Methods 3 and 4 into one procedure gives the following relationship for computing side resistance:

$$q_s = s_{avg} \tag{2.19}$$

where

s_{avg} = the average strength from results of the direct shear test,

q_s = the unit load transfer.

For computing the ultimate bearing pressure,

$$q_b = 10s \tag{2.20}$$

where

s = the average direct shear strength for weakly cemented soils, or the upper limit of data from direct shear tests for strongly cemented soils.

Table 2.2 summarizes the recommended design procedures for drilled shafts as installed in cemented alluvial fan deposits by the Arizona investigators.

Because of the erratic variations that occurred in the soil profile across the sites, a factor safety of 3.0 was suggested for design. This criterion limits working stresses to the elastic portion of the load-deformation curves, keeping settlements to a maximum of 0.25 inch.

Other Recommendations

Beckwith and Bedenkop (1973) also investigated the effect of cleaning the base and the effect shear collars might have on load capacity.

TABLE 2.2. DESIGN PARAMETERS FOR DRILLED SHAFTS IN ARIZONA CEMENTED FINE-GRAINED ALLUVIAL FAN DEPOSITS

SOIL TESTING METHOD	q_s (ksf)	q_b (ksf)	REMARKS
Pressuremeter Test: Large number of tests performed	s_o	$1.4p_L$	s_o = average shear strength computed from pressuremeter data p_L = limit pressure determined from pressuremeter data
Pressuremeter Test: Small number of tests performed	s_o*	p_L	same
Direct Shear Test	s_{avg}	10s	s_{avg} = average of the direct shear test results s = average of direct shear test results for weakly cemented soil, or the upper limit of direct shear test data for strongly cemented soil

*$q_s \leq 7.0$ ksf for shafts with a rough surface texture and

$q_s \leq 5.0$ ksf for shafts with a smooth surface texture

Two drilled shafts were used to study the effect of cleaning the base on ultimate capacity. One shaft was completely cleaned while the companion shaft was left with approximately 3 inches of cuttings on the base, which is the amount it is estimated "machine cleaning" generally leaves on the base. Upon completion of the load testing, it was observed that the "machine cleaned" shaft settled much more than the hand cleaned shaft for the same working loads. However, some contractors have claimed that in some soils careful techniques in "machine cleaning" can reduce the amount of cuttings left at the base to as little as one inch, or less. Therefore, cleaning techniques should be considered when design capacities are computed.

The investigation of the use of shear collars proved to be inconclusive. Numerous shafts installed with shear collars were tested and compared with companion straight-stemmed shafts. Some shafts with shear collars failed at greater capacities than their companion shafts while in other cases the straight-stemmed shafts exceeded the capacity of the drilled shafts with shear collars. In some cases both types of shafts sustained the same ultimate capacities. Consequently, the use of shear collars may not be economically justified, especially for the soils involved, because of extra machine time required as well as because of the need to mobilize additional equipment.

CHAPTER 3. RECOMMENDED DESIGN PROCEDURES

INTRODUCTION

A reevaluation was made of the current design procedures for drilled shafts in clay, clay-shale, sand, and fine-grained alluvial fan deposits. This study revealed a need for modifying certain current recommendations for the design of shafts in clay and in sand. These modifications are presented in the following sections.

Throughout the discussions that follow, design procedures are classified under two main categories: primary design procedure and secondary design procedure. The term "primary design procedure" refers to the procedure employed in computing the total ultimate resistance of a shaft when the engineer has reasonably good information on soil properties. Knowledge of these properties is obtained from an extensive field exploration effort combined with a complete laboratory testing program. For example, a sufficient number of samples should be recovered from a substantial number of borings, and laboratory tests should be performed in order to make a good estimate of actual soil conditions in the field. Details of laboratory testing will not be discussed, but, with regard to cohesive soils, unconsolidated-undrained triaxial tests at confining pressures equal to field overburden pressures should be run to determine the undrained shear strength parameters.

Many times a situation arises where an engineer has very little informaton regarding actual soil engineering properties obtained from laboratory tests. The only information available may be results from a dynamic penetration test, and, from such meager data, an engineer must try to determine the soil properties necessary to design drilled shafts. The term "secondary design procedure" refers to such a method of designing drilled shafts. In clays and clay-shales, the principal difference between the primary and secondary procedures is that dynamic penetration tests are employed in the secondary procedure to obtain the shear strengths. The ultimate base resistance in the secondary procedure is computed through use of an ultimate unit bearing pressure obtained from a direct correlation with dynamic penetrometer blowcounts.

Touma and Reese (1972) has suggested the use of Eq. 2.3 or 2.4 for estimating the undrained shear strength of clay from dynamic penetration tests. Later, Hamoudi, et al, (1974) made a more thorough investigation in correlating results from the SDHPT cone penetration tests with results from unconsolidated-undrained triaxial tests. A substantial number of tests were performed on a variety of soil samples which consisted of silty clays (CL), sandy clays (CL), homogenous clays of high plasticity (CH), and clays (CH) with a well-developed secondary structure. The Hamoudi correlations for silty clays (CL), sandy clays (CL), and homogeneous clays (CH) were adopted because in this study it was felt that the correlations gave better values for the undrained shear strength of these cohesive soils than did the relationships suggested by Touma and Reese. However, the correlation for clays (CH) with a well-developed secondary structure was not adopted because it seemed to underestimate greatly the undrained shear strengths of such soils.

The shear strength correlations adopted for clay-shales are those recommended by Aurora and Reese (1976).

For drilled shafts installed in sand, the primary procedures involves the use of the angle of internal friction which is determined by the best available method. The secondary procedure involves the use of correlations between blowcount and load transfer. These correlations were developed from the Texas load test program and are discussed in later sections of this chapter.

The nature of the design procedures for drilled shafts installed in fine-grained alluvial fan deposits, such as those tested in Arizona, did not warrant classification into a primary design procedure and a secondary design procedure as defined in the beginning of this chapter. Consequently, the Arizona design procedures were treated separately.

In the following sections of this chapter, recommended design procedures are presented for drilled shafts in soils thus far tested. In employing the suggested methods, the designer should appropriately modify parameters in cases where the soil profile and soil engineering properties differ significantly from those which were encountered in the research projects. Furthermore, such a situation should also call for a load test at the shaft installation site. In general, load tests are advisable at a site where correlations have not been obtained between load transfer characteristics and soil properties. Internal instrumentation should normally be used for obtaining the distribution of load with depth. The design correlations that follow will prove useful in designing such load tests and in exceptional circumstances may be useful in making final designs.

DESIGN PROCEDURES FOR CLAY

Primary Design Procedure

The routines of the computer program, SHAFT1, are based on the primary design procedures. Documentation of the program is presented in the next chapter. As shown below in Table 3.1, computations with SHAFT1 were used to develop a revised primary design procedure. In making the computations the top 5 feet of clay immediately below the ground surface was ignored because it was considered to contribute no side resistance. Furthermore, if the bottom portion of the shaft was embedded in clay, one diameter above the tip was regarded as offering no peripheral resistance (this differs from the 5 feet recommended by O'Neill and Reese, 1970).

For test shafts embedded in a soil whose profile included both clay and sand, the load transfer in clay layers was obtained from load-distribution curves developed from the field experiments. With the criteria stated above, SHAFT1 was used to compute the ultimate side resistance from undrained shear strengths, assuming the value of the shear strength reduction factor, α, to be equal to one. The top 5 feet of each shaft and one diameter above the tip were ignored in making the computations. The actual measured side resistance, reduced by the measured load transfer in sand for those shafts in layered soils, was divided by the computed side resistance to obtain α_{avg}.

For computing base resistance by the primary design procedure, the

TABLE 3.1. VALUES OF α_{avg} BASED ON LOAD TESTS
(ignoring top 5 ft. and bottom one diameter)

Test Shaft Designation	Diameter (in)	Effective Depth in Clay (ft)	Measured Side Resistance (tons)	Computed Side Resistance (tons)	α_{avg}
US59	30	4	70	47	1.49
HH	24	6	45	93	0.48
G1	36	28	130	261	0.50
G2	30	65	490	773	0.63
BB	30	25	225	258	0.87
S1	30	16	90	141	0.64
S2	30	11	90	90	1.0
S3	30	17	54	141	0.38
S4	30	38	179	358	0.50
Bryan	30	35	240	455	0.53
HBT	33.5	44	588	980	0.60

Note:
1. Weighted average value of α_{avg} = 0.61
2. Two load tests conducted in Puerto Rico were not included because the shafts were not loaded to failure.
3. See Table 3.10 for detailed information on these tests.

parameters used in the current design procedure are recommended.

Suggested design parameters for computing side and bearing resistances in clay are listed in Table 3.2. The design categories shown in Table 3.2 are the same as those discussed in Chapter 2 in relation to Table 2.1. In subcategory A.1, the suggested value for α_{avg} has been rounded from 0.61 to 0.60. In considering the reduction in load transfer due to shaft construction procedures, shaft geometry, and abrupt changes in soil strength with depth, O'Neill and Reese (1970) recommended reduced values of α_{avg}, which are also presented in Table 3.2 under different subcategories. Portions of the shaft to be ignored in computing side resistance are shown in the figure in Table 3.2.

To obtain a design load in clay, two working loads are computed and the lower of the two is selected as the design load. One working load is calculated by applying an overall factor of safety of 2.2 to the total ultimate capacity. To obtain the other working load, a factor of safety of one is used in computing the side resistance and a factor of safety of 3.0 is used in computing the base resistance. In selecting a factor of safety, one should keep in mind that as the base diameter increases in size, the factor of safety of the base must be increased because more settlement will be required to mobilize a given percentage of the base capacity. Reese and O'Neill (1971) have proposed that in clays, for base diameters greater than 9 ft, the factor of safety of 3.0 be increased linearly up to a value of 4.0 for shafts with a base diameter of 15 feet. The use of larger factors of safety for base resistance will insure that the load corresponding to a settlement of one inch will be no greater than 2.2 times the working load.

Secondary Design Procedure

As mentioned earlier in this chapter, the secondary procedure for designing drilled shafts in clay is based on obtaining undrained shear strengths and ultimate unit base resistances from dynamic penetrometer blow-counts. The following expressions were adopted from correlations suggested by Hamoudi, et al. (1974) for the SDHPT cone penetration tests.

TABLE 3.2. REVISED DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY
(Primary Design Procedure)

Parameter		Design Category					
		A.1	A.2	B.1	B.2	C	D
Side Resistance* in clay	α_{avg}	0.6	0.3[a]	0.3	0.15[c]	0	0
	Limit on side shear (tsf)	2.0	0.5[b]	0.5	0.3[d]	0	0
Tip Resistance** in clay	N_c	9	9	9	9	9	9

a May be increased to category A.1 value for segments of shaft drilled dry.

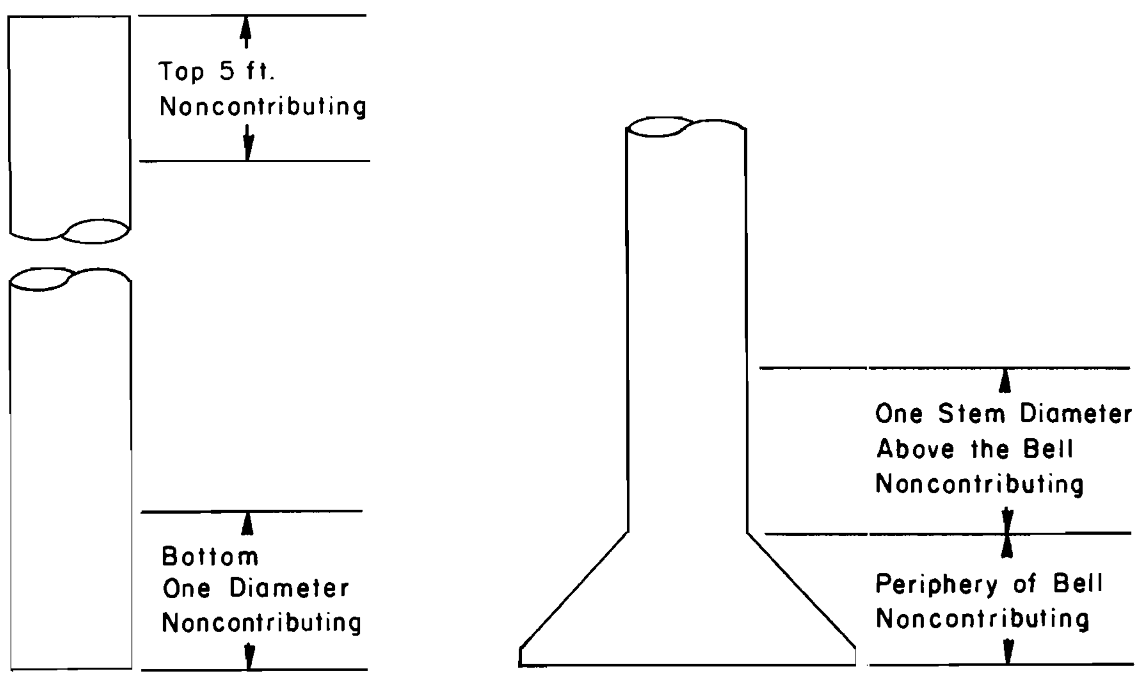
b Limiting side shear = 2.0 tsf for segments of shaft drilled dry.

c May be increased to category B.1 value for segments of shafts drilled dry.

d Limiting side shear = 0.5 tsf for segments of shaft drilled dry.

*Equation for computing side resistance: $(Q_s)_{ult} = \alpha_{avg} s_u A_s$

**Equation for computing base resistance: $(Q_B)_{ult} = N_c c_Q A_B$



For homogeneous clays (CH),

$$s_u = 0.07N_{SDHPT} \tag{3.1}$$

For silty clays (CL),

$$s_u = 0.063N_{SDHPT} \tag{3.2}$$

For sandy clays (CL),

$$s_u = 0.053N_{SDHPT} \tag{3.3}$$

where

s_u = the undrained shear strength, tsf, and

N_{SDHPT} = SDHPT cone penetrometer blowcount, blows/ft.

To establish similar correlations for results from the Standard Penetration Test, the following relationship (Touma and Reese, 1972) can be used:

$$N_{SPT} = 0.70\ N_{SDHPT} \tag{3.4}$$

where

N_{SPT} = the Standard Penetration Test blowcount, blows/ft, and

N_{SDHPT} = the SDHPT cone penetrometer blowcount, blows/ft.

Substituting Eq. 3.4 into Eqs. 3.1 through 3.3 gives the following relationships for SPT results.

For homogeneous calys (CH),

$$s_u = 0.10N_{SPT} \tag{3.5}$$

For silty clay (CL),

$$s_u = 0.09N_{SPT} \tag{3.6}$$

For sandy clay (CL),

$$s_u = 0.076N_{SPT} \tag{3.7}$$

In using the above correlations to obtain load transfer, one must select an appropriate value of α_{avg} from Table 3.2.

Table 3.3 includes the necessary parameters for computing base resistance from the results of dynamic penetrometer tests. The parameters are identical to those found on Table 2.1 except that an upper limit of 35 tsf is shown for the bearing pressure to keep within the range of our load test results for clays. The limiting pressure can be increased if it is justified by future research.

TABLE 3.3. DESIGN PARAMETERS FOR BASE RESISTANCE FOR DRILLED SHAFTS IN CLAY (Secondary Design Procedure)

Parameter	Design Category					
	A.1	A.2	B.1	B.2	C	D
p (SPT)	1.6	1.6	1.6	1.6	1.6	1.6
p (SDHPT)	2.8	2.8	2.8	2.8	2.8	2.8
Limit on bearing pressure (tsf)	35	35	35	35	35	35

Note: Equation for computing base resistance; $(Q_B)_{ult} = \frac{N}{p}A_B$

DESIGN PROCEDURES FOR CLAY-SHALE

The procedures discussed in Chapter 2 are recommended for the design of drilled shafts in clay-shale. The parameters to be used in the primary design procedure are presented in Table 3.4.

TABLE 3.4. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY-SHALE
(Primary Design Procedure)

	Parameter	Design Category A	B	C
Side resistance* in clay-shale	α_{avg}	0.75	0.50	0.50
Tip resistance** in clay-shale	N_c	8	8	7

Category A: Shafts installed by the dry method.

Category B: Shafts installed by the casing method.

Category C: Shafts installed by the slurry displacement method.

*Equation for computing side resistance: $(Q_s)_{ult} = \alpha_{avg}\ s_u\ A_s$

**Equation for computing base resistance: $(Q_B)_{ult} = N_c\ c_Q\ A_B$

The parameters required for the secondary design procedure are listed in Table 3.5. The N_{SPT} correlations with undrained shear strength and with unit base resistance were developed by using Eq. 3.4. Substituting Eq. 3.4 into Eqs. 2.7 and 2.8 gives the following equations:

$$c_Q = \frac{N_{SPT}}{53} \tag{3.8}$$

and

$$q_b = \frac{N_{SPT}}{7} \tag{3.9}$$

TABLE 3.5. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY-SHALE (Secondary Design Procedure)

	Parameter	Design Category A	Design Category B	Design Category C
Side resistance* in clay-shale	α_{avg}	0.75	0.6	0.5
Tip resistance** in clay-shale	p (SPT)	7	7	7
	p (SDHPT)	10	10	10

Category A: Shafts installed by the dry method.

Category B: Shafts installed by the casing method.

Category C: Shafts installed by the slurry displacement method.

*Equation for computing side resistance: $(Q_s)_{ult} = \alpha_{avg}\, s_u\, A_s$

**Equation for computing base resistance: $(Q_B)_{ult} = \frac{N}{p}\, A_B$

Note: $s_u = \frac{N_{SDHPT}}{75}$ or $s_u = \frac{N_{SPT}}{53}$

No definite recommendation can be made with regard to what portion of the shaft to ignore in computing the side resistance. Load tests have not been performed on shafts installed in clay-shale profiles which completely envelop the periphery of the shaft. Therefore, there is no way of telling how the load transfer along the shaft would vary with depth. Results of load tests conducted thus far have indicated that skin resistance is mobilized down to the tip of the shaft. However, this may not be the case for a shaft embedded solely in clay-shale. On the basis of the limited results presently available, it is suggested that, for computing side resistance for clay-shales, the top 5 feet of the shaft be ignored and that load transfer be assumed over the remaining portion of the shaft down to the tip.

DESIGN PROCEDURES IN SAND

Primary Design Procedure

No revisions are recommended in the procedure presented earlier in Chapter 2. The design parameters are summarized in Table 3.6.

TABLE 3.6 DESIGN PARAMETERS FOR DRILLED SHAFTS IN SAND

(Primary Design Procedure)

	Parameter	Value	Remarks
Side Resistance* in sand	α_{avg}	0.70	depth < 25 ft
		0.60	25 ft < depth < 40 ft
		0.50	depth > 40 ft Side resistance should be limited to 2.5 tsf.
Base resistance** in sand	q_b(tsf)	0	loose sand
		16	medium-dense sand
		40	very dense sand

*Equation for computing side resistance: $(Q_s)_{ult} = \alpha C \int_o^H \bar{p} \tan \bar{\phi} \, dz$

**Equation for computing base resistance: $(Q_B)_{ult} = \frac{\pi D^2}{4k} q_b$

Tip movement is limited to one inch. The ultimate bearing pressure, q_b, can be interpolated for intermediate densities.

Secondary Design Procedure

In developing a method for estimating load transfer in sand from dynamic penetration test results, load transfer information was acquired from load distribution curves from the load test results. This information was correlated with data from dynamic penetration tests. The data for the State Department of Highways and Public Transportation cone penetrometer correlations and the Standard Penetration Test correlations are listed in Tables 3.7 and 3.8, respectively. Because of the wide scatter in the data and because of the small number of data points that were available, the linear regression equations on pages 48 and 49 were simplified in order to limit computed load transfer to the lower bound of the measured load transfer data. For the State Department of Highways and Public Transportation load transfer correlation, the following expression is suggested:

$$q_s = 0.014 \cdot N_{SDHPT} \tag{3.10}$$

For the Standard Penetration Test relationship, the following equation can be used to compute load transfer:

$$q_s = 0.026 \cdot N_{SPT} \tag{3.11}$$

where

q_s = the load transferred along the sides of the shaft, tsf,

N_{SDHPT} = the State Department of Highways and Public Transportation cone penetrometer blow count, blows/ft, and

N_{SPT} = the Standard Penetration Test blow count, blows/ft.

Although load transfers greater than 3.50 tsf were achieved in the sand layers, an upper limit of 2.00 tsf is recommended when using Eqs. 3.10 and 3.11.

The load transfer that is calculated by Eq. 3.10 or 3.11 seems reasonable. These equations are similar to the Standard Penetration Test correlation suggested by Schmertamann (1967) for computing load transfer in sand for cylindrical driven piles:

$$q_s = 0.019 \cdot N_{SPT} \; (q_s \leq 1.14 \text{ tsf}) \tag{3.12}$$

TABLE 3.7. LINEAR REGRESSION ANALYSIS FOR SDHPT CONE PENETROMETER AND LOAD TRANSFER IN SAND

SDHPT Cone Penetration Test (blows/ft)	Measured load transfer (tsf)
67	1.02
*390	2.61
*450	3.06
35	0.87
46	0.91
58	1.17
67	1.17
63	1.17
90	1.58
60	1.46
68	1.82
*100	2.18

*Disregarded in linear regression analysis

Linear regression equation: load transfer = $0.37 + 0.014 \cdot N_{SDHPT}$

TABLE 3.8. LINEAR REGRESSION ANALYSIS FOR SPT RESULTS AND LOAD TRANSFER IN SAND

N_{SPT} Standard Penetration Test blowcount (blows/ft)	Measured load transfer (tsf)
30	0.94
49	2.02
*95	3.17
*90	3.31
83	2.09
30	0.82
]2	1.15
17	1.37
28	1.15
40	1.87
53	1.83
*130	2.00
*160	2.22
8	0.41
8	0.47
15	1.08
14	1.15
18	0.98

*Disregarded in linear regression analysis

Linear regression equations: load transfer = $0.20 + 0.021 \cdot N_{SPT}$

Schmertmann recognized that the above expression generally gave quite conservative estimates. When used wisely, it is felt that Eqs. 3.10 and 3.11 give reasonable estimates of load transfer in sand.

In calculating tip capacity, no deviation is made from the primary procedure. The density of the sand is estimated from a dynamic penetration test and the bearing pressure is limited so as to produce no more than one inch of settlement. The design parameters for drilled shafts in sand to be used in the secondary procedure are listed in Table 3.9.

TABLE 3.9. DESIGN PARAMETERS FOR DRILLED SHAFTS IN SAND (Secondary Design Procedure)

	Parameter	Value	Remarks
Side resistance* in sand	q_s(tsf)	$0.014N_{SDHPT}$	for SDHPT cone penetration test results
		$0.026N_{SPT}$	for SPT results
			side resistance should be limited to 2.0 tsf
Base resistance** in sand	q_b(tsf)	0	loose sand
		16	medium-dense sand
		40	very dense sand

*Equation for computing side resistance: $(Q_s)_{ult} = q_s A_s$

**Equation for computing base resistance: $(Q_B)_{ult} = \frac{\pi D^2}{4k} q_b$

Tip movement is limited to one inch. The ultimate bearing pressure, q_b, can be interpolated for intermediate densities.

When designing drilled shafts tipping in sand by the primary design procedure, an overall factor of safety of 2.0 appears reasonable. However, when shafts are being designed by the secondary procedure, an overall factor of safety of at least 2.5 should be employed. This stems from the fact that dynamic penetration tests have a limited reliability as far as using the test results as the sole means of determining soil properties.

DESIGN PROCEDURES FOR CEMENTED ALLUVIAL FAN DEPOSITS

The design methods recommended by Beckwith and Bedenkop (1973) appear to be the most practical for predicting drilled shaft capacities in cemented alluvial fan deposits. The design parameters are summarized in Table 2.2 and are based on the four best computational methods of a total of eight that were analyzed. A factor of safety of 3.0 applied to the total ultimate capacity gives a reasonable working load.

COMPARISON OF MEASURED CAPACITIES AND PREDICTED CAPACITIES

Texas and California Load Tests

The revised design procedures recommended in the previous sections were used to calculate the ultimate side resistances and total ultimate capacities of 16 instrumented drilled shafts tested by the Center for Highway Research and one uninstrumented drilled shaft tested by the California Department of Transportation.

Table 3.10 lists the load tests that were treated in this study. The soil properties at the sites where the test shafts were installed were found in Appendix A (Figs. A.1-A.22, 27, 28). Included in that appendix are shear strength profiles as well as results of dynamic penetration tests.

Two computer programs were used to compute the ultimate side and the total resistances: SHAFT1, which is based on the primary design procedure, and BSHAFT, which is based on the secondary design procedure. Figures 3.1 through 3.4 present the results of these computations. The graphs reveal that both design procedures in most instances predict the shaft capacities within 20 percent of the actual ultimate loads. In only one instance does a prediction method yield an error on the unsafe side greater than 20 percent. Predictions within 20 percent of the measured results are relatively good when one considers that shafts of identical geometry and size within a few feet of each other may differ in total ulti-

TABLE 3.10 DATA FROM LOAD TESTS OF DRILLED SHAFTS IN TEXAS AND CALIFORNIA
(Adapted from Reese, Touma, and O'Neill, 1975)

Test Code	Location	Dimensions: Shaft Diameter (ft)	Dimensions: Underream Diameter (ft)	Penetration (ft)	Construction Method	Date of Test
SA	San Antonio	2.5	None	26.8	Dry	May 68
HB&T	Houston	2.79	None	60	Slurry&casing to 54 ft; bottom 6 ft dry	Jun 69
S1	Houston	2.5	None	23.1	Dry	Aug 68
S2	Houston	2.5	7.5	23	Dry	Mar 69
S3	Houston	2.5	None	23	Dry	Oct 69
S4	Houston	2.5	None	45	Slurry&casing to 40 ft; bottom 5 ft dry	Dec 69
US59	George West	2.5	None	25.4	Dry	Sep 70
HH	George West	2.0	None	19.8	Dry	Oct 70
G1	Houston	3.0	None	54.8	Slurry	Oct 71
G2	Houston	2.5	None	73.5	Slurry	Oct 71
BB	Houston	2.5	None	45	Slurry	Oct 71
BRY	Bryan	2.5	None	42	Dry	May 73
MT1	Austin	2.5	None	23.75	Casing	Jan 75
MT2	Austin	2.5	None	23.5	Slurry	Jan 75
MT3	Austin	2.5	None	24	Dry	Nov 74
DT1	Dallas	3.0	None	20	Casing	Feb 75
CAL1	San Jose, California	4.0	None	83	Slurry	Mar 75

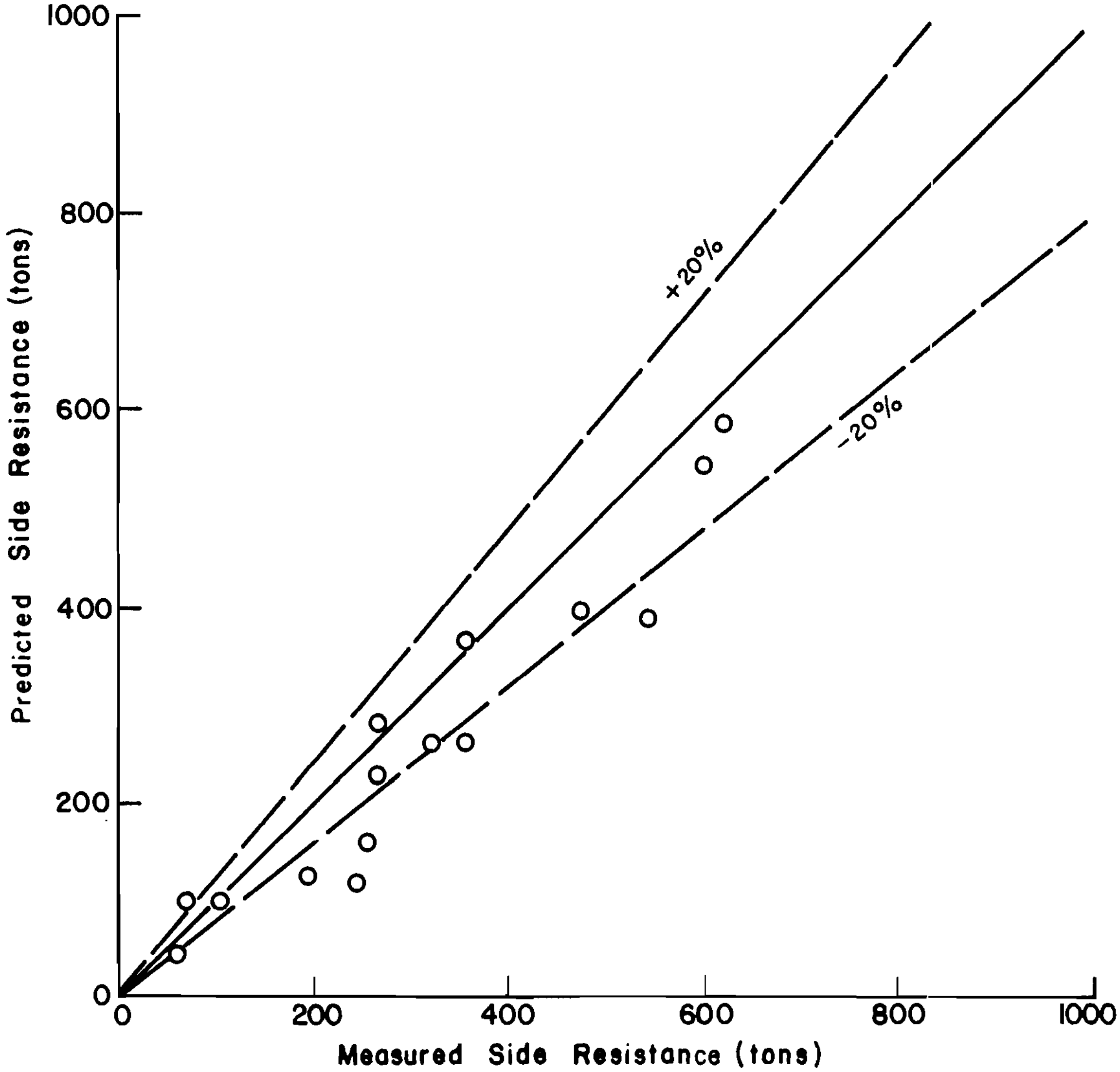


Fig. 3.1 Comparison of Predicted and Measured Side Resistances for Texas Load Tests, Revised Primary Design Procedure

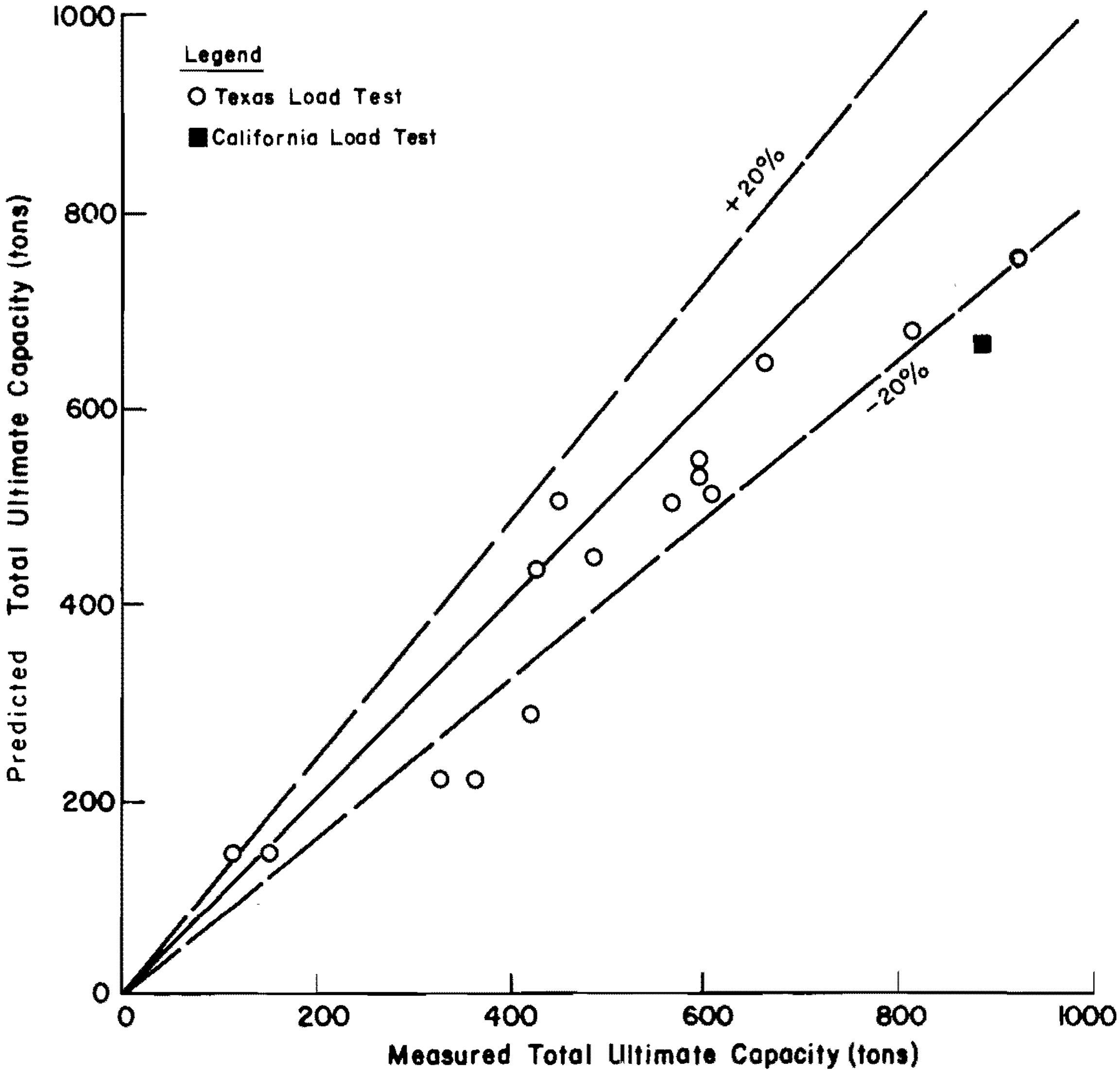


Fig. 3.2 Comparison of Predicted and Measured Total Ultimate Capacities for Texas and California Load Tests, Revised Primary Design Procedure

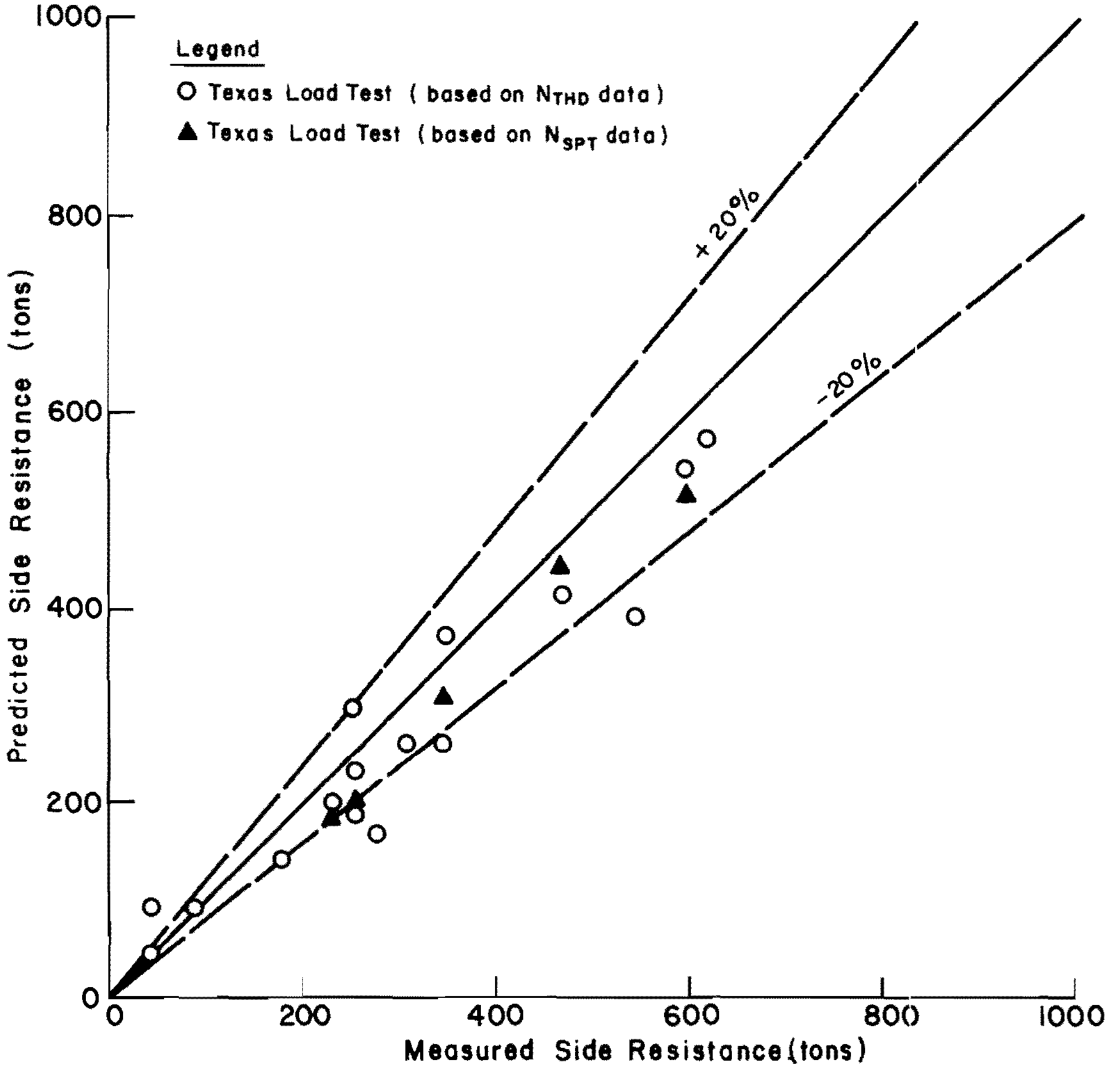


Fig. 3.3 Comparison of Predicted and Measured Side Resistances for Texas Load Tests, Revised Secondary Design Procedure

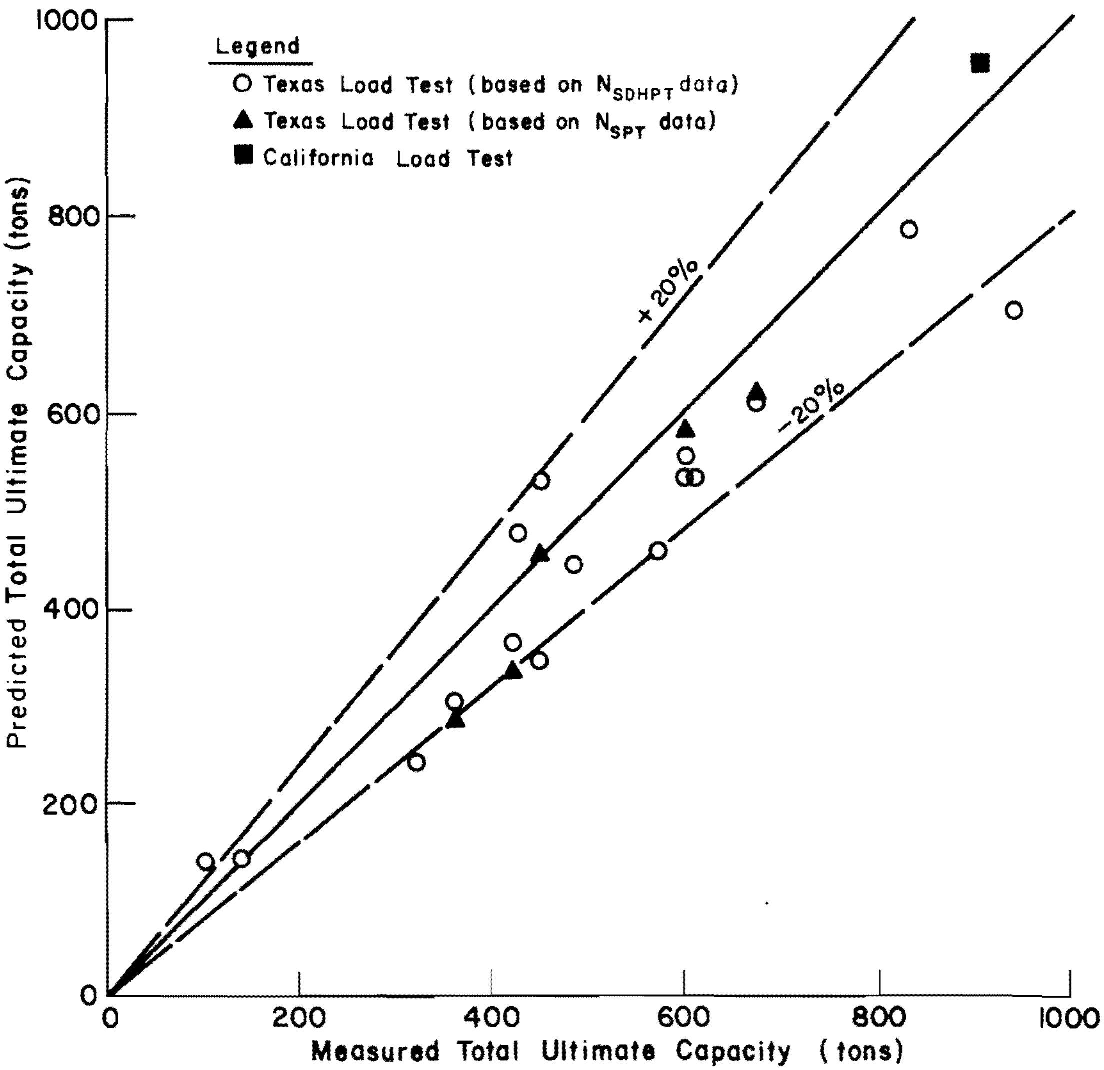


Fig. 3.4 Comparison of Predicted and Measured Total Ultimate Capacities for Texas and California Load Tests, Revised Secondary Design Procedure

mate capacity by as much as 20 percent.

The California test shaft was not instrumented. This precluded any opportunity for measuring the side resistance of the shaft. Consequently, only the total ultimate capacities could be compared.

As a matter of interest, shaft capacities were computed using the previously recommended secondary design procedures, and these results were plotted in Figs. 3.5 and 3.6. These figures demonstrate that the previous secondary procedures do not give as good an estimate of the actual resistances as the secondary procedures recommended in earlier portions of this chapter.

The State Department of Highways and Public Transportation (SDHPT) procedure for designing drilled shafts is discussed in Chapter 2 of this report. To compare our recommended primary and secondary design methods with the SDHPT method, total working loads were computed by the three methods and the results are listed in Table 3.11. The results show that the SDHPT procedure generally gives more conservative working loads than those computed by either of the other two methods.

Arizona Load Tests

The measured ultimate side resistances and total ultimate capacities for the Arizona load tests are compared in Figs. 3.7 through 3.14, with the predicted values using computational Methods 3 through 6, discussed in Chapter 2. From the figures, it is clear that the computations based on the pressuremeter test results (Methods 5 and 6) gave much better estimates of the actual capacities than computations based on Methods 3 and 4. Unfortunately, the pressuremeter test data are not always available and one must resort to Methods 3 and 4 for design purposes. Information on the drilled shafts for which capacities were computed is listed on Table 3.12.

Discussion of Comparisons

In the preceding sections, figures were plotted comparing measured resistances to predicted resistances. Comparisons of the side resistances were included in the previous section mainly so that the reader could

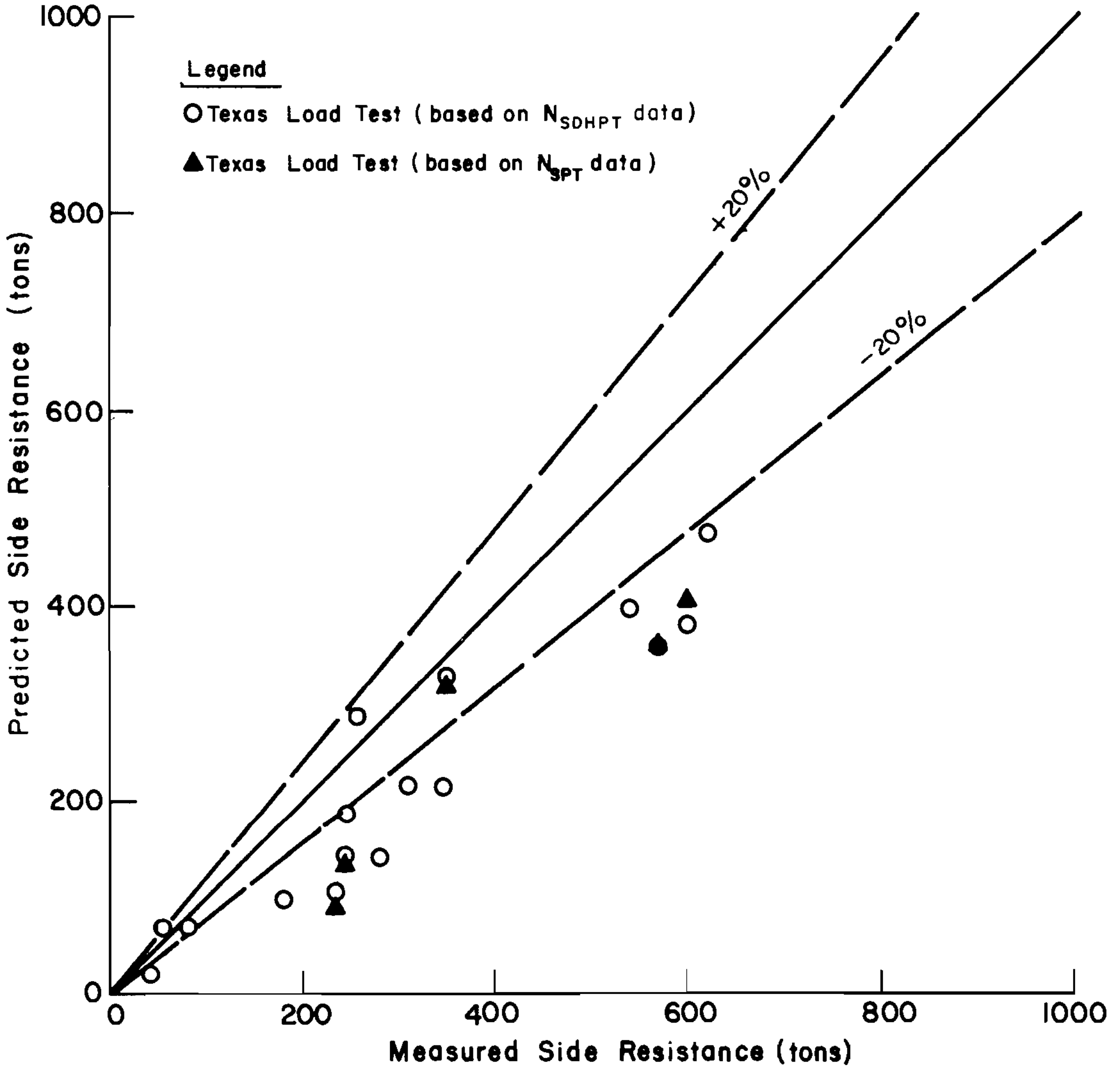


Fig. 3.5 Comparison of Predicted and Measured Side Resistances for Texas Load Tests, Previous Secondary Design Procedure

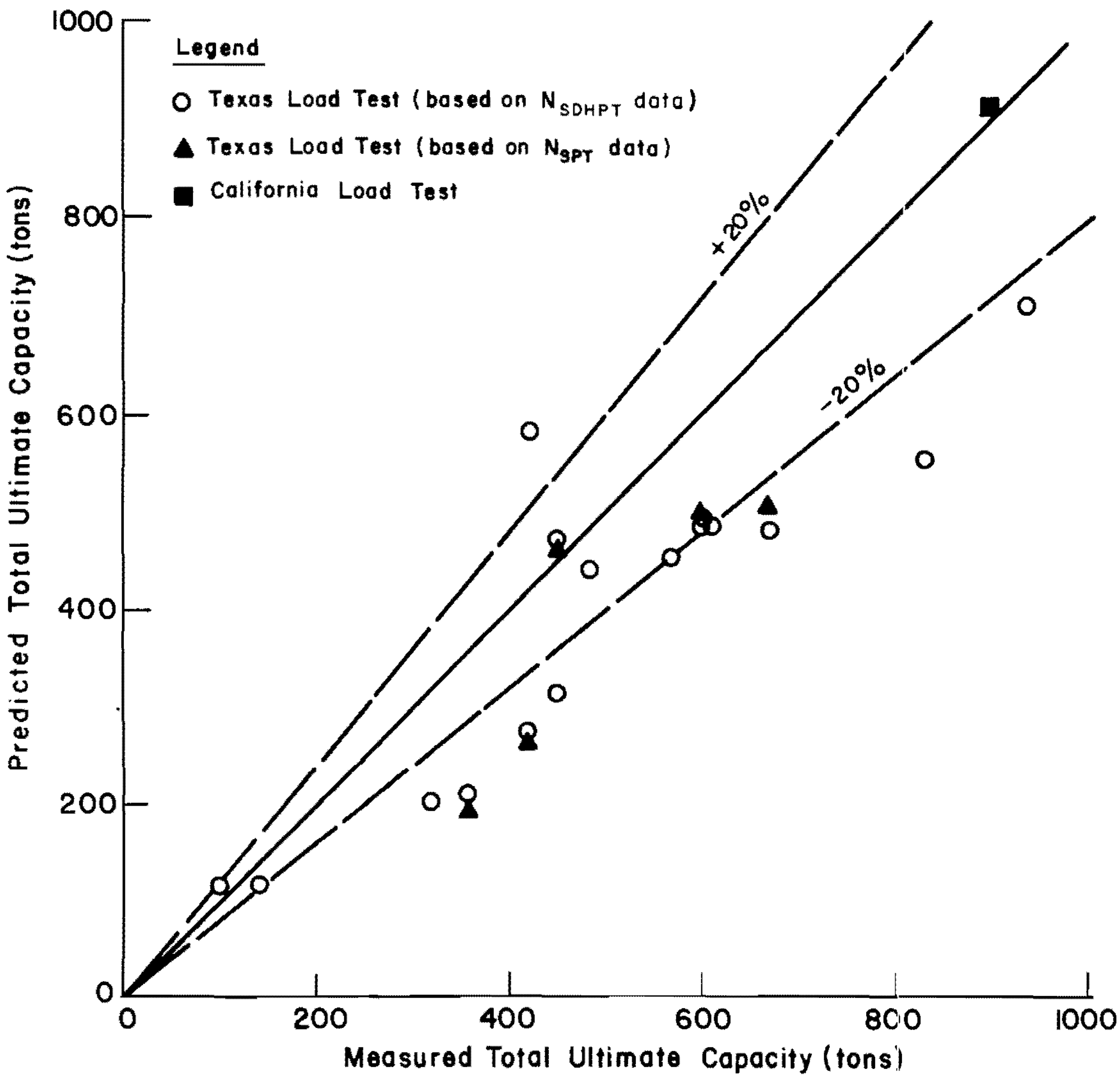


Fig. 3.6 Comparison of Predicted and Measured Total Ultimate Capacities for Texas and California Load Tests, Previous Secondary Design Procedure

TABLE 3.11 TOTAL DESIGN CAPACITIES FOR TEXAS SITES USING THREE DESIGN PROCEDURES

Test	SDHPT Procedure (tons)	Primary Design Procedure (tons)	Secondary Design Procedure (tons)
SA	334	366	367
HBT	242	313	304
S1	46	72	32
S2	85	160	185
S3	46	72	32
S4	85	145	99
US59	218	137	166
HH	85	106	139
G1	259	212	263
G2	216	294	357
BB	252	258	322
Bry	167	219	181
Mt1	224	398	311
MT2	224	379	311
MT3	191	364	322
DT1	157	-	259

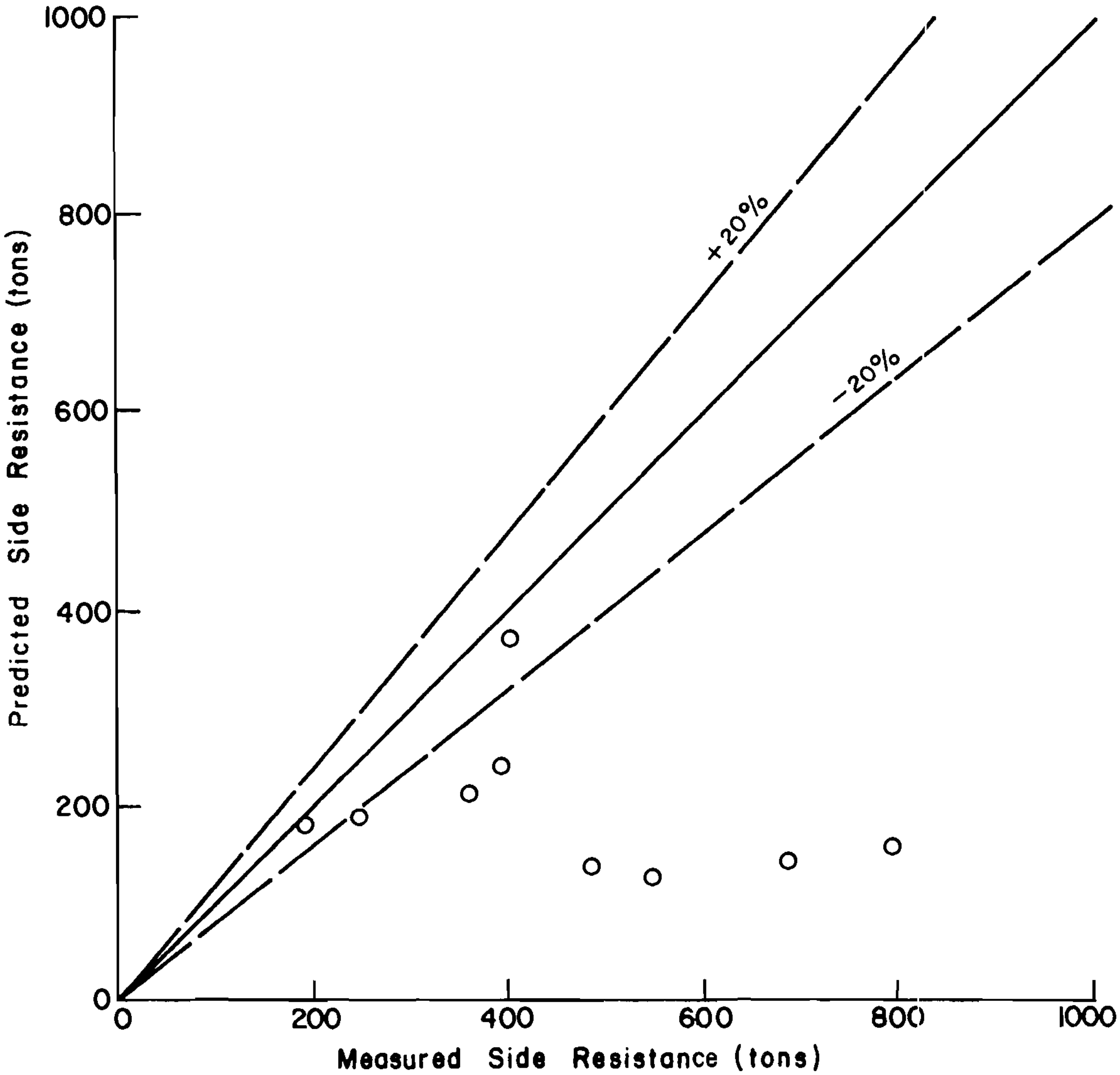


Fig. 3.7 Comparison of Predicted and Measured Side Resistances for Arizona Load Tests, Method 3

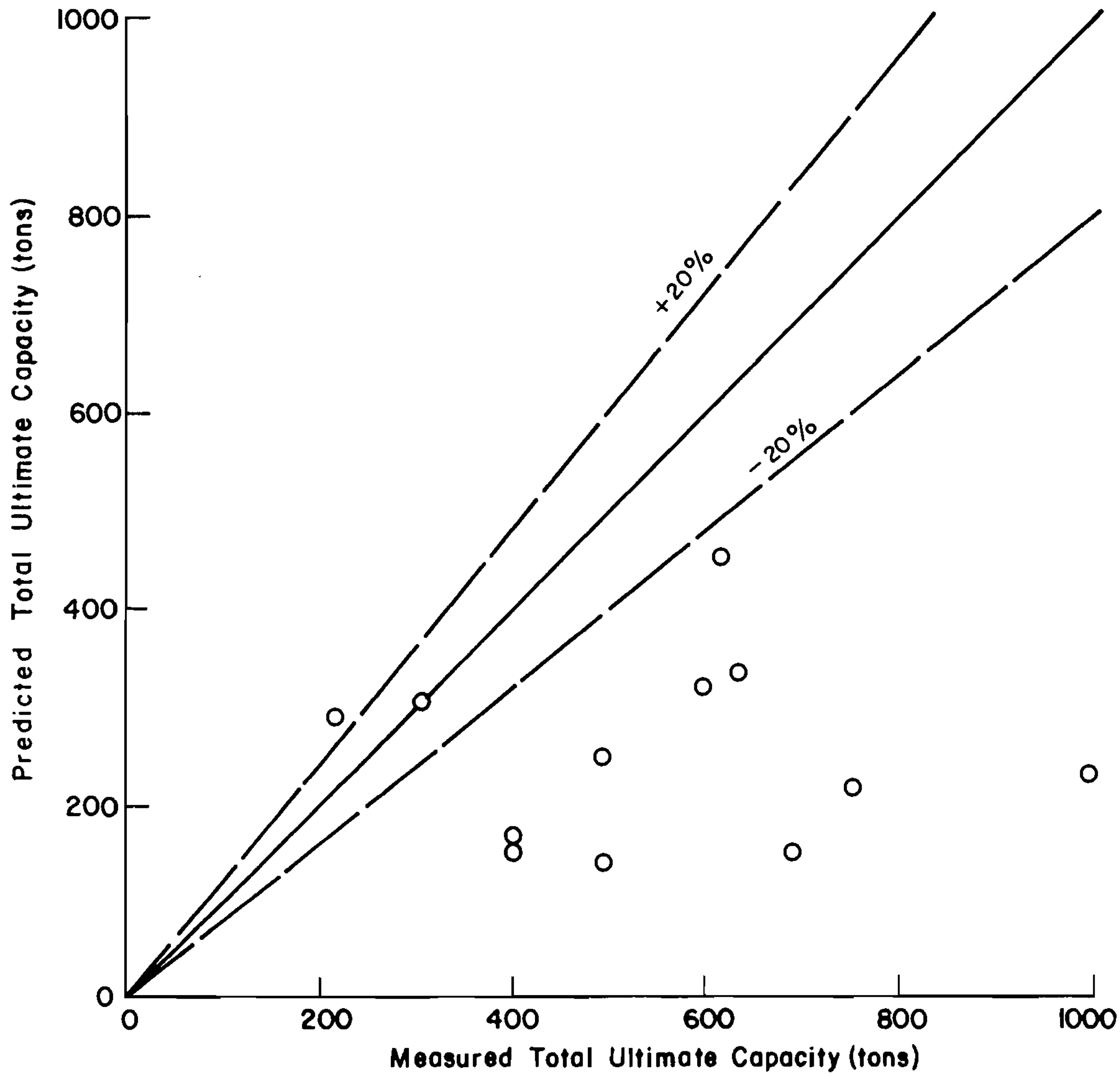


Fig. 3.8 Comparison of Predicted and Measured Total Ultimate Capacities for Arizona Load Tests, Method 3

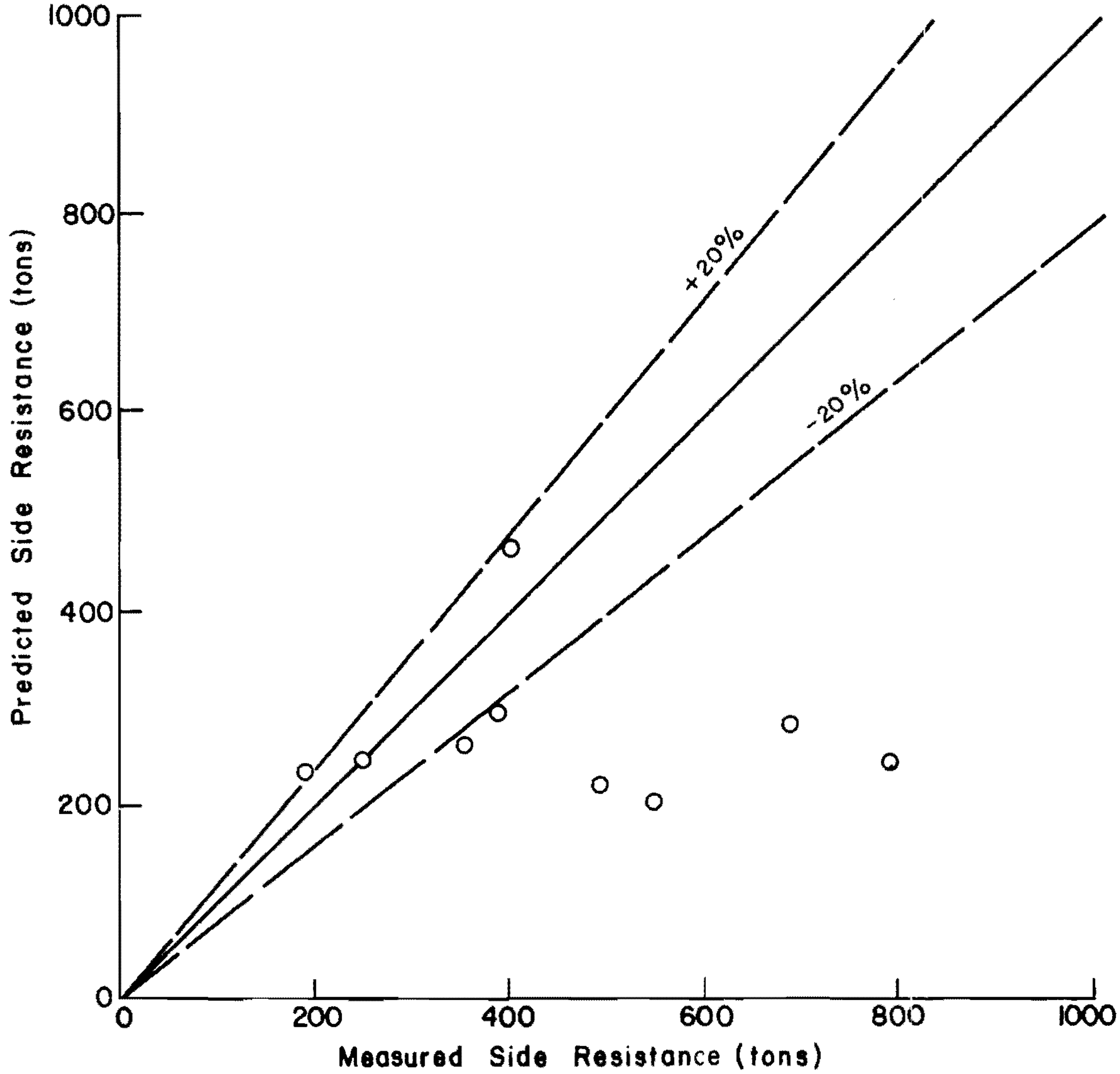


Fig. 3.9 Comparison of Predicted and Measured Side Resistances for Arizona Load Tests, Method 4

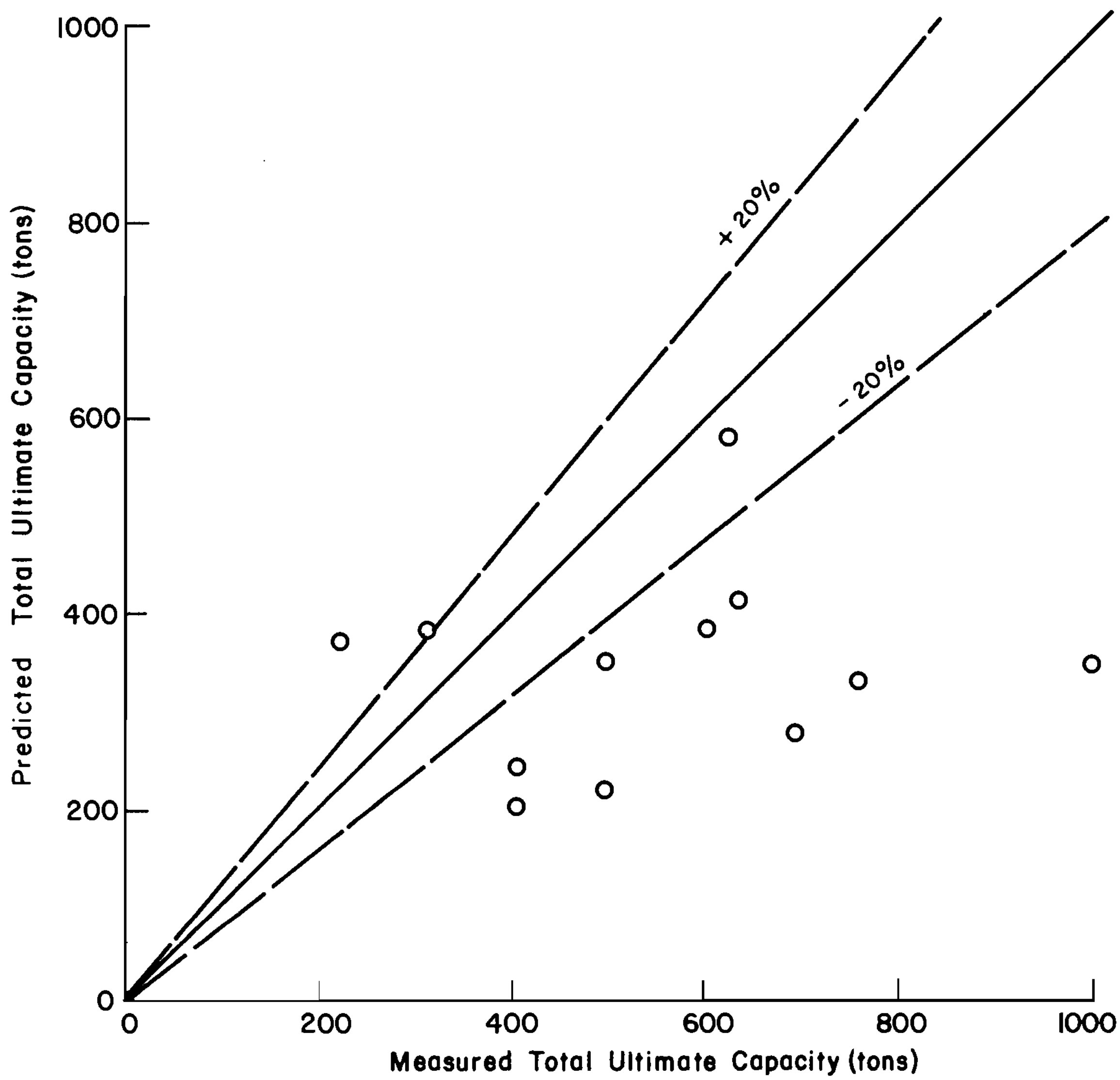


Fig. 3.10 Comparison of Predicted and Measured Total Ultimate Capacities for Arizona Tests, Method 4

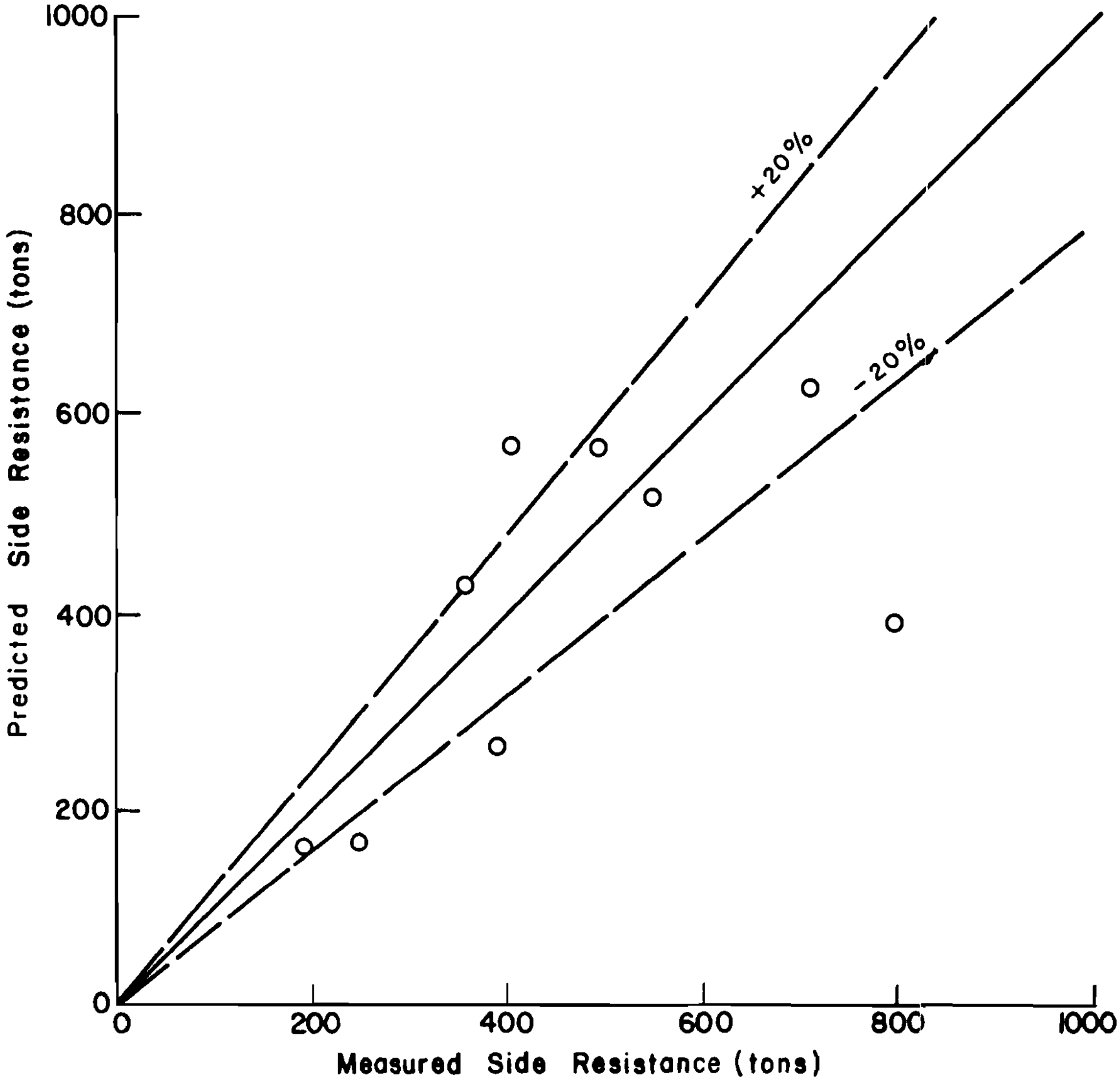


Fig. 3.11 Comparison of Predicted and Measured Side Resistances for Arizona Load Tests, Method 5

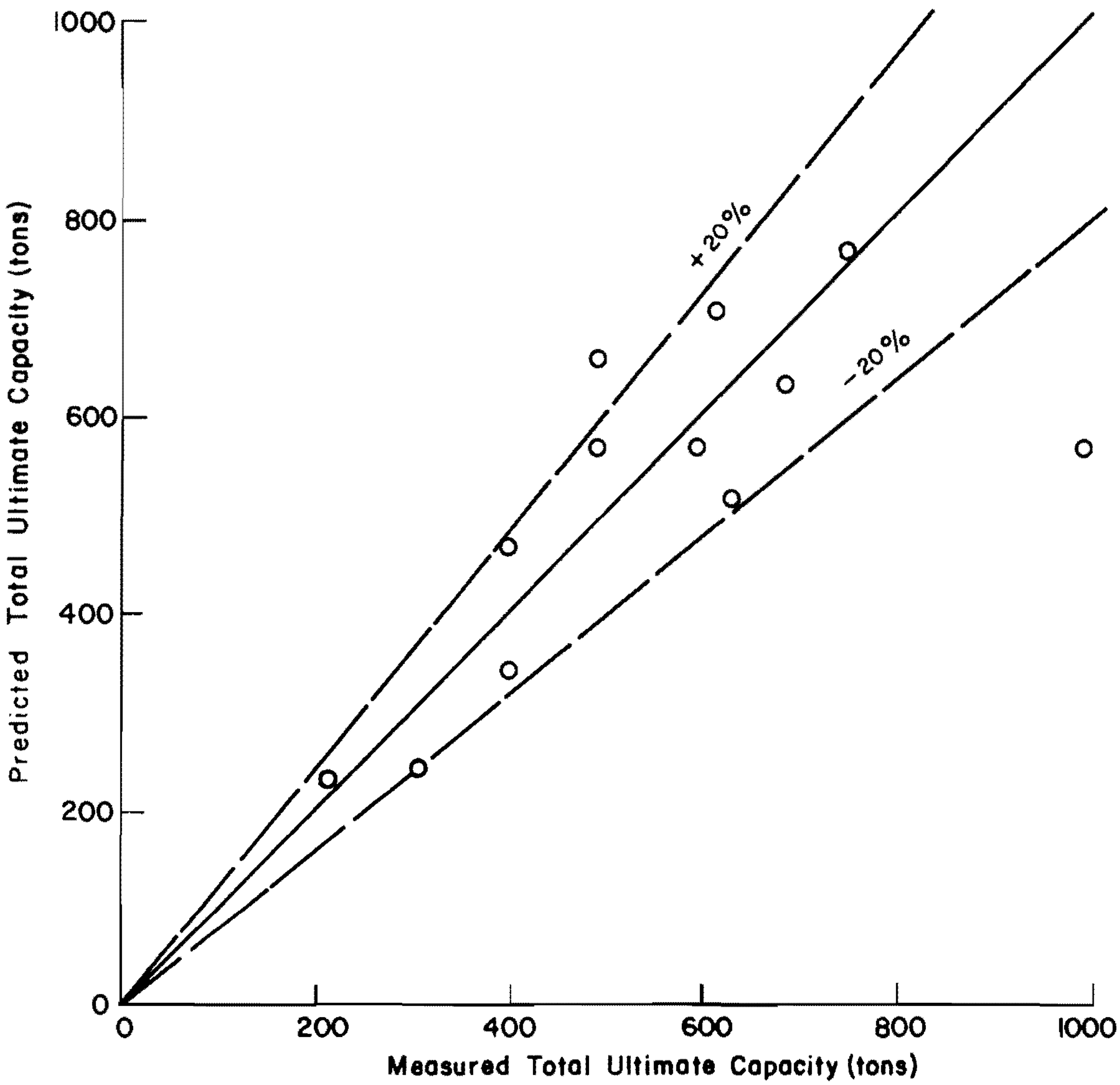


Fig. 3.12 Comparison of Predicted and Measured Total Ultimate Capacities for Arizona Load Tests, Method 5

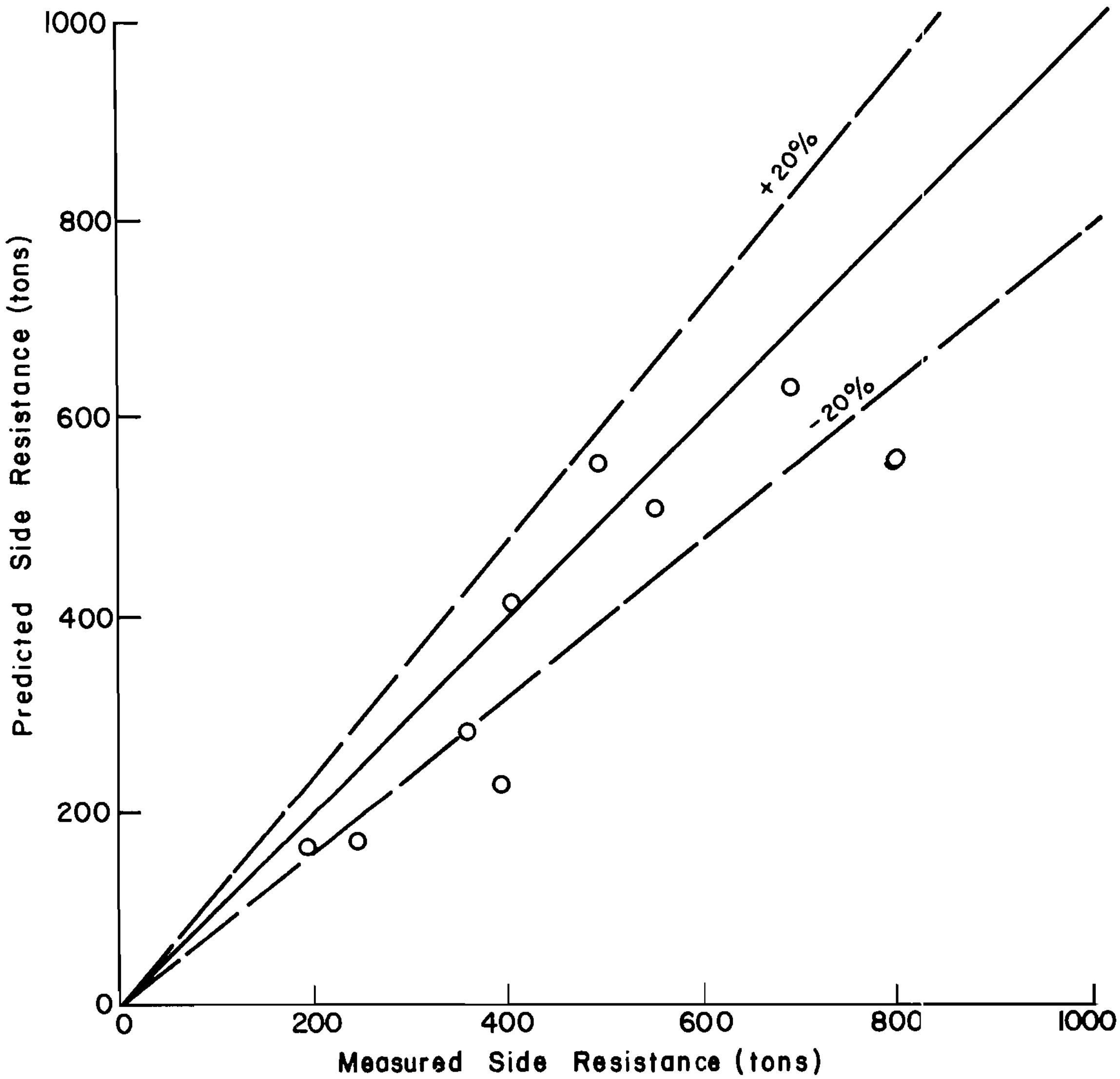


Fig. 3.13 Comparison of Predicted and Measured Side Resistances for Arizona Load Tests, Method 6

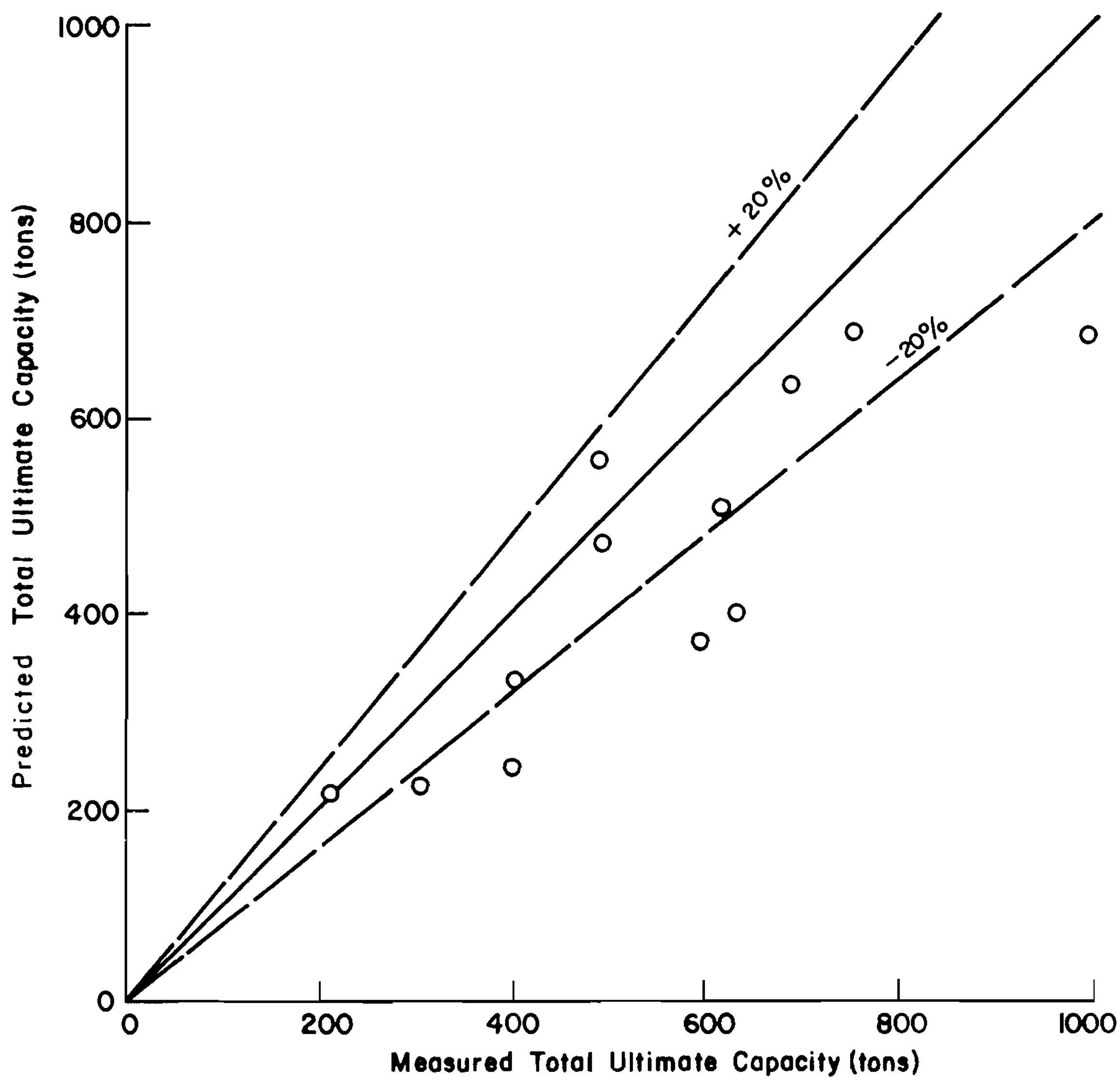


Fig. 3.14 Comparison of Predicted and Measured Total Ultimate Capacities for Arizona Load Tests, Method 6

TABLE 3.12

DATA FROM LOAD TESTS OF DRILLED SHAFTS IN ARIZONA

Test Code	Dimensions		Penetration (ft)	Type of Soil Resistance
	Shaft Diameter (ft)	Underream Diameter (ft)		
TPB-1	2.74	None	21.5	Side and base resistance
TPB-3	2.44	None	35.8	Side and base resistance
TPB-5	2.69	None	31.7	Side and base resistance
TPB-6	3.0	None	16.2	Side and base resistance
TPB-8	3.0	None	15.6	Side and base resistance
TPC-1	2.5	None	20,3	Side and base resistance
TPC-3	3.1	None	15.4	Side and base resistance
TPC-4	2.5	4.15	16.2	Base resistance only
TPC-5	2.5	3.54	16.1	Base resistance only
TPC-6	2.5	4.92	17	Base resistance only
TPC-7	2.5	None	22.4	Side resistance only
TPC-9	3.0	None	16.8	Side resistance only

have a general idea of how the computations for side resistance compared as a part of the total ultimate capacity. A design procedure would be unsafe and inadequate if it consistently overestimated the side resistance by a large margin.

Figures 3.1 through 3.6 indicate that the recommended design procedures for drilled shafts in clay, clay-shale, and sand give reasonably good estimates of the total ultimate capacity. For the Texas and California load tests, errors in the prediction of the total capacities varied from -41 to +34 percent. [A positive (+) percent error indicates overestimating the actual capacity and a negative (-) percent error indicates underestimating the actual capacity]. Sixty-five percent of the predictions varied within a range of ± 20 percent of the measured values. Of all the computed loads 76 percent were conservative. For the Texas and California load tests, total capacities predicted by the secondary design procedures had errors that varied from -21 to +41 percent. However, as many as 86 percent of the predictions were within ± 20 percent of the measured values. Sixty-eight percent of all the computed loads were conservative.

The four methods for computing shaft capacities in cemented alluvial fan deposits varied significantly with respect to margin of error. Computations by Method 3 show errors varying from -77 to +36 percent, with 92 percent of the loads computed conservatively. Unfortunately, only 8 percent of all the predictions fell within a range ± 20 percent of the measured values. Like Method 3, errors in loads computed by Method 4 varied widely, from -65 percent to +72 percent, with 83 percent of the predictions being conservative. Here again, only 8 percent of the computed loads fell within a range of ± 20 percent of the actual values. As discussed in Chapter 2, the large errors generated by Methods 3 and 4 were mainly the result of the lack of better techniques for obtaining undistrubed samples in soils similar to those tested in Arizona. Computational Methods 5 and 6, which are based on results from pressuremeter tests, gave superior predictions of total capacities. Loads computed by Method 5 varied within a range of error of -42 to +33 percent, with 50 percent of the predictions being conservative. Eighty-three percent of the computed loads were within ± 20 percent of the actual loads. Capacities computed by Method 6 gave a margin of error of

-40 to +14 percent, with 83 percent of the predictions being conservative. For this computational method, 58 percent of the predictions were within $\pm$ 20 percent of the actual values.

CHAPTER 4. COMPUTER PROGRAMS FOR THE DESIGN OF DRILLED SHAFTS

GENERAL

Two computer programs were developed to serve as design aids in computing the total ultimate capacity of drilled shafts by the primary design procedure as well as by the secondary design procedure. The following sections give a detailed explanation of input variables of the computer programs. In addition, a detailed discussion of the program routines is presented. Appendix B serves as a quick reference in finding the definitions of the input variables for both computer programs.

PROGRAM SHAFT1

The routines of program SHAFT1 are based on the primary design procedure for calculating the total ultimate capacity of a shaft.

Input Variables

To make it as easy as possible for a user of the program to become familiar with the terminology used for the input variables, variable names were selected to be as similar as possible to the actual terminology presented in previous chapters. The same pertinent soil information that is required to make design calculations by hand is fed into the computer. The variables consist of the bottom depth of a particular soil layer; the type of soil in that layer (sand, clay, shale, etc); the average total unit weight for that layer; the average undrained angle of internal friction for each layer; the undrained cohesion; the bearing capacity factor, N_c; the shear strength reduction factor, α; and the maximum load transfer that is allowed in each soil layer. Other information, such as water table depth, shaft geometry, and factor of safety, must also be input into the computer. In the following paragraphs, each input variable is defined and its role in the program routines is explained. The reader should refer to the input formats for this program in Appendix B.

NHEAD. This is the number of cards that comprise the heading description for each set of soil profile data. This integer variable must have a value of one or greater, but less than four. Setting NHEAD equal to zero will end the program.

HEADNG. These are singly subscripted alphanumeric variables used to store any information to describe a design case. The number of cards of description variables is defined by the value of NHEAD. The heading information can be punched anywhere on the cards.

NTYPDS. This integer variable serves as a flag to signal to the computer the type of calculations that are to be completed with one particular soil profile. When NTYPDS is set equal to one, ultimate capacities are computed for the soil profile information furnished in set A of the input format. Upon completion of the calculations, the computer returns to the beginning of the program to read a new set of soil profile data (i.e., a new set A). However, if one wishes to reutilize the same soil profile data to make computations with variables different from those found on the original card No. 3 of set A, NTYPDS is given a value of two. Immediately following set A, a new card No. 3 is placed with the revised data. If one wishes to reutilize particular soil profile data and change the shear strength reduction factor for each soil layer in addition to the variables found on card No. 3, NTYPDS is given a value of three. In this case, the cards that comprise set B must be placed immediately after set A (see input format).

DSTART. The computer program will calculate the total ultimate capacities for a series of stem diameters for both straight-sided shafts and underreamed shafts. The first diameter to be considered is DSTART. This value is increased in increments of 0.5 ft up to a limit determined by DIALIM. The units of DSTART are feet.

DIALIM. The units of this input variable are also feet. DIALIM represents the largest diameter to be considered of a series of stem diameters. If only one stem diameter is of interest for a particular soil profile, then DSTART is set equal to that diameter and DIALIM is set equal to zero.

RATIO. This variable is given a value of zero for a straight-sided shaft.

For belled drilled shafts, RATIO is the ratio of the base diameter to the stem diameter.

BELANG. This is the angle of the bell with respect to the vertical. For straight shafts, set BELANG equal to zero.

TOPLEN. This is the length of the top section of the shaft that does not contribute to side resistance. The units are in feet.

BS. This is the length of the bottom section of the shaft that does not contribute to side resistance. The units are in feet.

N. This variable represents the number of soil layers in a soil profile. N can be as large as 29.

WTD. WTD defines the water table depth. The units are in feet. When inputing the values for the bottom depth of each layer of soil (DEPTH), WTD must be made a soil boundary. If the water table depth actually falls within the boundaries of a soil layer, then that layer must be considered as two layers, with WTD as the common boundary of the two layers.

P. P is the design, or working, capacity of the drilled shaft. As soon as this design load is reached or exceeded for a particular soil profile and shaft diameter, the computations cease and the computer returns to the beginning of the program or begins calculations for another shaft diameter, depending on the values of NTYPDS and DIALIM.

QCHECK. The efficiency of a drilled shaft can be evaluated in terms of its ratio of tons of load capacity per cubic yard of concrete. A section of the computer program makes efficiency computations by comparing the values of the load-volume ratios of straight-sided and/or belled shafts of different lengths in one particular soil profile. Upon completion of the calculations for that soil profile, the length, stem and base diameters, load-volume ratio, and total ultimate resistance, as well as volume of concrete, are printed for the shaft size with the largest load-volume ratio.

It was thought unnecessary to make efficiency computations for drilled shafts with small penetrations at the very start of the analyses. Therefore, a load is selected called QCHECK. At computed loads less than QCHECK, the efficiency computations are eliminated. QCHECK has units in pounds.

FST. This is the factor of safety to be applied to the ultimate capacity, Q_{ult}.

FSB. This factor of safety is applied only to the ultimate base resistance to limit excessive settlement.

The following variables pertain to each layer of soil in a particular profile:

ALPHA. This is the strength reduction factor.

CNC. The bearing capacity factor for clay. If the soil is sand, CNC can be left blank.

GAMMA. This is the total unit weight in units of pounds per cubic feet. The program makes the necessary calculations to convert to buoyant, or effective, unit weight below the water table.

PHI. This variable represents the angle of internal friction. As presented in Chapter 2, the undrained angle of internal friction is used for clay and the effective angle of internal friction is used for sand. The units are in degrees.

CUTOP. CUTOP represents the undrained cohesion at the top boundary of a soil layer, in units of psf. For sand, CUTOP is equal to zero.

CUBOT. CUBOT represents the undrained cohesion at the bottom boundary of a soil layer, in units of psf. For sand, CUBOT is equal to zero.

For clay layers where the undrained cohesion is practically equal to the undrained shear strength, CUTOP and CUBOT can be set equal to the appropriate values and PHI can be set equal to zero.

TRANLM. This is the maximum load transfer (in psf) that is permitted for a particular soil layer.

SOIL. A code is used to represent the soil type. For sand, SOIL is given a value of 1.0. For cohesive soil, SOIL is set equal to 2.0.

DEPTH. This input variable defines the lower boundary of a soil layer (in feet). See the definition of WTD for information related to DEPTH.

Program Routines

In the main routine of program SHAFT1, the computer first calculates the shear strength profile for the soil deposit. Then the side resistance is computed by increasing the length of the shaft in one-foot increments. The starting length of the shaft is one foot plus the noncontributing sections along the shaft determined by the input variables TOPLEN and BS. For underreamed drilled shafts, the bell height, which is calculated by the computer, is also included as part of the shaft length not contributing to side resistance.

After side resistance for a particular length is determined, the base capacity is computed. In making this computation, the computer can use a number of subroutines. The influence of all soil within two base diameters below the base of the shaft is considered. When the two-base-diameter distance falls within the same clay layer, SUBROUTINE SUATB1 computes the bearing capacity. If the base capacity of the shaft is influenced only by sand, then the bearing capacity is determined by SUBROUTINE PHIATB. When two or more soil layers are within the base bearing capacity influence zone, SUBROUTINE SUATB2 is called to calculate the base resistance. A weighted average of the bearing capacities relative to thickness of the strata that are within the two-diameters influence zone below the base is used as the ultimate bearing capacity.

Once both side and base resistances are computed, the total ultimate resistance is calculated and the appropriate factors of safety (FST, FSB) are applied to obtain working loads. The length of the shaft is then increased one foot and the whole procedure is repeated. This process continues until any one of the following events occur:

(1) the design load is reached or exceeded or

(2) the shaft reaches a depth where no more soil information is available to the computer beyond a distance of two base diameters below the base.

In order to end the program, the computer must read in a value of zero for NHEAD. This calls for inserting a blank card following the last set of soil profile information; that is, the last set A or set B (see input format). The flow chart for SHAFT1 is shown in Fig. 4.1.

Program Applications

The computer program SHAFT1 lends itself to a number of useful applications. Primarily, it can be employed in obtaining curves which are useful in design. Figure 4.2 gives an example of such an application. The information for the graph was obtained from the check problem output in

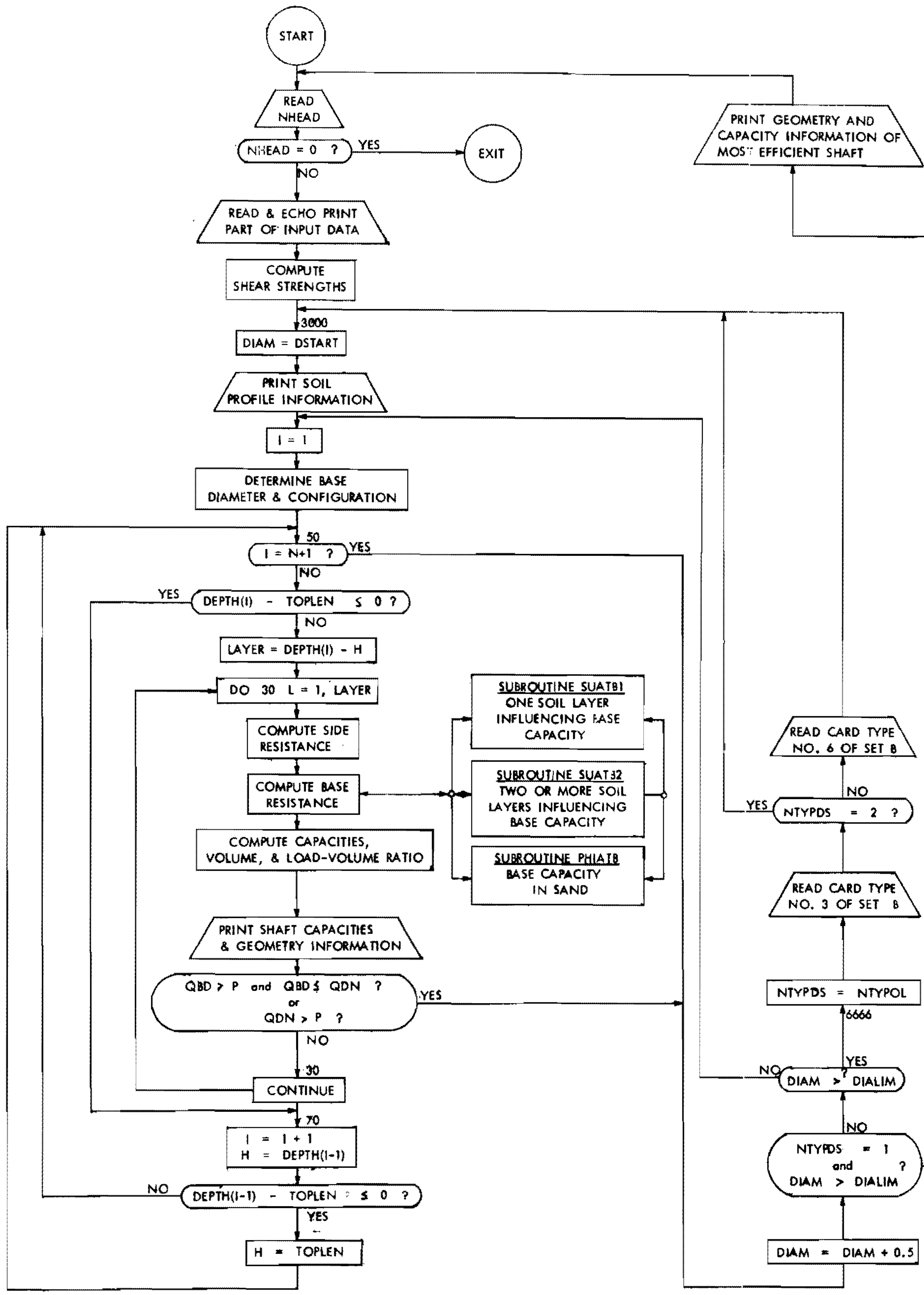


Fig. 4.1 Flowchart of SHAFT1

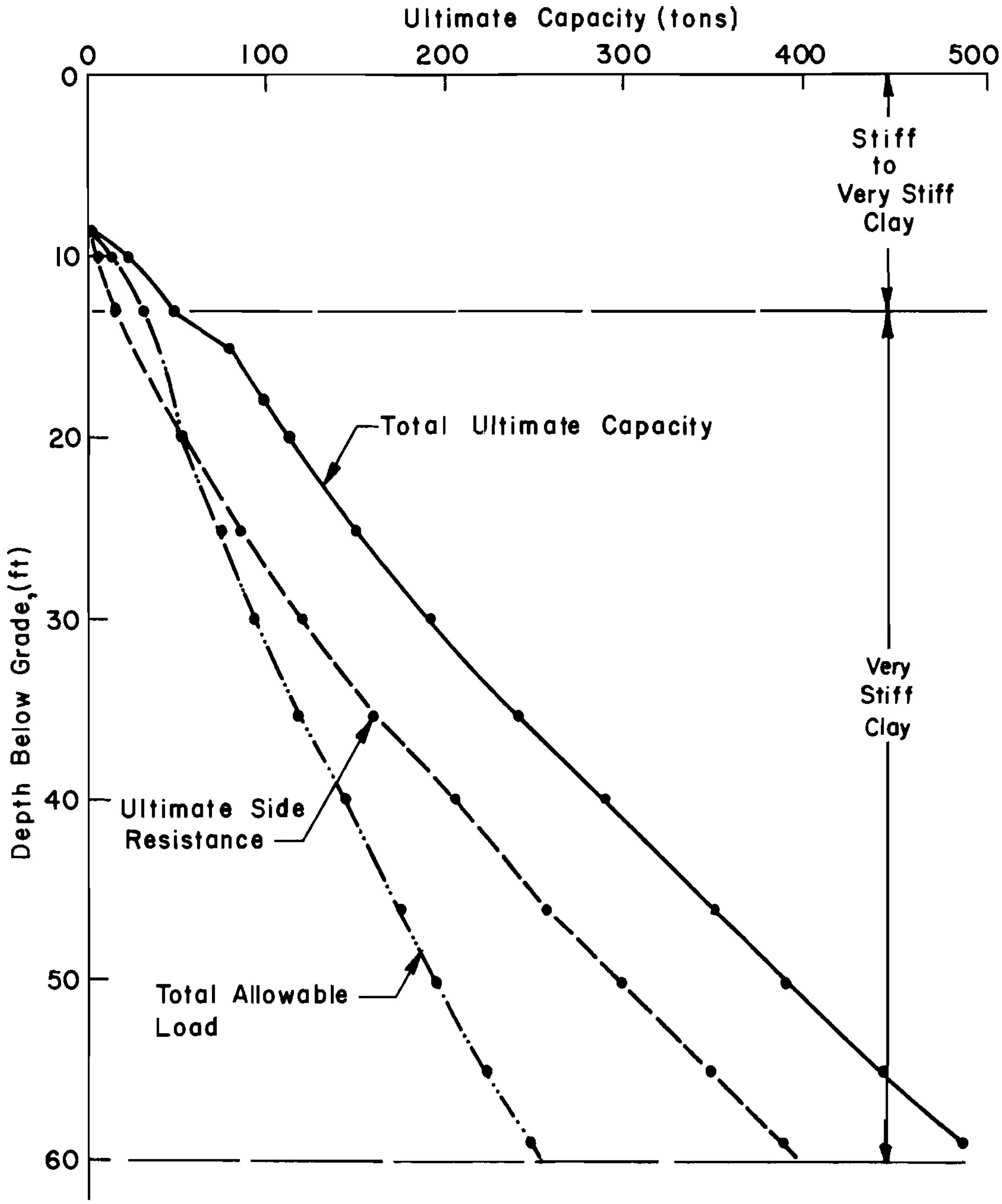


Fig. 4.2 Design Curves for Ultimate Capacity and Total Allowable Load for Texas G-2 Site

Appendix B.

The output information of SHAFT1 can also be used to design drilled shafts with a "step-tapered" geometry. This can be done by running the program for a series of diameters and using the difference in ultimate capacities with the respective difference in lengths for each diameter that will comprise the final design.

Another valuable feature of this program is that it can be easily modified to incorporate other design criteria that may be establised with future research. The base capacity and side resistance are computed in different routines of the program. Therefore, if later investigations demonstrate that there is a better method of calculating side resistance, the portion of the program that deals with side resistance can be modified without having to change the base capacity routine. Similarly, the base capacity routine can be modified and the side resistance routine left unaltered if future studies so indicate. Future investigations may call for modifying the method of computing side resistance as well as base capacity, and this can also be done without having to alter the whole program. For example, if the pressuremeter tests results prove to give the best estimate of actual soil engineering properties and this method of testing becomes economically available, program SHAFT1 can be easily modified to use the pressuremeter test results to calculate the total ultimate resistance of a drilled shaft.

A fourth possible use of SHAFT1 is in conjunction with another program, such as PX4C3, to produce a more detailed study of a proposed design. PX4C3 is a computer program developed at The University of Texas which enables one to establish load-settlement curves for a shaft of known diameter and length installed in a soil medium with known or estimated stress-strain properties. SHAFT1 can be used to select the desired diameter and length for a particular working load. That information is then supplied to program PX4C3 to determine an appropriate load-settlement curve.

PROGRAM BSHAFT

Input Variables

BSHAFT was developed to compute ultimate capacities by the secondary design procedure. All input variables for the program are the same as those for SHAFT1 with the following exceptions:

(1) On the card where NHEAD appears, TYPB must also be inserted. TYPB refers to the type of dynamic-penetration-test information available for a specific soil profile. For the SDHPT cone-penetration-test results, the letters SDHPT are inserted for TYPB; for the Standard Penetration Test results, SPT is used.

(2) The information for the each soil layer is also different. For BSHAFT, only ALPHA, BLOWS, SOIL, DEPTH, and TRANLM are required for each soil stratum. These are discussed in the following paragraphs.

ALPHA. For clay soil, the same parameters as those on Table 3.2 are suggested. For sand, however, a value for ALPHA of 1.0 can be employed because a direct correlation between blowcount and load transfer, which was established in Chapter 3, is used. If required, a lower value of ALPHA may be used.

BLOWS. This variable refers to the average numbers of blows per foot for a soil stratum for either kind of dynamic penetration test.

SOIL. Similarly to its use in program SHAFT1, SOIL identifies the soil type. For program BSHAFT, clay soils are subdivided into different groups. SOIL is assigned a value of 2.0 for homogeneous clays (CH). A value of 3.0 is for clay-shale, 4.0 for silty clays (CL), and 5.0 for sandy clays (CL). Sands are identified by a value of 1.0.

DEPTH AND TRANLM. These symbols are the same for BLSHAFT as they are for SHAFT1. However, since WTD, the water table depth, does not come into play in the blowcount-shear strength correlations in clays or the blow-count-load transfer correlations in sand, there is no need to consider WTD as a soil depth boundary as is required in program SHAFT1.

Program Routines

The program routines for BSHAFT are similar to those of SHAFT1 except for some minor modifications. The main routine in BSHAFT contains a section for computing the shear strengths and load transfers from dynamic-penetration-test results. Furthermore, since the shear strength is computed to be constant for each soil stratum, there is need for only one subroutine for calculating the base capacity of the drilled shaft.

Program Applications

The output information for BSHAFT is similar to that for program SHAFT1 (see sample output for BSHAFT in APPENDIX B). Therefore, the reader is referred to the previous section for a detailed discussion of applications of program SHAFT1.

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

Based on the results of this study, conclusions can be drawn as follows:

(1) The design parameters proposed in this report give satisfactory predictions of shaft ultimate capacities as demonstrated in Chapter 3.

(2) Although the revised correlations between dynamic penetration resistance and undrained shear strength for clays, and between dynamic penetration resistance and load transfer in sand, gave relatively good estimates of actual shaft capacities, the secondary design procedure should be used with caution because of the many factors that can affect the penetration resistance. The method should not be employed for soils with properties and conditions significantly different from those reported herein.

(3) Uncertainty of in situ soil properties remains a major obstacle in establishing a more generalized design procedure. Existing sampling techniques and testing methods are often inadequate for accurate determination of in situ soil properties. This situation was especially evident in the Arizona soils investigation. The pressuremeter shows great promise as an in situ testing device.

(4) The developed computer programs SHAFT1 and BSHAFT are time-saving tools that should be used as design aids. The programs can be easily modified should future research so indicate.

The following recommendations are made in connection with future research in the area of drilled shafts:

(1) More instrumented shafts should be tested to improve the present design procedures or to verify more definitely the existing criteria.

(2) An effort should be made to test shafts in soils different from those thus far encountered.

(3) Emphasis should also be placed on improving present sampling techniques and developing reliable in situ soil testing methods.

REFERENCES

1. Aurora, R. P., and Reese, L. C., Behavior of Axially Loaded Drilled in Clay-Shales, Research Report 176-4, Center for Highway Research, The University of Texas at Austin, March 1976.

2. Barker, W. R., and Reese, L. C., Load Carrying Characteristics of Drilled Shafts, Research Report 80-6, Center for Highway Research, The University of Texas at Austin, November 1969.

3. Barker, W. R., and Reese, L. C., Load Carrying Characteristics of Drilled Shafts Constructed with the Aid of Drilling Fluids, Research Report 89-9, Center for Highway Research, The University of Texas at Austin, November 1970.

4. Beckwith, G. H., and Bedenkop, D. V., An Investigation of the Load-Carrying Capacity of Drilled Cast-in-place Concrete Piles Bearing on Coarse Granular Soils and Cemented Alluvial Fan Deposits, Report: AHD-RD-10-122, Arizona Highway Department, Research Division, Phoenix, Arizona, May 1973.

5. Campbell, D. B., and Hudson, W. R., The Determination of Soil Properties in Situ, Research Report 89-7, Center for Highway Research, The University of Texas at Austin, November 1969.

6. Chuang, J. W., and Reese, L. C., Studies of Shearing Resistance Between Cement Mortar and Soil, Research Report 89-3, Center for Highway Research, The University of Texas at Austin, May 1969.

7. Ehlers, C. J., Reese, L. C., and Anagnos, J. N., The Nuclear Method of Soil Moisture Determination at Depth, Research Report 89-4, Center for Highway Research, The University of Texas at Austin, June 1969.

8. Engeling, D. E., Behavior of Three Instrumented Drilled Shafts Under Short Term Axial Loading, Research Report 176-3, Center for Highway Research, The University of Texas at Austin, May 1974.

9. Gibbs, H. J., and Holtz, W. C. (1957), "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, London, 1957.

10. Hamoudi, M. H., Coyle, H. M., and Bartoskewitz, R. E., Correlation of the Texas Highway Department Cone Penetrometer Test with Unconsolidated-Undrained Shear Strength of Cohesive Soils, Research Report 10-1, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1974.

11. O'Neill, M. W., and Reese, L. C., Behavior of Axially Loaded Drilled Shafts in Beaumont Clay, Research Report 89-8, Center for Highway Research, The University of Texas at Austin, December 1970.

12. Parker, F., Jr., and Reese, L. C., Experimental and Analytical Studies Of Behavior of Single Piles in Sand Under Lateral and Axial Loading, Research Report 117-2, Center for Highway Research, The University of Texas at Austin, November 1970.

13. Reese, L. C., Brown, J. C., and Dalrymple, H. H., Instrumentation for Measurements of Lateral Earth Pressure in Drilled Shafts, Research Report 89-2, Center for Highway Research, The University of Texas at Austin, September 1968.

14. Reese, L. C., and Hudson, W. R., Field Testing of Drilled Shafts to Develop Design Methods, Research Report 89-1, Center for Highway Research, The University of Texas at Austin, April 1968.

15. Reese, L. C., and O'Neill, M. W., Criteria for the Design of Axially Loaded Drilled Shafts, Research Report 89-11F, Center for Highway Research, The University of Texas at Austin, May 1971.

16. Reese, L. C., Touma, F. T., and O'Neill, M. W., "Behavior of Drilled Piers Under Axial Loading," Journal of the Geotechnical Engineering Division, GT, May, 1976.

17. Schmertmann, J. H., Guidelines for Use in Soil Investigations and Design of Foundations for Bridge Structures in the State of Florida, Florida State Road Department, Office of Materials and Research, Gainesville, Florida, September, 1967.

18. Skempton, A. W., "The Bearing Capacity of Clays," Proceedings of the Building Research Congress, 1951, Division I, Building Research Congress, London, 1951.

19. Terzaghi, K., Theoretical Soil Mechanics, John Wiley & Sons, New York, 1943.

20. Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 1967.

21. Texas Highway Department, Foundation Exploration and Design Manual, State Highway Department of Texas, Bridge Division, Austin, Texas, January, 1972.

22. Touma, F. T., and Reese, L. C., The Behavior of an Axially Loaded Drilled Shaft in Sand, Research Report 176-1, Center for Highway Research, The University of Texas at Austin, December 1972.

23. Vesic, A. S., "Ultimate Loads and Settlements of Deep Foundations in Sand," Proceedings, Symposium on Bearing Capacity and Settlement of Foundations, Duke University, Durham, North Carolina, 1965.

24. Vijayvergiya, V. N., Hudson, W. R., and Reese, L. C. (1969), Load Distribution for a Drilled Shaft in Clay Shale, Research Report 89-5, Center for Highway Research, The University of Texas at Austin, March 1969.

25. Welch, R. C., and Reese, L. C., Lateral Load Behavior of Drilled Shafts, Research Report 89-10, Center for Highway Research, The University of Texas at Austin, May 1972.

26. Wilhelms, R. C., Load Test-48" Diameter Pile, California Department of Transportation Division of Structures, Engineering Geology Section, California, May, 1975.

27. Wooley, J. A., and Reese, L. C., The Behavior of an Axially Loaded Drilled Shafts Under Sustained Loading, Research Report 176-2, Center for Highway Research, The University of Texas at Austin, May 1974.

APPENDIX A

SOIL INFORMATION FOR DIFFERENT TEST SITES

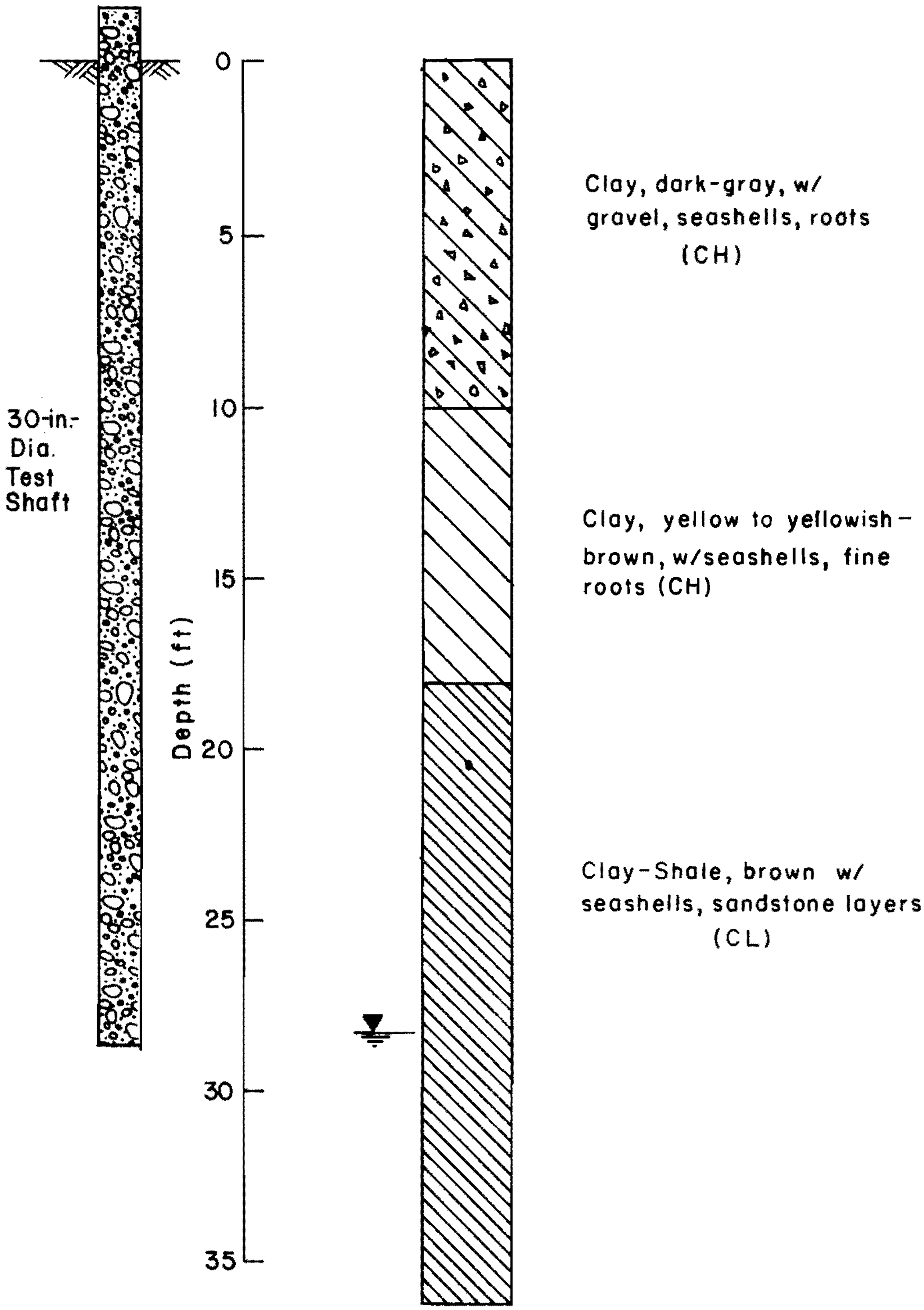


Fig. A.1 San Antonio SA Test, Soil Profile and Test Shaft (after Vijayvergiya, Hudson, and Reese, 1969)

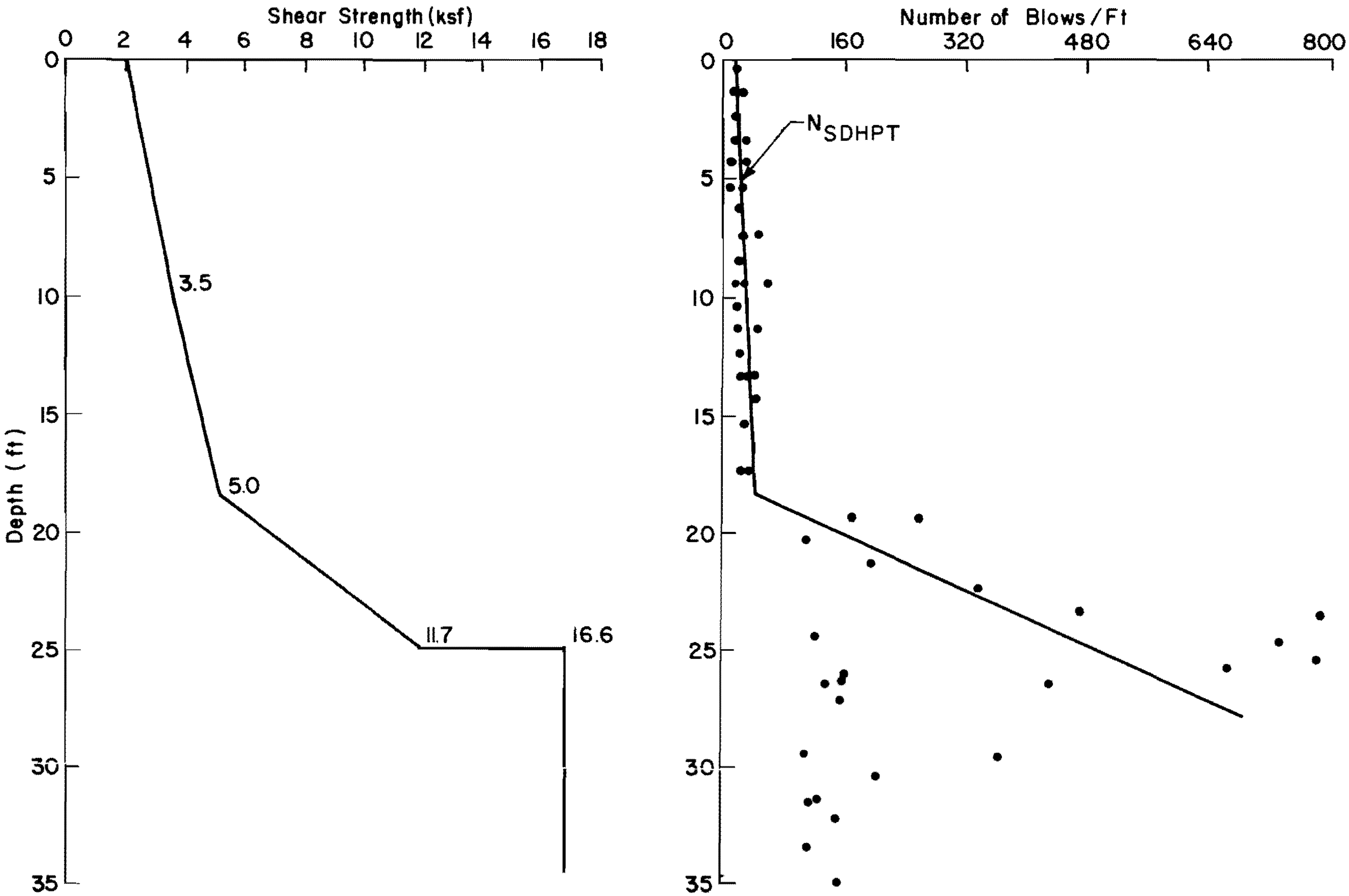


Fig. A.2 San Antonio Test, Shear Strength and Dynamic Penetrometer-Data Used in Analysis (after Vijayvergiya, Hudson, and Reese, 1969)

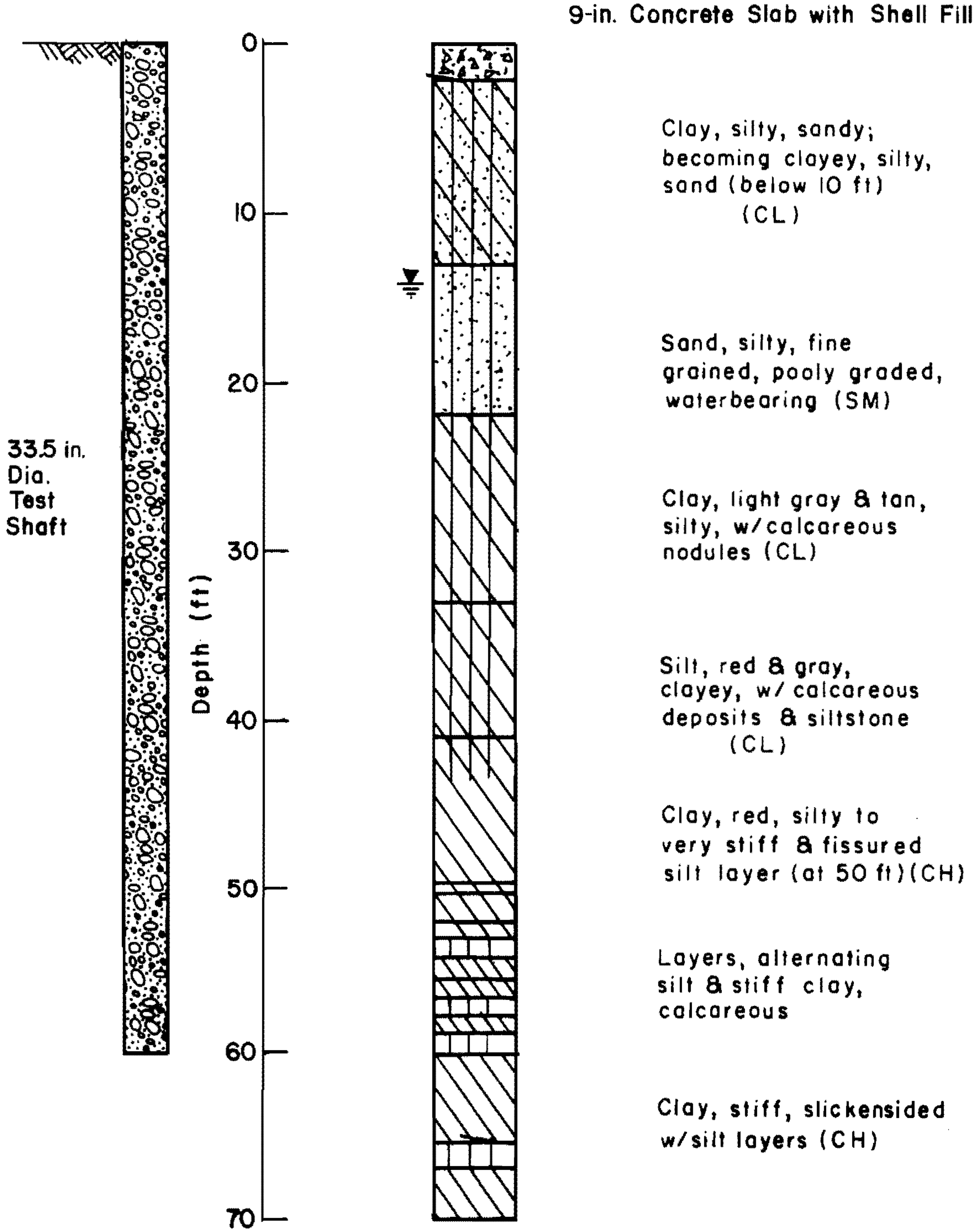


Fig. A.3 Houston HB&T Test, Soil Profile and Test Shaft (after Barker and Reese, 1970)

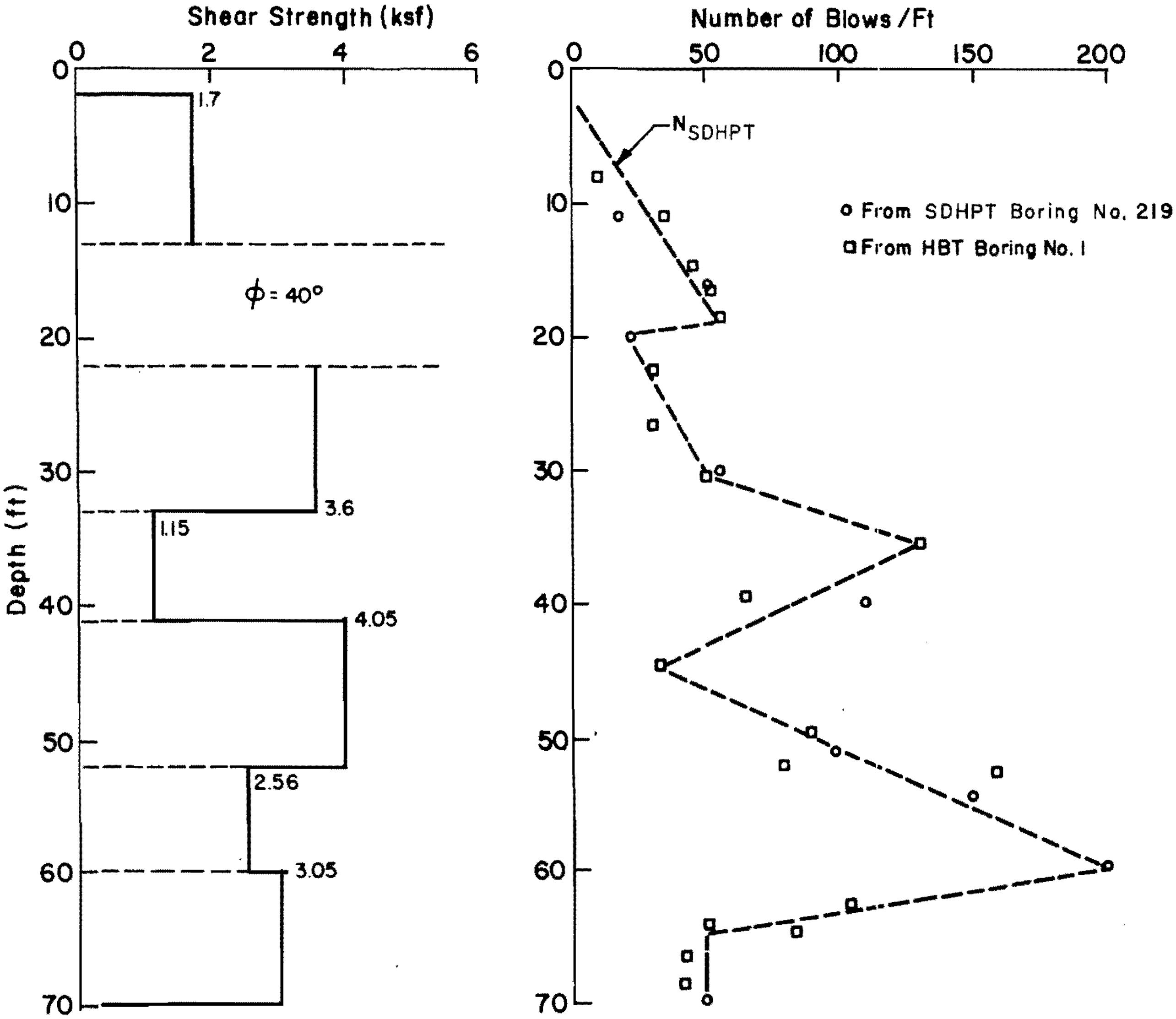


Fig. A.4 Houston HB&T Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Barker and Reese, 1970)

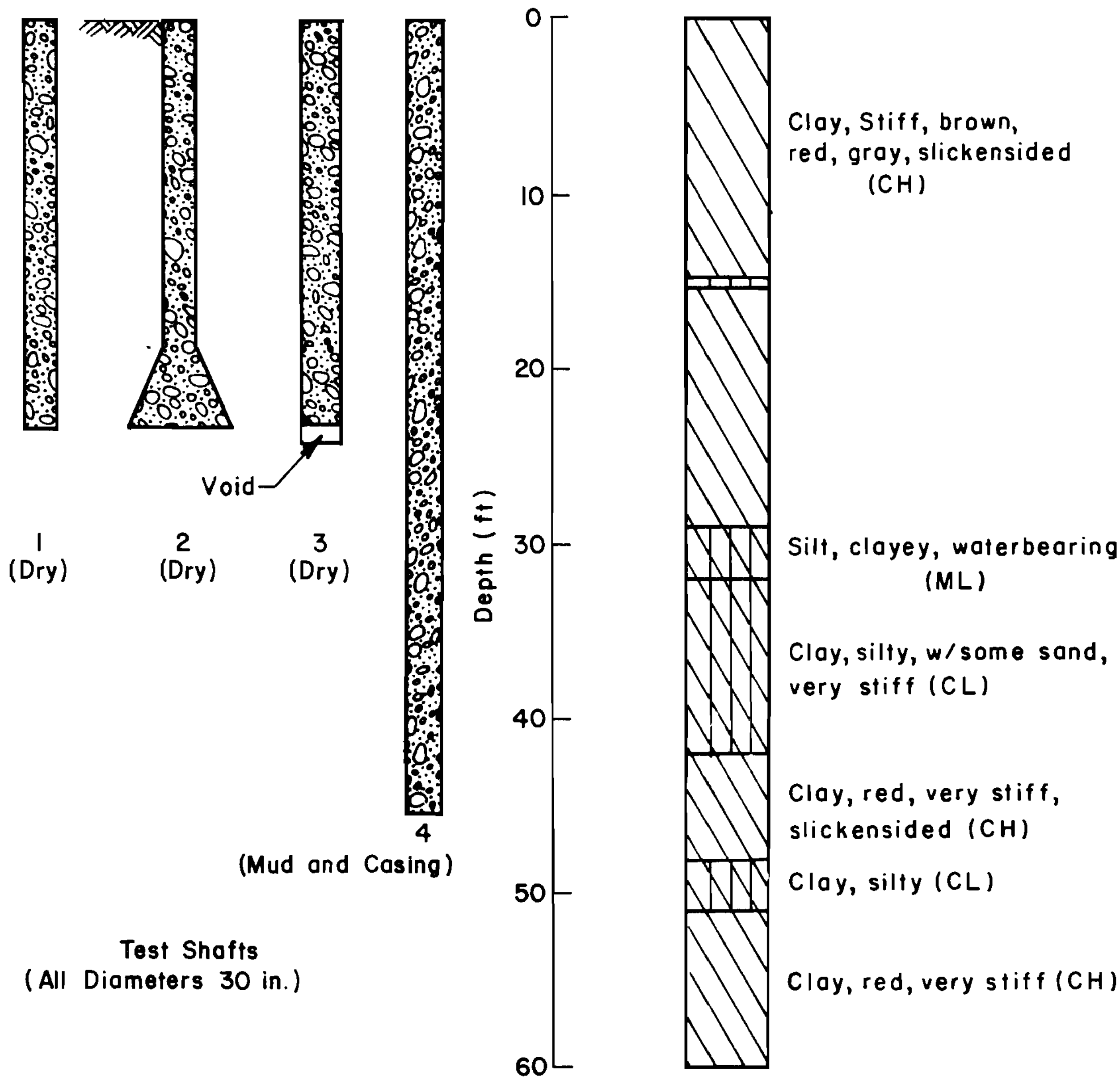


Fig. A.5 Houston S1, S2, S3, and S4 Tests; Soil Profile and Test Shafts (after O'Neill and Reese, 1970)

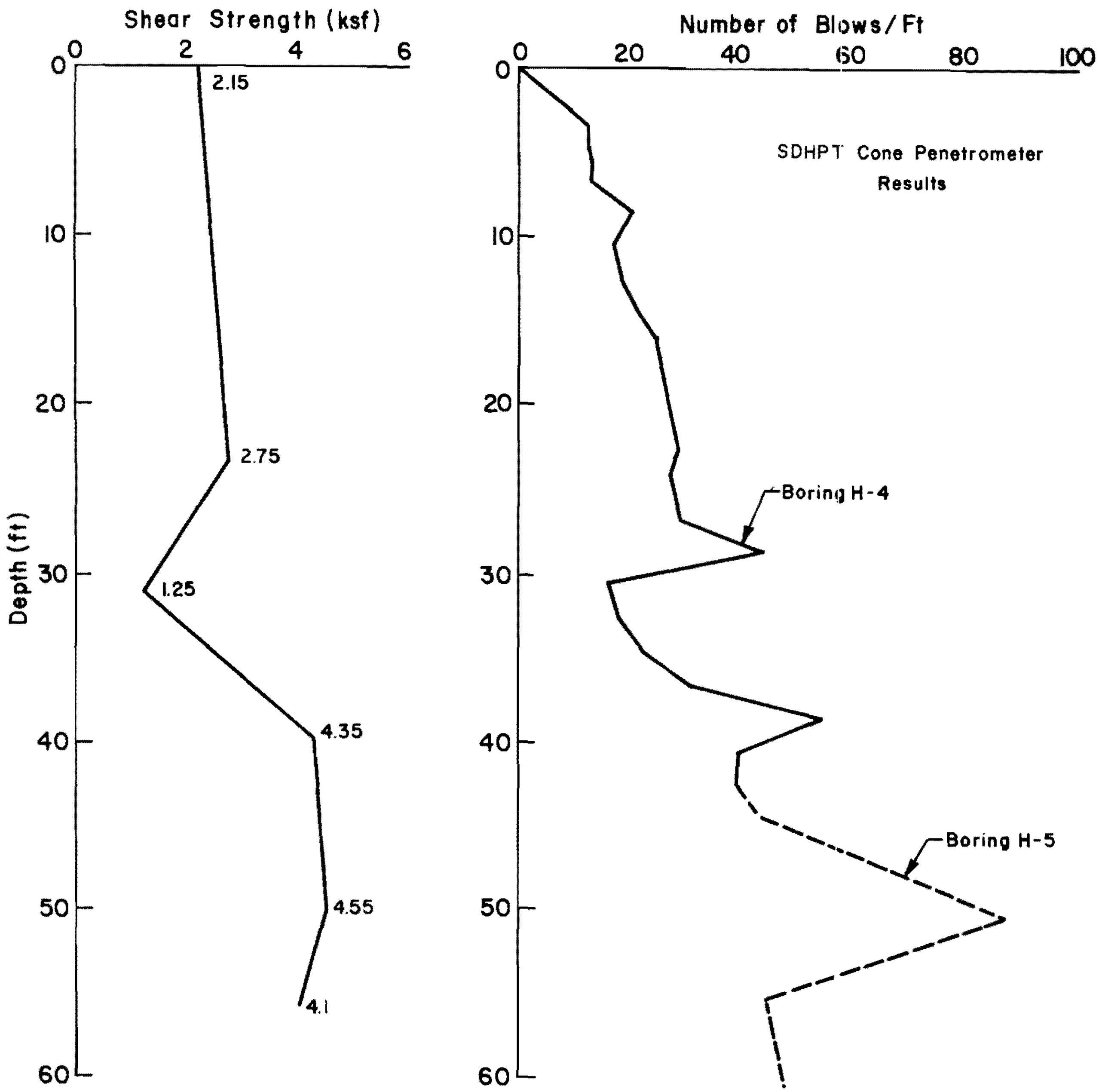


Fig. A.6 Houston S1, S2, S3, and S4 Tests; Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after O'Neill and Reese, 1970)

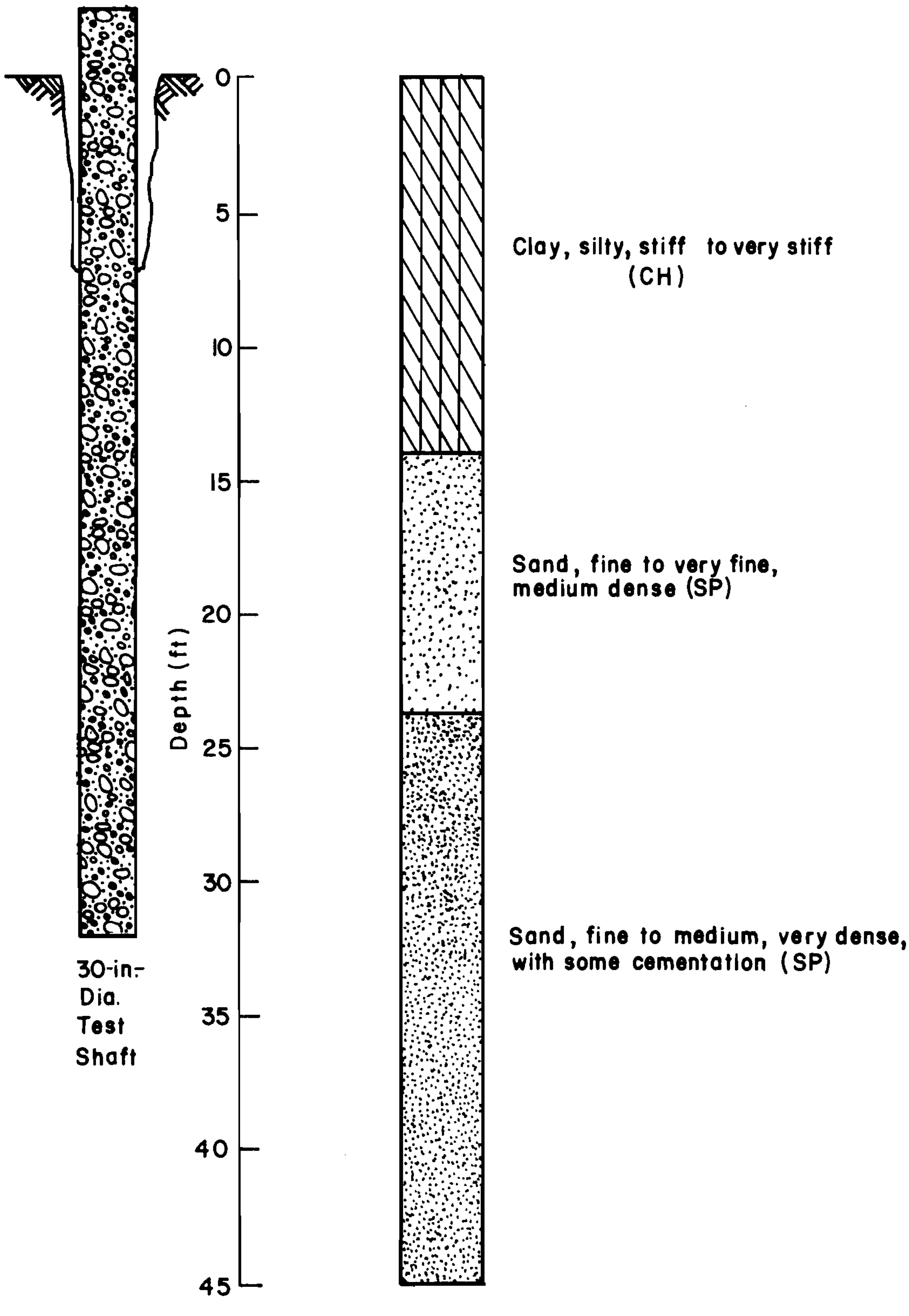


Fig. A.7 George West US59 Test, Soil Profile and Test Shaft (after Touma and Reese, 1972)

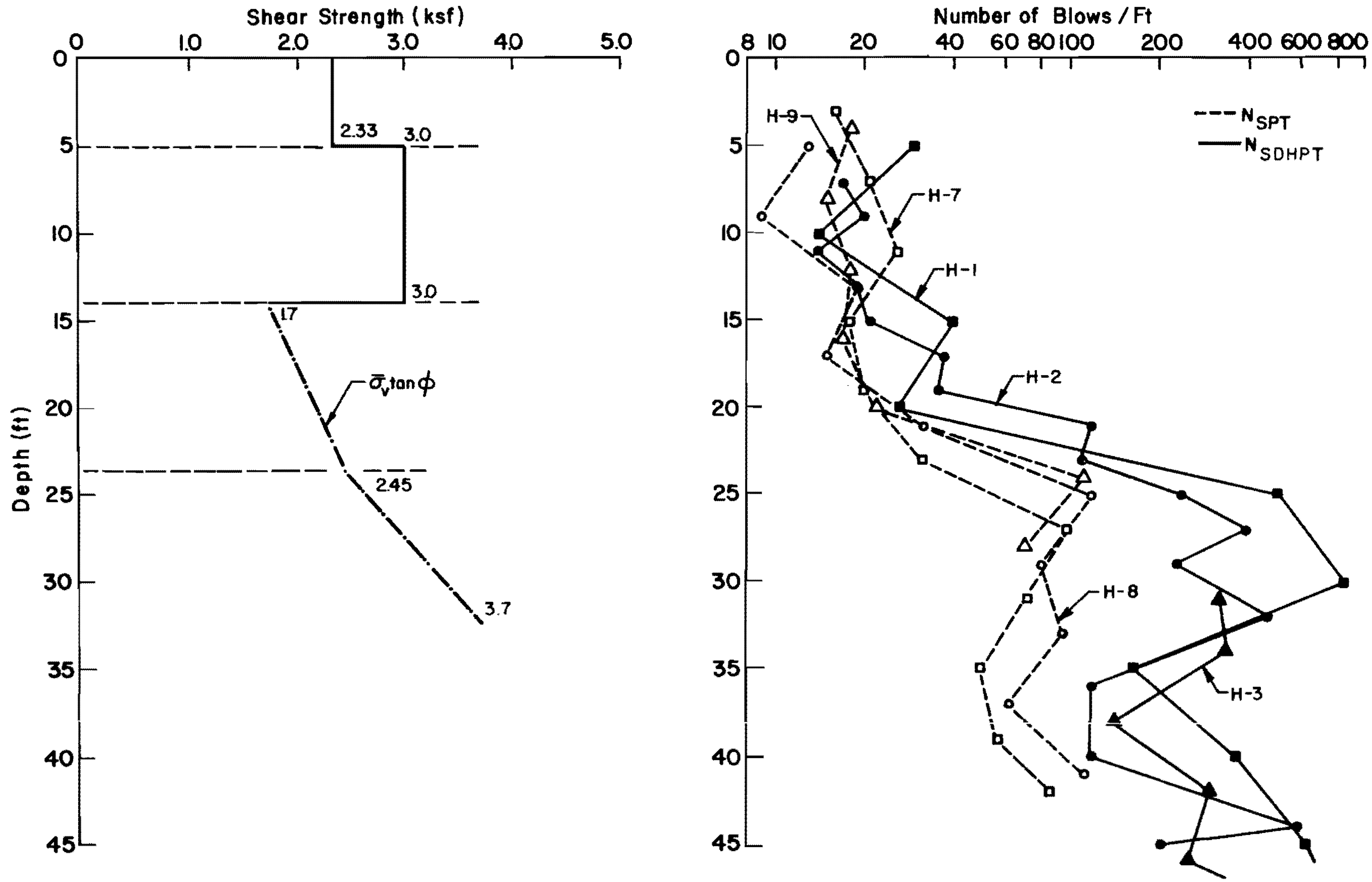


Fig. A.8 George West US59 Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Touma and Reese, 1972)

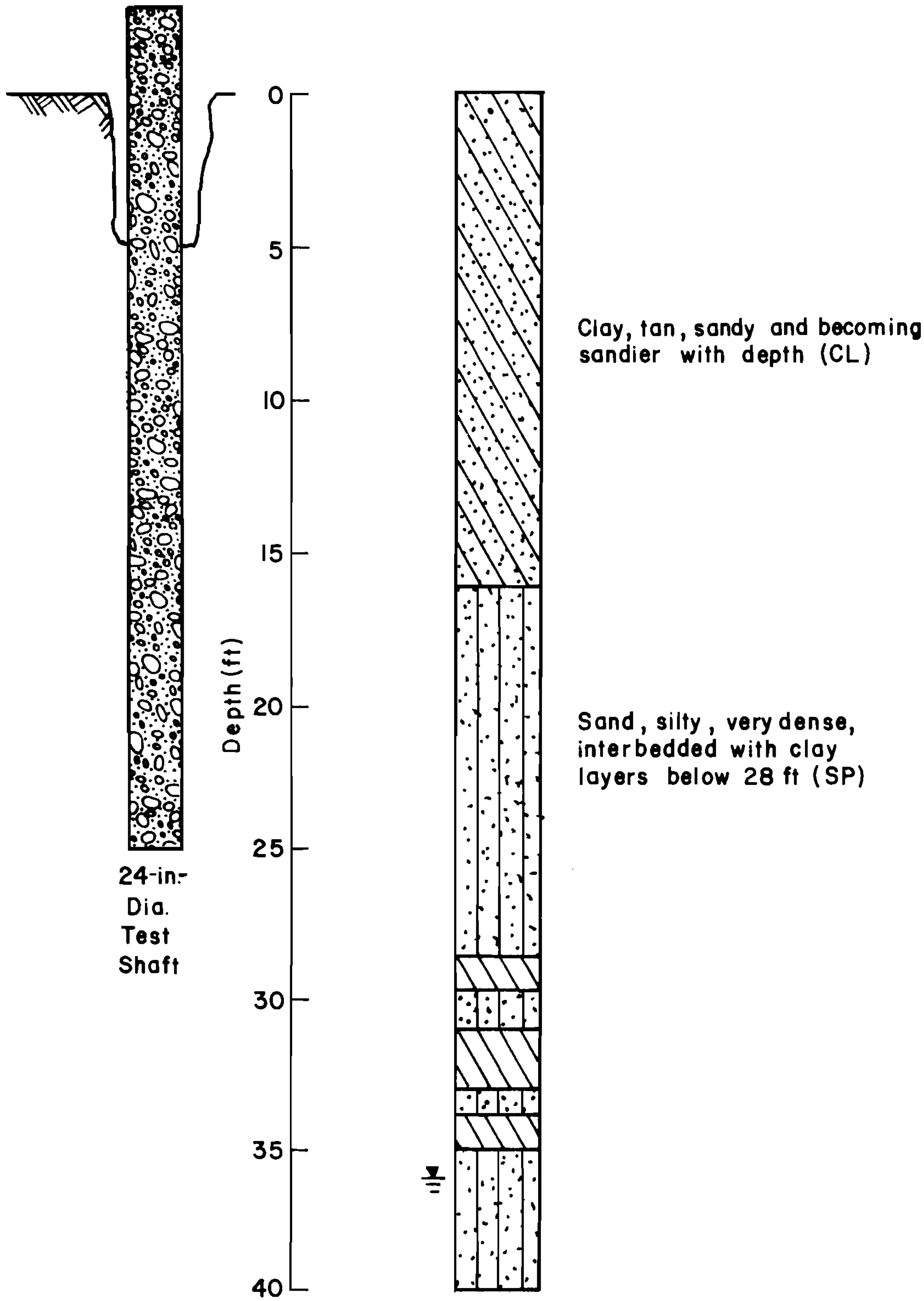


Fig. A.9 George West HH Test, Soil Profile and Test Shaft (after Touma and Reese, 1972)

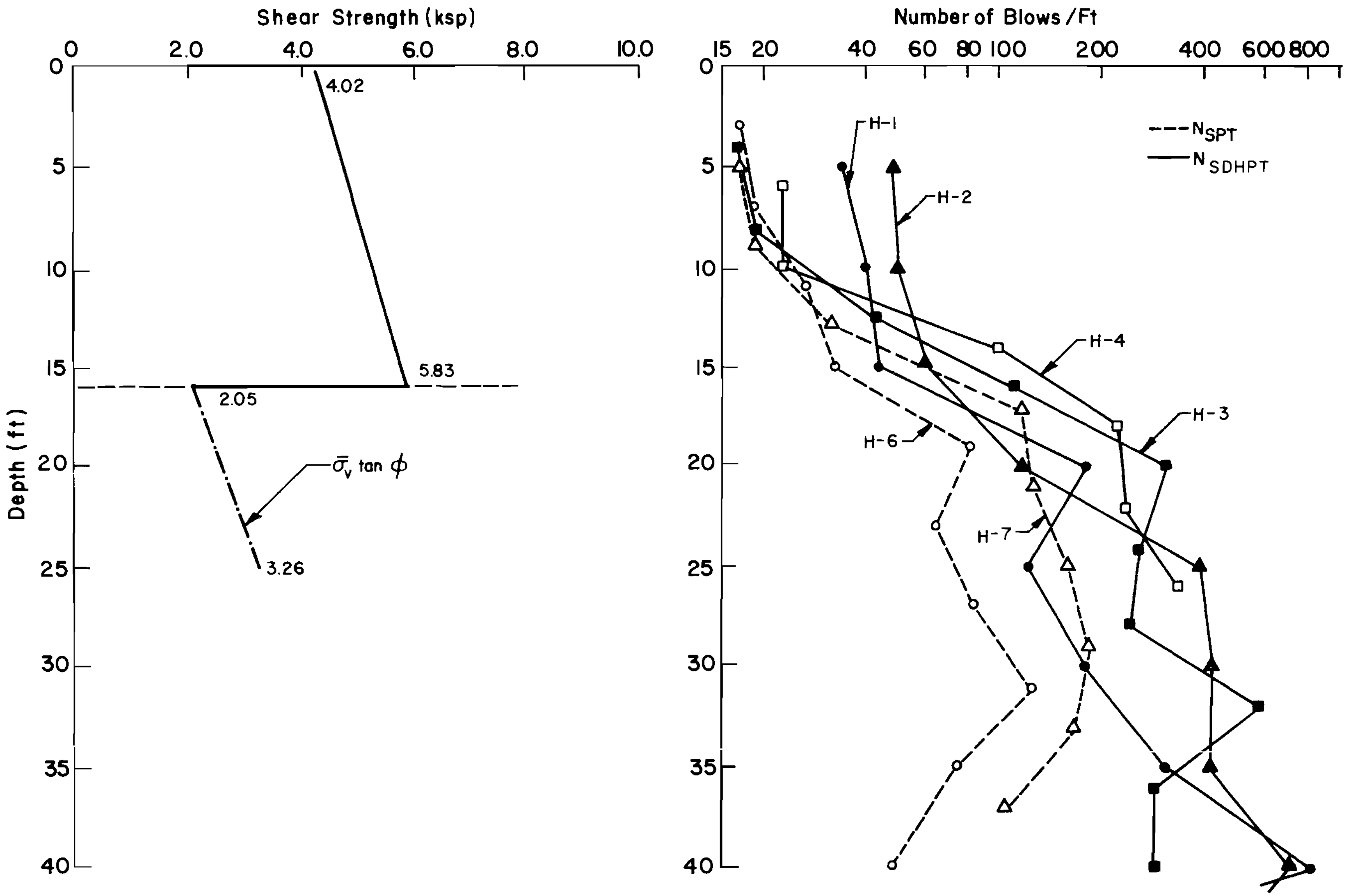


Fig. A.10 George West HH Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Touma and Reese, 1972)

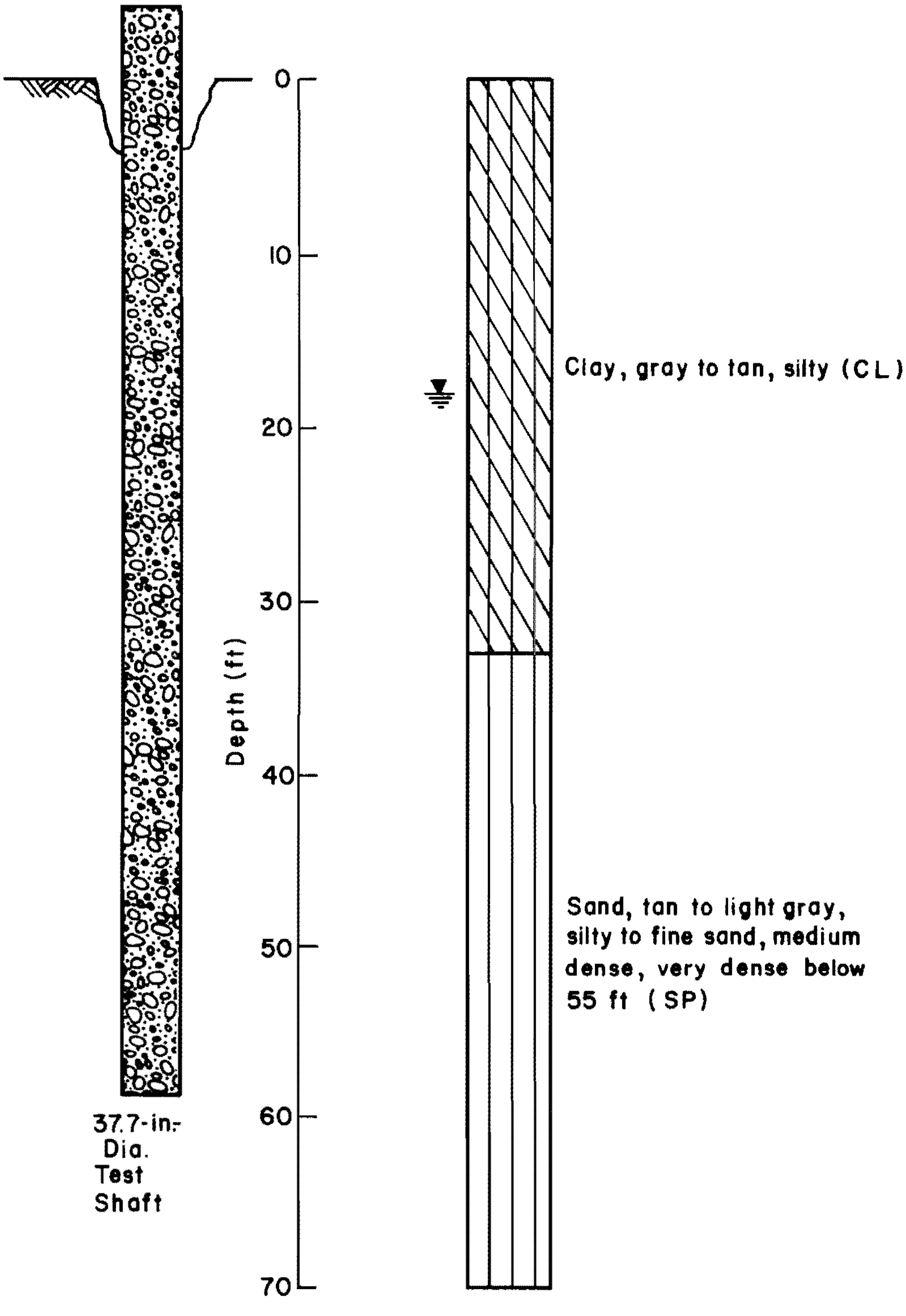


Fig. A.11 Houston Gl Test, Soil Profile and Test Shaft
(after Touma and Reese, 1972)

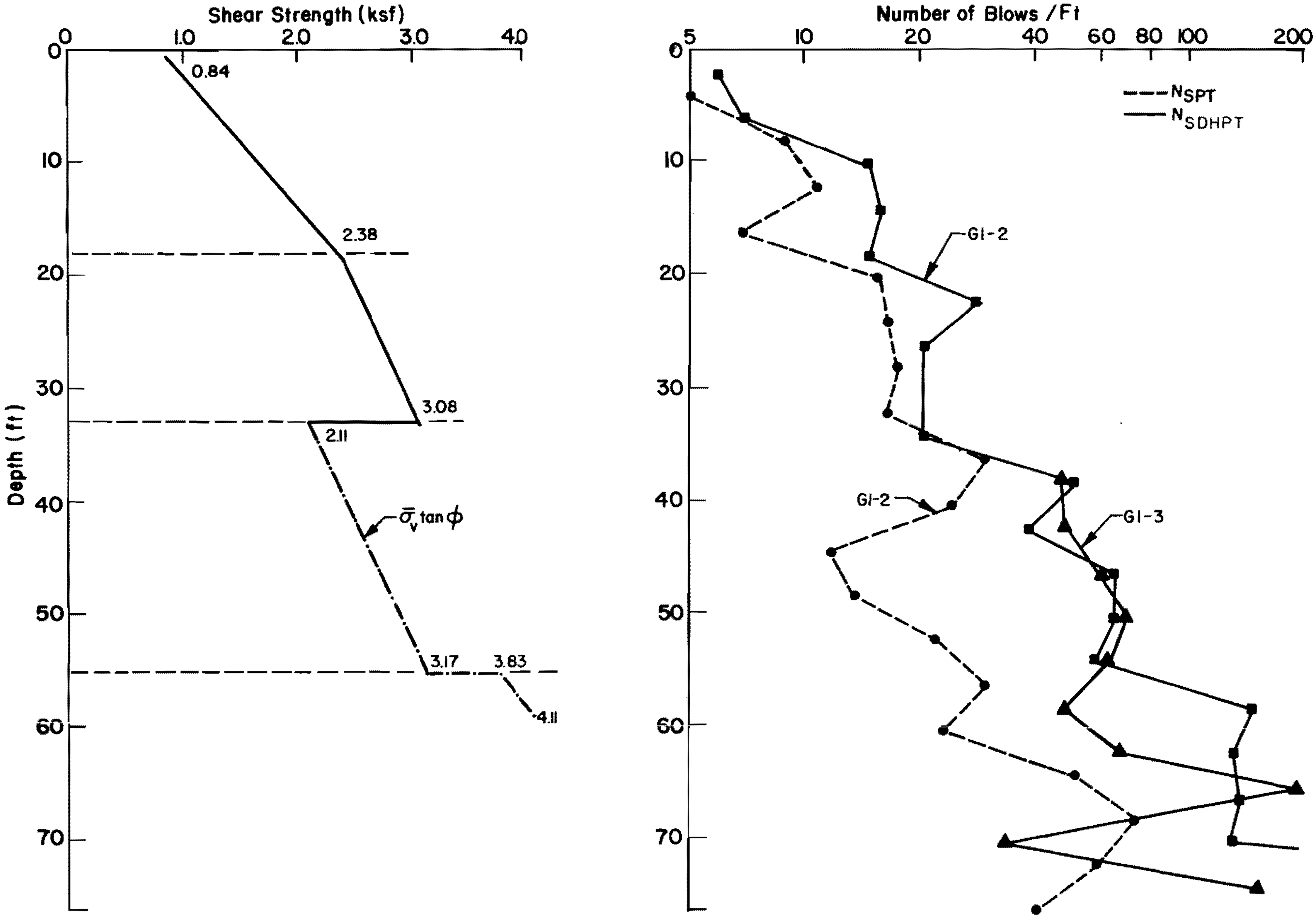


Fig. A.12 Houston G1 Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Touma and Reese, 1972)

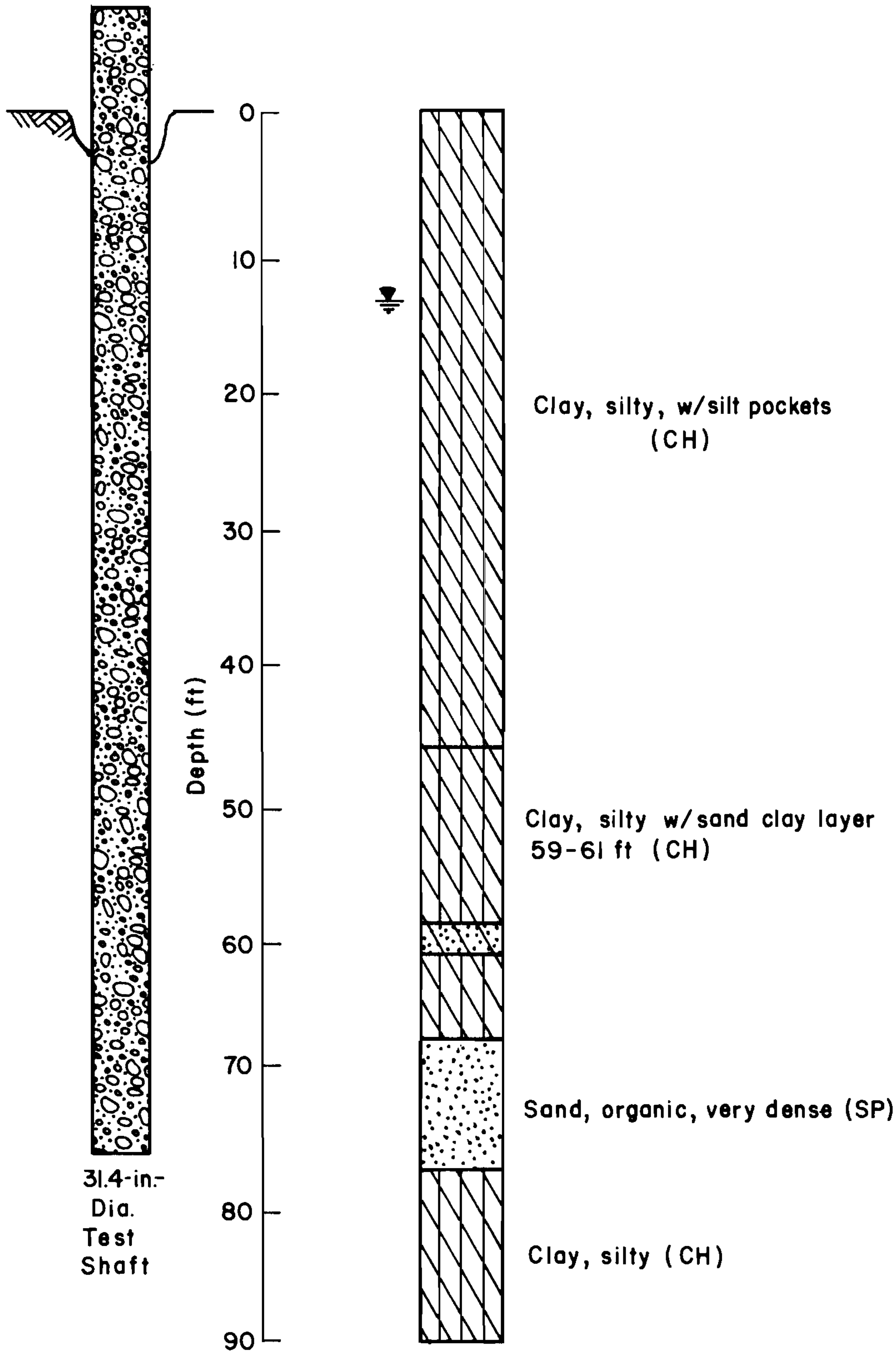


Fig. A.13 Houston G2 Test, Soil Profile and Test Shaft (after Touma and Reese, 1972)

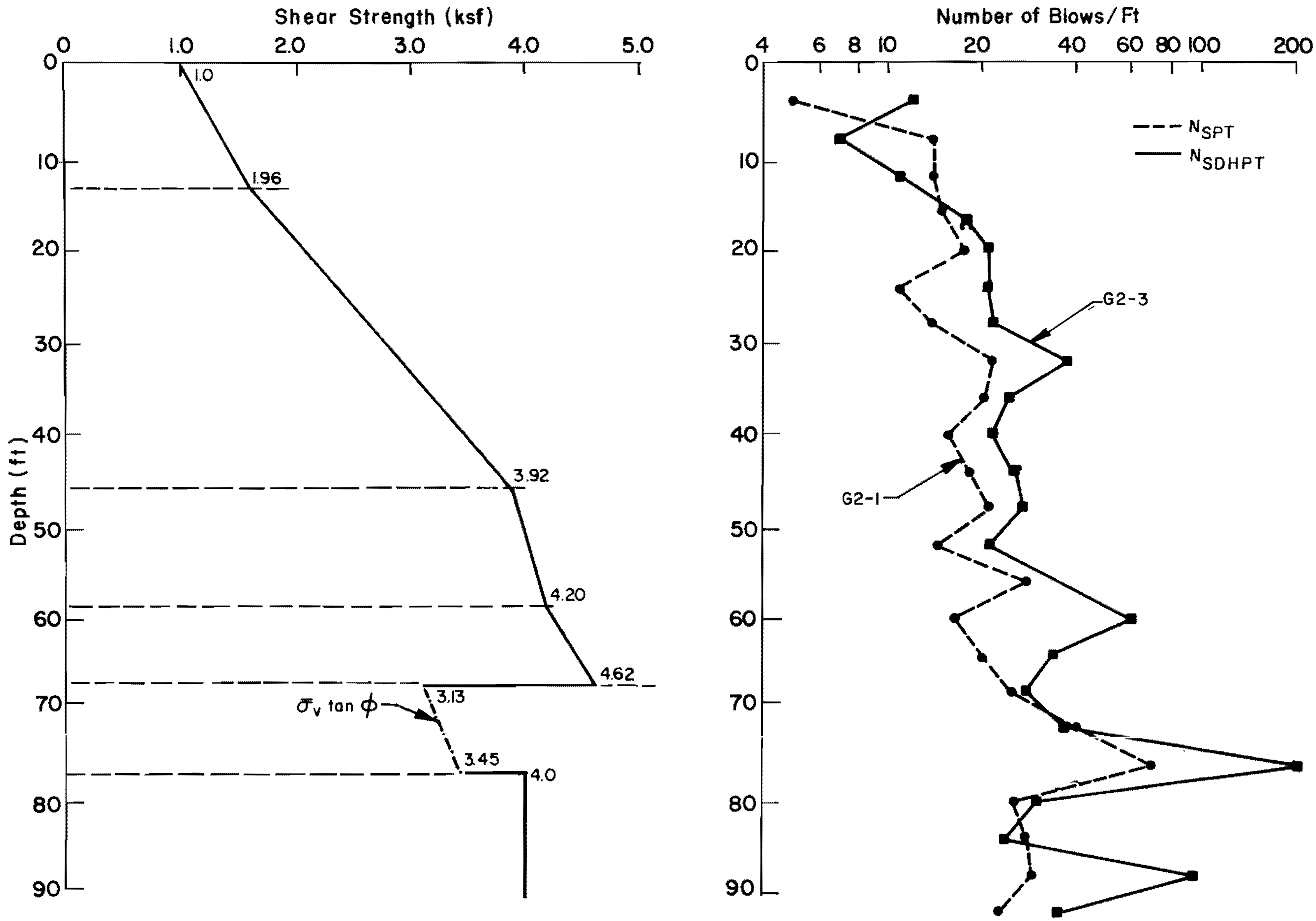


Fig. A.14 Houston G2 Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Touma and Reese, 1972)

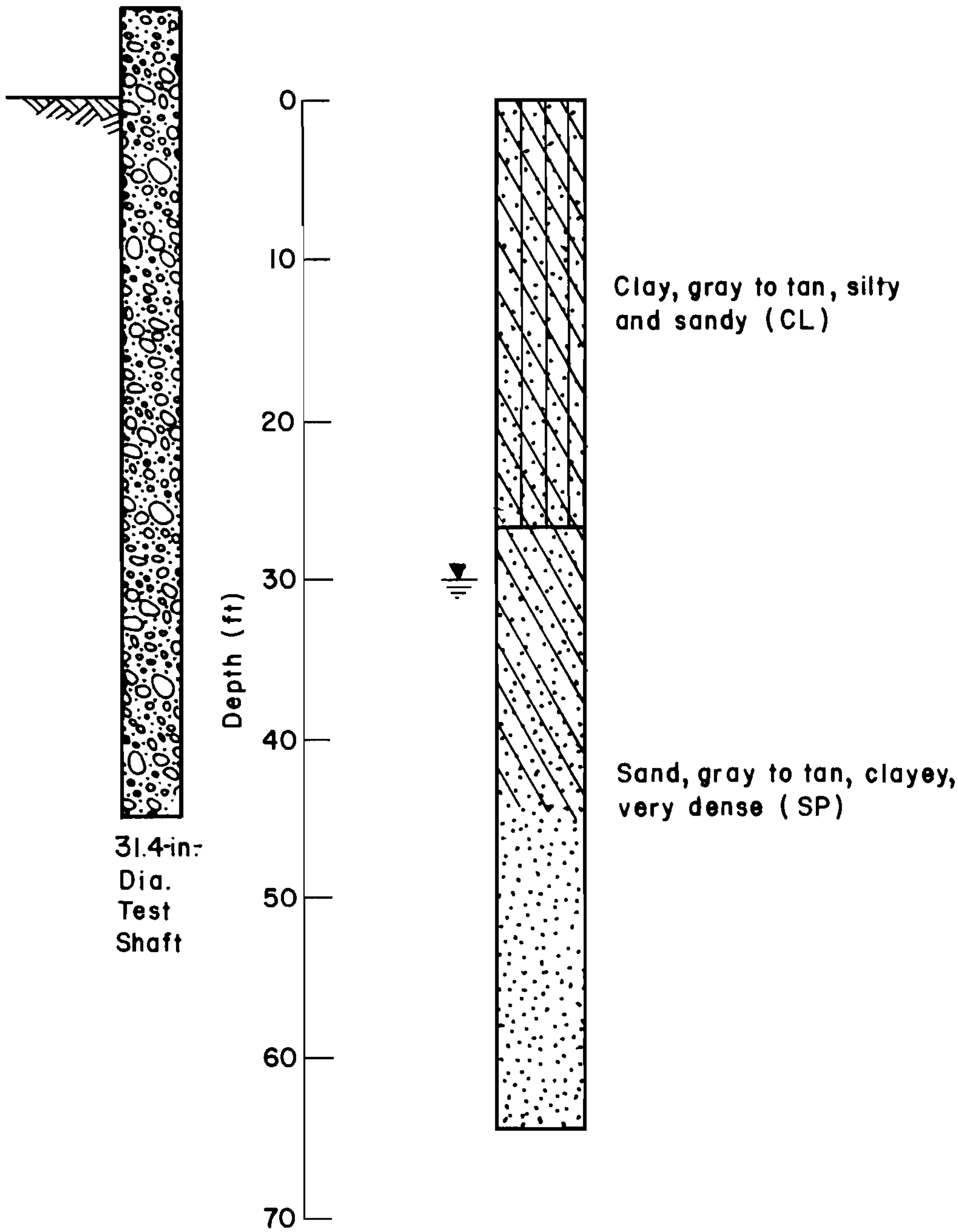
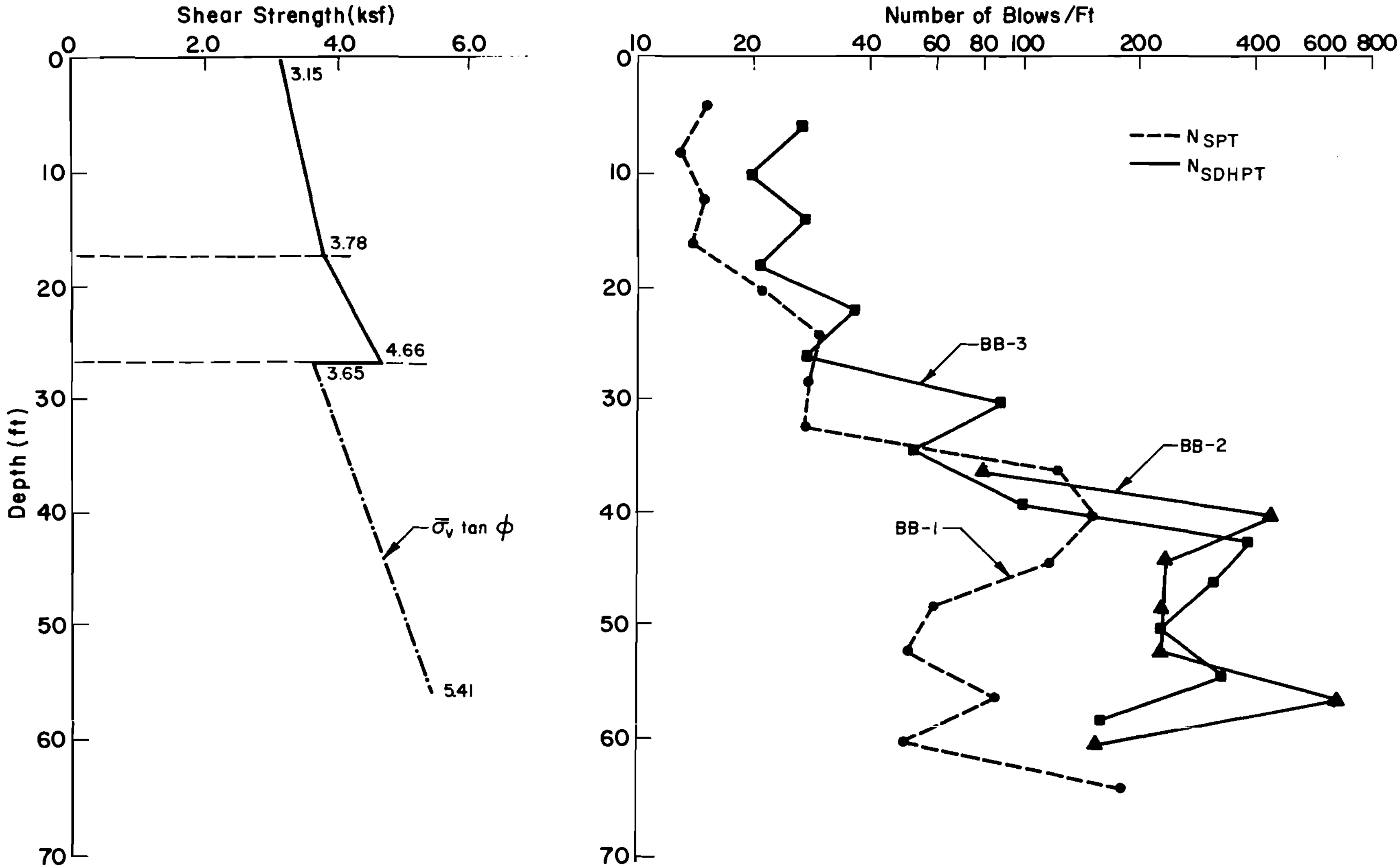


Fig. A.15 Houston BB Test, Soil Profile and Test Shaft (after Touma and Reese, 1972)



A.16 Houston BB Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Touma and Reese, 1972)

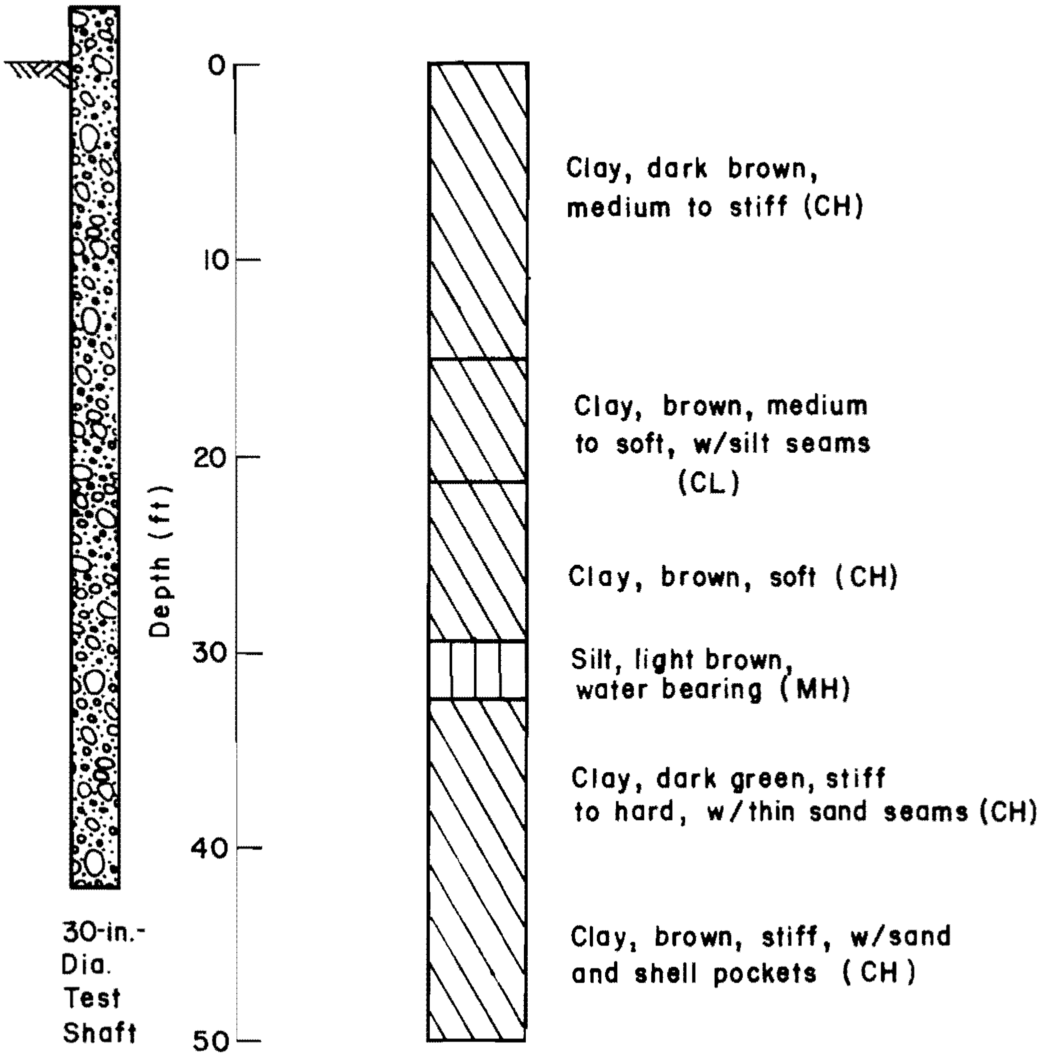


Fig. A.17 Bryan Test , Soil Profile and Test Shaft
(after Engeling and Reese, 1974)

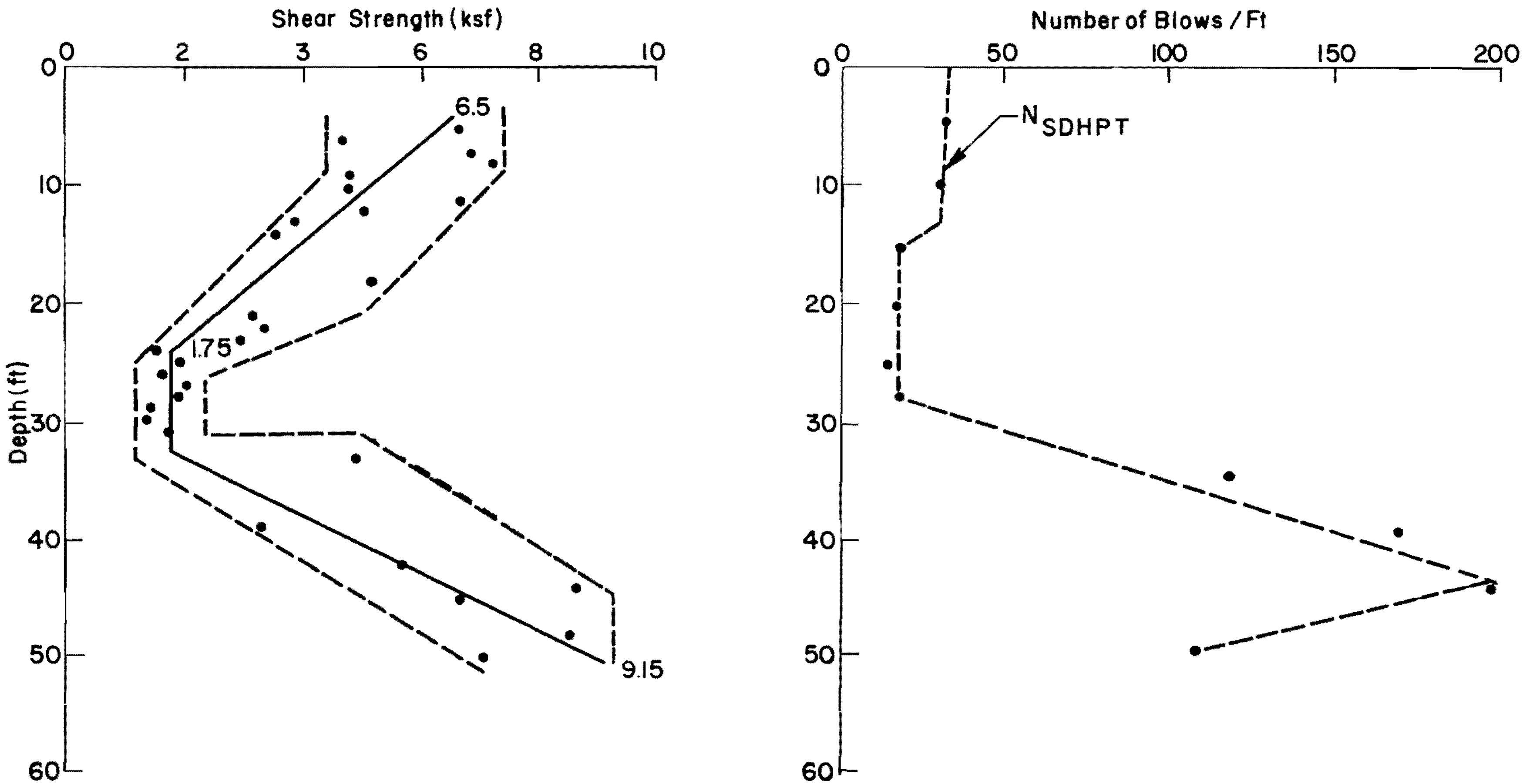


Fig. A.18 Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Engeling and Reese, 1974)

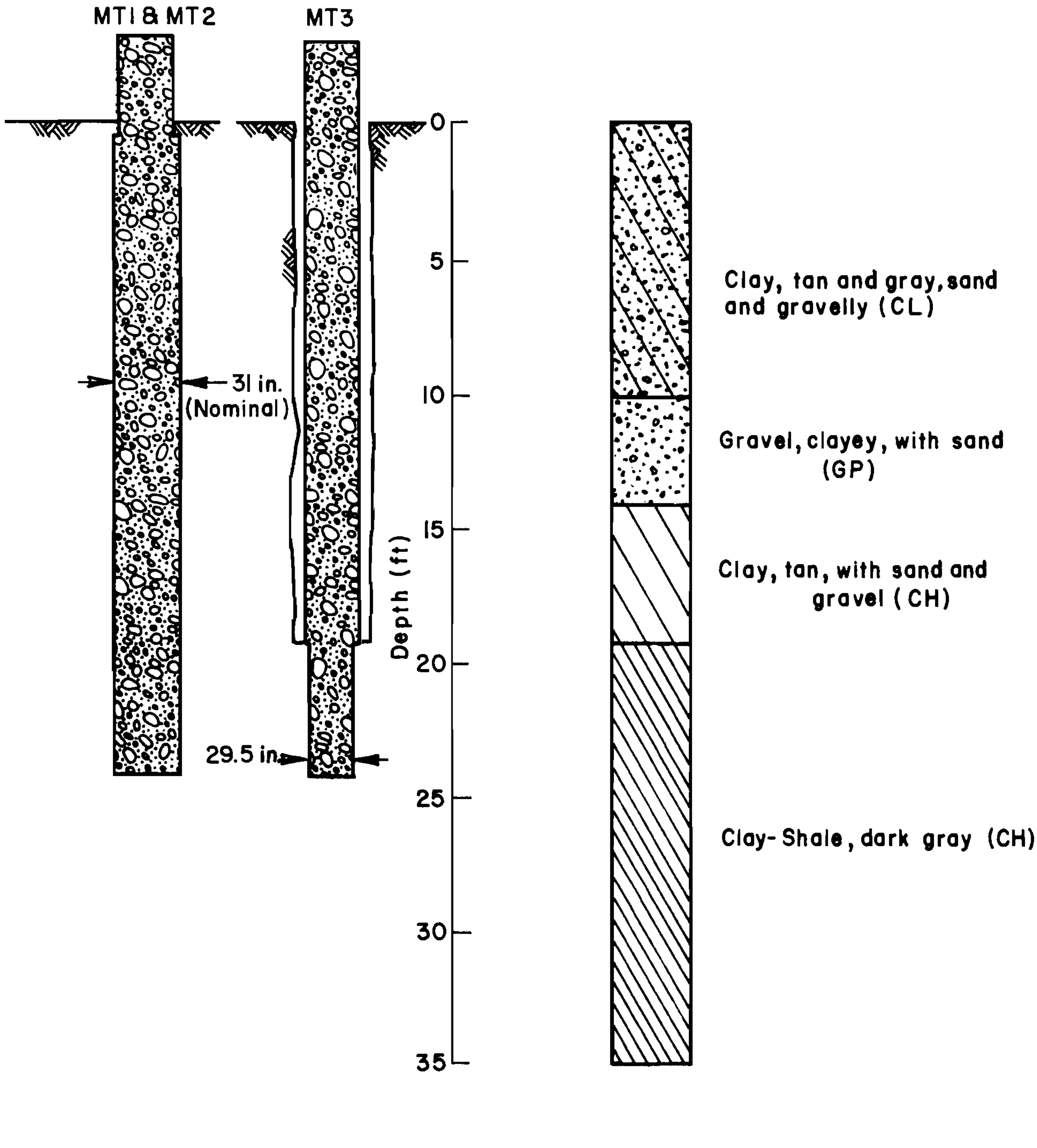


Fig. A.19 Montopolis MT1, MT2, and MT3 Tests ; Soil Profile and Test Shafts (after Aurora and Reese, 1976)

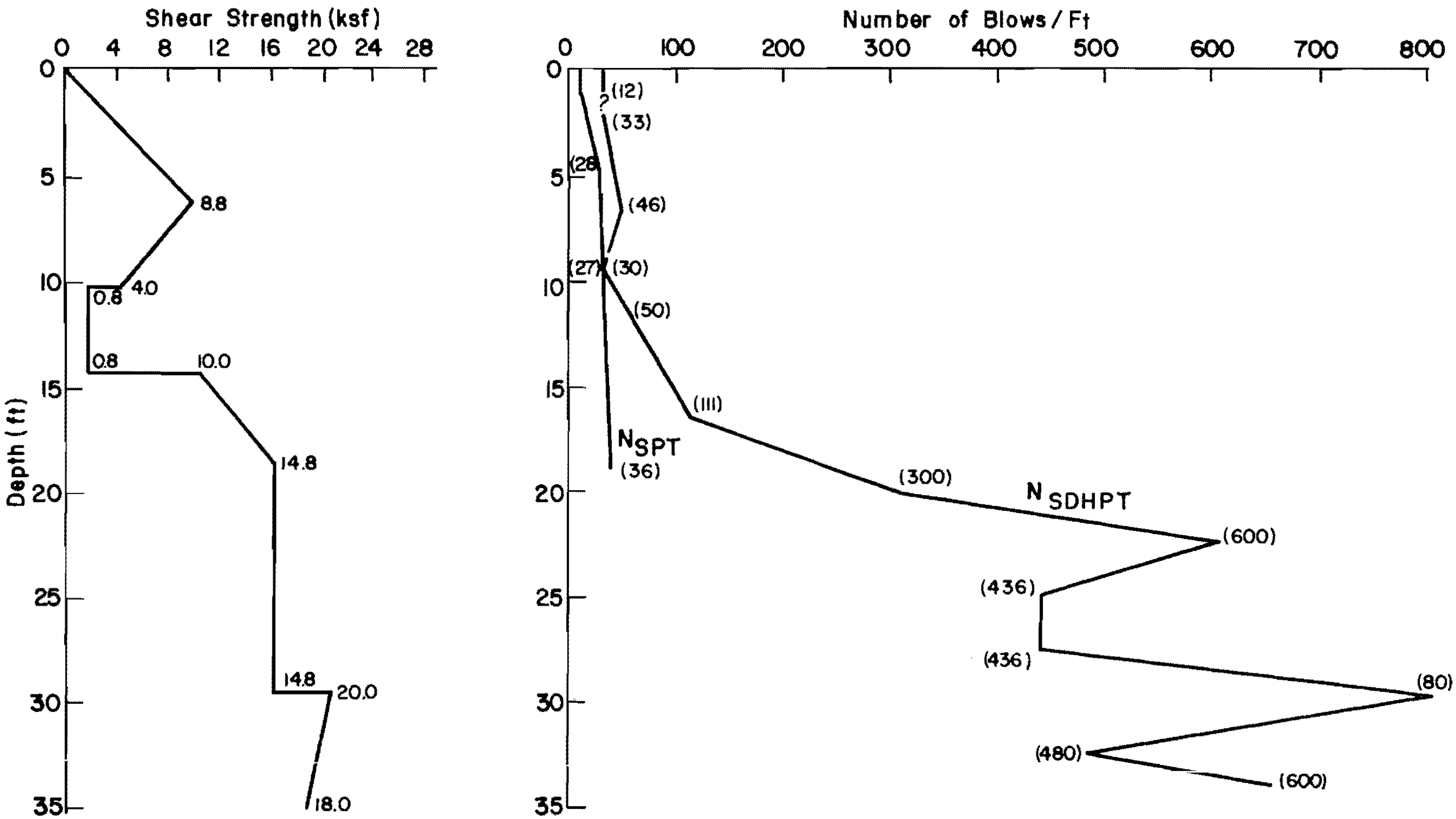


Fig. A.20 Montopolis MT1, MT2 and MT3 Tests; Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Aurora and Reese, 1976)

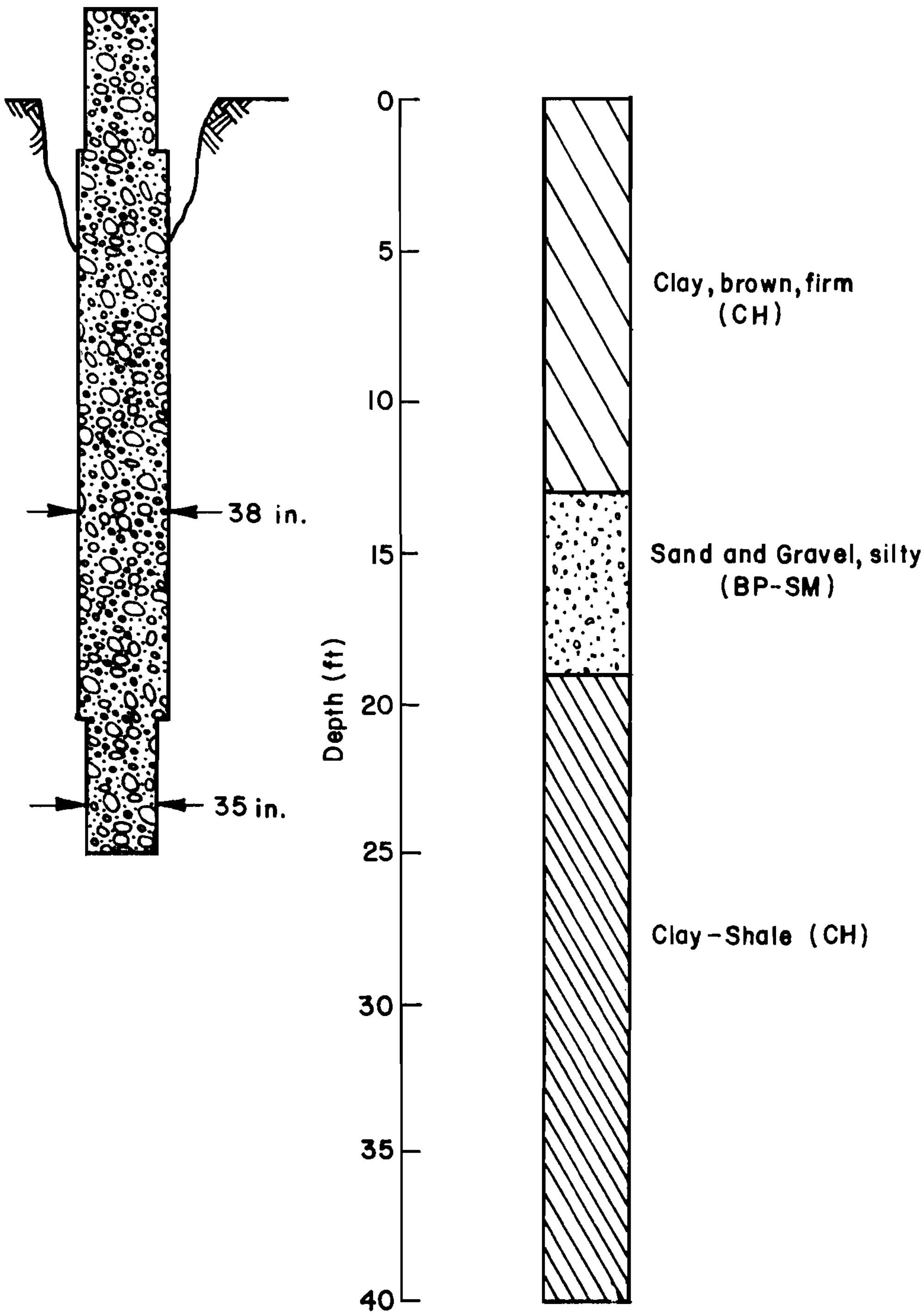


Fig. A.21 Dallas DT1 Test, Soil Profile and Test Shaft
(after Aurora and Reese, 1976)

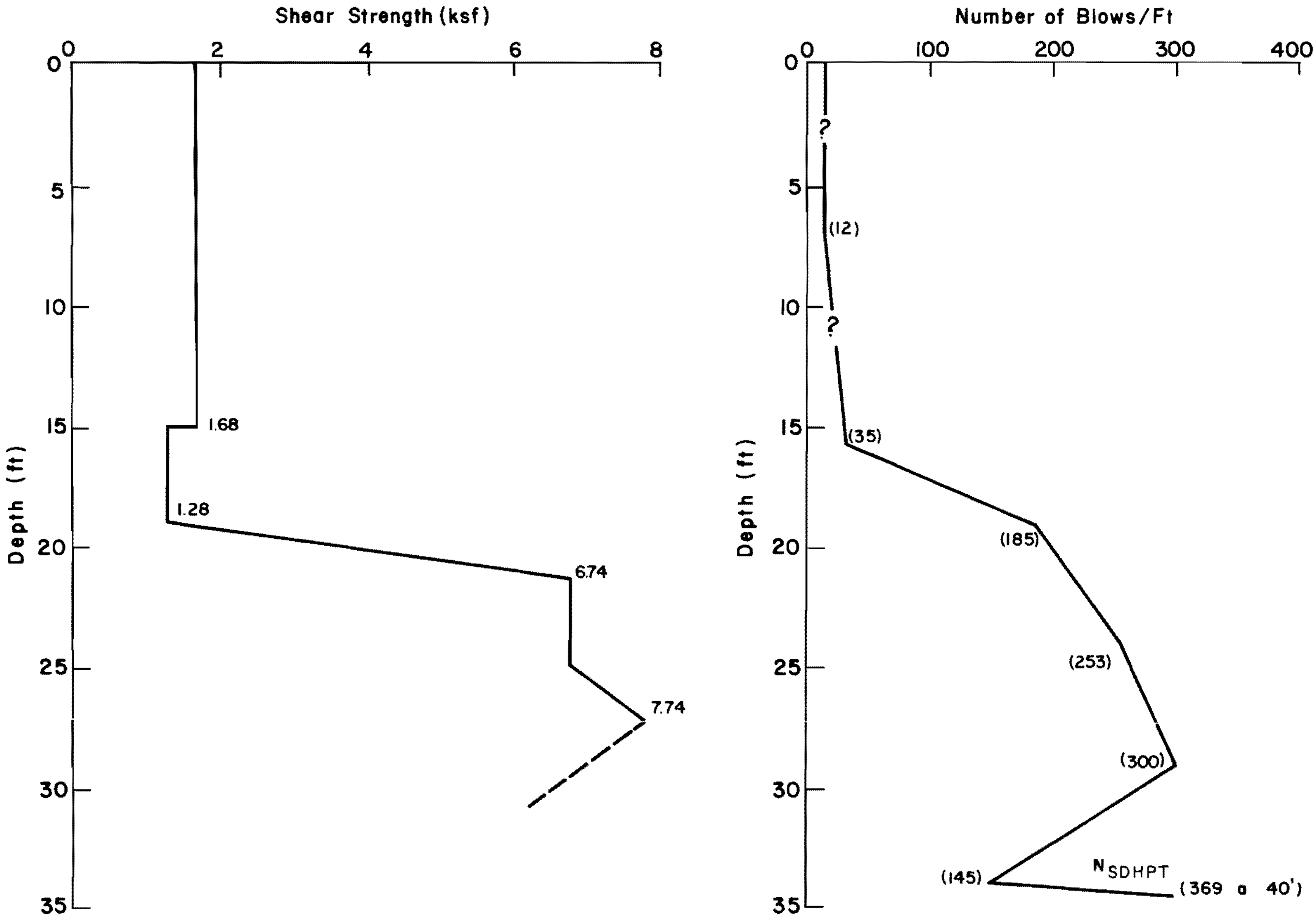


Fig A.22 Dallas DT1 Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Aurora and Reese, 1976)

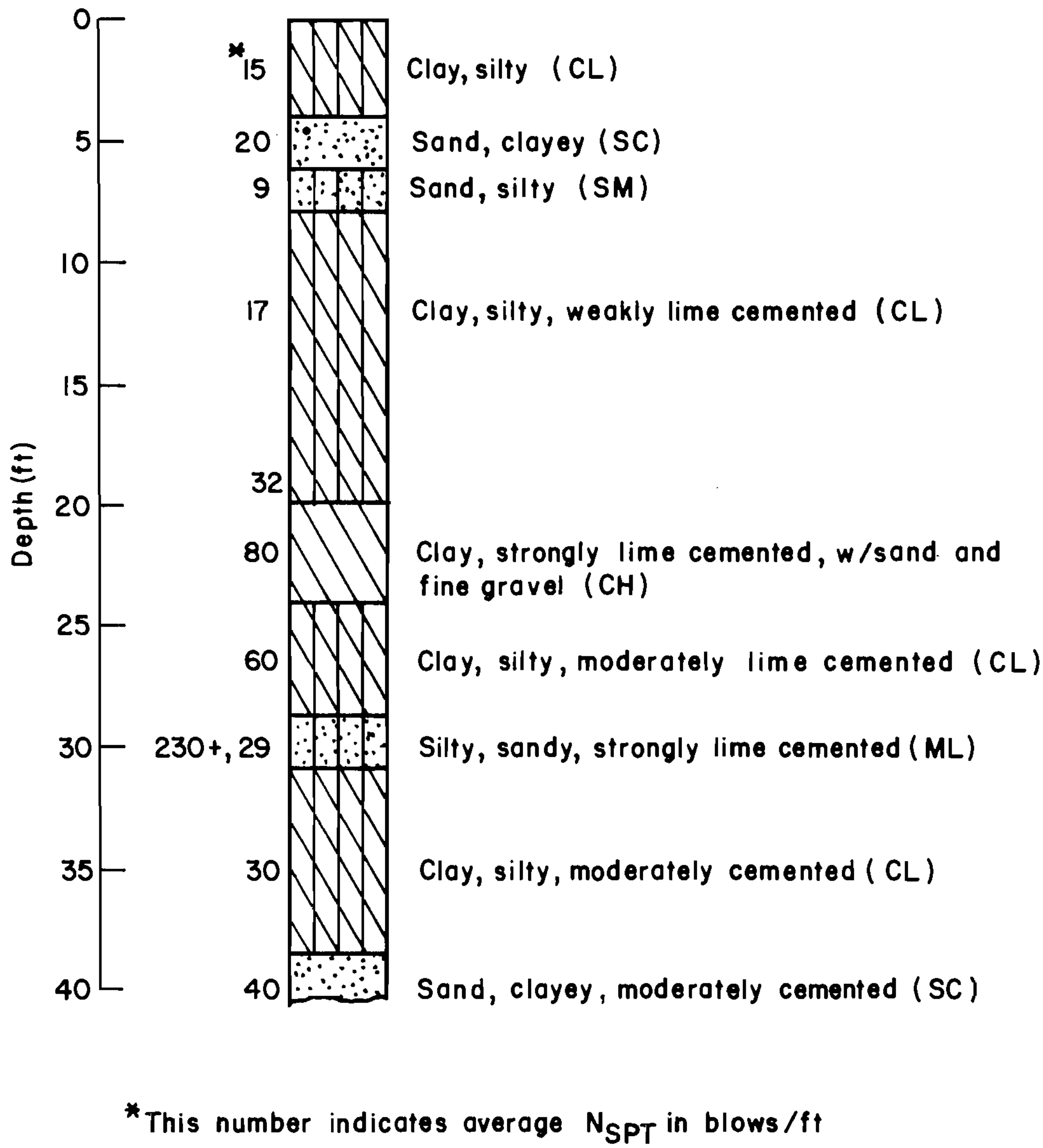


Fig. A.23 Arizona Site B Tests, Soil Profile
(after Beckwith and Bedenkop, 1973)

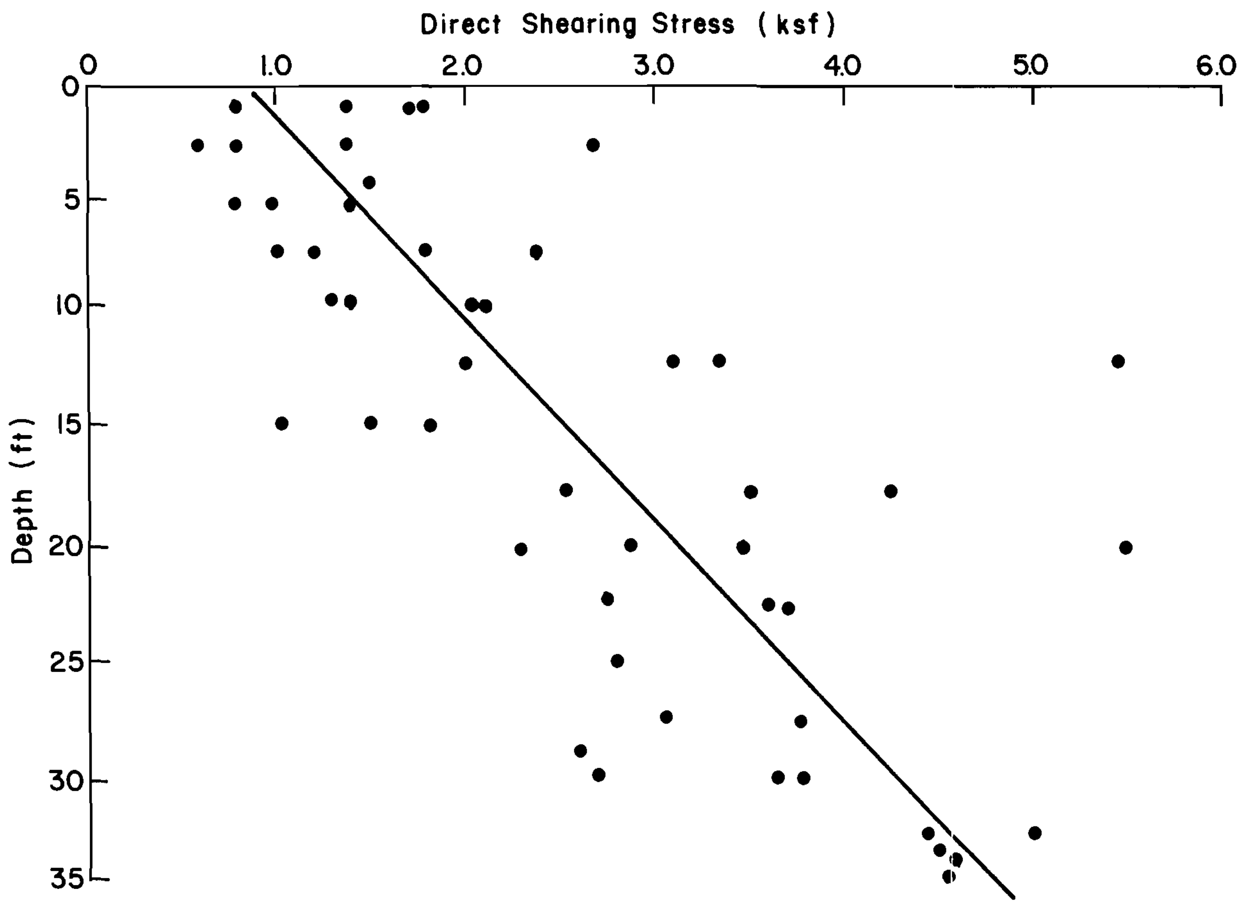


Fig. A.24 Arizona Site B Tests, Shear Strength Data Used in Analysis (after Beckwith and Bedenkop, 1973)

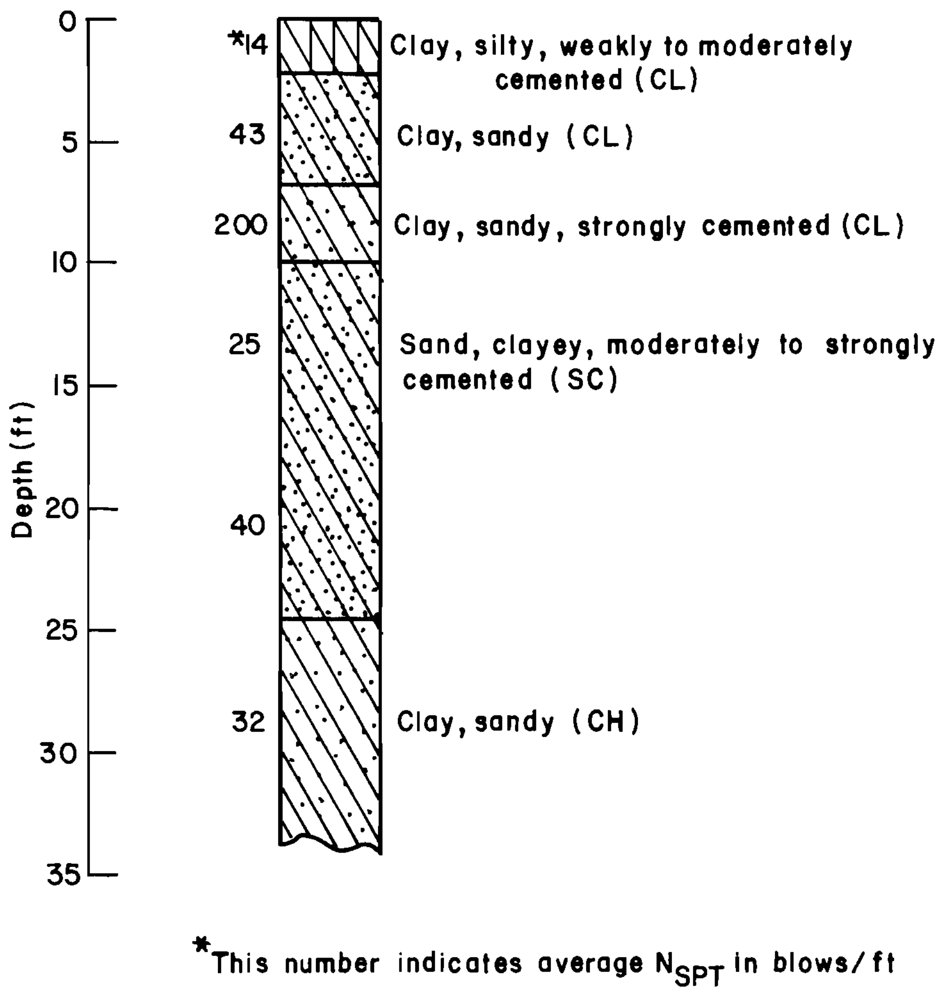


Fig. A.25 Arizona Site C Tests, Soil Profile (after Beckwith and Bedenkop, 1973)

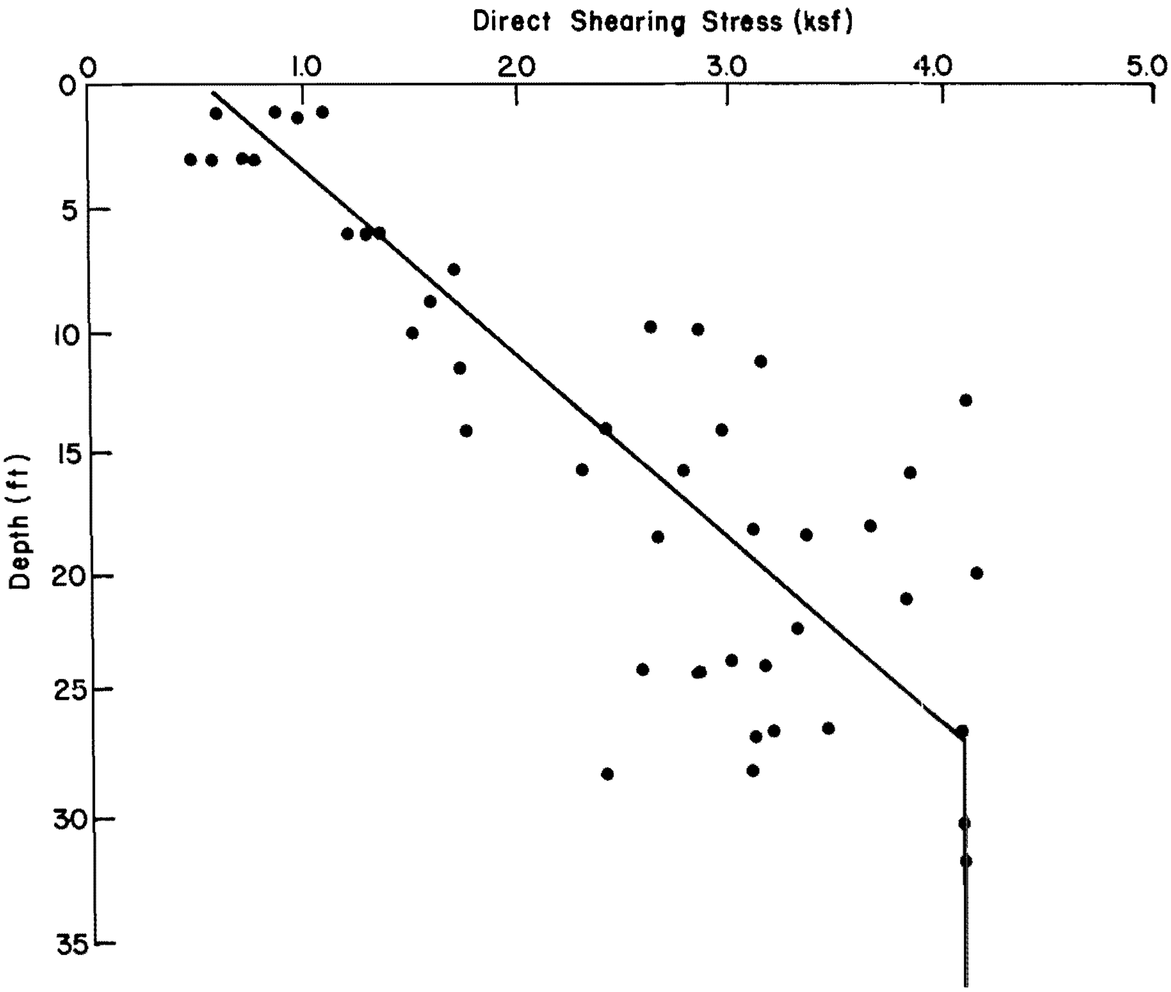


Fig. A.26 Arizona Site C Tests, Shear Strength Data Used in Analysis (after Beckwith and Bedenkop, 1973)

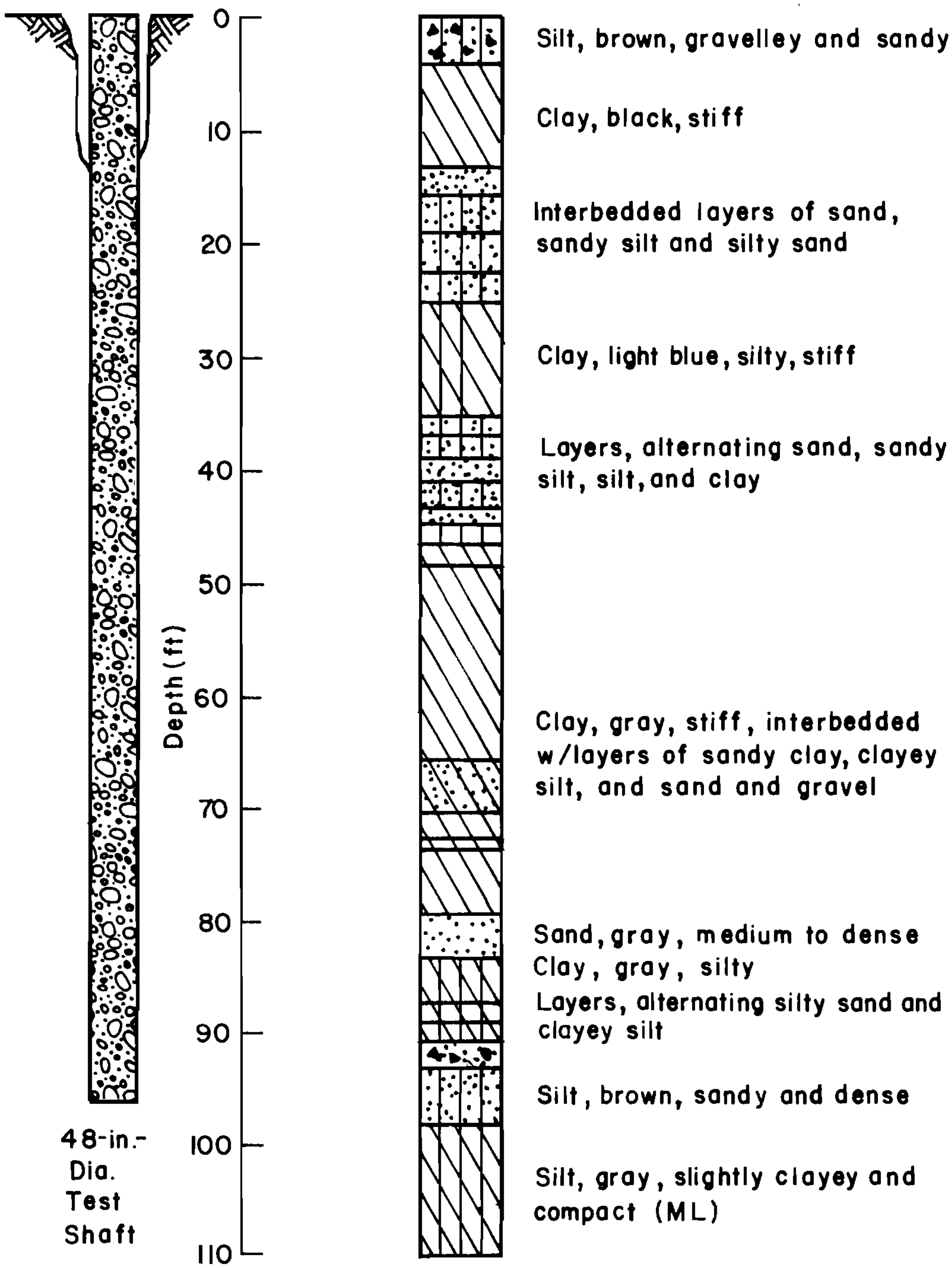


Fig. A.27 California Test, Soil Profile and Test Shaft (after Wilhelms, 1975)

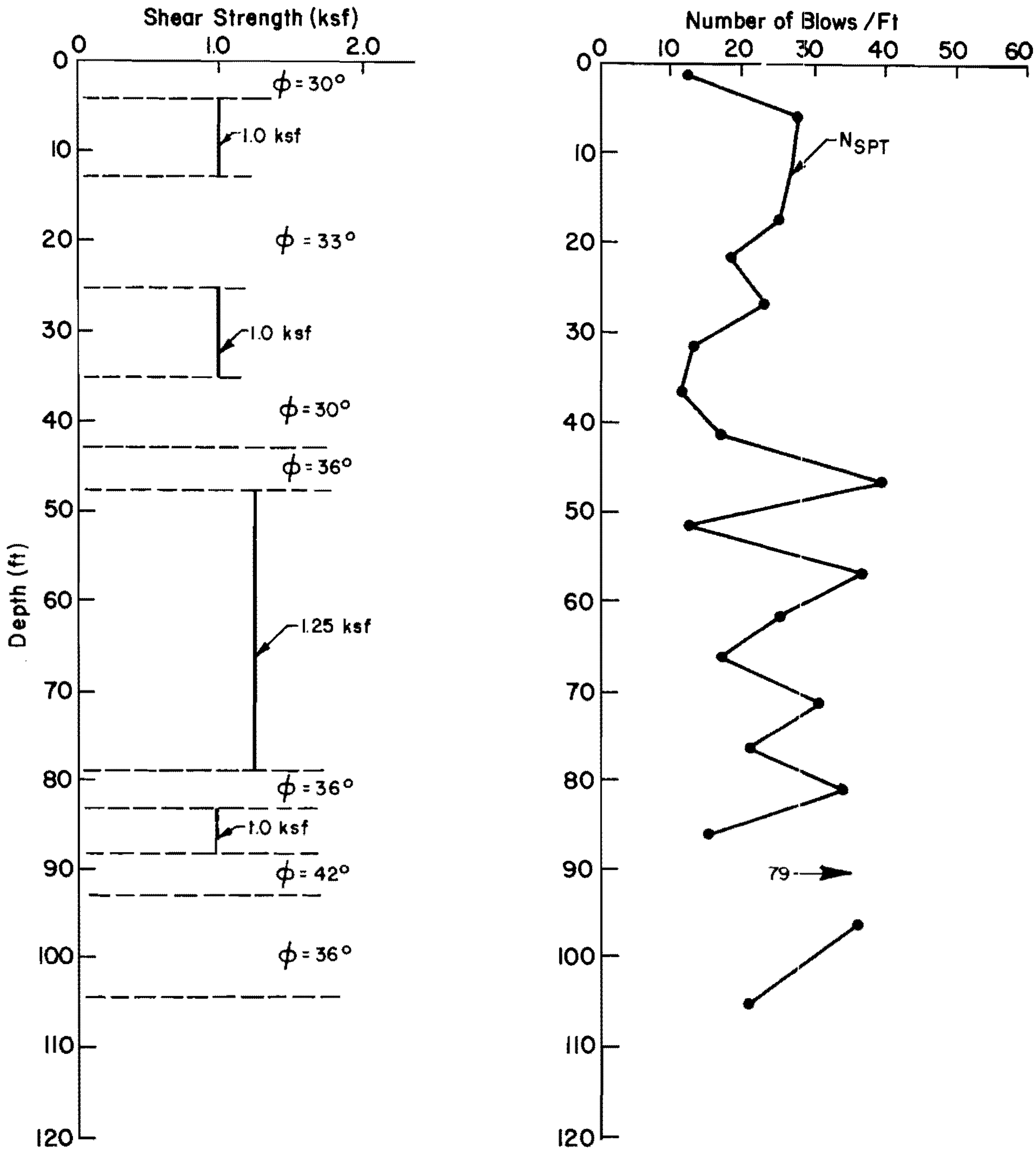


Fig. A.28 California Test, Shear Strength and Dynamic-Penetrometer Data Used in Analysis (after Wilhelms, 1975)

TABLE A.1 SUMMARY OF ARIZONA PRESSUREMETER TEST RESULTS, SITE B
(After Beckwith and Bedenkop, 1973)

Boring Location	Depth to Center (feet)	P_o (kg/cm^2)	P_f (kg/cm^2)	P_L (ksf)	E (ksf)	S_o (ksf)
1	3	3.84	5.46	12.37	327	0.68
4		1.19	6.76	18.19	520	2.79
1	6	3.43	8.20	21.46	351	2.72
2		3.26	7.28	20.95	1269	2.09
4		2.20	6.47	16.57	284	2.29
1	9	3.70	7.17	18.29	277	1.99
2		3.02	5.67	10.65	513	0.63
1	12	2.82	5.97	13.13	189	1.37
4		0.42	4.34	12.78	260	2.29
2	15	2.34	4.12	8.50	250	0.57
3		2.23	4.03	9.32	319	2.31
1	18	3.20	9.03	25.09	1441	2.76
2		2.65	8.76	33.05	923	4.85
3		1.77	4.22	20.36	1358	2.48
4		0.77	4.83	18.84	508	3.11
1	21	4.44	14.57	56.20	1397	8.07
3		1.72	13.90	62.53	1616	10.22
4		2.86	15.97	72.99	2124	11.02
1	24	5.53	22.26	81.31	3209	11.53
2		2.86	7.46	35.70	1170	4.96
3		5.64	21.42	91.42	3369	13.39
4	27	2.20	5.48	32.60	1168	4.73
1	30	1.58	16.41	54.03	601	10.40
2		3.30	7.44	31.99	1372	4.03
4		1.01	4.39	14.77	393	2.27
1	33	2.71	13.80	56.93	881	10.57
2		3.27	5.97	28.67	1075	3.58
4		2.33	8.80	31.33	516	5.30
1	36	2.55	12.80	60.62	818	11.86

LEGEND:

P_o = Initial Pressure, the beginning of the elastic stress range (sealing pressure)

P_f = Creep Pressure, the end of the elastic stress range

P_L = Limit Pressure, the failure pressure

E = Compression Modulus, derived from the slope of the compression curve between P_o and P_f

S_o = Shear Strength

TABLE A.2 SUMMARY OF ARIZONA PRESSUREMETER TEST RESULTS, SITE C
(After Beckwith and Bedenkop, 1973)

Boring Location	Depth to Center (feet)	P_o (kg/cm^2)	P_f (kg/cm^2)	P_L (ksf)	E (ksf)	S_o (ksf)
4	3	3.24	8.77	35.84	653	5.63
4	6	2.10	4.66	19.09	264	3.01
4	12	6.95	15.93	53.45	535	8.58
4	15	6.71	14.46	59.23	2382	7.31

<u>LEGEND</u>:

P_o = Initial Pressure, the beginning of the elastic stress range (sealing pressure)

P_f = Creep Pressure, the end of the elastic stress range

P_L = Limit Pressure, the failure pressure

E = Compression Modulus, derived from the slope of the compression curve between P_o and P_f

S_o = Shear Strength

APPENDIX B

COMPUTER PROGRAMS SHAFT1 and BSHAFT

SHAFT1 LISTING

```
      PROGRAM SHAFT1(INPUT,OUTPUT)
      DIMENSION HEADNG(50),SUB(20),TRANLM(30)
      DIMENSION ALPHA(30),PHI(30),GAMMA(30),SUTOP(30),SUBOT(30)
      DIMENSION CUTOP(30),CUBOT(30),CNC(30),DEPTH(30),SOIL(30)
      COMMON/SAME/   SUX,DEPTH,X,I,M,SUB,DELTA,MDELTA,H,MH,LONG,FACT,
     1ABASE,QBA,LTEMP,QBATEM,SOIL,DIAMB,BL,PHI,N,CNC,NACT,FINAL,
     2STAR,BS,TOPLEN,CUTOP,CUBOT
C*********************************************************************
C**** THIS PROGRAM IS BASED ON THE DESIGN CRITERIA ESTABLISHED IN
C**** CFHR REPORT NO. 176-3.
C**** ALPHA *** THE CORRELATION FACTOR BETWEEN THE SHEAR STRENGTH OF
C****           THE STRATUM TO THE SHEAR RESISTANCE DEVELOPED IN THE
C****           IN THE STRATUM.
C**** BELANG * THE ANGLE OF THE BELL (DEGREES) WITH RESPECT TO VERTICAL
C**** BS ****** THE BOTTOM PORTION OF THE SHAFT TO BE IGNORED(IN FEET)
C**** CUBOT *** UNDRAINED COHESION AT THE BOTTOM BOUNDARY OF
C****           A SOIL LAYER(IN PSF)
C**** CUTOP *** UNDRAINED COHESION AT THE TOP BOUNDARY OF
C****           A SOIL LAYER(IN PSF)
C**** DEPTH *** THE DEPTH TO THE BOTTOM OF EACH SOIL LAYER(FEET)
C**** DIALIM ** THE LARGEST DIAMETER (FEET) TO BE CONSIDERED
C**** DIAM **** THE DIAMETER (FEET) OF THE STEM
C**** DSTART ** THE FIRST VALUE DIAM IS TO ASSUME
C****           DIAM IS INCREASED IN INCREMENTS OF 0.5 FT.
C**** FSB ***** THE FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE
C****           CAPACITY, QB
C**** FST ***** THE FACTOR OF SAFETY APPLIED TO THE TOTAL ULTIMATE
C****           CAPACITY, QU, WHICH RESULTS IN  QDN
C**** GAMMA *** TOTAL UNIT WEIGHT (PCF). THE PROGRAM MAKES THE NECES
C****           SARY CHANGES BELOW THE WATER TABLE
C**** NHEAD *** THE NUMBER OF CARDS IN HEADING
C**** HEADNG * DESCRIPTION OF THE DESIGN
C**** N ******* NUMBER OF SOIL LAYERS
C**** CNC ***** THE BEARING CAPACITY FACTOR
C**** NTYPDS * SPECIFIES THE TYPE OF DESIGN CALCULATIONS DESIRED
C**** P ******* THE TOTAL DESIGN LOAD DESIRED (LBS.)
C**** PHI ***** ANGLE OF INTERNAL FRICTION OF THE SOIL LAYER(IN DEGREES)
C**** QB ****** THE ULTIMATE BASE RESISTANCE. AN ASTERISK BESIDE
C****           THIS VALUE SIGNIFIES THAT THE RESISTANCE WAS
C****           CALCULATED CONSIDERING TWO OR MORE SOIL LAYERS
C****           THAT ARE WITHIN TWO BASE DIAMETERS FROM THE BASE
C**** QBD ***** THE TOTAL DESIGN LOAD USING A FACTOR OF SAFETY
C****           OF  FSB  APPLIED TO THE ULTIMATE BASE RESISTANCE
C**** QCHECK ** EFFICIENCY COMPUTATIONS ARE ELIMINATED FOR COMPUTED
C****           LOADS LESS THAN QCHECK  (LBS.)
C**** QDN ***** TOTAL DESIGN RESISTANCE WITH A FACTOR OF SAFETY
C****           OF FST APPLIED TO  QU
C**** QS ****** ULTIMATE SIDE RESISTANCE
C**** QU ****** TOTAL ULTIMATE RESISTANCE
C**** RATIO *** THIS IS THE RATIO OF THE DIAMETER OF THE BASE
C****           TO THE DIAMETER OF THE STEM
C**** SOIL **** THE TYPE OF SOIL IN A GIVEN LAYER - (1)
C****           STANDS FOR SAND AND  (2) STANDS FOR CLAY
C**** TOPLEN * TOP PORTION OF THE SHAFT TO BE IGNORED(FEET)
C**** TRANLM * THE MAXIMUM LOAD TRANSFER ALLOWED WITHIN
C****           A GIVEN SOIL LAYER  (PSF)
C**** WTD ***** DEPTH OF THE WATER TABLE
C**** NOTE - ONE BLANK CARD IS REQUIRED TO END THE PROGRAM
C*********************************************************************
   61 READ 23,NHEAD
   23 FORMAT(I1)
```

```
      IF(NHEAD.EQ.0)GO TO 96
      PRINT 28
   28 FORMAT(1H1)
C**** READ COMMENTS **********
      DO 27 IT=1,NHEAD
      READ 25,(HEADNG(IH),IH=1,8)
   25 FORMAT(8A10)
      PRINT 26,(HEADNG(IH),IH=1,8)
   26 FORMAT(1X,8A10)
      QV2=0. $ QV1=0.
   27 CONTINUE
      READ  1000,NTYPDS,DSTART,DIALIM,RATIO,BELANG,TOPLEN,BS
 1000 FORMAT(I1,9X,6F10)
      READ 2,N,WTD,P,QCHECK,FST,FSB
    2 FORMAT(I2,8X,F10,2F20,2F10)
      PRINT 20, P
   20 FORMAT(///,1X*DESIGN LOAD =*F10.0*LBS.*//)
      PRINT 4,QCHECK
    4 FORMAT(1X*MAKE COMPARISONS OF QU/VOLUME*/1X
     1*FOR DESIGN LOADS GREATER THAN *2X,F10.0*LBS.*//)
      P=P/2000.
      QCHECK=QCHECK/2000.
      PRINT 5,N
    5 FORMAT(1X*NUMBER OF LAYERS =*I5//)
      PI = 3.142
      N=N+1
      PRINT 9, WTD
    9 FORMAT(1X*WATER TABLE DEPTH =*F10.0* FT.*//)
C**** READ SOIL INFORMATION **********
 3001 DO 500 I = 2,N
      READ  11,ALPHA(I),CNC(I),GAMMA(I),PHI(I),CUTOP(I),CUBOT(I),
     1TRANLM(I),SOIL(I),DEPTH(I)
   11 FORMAT(2F5,7F10)
  500 CONTINUE
      DEPTH(1)=0.0
      OVERP1=0. $ OVERP2=0.
      DO 1976 I=2,N
      OVERP1=OVERP2
      PHI(I)=PHI(I)/57.29
      IF(WTD.LT.DEPTH(I))OVERP2=(GAMMA(I)-62.4)*(DEPTH(I)-DEPTH(I-1))
     1+OVERP1
      IF(WTD.GE.DEPTH(I))OVERP2=GAMMA(I)*(DEPTH(I)-DEPTH(I-1))+OVERP1
      SUTOP(I)=OVERP1*TAN(PHI(I))+CUTOP(I)
      SUBOT(I)=OVERP2*TAN(PHI(I))+CUBOT(I)
      PHI(I)=PHI(I)*57.29
 1976 CONTINUE
 3000 DIAM=DSTART
  503 NRATIO=RATIO
      PRINT 1001,NTYPDS,DSTART,DIALIM,RATIO,FST,FSB
 1001 FORMAT(///1X*NTYPDS=*I5/1X*DSTART =*F5.2,2X*FT.*/1X*DIALIM =*F5.2
     1/1X*RATIO  =*F5.2/1X*FST     =*F5.2/1X*FSB     =*F5.2//)
      PRINT 10
   10 FORMAT(1X*ALPHA*7X*NC*7X*GAMMA(PCF)*7X*PHI(DEGREES)*7X
     1*CUTOP(PSF)*7X*CUBOT(PSF)*7X*TRANLM(PSF)*7X*SOIL*7X*DEPTH(FT.)*
     1//)
      DEPTH(1)=0.0
      PRINT 12,DEPTH(1)
   12 FORMAT(114X,F15.1/)
      DO 501 I=2,N
      PRINT 13,ALPHA(I),CNC(I),GAMMA(I),PHI(I),CUTOP(I),CUBOT(I),
     1TRANLM(I),SOIL(I),DEPTH(I)
   13 FORMAT(F5.2,F11.2,F14.0,F17.0,F19.0,F17.0,F19.0,F12.0//114X,
```

```
     1F15.1/)
 501 CONTINUE
1004 CONTINUE
C**** DETERMINE BASE DIAMETER **********
      SQDIAM=DIAM*DIAM
      AREAST=PI*SQDIAM/4.
      IF(NRATIO.EQ.0)GO TO 2010
      DIAMB=DIAM*RATIO
      SDIAMB=DIAMB*DIAMB
      ABASE=SDIAMB*PI/4.
      IF(DIAM.LE.3.52)TL=0.250
      IF(DIAM.GT.3.52)TL=0.50
      BELANG=BELANG/57.29
      BL=TL+((DIAMB-DIAM)/(2.*TAN(BELANG)))
      BELANG=BELANG*57.29
      VOLBL=((PI*(BL-TL)*(SQDIAM/4.+SDIAMB/4.+SQRT(DIAMB*DIAM/4.))
     1/3.)+(ABASE*TL))/27.
      GO TO 2015
2010 DIAMB=DIAM
      ABASE = PI *DIAMB*DIAMB/4.
      BL=0.
      VOLBL=0.
2015 CONTINUE
1005 SUX=0.
      QBATEM=0.
      OVERP=0.
      SQS=0.SQSI=0.SQBA=0.
      FINAL=0.
      I=1
      ASIDE=PI*DIAM
      FACT=2.*DIAMB
      NACT=IFIX(FACT)
      STEEL1=(.01*PI*DIAM**2/4.)*144.
      PRINT 21,DIAM,DIAMB,BL,BELANG,TOPLEN,BS,STEEL1,VOLBL
   21 FORMAT(1H1,*DIAMETER OF STEM          =*F5.2,2X*FT.*/1X
     1*DIAMETER OF BASE          =*F6.2,2X*FT.*/1X*END OF STEM TO BASE
     1   =*F6.2,2X*FT.*/1X*ANGLE OF BELL          =*F5.1,
     11X*DEGREES*/1X*IGNORED TOP PORTION      =*F5.2,1X*FT.*/1X*IGNORED
     1BOTTOM PORTION   =*F5.2,1X*FT.*/1X*AREA OF ONE PERCENT STEEL=*
     1F6.2,1X*SQ.IN.*/1X*VOLUME OF UNDERREAM      =*F6.2* CU.YDS.*//)
      PRINT 81
   81 FORMAT(1X*ESTIMATED*/1X*SHAFT LENGTH*6X*VOLUME*13X*QS*13X*QB*13X
     1*QU*12X*QBD*12X*QDN*13X*QU/VOLUME*/3X*(FEET)*9X*(CU.YDS.)*
     29X*(TONS)*9X*(TONS)*9X*(TONS)*9X*(TONS)*9X*(TONS)*9X
     3*(TONS/CU.YDS.)*)
C**** IF ONE NEEDS MORE SOIL INFORMATION THAN FURNISHED, END PROGRAM ***
   50 IF(I.EQ.(N+1))GO TO 800
      IF((DEPTH(I)-TOPLEN).LE.0.0)GO TO 70
      LAYER = (DEPTH(I) - H)
C***********************************************************************
C**** DETERMINING THE TYPE OF SOIL,SAND OR CLAY,TO MAKE SIDE
C**** RESISTANCE COMPUTATIONS
C***********************************************************************
      DO 30 L=1,LAYER
      X=L
      IF(ALPHA(I).LE.0.0)QSI=SQS
      IF(ALPHA(I).LE.0.0)GO TO 200
      SUX=SUTOP(I)+((SUBOT(I)-SUTOP(I))/(DEPTH(I)-DEPTH(I-1)))*(X-.50)
      TRANS=ALPHA(I)*SUX
      IF(TRANS.GT.TRANLM(I))TRANS=TRANLM(I)
      QSI=(TRANS*ASIDE/2000.)+SQS
  200 DELTA=X+BS+BL+2.*DIAMB
```

```
      M=I
      STAR=0.
      MDELTA=DELTA
C**** DETERMINING THE BASE RESISTANCE IF THE SECTION IS IN CLAY ******
      IF(MDELTA.LE.LAYER.AND.SOIL(I).EQ.2.)CALL SUATB1
      IF(MDELTA.LE.LAYER.AND.SOIL(I).EQ.2.)GO TO 55
C**** DETERMINING THE BASE RESISTANCE IF THE SECTION IS IN SAND ******
C**********************************************************************
      IF(MDELTA.LE.LAYER)CALL PHIATB
      IF(MDELTA.LE.LAYER)GO TO 55
      CALL SUATB2
      IF((I+1).EQ.(N+1))GO TO 800
      IF(FINAL.EQ.1.)GO TO 800
   55 Y=H+X+BS+BL
      NY=Y
      YN=NY
      IF((Y-YN).GE.0.50)YESTIM=YN+1.
      IF((Y-YN).LT.0.50)YESTIM=YN
      VOLY=VOLBL+(YESTIM-BL)*AREAST/27.
      QS = QSI
      QB = QBA
      QU=QSI+QBA
      QDN=QU/FST
      QBD=QS+QB/FSB
      QV2=QU/VOLY
      IF(STAR.EQ.1.)GO TO 90
      PRINT 91,YESTIM,VOLY,QS,QB,QU,QBD,QDN,QV2
   91 FORMAT(1X,F6.0,2F17.2,4F15.2,F17.2)
      GO TO 717
   90 PRINT 92,YESTIM,VOLY,QS,QB,QU,QBD,QDN,QV2
   92 FORMAT(1X,F6.0,2F17.2,F15.2,1H*,F14.2,2F15.2,F17.2)
  717 CONTINUE
C**** FIND THE LARGEST VALUE OF QU/VOLUME **********
      IF(QV2.LT.QV1)GO TO 821
      IF(QDN.LT.QCHECK)GO TO 821
      SAVEL=YESTIM$SAVED=DIAM$SAVEDB=DIAMB$SAVQV2=QV2
      SAVEV=VOLY$SAVEQU=QU
      QV1=QV2
  821 CONTINUE
      IF(QBD.GT.P.AND.QBD.LE.QDN)GO TO 800
      IF(QDN.GT.P)GO TO 800
      SQS = QSI
   30 CONTINUE
C**** SELECT A DEEPER LAYER OF SOIL **********
   70 I = I+1
      H=DEPTH(I-1)
      IF((DEPTH(I-1)-TOPLEN).LE.0.)H=TOPLEN
      GO TO 50
  800 CONTINUE
      DIAM=DIAM+0.5
      IF(NTYPDS.EQ.1.AND.DIAM.GT.(DIALIM+.02))PRINT 1346,SAVED,
     1SAVEDB,SAVEL,SAVEV,SAVEQU,SAVQV2
 1346 FORMAT(///*THE MOST EFFICIENT SHAFT FOR THIS SOIL PROFILE APPEARS
     1TO BE THE FOLLOWING ONE:*//1X*DIAMETER OF STEM      =*F5.2,1X*FT.*
     2//1X*DIAMETER OF BASE      =*F5.2,1X*FT.*//1X*ESTIMATED SHAFT LENG
     3TH=*F5.0,1X*FT.*//1X*VOLUME OF CONCRETE    =*F7.2,1X*CU.YDS.*//1X
     4*ULTIMATE LOAD CAPACITY=*F7.2,1X*TONS*//1X
     5*QU/VOLUME OF CONCRETE =*F7.2,1X*TONS/CU.YD.*)
      IF(DIAM.GT.(DIALIM+.02).AND.NTYPDS.EQ.1)GO TO 61
      IF(DIAM.GT.(DIALIM+.02))GO TO 6666
      GO TO 1004
C**** COME HERE IF THE SAME SOIL PROFILE IS TO BE USED
```

```
C**** BUT WITH THE FOLLOWING NEW VARIABLES ***********
 6666 NTYPLD=NTYPDS
      READ 1000,NTYPDS,DSTART,DIALIM,RATIO,BELANG,TOPLEN,BS
      IF(NTYPLD.EQ.2)GO TO 3000
      READ 6667,(ALPHA(J),J=2,N)
 6667 FORMAT(16F5)
      GO TO 3000
   96 CONTINUE
      END
      SUBROUTINE SUATB1
      DIMENSION         DEPTH(30),SUB(20),PHI(30),CNC(30),
     1SOIL(30),CUTOP(30),CUBOT(30)
      COMMON/SAME/   SUX,DEPTH,X,I,M,SUB,DELTA,MDELTA,H,MH,LONG,FACT,
     1ABASE,QBA,LTEMP,QBATEM,SOIL,DIAMB,BL,PHI,N,CNC,NACT,FINAL,
     2STAR,BS,TOPLEN,CUTOP,CUBOT
C***********************************************************************
C**** THIS SUBROUTINE CALCULATES THE BASE RESISTANCE
C**** IN A HOMOGENEOUS SOIL LAYER ***********
C***********************************************************************
      LONG=H+X+BS+BL
      IF(DEPTH(I-1).EQ.0.)DEPTH(I-1)=TOPLEN
      MD1=DEPTH(I-1)
      MD2=DEPTH(I)
      D=LONG-MD1
      SUBT=0.
      DO 36 JLM=1,NACT
      TJ=JLM
      DEPT=TJ+D-.5
      SUBT=SUBT+(CUTOP(I)+((CUBOT(I)-CUTOP(I))/(DEPTH(I)-DEPTH(I-1)))*
     1DEPT)/FACT
   36 CONTINUE
   13 QBA=(CNC(I)*SUBT*ABASE)/2000.
      RETURN
      END
      SUBROUTINE SUATB2
      DIMENSION         DEPTH(30),SUB(20),PHI(30),CNC(30),
     1SOIL(30),CUTOP(30),CUBOT(30)
      COMMON/SAME/   SUX,DEPTH,X,I,M,SUB,DELTA,MDELTA,H,MH,LONG,FACT,
     1ABASE,QBA,LTEMP,QBATEM,SOIL,DIAMB,BL,PHI,N,CNC,NACT,FINAL,
     2STAR,BS,TOPLEN,CUTOP,CUBOT
C***********************************************************************
C**** THIS SUBROUTINE FINDS THE BEARING CAPACITY.  IT TAKES INTO ACCOUNT
C**** THE CASES WHERE THE BOTTOM OF THE SHAFT IS IN ONE TYPE OF SOIL
C**** BUT IS CLOSE ENOUGH TO A LOWER LAYER WHICH MAY AFFECT THE BEARING
C**** CAPACITY. A STAR  IN THE OUTPUT DISTINGUISHES  A BEARING CAPACITY
C**** CALCULATED WHICH CONSIDERS  TWO OR MORE LAYERS OF SOIL
C***********************************************************************
      LONG=H+X+BS+BL
      IF(DEPTH(I-1).EQ.0.)DEPTH(I-1)=TOPLEN
      MH=H
   20 IF((I+1).EQ.(N+1))RETURN
      MDPTH1=DEPTH(I+1) $ KDEPTH=DEPTH(I)
      IF(MDPTH1.LE.LONG)I=I+1
      IF(MDPTH1.LE.LONG)GO TO 20
      IF(MDPTH1.LT.(LONG+NACT))GO TO 18
      IF(KDEPTH.GT.LONG)GO TO 3
      I=I+1
      IF(SOIL(I).NE.1.)GO TO 17
      CALL PHIATB
      I=M
      RETURN
   17 CALL SUATB1
```

```
      I=M
      RETURN
   18 IF((I+2).NE.(N+1))GO TO 19
      FINAL=1.0
      RETURN
   19 MDPTH2=DEPTH(I+2)
      IF(MDPTH2.LT.(LONG+NACT))GO TO 40
   30 I=I+1
    3 CONTINUE
      IF(SOIL(I).EQ.2..AND.SOIL(I+1).EQ.1.)GO TO 21
      IF(SOIL(I).EQ.1..AND.SOIL(I+1).EQ.1.)GO TO 11
      IF(SOIL(I).EQ.1..AND.SOIL(I+1).EQ.2.)GO TO 12
C**** COME HERE IF BOTH LAYERS BEING CONSIDERED ARE CLAYS *****
   22 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   31 MDIST=DIST1
      NDIST=DIST2
      JC=0
      DO 776 JM=1,MDIST
      TJM=JM
      DEP=TJM-0.5
      SUB(JM)=CUTOP(IT1)+((CUBOT(IT1)-CUTOP(IT1))/(DEPTH(IT1)-DEPTH(IT1-
     11)))*(DEP+LONG-DEPTH(IT1-1))
      JC=JC+1
  776 CONTINUE
      DO 777 JM=1,NDIST
      TJM=JM
      DEP=TJM-0.5
      SUB(JM+MDIST)=CUTOP(IT2+1)+((CUBOT(IT2+1)-CUTOP(IT2+1))/(DEPTH(IT2
     1+1)-DEPTH(IT2)))*DEP
      JC=JC+1
  777 CONTINUE
      QBA22=0.
      DO 778 JM=1,JC
      QBA22=QBA22+(((CNC(IT1)*SUB(JM)*ABASE)/2000.)/FACT)
  778 CONTINUE
      QBA=QBA22+QBATEM
      QBATEM=0.
      JC=0
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF THE TOP LAYER IS A CLAY AND THE LOWER
C**** LAYER IS A SAND **********
   21 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   32 MDIST=DIST1
      NDIST=DIST2
      JC=0
      DO 877 JM=1,MDIST
      TJM=JM
      DEP=TJM-0.5
      SUB(JM)=CUTOP(IT1)+((CUBOT(IT1)-CUTOP(IT1))/(DEPTH(IT1)-DEPTH(IT1-
     11)))*(DEP+LONG-DEPTH(IT1-1))
      JC=JC+1
  877 CONTINUE
      I=IT2+1
      CALL PHIATB
```

```
      I=IT1
      QBA21S=QBA
      QBA21C=0.
      DO 878 JL=1,JC
      QBA21C=QBA21C+(((CNC(I)*SUB(JL)*ABASE)/2000.)/FACT)
  878 CONTINUE
      QBA=QBA21C+QBATEM+((QBA21S*DIST2)/FACT)
      QBATEM=0.
      JC=0
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF BOTH LAYERS BEING CONSIDERED ARE SAND *****
   11 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   33 I=IT1
      CALL PHIATB
      QBA11A=QBA
      I=IT2+1
      CALL PHIATB
      I=IT1
      QBA11B=QBA
      QBA=(QBA11A*DIST1+QBA11B*DIST2)/FACT+QBATEM
      QBATEM=0.
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF THE TOP LAYER IS A SAND AND THE LOWER
C**** LAYER IS A CLAY **********
   12 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   34 NDIST=DIST2
      JC=0
      I=IT1
      CALL PHIATB
      QBA12S=QBA
      DO 977 JM=1,NDIST
      TJM=JM
      DEP=TJM-0.5
      SUB(JM)=CUTOP(IT2+1)+((CUBOT(IT2+1)-CUBOT(IT2+1))/(DEPTH(IT2+1)-
     1DEPTH(IT2)))*DEP
      JC=JC+1
  977 CONTINUE
      QBA12C=0.
      DO 978 JM=1,JC
      QBA12C=QBA12C+(((CNC(IT2+1)*SUB(JM)*ABASE)/2000.)/FACT)
  978 CONTINUE
      QBA=QBA12C+QBATEM+(QBA12S*DIST1/FACT)
      QBATEM=0.
      JC=0
      I=M
      STAR=1.
       RETURN
C**** COME HERE AND CALCULATE A WEIGHTED BEARING CAPACITY
C**** FOR A LAYER OF SOIL FULLY WITHIN THE TWO DIAMETER
C**** DISTANCE BELOW THE BASE OF THE SHAFT **********
   40 I=I+1
      IT1=I
```

```
      DIST1=DEPTH(I)-LONG
   60 I=I+1
      DIFFER=DEPTH(I)-DEPTH(I-1)
      IF(SOIL(I).EQ.1.)CALL PHIATB
      IF(SOIL(I).EQ.1.)QBA=QBA*DIFFER/FACT
      IF(SOIL(I).EQ.1.)GO TO 45
       SUSHAR=(CUTOP(I)+CUBOT(I))/2.
      QBA=0.
      QBA=QBA+(((CNC(I)*SUSHAR*ABASE)/2000.)/FACT)
   45 QBATEM=QBATEM+QBA
      IF((I+1).EQ.(N+1))RETURN
      MDPTH2=DEPTH(I+1)
      IF((MDPTH2-(LONG+NACT)).GE.0)GO TO 50
      GO TO 60
   50 IT2=I
      I=I+1
      DIST2=LONG+FACT-DEPTH(IT2)
      IF(SOIL(IT1).EQ.2..AND.SOIL(I).EQ.2.)GO TO 31
      IF(SOIL(IT1).EQ.2..AND.SOIL(I).EQ.1.)GO TO 32
      IF(SOIL(IT1).EQ.1..AND.SOIL(I).EQ.1.)GO TO 33
      GO TO 34
      END
      SUBROUTINE PHIATB
      DIMENSION        DEPTH(30),SUB(20),PHI(30),CNC(30),
     1SOIL(30),CUTOP(30),CUBOT(30)
      COMMON/SAME/   SUX,DEPTH,X,I,M,SUB,DELTA,MDELTA,H,MH,LONG,FACT,
     1ABASE,QBA,LTEMP,QBATEM,SOIL,DIAMB,BL,PHI,N,CNC,NACT,FINAL,
     2STAR,BS,TOPLEN,CUTOP,CUBOT
C***********************************************************************
C**** THIS SUBROUTINE FINDS THE BEARING CAPACITY OF THE SHAFT IN SAND.
C***********************************************************************
      IF(PHI(I).LE.30.0)GO TO 10
      IF(PHI(I).LE.36.0)GO TO 20
      IF(PHI(I).LE.41.0)GO TO 30
      QBA=(ABASE/(.6*DIAMB))*40.
      RETURN
   10 QBA=0.
      RETURN
   20 QBA=(ABASE/(.6*DIAMB))*(2.667*(PHI(I)-30.0))
      RETURN
   30 QBA=(ABASE/(.6*DIAMB))*(16.+(4.8*(PHI(I)-36.0)))
      RETURN
      END
```

EXPLANATION OF INPUT FORMAT FOR SHAFT1

As a matter of convenience in explaining the input format, the data cards are broken down into two groups: set A and set B (see later discussion). The cards in set A contain information for a soil profile. The cards in set B contain information related to the soil-profile data in set A. In using both sets of cards, one can reuse the soil-profile data in set A but with the modified values that are found in set B.

To use the data found in set A just once for a series of capacity computations in a soil profile without use of set B, NTYPDS is assigned a value of one. If another soil profile is to be considered, a second set A immediately follows the first set A.

NTYPDS is assigned a value of two when the information in set A is to be used again with modified values of the variables found in Card No. 3. The modified values are furnished in set B. If a new soil profile (set A) is to be considered following the capacity computations based on the modified values, then NTYPDS in set B must be given a value of one. However, if the values of Card No. 3 are to be modified another time, then NTYPDS in set B must be given a value of two. Any number of sets B may follow any set A.

NTYPDS is given a value of three for a case similar to the one in which NTYPDS is equal to two except that the strength reduction factor, ALPHA, for every soil layer can be modified. This information is furnished in set B in the order shown later. No more than two cards of the No. 6 type are required because not more than 29 layers of soil can be considered in this program.

In order to end the program or in order to consider a new set A, the last assigned value of NTYPDS must have been one. Furthermore, another requirement for ending the program is the use of a blank card following sets A (or set B when used), Card No. 7.

Card Type No. 1. NHEAD is an integer refering to the number of cards that comprise NHEADNG. NHEAD must be less than or equal to four. If NHEAD is equal to zero, the program will end; there is one card per set A.

Card Type No. 2. Data cards of this type consist of alphanumeric information which can be used to describe a design case. No more than four cards can comprise HEADNG for each set A.

Card Type No. 3. There is only one such card for each set A. However, any information on this card can be changed by the method explained earlier in this section by assigning a proper value of NTYPDS and the appropriate information in set B.

DSTART is the first stem diameter to be considered for each set A. Units are feet.

DIALIM is the size of the diameter to be considered when capacity computations are made for more than one shaft diameter. When only one shaft size is of interest, the DIALIM is assigned a value of zero. Units are feet.

RATIO is the ratio of the base diameter to the stem diameter. In the case of straight shafts, the value of RATIO must be zero.

BELANG is the angle of the bell with respect to the vertical. In the case of straight shafts, BELANG is zero. Units are degrees.

TOPLEN is the top portion of the shaft that is noncontributing to side resistance. Units are feet.

BS is the bottom section of the shaft that has no resistance from the soil. Units are feet.

Card Type No. 4. There is only one card per set A. N is an integer refering to the number of strata in the soil profile. N must be less than or equal to 29.

WTD is the water table depth with units in feet.

P is the design load. Units are pounds.

QCHECK also has units in pounds. As discussed in Chapter 4, efficiency computations are eliminated for computed loads less than QCHECK. QCHECK should be given a suitable value to avoid making unnecessary efficiency computations for drilled shafts with small penetrations.

FST refers to the factor of safety that must be applied to the total ultimate capactiy.

FSB refers to the factor of safety that must be applied to the ultimate base capacity.

Card Type No. 5. There are as many of these cards as there are layers of soil. The data cards are arranged in order of increasing depth.

ALPHA is the strength reduction factor, α.

CNC is the bearing capacity factor, N_c. For sand layers this parameter is given a value of zero.

GAMMA is the total unit weight in pounds per cubic foot.

PHI has units of degrees. PHI refers to the undrained angle of internal friction in cohesive material and to the effective angle of internal friction in granular soil.

CUTOP and CUBOT refer to the undrained cohesion at the top and bottom, respectively, of a cohesive soil layer. Units are in pounds per square foot. For granular soil layers, CUTOP and CUBOT are assigned values of zero.

TRANLM is the maximum load transfer permissible for a given stratum. Units are in pounds per square foot.

SOIL refers to the soil type. A value of 1.0 identifies a granular soil layer and a value of 2.0 identifies a cohesive soil stratum.

DEPTH specifies the bottom depth of each layer of soil. Units are in feet.

Card Type No. 6. This card is used in set B when the previously specified strength reduction factors are to be modified without changing any of the other information in set A. Even if only one ALPHA value is to be changed, desired values for all soil layers must be included. When this type of card is used, it must be preceded by Card Type No. 3 in set B. No more than two No. 6 cards can be used because of the limit of 29 soil layers per profile.

Card Type No. 7. This data card should always be the last one of the deck and may follow a set A or B, whichever is the case. Upon reading this blank card, the computer will assign NHEAD a value of zero, thus causing the program to be terminated.

INPUT FORMAT FOR PROGRAM SHAFT1

CARD TYPE NUMBER

1 — NHEAD (VALUE MUST BE ≤ 4) — ONE CARD

NHEAD
I1
1

2 — HEADING — NUMBER OF CARDS=NHEAD

HEADING
8A10
1–80

3 — ONE CARD

NTYPDS	DSTART	DIALIM	RATIO	BELANG	TOPLEN	BS
I1	F10	F10	F10	F10	F10	F10
1	11–20	21–30	31–40	41–50	51–60	61–70

4 — ONE CARD

N	WTD	P	QCHECK	FST	FSB
I2	F10	F20	F20	F10	F10
1–2	11–20	21–40	41–60	61–70	71–80

5 — AS MANY CARDS AS THERE ARE SOIL LAYERS

ALPHA	CNC	GAMMA	PHI	CUTOP	CUBOT	TRANLM	SOIL	DEPTH
F5	F5	F10	F10	F10	F10	F10	F10	F10
1–5	6–10	11–20	21–30	31–40	41–50	51–60	61–70	71–80

(Cards 1–5: SET A)

3 — ONE CARD

NTYPDS	DSTART	DIALIM	RATIO	BELANG	TOPLEN	BS
I1	F10	F10	F10	F10	F10	F10
1	11–20	21–30	31–40	41–50	51–60	61–70

6 — ONE OR TWO CARDS, AS REQUIRED

ALPHA(1)	ALPHA(2)	ALPHA(3)	. . . .	ALPHA(N)
F5	F5	F5	. . . .	F5
1–5	6–10	11–15	16 75	76–80

(Cards 3 and 6: SET B)

7 — (BLANK CARD TO END PROGRAM) — ONE CARD

SAMPLE OUTPUT FOR PROGRAM SHAFT1

```
HOUSTON(G2) - OCT 1971 (SLURRY)
73.5 FT, PENETRATION - QU=670 TONS - QS=600 TONS  QB=70 TONS


DESIGN LOAD =   1280000LBS,


MAKE COMPARISONS OF QU/VOLUME
FOR DESIGN LOADS GREATER THAN       200000LBS,


NUMBER OF LAYERS =    7


WATER TABLE DEPTH =       13 FT,



NTYPDES=    1
DSTART = 2,62  FT,
DIALIM =-0,00
RATIO  =-0,00
FST    = 2,00
FSB    = 3,00
```

ALPHA	NC	GAMMA(PCF)	PHI(DEGREES)	CUTOP(PSF)	CUBOT(PSF)	TRANSLM(PSF)	SOIL	DEPTH(FT,)
								0,0
.60	5,00	120	0	1000	1960	4000	2	
								13,0
.60	9,00	125	0	1960	3920	4000	2	
								46,0
.60	9,00	125	0	3920	4200	4000	2	
								59,0
.60	9,00	125	0	4200	4620	4000	2	
								68,0
.50	0,00	130	32	0	0	5000	1	
								75,0
.50	0,00	135	42	0	0	5900	1	
								77,0
.60	9,00	125	0	4000	4000	4000	2	
								85,0

DIAMETER OF STEM = 2,62 FT,
DIAMETER OF BASE = 2,62 FT,
END OF STEM TO BASE = 0,00 FT,
ANGLE OF BELL = -0,0 DEGREES
IGNORED TOP PORTION = 8,00 FT,
IGNORED BOTTOM PORTION =-0,00 FT,
AREA OF ONE PERCENT STEEL= 7,76 SQ,IN,
VOLUME OF UNDERREAM = 0,00 CU,YDS,

ESTIMATED SHAFT LENGTH (FEET)	VOLUME (CU,YDS,)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU,YDS,)
9	1,80	2,56	21,34*	23,90	9,67	11,95	13.30
10	2,00	5,74	23,29*	29,04	13,51	14,52	14.54
11	2,20	9,40	24,91*	34,31	17,70	17,15	15.62
12	2,40	13,53	26,19*	39,71	22,25	19,86	16.57
13	2,60	18,13	48,82	66,95	34,40	33,47	25.79
14	2,80	23,04	50,19	73,24	39,77	36,62	26.19
15	3,00	28,10	51,57	79,67	45,29	39,84	26.60
16	3,20	33,31	52,94	86,25	50,96	43,13	26.99
17	3,39	38,66	54,32	92,98	56,77	46,49	27.39
18	3,59	44,16	55,69	99,86	62,73	49,93	27.78
19	3,79	49,81	57,07	106,88	68,83	53,44	28.17
20	3,99	55,61	58,44	114,05	75,09	57,02	28.55
21	4,19	61,55	59,82	121,36	81,49	60,68	28.94
22	4,39	67,63	61,19	128,83	88,03	64,41	29.32
23	4,59	73,87	62,57	136,44	94,72	68,22	29.70
24	4,79	80,25	63,94	144,19	101,56	72,10	30.08
25	4,99	86,78	65,32	152,09	108,55	76,05	30.46
26	5,19	93,45	66,69	160,14	115,68	80,07	30.84
27	5,39	100,27	68,07	168,34	122,96	84,17	31.22
28	5,59	107,24	69,44	176,68	130,39	88,34	31.60
29	5,79	114,35	70,82	185,17	137,96	92,59	31.97
30	5,99	121,61	72,19	193,81	145,68	96,90	32.35
31	6,19	129,02	73,57	202,59	153,54	101,29	32.72
32	6,39	136,57	74,94	211,52	161,55	105,76	33.10
33	6,59	144,27	76,32	220,59	169,71	110,30	33.47
34	6,79	152,12	77,69	229,82	178,02	114,91	33.85
35	6,99	160,12	79,07	239,19	186,47	119,59	34.22
36	7,19	168,26	80,44	248,70	195,07	124,35	34.59
37	7,39	176,54	81,82	258,36	203,82	129,18	34.97
38	7,59	184,98	83,20	268,17	212,71	134,09	35.34
39	7,79	193,56	84,57	278,13	221,75	139,06	35.71
40	7,99	202,29	85,95	288,23	230,93	144,12	36.08
41	8,19	211,16	87,32	298,48	240,27	149,24	36.45
42	8,39	220,18	88,61*	308,79	249,72	154,39	36.82
43	8,59	229,35	89,72*	319,07	259,26	159,53	37.16
44	8,79	238,66	90,66*	329,32	268,88	164,66	37.48
45	8,99	248,12	91,42*	339,54	278,60	169,77	37.78
46	9,19	257,73	92,00	349,74	288,40	174,87	38.07
47	9,39	267,44	92,50	359,94	298,27	179,97	38.35
48	9,59	277,20	93,00	370,20	308,20	185,10	38.62
49	9,79	287,01	93,50	380,51	318,18	190,26	38.89
50	9,99	296,88	94,00	390,88	328,21	195,44	39.15
51	10,18	306,80	94,50	401,30	338,30	200,65	39.40
52	10,38	316,77	95,00	411,77	348,44	205,89	39.65
53	10,58	326,80	95,50	422,30	358,63	211,15	39.90
54	10,78	336,88	95,99	432,87	368,88	216,44	40.14
55	10,98	347,01	96,55*	443,56	379,20	221,78	40.38
56	11,18	357,20	97,22*	454,42	389,61	227,21	40.63
57	11,38	367,44	98,01*	465,45	400,11	232,73	40.89

58	11.58	377.73	98.92*	476.65	410.70	238.33	41.15
59	11.78	388.08	99.94	488.02	421.39	244.01	41.42
60	11.98	398.51	101.02	499.53	432.18	249.76	41.69
61	12.18	409.05	102.10	511.16	443.09	255.58	41.96
62	12.38	419.71	103.18	522.90	454.11	261.45	42.23
63	12.58	430.49	104.26	534.75	465.24	267.38	42.50
64	12.78	441.38	88.17*	529.55	470.77	264.78	41.43
65	12.98	452.39	71.03*	523.41	476.06	261.71	40.32
66	13.18	463.51	53.67*	517.17	481.40	258.59	39.24
67	13.38	474.74	36.09*	510.83	486.77	255.42	38.18
68	13.58	486.10	18.30	504.39	492.19	252.20	37.14
69	13.78	492.57	18.30	510.87	498.67	255.43	37.07
70	13.98	499.14	18.30	517.43	505.24	258.72	37.01
71	14.18	505.79	46.43*	552.22	521.27	276.11	38.95
72	14.38	512.53	69.13*	581.66	535.57	290.83	40.45
73	14.58	519.36	123.26*	642.61	560.44	321.31	44.08
74	14.78	526.27	115.59*	641.86	564.80	320.93	43.43
75	14.98	533.27	107.93*	641.20	569.25	320.60	42.81
76	15.18	543.48	100.27*	643.76	576.91	321.88	42.42
77	15.38	553.84	92.61	646.45	584.71	323.22	42.04
78	15.58	563.71	92.61	656.32	594.58	328.16	42.13
79	15.78	573.59	92.61	666.20	604.46	333.10	42.23
80	15.98	583.47	92.61	676.08	614.34	338.04	42.32

THE MOST EFFICIENT SHAFT FOR THIS SOIL PROFILE APPEARS TO BE THE FOLLOWING ONE:

DIAMETER OF STEM = 2.62 FT.

DIAMETER OF BASE = 2.62 FT.

ESTIMATED SHAFT LENGTH= 73 FT.

VOLUME OF CONCRETE = 14.58 CU.YDS.

ULTIMATE LOAD CAPACITY= 642.61 TONS

QU/VOLUME OF CONCRETE = 44.08 TONS/CU.YD.

BSHAFT LISTING

```
      PROGRAM BSHAFT(INPUT,OUTPUT)
      DIMENSION HEADNG(50),SIDRES(30),BRESIS(30),TRANLM(30)
      DIMENSION  ALPHA(30),BLOWS(30),SOIL(30),DEPTH(30)
      COMMON/SAME1/DEPTH,X,M,DELTA,MDELTA,H,MH,LONG,FACT,QBATEM,
     1SOIL,N,NACT,FINAL,STAR,J,BS,TOPLEN,SIDRES,BRESIS,I,QBA,
     2ABASE,DIAMB,BL
C***********************************************************************
C**** ALPHA *** THE CORRELATION FACTOR BETWEEN THE SHEAR STRENGTH
C****           OF THE STRATUM TO THE SHEAR RESISTANCE DEVELOPED
C****           IN THE STRATUM.
C**** BELANG * THE ANGLE OF THE BELL (DEGREES) WITH RESPECT TO VERTICAL
C**** BLOWS *** THE NUMBER OF BLOWS/FT. OBTAINED FROM THE T.H.D.
C****           DYNAMIC PENETRATION TEST OR THE S.P.T. FOR EACH
C****           LAYER OF SOIL
C**** BS ****** THE BOTTOM PORTION OF THE SHAFT TO BE IGNORED
C****           (IN FEET)
C**** DEPTH *** THE DEPTH TO THE BOTTOM OF EACH SOIL LAYER(FEET)
C**** DIALIM ** THE LARGEST DIAMETER (FEET) TO BE CONSIDERED
C**** DIAM **** THE DIAMETER (FEET) OF THE STEM
C**** DSTART ** THE FIRST VALUE DIAM IS TO ASSUME
C****           DIAM IS INCREASED IN INCREMENTS OF 0.5 FT.
C**** FSB ***** THE FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE
C****           CAPACITY, QB
C**** FST ***** THE FACTOR OF SAFETY APPLIED TO THE TOTAL ULTIMATE
C****           CAPACITY, QU, WHICH RESULTS IN QDN
C**** NHEAD *** THE NUMBER OF CARDS IN HEADING
C**** HEADNG * DESCRIPTION OF THE DESIGN
C**** N ******* NUMBER OF SOIL LAYERS
C**** NTYPDS * SPECIFIES THE TYPE OF DESIGN CALCULATIONS DESIRED
C**** P ******* THE TOTAL DESIGN LOAD DESIRED (LBS.)
C**** QB ****** THE ULTIMATE BASE RESISTANCE. AN ASTERISK BESIDE
C****           THIS VALUE SIGNIFIES THAT THE RESISTANCE WAS
C****           CALCULATED CONSIDERING TWO OR MORE SOIL LAYERS
C****           THAT ARE WITHIN TWO BASE DIAMETERS FROM THE BASE
C**** QBD ***** THE TOTAL DESIGN LOAD USING A FACTOR OF SAFETY
C****           OF  FSB  APPLIED TO THE ULTIMATE BASE RESISTANCE
C**** QCHECK ** EFFICIENCY COMPUTATIONS ARE ELIMINATED FOR COMPUTED
C****           LOADS LESS THAN QCHECK  (LBS.)
C**** QDN ***** TOTAL DESIGN RESISTANCE WITH A FACTOR OF SAFETY
C****           OF  FST  APPLIED TO  QU
C**** QS ****** ULTIMATE SIDE RESISTANCE
C**** QU ****** TOTAL ULTIMATE RESISTANCE
C**** RATIO *** THIS IS THE RATIO OF THE DIAMETER OF THE BASE
C****           TO THE DIAMETER OF THE STEM
C**** SOIL **** THE TYPE OF SOIL IN A GIVEN LAYER
C****           SOIL(I)= 1 REPRESENTS A SAND
C****           SOIL(I)= 2 REPRESENTS A HOMOGENEOUS  CH  SOIL .
C****           SOIL(I)= 3 REPRESENTS A CLAY SHALE
C****           SOIL(I)= 4 REPRESENTS A SILTY CLAY(CL)
C****           SOIL(I)= 5  REPRESENTS A SANDY CLAY(CL)
C**** TOPLEN * TOP PORTION OF THE SHAFT TO BE IGNORED(FEET)
C**** TRANLM * THE MAXIMUM LOAD TRANSFER ALLOWED WITHIN
C****           A GIVEN SOIL LAYER (IN PSF)
C**** TYPB **** THE TYPE OF DYNAMIC PENETRATION TEST USED. *SPT*  DE-
C****           NOTES THE STANDARD PENETRATION TEST AND  *SDHPT*
C****           DENOTES THE STATE DEPARTMENT OF HIGHWAYS AND PUBLIC
C****           TRANSPORTATION PENETRATION TEST.
C**** WTD ***** DEPTH OF THE WATER TABLE
C**** NOTE - ONE BLANK CARD IS REQUIRED TO END THE PROGRAM
C***********************************************************************
   61 READ 23,NHEAD,TYPB
```

```
      DATA A,B/5HSDHPT,3HSPT/
   23 FORMAT(I1,4X,A5)
      IF(NHEAD.EQ.0)GO TO 96
      PRINT 28
   28 FORMAT(1H1)
C**** READ COMMENTS ****
      DO 27 IT=1,NHEAD
      READ 25,(HEADNG(IH),IH=1,8)
   25 FORMAT(8A10)
      PRINT 26,(HEADNG(IH),IH=1,8)
   26 FORMAT(1X,8A10)
      QV2=0. $ QV1=0.
   27 CONTINUE
      READ  1000,NTYPDS,DSTART,DIALIM,RATIO,BELANG,TOPLEN,BS
 1000 FORMAT(I1,9X,6F10)
      READ 2,N,WTD,P,QCHECK,FST,FSB
    2 FORMAT(I2,8X,F10,2F20,2F10)
      PRINT 20, P
   20 FORMAT(///,1X*DESIGN LOAD =*F10.0*LBS.*//)
      PRINT 4,QCHECK
    4 FORMAT(1X*MAKE COMPARISONS OF QU/VOLUME*/1X
     1*FOR DESIGN LOADS GREATER THAN*2X,F10.0*LBS.*//)
      P=P/2000.
      QCHECK=QCHECK/2000.
      PRINT 5,N
    5 FORMAT(1X*NUMBER OF LAYERS =*I5//)
      PI = 3.142
      N=N+1
      PRINT 9, WTD
    9 FORMAT(1X*WATER TABLE DEPTH =*F10.0* FT.*)
C**** READ SOIL INFORMATION ****
 3001 DO 500 I = 2,N
      READ11, ALPHA(I),BLOWS(I),SOIL(I),DEPTH(I),TRANLM(I)
   11 FORMAT(5F10)
      TRANLM(I)=TRANLM(I)/2000.
  500 CONTINUE
C**** MAKE CONVERSION OF BLOWCOUNT TO UNIT SIDE RESISTANCE AND UNIT
C**** BASE RESISTANCE USING THE T.T.I.   CORRELATIONS ****
  888 DO 700 I=2,N
C**** STAY HERE IF USING T.H.D. PENETROMETER VALUES ****
      IF(TYPB.EQ.B)GO TO 505
      IF(SOIL(I).GT.1.)GO TO 771
      SIDRES(I)=0.014*BLOWS(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      IF(BLOWS(I).LE.20.)GO TO 760
      IF(BLOWS(I).LE.65.)GO TO 761
      IF(BLOWS(I).LE.110.)GO TO 762
      BRESIS(I)=40.0
      GO TO 700
  760 BRESIS(I)=0.0
      GO TO 700
  761 BRESIS(I)=0.3556*(BLOWS(I)-20.)
      GO TO 700
  762 BRESIS(I)=16.+(.5333*(BLOWS(I)-65.))
      GO TO 700
  771 IF(SOIL(I).GT.2.)GO TO 741
      SIDRES(I)=0.070*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/2.8
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
      GO TO 700
  741 IF(SOIL(I).GT.3.)GO TO 742
```

```
      SIDRES(I)=0.013*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/10.0
      GO TO 700
  742 IF(SOIL(I).GT.4.)GO TO 743
      SIDRES(I)=0.063*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/2.8
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
      GO TO 700
  743 SIDRES(I)=0.053*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/2.8
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
      GO TO 700
C**** COME HERE IF USING S.P.T. PENETROMETER VALUES ****
  505 CONTINUE
      IF(SOIL(I).GT.1.)GO TO 881
      SIDRES(I)=0.026*BLOWS(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      IF(BLOWS(I).LE.10.)GO TO 860
      IF(BLOWS(I).LE.30.)GO TO 861
      IF(BLOWS(I).LE.50.)GO TO 862
      BRESIS(I)=40.0
      GO TO 700
  860 BRESIS(I)=0.0
      GO TO 700
  861 BRESIS(I)=0.80*(BLOWS(I)-10)
      GO TO 700
  862 BRESIS(I)=16.+1.2*(BLOWS(I)-30.)
      GO TO 700
  881 IF(SOIL(I).GT.2.)GO TO 841
      SIDRES(I)=0.100*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/1.6
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
      GO TO 700
  841 IF(SOIL(I).GT.3.)GO TO 842
      SIDRES(I)=0.018*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/7.0
      GO TO 700
  842 IF(SOIL(I).GT.4.)GO TO 843
      SIDRES(I)=0.090*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/1.6
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
      GO TO 700
  843 SIDRES(I)=0.076*BLOWS(I)*ALPHA(I)
      IF(SIDRES(I).GT.TRANLM(I))SIDRES(I)=TRANLM(I)
      BRESIS(I)=BLOWS(I)/1.6
      IF(BRESIS(I).GT.35.)BRESIS(I)=35.
  700 CONTINUE
 3000 DIAM=DSTART
 7778 PRINT 7779
 7779 FORMAT(//1X*SOIL(I)= 1 REPRESENTS A SAND*/
     11X*SOIL(I)= 2  REPRESENTS A HOMOGENEOUS  CH  SOIL*/
     21X*SOIL(I)= 3  REPRESENTS A CLAY SHALE*/
     41X*SOIL(I)= 4  REPRESENTS A SILTY CLAY(CL)*/
     51X*SOIL(I)= 5  REPRESENTS A SANDY CLAY(CL)*//)
  503 NRATIO=RATIO
      PRINT 1001,NTYPDS,DSTART,DIALIM,RATIO,FST,FSB
```

```
1001 FORMAT(///1X*NTYPDS=*I5/1X*DSTART =*F5.2,2X*FT.*/1X*DIALIM =*F5.2
    1/1X*RATIO  =*F5.2/1X*FST     =*F5.2/1X*FSB     =*F5.2//)
     IF(TYPB.EQ.A)PRINT 6
   6 FORMAT(7X*REDUCTION FACTOR*7X
    1*SIDRES(TSF)*7X*BRESIS(TSF)*7X*SDHPT(BLOWS/FT.)*7X
    2*SOIL*7X*DEPTH(FT.)*//)
     IF(TYPB.EQ.B)PRINT 7
   7 FORMAT(7X*REDUCTION FACTOR*7X
    1*SIDRES(TSF)*7X*BRESIS(TSF)*7X*SPTN(BLOWS/FT.)*7X
    2*SOIL*7X*DEPTH(FT.)*//)
     DEPTH(1)=0.0
     PRINT 12,DEPTH(1)
  12 FORMAT(93X,F14.0/)
     DO 501 I=2,N
     PRINT13, ALPHA(I), SIDRES(I),BRESIS(I),BLOWS(I),SOIL(I),DEPTH(I)
  13 FORMAT(F17.2,F18.2,F19.2,F20.0,F17.0//93X,F14.0/)
 501 CONTINUE
1004 CONTINUE
C**** DETERMINE THE BASE DIAMETER ****
     SQDIAM=DIAM*DIAM
     AREAST=PI*SQDIAM/4.
     IF(NRATIO.EQ.0)GO TO 2010
     DIAMB=DIAM*RATIO
     SDIAMB=DIAMB*DIAMB
     ABASE=PI*SDIAMB/4.
     IF(DIAM.LE.3.52)TL=0.250
     IF(DIAM.GT.3.52)TL=0.50
     BELANG=BELANG/57.29
     BL=TL+((DIAMB-DIAM)/(2.*TAN(BELANG)))
     BELANG=BELANG*57.29
     VOLBL=((PI*(BL-TL)*(SQDIAM/4.+SDIAMB/4.+SQRT(DIAMB*DIAM/4.))
    1/3.)+(ABASE*TL))/27.
     GO TO 2015
2010 DIAMB=DIAM
     BL=0.
     VOLBL=0.0
     ABASE=PI*DIAMB*DIAMB/4.
2015 CONTINUE
     QBATEM=0.
     SQS=0.$QSI=0.$QBA=0.
     FINAL=0.
     H=0.
     I=1
C**** DETERMINE THE STEM UNIT SURFACE AREA ****
     ASIDE=PI*DIAM
     FACT=2.*DIAMB
     NACT=IFIX(FACT)
C**** DETERMINE THE AREA OF THE BASE ****
     STEEL1=(.01*PI*DIAM**2/4.)*144.
     PRINT 21,DIAM,DIAMB,BL,BELANG,TOPLEN,BS,STEEL1,VOLBL
  21 FORMAT(1H1,*DIAMETER OF STEM          =*F5.2,2X*FT.*/1X
    1*DIAMETER OF BASE          =*F6.2,2X*FT.*/1X*END OF STEM TO BASE
    1   =*F6.2,2X*FT.*/1X*ANGLE OF BELL          =*F5.1,
    11X*DEGREES*/1X*IGNORED TOP PORTION      =*F5.2,1X*FT.*/1X*IGNORED
    1BOTTOM PORTION   =*F5.2,1X*FT.*/1X*AREA OF ONE PERCENT STEEL=*
    1F6.2,1X*SQ.IN.*/1X*VOLUME OF UNDERREAM       =*F6.2* CU.YDS.*//)
     PRINT 81
  81 FORMAT(1X*ESTIMATED*/1X*SHAFT LENGTH*6X*VOLUME*13X*QS*13X
    1*QB*13X*QU*12X*QBD*12X*QDN*13X*QU/VOLUME*/3X
    2*(FEET)*9X*(CU.YDS.)*9X*(TONS)*9X*(TONS)*9X
    3*(TONS)*9X*(TONS)*9X*(TONS)*9X*(TONS/CU.YDS.)*)
  50 IF(I.EQ.(N+1))GO TO 800
```

```
      IF((DEPTH(I)-TOPLEN).LE.0.0)GO TO 70
      LAYER=(DEPTH(I)-H)
      DO 30 L=1,LAYER
      X=L
C***********************************************************************
C**** DETERMINING THE SIDE RESISTANCE IF THE SECTION IS IN CLAY
C***********************************************************************
      QSI=SIDRES(I)*ASIDE+SQS
  200 DELTA=X+BS+BL+2.*DIAMB
      M=I
      STAR=0.
      MDELTA=DELTA
C***********************************************************************
C**** DETERMINING THE BASE RESISTANCE IF THE SECTION IS IN CLAY
C***********************************************************************
      IF(MDELTA.LE.LAYER.AND.SOIL(I).GT.1.)QBA=BRESIS(I)*ABASE
      IF(MDELTA.LE.LAYER.AND.SOIL(I).GT.1.)GO TO 55
C***********************************************************************
C**** DETERMINING THE BASE RESISTANCE IF THE SECTION IS IN SAND
C***********************************************************************
      IF(MDELTA.LE.LAYER)QBA=BRESIS(I)*ABASE/(.60*DIAMB)
      IF(MDELTA.LE.LAYER)GO TO 55
      CALL SUATB2
      IF((I+1).EQ.(N+1))GO TO 800
      IF(FINAL.EQ.1.)GO TO 800
C**** DETERMINE THE LENGTH OF THE SHAFT ****
   55 Y=H+X+BS+BL
      NY=Y
      YN=NY
      IF((Y-YN).GE.0.50)YESTIM=YN+1.
      IF((Y-YN).LT.0.50)YESTIM=YN
C**** DETERMINE THE VOLUME OF CONCRETE ****
      VOLY=VOLBL+(YESTIM-BL)*AREAST/27.
C**** DETERMINE THE DIFFERENT SHAFT CAPACITIES ****
      QS = QSI
      QB = QBA
      QU = QSI + QBA
      QDN=QU/FST
      QBD=QS+QB/FSB
      QV2=QU/VOLY
      IF(STAR.EQ.1.)GO TO 90
      PRINT 91,YESTIM,VOLY,QS,QB,QU,QBD,QDN,QV2
   91 FORMAT(1X,F6.0,2F17.2,4F15.2,F17.2)
      GO TO 717
   90 PRINT 92,YESTIM,VOLY,QS,QB,QU,QBD,QDN,QV2
   92 FORMAT(1X,F6.0,2F17.2,F15.2,1H*,F14.2,2F15.2,F17.2)
  717 CONTINUE
C**** COMPARE THE PREVIOUS VALUE OF  *QU/VOLUME*  WITH THE NEW VALUE
C**** AND SAVE THE LARGEST VALUE OF THE TWO ****
      IF(QV2.LT.QV1)GO TO 821
      IF(QDN.LT.QCHECK)GO TO 821
      SAVEL=YESTIM$SAVED=DIAM$SAVEDB=DIAMB$SAVQV2=QV2
      SAVEV=VOLY$SAVEQU=QU
      QV1=QV2
  821 CONTINUE
C**** IF THE DESIGN LOAD EXCEEDS  *P*  GO TO 800 ****
      IF(QBD.GT.P.AND.QBD.LE.QDN)GO TO 800
      IF(QDN.GT.P)GO TO 800
      SQS = QSI
   30 CONTINUE
   70 HTEMP=H
      I = I+1
```

```
      H=DEPTH(I-1)
      IF((DEPTH(I-1)-TOPLEN).LE.0.)H=TOPLEN
      GO TO 50
  800 CONTINUE
      DIAM=DIAM+0.5
C**** IF ANOTHER DIAMETER IS TO BE CONSIDERED GO TO 3000.  OTHERWISE,
      IF(NTYPDS.EQ.1.AND.DIAM.GT.(DIALIM+.02))PRINT 1346,SAVED,
     1SAVEDB,SAVEL,SAVEV,SAVEQU,SAVQV2
 1346 FORMAT(///*THE MOST EFFICIENT SHAFT FOR THIS SOIL PROFILE APPEARS
     1TO BE THE FOLLOWING ONE:*//1X*DIAMETER OF STEM      =*F5.1,1X*FT.*
     2//1X*DIAMETER OF BASE      =*F5.1,1X*FT.*//1X*ESTIMATED SHAFT LENG
     3TH=*F5.0,1X*FT.*//1X*VOLUME OF CONCRETE    =*F7.2,1X*CU.YDS.*//1X
     4*ULTIMATE LOAD CAPACITY=*F7.2,1X*TONS*//1X
     5*QU/VOLUME OF CONCRETE =*F7.2,1X*TONS/CU.YD.*)
C**** GO TO 61 TO SEE IF ANOTHER SOIL PROFILE IS TO BE CONSIDERED
C**** OR IF THE PROGRAM IS TO BE ENDED ****
      IF(DIAM.GT.(DIALIM+.02).AND.NTYPDS.EQ.1)GO TO 61
      IF(DIAM.GT.(DIALIM+.02))GO TO 6666
      GO TO 1004
C**** COME HERE IF THE SAME SOIL PROFILE IS TO BE  RE-USED WITH
C**** THE FOLLOWING NEW VARIABLES ****
 6666 NTYPLD=NTYPDS
      READ 1000,NTYPDS,DSTART,DIALIM,RATIO,BELANG,TOPLEN,BS
      IF(NTYPLD.EQ.2)GO TO 3000
      READ 6667, (ALPHA(J),J=2,N)
 6667 FORMAT(16F5)
      GO TO 3000
   96 CONTINUE
      END
      SUBROUTINE SUATB2
      DIMENSION DEPTH(30),SOIL(30),SIDRES(30),BRESIS(30)
      COMMON/SAME1/DEPTH,X,M,DELTA,MDELTA,H,MH,LONG,FACT,QBATEM,
     1SOIL,N,NACT,FINAL,STAR,J,BS,TOPLEN,SIDRES,BRESIS,I,QBA,
     2ABASE,DIAMB,BL
C***************************************************************************
C**** THIS SUBROUTINE FINDS THE BEARING CAPACITY.  IT TAKES INTO ACCOUNT
C**** THE CASES WHERE THE BOTTOM OF THE SHAFT IS IN ONE TYPE OF SOIL
C**** BUT IS CLOSE ENOUGH TO A LOWER LAYER WHICH MAY AFFECT THE BEARING
C**** CAPACITY. A STAR  IN THE OUTPUT DISTINGUISHES  A BEARING CAPACITY
C**** CALCULATED WHICH CONSIDERS  TWO LAYERS.
C***************************************************************************
      LONG=H+X+BS+BL
      IF(DEPTH(I-1).EQ.0.)DEPTH(I-1)=TOPLEN
      MH=H
   20 IF((I+1).EQ.(N+1))RETURN
      MDPTH1=DEPTH(I+1) $ KDEPTH=DEPTH(I)
      IF(MDPTH1.LE.LONG)I=I+1
      IF(MDPTH1.LE.LONG)GO TO 20
      IF(MDPTH1.LT.(LONG+NACT))GO TO 18
      IF(KDEPTH.GT.LONG)GO TO 3
      I=I+1
      IF(SOIL(I).NE.1.)GO TO 17
      QBA=BRESIS(I)*ABASE/(.60*DIAMB)
      I=M
      RETURN
   17 QBA=BRESIS(I)*ABASE
      I=M
      RETURN
   18 IF((I+2).NE.(N+1))GO TO 19
      FINAL=1.0
      RETURN
   19 MDPTH2=DEPTH(I+2)
```

```
      IF(MDPTH2.LT.(LONG+NACT))GO TO 40
   30 I=I+1
    3 CONTINUE
      IF(SOIL(I).GT.1..AND.SOIL(I+1).EQ.1.)GO TO 21
      IF(SOIL(I).EQ.1..AND.SOIL(I+1).EQ.1.)GO TO 11
      IF(SOIL(I).EQ.1..AND.SOIL(I+1).GT.1.)GO TO 12
C**** COME HERE IF BOTH LAYERS BEING CONSIDERED ARE CLAYS *****
   22 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   31 QBA=(QBATEM+((BRESIS(IT1)*DIST1)+(BRESIS(IT2+1)*DIST2))*
     1ABASE)/FACT
      QBATEM=0.
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF THE TOP LAYER IS A CLAY AND THE LOWER
C**** LAYER IS A SAND **********
   21 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   32 QBA21S=BRESIS(IT2+1)*ABASE/(.60*DIAMB)
      QBA=(QBATEM+(QBA21S*DIST2)+(BRESIS(IT1)*ABASE*DIST1))/FACT
      QBATEM=0.
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF BOTH LAYERS BEING CONSIDERED ARE SAND *****
   11 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   33 QBA11A=BRESIS(IT1)*ABASE/(.60*DIAMB)
      QBA11B=BRESIS(IT2+1)*ABASE/(.60*DIAMB)
      QBA=(QBATEM+(QBA11A*DIST1+QBA11B*DIST2))/FACT
      QBATEM=0.
      I=M
      STAR=1.
      RETURN
C**** COME HERE IF THE TOP LAYER IS A SAND AND THE LOWER
C**** LAYER IS A CLAY **********
   12 IT1=I
      IT2=I
      DIST1=DEPTH(IT1)-LONG
      DIST2=LONG+FACT-DEPTH(IT2)
   34 I=IT1
      QBA=BRESIS(I)*ABASE/(.60*DIAMB)
      QBA=((QBA*DIST1)+(BRESIS(IT2+1)*ABASE*DIST2)+QBATEM)/FACT
      QBATEM=0.
      I=M
      STAR=1.
       RETURN
C**** COME HERE AND CALCULATE A WEIGHTED BEARING CAPACITY
C**** FOR A LAYER OF SOIL FULLY WITHIN THE TWO DIAMETER
C**** DISTANCE BELOW THE BASE OF THE SHAFT **********
   40 I=I+1
      IT1=I
      DIST1=DEPTH(I)-LONG
   60 I=I+1
      DIFFER=DEPTH(I)-DEPTH(I-1)
```

```
      IF(SOIL(I).GT.1.)GO TO 67
      QBA=BRESIS(I)*ABASE*DIFFER/(.60*DIAMB)
      GO TO 45
   67 QBA=BRESIS(I)*ABASE*DIFFER
   45 QBATEM=QBATEM+QBA
      IF((I+1).EQ.(N+1))RETURN
      MDPTH2=DEPTH(I+1)
      IF((MDPTH2-(LONG+NACT)).GE.0)GO TO 50
      GO TO 60
   50 IT2=I
      I=I+1
      DIST2=LONG+FACT-DEPTH(IT2)
      IF(SOIL(IT1).GT.1..AND.SOIL(I).GT.1.)GO TO 31
      IF(SOIL(IT1).GT.1..AND.SOIL(I).EQ.1.)GO TO 32
      IF(SOIL(IT1).EQ.1..AND.SOIL(I).EQ.1.)GO TO 33
      GO TO 34
      END
```

EXPLANATION OF INPUT FORMAT FOR BSHAFT

As a matter of convenience in explaining the input format, the data cards are broken down into two groups: set A and set B. The cards in set A consist of all necessary information for a soil profile. The cards in set B contain information related to the soil profile in set A. In using both sets of cards, one can reuse the soil profile data in set A with the modified values that are found in set B.

If one wishes to use the data found in set A just once for a series of capacity computations in a soil profile without use of set B, then NTYPDS is assigned a value of one. If another soil profile is to be considered, this second set A immediately follows the first set A.

NTYPDS is assigned a value of two when the information in set A is to be used again with modified values of the variables found in Card No. 3. The modified values are furnished in set B. If a new soil profile (set A) is to be considered following the capacity computations based on the modified values, NTYPDS in set B must be given a value of one. However, if the values of Card No. 3 are to be modified another time, NTYPDS in set B can be given a value of two. Any number of sets B can follow any set A.

NTYPDS is given a value of three for a case similar to the one in which NTYPDS is equal to two except that the strength reduction factor, ALPHA, for every soil layer can be modified. This information is furnished in set B in the order shown later. No more than two cards of the No. 6 type are required because not more than 29 layers of soil can be considered in this program.

In order to end the program or in order to consider a new set A, the last assigned value of NTYPDS must have been one. Furthermore, another requirement for ending the program is to follow set A (or set B when used) with a blank card, Card No. 7.

Card Type No. 1. NHEAD is an integer refering to the number of cards that comprise HEADNG. NHEAD must be less than or equal to four. IF NHEAD is equal to zero, the program will end. There is one card per set A.

TYPB is an alphanumeric identifier. If the SDHPT cone penetrometer results are used, then the letters "SDHPT" are punched in columns six through ten as shown later. However, if the Standard Penetration Test results are used, the letters "SPT" are punched instead of "SDHPT".

Card Type No. 2. Data cards of this type consist of alphanumeric information which can be used to describe a design case. No more than four cards can comprise HEADNG for each set A.

Card Type No. 3. There is only one such card for each set A. However any information on this card can be changed by the method explained earlier in this section by assigning a proper value of NTYPDS and the appropriate information in set B.

DSTART is the first stem diameter to be considered for each set A. Units are feet.

DIALIM is the final diameter size to be considered when capacity computations are made for more than one shaft diameter. When only one shaft size is of interest, DIALIM is assigned a value of zero.

RATIO is the ratio of the base diameter to the stem diameter. In the case of straight shafts, the value of RATIO must be zero.

BELANG is the angle of the bell with respect to the vertical. In the case of straight shafts, BELANG is zero. Units are degrees.

TOPLEN is the top portion of the shaft that is noncontributing to side resistance. Units are feet.

BS is the bottom section of the shaft that obtains no resistance from the soil. Units are feet.

Card Type No. 4. There is only one card per set A. N is an integer refering to the number of strata in the soil profile. N must be less than or equal to 29.

WTD is the water table depth with units in feet.

P is the design load. Units are pounds.

QCHECK also has units pounds. As discussed in Chapter 4, efficiency computations are eliminated for computed loads less than QCHECK. QCHECK should be given a suitable value so as to avoid making unnecessary efficiency computations for drilled shafts with small penetrations.

FST refers to the factor of safety that must be applied to the total ultimate capacity.

FSB refers to the factor of safety that must be applied to the ultimate base capacity.

Card Type No. 5. There are as many of these cards as there are layers of soil. The data cards are arranged in order of increasing depth.

ALPHA is the strength reduction factor, α. For sand, ALPHA may be assigned a value of 1.0 as discussed in Chapter 3. However, if a lower load transfer is expected than that calculated by the correlations listed in Table 3.9, ALPHA can be assigned a value lower than 1.0.

BLOWS refers to the average number of blows per foot for a soil stratum obtained from the dynamic penetration test (SDHPT or SPT).

SOIL is used to specify a type of soil layer. For this computer program, the soil types are broken down into five classes as follows:

1.0 defines a sand layer,

2.0 defines a homogeneous CH soil layer,

3.0 defines a clay-shale,

4.0 defines a silty clay (CL), and

5.0 defines a sandy clay (CL).

DEPTH specifies the bottom depth of each layer of soil. Units are in feet.

TRANLM is the maximum load transfer permissible for a given stratum. Units are in pounds per square foot.

Card Type No. 6. This card is used in set B when the previously specified strength reduction factors are to be modified without changing any of the other information in set A. Even if only one ALPHA value is to be changed, desired values for all soil layers must be included. When this type of card is used, it must be preceded by Card Type No. 3 in set B. No more than two No. 6 cards can be used because of the limit of 29 soil layers per profile.

Card Type No. 7. This data card should always be the last one of the deck and may follow a set A or B, whichever is the case. Upon reading this blank card, the computer will assign NHEAD a value of zero, thus causing the program to be terminated.

INPUT FORMAT FOR PROGRAM BSHAFT

CARD TYPE NUMBER	Variable	Format	Columns	Number of cards	Set
1	NHEAD	I1	1	ONE CARD	SET A
	TYPB	A5	6–10		
2	HEADNG	8A10	1–80	NUMBER OF CARDS=NHEAD	
3	NTYPDS	I1	1	ONE CARD	
	DSTART	F10	11–20		
	DIALIM	F10	21–30		
	RATIO	F10	31–40		
	BELANG	F10	41–50		
	TOPLEN	F10	51–60		
	BS	F10	61–70		
4	N	I2	1–2	ONE CARD	
	WTD	F10	11–20		
	P	F20	21–40		
	QCHECK	F20	41–60		
	FST	F10	61–70		
	FSB	F10	71–80		
5	ALPHA	F10	1–10	AS MANY CARDS AS THERE ARE SOIL LAYERS	
	BLOWS	F10	11–20		
	SOIL	F10	21–30		
	DEPTH	F10	31–40		
	TRANLM	F10	41–50		
3	NTYPDS	I1	1	ONE CARD	SET B
	DSTART	F10	11–20		
	DIALIM	F10	21–30		
	RATIO	F10	31–40		
	BELANG	F10	41–50		
	TOPLEN	F10	51–60		
	BS	F10	61–70		
6	ALPHA(1)	F5	1–5	ONE OR TWO CARDS, AS REQUIRED	
	ALPHA(2)	F5	6–10		
	ALPHA(3) . . .	F5	11–15, 16 . . . 75		
	ALPHA(N)	F5	76–80		
7	(BLANK CARD TO END PROGRAM)			ONE CARD	

SAMPLE OUTPUT FOR PROGRAM BSHAFT

```
BRYAN,TX, - MAR 1973(DRY)
42 FT. PENETRATION - QU=425 TONS - QS=255 TONS


DESIGN LOAD =    900000LBS.


MAKE COMPARISONS OF QU/VOLUME
FOR DESIGN LOADS GREATER THAN      100000LBS.


NUMBER OF LAYERS =    4


WATER TABLE DEPTH =        29 FT.


SOIL(I)= 1 REPRESENTS A SAND
SOIL(I)= 2  REPRESENTS A HOMOGENEOUS  CH  SOIL
SOIL(I)= 3  REPRESENTS A CLAY SHALE
SOIL(I)= 4  REPRESENTS A SILTY CLAY(CL)
SOIL(I)= 5  REPRESENTS A SANDY CLAY(CL)



NTYPDES=    1
DSTART = 2,50  FT.
DIALIM =-0,00
RATIO  =-0,00
FST    = 2,00
FSB    = 3,00
```

REDUCTION FACTOR	SIDERES(TSF)	BRESIS(TSF)	THDN(BLOWS/FT.)	SOIL	DEPTH(FT.)
					0
,60	1,30	11,07	31	2	
					12
,60	,76	6,43	18	2	
					32
,60	2,00	35,00	150	5	
					42
,60	2,00	35,00	170	5	
					50

DIAMETER OF STEM = 2,50 FT.
DIAMETER OF BASE = 2,50 FT.
END OF STEM TO BASE = 0,00 FT.
ANGLE OF BELL = -0,0 DEGREES
IGNORED TOP PORTION = 5,00 FT.
IGNORED BOTTOM PORTION = 2,00 FT.
AREA OF ONE PERCENT STEEL= 7,07 SQ.IN.
VOLUME OF UNDERREAM = 0,00 CU.YDS.

ESTIMATED SHAFT LENGTH (FEET)	VOLUME (CU.YDS.)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS.)
8	1,45	10,23	49,80*	60,02	26,83	30,01	41,26
9	1,64	20,45	45,24*	65,69	35,53	32,85	40,14
10	1,82	30,68	40,68*	71,36	44,24	35,68	39,25
11	2,00	40,91	36,12*	77,03	52,95	38,51	38,51
12	2,18	51,14	31,56	82,70	61,66	41,35	37.90
13	2.36	61,36	31,56	92,92	71,88	46,46	39,31
14	2,55	71,59	31,56	103,15	82,11	51.58	40.52
15	2,73	77,53	31,56	109,09	88,05	54,54	40,00
16	2,91	83,47	31,56	115,03	93,99	57,51	39,54
17	3,09	89,41	31,56	120,97	99,93	60,48	59.13
18	3,27	95,34	31,56	126,90	105,86	63,45	3*.77
19	3,45	101,28	31,56	132,84	111,80	66,42	38,45
20	3,64	107,22	31,56	138,78	117,74	69,39	38,16
21	3,82	113,16	31,56	144,72	123,68	72,36	37,90
22	4,00	119,10	31,56	150,66	129,62	75,33	37,66
23	4,18	125,04	31,56	156,60	135,56	78,30	37,44
24	4,36	130,97	31,56	162,53	141,49	81,27	37.25
25	4,55	136,91	31,56	168,47	147,43	84,24	37,06
26	4,73	142,85	31,56	174,41	153,37	87,21	36,89
27	4,91	148,79	31,56	180,35	159,31	90,17	36,74
28	5,09	154,73	59,61*	214,34	174,60	107,17	42,10
29	5,27	160,67	87,67*	248,33	189,89	124,17	47,10
30	5,45	166,60	115,72*	282,33	205,18	141,16	51,76
31	5,64	172,54	143,77*	316,32	220,47	158,16	56,12
32	5,82	178,48	171,83	350,31	235,76	175,15	60,21
33	6,00	184,42	171,83	356,25	241,70	178,12	59,37
34	6,18	190,36	171,83	362,19	247,63	181,09	58,59
35	6,36	206,07	171,83	377,90	263,34	188,95	59,38
36	6,55	221,78	171,83	393,61	279,05	196,80	60,13
37	6,73	237,49	171,83	409,32	294,76	204,66	60,84
38	6,91	253,20	171,83*	425,03	310,47	212,51	61,51
39	7,09	268,91	171,83*	440,74	326,18	220,37	62,15
40	7,27	284,62	171,83*	456,45	341,89	228,22	62,76
41	7,45	300,33	171,83*	472,16	357,60	236,08	63,33
42	7,64	316,04	171,83	487,87	373,31	243,93	63,88
43	7,82	331,75	171,83	503,58	389,02	251,79	64,41
44	8,00	347,46	171,83	519,29	404,73	259,64	64,91
45	8,18	363,17	171,83	535,00	420,44	267,50	65,38

THE MOST EFFICIENT SHAFT FOR THIS SOIL PROFILE APPEARS TO BE THE FOLLOWING ONE:

DIAMETER OF STEM = 2,5 FT.

DIAMETER OF BASE = 2,5 FT.

ESTIMATED SHAFT LENGTH= 45 FT.

VOLUME OF CONCRETE = 8.18 CU.YDS.

ULTIMATE LOAD CAPACITY= 535.00 TONS

QU/VOLUME OF CONCRETE = 65.38 TONS/CU.YD.

APPENDIX C

DRILLED SHAFT CONSTRUCTION METHODS

DRILLED SHAFT CONSTRUCTION METHODS

In the following paragraphs, three methods of installing drilled shafts are briefly discussed. However, the reader is advised that every contractor may have a construction method that deviates slightly from those presented below.

Dry Method

This method of construcion entails excavating the shaft without the aid of any fluid and following the excavation phase with complete concreting of the hole. This method of constructing drilled shafts is used in firm to stiff cohesive soils above the water table which do not have a pronounced secondary structure that will cause the soil to slough into the excavated hole. Drilled holes in similar soils below the water table will remain open for small time periods. However, at greater depths below the water table, the high seepage pressures will cause softening of the soil and induce caving of the shaft walls. Under such circumstances, the hole is advanced with the aid of a steel casing which is withdrawn as the hole is filled with concrete.

Slurry Displacement Method

In areas where excavation shafts will not remain open due to the softness or pronounced secondary structure of the cohesive soil, the presence of cohesionless soil or high seepage pressures, or a combination of any of these cases, a shaft hole can be advanced by filling the hole with a slurry of drilling mud of sufficient specific gravity to stabilize the hole. After the shaft is excavated to design grade, the concrete is poured with a tremie pipe in a cuatious operation so as to displace the slurry with concrete and ensuring that no slurry is trapped along the sides of the shaft.

Disadvantages of this method are that the use of the slurry precludes any visual inspection of the drilled hole and that a highly experienced drilling crew familiar with the method is necessary to ascertain proper displacement of the slurry during the concreting operation.

Casing Method

In this method of construction, which is also used for unstable soils, the shaft can first be advanced with drilling fluid until a strong clay layer is encountered beneath the caving soils. A steel casing is then inserted into the hole and pushed into the strong clay to form a tight seal. The slurry is then removed with a cleaning bucket and the hole is advanced in the clay. Upon completion of the drilling operation, concrete is poured to a suitable elevation so as to allow proper withdrawal of the steel casing. A major advantage of this method is that the hole the be visually inspected upon completion of the drilling operation.