# CRITERIA FOR THE DESIGN OF AXIALLY LOADED DRILLED SHAFTS

by

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Soil Properties as Related to Load-Transfer
Characteristics of Drilled Shafts

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## PREFACE

This report is the eleventh and final report in a series from Research Study 3-5-65-89 of the Cooperative Highway Research Program between the Center for Highway Research, the Texas Highway Department, and the United States Department of Transportation. It summarizes the six-year program of field testing and gives criteria for the design of drilled shafts in predominantly clay soil profiles.

This report is based upon the work of many former graduate research assistants including Vasant Vijayvergiya, Walter Barker, John Chuang, Clarence Ehlers, David Campbell, Fadlo Touma, Robert Welch, Crozier Brown, and Michael O'Neill. Technical contributions were also made by Harold Dalrymple, James Anagnos, Frederick Koch, and Olen Hudson. Mrs. Eddie B. Hudepohl supervised the typing and the editing of the manuscript of this report.

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LIST OF REPORTS

Report No. 89-1, "Field Testing of Drilled Shafts to Develop Design Methods," by Lymon C. Reese and W. Ronald Hudson, describes the overall approach to the design of drilled shafts based on a series of field and laboratory investigations.

Report No. 89-2, "Measurements of Lateral Earth Pressure in Drilled Shafts," by Lymon C. Reese, J. Crozier Brown, and H. H. Dalrymple, describes the development and evaluation of pressure gages to measure lateral-earth pressures on the drilled shaft.

Report No. 89-3, "Studies of Shearing Resistance Between Cement Mortar and Soil," by John W. Chuang and Lymon C. Reese, describes the overall approach to the design of drilled shafts based on field and laboratory investigations.

Report No. 89-4, "The Nuclear Method of Soil-Moisture Determination at Depth," by Clarence J. Ehlers, Lymon C. Reese, and James N. Anagnos, describes the use of nuclear equipment for measuring the variations of moisture content at the drilled shaft test sites.

Report No. 89-5, "Load Distribution for a Drilled Shaft in Clay Shale," by Vasant N. Vijayvergiya, W. Ronald Hudson, and Lymon C. Reese, describes the development of instrumentation capable of measuring axial load distribution along a drilled shaft, the development, with the aid of full-scale load testing, of a technique of analysis of observed data, and the correlation of observed data with the Texas Highway Department cone penetration test.

Report No. 89-6, "Instrumentation for Measurement of Axial Load in Drilled Shafts," by Walter R. Barker and Lymon C. Reese, describes the development and performance of various instrumentation systems used to measure the axial load distribution in field tests of full-scale drilled shafts.

Report No. 89-7, "The Determination of Soil Properties *In Situ*," by David B. Campbell and W. Ronald Hudson, describes the use of the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and The University of Texas *in situ* device in estimating soil properties *in situ* and estimating load transfer values obtained from drilled shaft tests.

Report No. 89-8, "Behavior of Axially Loaded Drilled Shafts in Beaumont Clay," by Michael W. O'Neill and Lymon C. Reese, describes the results of axial load tests of instrumented drilled shafts having varying geometry and differing methods of installation and presents a tentative design procedure for drilled shafts in Beaumont clay.

Report No. 89-9, "Load Carrying Characteristics of Drilled Shafts Constructed with the Aid of Drilling Fluids," by Walter R. Barker and Lymon C. Reese, describes the construction, instrumentation, and testing of a drilled shaft constructed with the use of drilling mud.

Report No. 89-10, "Lateral Load Behavior of Drilled Shafts," by Robert C. Welch and Lymon C. Reese.

Report No. 89-11F, "Criteria for the Design of Axially Loaded Drilled Shafts," by Lymon C. Reese and Michael W. O'Neill, summarizes the results of previous research and presents criteria for designing drilled shafts.

# ABSTRACT

In recent years drilled shafts have come into increasing use as foundation elements due to the economic advantage they afford. Prior to 1965 little information had been acquired concerning the magnitudes of skin friction and end bearing that are developed by drilled shafts. Consequently, design procedures, reflecting the lack of information, have been conservative, often allowing no skin friction at all.

Between 1965 and 1971 several instrumented drilled shafts were installed and load tested by the Center for Highway Research at various sites in Texas. The results of the tests were analyzed and, together with a thorough review of the work of other investigators, were used in establishing realistic criteria for design values of side resistance and base capacity.

A step-by-step procedure incorporating these criteria was developed for use in designing shafts in predominantly clay soils. This procedure includes the effects of construction technique and shaft geometry and is intended for use in the design office.



KEY WORDS: drilled shafts, foundation engineering, shear strength, construction, clay soil, design criteria

## SUMMARY

Results of load tests on instrumented drilled shafts in clay, clay shale, and sand have indicated that three variables primarily govern the behavior of drilled shafts under axial loading. They are soil conditions, geometry of shaft, and method of installation.

A design procedure relating permissible side shear and end bearing to these variables was developed during this study. It is intended for use primarily in clay soils, but it may be used with a measure of judgment in clay-sand and clay-silt profiles.

## IMPLEMENTATION STATEMENT

The design procedure and criteria presented in this report are recommended for use in Texas Highway Department district design offices. The design method will be useful in establishing capacities or design elevations of drilled shafts in predominantly clay soil profiles. The procedure is concise and is written to be of direct use to the design engineer. Examples of its application are given to familiarize the designer with the procedure.

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NOMENCLATURE

| Symbol | Definition |
|---|---|
| $A_B$ | area of base |
| $A_S$ | peripheral area of stem, or cylindrical part of shaft |
| $c$ | undrained cohesion |
| $d$ | thickness of stratum |
| $N$ | number of blows per foot for THD penetrometer |
| $N_c$ | bearing capacity factor (dimensionless) |
| $p$ | factor relating THD penetrometer blow count to maximum unit side resistance in tons per square foot |
| $p'$ | factor relating THD penetrometer blow count to unit base capacity in tons per square foot |
| $(Q_B)_{ult}$ | ultimate base load |
| $(Q_S)_{ult}$ | ultimate load carried in side resistance along the stem |
| $(Q_T)_{ult}$ | ultimate capacity of the shaft |
| $(Q_T)_{Design}$ | design load for the shaft |
| $(q_S)_{ult}$ | unit ultimate side resistance |
| $S$ | shear strength of soil |
| $w$ | effective unit weight of soil |
| $\alpha$ | ratio of maximum unit side resistance to shear strength of soil |
| $\alpha_{avg}$ | average value of $\alpha$ over a specified length of shaft |
| $\phi$ | angle of internal friction |

# CHAPTER I

# INTRODUCTION

## Scope

In 1965 the Center for Highway Research of The University of Texas at Austin began a study to investigate the behavior of axially loaded drilled shafts under realistic field conditions. The study was pursued by constructing and load testing full-scale drilled shafts with varying geometry in different geological formations throughout the eastern part of Texas. Since 1965 nine full-sized, fully instrumented shafts have been installed, tested, and evaluated. The locations of the test sites are shown in Fig. 1.

An additional instrumented drilled shaft was tested under lateral loading at Site III (Fig. 1) in stiff, fissured clay. Based upon the tests conducted on that shaft and a review of tests performed by others, criteria for soil resistance-lateral displacement relationships were developed. The reader is referred to Welch and Reese (1972) for those criteria.

One purpose of the axial load study was to develop a more economical design procedure for drilled shafts for the sponsors, the Texas Highway Department and the Federal Highway Administration. The procedure, as developed, incorporates the rational use of side shear and permits an accurate determination of the tip resistance mobilized by drilled shafts.

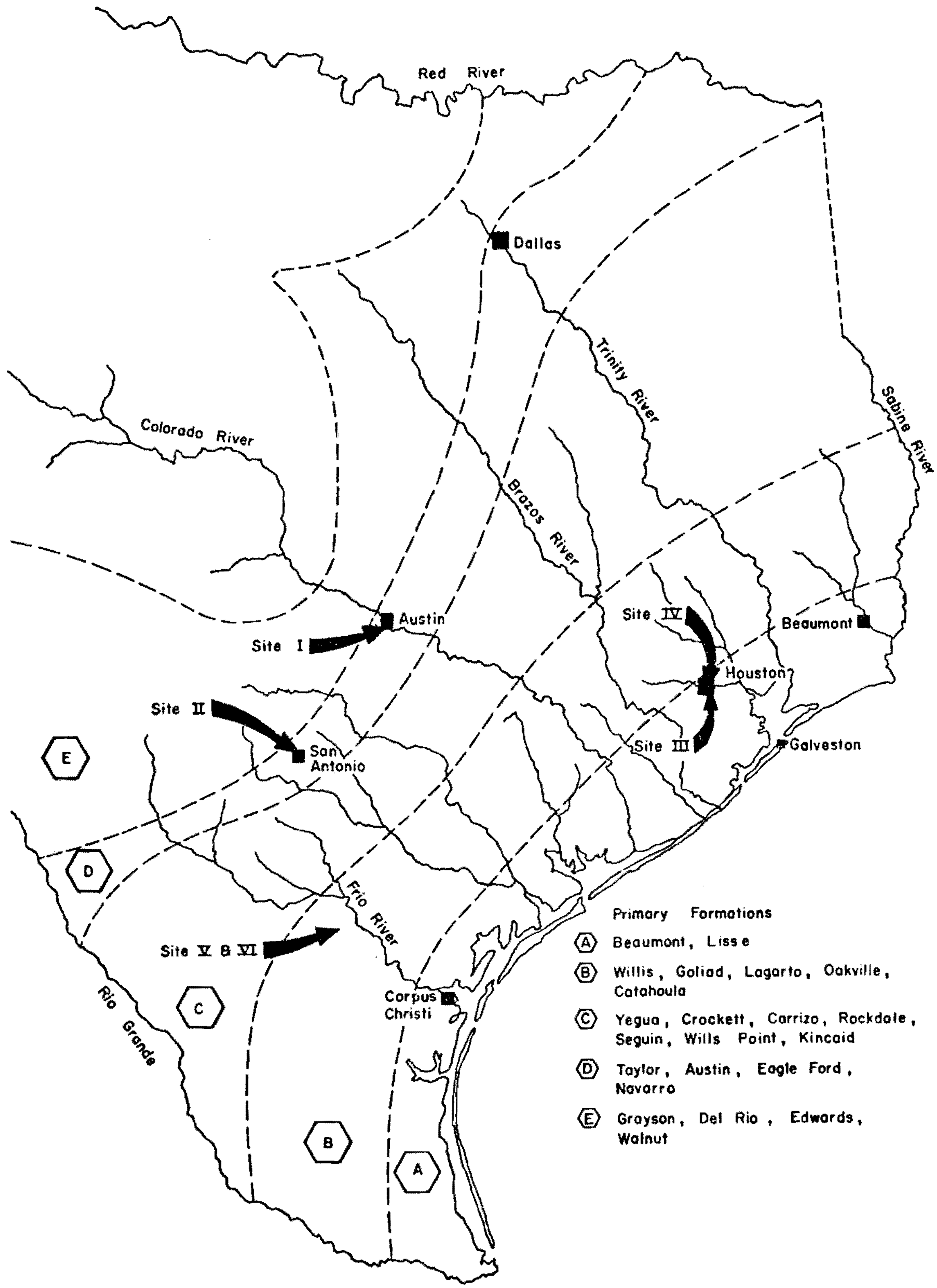



Fig 1. Location of drilled shaft test sites (After Barker and Reese, 1970).

This report summarizes the information gathered during this study (presented in detail in the previous reports of this series) and concisely gives a design procedure for sizing and calculating working capacities of axially loaded drilled shafts which is both practical and economical.

Only the behavior of single shafts was studied. Group effects are small in most soils for bridge bent foundations, in which shafts are usually widely spaced. Since the principal design application is expected to be in sizing drilled shafts for bridge bents, group behavior studies were not undertaken.

## General

Drilled shafts, also known by a variety of other names, including drilled piers, cast *in situ* piles, bored piles, and drilled caissons, are made by drilling a hole into the soil and casting concrete in the hole directly against the natural soil. Sometimes the base is enlarged (belled). The concrete is usually reinforced. Drilled shafts have a wide variety of uses in highway construction. The most predominant use is in foundations for bridges, although drilled shafts have also been used as anchorages and as earth retaining structures. Drilled shafts have been designed to support loads of from several tons to over 10,000 tons. One such heavily loaded drilled shaft, reported in the literature (*Engineering News-Record*, 1971), is over 140 feet deep with a 33-foot diameter bell and is designed to carry a load of over 13,000 tons. This is believed to be the largest drilled shaft constructed to date.

Drilled shafts are foundation alternatives to driven piles. The reason that the foundation engineer chooses drilled shafts instead of driven piles is often one of economics. For example, a $58,000 savings was realized on the foundation of a one-quarter-mile-long freeway bridge in Houston when drilled shafts were used instead of driven piles. Drilled shafts also have other advantages, including reduction of ground heave and reduction of noise and vibration on construction sites. The use of drilled shafts also permits the inspector to observe directly the type of soil in which the shaft is being founded. He cannot do so, of course, with driven piles.

There is, however, a singular disadvantage to using drilled shafts: the engineer is never completely sure of the structural integrity of the concrete beneath the ground surface unless the completed shaft is cored and the core carefully inspected. This problem is particularly evident when temporary casing is used to hold back sloughing soil or waterbearing soil and then removed as concrete is placed. Lack of structural integrity is very serious when drilled shafts are carried to bedrock for the purpose of giving shafts a high capacity. Furthermore, it is generally hard to terminate a drilled shaft in waterbearing sand, and problems with loss of ground are encountered in very soft clays.

## Construction Procedures

Three distinct methods exist for constructing drilled shafts. The first, the dry-hole procedure, permits rapid operation. It is employed in the absence of waterbearing sands and when drilling and concreting can be completed in a time span short enough to circumvent sloughing in clays and silts. It involves merely augering a hole, belling the base if desired, installing minimal reinforcing, and backfilling the hole with concrete.

The second method, used in soils where dry-hole drilling is not possible, involves using drilling mud to advance to hole, casing the hole whenever an impermeable founding stratum is reached, removing the drilling mud by pumping or bailing, placing the concrete, and finally removing the casing before the concrete begins to set up. This procedure is in common use today (1971) in the coastal areas of Texas.

The third method, called the direct displacement method, is similar to the second method, except that fluid concrete is used to displace the drilling mud directly. No casing is inserted, and the concrete is pumped or allowed to flow by gravity from the bottom of the hole toward the top through an initially closed tremie. Such a procedure involves the use of very high slump concrete and precise control of the viscosity of drilling fluids.

Only results from the first two procedures will be described in this report since they were the only two procedures investigated in the study. However, further investigations will be undertaken in a separate study in 1971 to determine the feasibility of using the third method, the direct displacement method, for the construction of drilled shafts in caving soils, primarily waterbearing sands.

Design: General Considerations

Two factors must be considered in the design of drilled shafts. First, there must be an adequate factor of safety against bearing failure. Second, settlement of drilled shafts at working load must be limited to a value that will not cause structural or esthetic damage to the bridge they support. The design criteria developed during this study incorporate these two factors.

In expansive soils embedment must be adequate to prevent excessive heave. In such soils, lower portions of the shaft may go into tension as the upper soils swell; hence, adequate reinforcement must be provided. Shafts may be anchored in expansive soils by belling into a stable, nonexpansive stratum. In many areas of the Southwest, stable strata are not reached at reasonable depths, and heave will occur despite the best efforts of the designer and drilling contractor. Many clay shales present a problem in this respect. In such cases, the structure must be flexible enough to withstand differential movements or an alternative foundation design employed.

Since drilled shafts resist load through a combination of end bearing and skin friction, the capacity of a drilled shaft can be calculated either by employing preemptive values for end bearing and side friction based on a physical description of the soil (O'Neill and Reese, 1970), or by a rational limiting equilibrium procedure. The design procedure recommended herein employs the limiting equilibrium procedure, in which Eq. 1 is used:

$$\left(Q_T\right)_{ult} = \left(Q_S\right)_{ult} + \left(Q_B\right)_{ult} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

$\left(Q_T\right)_{ult}$ is the ultimate axial load capacity of the shaft,

$\left(Q_S\right)_{ult}$ is the ultimate capacity of the sides,

$\left(Q_B\right)_{ult}$ is the ultimate capacity of the base.

The ultimate side and base capacities are calculated independently from results of laboratory tests on representative soil samples or from subsurface penetrometer soundings.

The following expressions are used to calculate the ultimate side and base resistances in predominantly clay profiles:

$$\left(Q_S\right)_{ult} = \alpha_{avg}\ S\ A_S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

$$\left(Q_B\right)_{ult} = N_c\ c\ A_B \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where

$\alpha_{avg}$ is the ratio of the peak mobilized shear stress to the shear strength of the soil averaged over the peripheral area of the stem,

S is the shear strength of the soil,

$N_c$ is a bearing capacity factor,

c is the average undrained cohesion of the soil for a depth of two base diameters beneath the base ("Shear strength" may be substituted for "cohesion" for soils having an undrained angle of internal friction of 10 degrees or less),

$A_S$ is the peripheral area of the stem,

$A_B$ is the area of the base.

Many studies have been reported in which the values for $\alpha_{avg}$ and $N_c$ have been measured for driven piles. However, since the disturbance and stress changes in the soil due to the installation of a drilled shaft are not the same as for driven piles, it is not logical to assume that $\alpha_{avg}$ and $N_c$ are the same for drilled shafts and driven piles. This fact was a principal reason for initiation of this research study.

The factor $\alpha_{avg}$ is always less than unity. The reduction in unit side shear capacity to a value less than the shear strength of the soil is due to several factors, including:

1. Remolding of the borehole walls during drilling.
2. Opening of cracks or fissures in the soil during and after drilling.
3. Migration of excess water from concrete into the soil, thereby softening (and weakening) the soil.
4. Shrinking of surface soils and mechanical interaction between the shaft and soil near the base (O'Neill and Reese, 1970).
5. Use of drilling mud during construction.

The behavior of drilled shafts in sands will be described briefly in Chapter III.

## Summary of Results

Briefly, the research has shown that utilization of side shear, together with end or point bearing is a logical and safe design procedure if the behavior of drilled shafts and the supporting soils are properly understood by the designer. The soil strength can be evaluated by undrained triaxial compression tests, unconfined compressions tests, and/or penetrometer correlation. The side shear should be utilized only below the point of maximum scour or below the zone of significant moisture fluctuation. The soil stratum in which the drilled shaft is terminated should be of uniform or increasing strength for at least two base diameters below the proposed depth of termination. It is not necessary that the wall of the excavation for the drilled shaft be smooth. In fact, it is desirable to leave it purposely rough in order to enhance bonding between concrete and the soil, but the wall should not contain large pockets if drilling fluid is to be used because the drilling fluid will be trapped in the pockets as the concrete is introduced, thus lowering the load transfer between the shaft and the soil. A concrete slump of at least six inches is desirable.

# CHAPTER II

# DRILLED SHAFTS IN CLAYS

## Previous Studies

A number of field studies of drilled shafts in clay soils have been conducted by other investigators. The results of those tests are summarized by O'Neill and Reese (1970). Essentially, those investigators have found that in stiff clays an average of approximately one-half of the undrained shear strength is mobilized ultimately along the periphery of the shaft. Furthermore, the tip capacity is approximately nine times the undrained cohesion intercept times the base area. These results, however, were valid for only drilled shafts installed in the dry. Little information is available in the literature concerning the variation of mobilized shear stress with depth or with displacement, and almost no information is available concerning the development of side shear and base resistance in shafts installed with the aid of drilling fluids.

## Center for Highway Research Study

*Austin Test Shaft*. In order to evaluate the parameters governing the development of load transfer in clays, several test shafts were instrumented, constructed and load tested by the Center for Highway Research. The first test shaft was constructed in Austin (Site I, Fig. 1) in 1966, primarily for the purpose of developing instrumentation systems and testing procedures (Reese and Hudson, 1968). The instrumentation schemes developed for this shaft were followed for the remainder of the test program, except that refinements were made in the strain transducers as more information was gained about their behavior.

The Austin test shaft and all subsequent shafts were instrumented by embedding strain transducers at several levels. The output of each transducer was converted to load in the shaft by comparing the signal to that of a set of calibration transducers at the ground surface. Instrumentation is discussed in detail by Vijayvergiya, Hudson, and Reese (1969), Barker and Reese (1969), and O'Neill and Reese (1970).

Loads were applied to each test shaft by hydraulic rams which jacked against a reaction frame anchored on each side of the test shaft by a single drilled shaft. Tests were conducted to failure loads in the range 100 to 1,000 tons. Loads were applied in small increments every 2.5 minutes. Time to failure was less than four hours in every case, thus producing essentially undrained failure in the soil.

Each time a load was applied to the butt of a shaft, the distribution of load along the shaft was obtained by reading the transducers at each level with a digital scanner. The load transfer relationship was calculated at several depths by finding the slope of the load-in-shaft versus depth curves for a number of values of displacement. From a family of such relationships, a complete picture of the way in which soil resisted load was obtained.

San Antonio Test Shaft. The second test shaft, described by Vijayvergiya, Hudson and Reese (1969), was an instrumented shaft in stiff fat clay and clay shale at the test site in San Antonio (Site II, Fig. 1). The soil profile at that site is shown in Fig. 2. The test shaft was installed in the dry, was 30 inches in diameter, and was terminated at

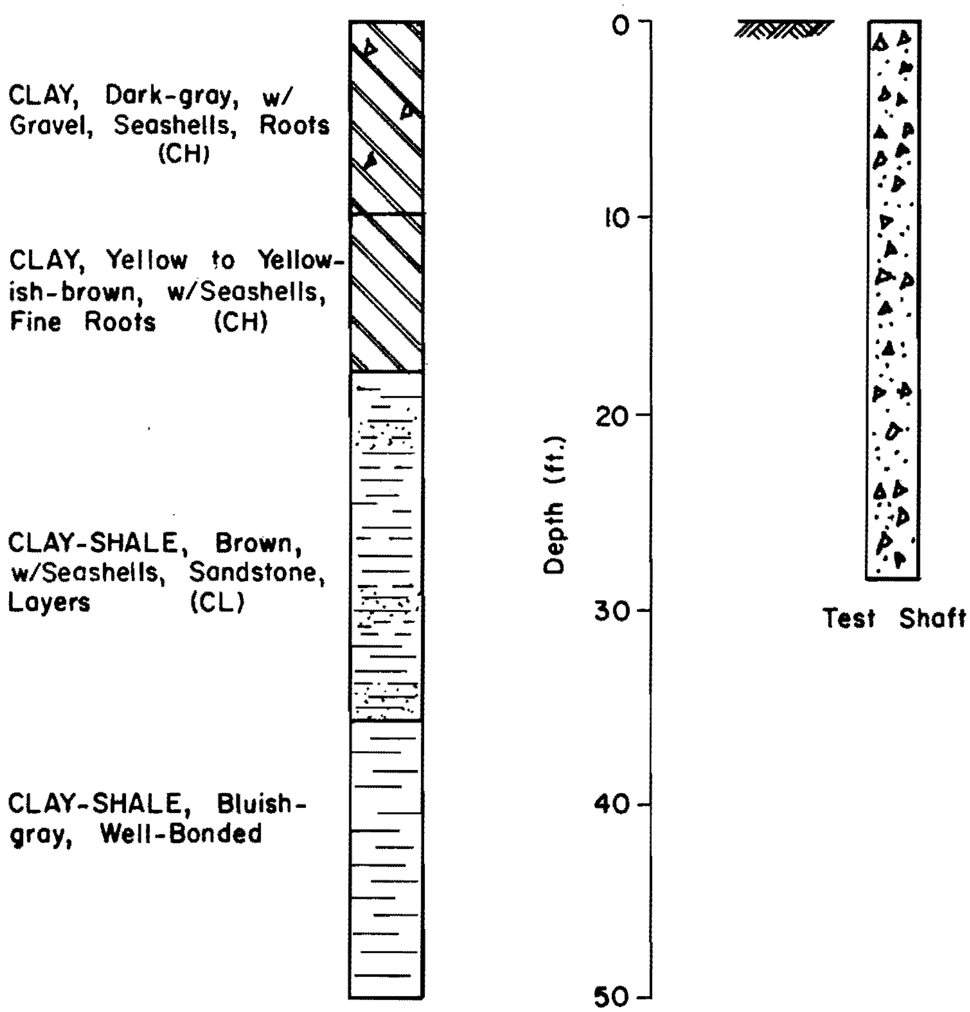



Fig. 2. Soil and shaft profiles, San Antonio test site (Site II) (After Vijayvergiya, Hudson, and Reese, 1969).

a depth of approximately 28.5 feet. The engineering properties of the soil and results of several load tests on that shaft conducted during 1967 and 1968 are reported in detail by Vijayvergiya, Hudson, and Reese (1969). One principal finding was that, on a random testing basis, the first fifteen feet of penetration was ineffective in resisting side shear. This was due to the expansive nature of the soils and due to the fact that some of the tests were conducted during dry periods. It was not possible to establish the maximum depth to which soil expansion may occur for design purposes because no tests were run during extreme droughts. However, on the basis of the several tests conducted at random on the San Antonio Test Shaft, it may be assumed that shearing resistances should be neglected in at least the top 15 feet of drilled shafts in that geographical area. The second conclusion was that the maximum side load transfer $(q_S)_{ult}$ may be related to the THD cone penetrometer test by Eq. 4.

$$\left(q_S\right)_{ult} = \frac{N}{35} \qquad (4)$$

where N is the number of blows per foot of a standard THD cone penetrometer and $(q_S)_{ult}$ is in tons per square foot.

Although difficulty was encountered in obtaining undisturbed samples for laboratory testing, it appeared that the shear strength of the soil at the test site varied between about one and four tons per square foot; hence, Eq. 4 appears to be valid in that range of soil strength. This equation was later verified at the State Highway 225 Test Site in Beaumont Clay in Houston, as will be discussed later, for shear strengths in the range of 1.0 to 1.5 tons per square foot.

The authors proposed the following equation for computation of the ultimate base bearing capacity of a drilled shaft from THD cone penetrometer soundings:

$$\left(Q_B\right)_{ult} = 3\ A_B\ N \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where $A_B$ is the cross-sectional area of the base in square feet.

The factor 3, which is both an emperical constant and a dimensional factor, was found to vary between about 1.9 and 3.8 at other clay soil sites. Hence, calculation of the base capacity strictly by using this equation may lead to fairly large errors; however, Eq. 5 represents a reasonable first estimate of the base capacity of a drilled shaft.

State Highway 225 Test Shafts. Following the completion of the tests in San Antonio, a test site at State Highway 225 and South Loop East in Houston, Texas, was selected for testing drilled shafts in the Beaumont Clay formation. The site was a basic research site in which several specific parameters were studied.

The parameters investigated were method of installation, shaft geometry, and soil conditions. Four shafts were installed; three of them were installed by the dry method. The fourth was installed with the drilling mud and casing technique, and the behavior of that shaft was compared with the other three (O'Neill and Reese, 1970). The soil and shaft profiles for this site (Site III) are shown in Fig. 3. It was found from these tests that when drilling mud is used in excavating, further reduction in the operational strength of the soil along the walls of the shaft can occur due to entrapment of drilling mud between

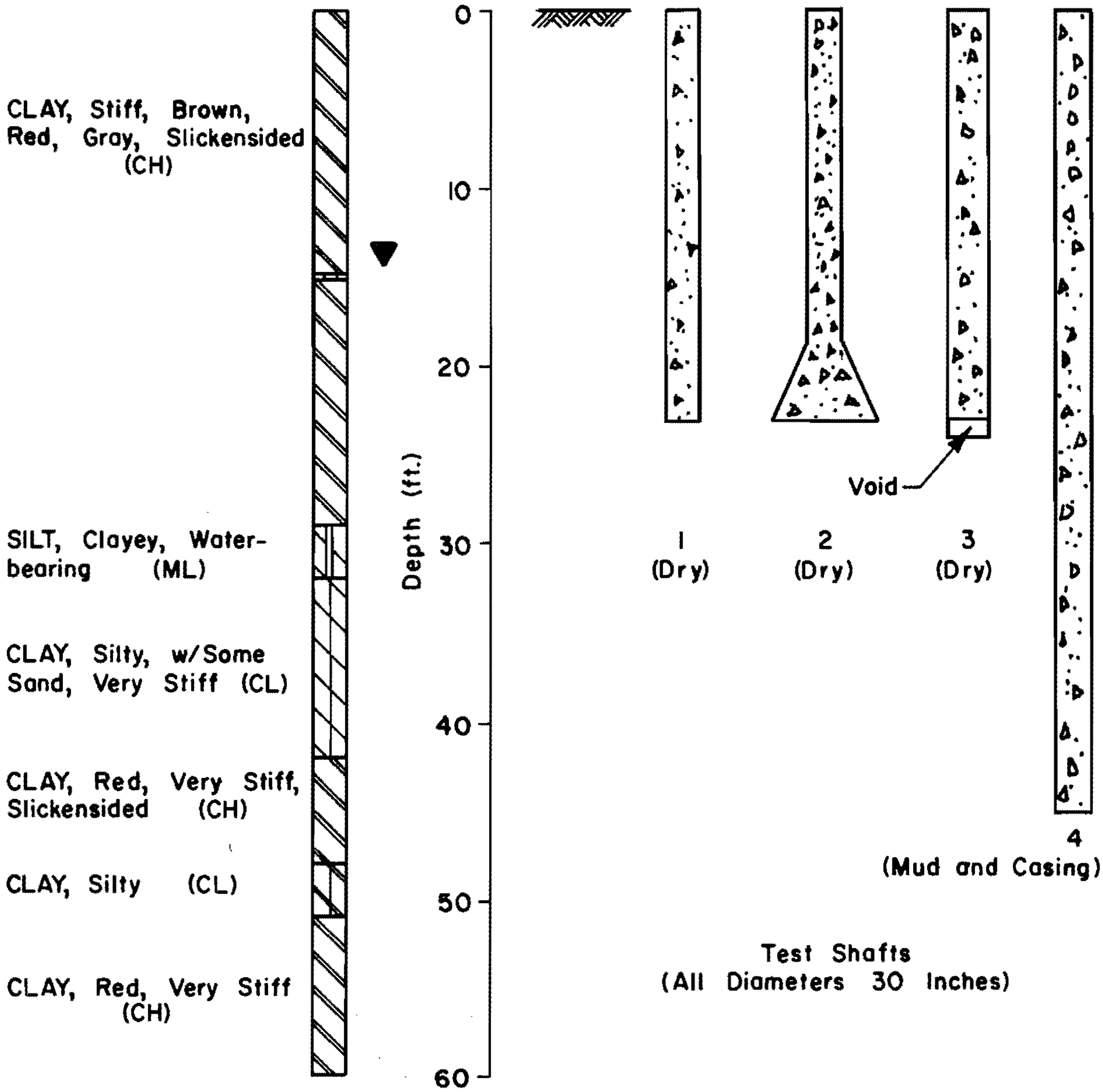



Fig. 3. Profiles of soil composition and test shafts, 5H225 test site (Site III) (After O'Neill and Reese, 1970).

the concrete and natural soils. The maximum average unit side shearing resistance in the three test shafts installed in the dry was approximately one-half of the undrained shear strength of the soil as indicated by triaxial compression tests on 1.4-inch diameter specimens. The average peak load transfer along the sides of the shaft installed with the aid of drilling mud was approximately one-third of the shear strength so indicated.

The patterns of distribution of shear stresses along the sides of the shafts were also investigated carefully during the test at State Highway 225. The shear stress distribution was approximately parabolic in the three shafts installed in the dry, with the largest shear stress being observed near the center of the shaft. Apparently, surface effects reduced the mobilized shear stress near the top and a base-soil interaction effect and migration of water from the test shaft reduced the load transfer near the bottom (O'Neill and Reese, 1970). In the test shaft installed with the aid of drilling fluid a larger shear stress was noted near the base. This effect was due to the fact that the last five feet of the shaft were installed in the dry, that is, without drilling mud having contacted the soil.

The three test shafts installed in the dry were placed in a relatively homogeneous stiff, fissured clay stratum. They were all 30 inches in diameter and 23 feet deep. They differed only in their base geometry. One was a perfectly cyclindrical shaft, one had a 7.5-foot-diameter bell, and one was cast above a void. The behavior of all three shafts in their resistance to load along their sides was very nearly the same, indicating that, at least for short-term behavior, the base geometry has a small effect on the development of shearing resistance along the sides. The fourth shaft, which was twice the length of the first three, and which had no bell, had a much different pattern of load transfer development due to the fact that it was installed through several different layers of soil and because drilling mud had been used in the installation. Little information could be obtained concerning the effect of soil type on load transfer from the fourth test shaft due to the masking effect of the mud.

Although little effect of base geometry was noted in these tests, it is not possible to say that considerable load shedding will not occur on a long-term basis for belled drilled shafts; however, no long-term tests were run in the present study.

HB&T Test Site. Another test site in Houston (Site IV) was selected at the IH610 crossing of the HB&T Railroad between Hardy and Gold Streets. A single test shaft, installed in alternating thin layers of clay, sand, and silt by using drilling mud and casing, was 36 inches in diameter and had a 60-foot penetration. The shaft and soil profiles are shown in Fig. 4. The results of the load tests on this instrumented shaft are reported in detail by Barker and Reese (1970). The results of the tests did not differ significantly from those obtained on the test shafts at the State Highway 225 site, except that there appeared to be no reduction in the mobilized load transfer due to using drilling mud.

Following the completion of the load test on all five shafts in Houston, access bore holes were drilled adjacent to the shafts, and the shafts were inspected by personnel from the Center for Highway Research. These inspections revealed that no drilling mud was trapped adjacent to

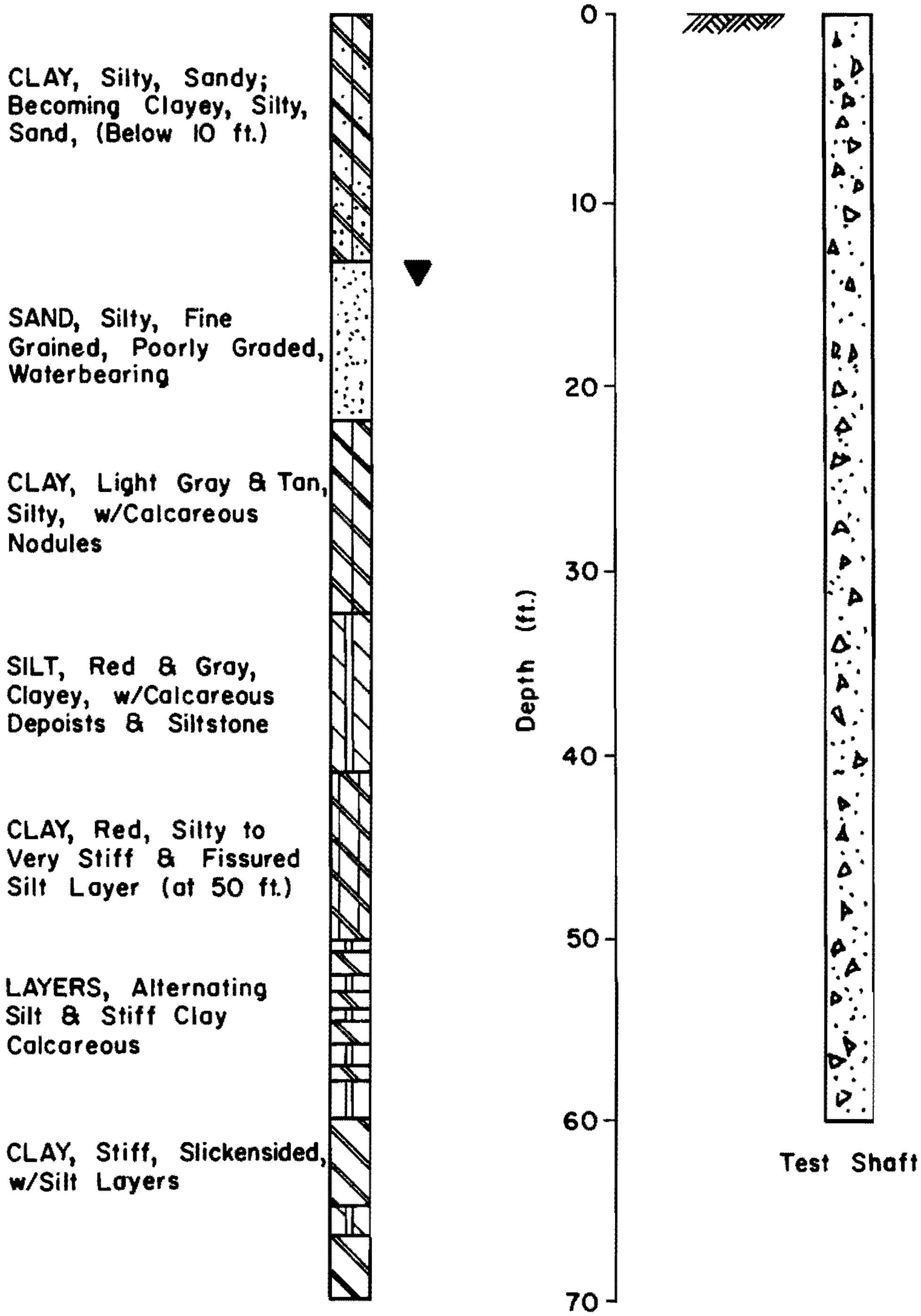



Fig. 4. Soil and shaft profile, HB & T test site (Site IV) (After Barker and Reese, 1970).

the top part of the shaft at the HB&T Test Site, while considerable amounts of drilling mud were trapped adjacent to the drilled shaft installed with drilling mud at the State Highway 225 Test Site. The results of these tests and observations of other installations lead to the conclusion that the entrapment of drilling mud is a random process in constructing drilled shafts by the mud and casing method. Therefore, it appears prudent, from the design standpoint, to assume that drilling mud will be trapped and to design for side friction accordingly.

On the basis of load tests at the HB&T Test Site and the State Highway 225 Test Site, Eq. 6, below, was found to be appropriate for computing the ultimate base capacity of both belled and straight-sided shafts in saturated clay soils.

$$\left(Q_B\right)_{ult} = N_c \; c \; A_B \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where $N_c$ is a bearing capacity factor equal to 9 and $c$ is the average undrained shear strength of the soil for a distance of two base diameters beneath the base.

Equation 6, of course, requires that undisturbed soil specimens be obtained in order to evaluate the parameter $c$ . When THD cone penetrometer soundings are made, Eq. 7 appears appropriate for calculation of ultimate base capacity:

$$\left(Q_B\right)_{ult} = \frac{N}{2.8} \; A_B \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

## Categories of Design in Clay Soils

Based upon the tests in San Antonio and Houston it appeared that four categories of design for drilled shafts in clay exist. They are:

Category A: Straight-sided shafts in either homogeneous or layered soil with no soil of exceptional stiffness below the base.

Category A.1: Shafts in Category A installed dry.

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

Category B: Belled shafts in either homogeneous or layered clays with no soil of exceptional stiffness below the base.

Category B.1: Shafts in Category B installed dry.

Category B.2: Shafts in Category B installed with drilling mud along some portion of the hole such that the 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.

Category D: Belled shafts with base resting on soils significantly stiffer than the soil around the stem.

Shafts in any category can be designed according to a Primary Procedure, in which triaxial test data are used, or an Alternate Procedure, in which penetrometer data are used. When the Alternate Procedure is used, the unit ultimate base or side capacity is computed by dividing the number of blows per foot in a given zone by a correlation factor $p$ (side shear capacity) of $p'$ (base capacity). (For straight shafts installed in the dry, $p$ is 35 and $p'$ is 2.8, for example.) Based on the field tests, the various parameters and limiting resistances to be used in calculating the ultimate capacity of a drilled shaft in each of the four categories previously enumerated are evaluated in Table 1. When the calculated side resistance is greater than the tabulated limiting value shown in Table 1, the limiting value should be used because insufficient data have been accumulated concerning development of side resistance in very stiff soils. Further extensive load testing in hard clays and clay shales will, in all probability, show that these limits are conservative.

Table 1 is intended for use in the design office. There is a delineation between $\alpha$ factors used when standard triaxial tests are employed for obtaining shear strength and when the Houston Urban Expressways Office multiple phase triaxial procedure is employed. The Houston Urban Expressways Office procedure incorporates a large (three-inch-diameter by six-inch-long) not completely failed specimen in each phase of the test. Hence, the procedure for testing undisturbed specimens of soil is reflected in the slightly higher allowable values for $\alpha_{avg}$ whenever that procedure is used. Those values for $\alpha_{avg}$ should not be used when other laboratory test procedures are employed for obtaining shear strength of clays.

The research has indicated that the bottom five feet of the stem should be considered as noncontributing to side shear because of the excess softening and base-soil interaction described by O'Neill and Reese (1970). None of the peripheral area of a bell should be considered in calculating the ultimate side shear. (See Fig. 5.)

An adequate number of triaxial tests must be conducted when determining the shear strength profile in fissured clay for design purposes. Enough tests must be run to establish a correct average shear strength

TABLE 1. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY.

| | Parameter | Design Category | | | | | |
|---|---|---|---|---|---|---|---|
| | | A.1 | A.2 | B.1 | B.2 | C | D |
| Primary Procedure, Standard Laboratory Triaxial Tests | $\alpha_{avg}$ | 0.5 | $0.3^{A}$ | 0.3 | $0.15^{C}$ | 0 | 0 |
| | Limit on Side Shear (tsf) | 0.9 | $0.4^{B}$ | 0.4 | $0.25^{D}$ | 0 | 0 |
| | $N_c$ | 9 | 9 | 9 | 9 | 9 | 9 |
| Primary Procedure, HUE Multiphase Triaxial Tests on 3-Inch Diameter Samples | $\alpha_{avg}$ | 0.65 | $0.4^{A}$ | 0.4 | $0.20^{C}$ | 0 | 0 |
| | Limit on Side Shear (tsf) | 0.9 | $0.4^{B}$ | 0.4 | $0.25^{D}$ | 0 | 0 |
| | $N_c$ | 9 | 9 | 9 | 9 | 9 | 9 |
| Alternate Procedure, Cone Penetrometer Soundings | P | 35 | $60^{A}$ | 60 | $120^{C}$ | 0 | 0 |
| | Limit on Side Shear (tsf) | 0.9 | $0.4^{B}$ | 0.4 | $0.25^{D}$ | 0 | 0 |
| | $p'$ | 2.8 | 2.8 | 2.8 | 2.8 | 2.8 | 2.8 |

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

B Limiting side shear = 0.9 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 = 0.4 tsf for segments of shaft drilled dry

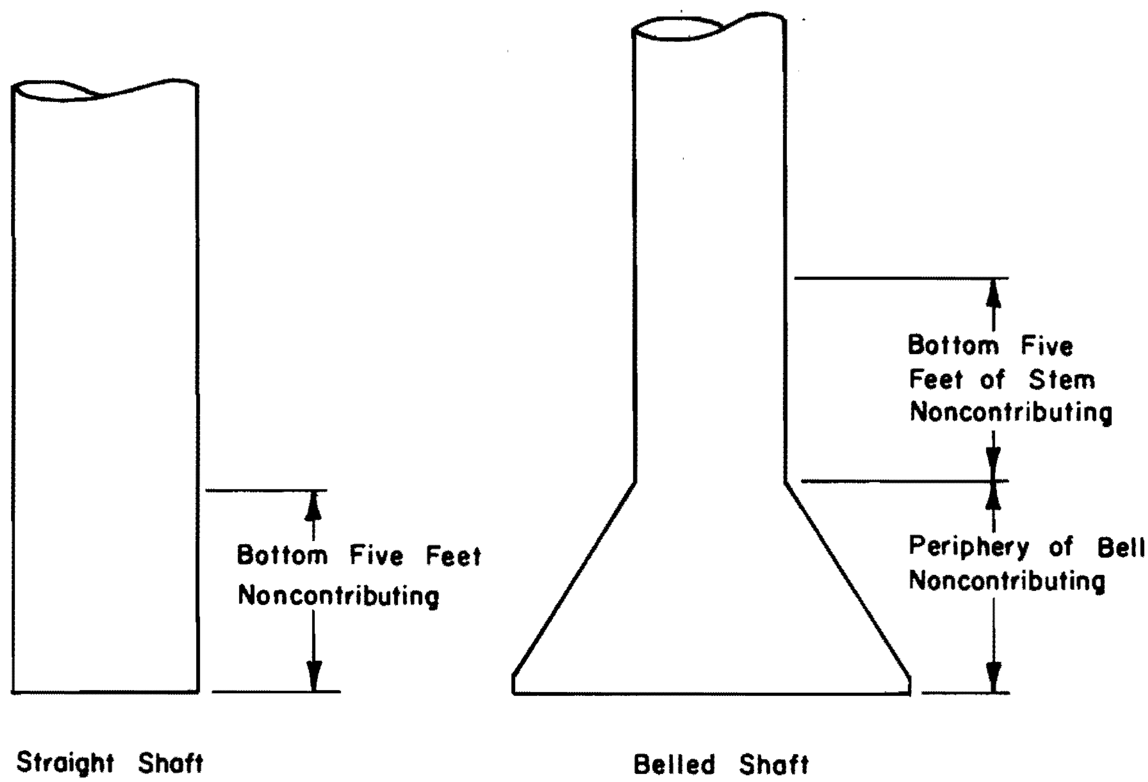



Fig. 5. Noncontributing zones for drilled shafts in clay.

can occur. A suggested procedure is to obtain at least one triaxial test per foot of hole in such soils. When an adequate number of tests cannot be performed, the $\alpha$ factors should be reduced according to the degree of uncertainty in accuracy of indicated shear strength.

Although no tests have been conducted on drilled shafts installed by the direct displacement method, one problem in such shafts may be that a firm bearing surface from the base may be difficult to obtain. Therefore, it is necessary to clean out the base immediately before placement of concrete when constructing shafts by that procedure. Furthermore, until results can be obtained from load tests on shafts installed by the direct displacement method, it is suggested that the design values for $\alpha$ from categories A.2 or B.2 be utilized.

The allowable maximum side shear values for belled shafts are less than those for straight shafts because the sides of a belled shaft will be in a failed condition at design load if the shaft is designed properly. That is, in order to mobilize an overall factor of safety of approximately 2 or 2.5, the deflection of a five-foot-diameter base must be in the neighborhood of one-half to one inch. Since the soil supporting the sides generally fails at a downward displacement of approximately 0.2 inches, the sides would be in a failed condition. In an overconsolidated clay the soil would then relax considerably with time causing a reduction in the amount of load that is resisted along the sides. This phenomenon is discussed by O'Neill and Reese (1970).

In Categories A and B, the top five feet should be considered as non-contributing in the Beaumont Clay soils of the Houston area and the top fifteen feet considered noncontributing in the upper Cretaceous clays and clay shales of the San Antonio area. In other locations the design engineer must still determine the depth to which side resistance should be ignored, pending performance of load tests on instrumented shafts in other soils and under different climatological conditions.

When designing under Categories C and D, no side resistance is allowed because downward displacements are likely to be too small to mobilize significant shear, especially when shafts are carried to bedrock. Concrete stress will often be very high for shafts in these categories. Hence, special attention must be paid to obtaining proper concrete integrity, such as by avoiding the use of temporary casing, and to insuring that the base of the shaft is free from loose material.

The use of Table 1 will be explained in several example problems in Chapter V.

## CHAPTER III

## DRILLED SHAFTS IN SANDS

### General

There is a notable lack of information concerning the behavior of drilled shafts in sands. It appears that the action of augering a bore-hole in medium and dense sands reduces the density of the sand surrounding the hole. The soil beneath the base appears particularly to be effected. The ultimate base capacity may be the same as would be computed using bearing capacity expressions from standard soil mechanics textbooks, but that capacity apparently is mobilized only at a very large displacement and is not a practical value to use. Considerable judgment is presently required in designing drilled shafts in sands. Load tests are indicated when it is feasible to employ them.

Where sand-clay soils appear in a predominantly clay profile, it appears justified to use the design parameters discussed in Chapter II for clay in computing the capacity of the drilled shaft.

### Live Oak County Tests

In 1970, two test shafts were constructed and tested by the Center for Highway Research in predominantly sand profiles in Live Oak County, Texas (Sites V and VI, Fig. 1). The results of those tests are presently being analyzed in light of soil data which has recently been obtained. A comprehensive report has not yet been written; however, some preliminary results are available, and they are presented briefly herein. Final results will be available in a future report from the Center for Highway Research.

To illustrate the behavior of drilled shafts in sand and to provide design guidance, a very brief discussion of the behavior of the test shaft at Site V (U.S. 59 and State Highway 9) will be discussed briefly. The test shaft at that site was fully instrumented in a fashion similar to the test shafts in Houston and installed to a penetration of 32.5 feet. The soil and pile profiles are shown in Fig. 6. The test shaft was 30 inches in diameter and was unbelled. Figure 6 also contains a plot of the number of blows per foot for both the THD cone penetrometer and the standard split spoon penetrometer at the U.S. 59 Test Site. There were intermittent layers of silt and sand at the site; the sand was not saturated and it was lightly cemented. The water table lay at a considerable depth beneath the base of the shaft. With this combination of conditions, it was possible to install the shaft in the dry even though it was terminated in sand. The shaft was reasonably well-formed and presented no particular difficulty in installation.

The load-settlement curve of the initial load test on the U.S. 59 Test Shaft is shown in Fig. 7. The rapid increase in downward deflection with load is particularly noteworthy. This behavior is quite different from that noted in testing drilled shafts in saturated clays. Drilled shafts in clays exhibit an initial elastic load-settlement response and then suddenly plunge to failure. The base of the U.S. 59 Test Shaft picked up load very slowly. In the range of the test the base load curve was nearly linear. It appears from the shape of the load-settlement curve that a criterion for determining permissible design load involving any emperical procedure, such as the double tangent method described by

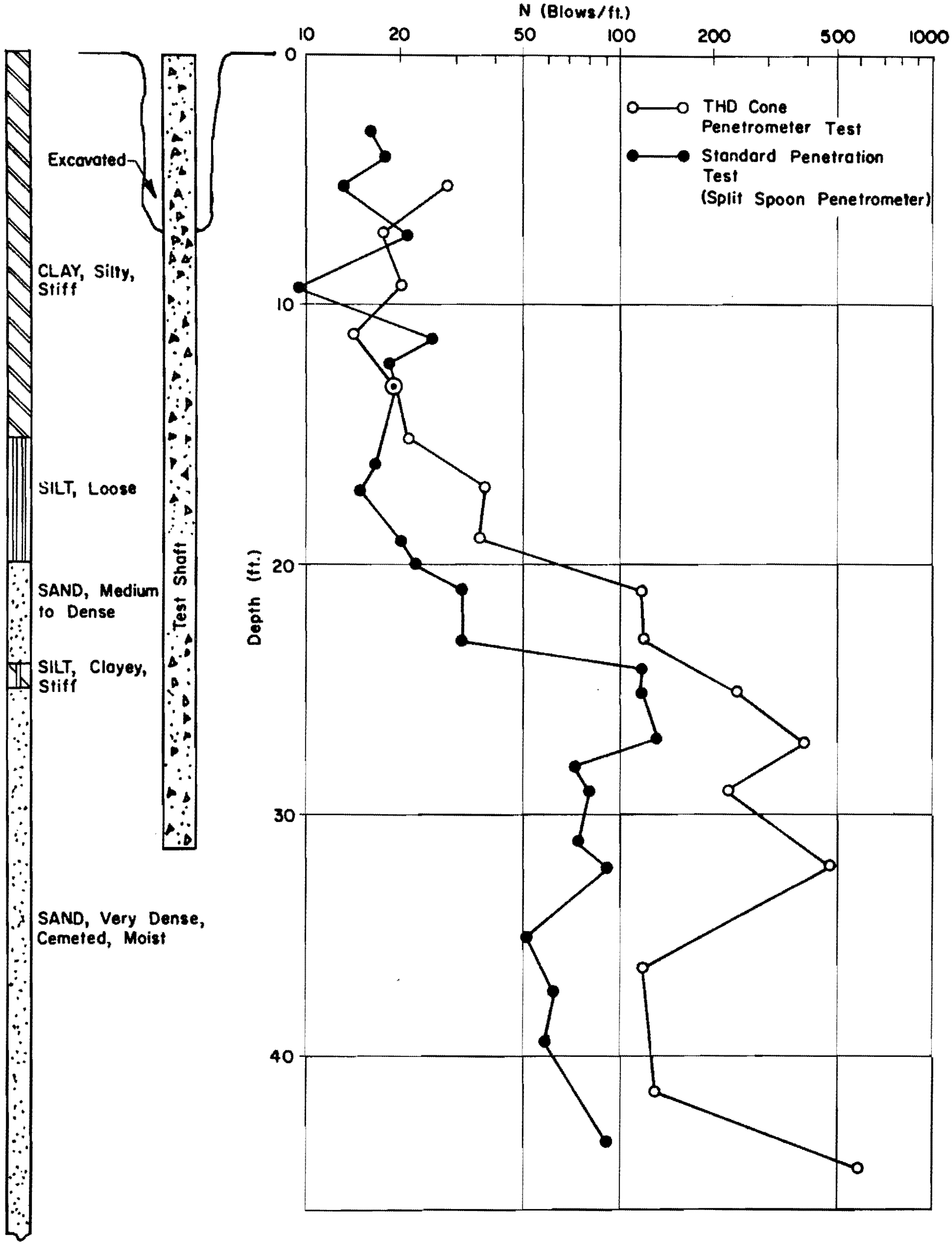



Fig. 6. Soil and shaft profiles, U.S. 59 Test Site (Site V).

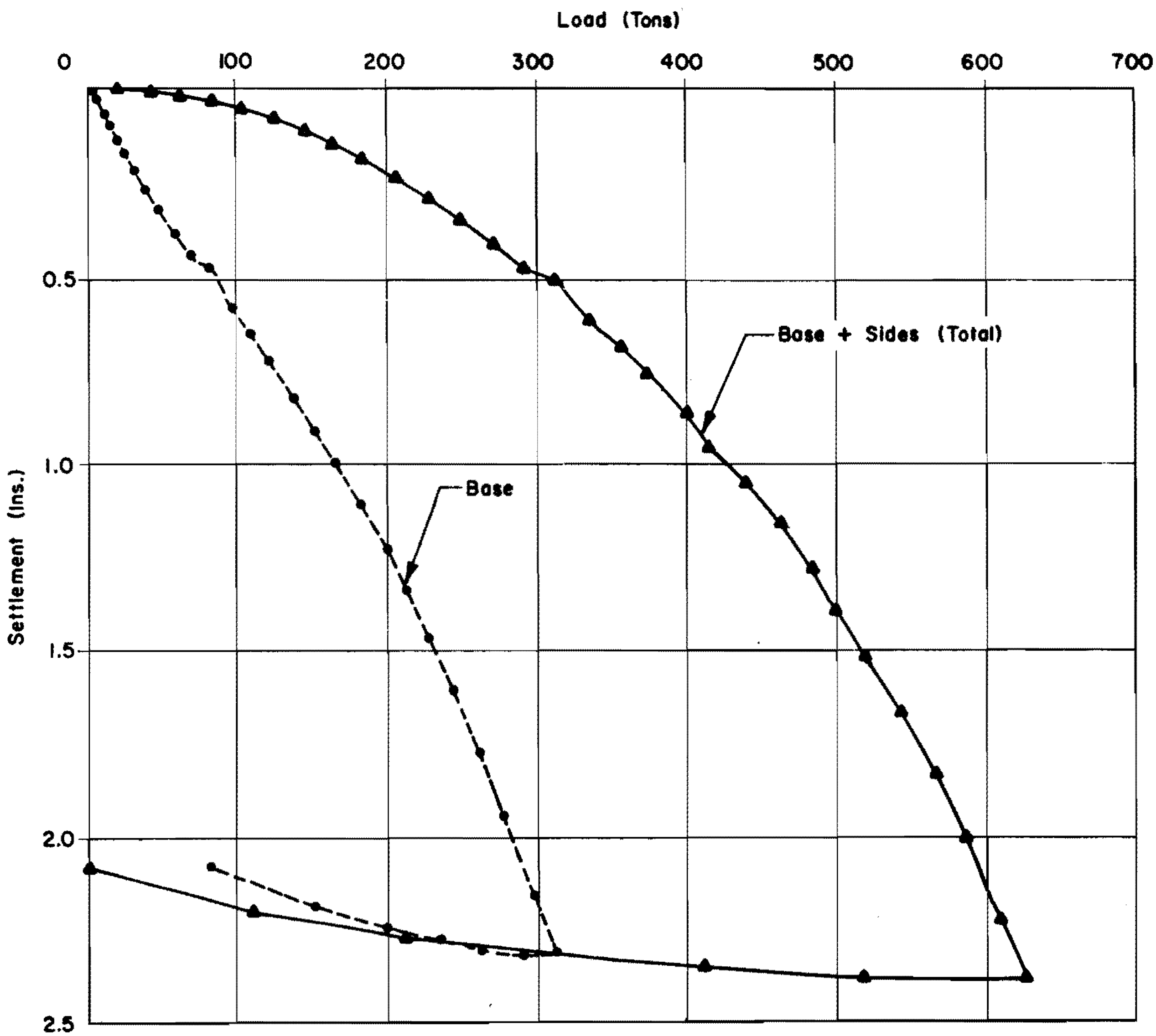



Fig. 7. Load-settlement curves for U.S. 59 test shaft.

O'Neill and Reese (1970), is probably not valid. Instead, one must use settlement as a criterion. In order to evaluate the permissible load on this shaft, it will be necessary that the structural engineer, by analyzing the behavior of the structure, establish a maximum permissible settlement. Only then can the design load be determined. For example, if the maximum permissible settlement on any single shaft is set at 0.4 inches, the design load, according to Fig. 7, would be about 270 tons.

Since the "flexible" load-settlement behavior apparently resulted from a loosening of the soil as the borehole was drilled and overburden released, a reloading of the shaft indicated a "stiffer" behavior. Upon loading the shaft a second time the initial portion of the load-settlement curve was approximately three times as stiff (settlements at corresponding loads were one-third as great) as on the first loading and remained nearly linear up to a point approaching the value of the maximum load on the first test. Hence, a single loading vastly improved the load-settlement qualities of the drilled shaft by redensifying the supporting soil.

The side load transfer behavior also appears to differ from that of shafts installed in clay. Two salient points are evident from the load tests:

1. The load transfer curves in sand and silt for the U.S. 59 Test Shaft do not peak out. An exception is the load transfer curve just above the base, which peaks between one-half and three-quarters of an inch downward displacement because of a base-side interaction effect analogous to that described for shafts in clay by O'Neill and Reese (1970). In sands and silts the load transfer is a function of the

effective stress. Since placing a load on a shaft increases the effective stress in the sand, the magnitude of load transfer continues to increase up to very large displacements. This, of course, is not true for tests in saturated clay conducted over a short period of time, because the effective stress is not allowed to change. The load transfer curves continue to show an increase in load transfer in the U.S. Highway 59 Test Shaft beyond a settlement of two inches.

2. The following values of load transfer were measured at a downward displacement of one-half inch, which would likely be a limiting value for design:

    a. Top silty clay: 0.6 tsf.

    b. Silt zone (15 feet to 20 feet): 0.1 tsf.

    c. Medium dense sand (20 feet to 24 feet): 0.3 tsf.

    d. Very dense sand (25 feet to 32 feet): 2.0 to 2.5 tsf.

In sand the load transfer appears to increase with depth, except in the vicinity of the base, where it is reduced.

Since it is difficult to take undisturbed specimens of sands, it appears that future design criteria will be based either on a static or a dynamic penetration test. Results of both types of tests are currently being analyzed, and criteria for the more appropriate type will be developed. The values given in the previous paragraph for load transfer are not intended as design values but are only intended to illustrate the way in which the shaft behaves. It still remains to determine what parameters most significantly affect the load transfer for drilled shafts in sands. When these parameters can be isolated and analyzed, design procedures will be developed and subsequently reported.

# CHAPTER IV

## ESTABLISHING PERMISSIBLE DESIGN LOADS

The capacities of drilled shafts computed in using Table 1 are ultimate (plunging) capacities. They must be reduced by a factor of safety or load factor in order to arrive at a safe design load. For drilled shafts installed completely in clay soils, two criteria should be checked concerning the factor of safety:

1. The overall factor of safety at design load should be at least 2.2.
2. The factor of safety on the base at design load must be at least 3.0.

The overall factor of safety of 2.2 against plunging corresponds to a factor of safety of 2.0 applied to the intersection of tangents to the initial and final parts of the load-settlement curve. This double tangent load used in the past by the Texas Highway Department to establish design loads for drilled shafts and piles in clay based on results of load tests.

It is emphasized that both of the above criteria must be met. The former will usually govern, but the latter should be checked to insure that immediate settlement is not excessive. If the base diameter is larger than 9 feet, the factor of safety on the base should be increased because added settlement will be required to mobilize a given percentage of the base capacity. For a base diameter of 15 feet, a factor of safety of 4 on the base should be specified, and for diameters between 9 and 15 feet the factor of safety should be linearly interpolated between 3 and 4.

For drilled shafts installed in sands, it is not appropriate to establish a numerical factor of safety against plunging. Instead, computed working capacity at a preset value of permissible settlement should be used. Methods of obtaining that capacity based strictly upon analytical techniques have not been developed. Therefore, it is appropriate to conduct load tests whenever possible to determine a safe value for design load.

## CHAPTER V

## PROCEDURES FOR DESIGN

### General

In this chapter, a step-by-step method for calculating the safe design load and establishing penetrations for drilled shafts in principally clay profiles is outlined. For a silt or sand-clay material, the $\alpha$ factors tabulated in Table 1 may be used with a measure of judgment by applying them to the average undrained shear strength of the layer being considered. The outline is followed by several realistic examples which illustrate the use of the design parameters enumerated in the preceding chapters.

The steps in design are as follows:

1. Calculate the required ultimate capacity of the shaft by multiplying the design load by 2.2.
2. Secure an accurate, representative shear strength or penetrometer profile for the soil at the construction site.
3. Make an initial estimate of the depth and diameter of the base and decide whether the shaft will be belled.
4. Determine whether drilling mud will be required. If it is, estimate the extent of the zones in which mud will come into contact with the sides of the borehole.
5. Calculate the ultimate base resistance for the shaft in the founding stratum chosen for the trial design.

a. Use the bearing capacity formula given in Eq. 3 or calculate the bearing capacity by dividing the N value by the penetrometer correlation factor p' . The appropriate penetrometer correlation factor p' is given in Table 1.

b. Make an appropriate reduction when concrete is used to displace mud directly if it appears that mud or sloughing will be trapped between the concrete and the natural soil beneath the base.

6. Calculate the required ultimate side resistance by subtracting the calculated ultimate base resistance from the required ultimate capacity.

7. Construct a cumulative ultimate side resistance curve from the soil strength profile with the aid of the shear strength reduction factors $\alpha$ tabulated in Table 1. Be sure to neglect any resistance developed in the top five feet in Beaumont Clay or top fifteen feet in the expansive upper Cretaceous clay shales or residual clayey overburden such as found in San Antonio.

8. From the cumulative curve, determine the depth required to develop the desired ultimate resistance. Add five feet to the depth at which the required resistance has been developed to locate the bottom of the stem. Add to that the height of the bell to locate the base of a belled shaft, if applicable.

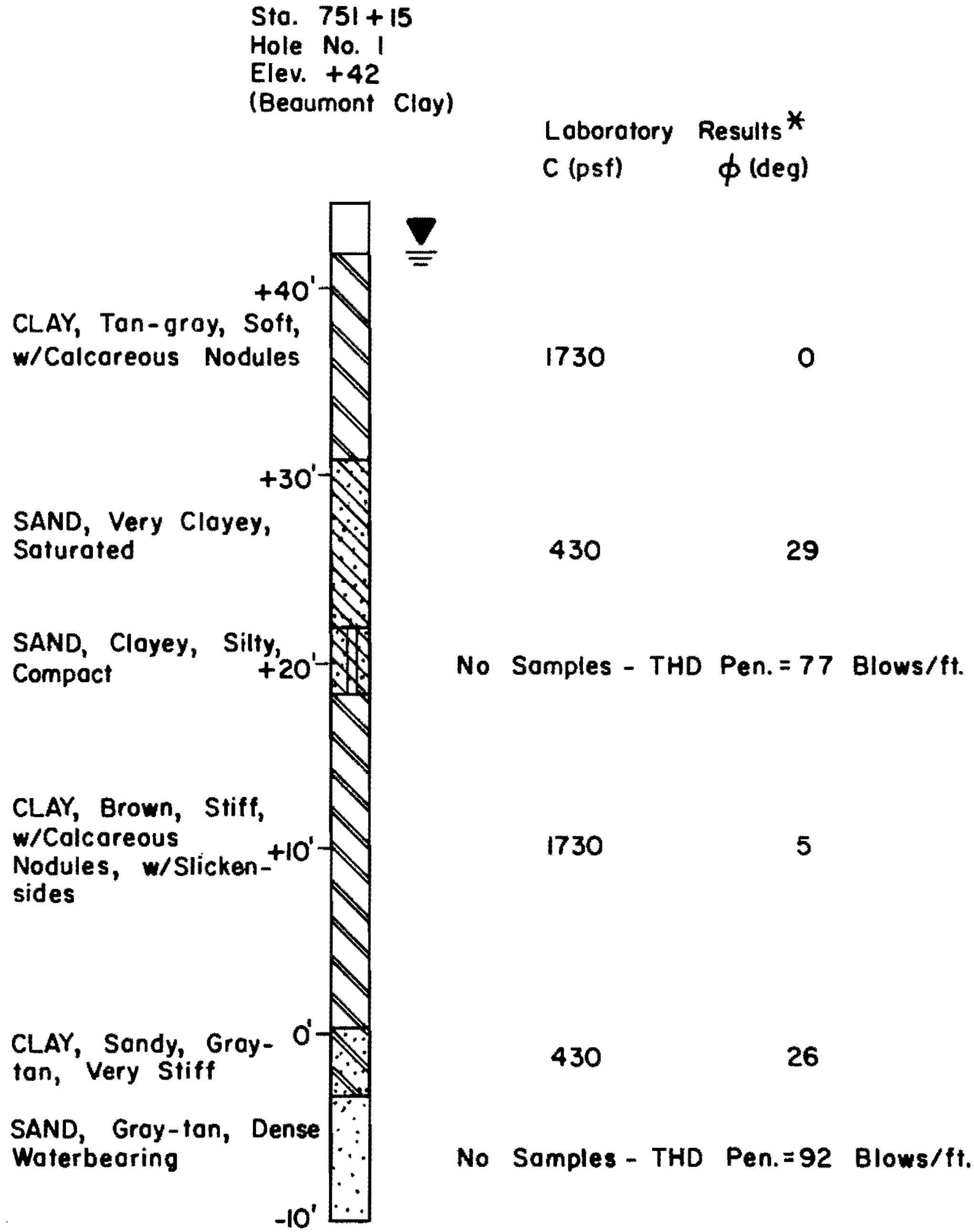
9. If the calculated depth of the base places the base in a stratum other than that which was assumed, or if the base is too near the bottom of the trial base stratum, the geometry of the trial design should be altered, and steps one through eight repeated.

10. Check to see that the sum of the ultimate side resistance and one-third of the ultimate base resistance is greater than the design load for the final design of a shaft founded in clay. Normally this criterion will be met automatically when designing under steps one through nine. Only for relatively short, belled shafts will this criteria govern.

To illustrate this procedure, four example problems are now presented. It is suggested that the potential designer of drilled shafts become thoroughly familiar with these examples and attempt to work through them himself in order to become familiar with the design procedures.

Example Problem No. 1

Given: A drilled shaft foundation is to be designed to carry a load of 90 tons per shaft. Normal foundation exploration, field, and laboratory test data are available and are shown in the following pages. The static water table fluctuates and is near the ground surface on occasion. The soil profile is given in Fig. 8. The shear strength values are tabulated in Table 2. They were from a test other than the HUE multiphase test, so increased $\alpha$ factors may not be used. Structure type and span lengths call for three-foot-diameter columns.




* Values shown are average of several unconsolidated, undrained triaxial tests within stratum.

Fig. 8. Soil profile for test hole No. 1, example problem No. 1.

TABLE 2. EXAMPLE PROBLEM NO. 1, TEST HOLE NO. 1 (FIG. 8).

| Elevation (ft) | Thickness of Stratum, d (ft) | Effective Unit Weight of Soil, w (pcf) | $\phi$[1] (deg) | c[1] (psf) | Shear[2] Strength, S (psf) | Shear[2] Strength, S (tsf) | Ultimate Stress $=\alpha(S)$ (tsf) per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter) per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter) cumulative |
|---|---|---|---|---|---|---|---|---|---|
| +42 to +37 | 5 | 55 | Disregard | | | | | | |
| +37 to +31 | 6 | 55 | 0 | 1730 | 1730 | 0.86 | 0.43 | 2.6 | 2.6 |
| +31 to +22 | 9 | 68 | 24 | 430 | 830 | 0.41 | 0.21 | 1.9 | 4.5 |
| +22 to +18.5 | 3.5 | 60 | THD Pen = 77 Blows/Ft. | | | | (0.9)[3] | 3.2 | 7.7 |
| +18.5 to + 0.5 | 18 | 60 | 5 | 1730 | 1870 | 0.94 | 0.47 | 8.4 | 16.1 |
| + 0.5 to - 3 | 3.5 | 68 | 26 | 430 | 1700 | 0.85 | 0.43 | 1.5 | 17.6 |
| - 3 to -10 | 7 | 70 | THD Pen = 92 Blows/Ft. | | | | (0.9)[3] | 6.3 | 23.9 |

1 Determined from laboratory undrained triaxial compression tests

2 Cohesion plus product of overburden stress and tan $\phi$. Overburden stress is sum of wd from strata above and $w(\frac{d}{2})$ for stratum under consideration.

3 Limiting values

Required: To determine the size and penetration of drilled shafts without bells to carry safely and economically the design load in the vicinity of Test Hole No. 1.

Solution: The design Category is A.1.

1. A study of Fig. 8 indicates that the sand from -3 to -10 may be waterbearing. There are no other free waterbearing strata. The clay stratum directly above is an adequate founding stratum (30 blows per foot) for the shafts, and founding in the clay will obviate the requirement for using drilling mud.

2. Select a diameter of 3.5 feet for the first trial design. A 3.5-foot-diameter shaft has the following geometric properties:

   Perimeter = 10.99 square feet per linear foot

   Base area, $A_B$ = 9.62 square feet

3. Ultimate total capacity required (assuming overall factor of safety of 2.2) = 198 tons

4. Ultimate capacity of the base (base founded in clay stratum between +18.5 and +0.5):

$$\left(Q_B\right)_{ult} = 9\ c\ A_B = (9)\ \frac{1730}{2000}\ \ (9.62)$$

$$= 9\ (0.86)\ \ (9.62)$$

$$= 74.5 \text{ tons}$$

5. Ultimate side resistance required:

$$\left(Q_S\right)_{ult} = \left(Q_T\right)_{ult} - \left(Q_B\right)_{ult}$$

$= 198 - 74.5 - 123.5$ tons

6. Required ultimate cumulative frictional resistance:

$$= \frac{123.5}{10.99} = 11.2 \text{ tons per foot of perimeter}$$

7. A cumulative resistance graph, Fig. 9, is plotted from Table 2. In computing overburden stresses, submerged unit weights are used. Where samples could not be obtained (sand-clay strata), THD penetrometer results are used along with the criteria outlined in Table 1. The ultimate side shear for strata where sampling was not possible is N/p , but not greater than the limiting value of 0.9 tsf. From the cumulative graph, 11.2 tons per foot of perimeter is achieved at an elevation of +10.5 feet.
8. Since the last 5 feet of the shaft are noncontributing, the shaft is then extended 5 feet, so that the required base elevation is <u>+5.5 feet</u> (penetration of 36.5 feet). This is well within the founding stratum assumed in the trial design.
9. The first of the two design criteria has been met in establishing the design length. Check the second of the two criteria:

$$\left(Q_T\right)_{Design} \leq \left(Q_S\right)_{ult} + \frac{\left(Q_B\right)_{ult}}{3}$$

$$90 \leq 123.5 + \frac{74.5}{3} = 148.3 \quad \underline{\underline{\text{OK}}},$$

Immediate settlement will be tolerable.

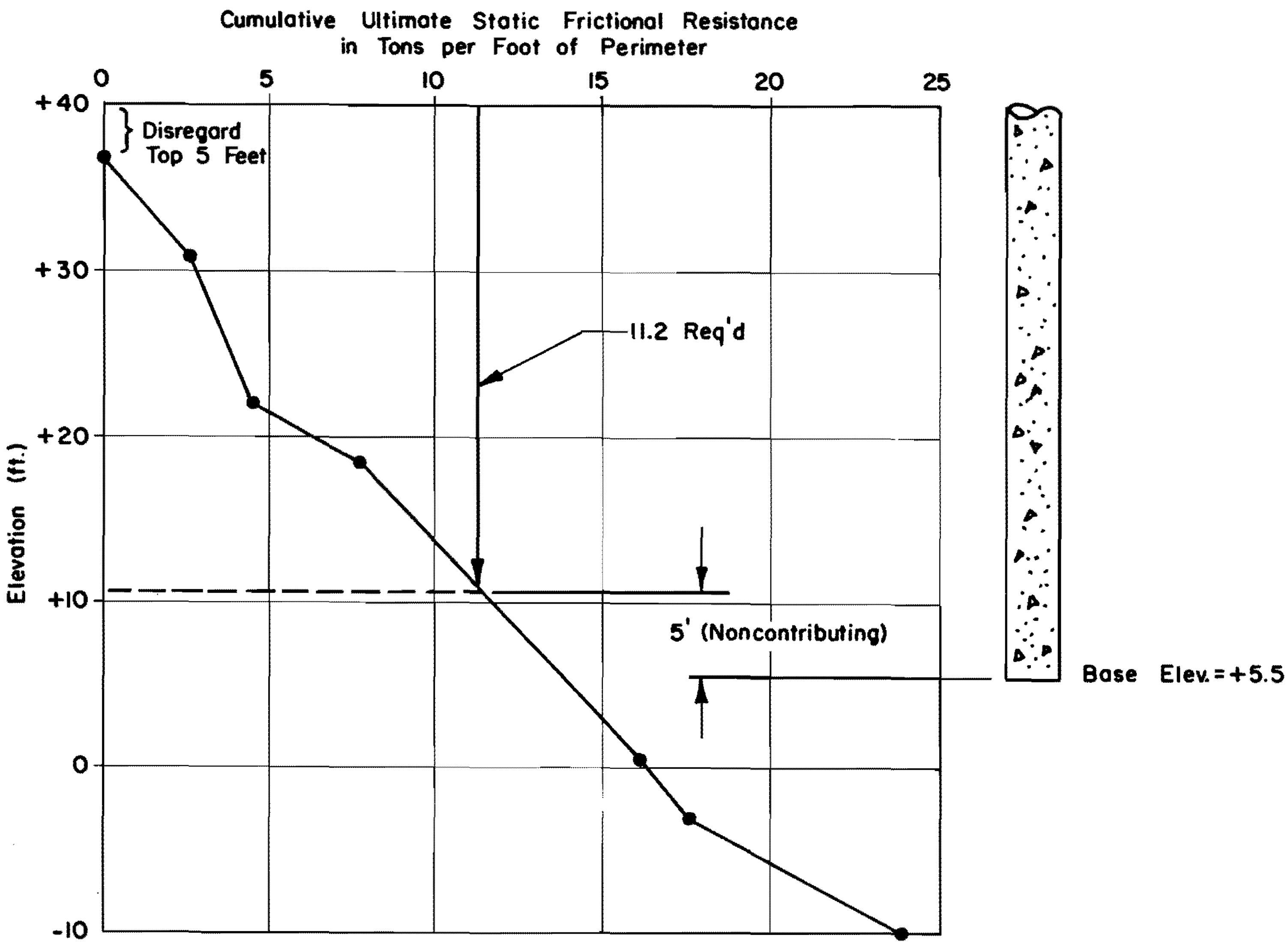



Fig. 9. Cumulative frictional resistance diagram, example problem No. 1.

10. In this case the trial design was valid. Had the trial design been inadequate, the design could have been altered by founding the shaft deeper, increasing its diameter, or specifying a bell in the clay stratum between +18.5 and +0.5.

Example Problem No. 2

Given: A drilled shaft foundation is to be designed to carry a load of 90 tons per shaft. Normal foundation exploration, field, and laboratory data are available and are shown in the following pages. The soil profile is given in Fig. 10. The shear strength values are tabulated in Table 3. They were obtained by other than the HUE multiphase triaxial procedure. Structure type and span lengths call for three-foot-diameter columns.

Since the silty sand from +22 to +18.5 is waterbearing, drilling mud will be required in the upper portion of the borehole, down to an elevation of +18.5 feet. The lower portion of the shaft will be drilled in the dry, after the upper part is cased and pumped dry. A possibility exists for drilling mud entrapment above +18.5 feet. Therefore, the reduced $\alpha$ factors corresponding to Category A.2 must be used.

Required: To determine the size and penetration of drilled shafts without bells to carry safely and economically the design load in the vicinity of Test Hole No. 1.

Solution: The Design Category is A.2.

1. A study of Fig. 10 indicates that the brown clay layer from +18.5 to +0.5 is the probable founding stratum.

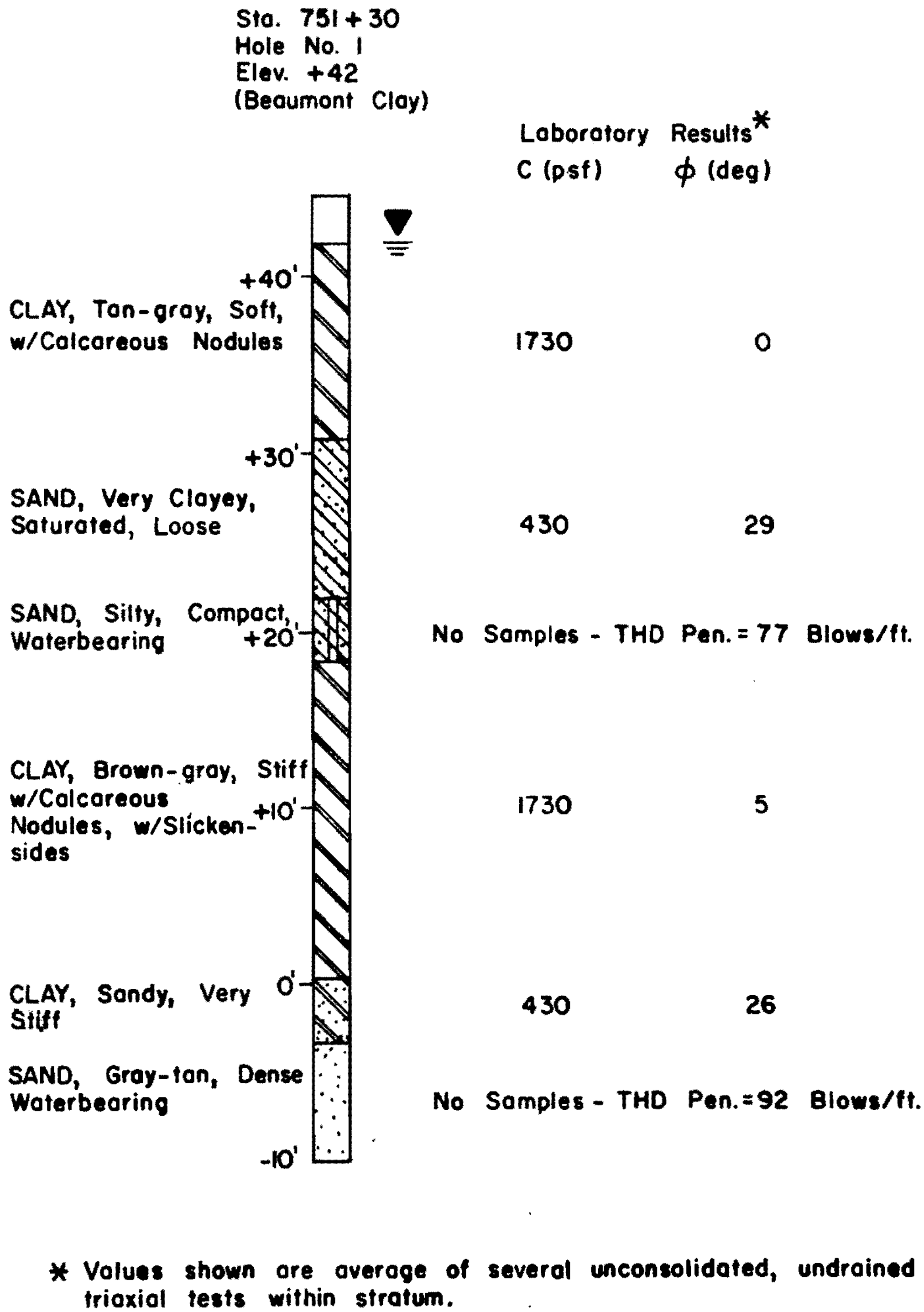



Fig. 10. Soil profile for test hole No. 1, example problem No. 2.

TABLE 3. EXAMPLE PROBLEM NO. 2, TEST HOLE NO. 1 (FIG. 10).

| Elevation (ft) | Thickness of Stratum d (ft) | Effective Unit Weight of Soil w (pcf) | $\phi$[1] (deg) | c[1] (psf) | Shear[2] Strength, S (psf) | Shear[2] Strength, S (tsf) | Ultimate Stress $=\alpha(S)$ (tsf) per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter) per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter) cumulative |
|---|---|---|---|---|---|---|---|---|---|
| +42 to +37 | 5 | 55 | Disregard | | | | | | |
| +37 to +31 | 6 | 55 | 0 | 1730 | 1730 | 0.86 | 0.26 | 1.1 | 1.1 |
| +31 to +22 | 9 | 68 | 24 | 430 | 830 | 0.41 | 0.12 | 1.1 | 2.2 |
| +22 to +18.5 | 3.5 | 60 | THD Pen = 77 Blows/Ft. | | | | (0.4)[3] | 1.4 | 3.6 |
| +18.5 to + 0.5 | 18 | 60 | 5 | 1730 | 1870 | 0.94 | 0.47 | 8.5 | 12.1 |
| + 0.5 to - 3 | 3.5 | 68 | 26 | 430 | 1700 | 0.85 | 0.43 | 1.5 | 13.6 |
| - 3 to -10 | 7 | 70 | THD Pen = 92 Blows/Ft. | | | | (0.9)[3] | 6.3 | 19.9 |

1 Determined from laboratory undrained triaxial compression tests

2 Cohesion plus product of overburden stress and tan $\phi$. Overburden stress is sum of wd from strata above and $w(\frac{d}{2})$ for stratum under consideration.

3 Limiting values

2. Select a diameter of 4 feet for trial design. A 4-foot-diameter shaft has the following geometric properties:

   Perimeter = 12.57 square feet per linear foot

   Base area, $A_B$ = 12.57 square feet

3. Ultimate total capacity required = 2.2(90) = 198 tons

4. Ultimate capacity of the base:

$$\left(Q_B\right)_{ult} = 9\ c\ A_B$$

$$= 9(0.86)(12.57) = 97.1 \text{ tons}$$

5. Ultimate side resistance required

$$\left(Q_S\right)_{ult} = \left(Q_T\right)_{ult} - \left(Q_B\right)_{ult}$$

$$= 198 - 97.1 - 100.9 \text{ tons}$$

6. Required ultimate cumulative frictional resistance

$$= \frac{100.9}{12.57} = 8.03 \text{ tons per foot of perimeter}$$

7. From the cumulative graph, Fig. 11, 8.03 tons per foot of perimeter is achieved at an elevation of +9 feet.

8. Since the last five feet of the shaft are noncontributing, the shaft is then extended five feet, so that the required base elevation is $\underline{+4}$ feet. This elevation is within the founding stratum assumed but is near the bottom of that stratum. Since the stratum from +0.5 to -3 has about the same shear strength as the founding stratum, no alterations in the design are necessary. Had the underlying stratum been significantly weaker than the founding

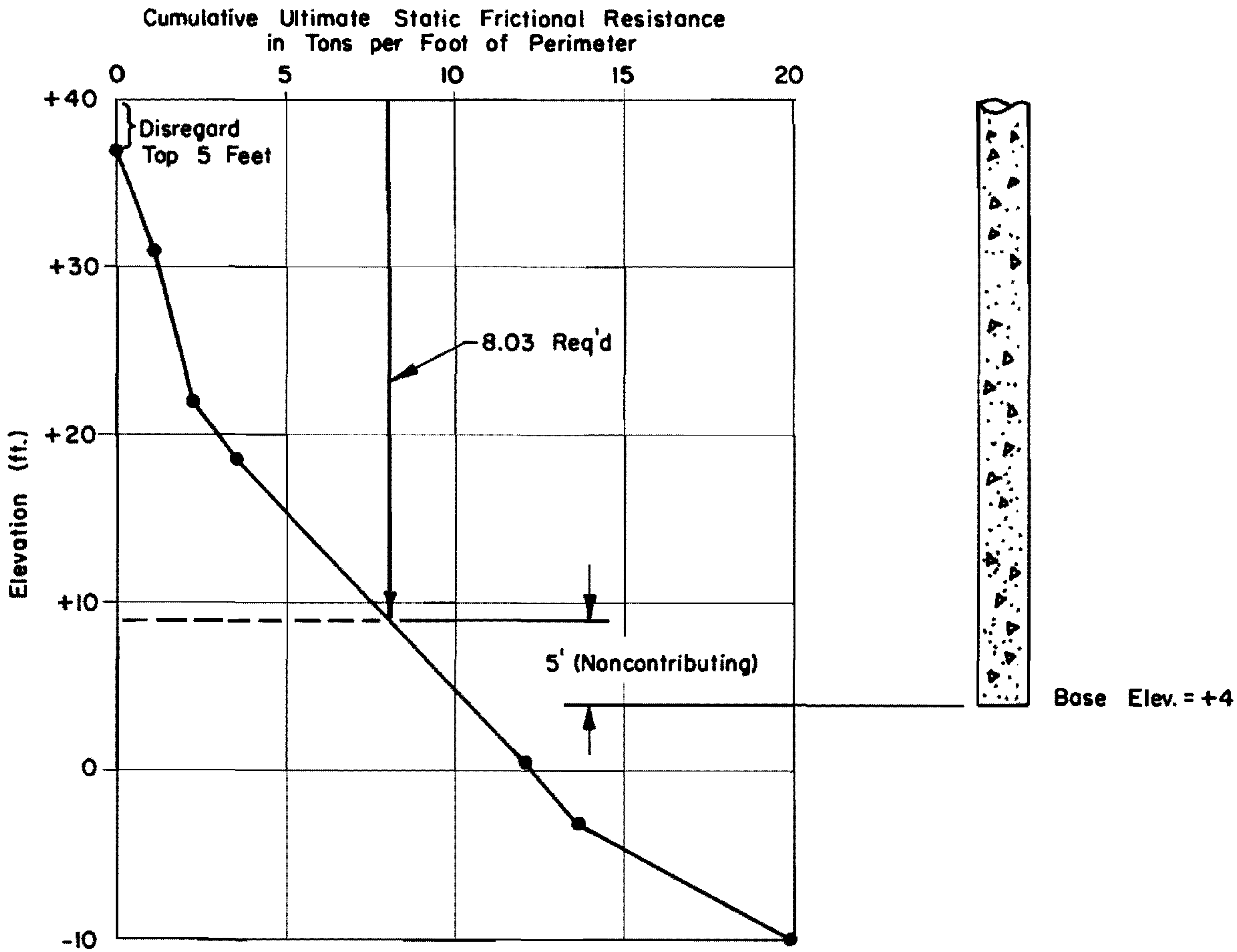



Fig. 11. Cumulative frictional resistance diagram, example problem No. 2.

stratum, it would have been necessary to revise the design to relocate the base.

9. Check second criterion:

$$\left(Q_T\right)_{Design} \quad \left(Q_S\right)_{ult} + \frac{\left(Q_B\right)_{ult}}{3}$$

$$\leq 100.9 + \frac{97.1}{3} = 133.3 \quad \underline{\underline{\text{OK}}},$$

Immediate settlement will be tolerable.

<u>Example</u> <u>Problem</u> <u>No</u>. <u>3</u>

<u>Given</u>: A drilled shaft foundation is to be designed to carry 90 tons per shaft. Foundation exploration data are available for the site (Test Hole No. 3, Fig. 12). Only THD cone penetrometer soundings were taken. Three-foot-diameter columns will be required.

<u>Required</u>: To determine the size and penetration of drilled shafts without bells to carry safely and economically the design load.

<u>Solution</u>: Since the first waterbearing stratum is at -12 feet, assume the design will be under Category A.1. The Alternate Procedure is followed.

1. For trial design assume that clay from +20 to +10 will be the founding stratum.
2. Select a diameter of 3.5 feet (Perimeter = 10.99 square feet per linear foot; $A_B$ = 9.62 square feet).
3. Ultimate total capacity required = (2.2)(90) = 198 tons

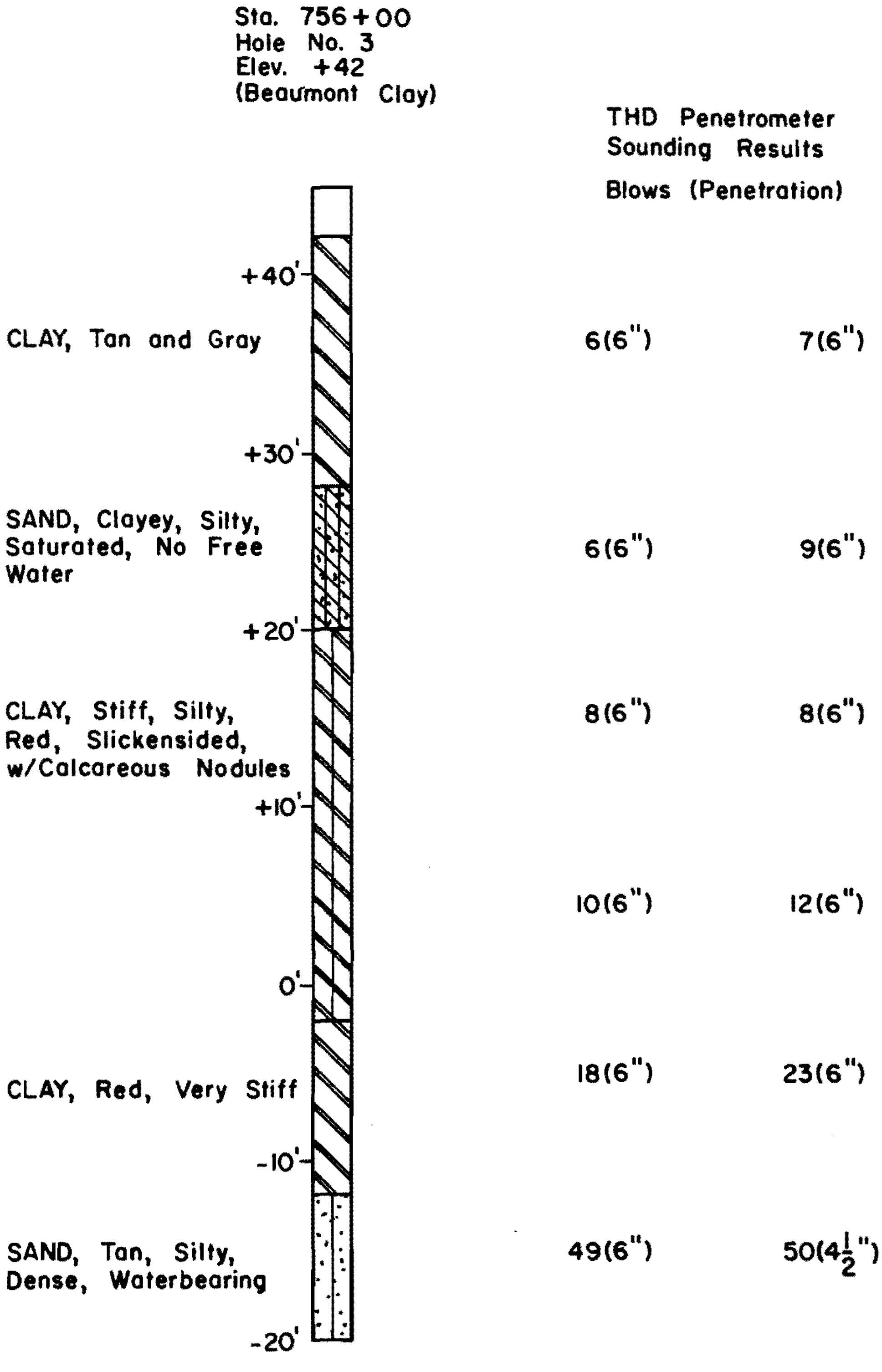



Fig. 12. Soil profile for test hole No. 3, example problem No. 3.

4. Ultimate capacity of the base:

$$\left(Q_B\right)_{ult} = \frac{N}{p'} = \frac{16}{2.8} \;(9.62) = 55 \text{ tons}$$

5. Ultimate side resistance required:

$$\left(Q_S\right)_{ult} = \left(Q_T\right)_{ult} - \left(Q_B\right)_{ult} = 198 - 55 = 143 \text{ tons}$$

6. Required ultimate cumulative frictional resistance

$$= \frac{143}{10.99} = 13.0 \text{ tons per foot of perimeter}$$

7. The shear strength values are tabulated in Table 4. From Table 4, the cumulative graph, Fig. 13, is obtained. A value of 13.0 tons per foot of perimeter is achieved at an elevation of +7.5 feet.

8. Since the last five feet of the shaft are noncontributing, the base of the shaft should be set at +2.5 feet. This is below the founding stratum assumed in the first trial.

9. Assume for a second trial that the base will be located in the clay stratum from +10 to -2. The new ultimate base capacity is:

$$\left(Q_B\right)_{ult} = \frac{N}{p'} \; A_B = \frac{22}{2.8} \;(9.62) = 75.7 \text{ tons}$$

10. Ultimate side resistance required:

$$198 - 75.7 = 122.3 \text{ tons}$$

11. Required ultimate cumulative frictional resistance

$$= \frac{122.3}{10.99} = 11.12 \text{ tons per foot of perimeter}$$

TABLE 4. EXAMPLE PROBLEM NO. 3, TEST HOLE NO. 3 (FIG. 12).

| Elevation (ft) | Thickness of Stratum $d$ (ft) | N (Blows/Ft) | Ultimate Stress $= \frac{N}{P}$ (tsf) per stratum | Ultimate Capacity $= d(\frac{N}{P})$ (tons per foot of perimeter) | |
|---|---|---|---|---|---|
| | | | | per stratum | cumulative |
| +42 to +37 | | Disregard | | | |
| +37 to +28 | 9 | 13 | 0.37 | 3.3 | 3.3 |
| +28 to +20 | 8 | 15 | 0.43 | 3.4 | 6.7 |
| +20 to +10 | 10 | 16 | 0.46 | 4.6 | 11.3 |
| +10 to - 2 | 12 | 22 | 0.63 | 7.6 | 18.9 |
| - 2 to -12 | 10 | 41 | (0.9)[1] | 9.0 | 27.9 |
| -12 to -20 | 8 | >100 | (0.9)[1] | 7.2 | 35.1 |

[1] limiting values

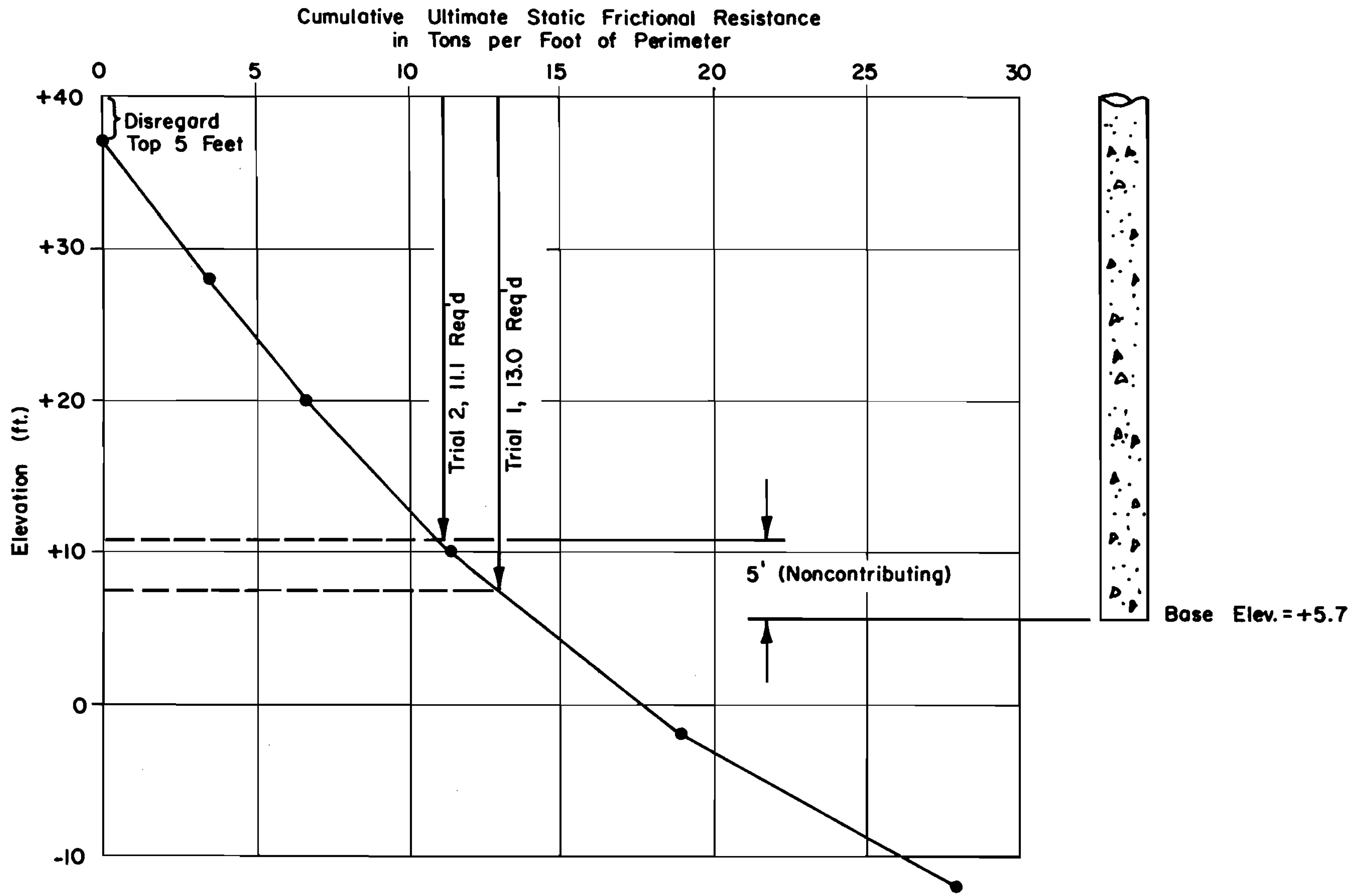



Fig. 13. Cumulative frictional resistance diagram, example problem No. 3.

12. From the cumulative graph, 11.12 tons per foot of perimeter is achieved at an elevation of +10.7 feet.

13. Since the last five feet of the shaft are noncontributing, the base of the shaft should be set at +5.7 feet, which is well within the stratum assumed in the second trial.

14. The first of the two design criteria has been met in establishing the design length. Check the second of the two criteria:

$$\left(Q_T\right)_{Design} \leq \left(Q_S\right)_{ult} + \frac{\left(Q_B\right)_{ult}}{3}$$

$$90 \leq 122.3 + \frac{75.7}{3} = 147.5 \quad \underline{\underline{OK}},$$

Immediate settlement will be tolerable. Since the shaft terminates at +5.7, the assumed category (A.1) remains valid.

Example Problem No. 4

Given: A drilled shaft foundation is to be designed to carry a load of 132 tons per shaft. Normal foundation exploration, field, and laboratory data are available. The soil profile is as given for Test Hole No. 1 in Fig. 8. The ultimate side resistances for each stratum are given in Table 5. The triaxial tests were conducted by a method other than the HUE multiphase shear test, so increased $\alpha$ factors may not be used. Structure type and span length call for 2.5-foot-diameter columns.

Required: To determine the size and penetration of drilled shafts with bells to carry safely and economically the design load in the vicinity of Test Hole No. 1.

TABLE 5. EXAMPLE PROBLEM NO. 4, TEST HOLE NO. 1 (FIG. 8).

| Elevation (ft) | Thickness of Stratum d (ft) | Effective Unit Weight of Soil w (pcf) | $\phi$[1] (deg) | c[1] (psf) | Shear[2] Strength, S (psf) | Shear[2] Strength, S (tsf) | Ultimate Stress $=\alpha(S)$ (tsf) per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter): per stratum | Ultimate Capacity $=d(\alpha)(S)$ (tons per foot of perimeter): cumulative |
|---|---|---|---|---|---|---|---|---|---|
| +42 to +37 | 5 | 55 | Disregard | | | | | | |
| +37 to +31 | 6 | 55 | 0 | 1730 | 1730 | 0.86 | 0.13 | 0.8 | 0.8 |
| +31 to +22 | 9 | 68 | 24 | 430 | 830 | 0.41 | 0.06 | 0.5 | 1.3 |
| +22 to +18.5 | 3.5 | 60 | THD Pen = 77 Blows/Ft. | | | | (0.25)[3] | 0.9 | 2.2 |
| +18.5 to + 0.5 | 18 | 60 | 5 | 1730 | 1870 | 0.94 | 0.28 | 5.0 | 7.2 |
| + 0.5 to - 3 | 3.5 | 68 | 26 | 430 | 1700 | 0.85 | 0.26 | 0.9 | 8.1 |
| - 3 to -10 | 7 | 70 | THD Pen = 92 Blows/Ft. | | | | (0.40)[3] | 2.8 | 10.9 |

1 Determined from laboratory undrained triaxial compression tests

2 Cohesion plus product of overburden stress and tan $\phi$. Overburden stress is sum of wd from strata above and $w(\frac{d}{2})$ for stratum under consideration.

3 Limiting values

<u>Solution</u>: The Design Category is B.2.

1. A study of Fig. 8 indicates that the brown clay from +18.5 to +0.5 is a likely founding stratum and will probably be good belling material. The waterbearing, sandy soil from +22 to +18.5 will require that drilling mud be used from the top of the borehole to elevation +18.5. The remaining portion of the hole can be completed in the dry.

2. Select a 3:1 bell (60°) with a 7.5-foot base diameter with a six-inch cylindrical pad on the bottom. Try a 2.5-foot stem diameter. The 7.5-foot diameter bell has a base area of 44.18 square feet, and a 2.5-foot diameter stem has 7.85 square feet of perimeter area per linear foot.

3. For a belled shaft, the design load is likely to be controlled by the second of the two design criteria; namely, that the factor of safety on the base should be not less than three. The design procedure is, therefore, modified as follows. The ultimate base capacity in the chosen stratum is given by

$$\left(Q_B\right)_{ult} = 9\ c\ A_B = 9(0.86)(44.18) = 342 \text{ tons.}$$

The maximum permissible load on the base is

$$\left(Q_B\right)_{Design} = \frac{342}{3} = 114 \text{ tons}$$

4. The ultimate side resistance must then be:

$$\left(Q_S\right)_{ult} = \left(Q_T\right)_{ult} - \left(Q_B\right)_{ult} = 132 - 114 = 18 \text{ tons.}$$

5. The required ultimate cumulative frictional resistance

$$= \frac{18}{7.85} = 2.3 \text{ tons per foot of perimeter}$$

6. From the cumulative graph, Fig. 14, 2.3 tons per foot of perimeter is achieved at an elevation of +18 feet.
7. Since the last five feet of the stem are noncontributing, and since the bell, which is also noncontributing, is approximately five feet high, the founding elevation for the base of the bell is +8 feet. This is within the founding stratum assumed in the trial design, and the soil for a depth of two base diameters (15 feet) is of sufficient strength that the calculated base capacity will be achieved.
8. The second of the two design criteria has been met in establishing the design length. Now, check the first criterion:

$$\left(Q_T\right)_{Design} = 132 \leq \frac{\left(Q_S\right)_{ult} + \left(Q_B\right)_{ult}}{2.2} = \frac{18 + 342}{2.2}$$

$$= 164 \quad \underline{\underline{\text{OK}}},$$

The overall factor of safety is adequate.

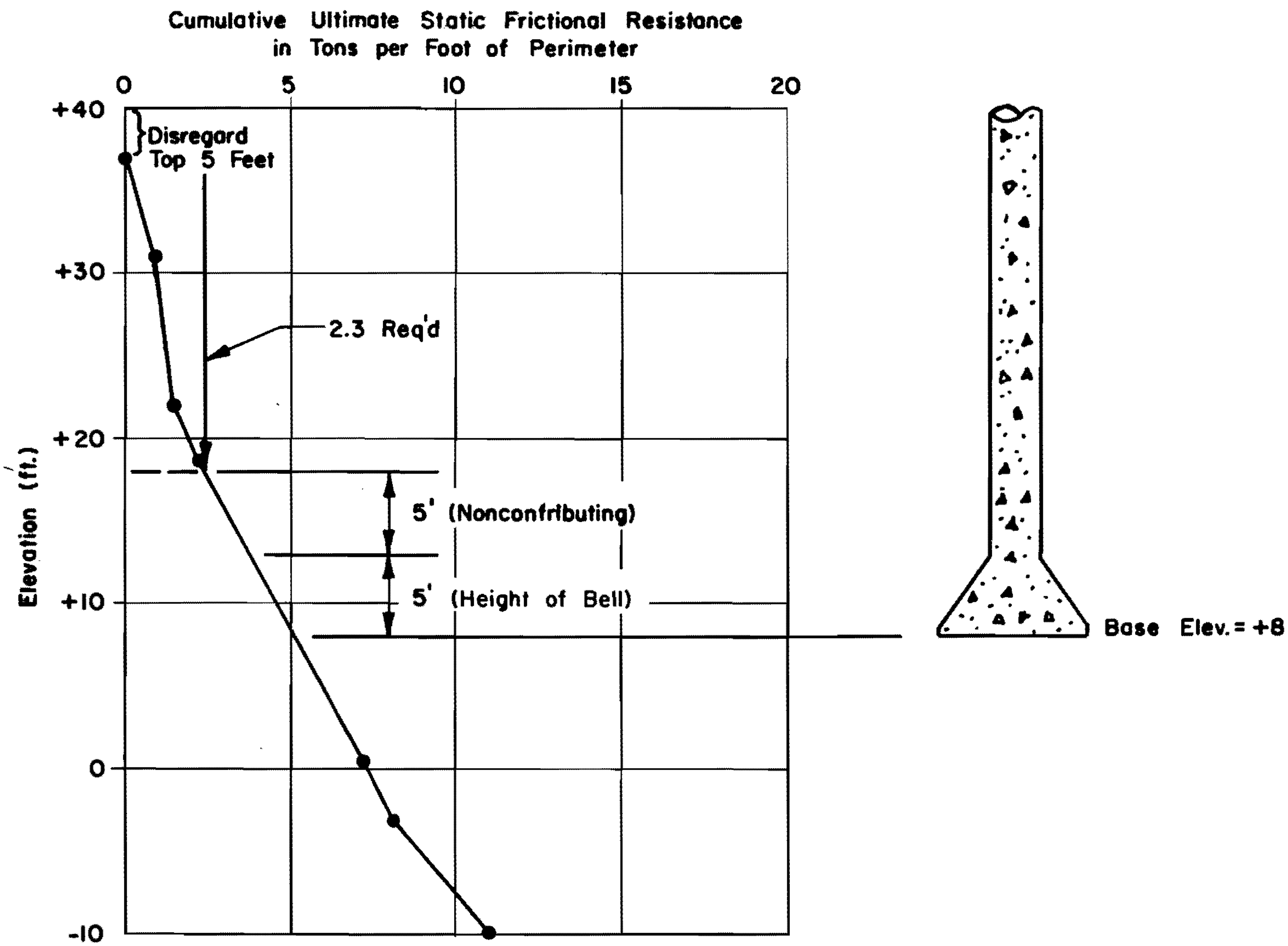



Fig. 14. Cumulative frictional resistance diagram, example problem No. 4.

# CHAPTER VI

## CONCLUSIONS AND RECOMMENDATIONS

Since this report is a summary of pertinent reports written previously, no specific conclusions are given. The design procedures and values of design parameters presented herein are recommended for use by the Texas Highway Department.

The following recommendations are made concerning future investigations of behavior of drilled shafts under axial loading:

1. Further tests on drilled shafts installed by existing methods, namely the dry method and the drilling mud and casing method, should be conducted in soils with engineering properties significantly different than the soils in which tests have been run to date, particularly in sands, very soft clays, and shales.
2. Tests should be conducted on shafts installed by the direct displacement method in order to evaluate the effects on the load transfer characteristics of placing shafts by directly displacing drilling mud with fluid concrete.
3. Long-term tests should be conducted in order to determine whether the $\alpha$ factors in Category B are relevant or whether they are too conservative.
4. Further tests should be conducted on drilled shafts in expansive clays in order to determine the depth of

complete loss of load transfer due to shrinkage and to determine the magnitude of uplift stresses that will be developed in such shafts.

REFERENCES

Barker, W. R., and Reese, L. C. (1969), "Instrumentation for Measurement of Axial Load in Drilled Shafts," Research Report 89-6, Center for Highway Research, The University of Texas at Austin, Austin, Texas, November, 1969.

Barker, W. R., and Reese, L. C. (1970), "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, Austin, Texas, August, 1970.

Engineering News-Record (1971), "Huge Cantilever Beam Supports Building Over Tunnel," Engineering News-Record, Vol. 186, No. 7, February 18, 1971, pp. 22-23.

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

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

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, Austin, Texas, March, 1969.

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

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