BEHAVIOR OF AXIALLY LOADED DRILLED SHAFTS IN BEAUMONT CLAY

PART FOUR - DESIGN INFERENCES AND CONCLUSIONS

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 eighth in a series of reports from Research Project
3-5-65-89 of the Cooperative Highway Research Program. The principal aim
of the report is to describe the results of axial load tests of full-scale,
instrumented drilled shafts in the Beaumont Clay formation in Houston, Texas.
The tests were conducted to measure side and base stresses in cylindrical
and underreamed shafts, constructed by both wet and dry procedures. The
distribution of shear stresses along the sides of the shafts was measured
to provide an insight into the mechanism affecting the load transfer behav-
ior of drilled shafts in clay. Maximum side shear stresses and base capac-
ities have been correlated with the undrained shear strength of the soil
as indicated by laboratory procedures and with results of Texas Highway
Department cone penetration tests.

The report is issued in five separately bound parts:

 Part One - "State of the Art" describes the historical develop-
     ment of drilled shafts, describes construction pro-
     cedures, presents the mechanics of shaft behavior,
     outlines current methods of design, and presents a
     summary of the results of field tests reported in
     the technical literature.

 Part Two - "Site Investigation and Test Shaft Instrumentation"
     gives details of the geotechnical investigation of
     the test site, describes the test shafts and anchor-
     age systems, describes the various instrumentation

systems, and presents results of monitoring the instrumentation under no-load conditions.

Part Three - "Field Tests" describes the field test procedures and presents the detailed results of the tests.

Part Four - "Design Influences and Conclusions" presents criteria, obtained through the field tests and from the literature review, for designing drilled shafts in Beaumont Clay.

Part Five - "Appendices" gives supporting data and details not contained in the main body of Parts One through Four.

It is not intended that the reader read the entire report in order to obtain information on any particular subject. The report was separated into the various Parts, any of which can be consulted for specific details, for this reason. It is expected that most readers will desire to consult only Part Four, which briefly summarizes Parts One through Three, and then consicely presents design criteria for axially loaded drilled shafts in Beaumont Clay. The Chapters are numbered continuously from Part One through Part Five. Although some cross-referencing exists, the various Parts are written to be as independent as possible. The reference list is contained in Part Four.

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 Pressure-meter, 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.

ABSTRACT

A drilled shaft is a foundation element formed by boring a cylindrical

hole into the soil and backfilling the hole with concrete. The recent

increase in the utilization of drilled shafts as foundations for major

structures has created a need for systematic investigations of their

behavior. One such investigation, in which four full-sized drilled

shafts of varying geometries were loaded axially to failure, was con-

ducted at a site in the stiff, fissured Beaumont Clay in Houston, Texas.

The test shafts were constructed by both wet and dry procedures. They

were fully instrumented for measurement of the distribution of axial

load, thereby permitting a calculation of the distribution of developed

side resistance and of base resistance.

Prior to and during the field tests, a careful site investigation was

conducted, and a shear strength profile was developed based on unconsoli-

dated, undrained triaxial test results and Texas Highway Department cone

penetrometer soundings. The maximum side shear stresses developed during

the load tests were compared to the shear strength profile and penetrometer

results in order to arrive at shear strength reduction factors that

could be relied upon in predicting design values for side friction.

The side shear stresses were observed to vary considerably from the

tops of the shafts to the bottoms, generally being quite small at both

ends. Overall, the shafts that were installed in dry boreholes developed

an average maximum side shear stress of about one-half of the shear

x

strength of the clay. The single shaft installed in a processed borehole developed an average of only about one-third of the shear strength of the clay along its sides.

The load measurements indicated that bearing capacity equations used for ultimate base resistance for piles in clay were valid for both belled and cylindrical test shafts.

After the tests were completed, soil adjacent to the walls of three of the shafts was sampled in an attempt to determine the nature of the mechanism of shear strength reduction in soil immediately adjacent to the sides of drilled shafts. In the shafts installed in dry boreholes, some soil softening due to an increase in moisture content occurred, particularly near the bases. This softening, produced by water from the setting concrete, accounted for some, but not all ot the measured strength reduction. Other reasons for shear strength reduction are reasoned to be the effects of remolding and opening of fissures as the boreholes were drilled and mechanical base-side interference. Samples taken adjacent to the shaft installed in a processed hole revealed pockets of trapped drilling mud between the sides of the borehole and the wall of the shaft.

Based upon the field study and a comprehensive review of related research conducted in similar soil formations, a tentative design procedure is suggested. That procedure includes criteria for providing an adequate factor of safety against plunging failure and for limiting immediate settlement at working load to an acceptable value.

KEY WORDS: piles, bored piles, drilled shafts, soil mechanics, undrained shear tests, cohesive soils, cone penetrometer, instrumentation, field tests, design criteria

SUMMARY

The purpose of this report is to describe the results of field tests
of full-sized, instrumented drilled shafts in the Beaumont Clay formation.
Drilled shafts with varying base geometry, length, and method of installa-
tion were load tested to obtain measurements of the distribution of axial
load with depth and of base load-settlement characteristics in order to
develop design criteria.

Pertinent soil parameters were obtained by various standard procedures,
including the unconsolidated, undrained triaxial test and the T.H.D. cone
penetrometer test to provide a basis for the correlation of test results.

The test shafts were observed to develop considerable resistance in
side friction. Furthermore, side resistance was observed to develop much
sooner than base resistance, with the result that side resistance predom-
inated over base resistance at design load. The shafts installed in dry
boreholes mobilized an average of one-half of the shear strength of the
soil in side friction, while the side frictional stresses in the shaft
installed in a processed borehole were significantly smaller. An investi-
gation showed that the shafts installed in the dry were well-formed and bonded
securely to the soil composing the borehole walls, while the shaft installed
in a processed hole contained pockets of drilling mud between the concrete
and natural soil. Based upon these observations, the numerical test results,
and field tests of other investigators in similar soil formations, a tenta-
tive design procedure incorporating side resistance is formulated.

IMPLEMENTATION STATEMENT

The study indicated that considerable load was resisted in side friction in axially loaded drilled shafts in stiff clay with both straight sides and underreams, installed in dry boreholes and in boreholes processe with drilling mud. The possibility that considerably smaller frictional resistance occurs in shafts installed in processed holes was observed, however. The test results generally agree with those of other investigators in similar soils.

Measured side shear and base capacities were correlated with standard soil strength tests. It appears that side friction can be reliably estimated for shafts in dry boreholes, and to some extent for shafts installed in processed holes, from laboratory soil tests or from penetrometer soundings. Therefore, a new design procedure for drilled shafts is suggested that incorporates side friction, a resistance component heretofore omitted from consideration. The incorporation of side friction in the design of drilled shafts will undoubtedly result in considerable monetary savings in bridge foundation construction.

The suggested general design parameters are, of necessity, somewhat conservative, because of the limited number of tests that were conducted and because field testing was limited to short-term loading in one specific soil formation. Further savings can be realized by extending the research into long-term testing, into testing in other soil formations, and into reevaluating construction techniques for installation of shafts in processed boreholes. Such research would provide a better definition of the design parameters in all situations and would therefore permit the design of drilled shafts to be more rational and less conservative.

CONTENTS

NOMENCLATURE

Symbol	Definition
A_B	area of base
A_c	transformed cross-sectional area of stem (including effects of reinforcing steel)
A_S	peripheral area of stem
A'_S	nominal peripheral area of the stem excluding sections at the top and bottom, each equal in height to twice the stem diameter
B	diameter of loaded area
$\bar{B}$	width of group of piles or shafts
C'	change in void ratio for increment of applied load
C_c	compression index
C_e	expansion index
c'	effective cohesion
c_{base}	average undrained cohesion of clay beneath base of shaft
c_{mean}	average undrained soil cohesion for fissured soil
c_{sides}	average undrained cohesion of clay along sides of shaft
c_u	undrained cohesion
c_v	coefficient of consolidation
D_r	relative density
d	diameter of shaft or pile
d_{stem}	diameter of stem
E_c	Young's modulus of concrete

Symbol	Definition
E_o	slope of initial tangent to nonlinear soil stress-strain curve; circuit output
$\dfrac{E_o}{c_u}$	ratio of E_o to half of maximum indicated undrained stress difference of clay
$e_{corrected}$	void ratio at beginning of loading increment of consolidation test corrected for elastic compression of consolidation apparatus
e_i	indicated void ratio at beginning of loading increment in consolidation test
e_o	void ratio of soil under overburden pressure, p_o
e'_o	void ratio after load increased to preconsolidation pressure, then decreased to overburden pressure in consolidation test
e_{50}	void ratio corresponding to t_{50}
e_{100}	void ratio corresponding to t_{100}
F.S.	factor of safety at working load
f_1, f_2	base shape factors
H	thickness of compressible layer
h	depth of base of shaft
I_ρ	settlement influence coefficient
K	gage factor
K_o	coefficient of lateral earth pressure, or the ratio of horizontal effective stress to vertical effective stress
L	unit length along shaft
L_s	length of stem

Symbol	Definition
l	length of shaft or pile
N	number of blows per foot for T.H.D. penetrometer
N_c, N_q, N_γ	bearing capacity factors
N_q^*	bearing capacity factor for sands
NMC	natural moisture content
P_i	point at center of ith layer at which consolidation settlement is computed
p	factor relating penetrometer results to maximum unit side resistance
Δp	increment of applied pressure causing consolidation
p'	factor relating penetrometer results to unit base capacity
P_c	preconsolidation pressure
p_i	i^{th} point on load transfer or load distribution curve
P_o	overburden pressure, or initial effective vertical pressure at the center of the compressible layer
$Q(z)$	function relating load in the shaft to depth
Q_B	total amount of load taken by the base
Q_S	total amount of load removed by the sides in shear
Q_T	applied load
$(Q_B)_{ult}$	ultimate base load
$(Q_S)_{ult}$	ultimate side load
$(Q_T)_{ult}$	ultimate load at top of pile or shaft
q	contact pressure
$(q_B)_{ult}$	unit ultimate bearing stress on the base

Symbol	Definition
$(q_S)_{ult}$	unit ultimate side resistance
$(q_B)_{ult, net}$	net unit ultimate bearing stress on the base
r	stem radius
S	mean shear strength of clay soil
S_r	degree of saturation
S_0	shear strength of soil before softening
S_1	shear strength of soil after softening
S1, S2, S3, S4	abbreviations for Test Shaft No. 1, Test Shaft No 2, Test Shaft No. 3, Test Shaft No. 4
S1T1, etc.	abbreviation for "Test No. 1 on Test Shaft No. 1," etc
s	shear stress, spacing between piles in a group
T_z	tensile force at depth z
t_{50}	time required to develop 50 per cent of primary consolidation (logarithm of time plot)
t_{100}	time required to develop 100 per cent of primary consolidation (logarithm of time plot)
v	applied voltage
w	downward movement, moisture content
w_T	downward displacement of the butt
$w_{\bar{z}}$	downward displacement at depth $\bar{z}$
z	depth coordinate
$\bar{z}$	generic depth
α	shear strength reduction factor
α_{avg}	average shear strength reduction factor over a specified length of shaft

Symbol	Definition
α_{min}	minimum shear strength reduction factor from a laboratory test series
α_{peak}	α_{avg} corresponding to peak side load
α_{ult}	α_{avg} corresponding to ultimate load
α_z	shear strength reduction factor at depth z
α_1	ratio of shear strength of soil around shaft after placing concrete to that existing before placing concrete
α_{11}	that part of α_1 due to softening because of migration of water from concrete into soil
α_{12}	that part of α_1 due to the shear strength reduction not accompanied by moisture migration (remolding, opening of surface fissures)
α_{13}	that part of α_1 due to surface effects and base-side mechanical interference
α_2	adhesion coefficient
$\bar{\alpha}$	average shear strength reduction factor over entire stem excluding top and bottom two diameters
β	settlement correlation coefficient, settlement interaction factor
γ'	effective unit of weight of soil
δ	angle of friction between the soil and concrete
δ_S	elastic compression of stem
ϵ	strain, general
$\epsilon_{circuit}$	circuit strain
ϵ_1	axial strain in triaxial or unconfined compression test
$\epsilon_{1_{steel}}$	strain in steel in longitudinal direction

Symbol	Definition
$\epsilon_{2_{steel}}$	strain in steel in transverse direction
ϵ_{50}	strain corresponding to one-half of the principal stress difference at failure
μv	abbreviation for microvolts
ν	Poisson's ratio
ξ	settlement ratio
ρ_B	average settlement beneath loaded area
ρ_c	total compression of compressible layer
σ	normal stress
σ_v'	vertical effective stress in the soil adjacent to the shaft
σ_Δ	principal stress difference in a triaxial or unconfined compression test
σ_1	maximum principal stress
σ_3	minimum principal stress
ϕ	angle of internal friction
ϕ' $(=\phi_d)$	effective angle of internal friction
ϕ_u	undrained angle of internal friction
ψ	additional shear strength reduction factor for shafts installed in a processed hole
ω	bearing capacity reduction factor for fissured clay

CHAPTER XIII

DESIGN INFERENCES OF THE FIELD TESTS

Review of Field Research

The manner in which side shear develops along an axially loaded drilled shaft has long been an imperfectly understood phenomenon. In order to study the development of side shear stresses in drilled shafts, the Center for Highway Research has conducted load tests on full-sized, instrumented shafts installed in several geological formations in Texas. One such field study was performed at a site in Houston, Texas, in the Beaumont Clay formation. The results of that study are presented in the five Parts of this report. Parts One through Three contain background information, in which details of the soil conditions at the site, instrumentation schemes, test results, and a review of information published by other investigators engaged in similar studies are presented. In Part Four the preceding Parts are summarized briefly and the information gained from the study is interpreted to provide design criteria. Supporting information, such as boring logs, raw data from the load tests, and soil stress-strain curves are given in Part Five.

Site Description. The test site, designated the SH225 Site, is described in detail in Part Two. For convenience, several pertinent facts are repeated in summary form here.

Four test shafts were constructed at the SH225 Test Site, which is located on Texas Highway Department right-of-way at the intersection of State Highway 225 and Interstate Highway 610 in southeastern Houston,

Texas. The shafts were designed to provide a means of investigating several variables that affect the development of side resistance and end bearing, including method of construction (dry borehole procedure or processed borehole procedure), geometry of the base, depth of penetration, and characteristics of the soil in which the shaft is located. Other parameters believe to affect the behavior of drilled shafts in clay to a lesser degree, including method of load testing, diameter of the stem, and composition of the concrete, were constants in the study. A stem diameter of 30 inches was chosen for all test shafts to represent a typical value for bridge foundations in the Houston area. All concrete for the test shafts was Texas Highway Department Class C concrete with a water-cement ratio of 6.5 gallons per sack (0.6 weight ratio) and a slump of 6 inches. Testing was accomplished by means of the Texas Highway Department standard quick load test method.

The four test shafts were constructed in close proximity to each other during 1968 and 1969. Test Shaft No. 1 was cylindrical and had an embedded length of 23 feet. Test Shaft No. 2 was identical to Test Shaft No. 1, except that it had a 7.5-foot-diameter bell with its base at a depth of 23 feet. Test Shaft No. 3 was identical to Test Shaft No. 1, except that it was cast above a void. These three shafts were cast in dry boreholes. Test Shaft No. 4, which was cast in a processed borehole, was cylindrical, with an embedded length of 45 feet. The first three shafts were load tested two to three months after casting. Test Shaft No. 4 was first tested approximately five months after casting.

The soil profile at the SH225 Test Site was as follows:

Layer I: ground surface to depth of 29 feet. Unified Classifi-

cation of CH. Mean undrained shear strength of 1.2 tsf.

Layer II: 29 feet to 32 feet. ML. 0.7 tsf.

Layer III: 32 feet to 42 feet. CL. 1.6 tsf.

Layer IV: 42 feet to 48 feet. CH. 2.3 tsf.

Layer V: 48 feet to 51 feet. CL. 2.3 tsf.

Layer VI: 51 feet to 60 feet. CH. 2.1 tsf.

The water table was located at a depth of 15 feet. The soil had a complex secondary structure containing many discontinuous, randomly-oriented fissures and slickensides. Shear strength values were obtained by conducting unconsolidated, undrained triaxial compression tests on 1.4-inch-diameter by 2.8-inch-long specimens trimmed from 3-inch Shelby tube samples.

Instrumentation. The test shafts were instrumented by embedding electrical resistance gages in the concrete at several levels in each shaft. The gages were essentially strain transducers, whose output signals were converted to load by obtaining an in-shaft calibration relationship for a set of gages at the ground surface and assuming that all subsurface gages responded in the same way as the calibration gages. In one shaft an electrical load cell was used to measure load at the base of the shaft. Details of instrumentation design and installation are given in Part Two.

Loading Arrangements. Loads were applied to each test shaft by hydraulic rams which jacked against a steel reaction frame anchored on each side of the test shaft by a single drilled shaft. Applied load

was measured by means of jack pressures, and settlement was measured by using dial indicators suspended from reference beams.

Data Interpretation. Each time a load was applied to the butt of a shaft, the distribution of load along the shaft was directly obtained by reading the various gages embedded at various depths. The load transfer, or shear reaction by the soil, was calculated at several levels for each application of butt load from the slope of the load-in-shaft versus depth curve. Furthermore, the shaft displacement corresponding to each value of load transfer was measured. In this manner, developed shear stress versus displacement curves were obtained. The ratio of maximum developed shear stress indicated from such a curve to the undrained shear strength of the soil was computed at a number of levels in each shaft to determine how much of the shear strength of the soil was actually mobilized. This ratio, denoted α , was observed to vary with depth, being quite small near both the tops and bottoms of the shafts.

In no case was all of the shear strength of the soil mobilized in side shear. The mean value for α along the stems of the three shafts installed in dry boreholes was found to be 0.50, while α for the shaft placed in a processed hole was 0.33. The entire in situ shear strength of the soil cannot be mobilized because of several factors, including disturbance of the soil by augering, migration of water into the soil from the concrete, a mechanical effect which exists whereby the presence of the base inhibits load transfer development near the base, and surface soil shrinkage, which removes soil support near the surface.

Development of base resistance was also measured during each load test. The maximum net base resistance was adequately predicted from

conventional bearing capacity theory for deep foundations resting on saturated clay.

Design Categories

The results of the SH225 tests, the HB&T tests (Barker and Reese, 1970), and the review of published results of load tests of drilled shafts presented in Part One suggest several conclusions concerning the safe design of drilled shafts in Beaumont Clay. Based upon those conclusions, simple criteria for the design of drilled shafts in Beaumont Clay and similar soil formations are given in this chapter. Since the majority of the tests from which design inferences can be drawn were conducted on a short-term basis, any design criteria derived from such test results must be considered tentative as related to long-term loading.

It appears that several categories of design exist, based on shaft geometry (straight or belled), principal mode of resistance (floating or end-bearing), and construction procedure (dry method or wet method). Four common major design categories determined by shaft geometry and mode of resistance can be established which require different criteria to be used in design. Two of the four categories can be further broken down into two subcategories based upon method of construction. The proposed design categories are:

    I. Category A: Floating straight shafts in either layered or homogeneous soil.

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

B. Category A.2: Shafts in Category A installed with drilling

mud such that the possibility of entrapment of drilling

mud between the sides of the shaft and the natural soil

exists.

II. Category B: Floating belled shafts in either layered or homo-

geneous soil.

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

B. Category B.2: Shafts in Category B installed with drilling

mud such that the possibility of entrapment of drilling mud

between the sides of the shaft and the natural soil exists.

III. Category C: Straight shafts with base on soil significantly

stiffer than soil around stem.

IV. Category D: Belled shafts with base on soil significantly stiffer

than soil around stem.

Most of the design information which can be derived from field tests

concerns Categories A and B. Hence, the suggested criteria for Categories

C and D are very tentative.

Criteria for the establishment of safe working load on drilled shafts

in the various categories will be developed in the following sections.

The allowable axial load for a drilled shaft is established by pro-

viding an adequate factor of safety against plunging failure and by

limiting settlement to a permissible value. These two design principles,

although interrelated, are treated separately herein.

Safe Design Against Plunging Failure

The methods of design and design factors enumerated below are all
formulated to insure that shafts in Beaumont Clay will have an adequate
factor of safety against plunging failure. The design procedures given
in this section were developed to give the proper factor of safety against
failure while providing reasonable assurance that immediate settlement will
not be excessive. Settlement under the design load nonetheless should be
checked by the method described in the next section.

The procedures are somewhat conservative, as well as subjective in
some respects; however, field implementation of the criteria, improve-
ment of construction procedures, and future load testing of instrumented
shafts should in due course motivate a reevaluation of values for the
various parameters and, perhaps, provide greater design economy.

It is appropriate to choose a design factor of safety based upon the
plunging failure load rather than the "failure load" given by empirical
techniques such as the double tangent method commonly used by the Texas
Highway Department (see Chapter IV, Part One), since plunging load can
be calculated from laboratory or in situ strength tests.

Factors of safety against plunging are computed for values of design
load equal to half the failure load indicated by the double tangent
method for the initial load tests on the SH225 test shafts in Table 13.1.
This comparison is made for the purpose of correlating factors of safety
based on plunging load to the standard factor of safety of 2 against the
double tangent load used currently by the Texas Highway Department for
driven piles and drilled shafts.

TABLE 13.1. FACTORS OF SAFETY BASED ON PLUNGING LOAD CORRESPONDING

TO FACTORS OF SAFETY OF 2 BASED ON DOUBLE TANGENT LOAD: SH225 TESTS

Shaft	Initial Plunging Load (tons)	Double Tangent Load (tons)	Working Load = One Half Double Tangent Load (tons)	Overall F. S. at Working Load Based on Plunging Load	Gross Butt Settlement at Working Load (inches)
1	140	122	61	2.3	0.03
2	537	460	230	2.3	0.47
3	121	118	59	2.1	0.03
4	321	294	147	2.2	0.06

The overall factor of safety based on plunging load for all shafts is reasonably consistent. A conservative value of 2.5 corresponding to a factor of safety of 2.0 against the double tangent load appears appropriate for design.

The gross settlements at computed working load likewise are consistent, except for that of Shaft No. 2, which is comparatively large because most of the applied load was resisted in end bearing. Since Shaft No. 2 was a belled shaft and the others were cylindrical, it appears that merely applying a total factor of safety against plunging failure may not be sufficient for designing belled shafts. Instead, a factor of safety should be applied to the base and side resistance components separately for belled shafts in order to insure that immediate settlement is restricted. From Fig. 12.68, one observes that a factor of safety of 3 against base failure gives a gross settlement of no more than 0.5 inches for a nine-foot diameter bell. That factor of safety should be the minimum value allowed for the base reaction. As stated in previous chapters, the soil along the sides would be in a "failed" condition when the load on the base is one-third of ultimate; hence, a residual side resistance with a factor of safety of one should be used for the side resistance component in belled shafts.

The factors of safety of 2.5 for total resistance and 1 and 3 for side and base resistances, respectively, are only suggested values and may be altered by experienced designers when conditions warrant.

Calculation of Plunging Load. For design purposes, the ultimate side and base loads are computed separately using the equations given

in this section. Values for the parameters to be used in the equations are tentatively suggested for each design category in Table 13.2.

Floating shafts installed with the aid of drilling mud can be placed in Categories A.1 or B.1 if there is assurance that no drilling mud will be trapped during construction.

If soil specimens are recovered for laboratory testing, the ultimate side load, $(Q_S)_{ult}$, for shafts in Beaumont Clay can be obtained from the expression

$$(Q_S)_{ult} = \alpha_{avg} S A'_S \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (13.1)$$

in which

α_{avg} = appropriate shear strength reduction factor, given in Table 13.2

S   = mean undrained shear strength of the clay soil as determined by the procedure described in Chapter VII and outlined below

A'_S = nominal peripheral area of the stem excluding sections at the top and bottom, each equal in height to twice the stem diameter.

The value of S is determined by recovering undisturbed specimens of soil and conducting a series of triaxial tests in which each specimen is tested under a confining pressure equal to the _in situ_ overburden pressure (to the nearest five psi). If the value of S varies with depth (such as in layered soils or in soils where taking the fixed percentage of shear strength will cause the side shear to exceed the limiting value at

TABLE 13.2. DESIGN PARAMETERS FOR DRILLED SHAFTS IN BEAUMONT CLAY

Parameter		Design Category					
		A.1	A.2	B.1	B.2	C	D
Primary Procedure*	α_{avg}	0.5	0.3^A	0.3	0.15^D	0	0
	Limiting Side Shear (tsf)	0.9	0.4^B	0.4	0.25^E	0	0
	N_c	9	9	9	9	9	9
Alternate Procedure**	p	35 ($\approx$17 for Std. Pen. Test)	60^C	60	120^F	0	0
	Limiting Side Shear (tsf)	0.9	0.4^B	0.4	0.25^E	0	0
	p'	2.8 ($\approx$1.4 for Std. Pen. Test)	2.8	2.8	2.8	2.8	2.8

* Based on laboratory or in situ strength tests.

** Based on T.H.D. cone penetrometer soundings.

A α_{avg} = 0.5 for segments of shaft drilled dry.

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

C p = 35 for segments of shaft drilled dry.

D α_{avg} = 0.3 for segments of shaft drilled dry.

E Limiting side shear = 0.4 tsf for segments of shaft drilled dry.

F p = 60 for segments of shaft drilled dry.

some depth), or if the design is according to Categories A.2 or B.2 (portion of borehole processed), the value of $(Q_s)_{ult}$ should be computed by summing values from individual increments from top to bottom of the stem in order not to overestimate the side capacity.

The method just described is denoted the "primary procedure." It should be used whenever it is feasible to obtain soil samples and to conduct laboratory tests on undisturbed specimens. The second method or "alternate procedure" involves only the use of penetrometer soundings from the T.H.D. cone penetrometer. The two procedures may be mixed in the design of a shaft where necessary, such as when stiff clay is interbedded with silt layers that cannot be sampled with available equipment. The ultimate side resistance, $(Q_s)_{ult}$, for the alternate procedure is given by the expression

$$(Q_s)_{ult} = \frac{N}{p} A'_S \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (13.2)$$

in which A'_S is as defined previously and

N = number of blows per foot for T.H.D. penetrometer

p = appropriate side resistance factor given in Table 13.2

The value of N is determined to be the average penetrometer reading over the length of the stem affording resistance to load. If a large variation in N along the stem is encountered, the value of $(Q_s)_{ult}$ should be computed incrementally.

It should be emphasized that the peripheral area of the bell is not counted upon to afford any resistance and that only the soil along the

stem (excluding the top and bottom two diameters) is considered. It should also be emphasized that the side resistance is computed from the mean shear strength profile determined in the way described, and not from the cohesion intercepts of the Mohr-Coulomb envelopes.

The tabulated values for the shear strength reduction factors should be used for silty clays and clayey silts in the design of drilled shafts in the Beaumont Clay, since field tests did not verify the laboratory conclusion that higher factors could be expected from such soils.

The ultimate base capacity, $(Q_B)_{ult}$, is computed from Eq. 4.7b in the primary procedure:

$$(Q_B)_{ult} = 9 \, c_u \, A_B \quad \dots\dots\dots\dots\dots\dots\dots\dots (4.7b)$$

in which

c_u = average undrained cohesion of soil for a distance of two base diameters beneath base. In Beaumont Clay c_u can be taken to be the mean shear strength in the same distance when the mean shear strength profile is determined from triaxial tests as described previously.

A_B = nominal area of base.

In the alternate procedure:

$$(Q_B)_{ult} = \frac{N}{p'} \, A_B \quad \dots\dots\dots\dots\dots\dots\dots\dots (13.3)$$

in which

N = average number of penetrometer blows per foot for distance
of two base diameters beneath base

p' = appropriate base capacity factor given in Table 13.2

A_B = nominal area of base.

The plunging load is calculated by summing the base and side components, respectively:

$$(Q_T)_{ult} = (Q_S)_{ult} + (Q_B)_{ult} \quad \cdots \cdots \cdots \cdots \cdots \cdots (4.1)$$

Calculation of Safe Design Load. It is proposed that the safe design load be calculated in two ways.

First, the plunging load, $(Q_T)_{ult}$, is calculated as outlined under the previous heading. The result is then divided by a factor of safety of 2.5 to obtain a tentative working load for the shaft.

Second, the ultimate side load is added to the ultimate base load divided by a factor of safety of 3 to obtain another tentative working load for the shaft.

The lesser of the two tentative working loads should be taken as the design load for the shaft. The first of the two calculations will usually govern for floating straight shafts, while the second will usually govern for floating belled shafts. The latter procedure is suggested to insure that design loads on belled shafts in Beaumont Clay do not produce gross settlements in excess of 0.5 inches.

In equation form:

$$(Q_T)_{design} = \frac{(Q_T)_{ult}}{2.5} \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots (13.4)$$

or

$$(Q_T)_{design} = (Q_S)_{ult} + \frac{(Q_B)_{ult}}{3} , \quad \ldots \ldots \ldots \ldots \ldots \quad (13.5)$$

whichever gives the smaller value.

Since gross base settlement at a given percentage of ultimate capacity appears to be directly proportional to base diameter, consideration should be given to providing factors of safety for the base load of greater than 3 in the second expression for very large-diameter bells. For bells with a diameter of less than nine feet, a factor of safety of 3 is appropriate to control initial settlement. As can be observed from Fig. 12.68, when the base diameter is 15 feet, a factor of safety of 4 will be required to limit gross settlement to 0.5 inches. Thus, for bells with diameters between 9 and 15 feet, the factor of safety should be determined by linear interpolation.

Calculation of Settlement at Design Load

After the design load has been established considering failure in plunging, it is advisable to check the settlement of the butt of the shaft.

An approximate method for making a check on the immediate settlement for shafts in Beaumont Clay is to use dimensionless load-settlement graphs developed from load tests, such as the graphs shown in Figs. 12.66 and 12.68. Average relationships from those figures for use in making design estimates for immediate settlement of shafts are given in Figs. 13.1 and 13.2. The applicability of Figs. 13.1 and 13.2 to soils with significantly different stress-strain properties from those of the soil at the SH225 site is unknown. Results from load tests of instrumented shafts
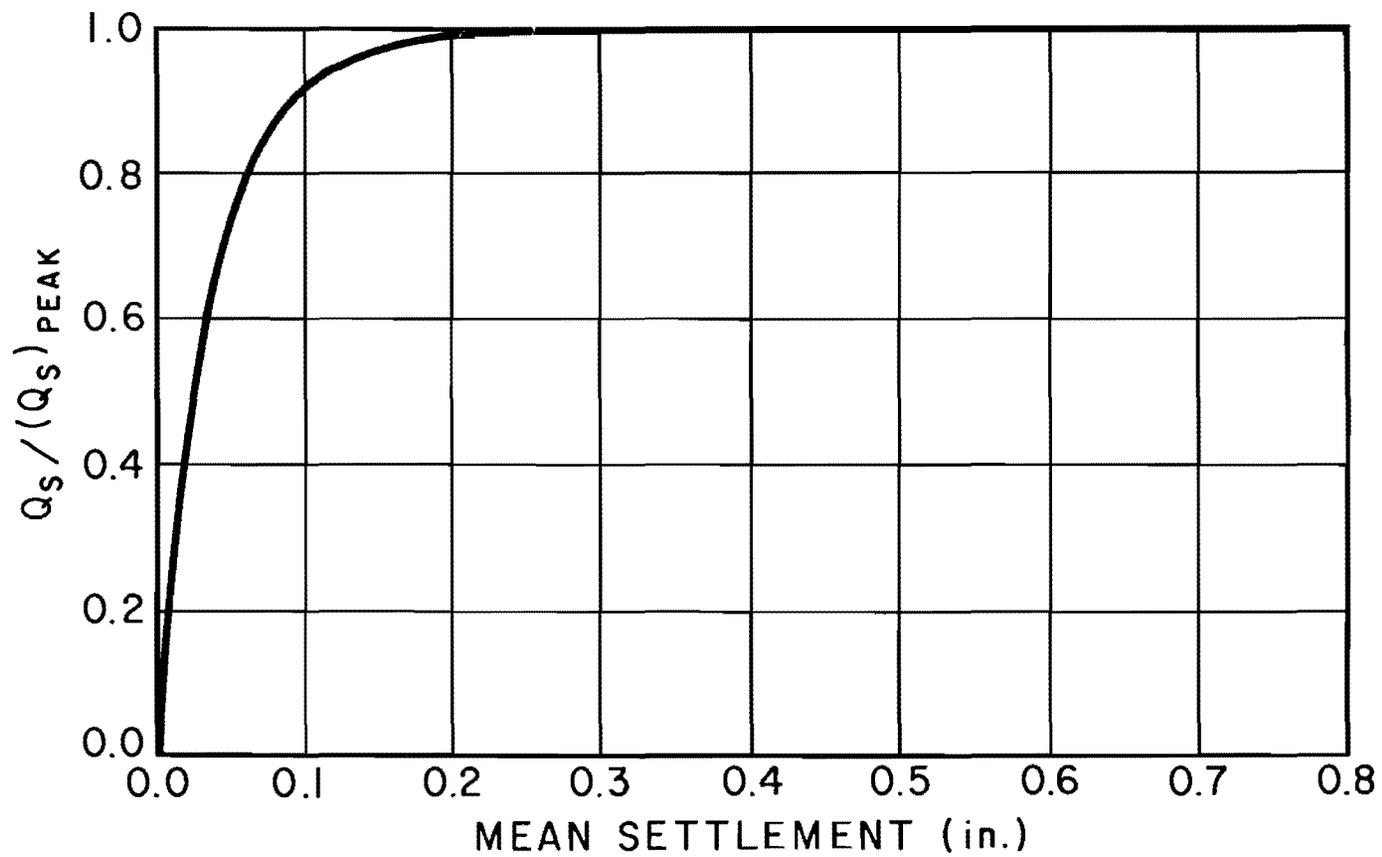
Fig. 13.1. Design Curve for Side Load Versus Settlement
for Drilled Shafts in Beaumont Clay
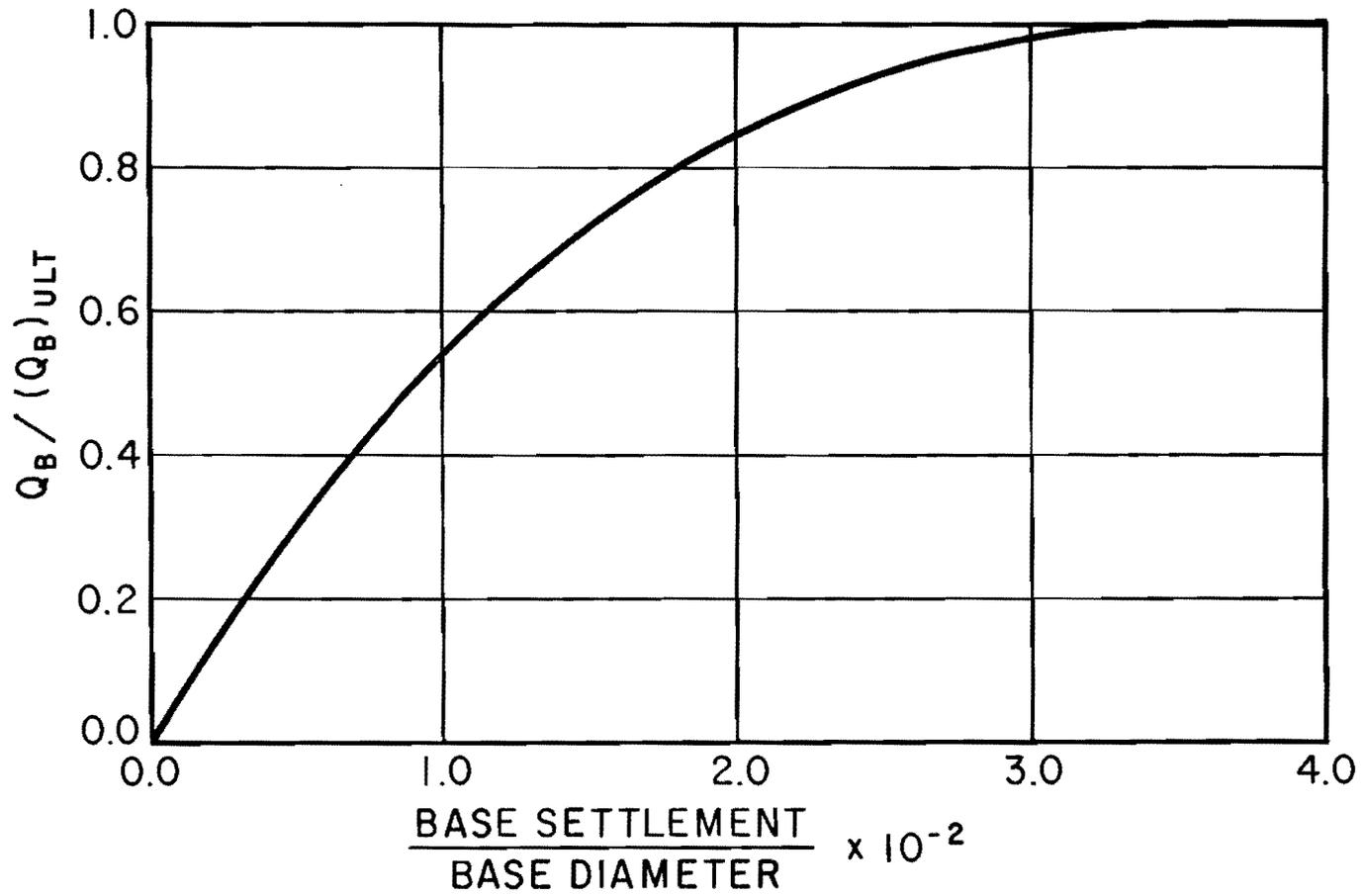
Fig. 13.2. Design Curve for Base Load Versus Settlement
for Drilled Shafts in Beaumont Clay

in other soils are necessary before such settlement graphs can be
generalized. The use of Figs. 13.1 and 13.2 is illustrated in the example
design problems which follow.

Although not usually required for design, more precise estimates of
immediate settlement can be made for applied load corresponding to the
design load or to any other value of applied load by employing one of
the methods of behavioral synthesis described in Chapter III. For example,
the load transfer curves obtained in the SH225 tests could be categorized
according to soil type, position on shaft, and method of construction,
and generalized to apply to all shafts of similar diameter installed in
Beaumont Clay. These empirical curves could then be used as input to
computational schemes, such as those described by Coyle and Reese (1966)
and Vijayvergiya, Hudson, and Reese (1969), to model the behavior of
a given shaft pursuant to design.

If necessary, long-term consolidation settlement can be estimated
by using the technique outlined in Chapter IV.

Concrete

It appears that the concrete specified for drilled shaft construction,
particularly where temporary casing is to be used, should have a minimum
slump of six inches, with a water-cement ratio of 0.6 by weight. The
customary slump presently employed is three to four inches. Furthermore,
the maximum coarse aggregate size should be limited to one inch when
temporary casing and drilling mud are to be used in order to give the
greatest assurance that concrete can flow upward between the casing and
sides of the borehole to displace the maximum amount of drilling mud.

Example Design Problems

Two example problems are now presented to illustrate the suggested design procedures.

Example Problem No. 1.

I. Field Conditions. A 36-inch-diameter straight shaft is to be installed in the Beaumont Clay at a site where the soil consists of layered stiff clay and clayey silt. Stiff clay is present from the ground surface, elevation +50 feet, to elevation +30 feet. Clayey silt is present from elevation +30 feet to elevation +25 feet. Stiff clay is again encountered from elevation +25 feet to elevation -20 feet. The shaft is to be terminated at elevation 0 feet. Exploration has indicated that the clayey silt zone is unstable. It will probably be necessary to process the hole from the surface to an elevation of +20 feet, after which the mud will be bailed out. The remaining segment (elevation +20 feet to elevation 0 feet) will be drilled in the dry. Undisturbed samples have been recovered from the upper and lower clay strata, but only T.H.D. penetrometer soundings are available from the clayey silt layer. The shear strength in the top clay layer is constant at 1.0 tsf and the shear strength in the lower clay layer is constant at 2.0 tsf. The T.H.D. penetrometer yielded an average of 15 blows per foot in the clayey silt layer.

II. Requirement. It is required to compute the working load on the shaft such that minimum factors of safety of 2.5 against total plunging failure and 3.0 against base failure are achieved.

III. <u>Solution</u>. It is necessary to compute the side capacity by an incremental procedure, as outlined below. The solution for side capacity is illustrated in Fig. 13.3.

 A. Design Category: A.2.

 B. Values of Design Parameters

  1. Elevation +50 to +30

$$\alpha_{avg} = 0.3$$

  2. Elevation +30 to +25

$$p = 60$$

  3. Elevation +25 to +20

$$\alpha_{avg} = 0.3; \text{ limiting side shear stress of 0.4 tsf.}$$

  4. Elevation +20 to 0

$$\alpha_{avg} = 0.5; \text{ limiting side shear stress of 0.9 tsf.}$$

$$N_c = 9$$

 C. Values of Unit Side Shear Capacities

  1. Elevation +50 to +30

$$(q_S)_{ult} = 0.3(1.0) = 0.30 \text{ tsf}$$

  2. Elevation +30 to +25

$$(q_S)_{ult} = \frac{15}{60} = 0.25 \text{ tsf}$$

  3. Elevation +25 to +20

$$(q_S)_{ult} = 0.3(2.0) = 0.6 \text{ tsf} > 0.4 \text{ tsf;}$$
$$\text{use 0.4 tsf}$$

  4. Elevation +20 to 0

$$(q_S)_{ult} = 0.5(2.0) = 1.0 \text{ tsf} > 0.9 \text{ tsf;}$$
$$\text{use 0.9 tsf}$$

Design Parameter	Unit Side Shear Capacity (tsf)	Effective Peripheral Area (sq. ft.)	Ultimate Side Capacity (tons)
$\alpha_{avg} = 0.3$	0.3	132	39.6
p = 60	0.25	47	11.8
Limiting Side Shear = 0.4 tsf	0.4	47	18.8
Limiting Side Shear = 0.9 tsf	0.9	132	118.7
		TOTAL	189
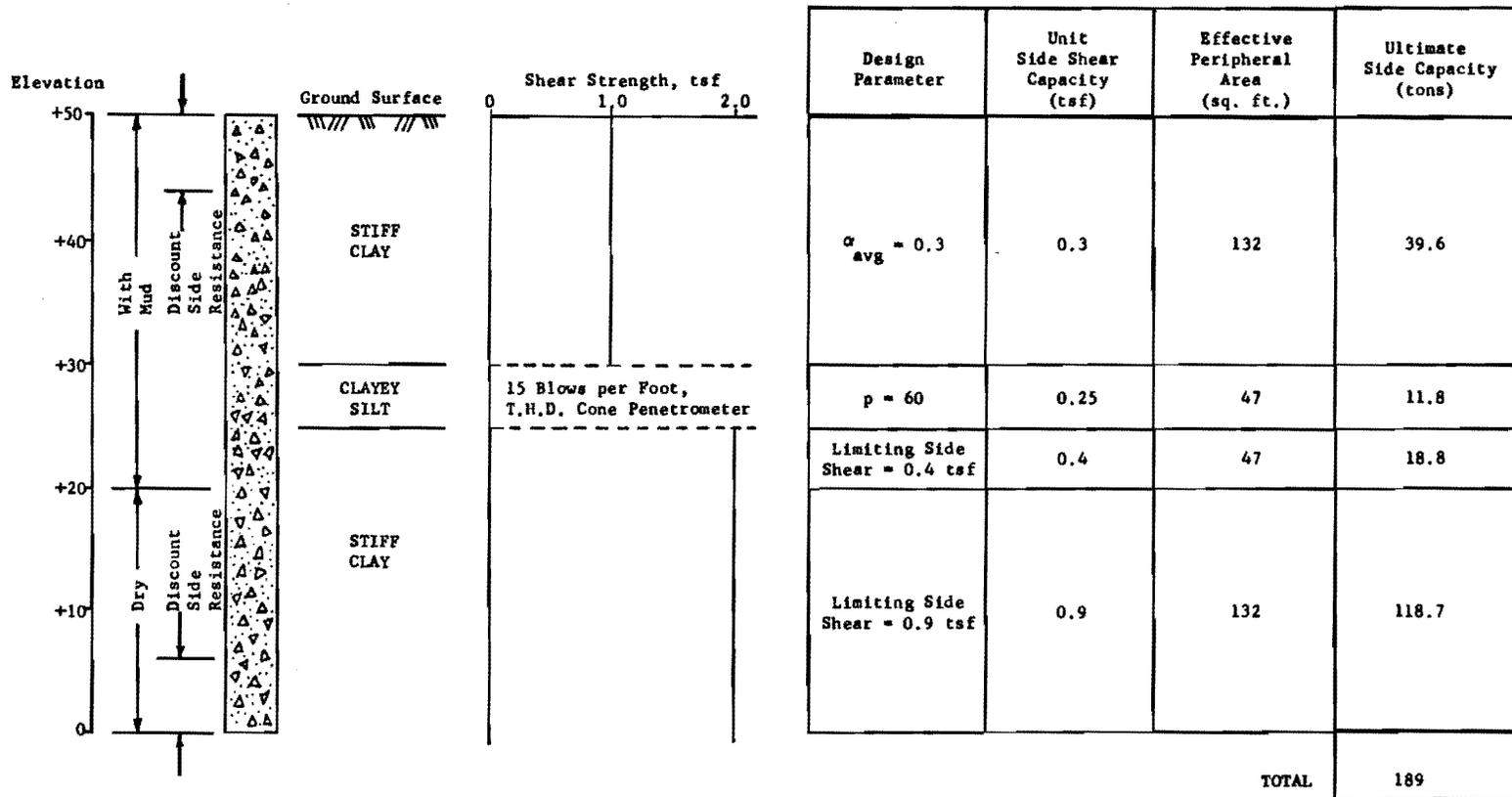
Fig. 13.3.   Side Capacity for Design Example Problem No. 1

D. Ultimate Side Capacity

   1. Elevation +50 to +30

        Effective length = 20 feet less

          top two diameters (6 feet), or 14 feet

        Peripheral area = 3π = 9.42

          square feet per linear foot

        $(Q_S)_{ult}$ = (9.42)(14)(0.30) =   39.6 tons

   2. Elevation +30 to +25

        $(Q_S)_{ult}$ = (9.42)(5)(0.25) =   11.8 tons

   3. Elevation +25 to +20

        $(Q_S)_{ult}$ = (9.42)(5)(0.4) =   18.8 tons

   4. Elevation +20 to 0

        Effective length = 20 feet less bottom

          two diameters (6 feet), or 14 feet

        $(Q_S)_{ult}$ = (9.42)(14)(0.9) =   118.7 tons

   5. Total Ultimate Side Capacity   189 tons

E. Ultimate Base Capacity:

   $(Q_B)_{ult}$ = $9(0.785)(3)^2(2.0)$ =   127 tons

F. Ultimate Shaft Capacity =   316 tons

G. Design Load Based Upon Total Factor of Safety

   of 2.5 = $\dfrac{316}{2.5}$ = <u>126 tons</u>

H. Design Load Based Upon Side Factor of Safety

   of 1 and Base Factor of Safety of 3

   = $189 + \dfrac{127}{3}$ = <u>231 tons</u> > 126 tons

I. Use Design Load Based Upon Total Factor of Safety

   of 2.5, or <u>126 tons</u>

J. Check Approximate Immediate Settlement From Figs. 13.1
and 13.2

1. Using trial and error procedure, assume immediate
butt settlement is 0.05 inches. Also assume butt
settlement equals mean shaft settlement (shaft
incompressible). From Fig. 13.1, mean settle-
ment of 0.05 inches produces 75 per cent of maxi-
mum side resistance, or 0.75 (189) = 142 tons.
Assuming base settlement equals butt settlement,

$$\frac{\text{Base settlement}}{\text{Base diameter}} = \frac{0.05}{36} = 0.14 \times 10^{-2} \text{ ,}$$

or according to Fig. 13.2, about 10 per cent of
the ultimate base load will be developed (13 tons).
The approximate butt load corresponding to a butt
settlement of 0.05 inches is, then, 142 + 13 = 155
tons > 126 tons. Try a smaller settlement.

2. Using the same method as in Step 1, assume butt
settlement = 0.03 inches. From Fig. 13.1, 62
per cent of maximum side resistance will be devel-
oped (117 tons). From Fig. 13.2, 6 per cent of
maximum base resistance will be developed (8 tons).
The butt load corresponding to an immediate butt
settlement of 0.03 inches is 117 + 8 = 125 ≈ 126 tons.
Therefore, the estimated immediate butt settlement
at design load is 0.03 inches.

Example Problem No. 2.

    I. Field Conditions. A belled shaft is to be installed in stiff clay at a site in the Beaumont Clay formation at which all boring and belling operations are to be accomplished in the dry. The stem is to be 30 inches in diameter, extending from the ground surface, elevation +25 feet, to -5 feet. The bell will have a base diameter of 90 inches, and the base will be set at an elevation of -9.5 feet. Laboratory testing indicated that the shear strength of the stiff clay soil varies linearly from 0.6 tsf at the surface to 2.6 tsf at an elevation of -15 feet. The stiff clay is underlain by a silty layer from -15 feet to -25 feet having a shear strength of 0.7 tsf.

    II. Requirement. It is required to compute the working load on the shaft such that minimum factors of safety of 2.5 against total plunging failure and 3.0 against base failure are achieved.

    III. Solution. It is necessary to compute the side capacity by an incremental procedure. The solution for side capacity is illustrated in Fig. 13.4.

        A. Design Category: B.1.

        B. Depth at which maximum allowable shear stress is reached

            1. Allowable side shear stress = (0.3)S or 0.4 tsf

            2. Elevation at which (0.3)S = 0.4 tsf is +10.4 feet

            3. Therefore, take increments as +25 to +10.4 and +10.4 to -5.

        C. Values of Design Parameters
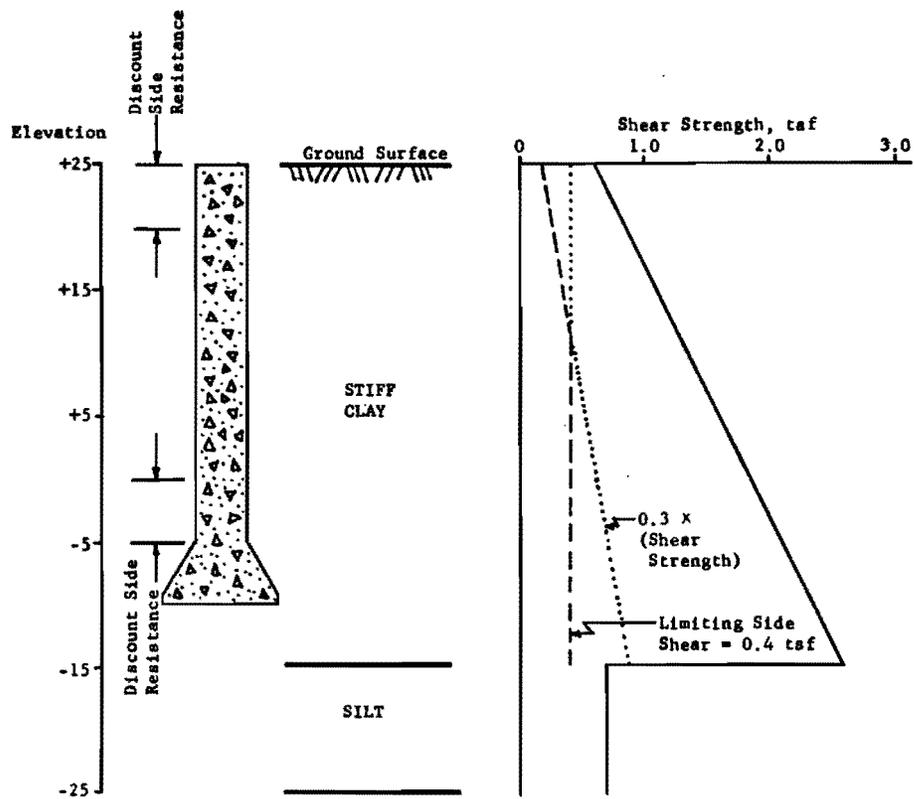
Design Parameter	Unit Side Shear Capacity (tsf)	Effective Peripheral Area (sq. ft.)	Ultimate Side Capacity (tons)
α_{avg} = 0.3	0.33	75.3	24.8
Limiting Side Shear = 0.4 tsf	0.4	81.5	32.6
		TOTAL	57.4

Fig. 13.4.   Side Capacity for Design Example Problem No. 2

1. Elevation +25 to +10.4

$$\alpha_{avg} = 0.3$$

2. Elevation +10.4 to -9.5

Limiting side shear stress = 0.4 tsf

$$N_c = 9$$

D. Values of Unit Side Shear Capacities

1. Elevation +25 to +10.4

$$(q_S)_{ult} = 0.3 \ \frac{0.85 + 1.33}{2} = 0.33 \text{ tsf}$$

2. Elevation +10.4 to -5

$$(q_S)_{ult} = 0.4 \text{ tsf}$$

E. Ultimate Side Capacity

1. Elevation +25 to +10.4

Effective length = 9.6 feet

Peripheral area = 2.5π = 7.85 square

feet per linear foot

$$(Q_S)_{ult} = (7.85)(9.6)(0.33) = 24.8 \text{ tons}$$

2. Elevation +10.4 to -5

Effective length = 10.4 feet

$$(Q_S)_{ult} = (7.85)(10.4)(0.4) = 32.6 \text{ tons}$$

3. Total Ultimate Side Capacity   57.4 tons

F. Ultimate Base Capacity

1. Average shear strength for two base diameters

(15 feet) beneath base

$$= \frac{(2.44)(5.5) + (0.7)(9.5)}{15} = 1.34 \text{ tsf}$$

2. $(Q_B)_{ult} = 9(0.785)(7.5)^2(1.34) =$ 533 tons

G.   Ultimate Shaft Capacity =   590 tons

H.   Design Load Based Upon Total Factor of Safety

    of 2.5   = $\dfrac{590}{2.5}$ =   236 tons

I.   Design Load Based Upon Base Factor of Safety

    of 3 and Side Factor of Safety of 1

    = $\dfrac{533}{3}$ + 57.4   =   235 tons < 236 tons

J.   Use Design Load of 235 tons

K.   Settlement check:

Following the procedure outlined in Example Problem No. 1, the indicated immediate settlement is about 0.5 inches. This value may be somewhat low because of the thick layer of silty soil present beneath the base.

Implementation of Design Procedures

The implementation by the Texas Highway Department of the suggested design procedures would represent a step toward more economical design for drilled shafts. Since the real factors of safety for shafts designed using these procedures would be less than those on shafts installed under present design methods, inspection of the installation of drilled shafts, therefore, should be conducted with added care.

CHAPTER XIV

CONCLUSIONS AND RECOMMENDATIONS

Based upon the results of the load tests of the four instrumented test shafts at the SH225 site, several conclusions concerning the behavior of axially loaded drilled shafts in Beaumont Clay have been drawn. Those conclusions, elaborated upon in previous chapters, are enumerated concisely below.

1. Floating drilled shafts resist an appreciable portion of the applied load in side friction.

2. The side resistance was found to vary with depth, becoming smaller near the top and bottom of the shaft. The reduced side resistance near the top appeared to be caused by drying of the soil surrounding the top of the stem which reduced lateral pressure between the soil and the concrete shaft. The reduced load transfer near the base appeared to be the result of increased wetting of the soil immediately adjacent to the concrete near the bottom of the stem and to mechanical base-side interference.

3. The maximum shear stress developed at or near the shaft-soil interface was less than the shear strength of the soil. This diminished shear capacity appeared to be the combined result of softening of the soil immediately adjacent to the shaft due to migration of excess water from the concrete, of surface and base effects, of shear strength reduction

due to remolding the soil along the borehole walls and opening of fissures during boring operations, and of drilling mud trapped between the concrete and soil when the shaft is installed in a processed hole.

4. Except for small mechanical base-side interaction differences, shaft geometry had no influence on behavior during initial loading. The belled shaft exhibited a greatly reduced side capacity upon delayed reloading, possibly reflecting the fact that the displacement required to mobilize the full capacity of that shaft was large in comparison to the displacement required to mobilize the full capacity of the straight shafts, thereby causing the strength of the soil along the sides to be greatly reduced.

5. The use of the wet process to install Test Shaft No. 4 caused a significant decrease in average side capacity, primarily because drilling mud was trapped between the concrete shaft and the soil and because some of the concrete at the soil-shaft interface was of very poor quality. That part of Test Shaft No. 4 drilled dry (bottom five feet) developed a side resistance comparable to that of the other three test shafts, which had been installed entirely in the dry.

6. The bearing capacity of the base can be estimated reliably from the usual bearing capacity theory for clay, using the average strength for two base diameters beneath the base as the appropriate value for undrained cohesion in the bearing capacity equation.

7. The instrumentation of drilled shafts to measure load
   distribution was shown to be feasible. The Mustran system
   proved to be an outstanding instrumentation scheme. The
   concrete embedment gages, when wired in full bridge con-
   figuration in the "improved tee," performed well, except
   that load resolution was not as good as with Mustran cells.
   The bottomhole cell performed adequately, as did the weld-
   able gages (except for rather low resolution). The tell-
   tales did not yield acceptable results. The hydraulic
   pressure cell also gave poor results.

8. The method of top-level, in-shaft calibration is adequate
   for shafts installed in the dry. Processed shafts, or
   those with otherwise irregular sides, are best calibrated
   by excavating soil from around the stem and reloading the
   shaft to obtain direct load-reading curves for gages in the
   freestanding portion.

9. Perhaps the greatest uncertainty related to the design of
   drilled shafts in Beaumont Clay is the adequate estimation
   of in situ soil properties. Shear strength profiles deter-
   mined by six different methods exhibited considerable
   variation. Those given by the T.H.D. cone penetrometer
   and the transmatic triaxial test had the most conservative
   values for shear strength. The variation in indicated
   shear strength reflects sampling disturbance and differ-
   ences in testing techniques, and it attests to the need
   for the development of better in situ strength measurement.

10. When the soil is fissured, considerable scatter in indicated shear strength exists. The mean profile is an appropriate representation of the soil strength as a function of depth, and side and base capacities may be computed from such a profile.

The following recommendations are made concerning the research reported herein.

1. The design procedure outlined in Chapter XIII should be incorporated as a tentative method for sizing drilled shafts in Beaumont Clay. If that procedure is implemented, particular attention should be paid to careful inspection during installation and to obtaining serviceability records for shafts designed by the new procedure.

2. Several of the categories of design listed in Chapter XIII remain inadequately investigated, requiring that values of design parameters be quite conservative. Therefore, it is important that long-term tests be conducted, particularly on belled shafts, to determine the effects of sustained loading on side capacity. Further investigations on the effect of varying the method of installing a shaft with the aid of drilling mud should also be made. These investigations would increase the confidence in the recommended values for shear strength reduction factors for processed shafts and would hopefully lead to an installation procedure whereby drilling mud can be consistently expelled completely from the borehole.

3. Testing of drilled shafts should be extended to other soil formations in order to develop general design procedures which can be used confidently at any location. Test shafts in such studies must be instrumented, and tests must be conducted to plunging failure in order to advance the state of the art. Mere proof testing, in which the test shaft is not failed, should be avoided when possible.

REFERENCES

Al-Layla, M. T. H. (1970), <u>Study of Certain Geotechnical Properties of Beaumont Clay</u>, thesis presented to Texas A&M University, at College Station, Texas, in 1970, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

American Concrete Institute (1963), "Recommended Practices for Concrete Formwork," <u>Standards</u>, American Concrete Institute, 1963, pp. 347-363.

American Railway Engineering Association (1950), "Steel and Timber Pile Tests - West Atchafalaya Floodway - New Orleans, Texas and Mexico Railway," <u>A.R.E.A. Bulletin</u>, Vol. 52, No. 489, September - October, 1950.

American Society for Testing and Materials (1970), <u>1970 Book of A.S.T.M. Standards</u>, Part 11, April, 1970.

Aschenbrenner, R. (1967), "Three-Dimensional Analysis of Pile Foundations," <u>Journal of the Structural Division</u>, A.S.C.E., Vol. 93, No. ST1, February, 1967.

Baker, C. N., Jr., and Kahn, F. (1969), "Caisson Construction Problems and Methods of Correction," Paper presented to 1969 A.S.C.E. Annual Meeting, Chicago, Illinois, Meeting Preprint No. 1020, October, 1969.

Barden, L., and Monckton, M. F. (1970), "Tests on Model Pile Groups in Soft and Stiff Clay," <u>Geotechnique</u>, Vol. XX, No. 1, London, March, 1970.

Barker, W. R., and Reese, L. C. (1969), <u>Instrumentation for Measurement of Axial Load in Drilled Shafts</u>, 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), <u>Load-Carrying Characteristics of Drilled Shafts Constructed with the Aid of Drilling Fluids</u>, Research Report 89-9, Center for Highway Research, The University of Texas at Austin, Austin, Texas, 1970.

Bernard, H. A., LeBlanc, R. J., and Major, C. F. (1962), "Recent and Pleistocene Geology of Southeast Texas," <u>Geology of the Gulf Coast and Central Texas</u>, Houston Geological Society, Houston, Texas, 1962, pp. 175-224.

Bhanot, K. L. (1968), <u>Behaviour of Scaled and Full-Length Cast-in-Place Concrete Piles</u>, thesis presented to The University of Alberta, at Edmonton, Alberta, in 1968, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Brooker, E. W., and Ireland, H. O. (1965), "Earth Pressures at Rest Related to Stress History," Canadian Geotechnical Journal, Vol. II, No. 1, February, 1965.

Burland, J. B. (1963), Discussion in Grouts and Drilling Muds in Engineering Practice, Butterworths, London, 1963, pp. 223-225.

Burland, J. B., Butler, F. G., and Dunican, P. (1966), "The Behaviour and Design of Large Diameter Bored Piles in Stiff Clay," Proceedings of the Symposium on Large Bored Piles, Institution of Civil Engineers, London, February, 1966.

Cambefort, H. (1953), "Le Comportement Des Pieux Forés Et Les Essais De Pénétration," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, 1953.

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

Carson, A. B. (1965), Foundation Construction, McGraw-Hill, New York, 1965.

Chadeisson, R. (1961), "Influence Du Mode De Perforation Sur Le Comportement Des Pieux Forés Et Moulés Dans Le Sol," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, 1961.

Chandler, R. J. (1968), "The Shaft Friction of Piles in Cohesive Soils in Terms of Effective Stress," Civil Engineering and Public Works Review, London, January, 1968.

Chellis, R. D. (1961), Pile Foundations, Second Edition, McGraw-Hill, New York, 1961.

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

Collins, L. E. (1953), "A Preliminary Theory for the Design of Under-reamed Piles," Transactions of South African Institution of Civil Engineers, Vol. 3, No. 11, Pretoria, 1953.

Coyle, H. M., and Reese, L. C. (1966), "Load Transfer for Axially Loaded Piles in Clay," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 92, No. SM2, March, 1966.

Crandall, L. L. (1948), "Electrical Resistance Strain Gauges for Determining the Transfer of Load from Driven Piling to Soil," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, 1948.

Cummings, A. E. (1949), "Lectures on Foundation Engineering," University of Illinois Bulletin, Vol. 47, No. 35, Urbana, Illinois, December, 1949.

D'Appolonia, E., and Romualdi, A. M. (1963), "Load Transfer in End-Bearing Steel H-Piles," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 89, No. SM2, March, 1963.

Davisson, M. T., and Salley, J. R. (1969), "Lateral Load Tests on Drilled Piers," Performance of Deep Foundations, A.S.T.M. S.T.P. 444, American Society for Testing and Materials, 1969.

Deb, A. K., and Chandra, S. (1964), "Short Bored Piles for Foundations in Expansive Clays," Proceedings of the Symposium on Bearing Capacity of Piles, Central Building Research Institute, Roorkee, India, 1964.

De Mello, V. F. B. (1969), "Foundations of Buildings In Clay," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, State of the Art Volume, 1969.

DuBose, L. A. (1955), "Load Studies of Drilled Shafts," Proceedings, Highway Research Board, Vol. 34, Washington, D. C., 1955.

DuBose, L. A. (1956), A Comprehensive Study of Factors Influencing the Load Carrying Capacities of Drilled and Cast-in-Place Concrete Piles, Texas Transportation Institute, A&M College of Texas, College Station, Texas, 1956.

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

Ellison, R. D. (1968), An Analytical Study of the Mechanics of Single Pile Foundations, thesis presented to Carnegie-Mellon University, at Pittsburgh, Pennsylvania, in 1968, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Evans, L. T. (1952), The Drilled Pile, Edwards Brothers, Ann Arbor, Michigan, 1952.

Fernandez-Renau, L. (1965), Discussion in Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, Vol. III, 1965, pp. 495-496.

Fleming, W. G. K., and Salter, T. H. (1962), "Reports on Loading Tests on a Large Diameter Underreamed Bored Pile," Civil Engineering and Public Works Review, London, October, 1962.

Frischmann, W. W., and Fleming, W. G. K. (1962), "The Use and Behaviour of Large Diameter Piles in London Clay," The Structural Engineer, London, April, 1962.

Fuller, F. M., and Hoy, H. E. (1970), "Pile Load Tests Including Quick Load Test Method, Conventional Methods, and Interpretations," Paper presented to National Meeting, Highway Research Board, Washington, D. C., January, 1970.

Geddes, J. D. (1966), "Stresses in Foundation Soils Due to Subsurface Loading," Geotechnique, Vol. XVI, No. 3, London, September, 1966.

Geddes, J. D. (1969), "Boussinesq-Based Approximations to the Vertical Stresses Caused by Pile-Type Subsurface Loadings," Geotechnique, Vol. XIX, No. 4, London, December, 1969.

Glossop, R. and Greeves, I. S. (1946), "A New Form of Bored Pile," Concrete and Constructional Engineering, London, December, 1946.

Gnaedinger, J. P. (1964), Discussion in Proceedings, Conference on Deep Foundations, Vol. II, Mexican Society of Soil Mechanics, Mexico City, 1964, pp. 625-627.

Golder, H. Q., and Leonard, M. W. (1954), "Some Tests on Bored Piles in London Clay," Geotechnique, Vol. IV, No.1, London, March, 1954.

Green, H. (1961), "Long-Term Loading of Short Bored Piles," Geotechnique, Vol. XI, No. 1, London, March, 1961.

Greer, D. M. (1969), "Drilled Piers State of the Art, 1969," W.C.A. Geotechnical Bulletin, Vol. III, No. 2, Woodward-Clyde and Associates, New York, September, 1969.

Hansen, V., and Kneas, F. N. (1942), "Static Load Tests for Bearing Piles," Civil Engineering, A.S.C.E., Vol. 12, No. 10, October, 1942.

Harris, F. A. (1951), "Plum Creek Load Test Results, Progress Report No. 2 on Skin Friction Investigation," Texas Highway Department, Houston Urban Expressways Office, May, 1951.

Hobbs, N. B. (1963), "An Analytical Approach to the Bearing Capacity and Settlement of the Large Diameter Bored Pile," Proceedings of the Institution of Civil Engineers, Vol. 25, London, 1963, Paper No. 6616.

Horner, J. M. (1969), Uplift Resistance of Straight Auger and Belled Auger Footings in Sandy Silt, United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, November, 1969.

Housel, W. S. (1969), "Design of Caissons on Granular-Cohesive Soils," Performance of Deep Foundations, A.S.T.M. S.T.P. 444, American Society for Testing and Materials, 1969.

Janbu, N., Bjerrum, L., and Kjaernsli (1956), "Soil Mechanics Applied to Some Engineering Problems," (in Norwegian), Norwegian Geotechnical Institute Bulletin No. 16, Oslo, 1956.

Kerisel, J. L. (1964), "Deep Foundations - Basic Experimental Facts," Proceedings, Conference on Deep Foundations, Mexican Society of Soil Mechanics, Vol. I, Mexico City, 1964.

Kerisel, J. L. (1967), "Vertical and Horizontal Bearing Capacity of Deep Foundations in Clay," Proceedings, Symposium on Bearing Capacity and Settlement of Foundations, Duke University, Durham, North Carolina, 1967.

Komornik, A., and Wiseman, G. (1967), "Experience with Large Diameter Cast-in-Situ Piling," Proceedings, Third Asian Regional Conference on Soil Mechanics and Foundation Engineering, Jerusalem Academic Press, Jerusalem, 1967.

Lambe, T. W., and Whitman, R. V. (1969), Soil Mechanics, Wiley, New York, 1969.

Leonards, G. A. (1962), "Engineering Properties of Soils," Foundation Engineering, ed. by G. A. Leonards, McGraw-Hill, New York, 1962.

Lysmer, J., and Duncan, J. M. (1969), Stresses and Deflections in Foundations and Pavements, Fourth Edition, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, California, 1969.

Martins, J. B. (1963), "Pile Load Tests on the River Pungué," Proceedings, Third Regional Conference for Africa on Soil Mechanics and Foundation Engineering, The Rhodesian Institution of Engineers, Salisbury, 1963.

Matich, M. A. J., and Kozicki, P. (1967), "Some Load Tests on Drilled Cast-in-Place Concrete Caissons," Canadian Geotechnical Journal, Vol. IV, No. 4, November, 1967.

Matlock, H., and Reese, L. C. (1962), "Generalized Solutions for Laterally Loaded Piles," Transactions, A.S.C.E., Vol. 127, Part I, 1962.

Mattes, N. S., and Poulos, H. G. (1969), "Settlement of Single Compressible Pile," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 95, No. SM1, January, 1969.

McClelland, B., Focht, J. A., Jr., and Emrich, W. J. (1969), "Problems in Design and Installation of Offshore Piles," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 95, No. SM6, November, 1969.

Meyerhof, G. G. (1951), "The Ultimate Bearing Capacity of Foundations," Geotechnique, Vol. II, No. 4, London, December, 1951.

Meyerhof, G. G., and Adams, J. I. (1968), "The Ultimate Uplift Capacity of Foundations," Canadian Geotechnical Journal, Vol. V, No. 4, November, 1968.

Meyerhof, G. G., and Murdock, L. J. (1953), "An Investigation of the Bearing Capacity of Some Bored and Driven Piles in London Clay," Geotechnique, Vol. III, No. 7, London, September, 1953.

Mindlin, R.D. (1936), "Force at a Point in the Interior of a Semi-Infinite Solid," Physics, Vol. 7, May, 1936.

Mohan, D., and Chandra, S. (1961), "Frictional Resistance of Bored Piles in Expansive Clays," Geotechnique, Vol. XI, No. 4, London, December, 1961.

Mohan, D., and Jain, G. S. (1961), "Bearing Capacity of Bored Piles in Expansive Clays," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, 1961.

Mohan, D., Jain, G. S., and Jain, M. P. (1967), "A New Approach to Load Tests," Geotechnique, Vol. XVII, No. 3, London, September, 1967.

Mohan, D., Murthy, V. N. S., and Jain, G. S. (1969), "Design and Construction of Multi-Under-Reamed Piles," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, 1969.

Moorhouse, D. C., and Sheehan, J. V. (1968), "Predicting Safe Capacity of Pile Groups," Civil Engineering, A.S.C.E., Vol. 38, No. 10, October, 1968.

Nair, K. (1967), "Load Settlement and Load Transfer Characteristics of a Friction Pile Subject to a Vertical Load," Proceedings, Third Pan American Conference on Soil Mechanics and Foundation Engineering, Vol. I, Venezuela, 1967.

Nair, K., Gray, H., and Donovan, N. C. (1969), "Analysis of Pile Group Behavior," Performance of Deep Foundations, A.S.T.M. S.T.P. 444, American Society for Testing and Materials, 1969.

Neville, A. M. (1963), Properties of Concrete, Wiley, New York, 1963.

Osgerby, C., and Taylor, P. T. (1968), "Vibrating Wire Load Cell," Experimental Mechanics, Vol. 8, No. 9, September, 1968.

Osterberg, J. O. (1968), "Drilled Caissons - Design, Installation, Application," Chicago Soil Mechanics Lectures - 1968, Northwestern University, Evanston, Illinois, 1968.

Palmer, D. J., and Holland, G. R. (1966), "The Construction of Large Diameter Bored Piles with Particular Reference to London Clay," Proceedings of the Symposium on Large Bored Piles, Institution of Civil Engineers, London, February, 1966.

Pandey, V. J. (1967), "Some Experiences with Bored Piling," Journal of the Soil Mechanics and Foundations Division, A.S.C.E. , Vol. 93, No. SM5, September, 1967, Part 1.

Peck, R. B. (1965), "Pile and Pier Foundations," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 91, No. SM2, March, 1965.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1953), Foundation Engineering, Wiley, New York, 1953.

Potyondy, J. G. (1961), "Skin Friction Between Various Soils and Construction Materials," Geotechnique, Vol. XI, No. 4, London, December, 1961.

Poulos, H. G. (1968), "Analysis of the Settlement of Pile Groups," Geotechnique, Vol. XVIII, No. 4, London, December, 1968.

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

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.

Reese, L. C., Hudson, W. R., and Vijayvergiya, V. N. (1969), "An Investigation of the Interaction Between Bored Piles and Soil," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, 1969.

Reese, L. C., O'Neill, M. W., and Smith, R. E. (1970), "Generalized Analysis of Pile Foundations," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 96, No. SM1, January, 1970.

Robinsky, E. I., and Morrison, C. F. (1964), "Sand Displacement and Compaction Around Model Friction Piles," Canadian Geotechnical Journal, Vol. I, No. 2, March, 1964.

Salas, J. A. J., and Belzunce, J. A. (1965), "Résolution Théorique De La Distribution Des Forces Dans Des Pieux," Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, 1965.

Saul, W. E. (1968), "Static and Dynamic Analysis of Pile Foundations," Journal of the Structural Division, A.S.C.E., Vol. 94, No. ST5, May, 1968.

Schlitt, H. G. (1952), "Group Pile Load Tests in Plastic Soils," Proceedings, Highway Research Board, Vol. 31, Washington, D. C., 1952.

Seed, H. B., and Reese, L. C. (1957), "The Action of Soft Clay Along Friction Piles," Transactions, A.S.C.E., Vol. 122, 1957.

Seed, H. B., and Reese, L. C. (1964), "Predicting the Performance of Friction Piles in Saturated Clays," Proceedings, Conference on Deep Foundations, Mexican Society of Soil Mechanics, Vol. I, Mexico City, 1964.

Sellards, E. H., Adkins, W. S., and Plummer, F. B. (1932), The Geology of Texas, Volume I, Stratigraphy, Bureau of Economic Geology, The University of Texas at Austin, Austin, Texas, August, 1932.

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

Skempton, A. W. (1959), "Cast-in-Situ Bored Piles in London Clay," Geotechnique, Vol. IX, No. 4, London, December, 1959.

Skempton, A. W., and Hutchinson, J. (1969), "Stability of Natural Slopes and Embankment Foundations," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, State of the Art Volume, 1969.

Skipp, B. O. (1966), Discussion in Proceedings of the Symposium on Large Bored Piles, Institution of Civil Engineers, London, February, 1966, pp. 146-147.

Snow, R. (1965), "Telltales," Foundation Facts, Vol. 1, No. 2, Raymond International, New York, Fall, 1965.

Sowers, G. B., and Sowers, G. F. (1961), Introductory Soil Mechanics and Foundations, Second Edition, MacMillan, New York, 1961.

Sowers, G. F. (1962), "Shallow Foundations," Foundation Engineering, ed. by G. A. Leonards, McGraw-Hill, New York, 1962.

Taylor, D. W. (1948), Fundamentals of Soil Mechanics, Wiley, New York, 1948.

Terzaghi, K. (1943), Theoretical Soil Mechanics, Wiley, New York, 1943.

Texas Highway Department (1962), Standard Specifications for Road and Bridge Construction, State Highway Department of Texas, Austin, Texas, January, 1962.

Texas Highway Department (1964), Foundation Exploration and Design Manual, State Highway Department of Texas, Bridge Division, Austin, Texas, January, 1964.

Texas Highway Department (1965), Special Provision to Item 405, Test Loading Piling, Standard Specifications for Road and Bridge Construction, State Highway Department of Texas, Austin, Texas, July, 1965.

Texas Transportation Institute (1965), Bibliography 65-10, Drilled Shafts, Texas Transportation Institute, Texas A&M University, College Station, Texas, 1965.

Thurman, A. G., and D'Appolonia, E. (1965), "Computed Movement of Friction and End-Bearing Piles Embedded in Uniform and Stratified Soils," Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, 1965.

Tomlinson, M. J. (1969), Foundation Design and Construction, Second Edition, Wiley-Interscience, New York, 1969.

Trollope, D. H., Freeman, McD., and Peck, G. M. (1966), "Tasman Bridge Foundations," The Journal of the Institution of Engineers, Australia, June, 1966.

Turner, E. A. (1962), "Uplift Resistance of Transmission Tower Footings," Journal of the Power Division, A.S.C.E., Vol. 88, No. PO2, July, 1962.

U. S. Army Corps of Engineers (1965), Standard Practice for Concrete, Engineer Manual 1110-2-2000, Department of the Army, Washington, D. C., December, 1965.

U. S. Army Engineer District, Fort Worth, Texas (1968), Investigation for Building Foundations in Expansive Clays, U. S. Army Corps of Engineers, Fort Worth, Texas, April, 1968.

Van Doren, L. M., Hazard, S. G., Stallings, J. R., and Schnacke, D. P. (1967), Drilled Shaft Foundation Test Program for State Highway Commission of Kansas, Van Doren-Hazard-Stallings-Schnacke Engineers-Architects, Topeka, Kansas, August, 1967.

Van Siclen, D. C. (1961), "Surface Geology in and Near Houston," Geology of Houston and Vicinity, Texas, Houston Geological Society, Houston, Texas, 1961.

Van Weele, A. F. (1957), "A Method of Separating the Bearing Capacity of a Test Pile into Skin-Friction and Point-Resistance," <u>Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering</u>, 1957.

Vesić, A. (1963), "Bearing Capacity of Deep Foundations in Sand," <u>Highway Research Record No. 39</u>, Highway Research Board, Washington, D. C., 1963.

Vesić, A. (1967), "Ultimate Loads and Settlements of Deep Foundations in Sand," <u>Proceedings, Symposium on Bearing Capacity and Settlement of Foundations</u>, Duke University, Durham, North Carolina, 1967.

Vesić, A. (1969), "Experiments with Instrumented Pile Groups in Sand," <u>Performance of Deep Foundations, A.S.T.M. S.T.P. 444</u>, American Society for Testing and Materials, 1969.

Vesić, A. (1970), "Tests on Instrumented Piles, Ogeechee River Site," <u>Journal of the Soil Mechanics and Foundations Division</u>, A.S.C.E, Vol. 96, No. SM2, March, 1970.

Vijayvergiya, V. N., Hudson, W. R., and Reese, L. C. (1969), <u>Load Distribution for a Drilled Shaft in Clay Shale</u>, Research Report No. 89-5, Center for Highway Research, The University of Texas at Austin, Austin, Texas, March, 1969.

Ward, W. H., Samuels, S. G., and Butler, M. E. (1959), "Further Studies of the Properties of London Clay," <u>Geotechnique</u>, Vol. IX, No. 2, London, June, 1959.

Watt, W. G., Kurfurst, P. J., and Zeman, Z. P. (1969), "Comparison of Pile Load-Test-Skin-Friction Values and Laboratory Strength Tests," <u>Canadian Geotechnical Journal</u>, Vol. VI, No. 3, August, 1969.

Whitaker, T. (1957), "Experiments with Model Piles in Groups," <u>Geotechnique</u>, Vol. VII, No. 4, London, December, 1957.

Whitaker, T., and Cooke, R. W. (1961), "A New Approach to Pile Testing," <u>Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering</u>, 1961.

Whitaker, T., and Cooke, R. W. (1966), "An Investigation of the Shaft and Base Resistance of Large Bored Piles in London Clay," <u>Proceedings of the Symposium on Large Bored Piles</u>, Institution of Civil Engineers, London, February, 1966.

Whitaker, T., Cooke, R. W., and Clarke, G. W. (1962), "100 Ton Load Cell for Pile Loading Tests," <u>Engineering</u>, Vol. 194, No. 5040, London, November 23, 1962.

White, E. E. (1967), "Developments in Large Diameter Piles or Caissons in the United States," Proceedings, Third Asian Regional Conference on Soil Mechanics and Foundation Engineering, Jerusalem Academic Press, Jerusalem, 1967.

Williams, G. M. J., and Colman, R. B. (1965), "The Design of Piles and Cylinder Foundations in Stiff, Fissured Clay," Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, 1965.

Woodward, R. J., Lundgren, R., and Boitano, J. D., Jr. (1961), "Pile Loading Tests in Stiff Clay," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, 1961.

Zienkiewicz, O. C. (1967), The Finite Element Method in Structural and Continuum Mechanics, McGraw-Hill, London, 1967.