BEHAVIOR OF AXIALLY LOADED DRILLED SHAFTS IN BEAUMONT CLAY

PART ONE - STATE OF THE ART

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 behavior of drilled shafts in clay. Maximum side shear stresses and base capacities 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 development of drilled shafts, describes construction procedures, 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 anchorage systems, describes the various instrumentation

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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.

This report is the manifestation of the efforts of many individuals.
The technical contributions of Dr. Walter R. Barker, Mr. Harold H.
Dalrymple, Mr. James N. Anagnos, Mr. Frederick E. Koch, and Mr. Olen L.
Hudson merit special recognition. Mr. James Holmes skillfully made the
drawings. Miss Mary Kern proficiently prepared the final copy. Thanks

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.

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

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 of 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 installation 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 predominated 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 investigation 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 tentative 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 I

INTRODUCTION

The past three decades have seen an unprecedented worldwide increase in the use of deep foundations. This growth in pile and pier construction has been fostered by society's demand for heavier structures, which are being increasingly located in areas having unfavorable near-surface soil characteristics.

During this period a new class of deep foundation, the drilled shaft, has evolved. Drilled shafts, also known by such terms as bored piles, drilled piers, drilled caissons, and cast-in-situ piles, have accounted for a significant part of the total number of deep foundation elements constructed recently. Their economic attributes often dictate their selection over driven piles, especially in stiff clay. In Chicago, for example, drilled shaft foundations can be built for at least 25 per cent less than pile foundations (Gnaedinger, 1964). An even greater cost reduction for bridge foundation construction in Houston, Texas, has been observed (Barker and Reese, 1970).

The newly gained status of the drilled shaft as an important foundation element has attracted the attention of many investigators concerned with its behavior under load. The objective of this study is to review the research of those investigators, to describe the state of the art concerning the behavior of drilled shafts in stiff clay, and to present the results of a field testing program undertaken on full-sized instrumented drilled shafts in the stiff Beaumont Clay formation of southeast Texas.

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Description of the Drilled Shaft

A drilled shaft is formed by boring an open cylindrical hole into the soil and subsequently filling the hole with concrete. Boring is usually accomplished with a portable drilling rig equipped with a large helical auger or a cylindrical drilling bucket with a cutting edge on the bottom face. Concrete in a drilled shaft is often reinforced to withstand tensile stresses produced by expansive soils or imposed flexural loading. Once in place, a drilled shaft acts essentially like a driven pile, except that its pattern of behavior under load may be different because of the dissimilar geometries and installation procedures.

The specific features of a drilled shaft which distinguish it from other forms of deep foundations are:

1. The drilled shaft is placed by boring a hole and removing the soil with a consequent minimization of soil disturbance. A displacement pile, on the other hand, has the effect of maximizing disturbance (a result often desired, particularly in loose, cohesionless soils and in some soft clays).

2. Wet concrete is cast and cures directly against the soil forming the walls of the borehole. Although a temporary casing may be needed to aid in keeping the borehole open, it is always extracted at the time the concrete is introduced.

Foundation elements that do not have both of the above features will be excluded by definition from consideration herein as drilled shafts. Franki piles and drilled-in-caissons (in which the casing remains

permanently in place) are examples of foundation elements which are similar to drilled shafts but which satisfy only one of the two criteria.

In its most common application, the drilled shaft is used to sustain large axial loads. However, more diverse functions are now emerging. For example, drilled shafts have been successfully employed in retaining walls and as anchors and tiebacks. They are also being used as extensions to large-diameter pipe piles supporting offshore structures to provide added penetration (McClelland, Focht, Emrich, 1969).

When feasible, the base of a drilled shaft to be loaded in compression is located on bedrock or on another otherwise sound stratum. If an adequate founding stratum is not reached at a reasonable depth, the base of the shaft is often enlarged to provide the required bearing capacity.

· The diameter of the stem, or cylindrical part of the drilled shaft, typically varies from 18 inches to 36 inches, although stems with diameters greater than 10 feet have been built. The enlarged base, called a bell or underream, is usually conical and at its base is two or three times the diameter of the stem. The sidewalls of the conical bell commonly make an angle of 30 to 45 degrees with the vertical.

A schematic drawing showing the essential components of a drilled shaft and its modes of resisting load is given in Fig. 1.1. The drilled shaft derives its bearing capacity from a combination of base and side resistance. Depending upon the soil profile and shaft geometry, either base or side resistance can be dominant, or both can contribute to the capacity in approximately equal proportions. Because of the possible alterations of soil properties along the sides of the borehole as a consequence of placing wet concrete against the soil and the potential

Axial Load

Diameter
18-36 inches
Typical

Depth Varies
With Soil
Conditions

Reinforcing
Steel

Side Resistance

Bell- May be used or
omitted as desired.
Size varies – commonly
three times shaft
diameter at base.
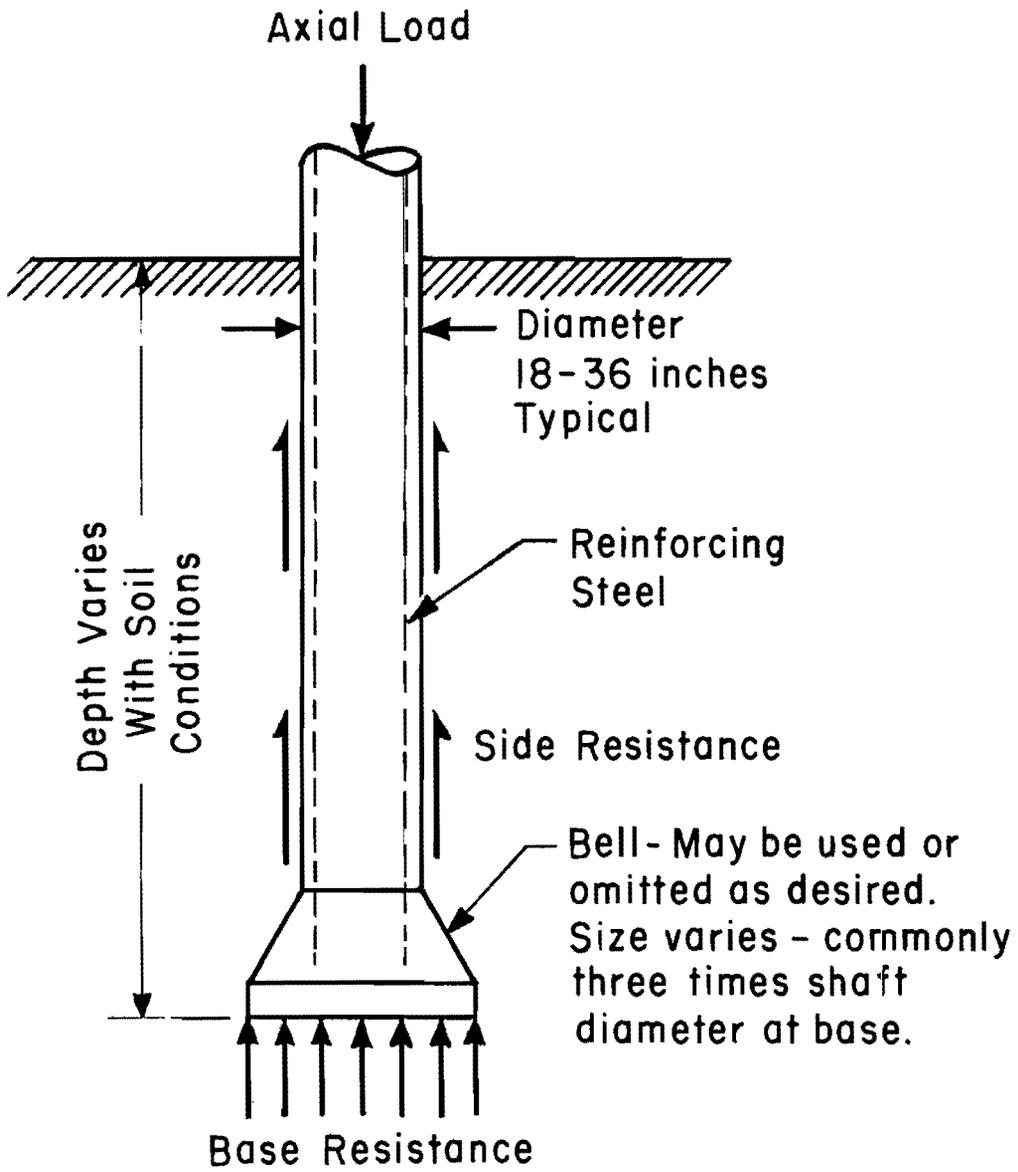
Base Resistance

Fig. 1.1. Typical Drilled Shaft

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removal of side support due to shrinkage of expansive clays, many designers
have been inclined to neglect side resistance when computing allowable
bearing values. The uncertainties surrounding these effects have persisted
because of scarce or inadequate published information concerning side capa-
cities of prototype shafts in many soil formations. The disallowance of
side resistance is of minor significance in the design of shafts carried
through weak soil to hardpan or bedrock, since the proportion of load
carried in side friction is small. However, drilled shafts formed
entirely within a homogeneous soil mass with no material of exceptional
rigidity below the base may actually carry a high percentage of load in
side resistance; thereby, making the exclusion of this mode of behavior
from consideration a source of overdesign with resultant loss of economy.
Because of the importance of side resistance, in surveying the work done
by others and in the field tests reported herein, emphasis has been
placed on the determination of actual side capacities of shafts, parti-
cularly those founded completely in stiff clay.

History of the Development of Drilled Shafts and Drilling Equipment

Present day drilled shafts, which are machine excavated, were predated
by hand-dug caissons such as the "Gow caisson," popular in the early part
of this century. Caissons strictly built by the Gow method fit the two
criteria for drilled shafts, although a related hand-digging technique,
known as the Chicago open well method, employed timber lagging that
remained in place inside the perimeter of the hole after the concrete
was poured. Gow caissons were formed by hand-excavating a series of
cylindrical holes, sometimes several feet in diameter. The holes were

made progressively smaller in diameter with depth and were usually cased
with telescoping metal tubes that were withdrawn during concrete place-
ment. Hand-dug caissons were used primarily in regions where they could
be carried to a hard bearing stratum. The subsurface hardpan in many
cities in the Great Lakes region provides good bearing at reasonable depth.
Consequently, many early high-rise structures in cities such as Chicago
and Detroit were supported on hand-dug caissons.

Gow caissons and other hand-dug shafts were tedious to construct,
however, and were generally competitive with driven piles only under condi-
tions where large axial loads had to be sustained and where the shafts
could be designed as end-bearing elements. Hand enlargement of bases
was occasionally permitted to increase the allowable load. In Chicago
the working load was computed using an allowable base bearing pressure
of 8,000 to 12,000 psf (Baker and Kahn, 1969).

Machine excavation for drilled shafts began to appear in the United
States in the 1920's. Greer (1969) has found records of horse-driven
rotary machines that were used to auger holes in San Antonio, Texas,
around 1920 for shafts 25 feet or more in depth. Osterberg (1969) des-
cribes an even earlier power-driven earth auger, built around 1908,
capable of making holes 12 inches in diameter and 20 to 30 feet deep.
Although it is not reported whether this auger rig was employed in the
construction of load-bearing drilled shafts, it seems probable that it
must have eventually been put to that use.

During the 1930's drilled shaft construction was limited primarily
to hand-dug caissons and a few machine-drilled elements. Shortly after

1930, steam shovels began to be modified for use as drilling rigs. These rigs normally employed buckets for soil excavation (Cummings, 1949). Drilled shafts began to find particular favor in underpinning operations about that time. But it was not until the Second World War that the full impact of machine drilling was felt. Truck-mounted post-hole augers, originally developed for utility companies, were adapted to the rapid construction of shallow pier foundations for many structures required by the armed forces (Greer, 1969). Efficient and economical techniques were soon devised for drilling and concreting operations.

Wartime foundation construction, and the resultant improvement and availability of high-speed portable drilling machines, spawned a new post-war industry composed of small drilling contractors in the United States and Great Britain. Energetic contractors became engaged in the construction of machine-excavated drilled shafts in areas geologically suited for this type of foundation. Many flourished and expanded their services. They operated principally in localities where cohesive soils permitted the excavation of free-standing holes, such as in parts of Texas, California, Michigan, and Illinois. Drilled shafts rapidly became popular particularly in the London, England, area, where small-diameter machine and hand-excavated shafts had been used for some time. Contractors on both sides of the Atlantic quickly created a demand for their services by demonstrating the economic advantages in many localities of drilled shafts over driven piles. Their ingenuity in developing portable drilling machines for making larger excavations, belling tools to form enlarged bases, and other appurtenances to speed construction

soon established a clear-cut economic advantage for drilled shafts where soils were suitable. This advantage was based mainly on the speed of construction and on lower material costs.

In the late 1940's and early 1950's, drilling contractors continued to expand their influence and to promote their product vigorously. Cutting devices and techniques to form sockets to allow boreholes to be advanced into rock were introduced. Sockets replaced bells in some instances. Large-diameter straight cylindrical shafts founded entirely in clay, and deriving a majority of support from side resistance, came into rather common usage in Britain. By introducing casing and drilling mud into boreholes, a procedure long established in the oil industry, many contractors found that boreholes could be cut through permeable soils below the water table and in caving soils. This procedure, known as "processing the hole," was most often employed in places where layered deposits of sand and stiff clay were encountered. Some contractors also found that boreholes could be terminated in sandy ground by injecting chemical grouts into the soil in advance of boring operations (Glossop and Greeves, 1946).

Rotary drilling rigs became standardized and began to be mass produced, giving further impetus to drilled shaft construction. The two basic types in use today, truck-mounted and crawler-crane-mounted rigs, came into prominence. Truck-mounted rigs are more mobile than crawler rigs. However, truck-mounted rigs are limited to drilling smaller boreholes, require good surface conditions for maneuvering, and experience difficulty handling casing. They were developed with a mast containing a square steel drill stem, called a kelly, which passes through a

turntable (or ring gear and yoke on bucket rigs) at the bottom of the mast.
The auger or bucket is attached to the kelly underneath the turntable or
yoke, through which torque is applied. The kelly is suspended from a
cable passing over a sheave in the crownblock at the top of the mast, and
is raised and lowered by a power wench on the bed of the truck. Many
truck-mounted rigs are fitted with "crowd" mechanisms which allow a verti-
cal force to be imparted to the kelly bar to facilitate drilling in hard
soils. The mast-turntable assembly can be lowered into a horizontal
position for transport and raised to a vertical position for drilling.
On more sophisticated auger drilling rigs, the mast-turntable assembly
can be run in and out along tracks which are mounted on a larger rotating
turntable on the bed of the truck. This arrangement, together with
leveling jacks on the sides of the truck, makes it possible to spot the
center of the auger over the point where the borehole is to be located
without maneuvering the truck. It also allows the operator to discharge
spoil from the auger by merely rotating the truck-bed turntable away
from the hole and spinning the auger rapidly to force the soil off the
blade. Truck-mounted bucket rigs have very little clearance between the
bottom of the ring gear assembly and the ground. Consequently, they
usually discharge spoil by disengaging the kelly, raising the bucket up
through the ring gear, swinging the kelly to one side with the aid of
a side boom, and dropping the spoil by opening the bottom of the bucket.
This process is more time consuming than discharging spoil from augers,
but bucket drilling is nonetheless preferred by many drillers in dry

and granular soils, in which cuttings tend to fall off an auger as it is being extracted. A typical, modern, truck-mounted rig is pictured in Fig. 1.2a.

Crawler-mounted rigs are more versatile and are capable of drilling larger boreholes than can truck-mounted rigs. However, they are more difficult to transport from site to site and, therefore, are less economical on small jobs. In principle, the operation of the crawler-mounted rig, pictured in Fig. 1.2b, is similar to that of the truck-mounted rig. A standard crawler crane is fitted with an assembly containing a diesel engine (or twin engines on very large rigs), transmission, and turntable to apply torque to the kelly. This assembly is supported from brackets near the heel of the mast and by cables from the top of the mast. The kelly is suspended and controlled in the same manner as for truck-mounted rigs. An additional line for handling casing and reinforcing steel is also employed. The kelly passes through the turntable, and the auger or drilling bucket is pinned to the bottom of the bar. Some rigs have telescoping kellys which permit drilling to depths in excess of 100 feet without breaking drill stem. Soil is discharged as described previously for truck-mounted rigs, except that rigs employing buckets do not require that the kelly be disengaged, since adequate clearance exists between the ring gear and the ground. The turntable assembly and kelly are removable, enabling the crawler crane to be freed for work other than drilling boreholes.

Auger or bucket rigs work well in clayey soils and sometimes in sands when proper drilling techniques are used. But when gravel or rocks
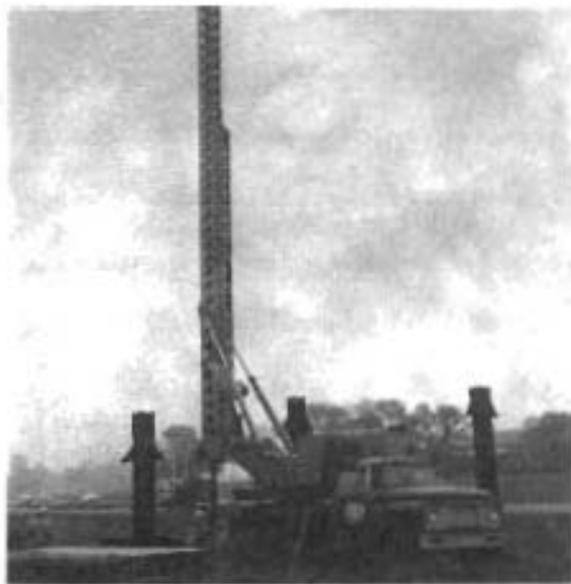
a. Truck-Mounted Rig

b. Crawler-Mounted Rig

Fig. 1.2. Drilling Rigs

are encountered, augers and buckets are inadequate for making a hole.
Contractors found that under favorable conditions a heavy casing or large
core barrel could be rotated under a crowd into the soil ahead of the
excavation, and the gravel or rocky soil inside removed with an auger or
clamshell. This operation, which would be clumsy with standard rigs,
began to be carried out with some success with special grab-type rigs
such as the Benoto, which sinks rotating sections of heavy casing ahead
of a drop-grab clamshell excavator. Grab-type operations are quite slow
in comparison with augering, and shafts are limited to a few standard
diameters.

As drilling rigs became available on a mass-produced basis, drilled
shaft construction spread throughout the world. In the 1950's, drilled
shafts quickly achieved paramount importance in Canada and several South
American countries.

Today, in cities such as Chicago, Edmonton, Winnipeg, and San Antonio,
most new deep foundations are drilled shafts, with driven piling or
shallow footings being used in rare instances where boreholes cannot be
economically made. The use of drilled shafts promises to increase still
further as drilling techniques are perfected and as research provides
more accurate criteria for their design.

Scope of Study

This study is concerned with the primary objective of describing the
behavior of axially loaded drilled shafts in stiff clay, since this is
the type of soil in which drilled shafts are most often specified.
Emphasis is placed on floating shafts (shafts deriving a significant

portion of their capacity from side resistance), in which difficulty may
be encountered in estimating design capacity and settlement. In discussing
design and construction, however, the current practices regarding other
types of soil are mentioned. This state-of-the-art presentation is
followed by a description of field tests that were conducted on instru-
mented floating drilled shafts in the Beaumont Clay foundation in Houston,
Texas. It is hoped that the results of these tests will contribute to
the still meager body of knowledge pertaining to drilled shaft behavior.

CHAPTER II

CONSTRUCTION PROCEDURES

Excavation Techniques

In order to understand the advantages, disadvantages, and behavior of drilled shafts, general knowledge of installation procedures is desirable; therefore, a description of typical methods is given in this chapter. Hole-drilling techniques are greatly influenced by the ingenuity of individual contractors. Consequently, there are many variations of the typical procedures presented herein.

Drilling is an art. The successful completion of a drilled shaft foundation, as well as the contractor's livelihood, depends upon the skill of the driller. While the steps outlined in this chapter for excavating a borehole are straightforward, the success of their execution is controlled by the ability of the driller to make timely decisions, such as how fast to drill, when to set casing, or whether to use mud.

At potential construction sites where exploration has shown the soil profile to be marginal for drilled shafts, full-sized test holes are frequently drilled to assess the practicability of this type of construction as compared with driving piles. Caving and waterbearing sands, rocky soils, predominantly stiff clay profiles containing layers of sand or rock, or soft clays are examples of soils that may or may not be suited to drilled shaft foundations and that may require the selection of other foundation alternatives.

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In having a test hole excavated, the foundation designer determines which drilling technique is best, if a bell can be cut at the desired elevation, how easily casing can be inserted and extracted (if needed), whether any loss of ground occurs as a result of squeezing in or sloughing of the sides of the borehole, if the hole is stable for a period of time sufficient to allow concrete placement, total length of time required for excavation, and whether potential problems involving groundwater intrusion exist. The test-hole excavation results allow more accurate cost estimates to be prepared in order to compare the relative economic merits of drilled shafts and driven piles or other foundation systems. If the designer decides to employ drilled shafts, test-hole results provide contractors with a basis for computing bid prices for the job.

The excavation techniques explained below are typical of those presently employed in Texas, although each job is unique and will often require departures from the procedures described. In Texas, augers are normally preferred over buckets for cutting holes.

Dry Method. Dry drilling, that is, drilling a freestanding hole without recourse to drilling mud, is of course the excavation method of choice. When boring and concreting operations can be conducted in the dry, without casing, construction is rapid. Subsoil conditions permitting dry drilling (for example, uniform, stiff clay) provide the clearest economic advantage for drilled shafts over driven piles. Drilling a dry hole is simple in principle. The rig is positioned and leveled. The kelly is then plumbed and the auger spotted over the hole. The hole is advanced by repeating successive cycles of dropping the auger to the

bottom of the hole, boring one or two feet by turning the auger, bringing up the cuttings, or spoil, on the auger, and discharging the spoil. Excavation proceeds as fast as the driller can get the auger into and out of the hole.

Each individual cut usually takes only a few seconds to perform. The spoil is discharged away from the hole to avoid interference with drilling activity. Excavation becomes slower as depth increases because of the increased time required in getting the auger to and from the bottom of the advancing hole, but a three-foot-diameter by thirty-foot-deep straight borehole in stiff clay can usually be excavated in less than a half-hour. With proper scheduling, drilling can be followed with concreting immediately, resulting in completion of a thirty-foot straight shaft in as little elapsed time as one hour. Reinforcing steel, if used, is placed in the hole just prior to casting.

The essential steps in the dry drilling process are shown in Fig. 2.1. Occasionally, a temporary casing may be placed into the hole to impede minor caving in situations which do not warrant the use of drilling mud. Casing techniques are treated in the section on wet drilling.

Tolerances for borehole alignment vary, but, typically, holes are bored plumb to less than one per cent from the vertical. The Texas Highway Department, for example, requires that a shaft be no more than 1.5 inches out of plumb for the first ten feet, with an additional tolerance of 0.05 inches per foot for depths exceeding ten feet (Texas Highway Department, 1962). Vertical alignment is checked occasionally by setting a level on the kelly with the cutting tool resting on the bottom
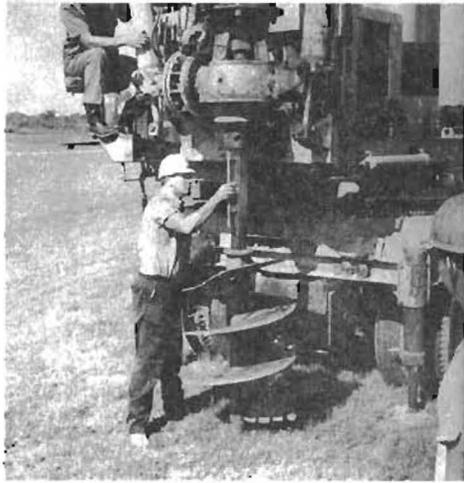
a. Spotting the Auger and
    Plumbing the Kelly
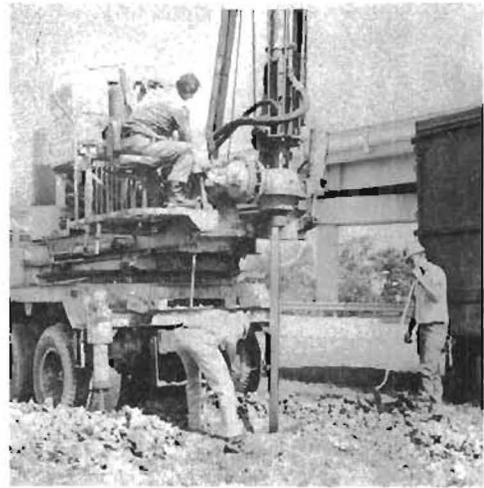
b. Boring the Hole

c. Extracting Spoil
    on the Auger

d. Discharging Spoil

Fig. 2.1. Steps in Dry Drilling

or by suspending a plumb bob periodically from the center of the top of the hole. Although drilled shafts are usually installed vertically, they may also be placed on a batter. A straight borehole is more difficult to achieve for battered shafts because of flexing in the kelly. Battered shafts, therefore, often have a characteristic bowed shape.

When an enlarged base is specified, a straight borehole is first excavated to the bottom elevation of the bell. The auger is then removed, and a mechanical belling tool, such as the one shown in Fig. 2.2, is attached to the bottom of the kelly. The belling tool is a cylindrical bucket, slightly smaller in diameter than the hole, with two cutting blades which fold up inside the bucket when the tool is picked up to be lowered into the hole. When the belling tool reaches the bottom of the borehole, the downward force of the weight of the kelly is allowed to bear on the joint that pins the top of the two cutting blades and kelly together. Pivot arms are pinned to each cutting blade and to the body of the bucket below the blade pins. The action of the vertical force on the kelly joint causes the blades to rotate outward through side openings in the bucket and bear against the soil. The belling tool, then in cutting position, is turned slowly through one or two rotations, thus cutting into the sides of the borehole and forcing spoil into the bottom of the bucket. When the capacity of the bucket is reached, the driller stops turning and pulls up on the kelly to cause the cutting blades to retract. The belling tool is then brought to the surface, and the spoil is discharged through a trap door in the bottom of the bucket. The belling tool must be inserted and extracted many times to form a good bell. Each time the belling tool is inserted, the cutting blades

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a. Belling Tool in
    Open Position

b. Belling Tool in Closed Position
    for Lowering Into Borehole

Fig. 2.2. Belling Tool

cut deeper into the sides of the hole until the bell is finally completed. After the bell is cut, good practice dictates augering a deeper seat for the bucket and using the belling tool to excavate uniformly a few inches of soil from the bottom of the bell to provide a cylindrical bearing pad. In many soils, slow cutting at all times is essential to keep the sides of the bell from caving. In fact, the entire process of belling is often quite slow in comparison with augering of the stem. It usually takes two hours to form a bell, three times the diameter of the stem, in a borehole that required thirty minutes to drill.

The finished bell cut with the tool shown in Fig. 2.2, is conical in shape. Other designs have different pivoting mechanisms that cause the cutting blades to rotate about a pin in the bottom of the bucket, thereby forming a hemispherical bell.

If a bell collapses where good cohesive soil is present below the bell, the driller must auger farther down, through the bottom of the aborted bell, and try to underream again at a lower level. Occasionally, this procedure has to be repeated several times, with the uneconomical result that the finished bell lies a large distance below the intended elevation. In rare instances, a bell cannot be formed at all. If side resistance is disallowed in design, such a shaft will probably have inadequate allowable capacity because of the unbelled base, unless the straight shaft is terminated in unusually strong material. This problem can require redesign of local segments of the foundation system. For example, it may be necessary to revert to driven piles or to use two or more cylindrical drilled shafts to carry the load originally intended for one belled shaft.

Underreaming on the surface of bedrock often requires the additional steps of hand preparation of the bearing surface at the base of the shaft and dewatering, since the disconformity between bedrock and overlying soil is frequently a groundwater channel.

Machine-formed bells in clay are "cleaned up" by sweeping up crumbs of soil from the bottom of the underream by careful use of the belling tool or by hand, as specified by the designer. Hand cleaning is preferable, but time consuming, because it necessitates placing a temporary casing and often operating a fresh air supply for the protection of workers.

The technique of forming bells at two or more elevations in a shaft has received attention lately (Mohan, Murthy, and Jain, 1969). Evidence obtained from model tests in clay indicates that such shafts fail by shearing along the periphery of the bells as well as by end bearing. Although such shafts are in limited service, the future possibility exists that even greater monetary savings can be effected by employing multiply underreamed shafts.

Wet Method. Wet excavation is used in soils that do not permit a freestanding borehole to be drilled or where groundwater will leak into the hole at an excessive rate.

The essential steps of wet drilling or "processing" are illustrated in Fig. 2.3. A typical soil profile requiring this operation is shown. The hole is first excavated to the top of the caving or waterbearing soil in the dry (Fig. 2.3a). Bentonite and spoil are dumped into the hole and mixed with water by the auger to fill the hole. The bentonite-soil slurry or "mud" is mixed in varying proportions of each ingredient
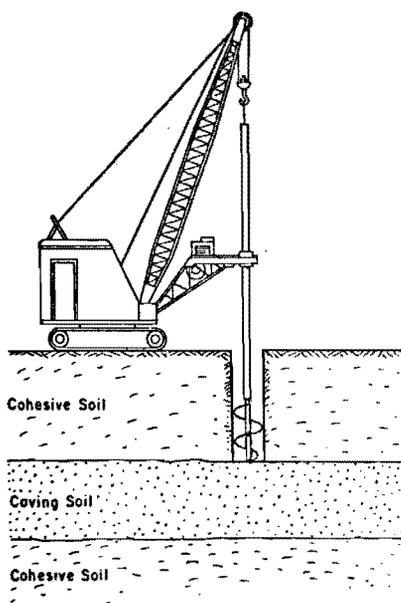
a. Dry Drilling Through
    Cohesive Soil
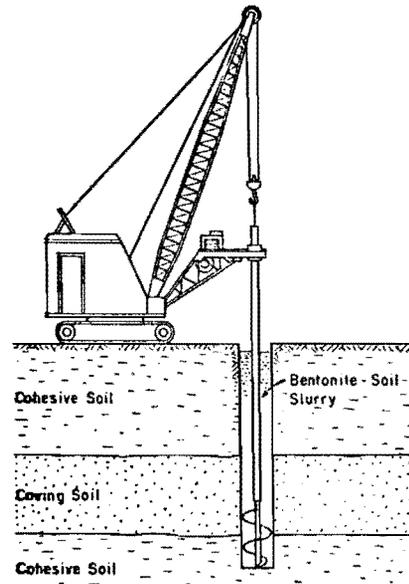
b. Drilling Through Caving
    Soil Under Mud
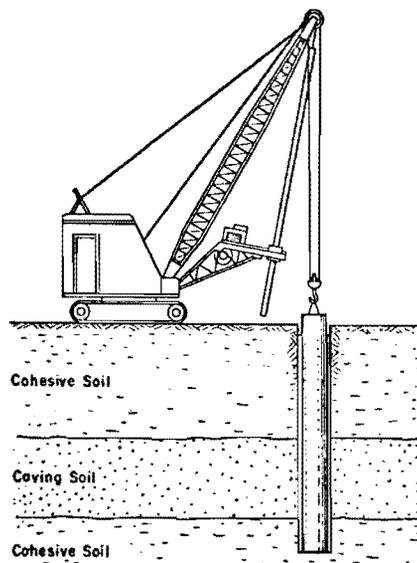
c. Setting Casing

Fig. 2.3. Steps in Wet Drilling

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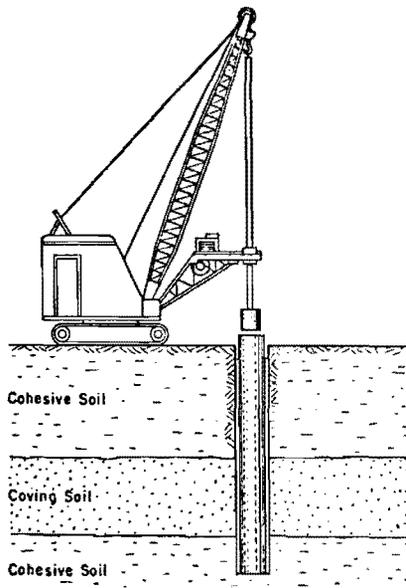
d. Bailing Mud from
   Inside Casing
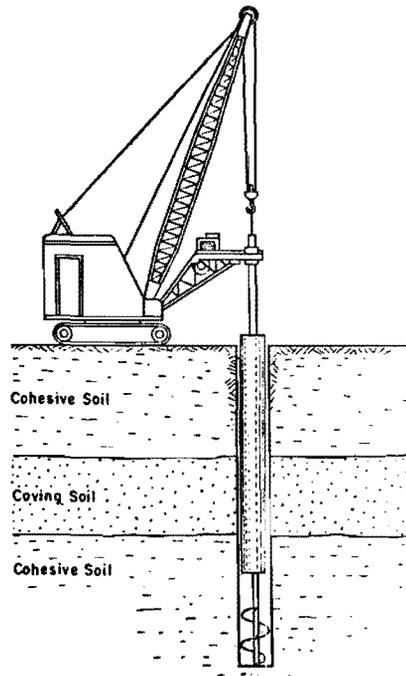
e. Dry Augering Below
   Cased Section
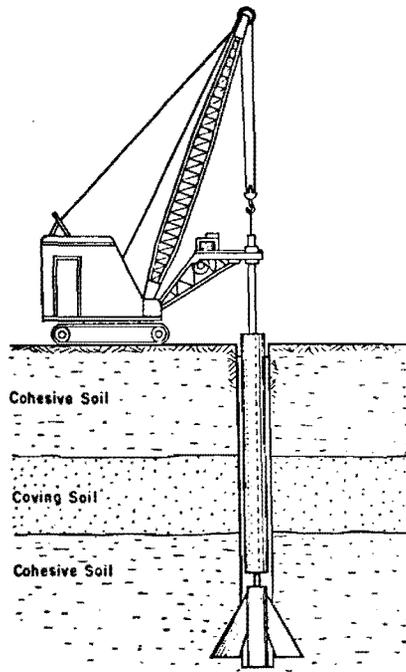
f. Belling

Fig. 2.3. Continued

Level of
Fluid Concrete

Drilling Fluid
Forced from
Space Between
Casing and Soil
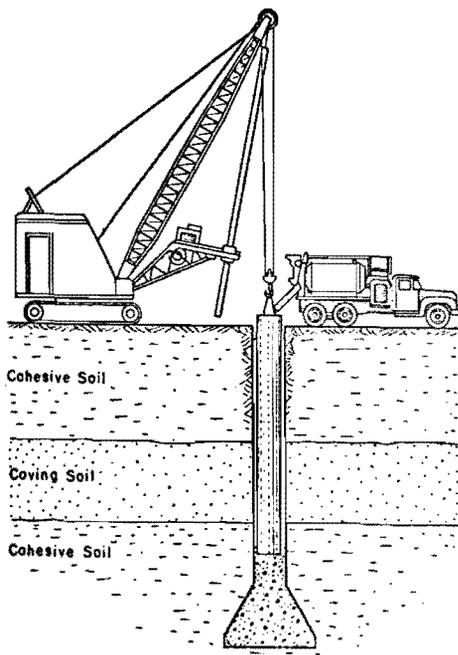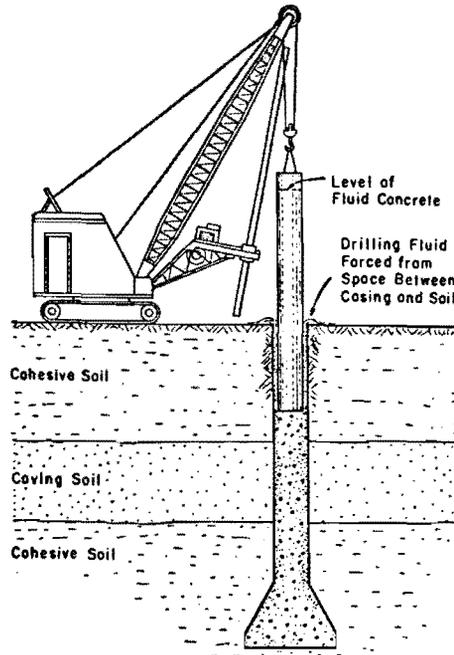
g. Placing Concrete   h. Extracting Casing
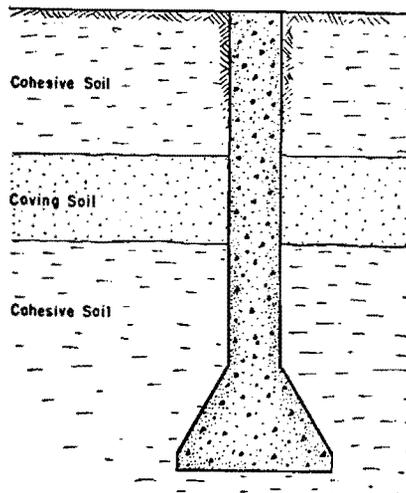
i. Completed Shaft

Fig. 2.3. Continued

according to individual site conditions and experience. Mud is a dense, viscous fluid in which the bentonite causes the soil to go into suspension. Barite is occasionally used in place of bentonite when heavy mud is desired.

The driller then continues augering through the mud, which stabilizes the borehole, as drilling advances through the caving soil (Fig. 2.3b). The lateral pressure of the mud against the sides of the borehole counter-acts caving and retards inward migration of groundwater. As the hole is deepened, more water and bentonite are added to keep a constant level and consistency of the mud. The mud can be circulated through a slush pit to remove cuttings, but enough soil is usually brought up on the auger to make this procedure unnecessary.

After the caving stratum has been fully penetrated, and impermeable, cohesive soil is again encountered, a temporary steel casing, with a diam-eter slightly greater than that of the auger, is inserted (Fig. 2.3c). The casing is normally in a single piece. If the middle stratum is waterbearing, the casing must be screwed into place, using the kelly and a special yoke, to form a seal in the cohesive soil below to prevent intrusion of groundwater.

Once the casing is sealed in place, the mud inside the casing is bailed out using a bailing bucket attached to the end of the kelly (Fig. 2.3d). The remainder of the hole is then augered in the dry to permit concrete to be placed against dry soil at the base (Fig. 2.3e). If speci-fied, a bell is formed as in the dry method (Fig. 2.3f). Care must be taken to insure that the bell is cut well within the cohesive stratum to circumvent a bell cave-in.

The reinforcement is then placed and the concrete is poured either directly into the hole, through a tremie, or through a downchute, depending

upon local specifications (Fig. 2.3g). The steel casing is expensive and is, therefore, routinely removed and reused. As soon as a sufficient head of fluid concrete is achieved inside the casing, the casing is raised slightly, breaking the outside seal against the lower stratum.

When the concrete level reaches a point near the ground surface, the casing is raised 10 to 15 feet. As this action occurs, the fluid concrete fills any spaces that existed between the casing and the side of the borehole from below, ideally forcing the mud that had occupied those spaces toward the surface. The partially extracted casing is then filled nearly to the top with wet concrete and then completely removed (Fig. 2.3h). The excess hydraulic head provided by the wet concrete inside the casing above ground ideally expels all of the mud around the casing near the top of the hole, insuring good contact between the borehole walls and concrete. Mud may not be completely expelled if the concrete slump is too low, if the casing is extracted too quickly, if the hydraulic head is too small, or perhaps for other reasons. The completed shaft is shown in Fig. 2.3i.

The stem of the finished drilled shaft tends to be tapered slightly inward, toward the bottom, as a result of screwing the casing to form the seal. It may also be "collared" due to erosion of the sides at the depth where the surface of the mud was located. The interface between the concrete and the natural soil will invariably contain at least a thin film of mud, which introduces a further uncertainty concerning the side capacity of drilled shafts installed by the wet method.

Wet drilling, although slow, can still be less expensive than driving piles, but the driller must be especially careful when augering through

caving soils with mud in the hole. The cutting operation and extraction

of the auger must be done slowly to avoid "sucking in" the sides of the

hole.

Occasionally, cohesive soil is not encountered in sufficient thickness

to terminate the borehole in the manner described. Such an occurrence

is usually unexpected, since drilled shafts are not normally specified in

such soil. However, under these circumstances, a cylindrical shaft may

be completed in waterbearing soil by augering all the way to the bottom

with mud in the hole. Then, by placing the concrete through a tremie,

all of the mud is displaced by the concrete. Using this procedure there

is a question as to whether some mud may have been trapped under the con-

crete at the base of the shaft at the beginning of the pour; however,

better displacement of mud along the sides may occur than when casing is

used.

As an alternative in some situations, the hole can be cased and the

soil beneath the base stabilized chemically. After setup occurs, the

mud is bailed out and the concrete placed in the dry.

Belling is rarely attempted through mud. If a borehole for an under-

reamed shaft cannot be terminated in soil for dry belling, the foundation

design is usually revised. Further consideration of the wet process and

the behavior of shafts constructed by this process is given by Barker

and Reese (1970).

Reinforcement

The sizing of axially loaded drilled shafts entirely in clay is

usually based solely on providing enough base and side area to develop

the required bearing capacity and to control settlement at working load.
When these requirements are met, compressive working stresses in the
concrete will usually be below the maximum allowable, and reinforcement
will be required only to protect against the possible development of
tension as a result of concrete shrinkage, flexural loading, or swelling
soils. Nominal vertical reinforcement in the form of intermediate-grade
deformed bars, composing about one per cent of the cross-sectional area,
is routinely used in nonexpansive soils when shafts carry little or no
bending moment. Many agencies require less reinforcement, and some require
no reinforcement at all. The vertical rebars are usually tied together
in a circular pattern with spiral hooping or horizontal ties to form a
cylindrical cage, which is ordinarily equal in length to the depth of the
borehole, as well as several inches smaller in diameter. Tie bars to be
used in tying into the superstructure are attached to the top of the cage.
The reinforcing cage is usually assembled on the construction site and
placed in the borehole as a single unit just prior to concreting. The
cage is centered in the hole with side blocks tied to the cage.

If the drilled shaft is to be founded on a hard stratum, compressive
stresses in the concrete become a matter of consideration, with the
working load being potentially limited by the allowable concrete stress
and not by the allowable bearing pressure on the base. Allowable con-
crete stresses are usually in the order of one-fourth of the compressive
strength of the concrete. Such a low value is dictated by the fact that
undetected discontinuities can occur in the concrete, particularly during
operations requiring use of temporary casing. The undesirable condition

of allowing lower stresses in the concrete than those permitted against

the bearing stratum occurs most often when shafts are belled on top of

or socketed securely into bedrock. To rectify the problem, the stem

may be enlarged, or alternatively, additional vertical reinforcement

may be provided. Added reinforcement is furnished by placing more rebars

in the section, embedding structural steel members in the core of the

section, or leaving the casing permanently in place (in which case the

element ceases to be a drilled shaft). The last procedure has become

generally accepted for extremely heavily loaded caissons supporting major

structures. Some building codes allow much higher concrete stresses when

permanent casing is used due to the smaller probability of occurrence of

concrete discontinuities.

Concrete

To this point, emphasis has been placed on the importance of the

drilling operation. Of no less concern is the careful control and place-

ment of the concrete.

Concrete for drilled shafts should be of good quality, with a mini-

mum compressive strength of about 3000 psi. Highly stressed end-bearing

shafts require stronger concrete. The maximum aggregate size should be

limited to 1.5 inches, especially in operations involving extraction of

casing, where larger aggregates can hang up between the casing and rein-

forcing cage and make proper casing extraction difficult. A concrete

slump of at least six inches is desirable, especially where casing is to

be removed during placement. Many agencies, however, specify slump

values in the order of four inches. Retarding admixtures should be used

as a matter of course in warm weather if a temporary casing is involved. Concrete with retarded set, high slump, and small aggregate size will tend to consolidate without honeycombing, will allow the casing to slip out freely, and will flow more easily into the annular space between the casing and borehole wall as the casing is pulled.

Concrete placement should follow normal good practice. Concreting of drilled shafts is a continuous operation, except that the bell and stem are sometimes concreted separately. Placement of concrete should be accomplished as soon as possible after the concrete is mixed, with intermediate agitation provided. Concrete is often placed into the shaft through a downchute or tremie supported from the drilling rig. The tremie is raised as the concrete rises in the shaft in order to keep it from becoming too deeply embedded. Ports are cut at various levels in the side of the tremie to permit convenient introduction of concrete as the tremie is raised. The concrete is rarely vibrated.

Casing extraction should never be delayed. The total elapsed time from the beginning of the placement of concrete inside the casing until removal is started should not exceed one hour if the set is retarded, or one-half hour if it is not retarded (Texas Highway Department, 1962).

Test cylinders should be made routinely to provide a check on the quality of concrete being used.

Typical Drilled Shaft Construction Problems

Many difficulties can be encountered during the construction of drilled shafts, which if not properly controlled, can endanger the structural integrity of the finished element. Problems associated with drilled

shaft construction have been reported by a number of investigators (Carson, 1965; Peck, 1965; Palmer and Holland, 1966; Pandey, 1967; White, 1967; Osterberg, 1968, Baker and Kahn, 1969; and Greer, 1969). A few typical problems are considered briefly in the following paragraphs.

Extraneous Water in the Borehole. Despite precautions taken against groundwater intrusion, water may enter the bottom of the excavation from beneath the base or from around a poorly sealed casing. If the quantity of water is small (one or two inches in the bottom of the hole), the usual practice is to concrete the shaft as if it were dry. When significant water is present, placing concrete with a dry-hole technique may result in very low strength concrete in the bottom several feet because of a significant increase in the water-cement ratio.

To circumvent the problem, concrete is placed without delay after the boring is completed. If quick concrete placement is not feasible, the hole should be pumped free of water immediately before casting.

Occasionally, concrete must be placed under water. This operation should be done with a tremie, with concrete being discharged beneath the surface of concrete already in the hole (Baker and Kahn, 1969).

Rising Steel. When the first pull is made on casing, the top of the reinforcing steel should be carefully observed to determine if steel is coming up with the casing. In many cases the reinforcing steel will be directly visible to the inspector when extraction begins. If it is not, the inspector should endeavor to provide a means of remote sensing, such as a mirror, to detect steel movement. The inspector should insist on a slow pull, and he should require that pulling cease if any upward motion of the steel is noted. Rising steel may be an indication of concrete

rising with the casing (and that the shaft is tending to separate somewhere in the stem), or that the reinforcing cage is binding on the casing. In any event, when a steel rise occurs, the casing should be left permanently in place to avoid major damage to the shaft.

An associated problem is the unravelling of the spiral hooping on the cage caused by unscrewing the casing to break the seal with the soil prior to pulling. The inside of the casing impinges on the spiral reinforcement with a force sufficient to break the ties with the vertical reinforcing and causes the spiral to coil up. The problem is often experienced in drilled shafts installed on a batter, in which difficulty in keeping the cage out of contact with the casing can occur. The entire reinforcing cage often collapses, and considerable effort is necessary to fish out and reset the vertical rebars and dowels before concreting can be completed. Again, the casing is usually left permanently in place after such an occurrence.

Necking. When casing is extracted with insufficient head of fluid concrete inside the casing, caving soils can squeeze in on the concrete within the stem, forming a neck, or section of reduced diameter, as shown in Fig. 2.4. This defect normally goes unnoticed, although clues to its existence are occasionally offered by concrete rising in the casing during extraction or by formation of depressions around the casing at the surface. Therefore, prevention through the proper installation is quite important.

Separation. When an excessive amount of time elapses between concrete placement and casing removal, bond can develop between the casing and the concrete, which will cause concrete in the upper part of the stem to rise and completely separate from the concrete below. This action results in a discontinuity in the concrete bridged only by the reinforcing steel.

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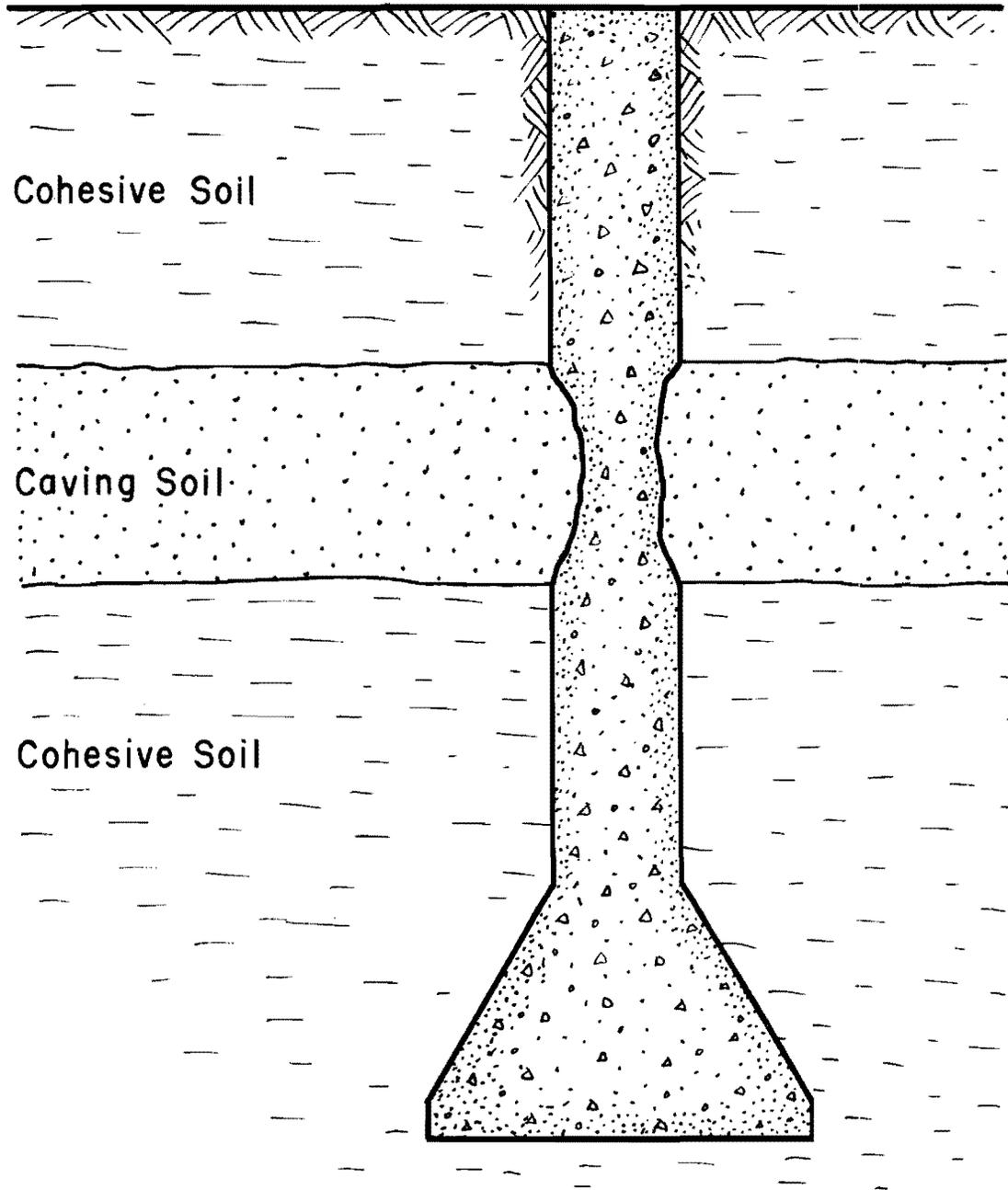
Fig. 2.4.   Shaft with Necked Section

A separated shaft is illustrated in Fig. 2.5. The void usually fills
with water or loose soil.

Miscellaneous Problems. Numerous other difficulties can arise in
drilled shaft construction. Sometimes, plastic clay will set up around
a long casing, and the contractor will be unable to remove it. This, of
course, does not affect the structural integrity of the shaft, unless the
stem is separated during attempts to dislodge the casing. However, loss
of casing represents a financial setback to the contractor. If casing
remains permanently in the hole, it also may cause the side resistance to
be appreciably different than if it were removed and concrete allowed
to cure against the soil.

An effect opposite to necking sometimes occurs, whereby concrete dis-
places weak soils in the walls of the borehole under a high hydraulic
head. This action results in the formation of a collar around all or
part of the perimeter of the borehole at a particular level. Collars do
not impair the effectiveness of the shaft. In fact, they add to the
bearing capacity, but they are a matter of concern to contractors because
considerable concrete can be wasted.

Sloughing of soil from an uncased borehole during casting operations
can contaminate concrete. This problem can usually be avoided by
inspecting the sides of the borehole to evaluate their stability prior to
concreting. The soundness of the concrete in the shaft also is affected
by the design of the reinforcing cage. Adequate spacing should be allowed
between rebars or ties, and between the perimeter of the cage and the
sides of the borehole or inside of the casing. In the latter instance, a
3-inch clearance is desirable for 1 1/2-inch maximum-sized coarse

Cohesive Soil

Caving Soil

Cohesive Soil
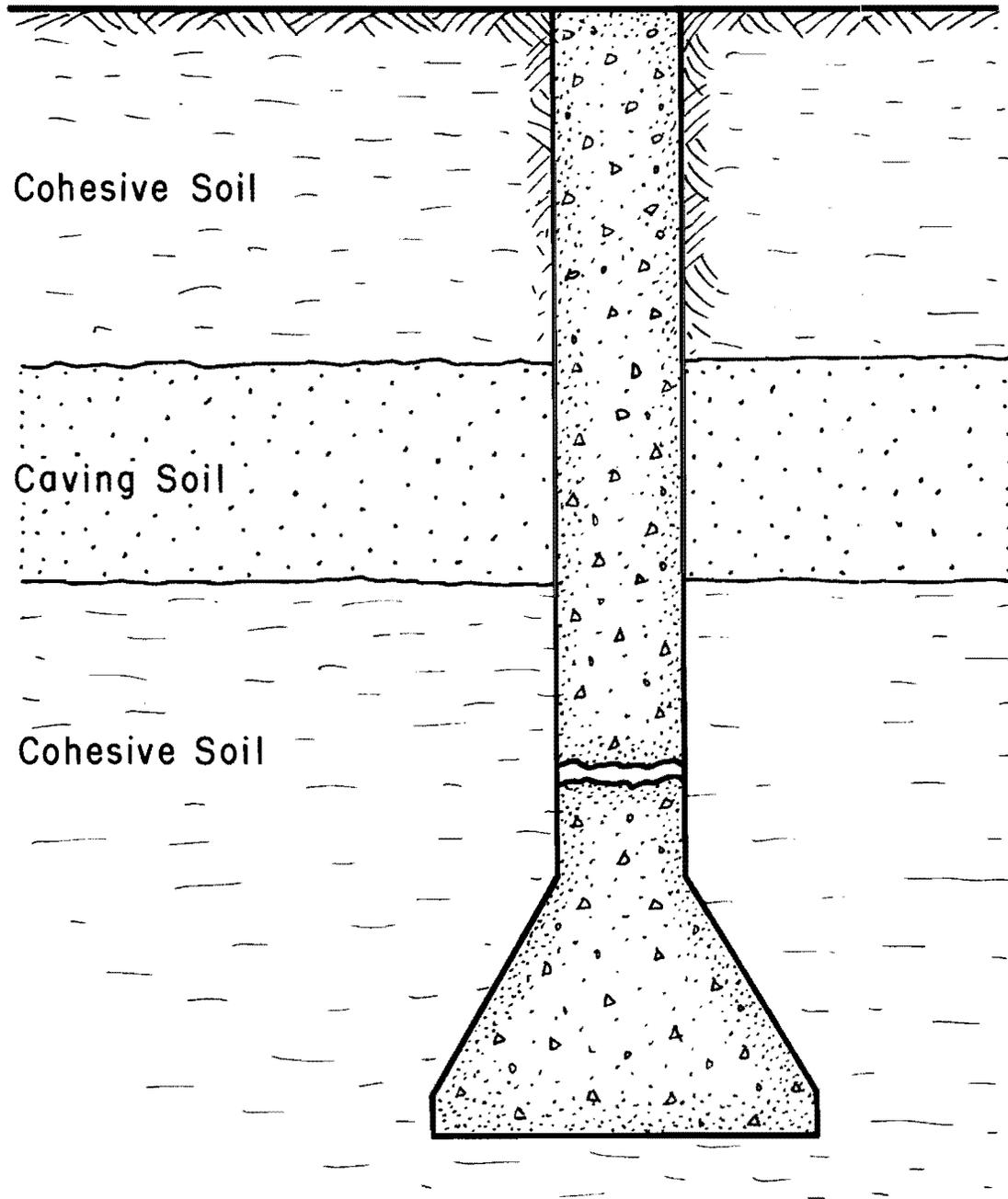
Fig. 2.5. Separated Shaft

aggregate. Failure to provide enough space for the concrete to flow freely around reinforcing steel can result in honeycombing between the cage and borehole wall. Side-resistance characteristics can thereby be potentially altered.

Although drilled shafts are rarely installed entirely in soft clay deposits, it is common practice to penetrate surface strata composed of soft clay to reach good bearing material below. Loss of ground readily occurs in uncased boreholes in soft clay as the soil squeezes inward upon release of confining pressure. This phenomenon not only causes the borehole to become smaller than desired, but it also poses a hazard to nearby structures. Immediate insertion of casing and rapid construction are used to minimize loss of ground.

An additional problem, associated with large diameter shafts, is the lateral buckling of casing. Large diameter, thin-walled casing, if not properly reinforced, can buckle under groundwater pressure (Osterberg, 1968).

Another important category of construction problems is associated with failure by the contractor to construct the shaft according to plans. Improper construction can be either intentional or unintentional. Proper inspection, however, will eliminate such major discrepancies as omitting a bell or terminating the borehole too high.

Correction of Deficiencies Caused by Poor Construction

Most agencies emphasize close inspection of drilled shaft construction to insure that techniques and practices employed are sufficient to produce sound shafts. For many foundations, drilled shafts are assumed to be

satisfactory if the inspector does not observe problems during construction, such as those explained in the previous section. However, large shafts supporting major structures are often cored or otherwise carefully checked for deficiencies by such procedures as seismic wave and velocity measurement if there is any question regarding their soundness (Baker and Kahn, 1969). Coring and sonic inspection are not economically warranted for minor structures except in rare instances. Consequently, the effects of suspected flaws must be evaluated, and a judgement must be made concerning the need for corrective action.

Unsound shafts can be repaired by coring the length of the shaft with several holes and inserting extra reinforcing steel, which is then grouted to the existing concrete. If voids are found during exploratory coring, they can be filled through the coreholes with pumped grout. Alternatively, new boreholes can be drilled alongside the defective shaft and unsound concrete cut away and replaced (Baker and Kahn, 1969). However, in many cases the practice is to abandon the unsound or suspect shaft and construct a new shaft on either side. The new shafts are spanned by a heavy transfer girder that takes the load originally intended for the defective shaft.

Effect of Construction Method on Behavior Under Load

The preceding sections illustrate a variety of construction procedures and problems. It is obvious that the behavior of an axially loaded drilled shaft will be quite dependent on the techniques used to install the shaft and problems encountered during placement, as well as on shaft geometry and natural soil conditions at the site. Construction procedures

to be used and possible on-site variations thereof should be of primary

concern to the designer of drilled shaft foundations.

Comparison of Drilled Shafts and Driven Piles

The merits and shortcomings of drilled shafts in comparison to driven

piles are related mainly to the construction practices previously described.

The most significant advantages and disadvantages which have been mentioned

or implied in conjunction with installation techniques are outlined con-

cisely below.

Advantages:

1. One drilled shaft can be used in place of a pile group

    because the capacity of a single shaft may be equiva-

    lent to that of several driven piles.

2. The overall foundation construction time is shorter.

3. There is a minimum of soil displacement and surface

    heave.

4. The drilling operation permits direct observation of the

    soil in which the shaft is being constructed. The physical

    properties of the bearing stratum and sidewall soil can be

    evaluated visually on the site and compared with those

    estimated for the design of the shaft. On-the-spot cor-

    rective measures possibly can then be taken, if necessary.

5. Ground vibration is kept to a minimum.

6. The noise caused by the pile hammer is eliminated.

Disadvantages:

1. Drilled shafts are difficult to install in soft clays. Loss of ground is also likely in such soils.

2. Difficulty is encountered in terminating a drilled shaft in waterbearing granular soil, and belling therein is impractical. Large bells formed in fissured clay below the water table tend to collapse easily.

3. Design of a drilled shaft foundation requires a more complete knowledge of the soil properties at the site.

4. Because of the many potentially serious problems that can appear during construction, more careful inspection is required.

5. Design specifications for drilled shafts are overconservative, especially in regard to allowable side friction and base bearing stress values. This fact is due in large part to the lack of information generally available concerning the behavior of drilled shafts under load.

CHAPTER III

MECHANICS OF DRILLED SHAFT BEHAVIOR

Detailed descriptions of the mechanical behavior of axially loaded drilled shafts have been presented elsewhere (Reese and Hudson, 1968; Barker and Reese, 1969). However, it is appropriate to describe briefly some of the salient points at this time, in order to provide a clear basis for understanding the results of the research reported herein and to define the terminology used to explain those results.

Removal of Applied Load By Soil Surrounding Stem

When a drilled shaft is acted upon by an applied load, Q_T , it is displaced downward, causing distortions in the soil adjacent to the side-walls (Fig. 3.1). These distortions produce shearing stresses that resist the movement of the shaft and cause reduction of load and compressive strain in the shaft with depth. Because of shaft compression, the absolute downward displacement of a point on the shaft becomes smaller with depth; hence, a lesser shearing distortion exists along the side-walls at lower levels. Assuming that slippage has not occurred in the soil or at the shaft-soil interface, the smaller shearing distortions at greater depths produce smaller shear stresses. The effect of shaft compressive flexibility is more significant as the supporting soil becomes stiffer. The portion of the applied load which has not been removed from the shaft by side shear is resisted by the base.
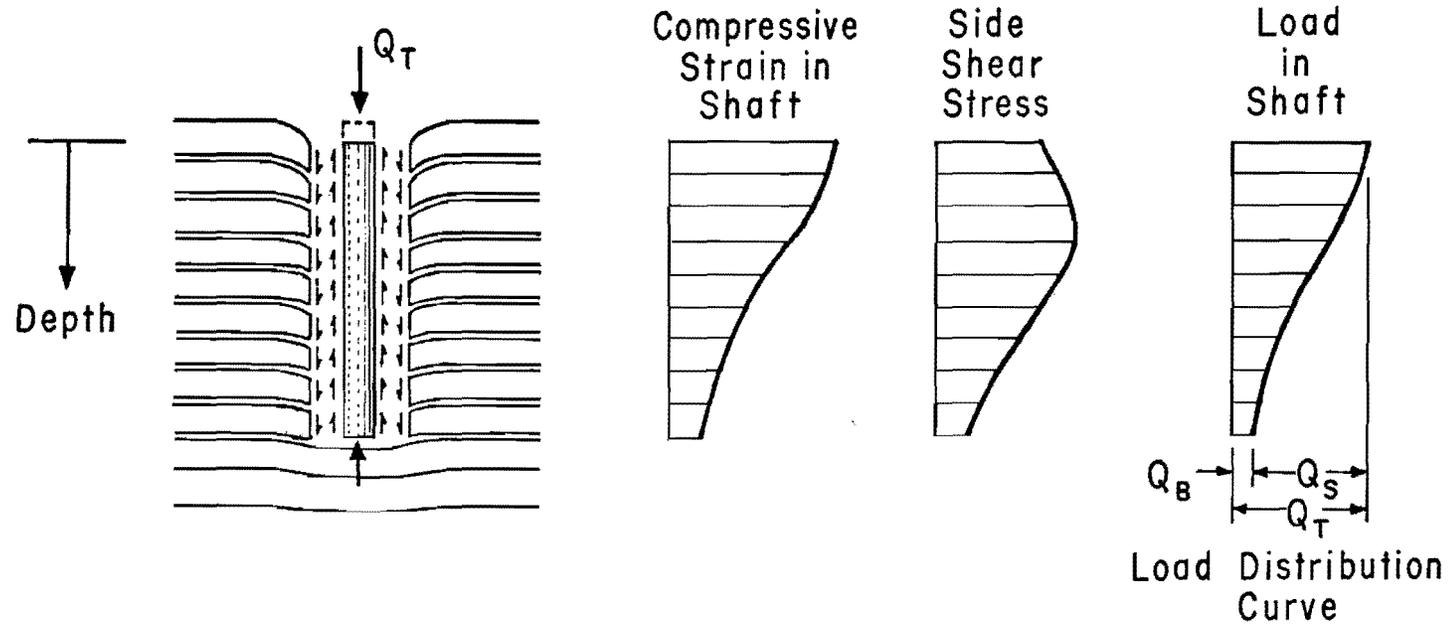
Fig. 3.1. Action of Drilled Shaft-Soil System Under Axial Load

Typical variations of side shear, compressive strain, and load in

the shaft are plotted as functions of depth in Fig. 3.1. The shape of

the shear stress diagram suggests that shear failure of the soil sur-

rounding the upper portion of the shaft may have occurred under load

Q_T or that the soil in that region may have low shear strength. If the

side shear stress is integrated over the peripheral area of the shaft

from the ground surface to a given depth and the result subtracted from

the applied load, the load remaining in the shaft at that depth is

obtained. A plot of load remaining in the shaft as a function of depth

is denoted a "load distribution curve." The total amount of load removed

by the sides in shear is denoted Q_S and the amount taken by the base,

Q_B. If the shear stress is constant along the sides, the load distri-

bution curve will be linear. Normally, however, the shear stress will

vary, giving the characteristic shape to the load distribution curve

shown in Fig. 3.1.

When values of applied load are plotted against the corresponding

settlements which occur at the top, or butt, of the shaft, a load-

settlement curve is obtained (Fig. 3.2). The load-settlement curve is an

important relationship, since it describes the response of the shaft

to loads that are imposed from the superstructure. Load distribution

curves can be plotted for different values of applied load, for example

$(Q_T)_1$, $(Q_T)_2$, and $(Q_T)_3$ in Fig. 3.2. Load $(Q_T)_1$ represents a

load for which the resisting shear forces are less than maximum; $(Q_T)_2$

is a load near the maximum; and $(Q_T)_3$ is a load beyond the maximum,

for which all soil along the sides has been completely sheared. It is

noteworthy that the increment of applied load $\left[(Q_T)_3 - (Q_T)_2\right]$ is
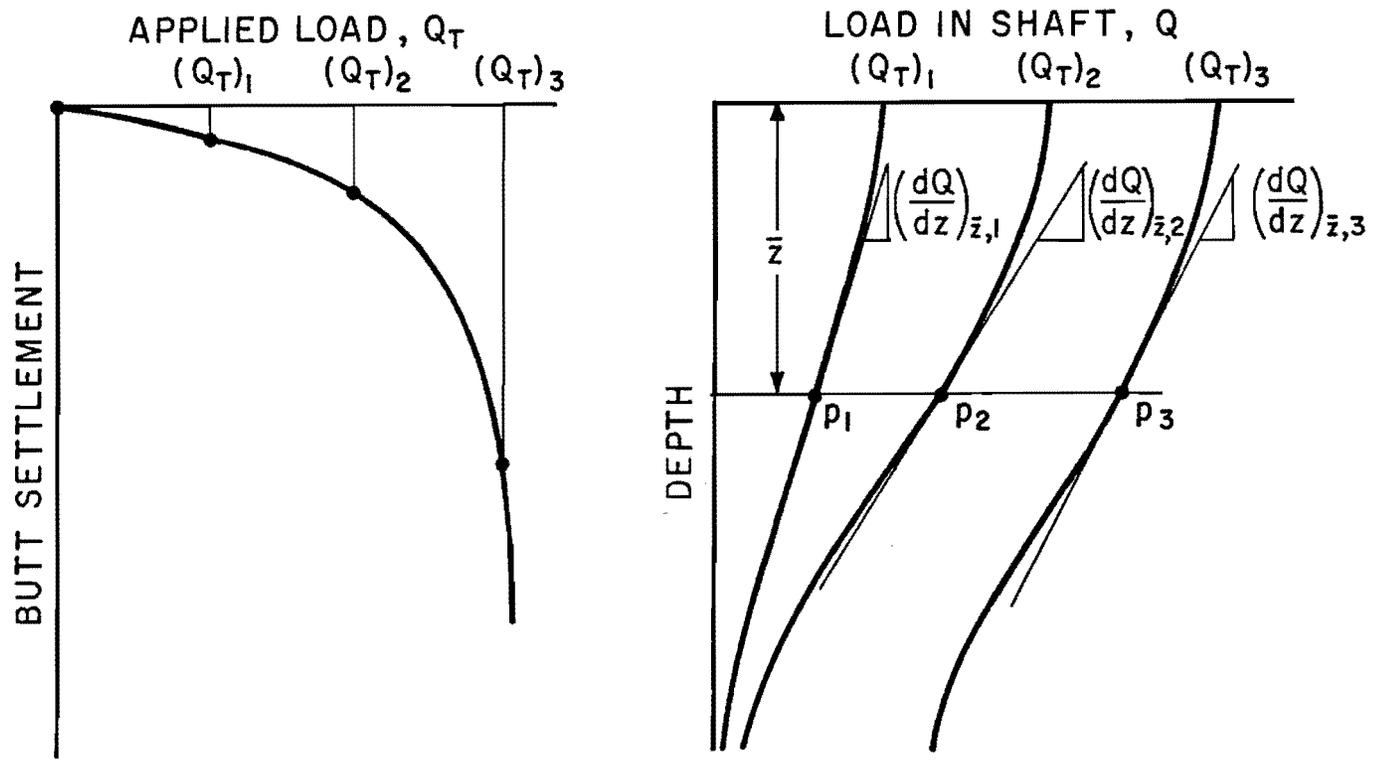
Fig. 3.2. Load-Settlement and Load Distribution Relationships

transmitted in its entirety to the base, since no more side resistance was available from the soil to resist that increment. In fact, with the added displacement imparted by the increment $\left[(Q_T)_3 - (Q_T)_2\right]$ the soil along the sides may be remolded or otherwise lose strength, with the result that less load is carried by side shear when $(Q_T)_3$ is applied than when $(Q_T)_2$ is applied. This phenomenon is evidenced by the fact that the separation between the load distribution curves corresponding to applied loads of $(Q_T)_2$ and $(Q_T)_3$ is greatest at the bottom. In other words, for an increment of applied load of a prescribed amount, the base load can increase by more than that amount. This phenomenon is known as load shedding. It is promoted by several factors, including relaxation of soil under long-term loading.

The slope of the load distribution curve corresponding to a given applied load at a generic depth $\bar{z}$ is equal to the product of the shear stress and the circumference of the shaft. By taking derivatives of the load distribution curves for several values of applied load at any level, values of shear stress corresponding to different magnitudes of downward movement can be obtained. The magnitude of the downward movement, w , at depth $\bar{z}$ under a prescribed applied load is given by

$$w_{\bar{z}} = w_T - \int_{z=0}^{z=\bar{z}} \frac{Q(z)}{A_c E_c} \, dz \quad \ldots \ldots \ldots \ldots \quad (3.1)$$

in which

$w_{\bar{z}}$ = the downward displacement at depth $\bar{z}$

w_T = the downward displacement of the butt

$Q(z)$ = the function relating load in the shaft to depth

$A_c E_c$ = the product of shaft cross-sectional area and modulus

of elasticity

When values of downward movement are plotted against corresponding

values of resisting shear stress at a particular depth, a fundamental

relationship called the load transfer curve is generated. A typical

load transfer curve is shown in Fig. 3.3. Points p_1 , p_2 , and p_3

represent stresses and displacements corresponding to points P_1 , P_2 ,

and P_3 in Fig. 3.2. The diagrams above the load transfer curve in

Fig. 3.3 show the state of distortion in the soil at depth $\bar{z}$ for the

three stages of loading. Such distortions generally occur in a limited

zone quite close to the wall of the shaft (DuBose, 1956). In the first

and second diagrams, the soil at the interface with the shaft has moved

downward with the shaft. At a large value of displacement, the soil

slips with respect to the shaft (or shear failure occurs at some short

distance from the interface in the soil) as shown in the third diagram.

Depending upon the soil characteristics, slippage or shear failure may

result in a relaxation of shear stress with further displacement, as

shown in the example load transfer curve, or the shear stress may

become constant, making the curve horizontal beyond point a.

It should be pointed out that the load transfer curve gives all the

necessary information pertinent to the behavior of soil along the sides

of drilled shafts under load. An understanding of the load transfer

behavior of the supporting soil is fundamental to the understanding of

the behavior of drilled shafts.
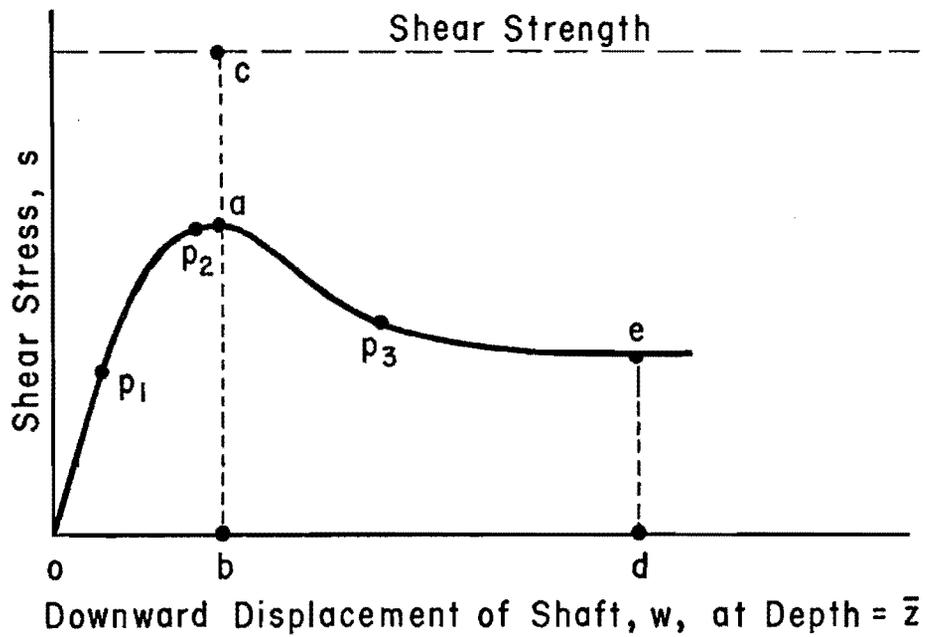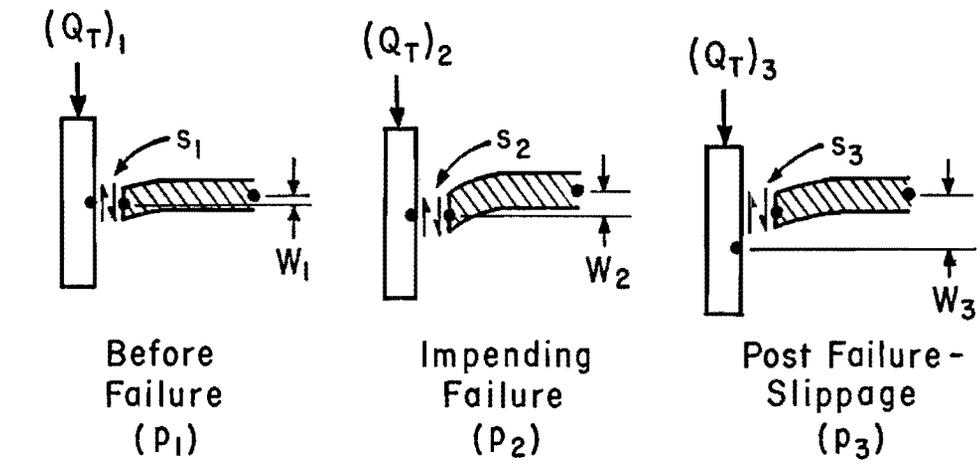
Fig. 3.3. Load Transfer Curve

Complete load transfer curves presently can be reliably obtained by empirical means only, such as conducting load tests on instrumented drilled shafts. Seed and Reese (1957, 1964) have proposed plotting tangential movements against tangential shear stress from field vane shear tests to produce load transfer curves for driven piles in soft clays. Some of the mathematical methods of behavioral synthesis listed later provide the capability of generating load transfer relationships in some soils, but they still require some semiempirical input, such as the ratio of maximum resistance to shear strength.

The load transfer relationship may be different at different levels, even in a homogeneous deposit. Therefore, a set of such curves spanning the entire length of the shaft is necessary to describe completely the action of soil shearing resistance.

Referring again to Fig. 3.3, several important characteristics of the load transfer curve are evident. First, the peak resistance, ab , is seldom equal to the shear strength of the soil, bc. The ratio of the peak resistance to the shear strength (ab/bc) is denoted as the "shear strength reduction factor," α. This factor is an important parameter which must be known in order to compute the frictional capacity of the shaft. The parameter α is a complex function of many variables, including:

1. Type of soil

2. Strength of soil

3. Type of concrete used in shaft

4. Depth of soil level under consideration

5. Method of construction of shaft

6. Time between casting and loading

7. Type of loading, fast or slow

Approximate average values for α have been evaluated from load
tests on drilled shafts. Nearly all such tests have been conducted in
stiff clay and glacial till. A review of research conducted over the
past two decades concerning determination of shear strength reduction
factors is presented in Chapter V. That research provides some limited
indication in stiff clays of the effects of variables listed in 3, 5, and
6. Little is known about the variations of α with depth in the shaft
(proximity to base or ground surface), conditions of loading (drained
versus undrained), and type of soil in which the shaft is installed
(other than stiff clay or till).

Other characteristics of the load transfer curve depend upon the
same variables. Among those most important to the designer are the
initial slope of the curve, the displacement, ob , at which the maxi-
mum resistance is mobilized, and the ratio of the residual resistance,
ed , to the peak resistance, ab. The residual resistance may be nearly
equal to the peak resistance in some deposits, but it may be considerably
less in others, such as in highly overconsolidated clays. Numerical
values obtained in load tests for the displacement necessary to mobilize
maximum shear and indications of residual resistance values are given
in Chapter V.

Resistance of Soil Beneath Base

That part of the applied load not resisted by side shear is sup-
ported by the soil underlying the base. The maximum load which can be

carried in base resistance is given by an appropriate bearing capacity
formula, such as one of those cited in the following chapter.

Qualitatively, the observation may be made that the base load-
settlement relationship is more "flexible" than the side shear load-
settlement function on a unit load basis. That is, far larger displacements
are required to mobilize maximum base loads than are required to mobilize
maximum side loads. For instance, downward movement in the order of 0.1
to 0.3 inches will produce side failure or slippage in stiff clay, whereas
settlements of 5 to 20 per cent of the base diameter are required to plunge
the base. This fact is illustrated graphically in Fig. 3.4, which shows
the load-settlement relationships for the base and the sides for hypothet-
ical shafts. Fig. 3.4a shows curves for a straight shaft and Fig. 3.4b
for an underreamed shaft in the same soil with the same overall length
and with a base diameter twice that of the straight shaft. Settlement in
either case may be assumed to be the butt settlement. In both shafts the
side resistance dominates for smaller settlements such as those which
occur at working load.

The example shown is a special case. Obviously, every shaft will
have its own characteristic pair of load-settlement relationships. For
a shorter shaft, or one with a larger base, the base load-settlement
relationship becomes more dominant, while the converse is true for a
longer shaft or one which has a smaller base.

The fact that base and side resistances are mobilized at different
rates gives rise to a need for considering the factors of safety against
base and side failure separately. For any value of applied load, Q_T ,
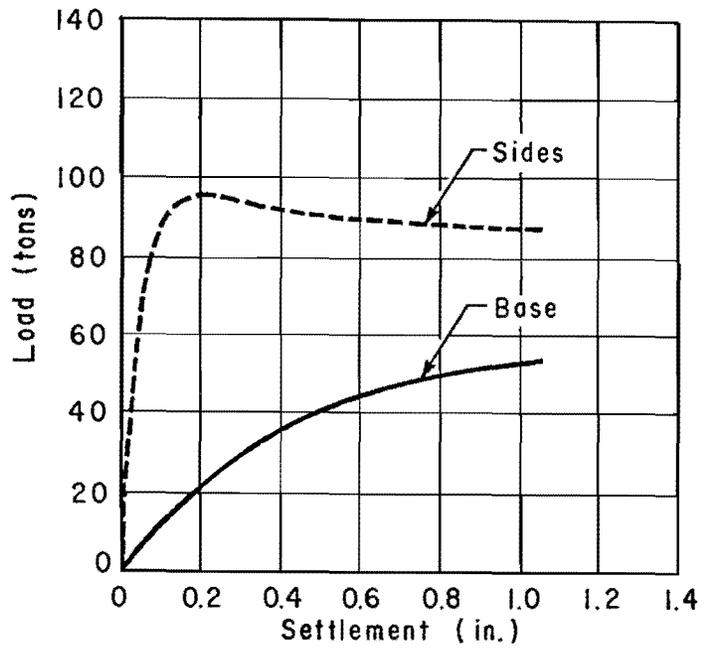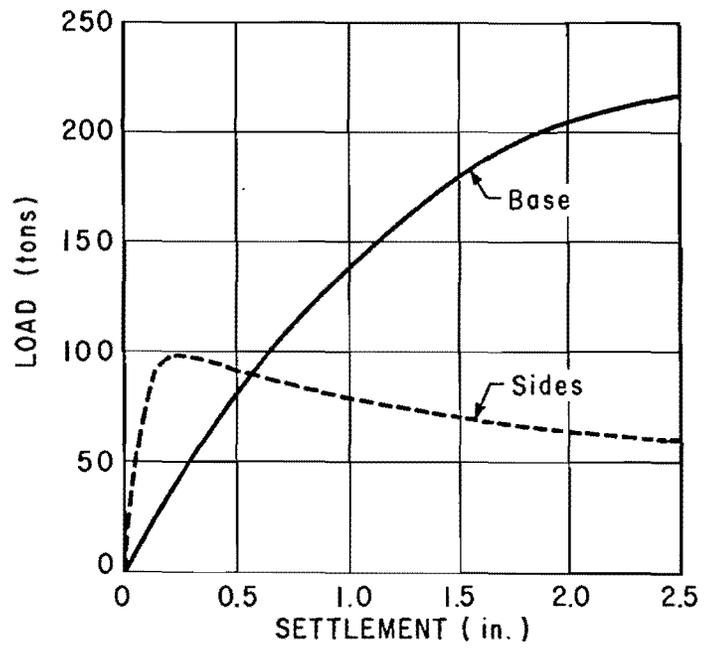
Fig. 3.4. Hypothetical Load-Settlement Relationships
for Side- and Base-Load Components

$$Q_T = Q_S + Q_B \quad \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \quad (3.2)$$

in which

Q_S = side resistance

Q_B = base resistance

It is clearly seen in Fig. 3.4 that whenever a load is applied, the factor of safety against base failure will not be equal to the factor of safety against side failure. For example, for a working load of 125 tons on the belled shaft, about 95 tons will be carried through side shear and about 30 tons by base resistance. These two reactions represent factors of safety of 1.0 (against peak resistance) and 7.3 for the sides and base, respectively. The overall factor of safety against ultimate failure is about 2.25. The gross settlement is 0.2 inches.

Suggestions have been advanced that the separate factors of safety for base and sides should be considered to insure adequate stability (Burland, Butler, and Dunican, 1966; Tomlinson, 1969). The working load on the butt is given by

$$(Q_T)_{\text{working load}} = \frac{(Q_T)_{\text{ult}}}{(F.S.)_{\text{shaft}}}$$

$$= \frac{(Q_S)_{\text{ult}}}{(F.S.)_{\text{sides}}} + \frac{(Q_B)_{\text{ult}}}{(F.S.)_{\text{base}}} \quad \dots \dots \dots \dots \dots \quad (3.3)$$

in which

F.S. = factor of safety at working load

ult = ultimate resistance values.

It is suggested that the working load be computed in two ways. First, $(Q_T)_{ult}$ is determined from considerations of limiting equilibrium, discussed in the next chapter, and an overall $(F.S.)_{shaft}$ of 2 to 3 is applied to $(Q_T)_{ult}$. Second, separate factors of safety of 1 and 3 are applied to $(Q_S)_{ult}$ and $(Q_B)_{ult}$, respectively, and the working load computed from Eq. 3.3. The lesser of the two values should be used for design load. The first method normally governs for straight shafts, while the second usually controls for belled shafts. For the design of belled shafts in heavily overconsolidated clays, it may be appropriate to take $(Q_S)_{ult}$ to be the residual side resistance rather than the peak resistance. Methods for calculating $(Q_S)_{ult}$ and $(Q_B)_{ult}$ are considered in Chapter IV.

Bhanot (1968) has shown that the individual factors of safety for base and sides as functions of the total factor of safety plot approximately as straight lines on log-log scales. An example showing the base and side factors of safety (side factor of safety based on peak resistance) as functions of the overall factor of safety for the belled shaft illustration from Fig. 3.4 is given in Fig. 3.5. This type of graph can be useful in design and in illustrating behavior, but its construction requires knowledge of the side and base responses to load, which can only be obtained through load tests on instrumented shafts or by one of the mathematical methods of synthesis outlined below.

Further details of procedures for utilizing the separate base and side factors of safety in analysis are given by Hobbs (1963) and Whitaker and Cooke (1966).
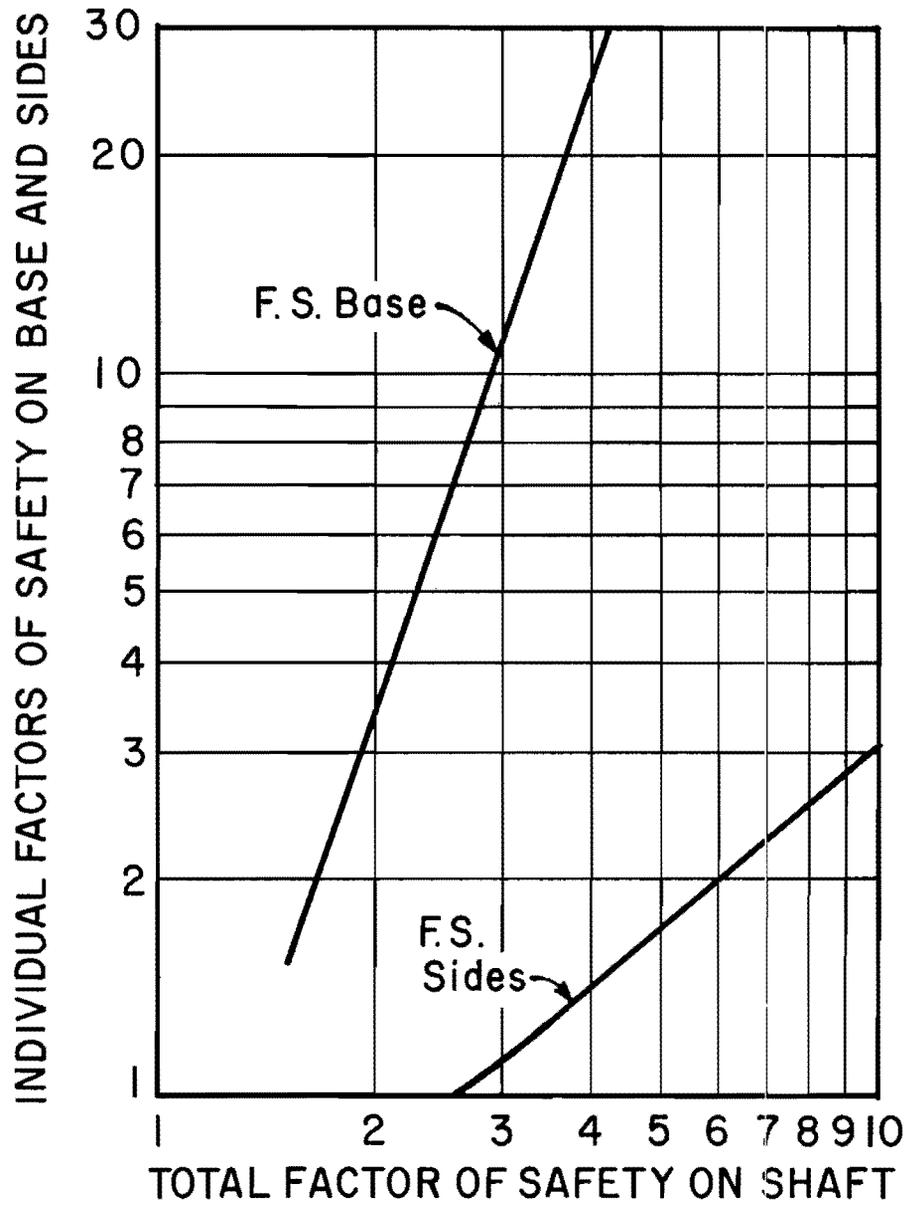
Fig. 3.5. Relationship Between Base, Side, and Total
Factors of Safety on a Drilled Shaft

Mathematical Synthesis of Behavior

The introduction of digital computers has made possible the mathematical simulation of mechanical behavior of axially loaded drilled shafts. Several methods are now available for accomplishing this simulation. They are tedious, if not impossible, to apply by hand, but they are well-suited to be programmed for the computer, which can efficiently and quickly carry out the necessary numerical computations. These methods, summarized briefly in the following paragraphs, can be used to obtain complete load-settlement and load distribution relationships, provided adequate information is available to describe the behavior of the soil. They provide a vehicle for studying the various parameters affecting drilled shaft behavior and may be used in design applications.

Discrete Element Method Requiring Load Transfer Curves as Input.

Seed and Reese (1957) and Coyle and Reese (1966) present a numerical scheme for determining the load distribution and load-settlement characteristics of a single axially loaded pile or drilled shaft. A model for the pile, composed of rigid blocks connected by springs representing the compressibility of the pile, is described mathematically. Nonlinear leaf springs, describing the shear resistance of the soil as a function of displacement, are introduced at each block and modeled mathematically. A nonlinear coil spring describing the base load-settlement relationship is also provided. For a prescribed base displacement, the movement and shear stress at each block are computed using the requirement that all blocks be in static equilibrium and that forces in the resisting soil springs be compatible with the stress-displacement relationship described by the appropriate load transfer curve, which is one of a family of such

curves provided as input. Iterative schemes are required to accomplish this computation. Finally, a value for the load at the top of the pile or shaft is computed which is necessary for overall static equilibrium. This is the value of the applied load which is necessary to produce the base displacement originally assumed. The computations yield a load distribution curve, butt settlement, and a strain diagram for the pile. By varying the value of base displacement, an entire load-settlement relationship can be generated along with the corresponding load distribution curves. The primary limitation of this method is the need for prior determination or estimation of load transfer and base load-settlement curves. Empirical procedures for producing approximate load transfer and base load-settlement relationship based on the load tests of drilled shafts in stiff clay described herein are given in Chapter XIII.

Discrete Element Method Employing Mindlin's Solution. Mindlin (1936) derives expressions for stresses and displacements due to a force acting inside a semi-infinite elastic solid. Several investigators (D'Appolonia and Romualdi, 1963; Thurman and D'Appolonia, 1965; Salas and Belzunce, 1965; Nair, 1967; Mattes and Poulos, 1969) have utilized these expressions to develop mathematical models for axially loaded piles. Basically, the numerical procedure involves solving for soil reaction forces, compatible with the stress-strain behavior of the soil at nodal points along the pile or shaft, which will put the pile in static equilibrium. This is done by developing a matrix stiffness equation which relates the soil reaction force at each node (including base reaction at the bottom node) to the displacement of every node along the pile by using Mindlin's equation for displacement at one point due to a force at that or another

point. (The Mindlin solution is singular at the point at which a load is applied. This problem is circumvented by assuming the soil reaction forces to be distributed around the circumference of the element. Displacement is computed for nodes located at the center of the elements.) A similar matrix equation is set up to relate elastic compression forces in the pile to the nodal displacements. The equations for pile and soil forces are combined by assuming displacement continuity across the pile-soil interface, and the resulting single stiffness equation is solved for a given value of imposed load. The results yielded are the same as for the previous method. The Mindlin-type procedure requires the assumption that the supporting soil is elastic, and, hence, gives best results for relatively small loads. Nonlinear behavior has been simulated by considering local yield between the soil and the pile, requiring prior information concerning the stress required to cause slippage at the soil-element interface, and by assuming the base to have an elastic-perfectly plastic load displacement relationship.

Finite Element Method. The finite element method (Zienkiewicz, 1967) has been increasingly applied to problems in structural mechanics for several years and has been recently extended to problems in soil mechanics. Skipp (1966), Zienkiewicz (1967), and Ellison (1968) have reported applications of the method to problems in pile-soil interaction. Ellison has presented specific solutions for drilled shaft behavior for straight shafts in stiff clay.

The finite element method involves dividing or discretizing the pile-soil system into many simply-shaped regions of finite dimensions. Each region or "element" is ascribed the stress-strain properties of the

part of the pile or soil mass it represents. The elements are connected
at discrete points called nodes. Linear stiffness equations relating
nodal displacements to nodal forces for every element are obtained using
the principle of minimum potential energy. The stiffness equations for
each element are combined to form a global stiffness matrix equation for
the entire system, which relates force to displacement at every node.
After imposing the necessary boundary conditions, the overall stiffness
equation is solved for the nodal displacements using an appropriate algo-
rithm for solution of simultaneous linear equations. Stresses and strains
are evaluated numerically from the computed nodal displacements. Non-
linear material properties can be handled by employing step-by-step
loading or by using iterative techniques (Zienkiewicz, 1967).

The finite element method can handle arbitrary geometries and can
be adapted to take account of material discontinuities such as tension
cracks and interface slippage. It still requires knowledge of the mag-
nitude of the stress at which pile-soil slippage occurs, however. It
needs a larger computer and more execution time than either of the other
two methods, but it has the potential of giving the most accurate infor-
mation with the fewest assumptions regarding soil behavior.

CHAPTER IV

CURRENT METHODS OF DESIGN AND ANALYSIS

While this study is concerned primarily with describing the behavior of isolated drilled shafts under axial load, it is appropriate to provide some perspective on the problem by mentioning important behavioral factors that must be considered in design and by describing current methods used in the design and analysis of drilled shafts. Detailed treatment of the state of the art of the design of deep foundations in clay is given by De Mello (1969).

General Design Concepts

The design of drilled shaft foundations, like that of other foundation systems, is predicated on two principal requirements:

1. There should be an adequate factor of safety against bearing failure.

2. The settlement at working load should be within allowable limits.

In the past, emphasis has been placed on the former requirement, with settlements being scrutinized only in problem soils or for heavily-loaded shafts. In fact, settlements of drilled shafts at working load are often of little concern where the shafts are essentially end bearing. However, whenever floating shafts are employed, settlements can be important. Settlements of floating shafts depend on a number of factors, including

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individual shaft geometry, soil properties, and geometry of the group in which the individual shaft is located. In many floating shaft installations, such as bridge bents in the stiff clays and clay-shales of Texas, experience has indicated that satisfaction of the first requirement automatically satisfies the second.

For floating shafts, a basic decision must be made whether to provide the necessary bearing capacity by using longer straight shafts or shorter belled shafts. Straight shafts, deriving a majority of support from side friction, settle less at comparable working loads than do belled shafts, which resist a larger part of the load in bearing. This phenomenon was indirectly illustrated in Fig. 3.4. By varying the lengths of the stems and the sizes of the bells, a designer can effectively control settlements in a drilled shaft foundation. This option is often taken out of the designer's control by codes which prohibit or severely limit the use of side resistance. In stratified deposits or for end-bearing shafts, the depth and nature of potential founding strata usually govern the design length.

Principal features of procedures presently used to determine allowable loads on single shafts and groups of shafts and corresponding settlements will now be treated. Most methods for design of drilled shafts have been adapted primarily from similar procedures for design of driven piling, with appropriate allowances for differences in installation procedures.

Prediction of Allowable Compressive Load on an Isolated Drilled Shaft

Semiempirical Procedures. A common method of computing the allowable loads for potential designs is to consider the shaft as a deep footing

and compute the allowable load on the base by multiplying a safe bearing pressure given in an appropriate local code by the base area. Representative safe bearing pressures for several soil and rock types where drilled shafts are commonly used are given in Table 4.1.

Side friction is usually disallowed except for sockets in rock, so that the resulting allowable base loads are also the working loads permitted on the butt, provided the allowable concrete stress is not exceeded. Permissible shear stresses for shafts socketed into rock are also given in Table 4.1.

Penetrometer soundings provide a basis for obtaining design bearing values. Charts have been developed (Peck, Hanson, and Thornburn, 1953; Texas Highway Department, 1964) which give allowable contact pressures as functions of results of penetrometer tests.

Allowable loads are then sometimes reduced for floating shaft groups by applying efficiency factors computed from empirical formulas. Group behavior is treated in more detail later. If settlements are to be considered, they are usually estimated by rule of thumb or from experience with drilled shaft foundations in similar soils.

Rational Procedures. Most rational design procedures for piles or drilled shafts are based on the following limiting equilibrium formula for ultimate capacity:

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

in which

$(Q_T)_{ult}$ = ultimate load at top of pile or shaft

TABLE 4.1. REPRESENTATIVE ALLOWABLE BEARING

AND FRICTION VALUES FOR DRILLED SHAFTS

Material	Allowable Stress (tsf)	Source of Information
Very Sound Limestone (Chicago)	up to 200 (bearing)	Osterberg, 1968
Sound Limestone (Chicago)	120 (bearing)	White, 1967
Hardpan (Till) (Chicago)	6 - 12 (bearing)	Osterberg, 1968
Hardpan (Till) (Detroit)	25 (bearing)	Housel, 1969
Shale (Texas)	5 - 30 (bearing)	Texas Highway Department, 1964
Expansive Shales and Clay-Shales (Texas)	6 (bearing)	U.S. Army Engineer District, Fort Worth, Texas, 1968
Clay (Chicago)	Unconfined compressive strength (bearing)	Osterberg, 1968
Sound Limestone	18 (shear)	White, 1967
Average Limestone	14 (shear)	White, 1967
Poor Limestone	7 (shear)	White, 1967
Shale	0.8 - 3.0 (shear)	Texas Highway Department, 1964

$(Q_S)_{ult}$ = ultimate side load

$(Q_B)_{ult}$ = ultimate base load

The use of Eq. 4.1 requires a reliable estimate of profiles of the soil

parameters: cohesion and internal friction. The allowable axial load

is then computed as indicated in Chapter III. At working load, the indi-

vidual factors of safety against side and base failure will be different

because ultimate side and base loads are mobilized at different displace-

ments.

The quantities $(Q_S)_{ult}$ and $(Q_B)_{ult}$ are computed separately and

are assumed to be mutually independent. In reality, they are probably

not independent, but the exact nature of base and side interaction is

not understood well enough to permit a reasonable analytical formulation

of this phenomenon.

The unit ultimate side resistance $(q_S)_{ult}$ is normally obtained

from a total stress analysis using a modification of Coulomb's equation,

which assumes that failure occurs at or near the soil-shaft interface:

$$(q_S)_{ult} = \alpha c_u + K_o \sigma_v' \tan \delta \quad \ldots \ldots \ldots \ldots \ldots (4.2)$$

in which

α = cohesion reduction factor

c_u = undrained cohesion of the soil along the side of the shaft

σ_v' = vertical effective stress in the soil adjacent to the shaft

K_o = coefficient of lateral earth pressure, or the ratio of

horizontal effective stress to vertical effective stress

δ = angle of friction between the soil and concrete

The determination of values for α is an important step in calculating the side resistance. Considerable research has been done in recent years on the ultimate side resistance of drilled shafts in cohesive soils. A review of that work is presented in the next chapter. Briefly, however, it can be stated that previous research has shown that α can be taken to be approximately 0.45 in deep shafts in stiff clay with the understanding that the product αc_u should not exceed some set value in the range of 1500 to 2000 psf. The variation of α with depth is largely unknown, and the value of 0.45 represents an average over the length of the shaft.

It is likely that clay soils in the zone of seasonal moisture fluc-tuation (usually extending several feet below the ground surface) can shrink away from the concrete periodically as they become deficient in moisture, with the result that α can approach zero. This fact must be taken into account when computing side resistance for design. Uncer-tainties regarding the depth of this effect have resulted in low allow-ables for α in short shafts. In many areas having highly active soils and unfavorable rainfall situations, α is always taken as zero for the entire length of the stem, although such a drastic practice is probably unwarranted.

The value of the quantity K_o is unknown for drilled shafts. It is certainly likely to be less than that for driven piles because of the inward movement of the soil that will occur during drilling operations, and it may approach the active earth pressure coefficient in sands. Since information is scarce concerning development of side resistance in gran-ular soils, a conservative value for K_o in the order of one-third should be taken for design purposes for shafts in sand. Lower values may be

anticipated when the borehole is augered dry (some cohesion in the sand) than when it is augered with mud, which tends to prevent inward movement of the soil. Considerable research is required to determine reasonable values for coefficients of lateral earth pressure for drilled shafts installed in granular materials.

The quantity σ_v' must be determined for analysis of shafts in sand or for long-term capacity for shafts in clay. It is customarily taken as the effective overburden pressure at the depth in question. Little is known about the behavior of drilled shafts in sand. Recent research (Vesić, 1963; Kerisel, 1964; Robinsky and Morrison, 1964) concerning the behavior of buried and driven cylindrical piles has shown evidence that the action of pushing an element into sand, beyond a depth of 5 to 20 diameters, causes a release of vertical pressure in the sand adjacent to the element. The magnitude of vertical pressure release depends on the relative density of the soil. It is suggested that the stress release occurs as a consequence of the withdrawal of vertical support from the sand surrounding the pile directly above the tip, as illustrated in Fig. 4.1. This action in turn promotes arching of horizontal stresses around the pile with a resultant decrease in both vertical and horizontal stress in the soil which is being sheared near the pile wall. Consideration of this type of behavior leads to the conclusion that unit side resistance approaches some constant value as depth increases. This arching phenomenon may be quite pronounced for shafts with enlarged bases.

For purposes of design, the confining stress value is obtained by limiting the effective confining pressure σ_v' to be the overburden pressure present at a depth of a few diameters.

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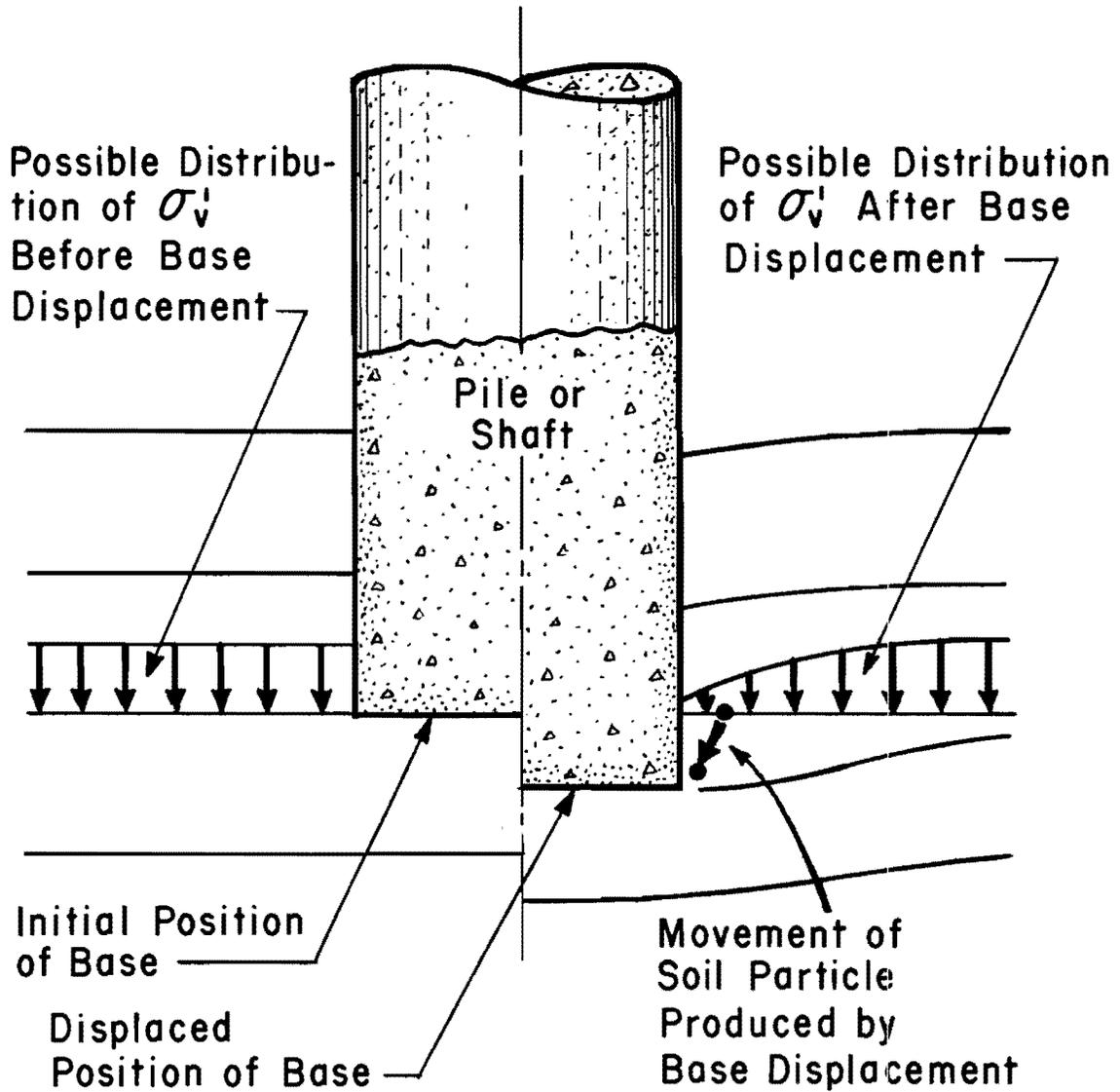
Fig. 4.1. Action of Soil Near Base of Drilled Shaft

In an actual drilled shaft in sand, installed by a non-displacement method, some horizontal arching of stresses in the sand mass may occur due to the inward movement of the sand which occurs during drilling. Furthermore, when the wet concrete is placed and the slurry or casing support withdrawn, the horizontal pressure against the side of the shaft is limited by the lateral fluid pressure of the concrete, which becomes constant at perhaps 10 to 20 psi at a depth of several diameters, as discussed later. Thus, the lateral pressure of the concrete may be less than that of the mud used to keep the hole open during drilling. Further arching evidently may then occur at larger depths.

Hence, the horizontal stresses around a drilled shaft may not be proportional to the overburden pressure, even before the shaft is loaded. In any event, horizontal stresses almost certainly will be reduced by the phenomenon of removal of support described previously after load is applied. The possibility of these various occurrences suggest a conservative working hypothesis that, lacking experimental evidence to the contrary, σ_v' should be limited to the overburden pressure at perhaps ten stem diameters for straight drilled shafts installed in dense sands.

Vesić (1970) presents the following empirical expression for limiting values for shaft resistance against metal cylinders buried in sand, based on model tests. It represents only an approximation to maximum values to be expected for drilled shafts, which have rough sides, and which may have different horizontal stress distributions than do buried cylinders due to differences in method of placement.

$$(q_s)_{ult} = (0.025)(10)^{1.5D_r^4} \quad \ldots \ldots \ldots \ldots \ldots \ldots (4.3)$$

in which

D_r = relative density of the sand

$(q_S)_{ult}$ = ultimate unit side resistance in tsf

Even more stringent limitations may be needed for underreamed shafts in sand.

The angle of skin friction δ is taken as zero in designing shafts in clay. Experimentally, δ has been found to be approximately equal to ϕ , the angle of internal friction, for sand in contact with rough concrete (Potyondy, 1961).

Values of $(q_S)_{ult}$ from Eq. 4.2 are integrated analytically or numerically over the length of the shaft to arrive at a value for $(Q_S)_{ult}$.

A procedure for estimating ultimate side resistance of a shaft under sustained loading in clay, using effective stress parameters, has been proposed recently (Chandler, 1968). The method assumes that drained shear conditions exist in the soil. It makes use of the following expression for $(q_S)_{ult}$:

$$(q_S)_{ult} = c' + K_o \sigma_v' \tan \phi' \quad\dots\dots\dots\dots\dots\dots\dots (4.4)$$

in which

c' = effective cohesion of the clay

σ_v' = effective vertical stress due to overburden

ϕ' = effective angle of internal friction of the clay

The use of Eq. 4.4 permits a realistic consideration of the effects of possible reconsolidation of soil remolded during drilling along the

walls of the borehole and can also take account of possible relaxation of the soil under drained shear conditions at high stress levels due to dilatancy in heavily overconsolidated clays. The latter consideration is important because shear stress levels high enough to promote dilatancy may exist in short, belled shafts at working loads.

Chandler states that the horizontal stresses in the clay around the borehole return to the full magnitudes that existed in situ before the shaft was installed, and that in situ K_o values should then be used in computations for long-term side resistance. It seems unlikely, however, that full reestablishment of K_o will actually occur. Therefore, considerable judgement must be exercised concerning the correct value of K_o when used in a computational procedure. Chandler, using $c' = 0$ (assuming the clay to be initially softened and remolded), $\phi' = 21$ degrees , and an in situ $K_o = 2$ to 3 (for overconsolidated soil), predicted ultimate side resistance stresses for drilled shafts in London Clay. He compared his values with average values measured by several investigators during short-term load tests. His computations represented an upper limit to the measured values, but would have probably been nearer the average for the long-term tests.

This procedure can be extended to clays other than London Clay by conducting appropriate soil tests and estimating K_o. Brooker and Ireland (1965) have published useful graphs which relate K_o (in situ) to the overconsolidation ratio and plastic limit of clay.

Chandler's method appears promising for estimating long-term shearing resistance, provided procedures for determining actual horizontal stresses against the sides of a drilled shaft can be devised.

The ultimate bearing capacity of the base is customarily estimated from a bearing capacity formula, usually presented in the general form:

$$(q_B)_{ult} = f_1 c_u N_c + \sigma_v' N_q + f_2 \gamma' \frac{B}{2} N_\gamma \quad \ldots \ldots \ldots \ldots (4.5)$$

in which

$(q_B)_{ult}$ = ultimate unit bearing stress on the base

c_u = average undrained cohesion in the soil beneath the base

σ_v' = effective vertical stress in the soil on the horizontal plane passing through the base

γ' = effective unit weight of the soil

B = diameter of the base

f_1, f_2 = base shape factors

N_c, N_q, N_γ = bearing capacity factors

Methods are available in the technical literature for numerical evaluation of the shape and bearing capacity factors (Terzaghi, 1943; Meyerhof, 1951). The methods differ in that various logical modes of failure are assumed. Many investigators eliminate the expression involving N_γ in computing the bearing capacity of deep foundations, since its contribution is relatively minor, and that expression will be ignored in the discussions here.

In clays, N_q and f_2 are equal to 1, and σ_v' is normally taken as the stress due to overburden. Furthermore, for a specified base geometry, N_c can be redefined to be the original product $f_1 N_c$, so that

$$(q_B)_{ult} = c_u N_c + h\gamma' \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.6)$$

The value of the base pressure at failure due to the weight of the shaft is approximately equal to $h\gamma'$, where h is the depth of the base. Thus the net bearing capacity is given by

$$(q_B)_{ult,net} = c_u N_c \quad \ldots \ldots \ldots \ldots \ldots \quad (4.7)$$

Skempton (1951) has quoted the Mott-Gibson theory as applicable for N_c for deep footings in clay. That theory yields the following expression for N_c:

$$N_c = \frac{4}{3}\left[\log_e\left(\frac{E_o}{c_u}\right) + 1\right] + 1 \ldots \ldots \ldots \ldots \quad (4.8)$$

in which $\dfrac{E_o}{c_u}$ is the ratio of the initial Young's modulus to the cohesion of the clay for undrained conditions. Equation 4.8 gives N_c values of 7.6 to 9.4 for the usual range of $\dfrac{E_o}{c_u}$ for clays (50 to 200). Meyerhof (1951) derived a value of 9.34 for N_c for deep foundations in purely cohesive soils. Model and full-scale tests have tended to confirm a fairly consistent value of about 9 for N_c in saturated clay. Therefore, it appears appropriate to take N_c equal to 9 for use in Eq. 4.6. Hence:

$$(q_B)_{ult,net} = 9\,c_u \quad \ldots \ldots \ldots \ldots \ldots \quad (4.7a)$$

and

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

where A_B is the area of the base.

It may be appropriate to use a drained shear analysis for base capacity in clay soils in some instances. For example, base capacity

calculated from drained shear strength parameters are used in conjunction with Chandler's side resistance method if the failure load is approached slowly enough to permit full drainage beneath the base. In fact, in some heavily overconsolidated clays, the bearing capacity based on drained strength parameters may be less than that computed from undrained parameters, while the opposite result is normally expected.

Bearing capacity should be checked according to both drained and undrained criteria if the possibility exists for drained shear failure in heavily overconsolidated clay supporting the base, and the minimum value used. (In a drained analysis, drained cohesion is used in place of c_u in Eq. 4.5, and the drained angle of internal friction is used in estimating N_c and N_q. Undrained base capacity is computed by using Eq. 4.7b.) Otherwise, if the soil is not heavily overconsolidated, or if base failure can only be produced by a fairly rapid overload, as is most often the case considered in design, Eq. 4.7b should be used for computing base capacity in clay.

In sandy soil, N_c and N_q can be evaluated from charts based on expressions for N_c and N_q as functions of the angle of internal friction. Vesić (1967), however, simplified the computation for cohesionless soils by introducting a single factor N_q^* which incorporates the shape factors, so that

$$(q_B)_{ult} \approx (q_B)_{ult,net} = \sigma_v' N_q^* \quad \ldots \ldots \ldots \ldots \ldots \ldots (4.9)$$

and

$$(Q_B)_{ult} = \sigma_v' N_q^* A_B \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.9a)$$

The quantity σ_v' is the effective vertical stress at the base of the shaft. Emphasis should be placed on the fact that σ_v' is not necessarily equal to the overburden pressure, if sand overlies the base, due to the phenomenon of vertical stress release mentioned in connection with side resistance. Values for σ_v' , computed according to the same criteria used in computing side resistance, gives base resistances consistent with measured ultimate base loads.

Vesić (1967) presents graphs of N_q^* versus ϕ for circular footings according to various bearing capacity theories. He gives, for example, N_q^* of about 120 according to his own theory for $\phi = 40$ degrees.

Tomlinson (1969) suggests that the base capacity of a drilled shaft in sand is less than that computed from the bearing capacity equation using in situ values for angle of internal friction. Apparently, the action of augering and stress release loosens sand supporting the base, requiring that a reduced friction angle be used in calculations.

Table 4.2 gives a concise summary of the equations for base and stem capacity just considered and suggests numerical values for appropriate parameters based on the present state of the art.

Recent experience with predicting pile capacities by static loading of penetrometers, similar in design to the Dutch cone penetrometer, have been encouraging. Such devices can measure point and skin resistances independently. Static cone resistance values have correlated much better with measured skin friction and point bearing in driven piles in sand than have results from the dynamic standard penetration test. All of the arching effects which occur in driven piles or drilled shafts also occur on the static cone penetrometer; thus, the indicated skin friction and point

TABLE 4.2. SUMMARY OF EQUATIONS FOR USE IN COMPUTING CAPACITIES OF NON-END-BEARING DRILLED SHAFTS FOR DESIGN PURPOSES.

GENERAL EQUATION:

$$(Q_T)_{ULT} = (Q_S)_{ULT} + (Q_B)_{ULT}$$

OR

$$(Q_T)_{ULT} = A_S \alpha c_{SIDES} + 2\pi r K_0 \tan \delta \int_{z=0}^{z=\text{stem length}} (\sigma_v')_{SIDES} \, dz +$$

$$A_B \left\{ c_{BASE} N_c + (\sigma_v')_{BASE} N_q^* \right\}$$

Where r = stem radius; A_S = peripheral area of stem; A_B = base area.

SPECIAL FORMS

In Clay	In Sand
(Undrained Conditions)	(Drained Conditions, Straight Shafts[1])

In Clay

$$(Q_T)_{ULT} = A_S \alpha c_{SIDES} + A_B c_{BASE} N_c$$

where

$\alpha = 0.45$
(range 0.3 - 0.6)

c_{SIDES} = Avg. undrained cohesion along sides

c_{BASE} = Avg. undrained cohesion for two base diameters beneath base.

Note: $\alpha c_{SIDES} \leq 2000$ psf

In Sand

$$(Q_T)_{ULT} = 2\pi r K_0 \tan \delta \int_{z=0}^{z=\text{stem length}} (\sigma_v') \, dz + A_B (\sigma_v')_{BASE} N_q^*$$

where

$K_0 = 0.2 - 0.4$

$\delta = \phi_d$

(σ_v') = Effective overburden stress at depths up to ten stem diameters, becoming equal to overburden at depth of ten stem diameters for greater depths.

$(\sigma_v')_{BASE}$ = Same definition as above.

N_q^* = Bearing capacity factor, function of ϕ_d. Numerical values given by Vesić (1963; 1967).

[1]Applicability to belled shafts unknown

capacity become constant at some depth. It has been shown in one case (Vesić, 1970) that static cone results can be used directly to predict driven pile capacity with reasonable accuracy. If direct correlations with field static penetrometers can be made for loads mobilized by drilled shafts, considerable expediency in the design process can be realized.

Whenever shafts are installed in layered deposits or in true c-ϕ materials, a measure of engineering judgement must be employed when using rational analysis in design, particularly with respect to obtaining the correct bearing capacities. Errors in estimating appropriate effective confining pressures and earth pressure coefficients are greatest in layered soils, since the soil may behave differently than it would in a uniform deposit. A hypothetical example is the possible underestimation of the base capacity of a deep drilled shaft placed through clay with the base resting on (not in) a sand stratum. Under such conditions, the arching phenomenon above the base may not develop, with the result that the confining pressure may remain equal to the effective overburden pressure and the base capacity correspondingly increased.

Load Tests. Performance of full-scale load tests on prototype shafts remains the best method for determining carrying capacity of drilled shafts. However, such tests are difficult and expensive to perform because of typically high shaft capacities. In fact, load tests carried to failure may be impossible for end-bearing shafts, although they do provide a means of proving the design load. Therefore, as a general rule, heavy reliance is made on rational or semiempirical design methods to estimate allowable loads for routine designs.

A typical test arrangement is shown in Fig. 4.2. The load is applied by jacking against a reaction beam with a high-capacity jack. The reaction beam is anchored, in turn, by two to four piles or drilled shafts placed some distance from the test shaft. The load is obtained by reading a calibrated hydraulic pressure gage on the jack or by using a load cell between the jack and the reaction beam. Settlements corresponding to various values of applied load are recorded by reading dial gages which are supported from independent reference beams and whose stems rest on protrusions from the test shaft. Dial gages are often placed in pairs on opposite sides of the shaft in order to determine whether tilting of the butt occurs and to eliminate that effect from plotted load-settlement graphs.

An important consideration is that the anchor shafts be sufficiently far from the test shaft to minimize undue influence on the behavior of the test shaft. Whitaker and Cooke (1966) report results of model tests in clay with four anchor shafts spaced symmetrically about a test shaft. The diameters of all shafts were equal. At spacings of 3.5 diameters or more, the load-settlement relationship of the model test shaft was uneffected by the presence of the anchors. Hence, it appears that spacings in the order of 3 to 4 diameters are required. The supports for the reference beams should also be placed at least that far from both test and anchor shafts. An alternative to using anchor shafts is to jack against a platform loaded with kentledge and resting on cribbing or to dead load the shaft directly.

Another important factor is the friction which develops in the jack piston. Care should be taken to make certain that the piston and the
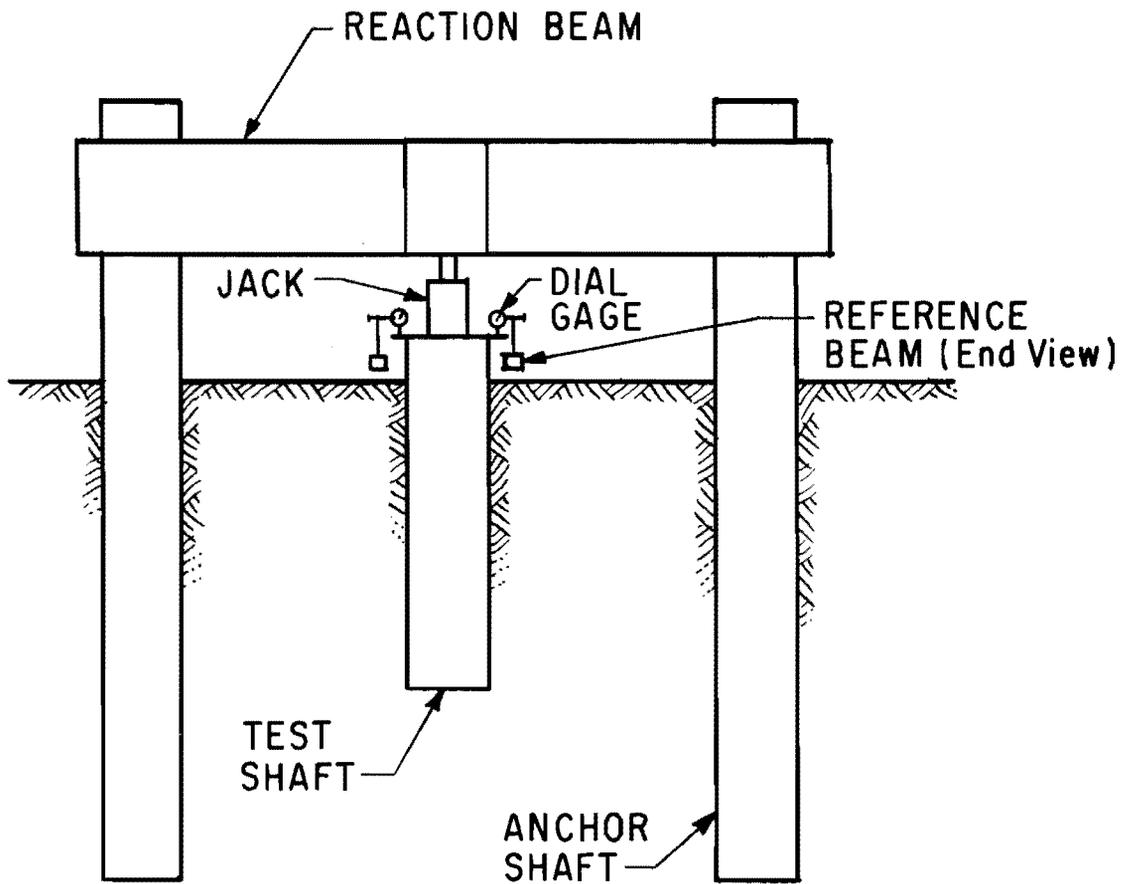
Fig. 4.2. Typical Arrangement for Testing a Drilled Shaft

loading surface of the reaction beam are perpendicular if jack pressure readings are to be used to measure the applied load. Otherwise, eccentric loads will develop on the piston that will cause some amount of frictional binding and result in indicated loads that may be too high by as much as five to ten per cent.

Several different procedures exist for conducting load tests on piling and drilled shafts. These procedures, along with references to specifications or articles describing the details of each procedure, are tabulated in Table 4.3.

TABLE 4.3. LOAD TEST PROCEDURES

Procedure	Reference
Maintained Load (ML)	American Society for Testing and Materials (1970), Test D 1143-69 Texas Highway Department (1962) Specification Item 405
Quick Load (QL)	Texas Highway Department (1965) Special Provision to Specification Item 405 Fuller and Hoy (1970)
Constant Rate of Penetration (CRP)	Whitaker and Cooke (1961)
Cyclic Method	Van Weele (1957)
Equilibrium Method	Mohan, Jain and Jain (1967)
Pullout Tests	May be conducted according to any of the above five procedures.

The indicated capacity and settlement of a test shaft may be influenced considerably by the procedure employed and by the care with which the test is conducted.

The ML test is the most universally accepted method for testing a pile or drilled shaft. The load is applied in increments, with sufficient time elapsing between load applications to allow settlement to cease or to decrease below a specified small rate. Loadings are increased to twice the design load, and the last load is held for an extended period of time to assure that the element is stable. The load is then either removed without achieving failure or increased to the value required to plunge the element. Normally such a test takes from several days to several weeks to perform. Settlements which occur are combinations of elastic and consolidation effects, and mobilized shear strengths of clay soils lie somewhere between those existing in the undrained and fully drained states.

The QL test is performed by adding prescribed increments of load in prescribed short increments of time. For example, the Texas Highway Department QL procedure (Texas Highway Department, 1965) allows 5-to 10-ton load increments in 2 1/2-minute intervals. QL tests require only a few hours to perform and generally result in more nearly undrained conditions of shear failure than do ML tests. Settlements correspond closely with ML results up to about one-third the ultimate load for straight shafts. Beyond that point, the QL test is expected to give smaller settlements for corresponding values of load, especially in clays. Failure loads obtained by both QL and ML methods are usually nearer the same value (Fuller and Hoy, 1970), although in soils subject to creep failure (highly overconsolidated clays or clay-shales), the QL procedure may indicate higher ultimate capacities.

It appears that the QL procedure is more acceptable than the ML procedure in determining load capacity (due to reduced testing time) and in providing a condition in which capacities can be more rationally correlated to conventional undrained laboratory shear tests. The converse is true with respect to obtaining realistic values of settlement, although settlement results will not be too divergent at working load for shafts in sand or for shafts deriving most of their support from side friction. Thus, the designer should consider whether load capacity or settlement is the parameter that is to be investigated before choosing between the ML and QL procedures.

The other procedures listed in Table 4.3 are designed for special uses. The CRP method, as the name implies, involves forcing the shaft into the ground at a constant rate of settlement. CRP load tests give a better definition of post-failure behavior than do other methods. CRP tests are usually short term in nature, with a rate of penetration in the order of 0.03 inches per minute being employed. They can be used in conjunction with other procedures to define accurately the last portion of the load-settlement curve.

The cyclic method allows the investigator to separate side and base resistance in an approximate fashion without instrumentation (Van Weele, 1957). This method is based on the assumption that cyclic loading causes the load-versus-settlement relationship for the base of the foundation element to become linear.

The equilibrium method is a short-term test which will produce a load-settlement curve similar to ML tests (Mohan, Jain, and Jain, 1967). It is performed by applying load increments through jacks as in the other

methods. But, instead of maintaining a constant load by pumping, the load is allowed to fall off as the shaft settles and the jack pressure decreases. After settlement ceases (usually within a few minutes, since the load is allowed to drop off), the final load is read. It is this reduced load which is plotted against the final settlement to obtain points on the load-settlement curve. Very little information on load tests using this procedure is available, but it seems quite promising for future use because a load-settlement curve more nearly approximating that for the ML test can be rapidly obtained.

Pullout tests may be conducted according to the various procedures just mentioned. Because of the difficulty in making a tension connection, pullout tests are hard to perform on drilled shafts, and are not commonly specified. Pullout tests do provide a direct indication of the amount of side resistance that will be mobilized, although the maximum side resistance of a shaft may be different in pullout than in compression.

Once the load test has been completed, it then remains to arrive at a suitable definition of the failure load. Figure 4.3 shows two standard procedures. In the first, extensions of the initial and final straight-line portions of the load-settlement curve are drawn. The load corresponding to the point of intersection is the "failure" load (point A), and the design load is taken to be one-half of that value.

A second procedure is to take the load which, if the load were removed, would produce a permanent set (net settlement) of 0.25 inches (point B). That load is estimated by cycling the applied load in gradually increasing increments or simply by drawing a line parallel to the initial tangent or rebound part of the load-settlement curve.
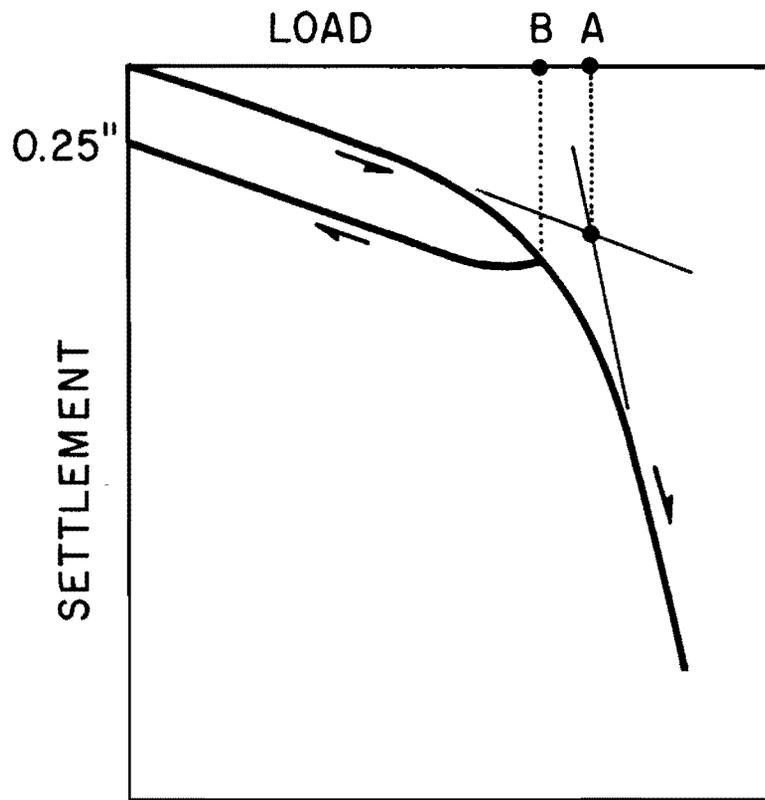
Fig. 4.3. Methods of Estimating Failure Load

Other common methods are to take the load corresponding to some particular slope of the load-settlement curve (Chellis, 1961) or to take the load at which the shaft plunges into the ground, if that mode of failure actually occurs.

Prediction of the Settlement of a Single Drilled Shaft

The prediction of settlement at working load is more difficult than the prediction of load capacity. Fortunately, experience shows that settlement does not control the design of drilled shaft foundations in many cases, in particular when shafts are end-bearing. Various limiting total and differential settlements have been established based on structrual requirements (Sowers, 1962). Generally, for example, reinforced concrete structures should be limited to total settlements of no more than 2 to 4 inches, with differential movements not exceeding 0.003 times the spacing between any two columns which settle differentially.

Shaft settlement consists of three components: initial settlement of the base due to elastic distortion of the soil beneath the base, elastic compression of the stem, and long-term compression (consolidation) of the soil supporting the shaft. (In addition, some additional movement of a shaft may occur because of volume change in expansive soils. This topic is treated briefly later.) The sum of the first two is the immediate butt settlement, and the sum of all three is the ultimate butt settlement. Both immediate (short-term) and ultimate (long-term) settlements should be checked against limiting settlement requirements. In analytical procedures, short-term settlements are computed using dead load only, while long-term settlements are computed using dead load plus live load. Transient live loading produces only minor settlement, if any, in clay, and

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should therefore be excluded from the analysis. For shafts in sand, transient live load should be included, since settlements occur almost instantaneously.

If necessary, shaft sizes are proportioned on the basis of differential settlement restrictions. For example, if individual floating shafts carry different column loads, the stem lengths can be varied to reduce differential settlement. Settlement characteristics of drilled shafts and similar foundation elements already in place have been improved on occasion by cyclic preloading before adding the superstructure (Trollope, Freeman, and Peck, 1966).

Immediate Settlement. Several procedures for determining the immediate settlement of a drilled shaft are detailed below.

Load Tests. Load testing of full-scale shafts, described previously, is the surest means of determining immediate settlements. Whenever severe settlement problems are expected on proposed major structures, such tests should be conducted. Careful judgement should be exercised in extrapolating the results of a single load test at one point on a site to expected shaft behavior at other points, particularly in irregular deposits. For general behavior, that is, for shafts of different geometry, a large number of load-settlement curves would have to be obtained.

Nondimensional Load-Settlement Relationships. A simple empirical procedure based on nondimensional load-settlement curves for the sides and base can be employed, provided such curves have been developed for shafts in the type of soil under consideration. One such group of curves, giving the relative shaft load versus mean shaft settlement and relative base load versus base settlement has been developed experimentally for

drilled shafts in stiff London Clay by Whitaker and Cooke (1966). Some
dependence on shaft diameter was found to exist. The curves for 30-inch-
diameter shafts are given in Fig. 4.4. Similar curves are developed in
Chapter XII of this study for drilled shafts in stiff Beaumont Clay.

To use such graphs, the designer first computes the ultimate side and
base loads according to criteria previously explained. Then, as a first
approximation for obtaining settlement, he may assume the shaft to be
incompressible, so that the base settlement and mean shaft settlement are
the same. By trial and error both curves are entered with several iden-
tical values of settlement, and the corresponding side and base loads
computed from the load ratios are obtained. When the sum of the two
becomes equal to the working load, the corresponding settlement is approx-
imately equal to the desired value of butt settlement. For a better esti-
mate, the elastic compression of the stem may then be computed using the
base and butt loads found in the first approximation, and new (larger)
mean shaft and butt settlements obtained by adding elastic compression
effects to the settlement just found. The side load is then recomputed
using the new mean shaft settlement, and a new base load is found by sub-
tracting the new side load from the applied working load. The corresponding
new value of base settlement is then found from the graph, and the elastic
compression recomputed to give refined mean shaft and butt settlements.
This procedure then continues through as many iterations as required to
obtain the desired accuracy for immediate butt settlement.

Approximate Methods Based on Theory of Elasticity. Settlement of
the base of a drilled shaft at small base loads can be estimated by appealing
to the theory of elasticity. Some judgement must be used in extrapolating
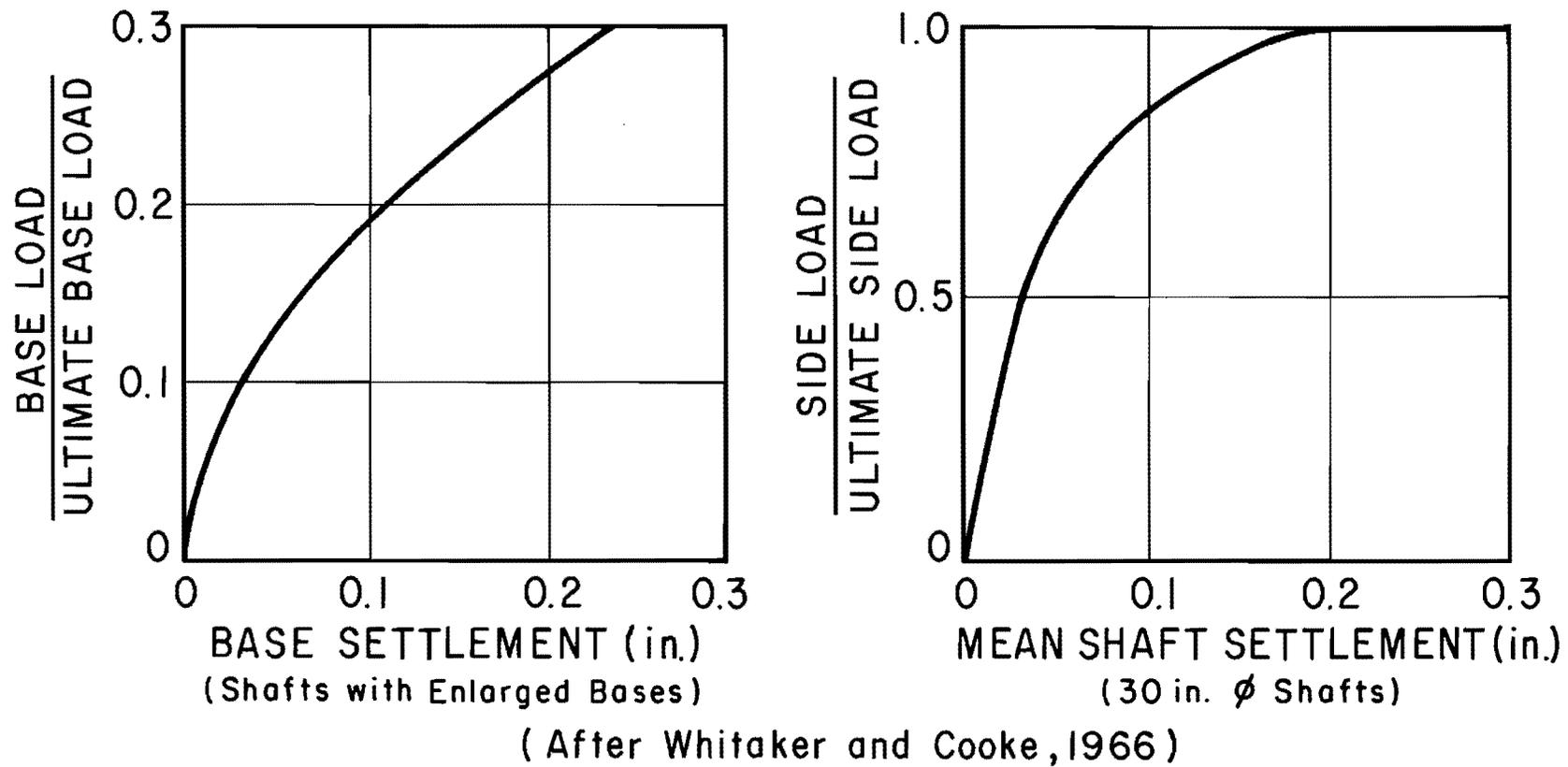
Fig. 4.4. Normalized Side and Base Load-Settlement
Curves for 30-Inch Shafts in London Clay

the results to arrive at butt settlement; however, this method will give
reasonable rough estimates of butt settlement whenever other methods of
settlement prediction cannot be used.

The well-known Boussinesq equations for stress and deformation beneath
a loaded point on the surface of a semi-infinite elastic solid can be used
to estimate the average settlement beneath a uniformly loaded circular area,
such as the base of a drilled shaft. The solution is of the form:

$$\rho_B = \frac{qB \ (1 - \nu^2)}{E_o} I_\rho \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.10)$$

in which

ρ_B = average settlement beneath loaded area

q = contact pressure

B = diameter of loaded area

E_o = Young's modulus of soil

ν = Poisson's ratio of soil

I_ρ = influence coefficient, depending on depth of loaded area

For deep footings, the Boussinesq solution given by Eq. 4.10 is valid
at any depth, provided I_ρ is appropriately adjusted. Young's modulus and
Poisson's ratio are estimated or are obtained from appropriate soil tests.

Skempton (1951) made a useful simplification of Eq. 4.10 for footings
in clay. He first modified Eq. 4.10 to:

$$\rho_B = I_\rho \frac{q}{\left(q_B\right)_{ult}} \frac{\left(q_B\right)_{ult}}{c_u} \frac{3B}{4\left(\dfrac{E_o}{c_u}\right)} \quad \ldots \ldots \ldots \ldots \ldots .(4.10a)$$

by observing that $\nu = 0.5$ for saturated clay in undrained shear, where the quantities c_u and $(q_B)_{ult}$ have the same definitions as have been used previously. Skempton observed that $\dfrac{(q_B)_{ult}}{c_u}$ increases and I_ρ decreases with depth, but that their product remains nearly constant at about 5.35. He also noted from the laboratory stress-strain curve that:

$$\frac{E_o}{c_u} = \frac{\sigma_\Delta}{(\sigma_\Delta)_{failure}} \frac{(\sigma_\Delta)_{failure}}{c_u} \frac{1}{\epsilon} = \frac{2}{\epsilon} \frac{\sigma_\Delta}{(\sigma_\Delta)_{failure}} \quad \ldots \ldots (4.11)$$

in which σ_Δ is the principal stress difference in a triaxial or unconfined compression test.

By making the substitutions for $I_\rho \left[\dfrac{(q_B)_{ult}}{c_u} \right]$ and $\dfrac{E_o}{c_u}$ suggested above in Eq. 4.10a and observing that

$$\frac{\sigma_\Delta}{(\sigma_\Delta)_{failure}} = \frac{q}{(q_B)_{ult}} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.11a)$$

the following simple expression for settlement is obtained:

$$\rho_B = 2B\epsilon \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.10b)$$

Skempton further observed that the strain corresponding to one-half of the principal stress difference at failure, ϵ_{50}, varies from 0.005 to 0.02 for stiff clays, which exhibit generally linear behavior up to that point. This strain value also applies to foundations in which the contact pressure is one-half of the ultimate. Hence, the immediate

settlement for the base of a drilled shaft in stiff clay in which the contact pressure, q , is one-half or less of the ultimate is given by:

$$\rho_B = 4 \frac{Q_B}{(Q_B)_{ult}} B \; \epsilon_{50} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.10c)$$

For example, if the base load is one-fourth of the ultimate computed base load and the diameter of the base is 36 inches, the corresponding settlement is 0.36 inches for an ϵ_{50} of 0.01.

Other investigators have used the theory of elasticity approach to arrive at expressions for ρ_B in clay. Janbu, Bjerrum, and Kjaernsli (1956) presented a method for obtaining settlements in clay when a rigid stratum lies some distance below the base. Lambe and Whitman (1969) describe the use of the stress path method for settlement estimation. Burland, Butler, and Dunican (1966) also give a procedure similar to that given above and quote strain factors for London Clay based on plate loading tests.

For buried cylinders in sands, which are expected to behave like straight drilled shafts, Vesić (1967) noted that:

$$\frac{1 - \nu^2}{E_o} = \frac{1}{\beta(q_B)_{ult}} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.12)$$

in which E_o is the initial Young's modulus of sand in a triaxial compression test with appropriate confining pressure, and β is a settlement correlation coefficient. Experimentally, Vesić observed β to vary from 6 in loose sand (relative density about 0.3) to 9 in dense sand (relative density about 0.8) for load tests on buried cylinders in dry sand. Using

this empirical observation, Vesić (1970) modified Eq. 4.10 for computing

base settlements for buried cylinders in dry or submerged sands as follows:

$$\rho_B = \frac{0.18 \ Q_B}{\left(1+D_r^{\ 2}\right) B \left(q_B\right)_{ult}} = \frac{0.14}{\left(1+D_r^{\ 2}\right)} \frac{Q_B}{\left(Q_B\right)_{ult}} B \ . \ . \ . \ . \ . \ . \ . \ . \ .(4.10d)$$

in which all quantities are as defined earlier. Equation 4.10d holds for

small displacements only, perhaps those corresponding to one-third of the

maximum base load or less.

For example, for a 36-inch diameter straight shaft in a dense sand,

D_r = 0.7 , the settlement at one-tenth of the base failure load is 0.34

inches.

Equation 4.10d is quite approximate and is somewhat difficult to

employ in practice because of the uncertainties in calculating $(Q_B)_{ult}$.

Furthermore, even approximate validity has not been established for belled

shafts in sand. The equation is probably not at all valid for shafts

installed in stabilized soils.

Some information concerning base settlement in sands can be obtained

from penetrometer tests. Very approximate estimates can be made by

knowing standard penetrometer readings and bearing pressures and inferring

corresponding settlements from charts (Peck, Hanson, and Thornburn, 1953,

p. 225) relating blows per foot, footing width, and contact pressure at

some known value of settlement.

In order to use Eq. 4.10c (base in clay) or Eq. 4.10d (base in sand)

to compute immediate butt settlement, the relative distribution of applied

load to stem and base must first be estimated. This estimate is usually

based on the designer's experience with load testing, his insight into the

behavior of drilled shafts, or it may represent a "worst case," such as all load going to the base. For shafts in uniform deposits of clay, the following percentages of maximum side capacity mobilized at working load (40 to 50 per cent of total ultimate capacity) have been observed in load tests and may be used to obtain very approximate settlement estimates in the absence of other information:

a. Short belled shaft   100 per cent
   (3-to-1 bell)

b. Long belled shaft   60 to 90 per cent depending
   (3-to-1 bell, bell   upon depth of bell
   diameter < 4 to 6
   times depth of base)

c. Long straight shaft   Assume base load 5 to 10 per cent
                               of applied load for length-to-
                               diameter ratio in range of 10 to 20.

Immediate settlement at working load will be most important in short shafts with enlarged bases, while it will likely be quite small for long, straight shafts. Fortunately, the percentages just quoted are more accurate for short shafts, since nearly all of the shear strength of the soil will have been mobilized around the stem at the point at which enough load has been taken by the base to provide the overall factor of safety of 2 to 3. If the stem capacity, $(Q_S)_{ult}$, is 100 tons, and the base capacity, $(Q_B)_{ult}$, is 200 tons for a short belled shaft, the net base load at a working load of one-half ultimate (150 tons) will be 150 - 100 = 50 tons. The base settlement is then computed using Q_B = 50 tons in the appropriate version of Eq. 4.10, and the elastic stem compression is added to obtain the total immediate butt settlement.

Stem compression may be computed from the following formula, assuming a linear distribution of load in the stem:

$$\delta_S = \frac{Q_T + Q_B}{2A_c E_c} L_S \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.13)$$

in which

δ_S = elastic compression of stem

Q_T = applied, or butt, load

L_S = length of stem

A_c = transformed cross-sectional area of stem (including effects of reinforcing steel)

E_c = Young's modulus of concrete in stem

Analytical Methods for Synthesis of Complete Behavior. One or more of the analytical procedures described in the previous chapter for synthesizing complete behavior may be employed, provided a computer of adequate size, the necessary computer programs, and appropriate soil information are available. Such methods are, of course, less approximate than the simpler hand procedures outlined under the preceding two headings, but they are usually justified only on major jobs. Such procedures are particularly useful for obtaining shaft capacities and settlements in stratified deposits, for which the parameters used in simple hand methods must be obtained with a considerable amount of guesswork.

Long-term Settlement. The long-term, or consolidation, settlement of drilled shafts may exceed the immediate settlement (in the case of floating shafts in clay) or may be insignificant (in the cases of shafts in sand

or shafts bearing on hard rock). It is possible to determine long-term settlement by means of load tests, but this method is uneconomical for design purposes because of the excessive time required. Instead, if the designer feels that long-term settlement is a matter of concern, estimates are usually based on the one-dimensional theory of consolidation (Taylor, 1948). The amount of consolidation, or compression, which a layer of soil beneath the base of the shaft ultimately undergoes is given by:

$$\rho_c = \frac{C'}{1 + e_o} H \log_{10} \frac{p_o + \Delta p}{p_o} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.14)$$

in which

ρ_c = total compression of the layer

H = thickness of the compressible layer

p_o = initial effective vertical pressure at the center of the layer

Δp = increment of applied pressure causing consolidation

C' = mean slope of e-log p curve between p_o and $p_o + \Delta p$ (equal to compression index in normally consolidated clays); specifically, change in void ratio per log cycle of pressure

e_o = void ratio of soil under pressure p_o

Equation 4.14 can be applied to the determination of long-term settlement for floating shafts with adequate accuracy as follows (consult Fig. 4.5):

1. Estimate the distribution of load between base and sides as in computations for immediate settlements.

2. Assume the side load Q_s is applied to the soil uniformly
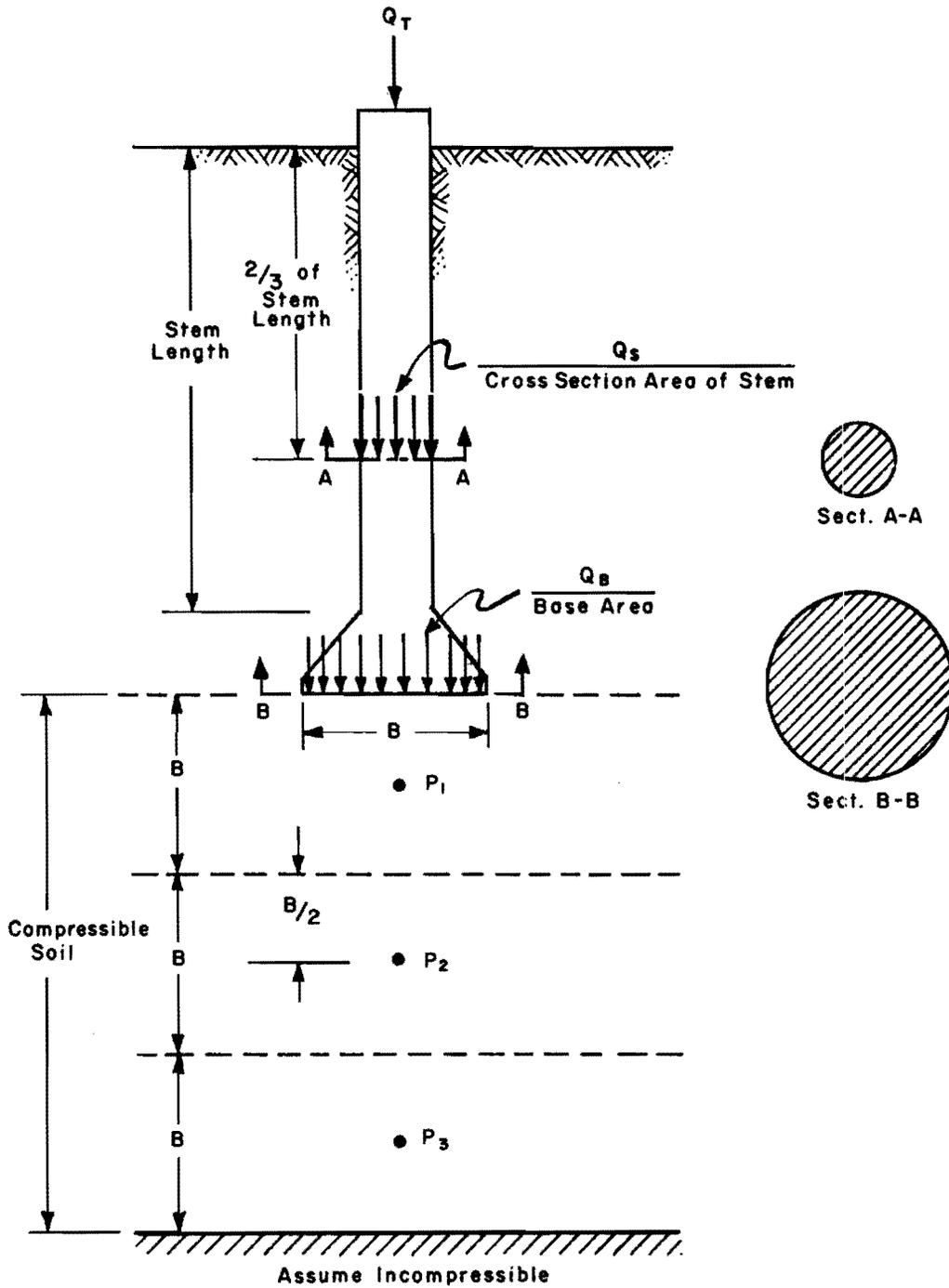
Fig. 4.5. Idealization of Shaft-Soil System
for Settlement Computations

over an imaginary footing equal in size to a cross-section

of the stem at a distance of two-thirds the stem length

below the ground surface. Assume the base load Q_B is

applied uniformly over the area of the base. Also assume

that no consolidation occurs between the level of applica-

tion of Q_S and that of Q_B.

3. Subdivide the soil below the base into three imaginary

layers, each of which has a thickness H equal to the

base diameter B. This procedure is based on the tacit

assumption that all consolidation occurs within a depth

of three diameters below the base.

4. Compute the initial effective vertical pressures, p_o ,

at points P_1 , P_2 , and P_3 , which lie directly

beneath the center of the base and are respectively at

the centers of each of the three compressible layers.

These pressures are usually taken as the product of the

effective soil unit weight and the depth to the point

in question.

5. Compute the pressure increments, Δp , at points P_1 ,

P_2 , and P_3 , due to the applied loads Q_S and Q_B.

Influence charts for stress, such as that devised by

Newmark for Boussinesq's equation for vertical stress

beneath a loaded area, may be used in such computations

(Peck, Hanson, and Thornburn, 1953), or the 2:1 stress

spreading approximation alternately may be employed (Sowers

and Sowers, 1961).

6. For each layer, select appropriate values of C' and e_o from laboratory consolidation test data.

7. Calculate the ultimate compression of each of the three layers by Eq. 4.14, and add the results to obtain long-term settlement.

8. To obtain total settlement, add value obtained in Step 7 to immediate butt settlement.

This procedure is only approximate. It is based on the assumptions that load in the stem decreases linearly and that neither side nor base changes in magnitude during consolidation, among others.

Consolidation settlement of drilled shafts in overconsolidated soils is usually small at working load. However, the method just outlined will probably yield computed settlements which are too high because slopes of laboratory e-log p curves in the range of pressures being considered are materially increased by pressure release caused by sampling.

A better estimate of settlement can be made for overconsolidated clays by performing laboratory tests on undisturbed samples in which the specimen is first consolidated to the estimated preconsolidation pressure and then rebounded to p_o (Leonards, 1962). The specimen is then consolidated in increments as in the standard test. The e-log p relationship obtained from the second loading should then be used in computations. A typical e-log p relationship for such a test is shown in Fig. 4.6. The quantity e_o' should be used in place of e_o in Eq. 4.14.

If the soil beneath the base is stratified, it is appropriate to take the boundaries of the imaginary compressible layers to be the boundaries
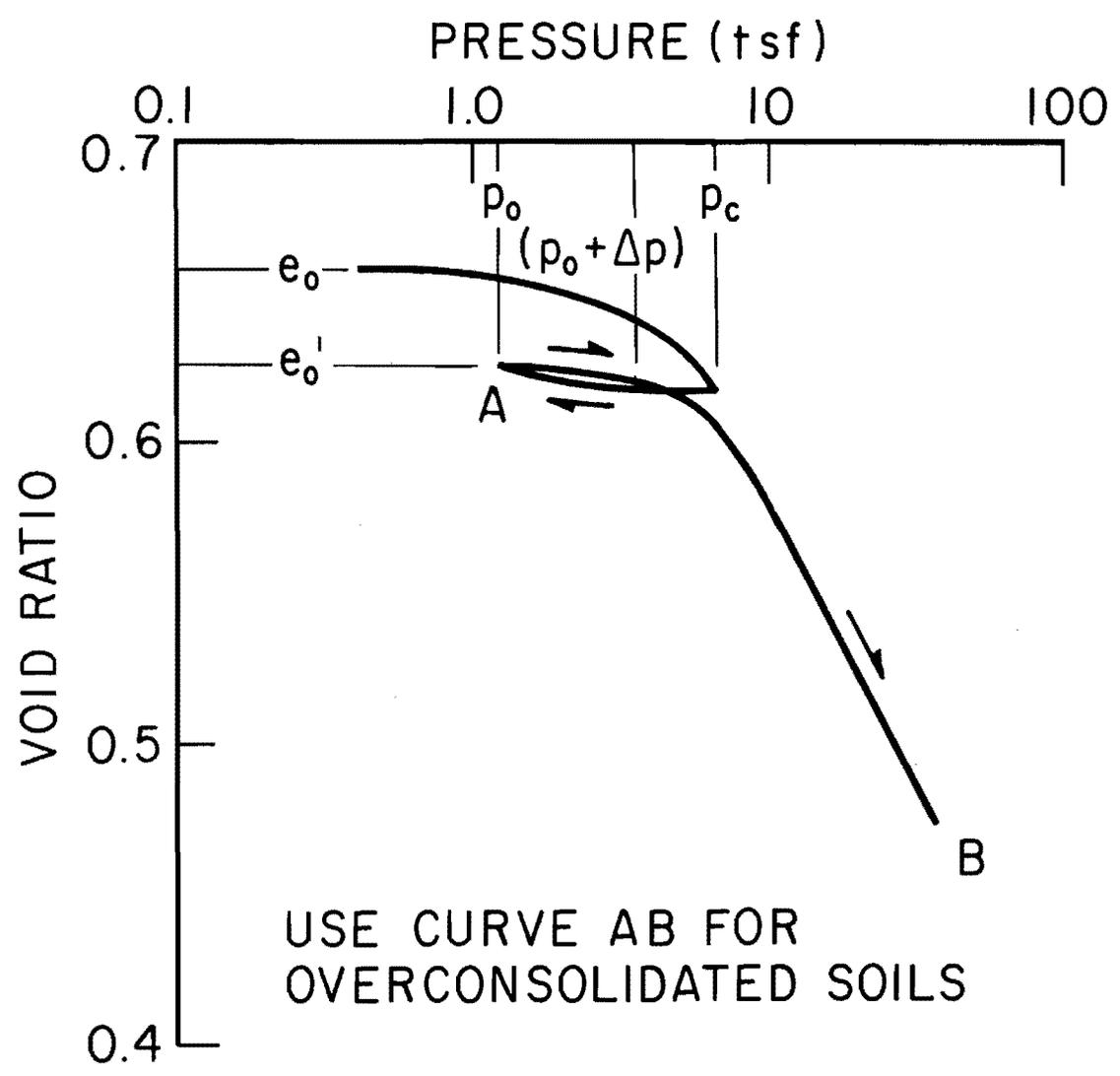
Fig. 4.6. Cyclic e-log p Curve for Use in Computing
Settlements in Overconsolidated Clays

of the natural soil strata, with subdivisions within any stratum thicker than the base diameter.

More accurate settlement computations can be made by using smaller subdivisions of the compressible soil beneath the footing. In addition, more rational procedures for predicting normal stresses at points below pile or drilled shaft bases are available (Geddes, 1966; Geddes, 1969), in which only Q_S , Q_B , and the pattern of shear stress distribution along the sides of the stem need be estimated. Influence charts for stress distributions around and beneath piles have been constructed (Lysmer and Duncan, 1969). These charts are based on idealized distributions of side shear.

The rate of settlement can also be roughly forecast based on the one-dimensional theory of consolidation (Peck, Hanson, and Thornburn, 1953; Taylor, 1948).

Design of Drilled Shafts in Expansive Soils

In areas of highly expansive soils, drilled shafts are commonly carried through the zone of expansion and belled or socketed in a non-expanding stratum. The bell or socket provides an anchor for uplift forces created by upward-directed side shear when the soil expands. For design purposes, under the worst conditions of expansion, it is usually assumed that the entire shear strength of the soil will be mobilized in upward side shear, giving the following equation for tensile force T_z in the shaft at any depth (Collins, 1953):

$$T_z = \frac{\pi}{2} \left(d_{stem} \right) (2c'z + K_o \gamma' z^2 \tan \phi') \quad \ldots \ldots \ldots \ldots \ldots (4.15)$$

in which

z = distance below top of expansive layer

d_{stem} = diameter of stem of drilled shaft,

and all other terms are as previously defined.

Collins suggested a value of K_o between 1 and 2 for overconsolidated clays. In designing a shaft in such soils, the ultimate load and immediate settlement are computed using procedures for nonexpansive soils previously described, since the ultimate bearing value will not be greatly altered by expansion of the soil, and elastic settlement will occur before uplift. Consolidation settlement may very likely be nonexistent. Forces in the stem should be checked at working load by computing T_z from Eq. 4.15 and decreasing that value by the amount of compressive working load applied at the top of the shaft. The working load is considered to be transmitted wholly to the depth in question for conditions of maximum uplift. The critical section of the stem will be at the top of the bell or the bottom of the expansive stratum. The net tension force (T_z minus working load) must then be resisted by providing a sufficient area of reinforcing steel, which might be as great as four to five per cent of the cross-sectional area of the stem.

If enough reinforcement against tension is provided, and the bell is properly anchored into a nonexpansive stratum, heave at the butt will usually not be a problem. However, the designer must be aware of the possibility that, under certain circumstances, placing the bell in an expansive, although strong, material can still result in butt heave, which may be detrimental to the structure. An example of such a situation is

a drilled shaft which passes through expansive overburden soils and is terminated in clay-shale that is below the zone of seasonal change yet deficient in moisture. The process of installation may open channels of water supply into the water-deficient stratum, causing subsequent soil expansion and heave of the base and, consequently, the butt. The amount of heave is variable and is difficult to predict. Serious structural distress has been reported due to this phenomenon (U.S. Army Engineer District, Fort Worth, Texas, 1968). Upward-directed shear stresses as high as 75 per cent of the shear strength of the soil have been measured along the sides of experimental drilled shafts placed in initially moisture-deficient clay-shales after introduction of moisture (U.S. Army Engineer District, Fort Worth, Texas, 1968).

In the case of a shaft with the bell or socket in nonexpansive soils, the use of bond-breaking material, such as Vermiculite, between the concrete and soil along the periphery of the stem may be desirable because tension in the shaft may be significantly reduced. But when the base is in a moisture-deficient stratum, bond-breakers merely provide a direct path for water to be transmitted to the soil beneath the base, resulting in heave. It appears that, under such conditions, other types of foundation systems may be more desirable than drilled shafts for structures not flexible enough to withstand considerable differential heave (U.S. Army Engineer District, Fort Worth, Texas, 1968).

Negative Side Resistance

Drilled shafts are occasionally installed through consolidating fills. Under such circumstances, negative side resistance, or downdrag, occurs. Downdrag is brought about as consolidating soils move downward with respect

to the shaft. Downdrag forces in the shaft must be added to the applied load to determine the actual load present at any depth for purposes of long-term bearing capacity and settlement analysis and should be considered as part of the applied load when computing a factor of safety. Procedures for estimating downdrag forces on driven piles are given by Tomlinson (1969). It is assumed that these procedures also apply to drilled shafts. Downdrag can be reduced by bond-breaking techniques such as that described in conjunction with design of drilled shafts in expansive soils.

There is a potential for negative side resistance in soils which shrink due to loss of moisture. However, since shrinkage implies an outward as well as a downward movement of the soil relative to the shaft, negative side resistance can usually be discounted under these circumstances.

Lateral Load

The permissible lateral load acting against the top of a drilled shaft is normally taken as some small arbitrary value when the drilled shaft is designed primarily as an axial-load-carrying element. A typical range of values is five to ten kips. Whenever the foundation must sustain large lateral loads, battered shafts or piles are generally provided. Few experimental and analytical studies have been reported concerning behavior of drilled shafts under lateral loading or combined lateral and axial loading. One recent paper (Davisson and Salley, 1969) reports results of lateral load tests on full-scale shafts, with vertical reinforcement on the order of 0.9 to 2.6 per cent, carried through granular overburden into shale bedrock. A cycled load of 100 kips produced deflections of less than 0.3 inches in four-foot diameter shafts, both socketed and belled in the shale. Embedded shaft lengths varied from 14 to 45 feet.

The results of these tests cannot be extrapolated numerically to other soil conditions, but they do indicate that drilled shafts are capable of resisting high lateral loads.

It is also uncertain whether criteria developed for predicting loads and deflections of laterally-loaded driven piles composed of elastic material (for example, Matlock and Reese, 1962) apply to drilled shafts. More experimental data need to be obtained before general design guides for lateral load behavior can be developed. Meanwhile, lateral load tests on proposed construction sites will provide the most realistic information concerning capacities and deflections under lateral loads, if such information is necessary for design.

Uplift Capacity

For straight shafts in clay, it is appropriate to equate the uplift capacity with the maximum side friction expected in compression. In sands, the capacity is probably somewhat less in uplift than in compression. No rational guidance is available at present concerning the exact amount of reduction to be expected. For belled shafts in sand, clay, and rock, the design is complicated by the anchoring action of the bell. A general uplift theory, appropriate for design application, has been recently presented by Meyerhof and Adams (1968).

Concrete Deterioration

If a drilled shaft is to be installed in a soil high in sulfate content, consideration should be given to methods for assuring that concrete deterioration due to sulfate attack at the soil-concrete interface is minimized. If the shaft is installed entirely in an impermeable soil with high sulfate

content, but with no channels permitting general groundwater movement to occur, deterioration will be minor (Neville, 1963). On the other hand, if a drilled shaft is located in a permeable sand in which the groundwater is high in sulfates and tends to flow appreciably, sulfate action can lead to friable or soft concrete.

As a rule of thumb, whenever sulfate concentration exceeds 0.2 per cent as water-soluble sulfates in the soil, or is greater than 1000 parts per million in moving groundwater, sulfate resistant cement (Type V) should be used in the concrete (U.S. Army Corps of Engineers, 1965).

Little is known about the effects of sulfate attack or of using sulfate resistant cement on the amount of side resistance which can be mobilized. Green (1961) quoted the observations of W. H. Ward of the Building Research Station, England, who uncovered and examined a shaft, made with sulfate resistant cement, located in a clay soil containing calcium sulfate about one year after casting. Ward noted that components of the concrete had evidently diffused into the clay for a distance of about one-half inch, causing the soil in that zone to become faded in color, more brittle, and harder.

Behavior of Groups of Axially Loaded Drilled Shafts

Although sizing of drilled shafts in a foundation system is usually determined by considering the capacity and settlement of all shafts to be identical to those for single, isolated shafts, the final design must involve some consideration of the interaction of the various shafts in the system. This consideration may be nothing more than observing from experience that group action is unimportant for widely spaced shafts

supporting a particular type of structure in a given soil formation and
need not be taken into account in design. It may, however, involve some
procedure for reduction of allowable loads from the values calculated
for isolated shafts, together with a numerical estimation of increases in
total and differential settlement for the shafts in the group.

Very little published information exists describing group action in
drilled shafts, particularly those groups containing shafts with enlarged
bases. It must be assumed, at least for the present, that methods that
have been developed for predicting group behavior and computing reduction
of allowable loads in driven piling also apply to drilled shafts, with
the exception that influences of installation methods and of enlargement
of bases must be somehow introduced. Because of this paucity of informa-
tion, no specific recommendations are made herein for forecasting group
behavior in drilled shaft foundations. However, the following short
review, principally based on driven pile behavior, may provide some
guidance for selection of design criteria for drilled shaft groups.

Two terms often encountered in discussions of group behavior are
"efficiency" and "settlement ratio." The efficiency of a group of piles
or drilled shafts is defined as the ratio of the ultimate capacity of the
group to the sum of the ultimate capacities of the individual elements
acting as isolated units. The settlement ratio is defined as the ratio
of the settlement of the group at a certain percentage of ultimate capa-
city to that of a single element at the same percentage of its ultimate
capacity. It will be convenient to use these two terms in the following
presentation.

Drilled shaft groups may be broadly classified under two categories: those in which individual shafts are connected at the top by a rigid cap (approximately equal settlement in all shafts; loads vary) and those in which individual shafts are connected by a flexible cap (each shaft considered to be loaded independently; settlements vary). The characteristics of behavior of the two types of groups are somewhat different; therefore, group action descriptions are given separately herein for each category.

Group With Rigid Cap. When groups of drilled shafts must be rigidly capped, an estimation of allowable load reduction on each shaft may be required. Virtually the only design guidance in this area comes from theoretical considerations or from tests of groups of small-sized jacked or driven piles.

The easiest way to determine the average load reduction on a single shaft in a group due to group action for design purposes (although hardly the most rational) is to use efficiency formulas such as the Converse-Labarre formula or Feld's rule (Moorhouse and Sheehan, 1968). Efficiency formulas always indicate that allowable loads should be reduced over those for isolated piles or shafts. They will give best results for groups of floating shafts in clay, but are at best highly approximate.

Groups in Sand. When driven piles are installed in sand, an increase in efficiency may be observed due to increased horizontal stresses against the piles caused by driving (Vesić, 1969). However, with drilled shafts in sand, actual loosening of the soil may occur as shafts are installed adjacent to those already in place. This action may be minimized if individual shafts are bored and concreted in one operation, instead of

boring several shafts before starting to place concrete. Experiments with pairs of small-diameter straight drilled shafts in sand by Press in Germany (quoted by Vesić, 1969) have indicated that efficiencies may be as low as 0.6 at spacings of three diameters. The methods of installation and testing were not mentioned.

Tomlinson (1969) suggests that drilled shafts not be installed in sand at a spacing closer than 2 feet 6 inches or twice the smallest diameter, whichever is least, with the design efficiency taken as unity.

Even if a small or nonexistent load reduction (near 100 per cent efficiency) is expected for a group of drilled shafts in sand, the settlement of the group at working load will be increased over that of a single shaft because of the overlapping and deepened stress fields mentioned earlier. As with single shafts, that added settlement is immediate and is likely to be small when the shafts are truly end-bearing. The settlement of a floating square group in sand can be roughly compared to the settlement of a single shaft in the group, at comparable degrees of ultimate load mobilization, by the following relationship presented by Vesić (1969):

$$\xi = \sqrt{\frac{\bar{B}}{B}} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.16)$$

in which

ξ = settlement ratio

$\bar{B}$ = width of group

B = diameter of single shaft

It is unclear whether the base or stem diameter should be used for B in Eq. 4.16 for belled shafts. It seems appropriate to take some intermediate value, since part of the load is resisted by side friction and part by base bearing at working load. If one of the two modes of resistance is expected to dominate at working load, the diameter corresponding to that mode may be appropriate.

Groups in Clay. Some insight into the behavior of groups of driven piles or drilled shafts in clay is available through reported results of model tests. Whitaker (1957) determined efficiencies and settlement ratios (for immediate settlements) for square groups of model piles in soft clay with various spacings and numbers of piles. He observed that "block" failure (shear failure around the periphery of the group) occurred when the spacing was 1.5 diameters for shallow piles in small groups to 2.2 diameters for deep piles in large groups. When the spacing was large enough to prevent block failure, group efficiencies varied from about 0.65 (impending block failure) for all groups to 1.0 (widely spaced piles). Whenever block failure occurred, efficiencies were observed to drop very sharply. Results of increased immediate settlement due to group action were presented graphically by Whitaker. As would be expected, the settlement ratios increased with increasing group size, increasing pile length, and decreasing spacing. As an example, for a three-by-three group in which the pile lengths were 48 times the diameters, the settlement ratio at failure was about 4 for a spacing of three diameters. Under the same conditions, the settlement ratio for a five-by-five group was about 9. Settlement ratios at working load were about the

same as at failure for small groups, but were considerably less than those at failure in larger groups.

Short-term tests of full-sized groups of driven piles in soft and medium clay have been reported (Schlitt, 1952; American Railway Engineering Association, 1950). Schlitt tested a three-by-three group of 12-inch-diameter Monotube piles on a 3.75-diameter spacing, with the result that the efficiency was about 0.9, and the settlement ratio at failure was near 1.75.

The A.R.E.A. tests on a three-by-three group of instrumented steel Monotube piles spaced at three diameters showed that the centroid of the side shear-resistance-versus-depth diagram occurs at lower levels for piles in a group than in a single pile. This fact can be attributed to the additional downward displacement of the mass of soil around the sides of the piles inside the group. This added downward soil movement is greater near the ground surface at working load, thereby inhibiting development of resisting shear stresses. Therefore, the applied load must be resisted at a lower level in the piles, which implies that a larger percentage of load is carried by the base for piles in a group than for single piles at comparable applied loads. This fact has been verified analytically (Poulos, 1968). The settlement ratio in the A.R.E.A. tests was on the order of three to four. Efficiencies were not reported, but they appear to be about 0.7.

The caps in both the Schlitt tests and A.R.E.A. tests were probably not completely rigid. A fairly uniform distribution of load to the various piles in the group was observed.

A simple method for determining group efficiency is outlined by Peck, Hanson, and Thornburn (1953). For a given trial design, the capacity of the group is computed first by summing the capacities of the individual elements (with no load reduction) and then by calculating the capacity of the block (shear around perimeter of group plus bearing capacity of horizontal gross area at the base bounded by the perimeter of the group). The smaller of the two is taken to be the design capacity. Although actual block failure may not occur at a spacing for which computed block capacity is slightly less than the sum of individual element capacities, the method, in effect, reduces the efficiencies of the elements for that spacing. This method would seem to be reasonable for groups of drilled shafts, especially those with elements having enlarged bases.

Kerisel (1967) gives a table of load reduction coefficients for groups of driven piles in clay, based simply on spacing. Typical values for reduction factors are 1.0 for a spacing of 10 or more diameters, 0.9 for 6 diameters, 0.75 for 4 diameters, and 0.55 for 2.5 diameters. When Kerisel's factors are extrapolated to drilled shaft foundations, the diameter to which the spacing is referred is again appropriately taken as a value intermediate between the stem diameter and base diameter.

The determination of the long-term settlement of a group of drilled shafts with a rigid cap may proceed in a manner similar to that for a single shaft, except that an equivalent pier concept is used (Sowers and Sowers, 1961). The group is considered to be replaced by a prismatic pier with a cross section identical in shape and area to the gross cross section of the group at corresponding depths. Distributions of applied load to sides and bases of shafts in the group at working load is assumed

to be the same as in the individual shafts at working load, although, as
mentioned previously, the base probably takes a somewhat greater propor-
tion when shafts are in groups. The part of the group load resisted
along the stems is applied uniformly over the cross-sectional area of the
equivalent pier at the same relative depth (bottom third point of the stem)
as for single shafts. The base loads are applied uniformly over the base
of the equivalent pier. The settlement computations then proceed as before,
with B being taken as the minimum dimension of the group. This proce-
dure is approximate but will provide a means of assessing whether a group
will experience excessive settlement due to consolidation.

Groups with rigid caps are less common in drilled shaft foundations
than in foundation systems consisting of driven piles. Rigid caps are
usually provided to tie a cluster of piles together to enable the piles
to carry the load from a large column or pier. The capacity of a single
large drilled shaft, however, may be equal to that of several driven piles;
hence, it is not necessary to install more than one element to carry the
load of a single column when the magnitude of the load is not extremely
great.

Group With Flexible Cap. A common example of a group of drilled shafts
with a flexible cap is a bridge bent in which the columns are extensions
of the foundation elements and are connected through a flexible reinforced
concrete beam across their tops. In designing a group of drilled shafts
with a flexible cap, the load reduction due to group action can be computed
as for groups with rigid caps. However, since the individual elements can
settle differentially, it is inappropriate to compute group settlement
ratios. Instead, the individual shaft settlements should be estimated.

One approximate method of estimating additional immediate settlement of a shaft at working load due to group action is to duplicate the procedure for finding normal stresses at various distances below the base, described in conjunction with long-term settlements of single shafts, except that the zone of influence is deepened to at least two or three times the minimum dimension of the group. Additional normal stresses, at the same points, contributed by surrounding shafts are then computed again using stress influence charts or by the equations given by Geddes (1966, 1969). The ratio of the sum of the additional stresses to the sum of the original stresses is assumed to be the ratio of the settlement of the shaft as it exists in a group to its settlement as an isolated element. This method is highly approximate, and is not at all rational because the increase in shear stress (distortion), not the increase in normal stress, causes most of the immediate settlement.

Long-term settlements in clay are computed by the same procedure. That is, pressure increases at points below the center of each shaft are computed considering the influence of surrounding shafts, and the consolidation settlement in each layer is found by employing Eq. 4.14 as in the analysis of compression under single shafts.

Increases in immediate settlement due to the presence of nearby shafts can be obtained from an extension of the procedures which employ Mindlin's solution (Poulos, 1968). Figure 4.7 gives the relationship between length, spacing, and settlement increase obtained by this approach for two rigid elements in an elastic medium carrying identical loads. The settlement interaction factor, β , is the ratio of the increase in immediate settlement to the original immediate settlement for one element
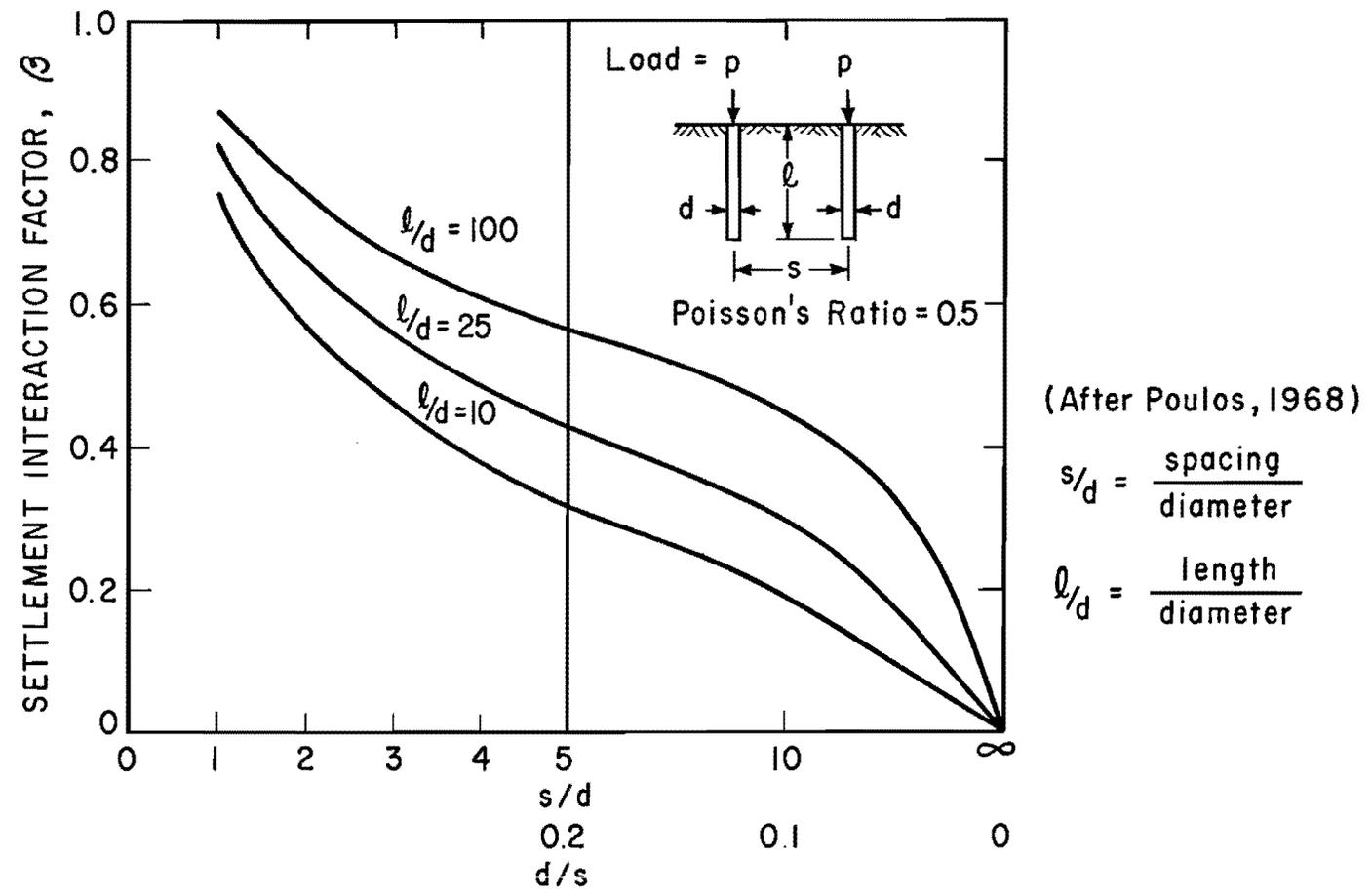
Fig. 4.7. Settlement Interaction Between Two Shafts
or Piles in a Semi-Infinite, Elastic Mass

due to the presence of the other. This graph can be used for any number
of elements by employing the principle of superposition. For example,
for three straight shafts three feet in diameter and 75 feet deep placed
in a line and spaced 15 feet on centers, the β factors for the outside
shafts would be 0.43 (influence of center shaft) plus 0.30 (influence of
other outside shaft), or 0.73. In other words, the outside shafts would
settle 73 per cent more in the three-shaft group (at working load) than
they would if loaded individually. The center shaft would settle 86 per
cent more in the group than it would singly, and the differential settle-
ment becomes equal to 13 per cent of the initial immediate settlement.
This method is not appropriate for determining long-term settlement
because the principle of superposition would not apply. Barden and
Monckton (1970) have verified Poulos' method experimentally for model
piles in stiff clay.

Other Considerations. Other factors which need to be considered in
a particular design are the amount of added load which can be allowed on
the group if the cap is in contact with the soil (Vesić, 1969) and the
distribution of loads to the various elements in case the group is
unsymmetric or is loaded eccentrically. Normally, simple structural
analysis methods are used to estimate this distribution if the geometry
is relatively simple. Numerical procedures for obtaining distribution of
load to elements in a group with complicated geometry and loading have
been presented recently (Aschenbrenner, 1967; Saul, 1968; Nair, Gray,
and Donovan, 1969; Reese, O'Neill, and Smith, 1970). These procedures
are rational, but require some knowledge of the axial and lateral load-
deflection relationships for each element in the group.

CHAPTER V

PREVIOUS FIELD STUDIES

In the early 1950's large diameter drilled shafts came into general
use as foundation elements for heavy structures. One primary focus of
drilled shaft construction was the stiff, overconsolidated, heavily-
fissured, London Clay in England. Drilled shafts in the London area have
been designed to behave as combined friction piles and deep footings,
since hard bearing strata are quite deep. It was correctly postulated
quite early that the load-settlement behavior of the base and sides were
different, with the sides mobilizing maximum shear at very small movement,
while settlements of several inches might be required to mobilize completely
the base reaction. Hence, under small loads, most of the applied load
would be carried in side shear in all but the shortest underreamed shafts.
Designers in London took advantage of this fact to control settlement by
specifying deep shafts, both straight and belled, whenever possible.

Until about 1950 very little knowledge had been acquired concerning
the behavior of floating drilled shafts, particularly with respect to the
amount of load resisted by side friction. It became evident to the London
designers, as well as to foundation engineers in other areas, that insuf-
ficient information on side-shear action was available to allow rational
methods to be applied to the design of deep, floating drilled shafts.
Therefore, during the years that followed, up to and including the present
(1970), numerous field load tests on large diameter, floating drilled
shafts were conducted in various locations to furnish design guides. The

preponderance of tests reported in the technical literature have been in London Clay. Some of the reported results provide useful information concerning side shear behavior, and a few give base load-settlement relationships.

Results of many of the field tests conducted throughout the world between 1950 and 1970 are summarized in some detail in Table 5.1. A number of reported test results reviewed as background for the study reported herein were omitted from this tabulation because of incomplete soil data. Most of the tests tabulated were conducted in stiff clay on shafts installed without the use of drilling mud. The results generally were obtained from short-term load tests. Further information is available in an annotated list of reports of other proof tests, model studies, and descriptions of construction projects employing drilled shafts, compiled by the Texas Transportation Institute (1965).

Several important factors expected to influence reported results are included in Table 5.1. Among these factors are method of determining soil strength, soil description, construction procedure, method of conducting field tests, and dimensions of test shafts. Other pertinent information such as position of anchor piles, time between casting and testing, and estimated reliability of load and settlement measurement devices is not generally available in the literature and is not included in the table.

While the earliest tests were conducted on uninstrumented shafts that could not provide a differentiation between side and base loads, valuable information was nevertheless recovered concerning the ultimate side shear capacity. Early investigators measured the failure load, and then calculated the load on the base by using an appropriate bearing capacity equation

TABLE 5.I. SUMMARY OF FULL SCALE LOAD TEST RESULTS

INVESTIGATOR (S) & REFERENCE		Meyerhof & Murdock (1953)	Golder & Leonard (1954)	Harris (1951)	Green (1961)	DuBose (1955, 1956)
LOCATION		Southall and Barnet, London Area, England	Kensal Green, London Area, England	Plum Creek, Houston, Texas	Borehamwood, Hartfordshire, England	College Station, Texas
DATE OF TEST (S)		1950 – 1952	1950 – 1951	1951	1952 – 1955	1953 – 1955
TESTING METHOD		Quick. Shafts Loaded to Failure within Several Hours.	Cyclic with Load Maintained until Equilibrium Reached in Each Cycle. Tests Required 3 Days per Shaft.	Cyclic. Load Maintained Several Days Each Cycle.	Static, Long-term. Design Load Maintained 3 Years, 8 Months. Also Short-term M.L.	Maintained Load Method. Most Tests Required 4-5 Days. Also a Few Pullout and Quick Tests.
SOIL DESCRIPTION		London Clay. Preconsolidated, Fissured CH Material. No Ground Water. Shear Strength Varies from 0.5 tsf at 5' to 2.3 tsf at 25'-30'. Constant Below 30'.	London Clay. Typical. Shear Strength Varies from 0.4 tsf at Top of Clay to 1.5 tsf at 30' Depth.	Beaumont Clay. Firm Yellow-Blue jointed Clay Interbedded with Waterbearing Silts. Strength not Explicitly Stated but about 1.0 tsf Indicated.	London Clay. Typical. Shear Strength about 1.1 tsf in Test Zone as Measured in Unconfined Compression Tests.	Layered Sandy CL and CH Material. Water-bearing Silt at 12'. Avg. Shear Strength of 0.5 tsf along Sides.
NUMBER AND SIZE OF SHAFTS	STRAIGHT	11: 12"-14"∅, 20'-40' Depth.	3: 18"-24"∅, 23'-34.6' Depth.	1: 20"∅, 44' Depth.	8: 12"-14"∅, 10' Depth.	30: 6"-23"∅, 6'-21' Effective Depth.
	BELLED	0	0	0	0	5: 15"∅ Stem, 3'∅ Bell, 10'-19' Effective Depth.
CONSTRUCTION PROCEDURE		Boreholes Drilled with Both Derrick-type Hand Augers and Lorry-mounted Power Augers. Dry Process. Required 1-5 Days to Install Each Shaft with Hand Augers. 45 Minutes - 4 Hours with Power Augers.	Presumably with Power Augers. Plate Loading Tests Conducted at Several Levels in Boreholes. Hence, Holes Presumably Open (but Cased) for Periods of Time Longer than Normal.	Power Auger with Casing. No Mud. Sides of Hole Wet. Considerable Sloughing. 4' Water in Bottom of Borehole. Installation Presumably Completed in One Day.	Mechanical Auger to 8'. No Mud Used. Hand Auger to Finished Depth (10'). Three Shafts Cast in Wet Holes.	All 6" and 7" Shafts Hand Augered. Others Installed with Power Augers. Dry Process in Each Case.
INSTRUMENTED ? IF SO, HOW ?		No. Side Resistance Deduced from Bearing Capacity Equation.	No. Side Resistance Deduced from Bearing Capacity Equation.	No. Side Resistance Deduced from Bearing Capacity Equation.	No.	Yes. Load Cells Used at Base of 6 Straight Shafts. SR-4 Gages Used on Re Bars of Several Shafts. Some Stability Problems with SR-4 Gages on Re Bars
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		0.21-0.42 at Southall. Water-Cement Ratio = 0.4. 0.4-0.6 at Barnet. Water-Cement Ratio = 0.2. (Average of UU Trioxial and Unconfined Tests)	0.64-0.74 as referred to Average Soil Strength. 1.0 as Referred to Envelope of Minimum Soil Strengths. (Triaxial Compression Tests)	0.6 at 1/4" Gross Settlement Computed by Harris. Probably High, but α Not Less Than 0.5 (Average of Trioxial Tests)	Not Measured, but 70%-100% of Short-term Value Indicated from Load Settlement Curves for Long Term Tests.	1.0 (Average of UU TH.O. Trioxial and Unconfined Tests.)
BEARING CAPACITY FACTOR Nc. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Measured in Shafts; but 9.4 Obtained for Plate Loading Tests in Boreholes. (Average of UU Trioxial and Unconfined Tests)	Not Measured in Shafts; but Factors Varying from 9.7 to 16.6 Measured in Plate Loading Tests in Lined Boreholes. (Average of Triaxial Tests Beneath Plates)	Not Measured.	Not Measured.	12. Obtained from Shaft with Side Resistance Destroyed. (Average of UU T.H.O. Trioxial and Unconfined Tests.)
SETTLEMENT TO PRODUCE SIDE FAILURE		Not Given.	Not Given.	Not Given.	Not Given.	0.02" - 0.10" with Larger Values Occurring in Shafts with Larger Diameters.
LOAD DISTRIBUTION INFORMATION		None.	None.	None.	None.	Good Information Obtained in at Least One Test on Straight Shaft. Approximately Linear Reduction in Load with Depth
TIME EFFECT		No Increase in Resistance with Time up to 18 Months	Not Given.	Not Given.	See Below.	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		1.) 2% to 7% Increase in Moisture Content in Soil within 2 inches of Shaft-Soil Interface at Southall. Higher Increases at Greater Depths. 2.) Also Tested Driven Piles at Barnet Site. Gave Higher Resistance Values by Factor of 2. Lower Values in Shafts Attributed to Softening of Soil by Migration of Water from Concrete.	These Tests Included in Skemptons 1959 Review, Skempton States Shear Strength Values Used by Investigators too Low. Kensal Green α Values Recomputed by Skempton as 0.56 - 0.60.		After 3 Years 8 Months, All Shafts Loaded to Approximately Twice Design Load. The Measured Settlement for this Loading Increment less than Corresponding Settlement in Short-term Shaft Tests. Indicates Prolonged Loading Not Detrimental to Load-Settlement Behavior	1.) Good Agreement Between Quick and M.L. Tests for Small Diameter Shafts. Not as Good for Large Diameter Shafts. 2.) Extracted Shafts Had 1/4"-1/2" Soil Adhering to Sides. 3.) In Model Studies, no Moisture Content Increase in Soil Adjacent to Shafts Except at Very Low Initial Moisture Content 4.) Pullout Capacity was 45%-79% of Compressive Capacity

TABLE 5.I. (Continued)

INVESTIGATOR (S) & REFERENCE		Mohan & Jain (1961)	Mohan & Chandra (1961)	Skempton (1959)	Woodward, Lundgren & Boitano (1961)	Burland, Butler & Dunican (1966)
LOCATION		Jabalpur, India	Poona, Bhopal, Ujjain and Jabalpur, India	Ten Sites in London Area, England	Lemoore, California	Moorfields, London Area, England
DATE OF TEST (S)		1957 - 1958	Late 1950's	1950 - 1959	Not Reported	Not Reported
TESTING METHOD		Short-term Pullout and Compression. Time Increments for Loading not Specified.	Short-term Pullout, Compression and Cyclic. Time Increments for Loading Not Specified	Varied	Short-term M.L.	Short-term M.L. to 1.5 Times Working Load Followed by C.R.P. to Failure.
SOIL DESCRIPTION		Black Cotton Soil. Highly Expansive Clay. Shear Strength about 1-2 tsf.	Black Cotton Soil. Highly Expansive Clay. Shear Strength about 0.7 - 1.6 tsf.	London Clay Typical. Shear Strength Varies from 0.4 tsf at Top of Clay to 2.5 tsf at 50' Depth.	Layered Stiff, Silty and Sandy Clay. Shear Strength about 1.0 tsf.	London Clay, Typical. Shear Strength about 1 tsf at Surface to 2.0 tsf at 50 Feet.
NUMBER AND SIZE OF SHAFTS	STRAIGHT	11: 9"-12"∅, 6'-12' Depth	45: 6"-12"∅, 5'-12' Depth.	34: 12"-36"∅, 8'-65' Penetration of London Clay.	3 (In Clay): 18"∅, 36'-46' Depth.	1: 36"∅, 31' Depth
	BELLED	12: 9"-12"∅ Stem, 21"-30" Bell, 6'-12' Depth	0	0	0	2: 36"∅ Stem, 72"∅ Bell, 21' and 31' Deep
CONSTRUCTION PROCEDURE		Used Hand Spiral Augers and Portable Hand Under-Reamers.	Not Specified	Varied. Some by Hand and Some by Power Augering.	Mechanical Bucket Rig with Casing. No Mud Used. Shafts Concreted Immediately After Drilling Finished.	Mechanically Bored. Straight Shaft Bored and Concreted Same Day. Belled-Shafts Concreted Day After Boring.
INSTRUMENTED ? IF SO, HOW ?		No. Side Resistance Determined from Pullout Tests and from Compression Tests on Shafts with False Bottoms.	No. Side Resistance Determined from Pullout Tests, Tests on Shafts with False Bottoms and Cyclic Tests Using Van Weele's Method of Separation of Base and Side Loads.	No. Side Resistance Deduced from Bearing Capacity Equation	No. All Shafts Had False Bottoms to Give Direct Reading of Side Capacity.	No. Side Resistance Deduced from Bearing Capacity Equation.
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		0.3 \ (Average of Unconfined Compression Tests)	0.45 - 0.54 \ (Soil Tests Not Specified)	0.3 - 0.6, with 0.45 Average. \ (Average of Triaxial Tests)	0.49 - 0.52 \ (Average of Unconfined Compression Tests)	0.8, Peak; 0.35, Residual for Large-displacement C.R.P. Tests. \ (Average of UU Triaxial Tests)
BEARING CAPACITY FACTOR N_c (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		8-9 for Straight Shafts, 3-4 for Belled Shafts. Calculated by Subtracting Shaft Area Times Average Shaft Resistance from Pullout Tests from Ultimate Load. (Average Unconfined Compression)	Not Given.	Not Given.	Not Given	Not Given. Results of Several Plate Bearing Tests Reported.
SETTLEMENT TO PRODUCE SIDE FAILURE		Approximately 0.25".	Not Given	0.4" for two Shafts Constructed with Void Under Base.	Not Given.	Approximately 0.25".
LOAD DISTRIBUTION INFORMATION		None.	None.	None.	None.	None
TIME EFFECT		No Long-term Settlement on Shaft Loaded to $\frac{1}{3}$ Ultimate Capacity for Two Years.	Retesting after Lapse of One Year Showed No Increase in Frictional Resistance.	No Significant Change in Capacity on Retesting Several Shafts.	Not Given.	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		Low N_c Values Calculated for Belled Shafts May Be Due to Reduced Side Resistance Instead of Reduced Base Capacity.	1.) Friction Tests Between Soil and Dry Concrete Gave Coefficient of Friction of 0.75 at 1 Tsf Normal Pressure. 2.) Measured Radial Moisture Gradients Adjacent to Some Piles. 2-3 % Increase in Moisture Content in 2" Nearest Shaft. Greatest Increase Near Bottom.	1.) 1 Tsf Recommended as Limit for Side Resistance 2.) Low α Factors Associated with Sites where Construction was Slow and where Water Caused Deterioration of Sides of Borehole.	Driven Pipe Piles of Slightly Smaller Diameter Gave α Factors of about 0.6 in Some Soil. Other Shaft Tests Performed in Sandy Soil.	1.) Authors Conclude Residual Value of α Should be Used in Design of Belled Shafts. 2.) Authors also Report Results of Tests on 4 Large Piers by Oerrington in which α > 0.5 for 2 Straight Shafts and = 0.3 for 2 Belled Shafts.

TABLE 5.I. (Continued)

INVESTIGATOR (S) & REFERENCE		Burland (1963)	Frishmann & Fleming (1962)	Fleming & Salter (1962)	Williams & Colman (1965)	Whitaker & Cooke (1966)
LOCATION		London, England	Blackfriars Road; St. Giles Circus; London, England	Cromwell Rd., London, England	Great St. Helens, London, England	Wembley, Middlesex, London, England
DATE OF TEST (S)		Not Reported	Not Reported	Not Reported	1962 - 1963	1962 - 1963
TESTING METHOD		Short-term M.L. and C.R.P.	Maintained Load.	Maintained Load	Maintained Load and Quick Tests.	Maintained Load to 78% Ultimate Load. C.R.P. to Failure.
SOIL DESCRIPTION		London Clay. Shear Strength Profile Not Given.	London Clay. Blackfriars Site Silty. Avg. Shear Strength along Sides = 1.2 tsf (Blackfriars), 1.8 tsf (St. Giles). Avg. Shear Strength Beneath Base = 2.0 tsf (Blackfriars), 2.8 tsf (St. Giles)	London Clay. Shear Strength Varied from 0.9 tsf at Top of Clay to 2.4 tsf at 37 feet Penetration.	London Clay. Typical Strength Profile. Minimum Strength Envelope Varies from 1 to 2 tsf.	London Clay Typical. Shear Strength Variation: 0.7 tsf at Surface of Clay to 1.8 tsf at 60' Depth.
NUMBER AND SIZE OF SHAFTS	STRAIGHT	2 : (Diaphragms) 4' x 1.6', 40' Depth	0	0	See Remarks	5 : 2'-3'ø , 30.5'-50' Penetration of Clay.
	BELLED	0	2 : 2'-6" and 3'-0"ø Stems, 5'-0" and 6'-1" ø Bells, 50' Penetration of Clay.	1 : 4'ø Stems, 10'-6" ø Bell, 35' Depth.	See Remarks	7 : 2'-3'ø Stems, 4'-6' Bells, 26'-48' Penetration of Clay.
CONSTRUCTION PROCEDURE		Each Element Excavated with Grab. One Diaphragm installed Dry. One installed using Bentonite Slurrey. Each element Took Two Days to Install. Both Cured Three Weeks Before Test.	Mechanically Bored. Blackfriars Concreted Immediately. Borehole Open Three Days at St.Giles. Dry Process.	Mechanical Auger.	Hand Excavated 6'. Open Shaft to 124' Depth. Hole Mostly Dry Except in One Zone.	Calweld Power Equipment, using Normal Good Practice. Bases of Belled Shafts Cleaned by Hand. Most Shafts Open One Day or Less.
INSTRUMENTED ? IF SO, HOW ?		No.	Yes. St.Giles Shaft Instrumented with Flatjack at Top of Bell, Foil-type Strain-Gages on Vertical Steel, Mechanical Strain Wires. Flatjack and Strain Wires Performed Well.	No.	No.	Yes. Electrical Load Cells Placed at Top of Bells or Bottom of Straight Shafts.
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given. Element Installed with Bentonite Slurry Had Nearly Identical Load Settlement Curve as Dry Element. Approximately Same α Factor Implied for Each.	Approximately 0.2 at Blackfriars and 0.35 at St.Giles. Low Value at Blackfriars Attributed to Higher Silt Content. (Soil Test Procedure Not Given)	Not Tested to Failure, but α Probably about 0.6. (Soil Test Procedure Not Given)	1.0 when Sides of Borehole are Dry. (Minimum Envelope of UU Triaxial Tests)	0.44 - All Shafts. Little Variation with Depth or with Size of Base. (Average of UU Triaxial Tests)
BEARING CAPACITY FACTOR N_c. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given.	Not Obtained at Blackfriars. St.Giles Shaft not Completely Failed.	Not Given.	9 for Bearing Tests on Blocks, Occurring at Settlement of 5% of Base Diameter in Quick Tests. (Minimum Envelope of UU Triaxial Tests)	6.75- All Shafts Mobilized at Settlement of 10%-20% of Base Diameter. (Average UU Triaxial) 9.0 (Minimum Envelope of UU Triaxial Tests)
SETTLEMENT TO PRODUCE SIDE FAILURE		Approximately 0.2" for Both Dry and Bentonite-coated Elements.	Not Given.	Not Given.	0.05"- 0.1" Where Soil Dry Behind Grout. 0.6" Where Soil Wet Behind Grout.	0.15" for 2'ø Shafts to 0.30" for 3'ø Shafts. Little Difference Between Belled and Unbelled Shafts.
LOAD DISTRIBUTION INFORMATION		None.	Not Reported	None	None.	None
TIME EFFECT		Not Given.	Not Given.	Not Given.	Not Given.	Approximately 12% Increase in Side Resistance in One Year Based on Pull Tests of Anchor Shafts. Increase Proportional to Logarithm of Time.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		1.) Generally 3%-4% Increase in Moisture Content in Soil Near Both Elements. Extent of Moisture Migration was 2-3 inches. 2.) Using Skempton's Shear Strength Profile for London Clay and Bearing Capacity Equation, α was Approximately 0.6.	Strain Gages Experienced some Instability, but did Show Decreasing Load with Depth. Numerical Results Not Given.		1.) Jacked Between Base Blocks and Sections of Precast Concrete Shaft Liner Grouted to Soil to Obtain Frictional Resistance and Base Capacity at Several Levels. 2.) "Immediate Settlement" Achieved in 2 Minutes for the Liner.	1.) Side Resistance Developed Was Greater Than Fully Softened Shear Strength of Soil. 2.) Proposed Design Methods Presented.

TABLE 5.I. (Continued)

INVESTIGATOR (S) & REFERENCE		Deb & Chandra (1964)	Matich & Kozicki (1967)	Komornik & Wiseman (1967)	Van Doren, Hazard, Stallings & Schnacke (1967)
LOCATION		Jabalpur, Ujjain, Poona and Indore, India	Brookfield, Nova Scotia	North Tel Aviv, Israel	Wichita, Kansas
DATE OF TEST (S)		Not Reported	1964	Not Reported	1966
TESTING METHOD		Long-term (2½ years) Maintained Load and Short Term Maintained Load	Maintained Load Compression on Unrifled Shafts, M L Pull-out on Rifled Shafts	Cyclic	Maintained Load
SOIL DESCRIPTION		Black Cotton Soil Highly Expansive Clay. Shear Strength about 1-2 tsf. Zone of Seasonal Moisture Change to 12' Depth	Dense Glacial Till (CL) Approximately 30' Thick Overlying Weathered Shale Average Shear Strength; for Till, 1.0 tsf; for Shale, 2.5 tsf	Slightly Sandy Fat Clay Along Sides Some Sand Base Founded in Cemented Yellow Sand. Average Shear Strength of Clay 1.5 tsf	Silt, Sand and Silty Clay Overburden 25 Feet Thick Overlying Weathered-to-intact Shale and Gypsum Shear Strength of Shale Was 1-10 tsf. No Shear Strength Tests in Overburden.
NUMBER AND SIZE OF SHAFTS	STRAIGHT	6: 9"-12"ø, 6'-12' Depth.	4: 2.5mooth Sided, 32"ø, 40' Depth in Till and Shale 2 Rifled, 24"ø, One 20' in Till, One 13'6" in Shale.	1: 26"ø, 63' Depth	5: 30"ø, 44'-74' Depth
	BELLED	7: 8"-12"ø, 25"-35" Bells, 12' Depth, One Double Bell	0	0	0
CONSTRUCTION PROCEDURE		Method Not Specified Presumably Portable Hand Auger	Mechanical Boring. Two Shafts Had Rifled Boreholes with a Spiral Groove 3" Deep with 18" Pitch	Benoto Method	Mechanical Auger Borehole Through Overburden Processed with Bentonite Slurry, Cased and Gated Out. Slurry Left in Holes Overnight Drilling into Shale Accomplished in the Dry
INSTRUMENTED ? IF SO, HOW ?		No	No	No.	Yes Hydraulic Pressure Cells and Borehole Extensometers at Different Levels
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Approximately 0.5 - All Shafts (Soil Tests Not Specified)	Unrifled: Tests Not Conducted to Failure, but α about 0.3 in Till Based on Shape of Load Settlement Curve Rifled: Approximately 1.0 on Basis of 24" Shaft Diameter in Both Shale and Till (Average of UU Unconfined and Triaxial Tests)	Complete Failure Not Achieved, but at Least 500 psf Developed in Side Friction in Clay Thus α > 0.17 (Average of Vane Shear Tests)	Complete Failure Not Achieved, but Maximum of 2000 psf Side Friction Indicated in Overburden Material. Shear Failure Not Achieved in Shale
BEARING CAPACITY FACTOR N_c (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given	Not Obtained in Tests	Not Obtained in Tests	Not Obtained in Tests
SETTLEMENT TO PRODUCE SIDE FAILURE		Not Given.	Complete Failure Not Quite Achieved in Pullout Tests in Rifled Shafts of 0.8"-1.0" Movement	About 0.1" Implied from Load Settlement Curve	Not Given
LOAD DISTRIBUTION INFORMATION		None	None	None	Load Decreased Somewhat Erratically with Depth
TIME EFFECT		Settlements Stabilized in 1-3 Months at 1.0-1.5 Times Working Load. Long-Term Shear Deformation of About 0.05 inches at 0.3 Times Ultimate Load.	Not Given	Not Given	Not Given
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		Comparison of Normal Belled Shafts with Belled Shafts with Side Friction Destroyed Indicated that Side Friction is Essentially Constant with Time at Loads of 0.3-0.6 of Ultimate in Long Term Tests.		Authors Discuss Shaft Installed in Sand of Nearby Site with Bentonite Slurry Technique Test Results Indicated High Side Resistance	1) No Evidence of Plane of Weakness Between Shaft and Soil in Overburden where Slurry Was Used 2) Evidence Some "Belling" May Have Occurred at Interface Between Overburden and Shale

TABLE 5.1. (Continued)

INVESTIGATOR (S) & REFERENCE		U.S. Army Engineer District, Fort Worth, Texas (1968)	Reese & Hudson (1968)	Bhanot (1968)	Watt, Kurfurst, & Zeman (1969)	Reese, Hudson & Vijayvergiya (1969)
LOCATION		Lockland Air Force Base, San Antonio, Texas	Austin, Texas	Edmonton, Alberta	Saskatoon, Saskatchewan	San Antonio, Texas
DATE OF TEST (S)		1966-1967	1966-1967	1967	1967	1967-1968
TESTING METHOD		Short and Long Term Maintained Load.	Quick Tests.	Maintained Load. Load Cycled for Shafts in Till.	Equilibrium Method.	Quick Tests.
SOIL DESCRIPTION		13 Feet of CH and GC material Overlying Jointed Shale. Shale and Overburden Highly Expansive. Contain 75% Montmorillonite. Average Shear Strength of Overburden 1 tsf; of Shale (In Test Zone) 3 tsf.	Fat Clay with Calcareous Material to 6' underlain by Leon Clay. Overconsolidated Shear Strength Averaging 2.5 tsf. No Water.	Top 20', Stiff Silty Clay. Shear Strength = 1 tsf; Underlain by 5' of Silt. Glacial Till, Shear Strength = 3 tsf, Beneath Silt.	Highly Plastic Clay 0'-13' with Shear Strength = 0.9 tsf. Water bearing Silty Clay 13'-18', with Shear Strength = 0.57 tsf. Glacial Till 18'-28' with Shear Strength = 1.0 tsf.	Fat Clay to 18' Underlain by Clay-Shale. Considerable Gravel. Shear Strength Varied from 1 to 5 tsf where Samples Could be Recovered.
NUMBER AND SIZE OF SHAFTS	STRAIGHT	3 : 18"ø, 13' Depth, Base Resistance Destroyed	1 : 24"ø, 12' Depth	4 : 2 in Clay, 16"-20ø,14'-17' Embedment. 2 in Till, 20"-28"ø, 11'-22' Embedment.	3 : 24"ø, 10.5'- 26.9' Embedment. Voids Beneath Bases.	1 : 30"ø, 28.5' Depth
	BELLED	4 : 18"-30"ø Stem, 36"-48" Bells, 34'-36' Depth	0	0	0	0
CONSTRUCTION PROCEDURE		Power Augers with Casing. Dry Process. After Initial Load Testing, Site was Ponded for One Year to Produce Soil Expansion.	Mechanical Auger. Installed in the Dry in Several Hours.	Mechanical Auger. Installed in the Dry in 1-2 Hours.	Mechanical Auger. Orilled Below Water Table Without Use of Mud. Water in Longest Shaft Installation Required 4 Hours Per Shaft.	Power Auger. Dry Procedure. Installed in One Day.
INSTRUMENTED ? IF SO, HOW ?		Yes. Bonded Strain Gages Along Reinforcing Steel. Carlson Stress Meters Installed Along Sides to Measure Lateral Pressure.	Yes. Concrete Embedment Gages, Strain Rods, Lateral Earth Pressure Cells at Several Levels.	Yes. All Shafts Had Electronic Load Cells Near Base. Generally Reliable Results. Also Used Strain Rods.	Not Reported.	Yes. Concrete Embedment Gages, Strain Rods, Lateral Pressure Gages.
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		About 0.2 to 0.6 for Short Shafts in Overburden, on the Order of 0.3-0.5 in Shale. Based on Short Term Testing. (Average of UU Triaxial Tests)	0.55 (Average of Unconfined Compression Tests)	0.43 in Clay. 0.65 in Till (Average of Undrained Triaxial and Unconfined Tests)	0.3 in Clay. 1.0 in Silty Clay and Till. (Average of Direct Shear and Torvane Tests)	Maximum of Approximately 800 Tons Transferred in Side Friction. α Not Computed Since Undisturbed Samples Could Not Be Recovered for Entire Length of Embedment.
BEARING CAPACITY FACTOR N_c. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Ultimate Factor Not Clearly Defined.	9.2 (Average of Undisturbed Unconfined and Fully Softened Unconfined Tests)	Complete Bearing Capacity Equation of Meyerhof Recommended. $N_{c\phi}$ = 13-25 Found in Till.	None.	Not Given.
SETTLEMENT TO PRODUCE SIDE FAILURE		0.05"-0.2" in Short Shafts in Overburden. Not Clearly Defined for Shale.	Approximately 0.1"	0.12" in Clay. Varied in Till : 0.4"-1.5"	Approximately 0.15" for Shaft Embedded in Plastic Clay and Silty Clay. Not Clearly Defined for Others.	Not Well-defined. Gradual Failure from 0.2" to 0.6" Settlement.
LOAD DISTRIBUTION INFORMATION		Gages Fairly Stable Over Short Term. Showed Increased Load Transfer Near Bottom of All Shafts.	Load Distribution Approximately Linear. Only Four Levels of Axial Load Transducers.	None.	None.	Very Little Load Transfer in Top of Shaft; Highest Load Transfer (6-14 tsf) in Clay-Shale Near Base.
TIME EFFECT		One Underreamed Shaft with Bell in Shale Under 120 Tons Sustained Load Showed About 40 Tons Tension in Shaft Above Bell One Year After Test Site Ponded.	Not Given.	Not Given.	Not Given.	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES ; OTHER REMARKS.		1.) Strain Gages Indicated Tension During Curing. 2.) Load Tests Conducted on Only Four Shafts. 3.) Tensile Stresses on Order of Half of Maximum Shear Mobilized During Compression Testing Indicated in Shale. 4.) Lateral Pressure Cells Indicated Earth Pressure Coefficients Greater Than 1.0 in Shale One Year After Ponding Site.	Tests Conducted Primarily to Evaluate Methods of Instrumentation.	1.) Presents Results of Model Tests of Shafts in Silt. N_c About 6 , α About 0.8. 2.) 24 Hour Settlement Readings Become "Plotably Different" from Immediate Settlements at About 2/3 of Plunging Load.	Borehole Direct Shear Device Used to Obtain Shear Stress vs. Vertical Movement Curves for Various Normal Pressures against Walls of Borehole. From Results, Authors Deduced Maximum Pressure of 10 psi Acting Laterally During Actual Load Test.	Load Transfer and Load-settlement Correlation Made on Basis of Texas Highway Department Cone Penetrometer Tests.

(for example, Eq. 4.7b). Some investigators assumed a value for N_c of 9 in clays, based on previous experience, while others conducted deep plate loading tests at the test site and applied the results to the bearing capacity formula.

Correlation of Field Test Results With Soil Properties

The main thrust of the early testing in London Clay was to determine design values for α , the average side shear strength reduction factor. Before α could be computed, an accurate shear strength profile had to be established to which the deduced average side shearing stresses for each test site could be referred. However, difficulties were encountered in obtaining representative in situ shear strength values. In most cases, the shear strength profile was obtained by recovering undisturbed samples and performing triaxial and unconfined tests in the laboratory.

Considerable scatter appears in results of laboratory shear strength tests on undisturbed samples of fissured clay, such as London Clay (Skempton, 1959). In addition, sample disturbances, sample size, testing method, orientation of test specimen in testing apparatus (or the orientation of the fissures), and elapsed time between sampling and testing all influence the indicated shear strength of stiff, fissured clay. For example, since larger undisturbed soil samples have a greater probability of containing more fissures than do smaller samples, the larger samples tend to indicate a lower shear strength. Skempton and Hutchinson (1969) summarized the effect of sample size on the undrained shear strength of London Clay as given by triaxial tests. Taking the average shear strength for the standard triaxial specimen size (1.5 inches in diameter by 3.0 inches high) to be 1.0, they reported relative indicated strength values

of 0.6 - 0.8 for specimens 4 to 12 inches in diameter and 8 to 24 inches high. Conversely, small chunk samples containing no fissures yielded shear strengths of 1.5 to 1.9. Unconfined tests are likely to give completely erroneous strength values, since premature failures along fissures can occur freely.

For any given natural soil, influence of sample size is dependent on the size and arrangement of the fissures. For a severely fissured soil in which fissures are spaced only a fraction of an inch apart and are arranged in a random pattern, samples of various sizes and of various orientations in the testing machine would not be expected to yield very different values of strength. On the other hand, when the fissures are continuous, more widely spaced, and appear in distinct patterns of preferred orientation, larger samples, having a greater probability of containing fissures, will exhibit lower strength than smaller specimens. Furthermore, test samples oriented in such a way that the plane of maximum shear stress is parallel to the plane of fissure orientation are likely to yield much lower values of shear strength than specimens oriented such that the plane of maximum shear stress is at some angle to the direction of fissure orientation.

Figure 5.1 presents hypothetical results from sets of UU (Unconsolidated Undrained) triaxial tests run on both large and small specimens of soil from a drilled shaft test site. The profiles of average shear strength are shown. The usual procedure of shearing each specimen at a confining pressure equal to the computed overburden pressure has been followed.

Several important points are evident in the figure. First, the average strength for the large specimens represents a minimum envelope to the
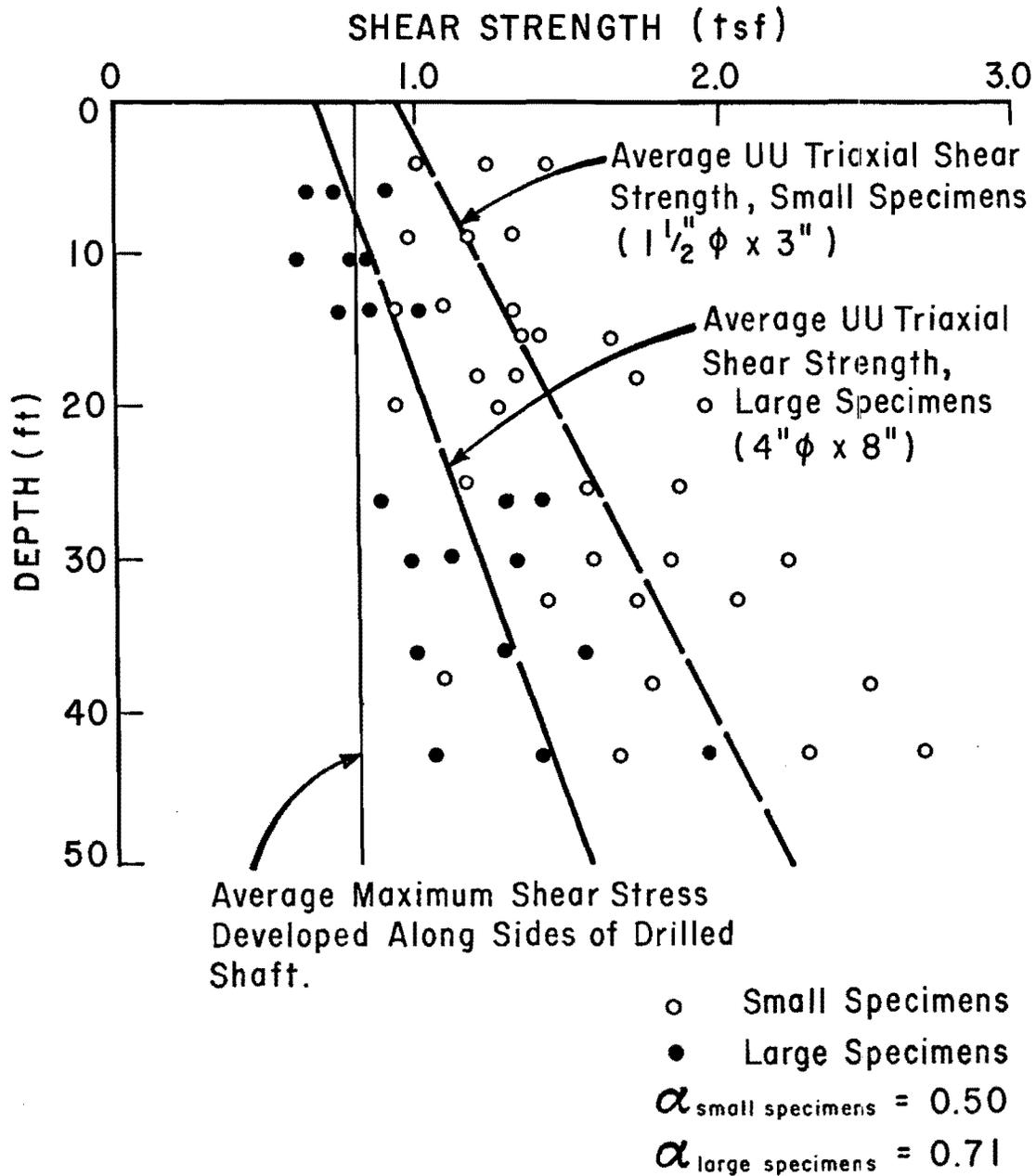
Fig. 5.1. Hypothetical Comparison of Average Shear Strengths Indicated by Tests on Small Specimens and on Large Specimens

shear strength values obtained for the small specimens. Assuming that all large specimens contained representative fissures, the lesser average is expected to be more nearly equal to the bulk, in situ shear strength of the soil. However, since shearing of the soil along the side of the shaft is forced to occur on a surface parallel to the sides of the stem, more intact soil is likely to be sheared than in laboratory tests on large samples, which tend to fail along fissures. Hence, for this reason, it appears qualitatively that α is more correctly referred to shear strength from the small specimens, although selection of exact size to represent the strength of the soil being sheared along the sides is highly indeterminate. On the other hand, base failure may tend to occur more readily along existing fissure surfaces, and the average of the larger samples may give a more appropriate description of the shear strength for purposes of calculating base capacity.

Second, the scatter is greatest for the smaller specimens. This reflects the fact that some specimens contain fissures, while others do not. Thus, the average is less clearly defined with small specimens.

Third, the α factor is obviously lower when the developed shear stress is referred to the average of tests on smaller specimens.

It can be inferred from the preceeding discussion that, during any research study employing load tests to establish side shear capacity in a particular soil, the soil test procedures employed should be consistent with those which are used in design. In this way the "correct" value of shear strength becomes a moot question, although it should be understood that the value of α so obtained is test-method-dependent. When the soil test method to be used in design is expected to vary, the mobilized

shear stresses in a field study should be referred to the procedure giving maximum shear strength, when α is to be calculated, in order to arrive at conservative factors.

Meyerhof and Murdock (1953) describe the effect of allowing various periods of time to elapse between sampling and testing of fissured clay. They present results which suggest that as much as one-third to one-half of the unconfined or undrained triaxial shear strength may be lost by testing one week to several months after sampling because of gradual opening of the fissures.

Ward, Samuels, and Butler (1959) discuss the effects of sample disturbance on indicated undrained shear strength and elastic modulus. They conclude, in general, that sampling with the usual tube samples causes a reduction in both the indicated strength and the elastic modulus.

A number of other factors effecting the laboratory measurement of shear strength are detailed by Skempton and Hutchinson (1969).

The foregoing brief description of possible sources of error in obtaining a "true" shear strength profile is included to point out that the α factors obtained in field load tests reported in Table 5.1 are influenced not only by inaccuracies in estimating or directly measuring side shear, but also by the sampling and testing technique used to obtain the shear strength profiles to which the test values have been referred. Shear strength values reported in Table 5.1 undoubtedly have been obtained by procedures that were inconsistent among investigators. Nonetheless, when studied in their entirety, the field tests do indicate definite trends of behavior, especially in regard to the mobilization of maximum side shear.

Table 5.1 exhibits the laboratory test method that each investigator used for determining the shear strength profile, such as passing a curve through the average UU triaxial strength at various depths. Sample sizes generally were not reported. It is common practice in Britain to use 1.5-inch-diameter by 3-inch-high triaxial specimens, and it may be assumed that most of the results in London Clay came from triaxial tests on samples of those dimensions. The T.H.D. triaxial method mentioned in some of the references uses test specimens that are approximately twice the dimensions given above. The T.H.D. procedure often incorporates the use of multiphase shear devices which reportedly provide a complete failure envelope from one test specimen by shearing the same specimen repeatedly at several confining pressures. Further details concerning laboratory testing are given in many of the references.

In summary, while the α factors and N_c values reported in Table 5.1 were obtained by investigators who employed inconsistent testing methods, and who determined α from different soil test procedures, enough information has been given in each case to allow the reader to make reasonable comparisons of reported numerical values. An elaboration of the test results summarily presented in Table 5.1 is given in the following sections.

Studies in London Clay

Meyerhof and Murdock (1953) performed a series of load tests on unin-strumented drilled shafts at Southall and Barnet in London Clay in which they determined that α varied from 0.2 to 0.4. After the tests, they obtained samples of soil from the zone immediately adjacent to the shaft and observed that the soil was considerably wetter (by 2 to 7 per cent)

than the soil some distance away from the shaft. They explained this phenomenon by proposing that part of the water not required for hydration of cement had migrated into the soil, causing the soil to soften. They tested their hypothesis by installing other shafts such that the amount of water added to the concrete mix was just enough to satisfy hydration requirements. No increase in moisture content was observed in the soil next to these shafts; however, the quality of the concrete was so poor that the concrete crushed under load before failure of the supporting soil occurred.

These tests indicated that one important parameter affecting the values of α was the water-cement ratio of the concrete mix: the higher the ratio, the lower the α value. The unusually low α values which were observed, however, were probably influenced by other factors, especially the method of construction. Some of the test shafts had been excavated by hand digging, which required several days and which may have permitted deterioration of the walls of the boreholes before concrete was placed.

The shafts were tested to failure at about one month, six months, and 18 months after casting. In each case, the ultimate resistance was about the same.

Golder and Leonard (1954) conducted similar tests in London Clay at Kensal Green. They obtained considerably higher α values (0.6 to 0.7) than did Meyerhof and Murdock. The reasons for the differences between the results of the two groups of tests are not clear. Part of the differences may be due to inconsistencies in determination of shear strength profiles, and part may be due to differences in construction procedures.

Both teams of investigators measured bearing capacity factors on the order of 9 by performing plate loading tests in the boreholes.

Skempton (1959) reviewed the test results of Meyerhof and Murdock, Golder and Leonard, and others. By tabulating ultimate loads and computed end bearing values, he concluded that α could vary from 0.3 to 0.6 in London Clay, at least on the basis of short-term loading. He suggested a design value for α of 0.45, unless the product of α and the shear strength was greater than 2000 psf, in which case a maximum of 2000 psf was to be allowed. He further recommended that smaller values be used for short shafts.

Skempton reasoned that the α factor was a function of two phenomena: the softening of the soil brought about by absorption of water, and the adhesion between the concrete and soil, which he stated was approximately 80 per cent of the shear strength of the softened soil. He attributed softening to the following four basic factors:

1. Stress release upon opening the borehole reduces pore pressures in the soil near the walls, promoting an inward migration of pore water from the mass of soil around the borehole. This action is accelerated when fissures are present.

2. Cracks, fissures, or permeable seams are often opened below the water table during drilling, allowing free water to leak onto the sides and bottom of the borehole before concrete is placed.

3. Soil composing the borehole wall is softened by water used to facilitate cutting operations. (In addition, when drilling mud is employed, copious amounts of water are available to soften the walls of the borehole.)

4. Soil composing the borehole wall is softened by free water
   which migrates from the wet concrete into the soil.

The degree to which each of these factors contributes to reduction in
shear strength is obviously highly dependent on soil and groundwater condi-
tions at the construction site. The length of time the borehole is allowed
to remain open may also be an important factor, since holes which are open
only a short time prior to concreting are not influenced as greatly by
factors 1 or 2. It should be noted that factors 1, 2, and 3 work in oppo-
sition to factor 4. That is, if the sides of the borehole become wetter
because of any of the first three factors, migration of water from the
concrete will take place under a smaller suction potential and will there-
by have less influence than if the sides were drier. It was generally
observed by Skempton that lower values of α tended to be produced when
the hole was open for prolonged periods and when silty, waterbearing soil
was present.

Although not mentioned specifically by Skempton, other factors in
addition to softening of the soil also apparently influence α. Among
these are:

1. Remolding of soil around walls of borehole by augering.

2. Drying of surface soil, after the shaft is placed, possibly
   causing soil to shrink away from shaft.

3. Mechanical interference with side shear by the base, causing
   smaller α factors near the base.

4. Opening of fissures, causing a reduction in shear strength
   of soil along the walls by an action similar to that explained
   by Meyerhof and Murdock for reduction in indicated shear

strength in laboratory specimens tested several weeks

after sampling.

Green (1961) reported results of long-term, static tests on small-diameter shafts. Design loads were maintained for up to 3 years, 8 months. At the end of that period, loads were increased to twice the design load. The settlement corresponding to the latter increment of load was observed to be less than that occurring under the same increment in identical shafts during short-term loading, indicating that long-term loading was not detrimental to load-settlement behavior. The exact effect of long-term loading on side shear was uncertain, since part of the increased stiffness could have been due to consolidation of soil beneath the base.

Further research on larger belled shafts in London Clay was performed by Frischmann and Fleming (1962) and Fleming and Salter (1962). The test at St. Giles Circus in London reported by Frischmann and Fleming marked an early attempt at comprehensive instrumentation for determining load distribution. An hydraulic load cell was placed at the top of the bell, and foil-type strain gages were installed on the reinforcing steel. Good results were obtained with the load cell, but some stability problems were experienced with the strain gages. The tests, on three belled shafts, yielded α factors varying from about 0.2 to about 0.6. The low value was obtained for a test shaft on Blackfriars Road, where the soil had a high silt content. The test shaft at St. Giles Circus yielded an α factor of about 0.35 despite the fact that the borehole was open for several days prior to concreting.

Williams and Colman (1965) conducted tests on sections of concrete liner grouted to the wall of a six-foot-diameter borehole. They also ran

bearing tests at several levels as the borehole was advanced. When the sides of the borehole were dry before placing the section of liner, they obtained an α factor of 1.0 with respect to the minimum envelope of UU triaxial tests. In doing this kind of correlation, they felt that the fissured strength of the clay controlled the maximum side resistance, and that the minimum envelope represented fissured strength, or the average strength of very large test specimens.

Whitaker and Cooke (1966) pointed out that considerable difficulty is incurred in establishing the minimum envelope to the results of shear strength tests, since drawing the envelope involves considerable subjective judgement. Although, according to Williams and Colman, it may be more rational to use the minimum envelope, the average strength-versus-depth line is much more accurately defined; therefore, it appears that α should be referred to the curve of the average shear strength by convention in order to obtain more accurate values. The tests of Williams and Colman suggest that the α factor is in fact primarily a measure of the amount that the average laboratory shear strength differs from the bulk, fissured strength of the clay, and that side shear capacity is governed by the strength of the clay along fissures, at least for grouted cylinders. The implication is that shearing occurred almost completely through fissures and not through intact soil. This tends to contradict the earlier observations of Meyerhof and Murdock (1953) and Skempton (1959) for actual drilled shafts, who indicated that softening of the intact soil due to water migration governed the side shear capacity. The actual cause of shear strength reduction in the tests on grouted cylinders (shear through fissures as opposed to softening of soil) is in reality not

determinate, since an α factor of 1.0 with respect to the minimum envelope could be coincidentally an indication either of fissure failure or softening.

Williams and Colman also conducted bearing tests on blocks at the bottom of the borehole as excavation progressed, from which they calculated an N_c value of 9 using Eq. 4.7b and shear strength from the minimum envelope. They also observed that vertical movements of about 0.05 to 0.10 inches were required to produce failure in side shear, while downward movements of about 5 per cent of the diameter of the base blocks were required to generate full mobilization of the soil in bearing.

Burland, Butler, and Dunican (1966) reported tests of uninstrumented drilled shafts in which the displacements were carried to seven to eight inches by the CRP test method. They obtained a shear strength profile and estimated base capacities by conducting plate bearing tests in a borehole. Using an N_c value of 9, (or other appropriate value for shallow depth) they computed shear strengths from Eq. 4.7b. All α values were calculated from this profile. They observed that the average α value reached a maximum of about 0.8 as initial failure in side shear took place, but that it decreased to an asymptote of approximately 0.35 at large displacement (assuming base resistance remained constant). The CRP tests were conducted over a period of several hours. The tests reflect the effect of remolding and the tendency to develop the residual shear strength of overconsolidated clay at large displacement. The tests indicated that peak side resistance was developed at a displacement of about 0.25 to 0.30 inches. The authors observed that underreamed shafts suffer greater displacement than straight shafts at comparable overall factors

of safety. In fact, in many instances, the sides are in a failed

condition by the time adequate load has been developed on the base to

bring the overall factor of safety to the desired value. For this reason,

Burland, Butler, and Dunican suggested that the lower or "residual" α

factor should be used in the design of belled shafts.

A comprehensive series of load tests on instrumented shafts was con-

ducted by Whitaker and Cooke (1966) in the early 1960's. Tests on twelve

straight and belled shafts with electrical load cells at the bottom of the

shaft or the top of the bell verified the α value of 0.45 which Skempton

had proposed for design applications in London Clay. Whitaker and Cooke

measured an average α of 0.44 with respect to the average of UU triaxial

tests by conducting maintained load tests to approximately three-fourths

of the estimated total capacity of each shaft. In reality, the α factors

might have been slightly higher had the tests been carried to failure

under maintained load, but the differences would probably have been insig-

nificant. The shafts were then loaded to failure by the CRP method to

obtain the maximum base resistances.

Several parameters, including shaft length, base diameter, and stem

diameter were varied in the study. Length was observed to have no effect

on side resistance other than to increase it in proportion to depth of

embedment (with due consideration to variation of shear strength with

depth). There was no observed effect of base enlargement on the side

capacity or on load-settlement behavior of the sides. Settlement required

to produce failure in side shear was observed to increase in proportion to

the stem diameter, however. About 0.15 inches was required to produce

peak side resistance in two-foot-diameter stems, while about 0.3 inches was required in three-foot-diameter stems.

Whitaker and Cooke reported that settlements of 10 to 20 per cent of the base diameter were required to produce complete base failure in each case. A bearing capacity factor of 9 with respect to the minimum envelope of the shear strength tests was observed. In this regard, the authors proposed the following formula for bearing capacity of deep foundations in stiff clays:

$$(Q_B)_{ult} = 9 \; \omega \; A_B \; c_{mean} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (5.1)$$

in which

ω = bearing capacity reduction factor for fissured clay
(varies according to soil formation)

A_B = area of base

c_{mean} = average soil cohesion beneath the base.

Whitaker and Cooke reason that the base failure occurs by shearing action primarily through fissures. Hence, when the theoretical bearing capacity factor of 9 is applied in fissured clays, it should be with respect to the "fissured" shear strength, or, if applied with respect to the mean triaxial shear strength, it should be reduced by the factor ω. Since fissured shear strength is estimated by evaluation of the minimum shear strength envelope, a subjective and often inaccurate procedure, they felt that the use of Eq. 5.1, with a value of ω appropriate for the soil in question, would be a more reliable procedure. They suggested a factor of 0.75 for ω for the soil at Wembley, where the tests were performed.

Pullout tests were conducted on some of the straight anchor shafts at various periods of time after casting, ranging from a few weeks to over a year. Side capacity was observed to increase approximately linearly with the logarithm of time. Shafts tested after the greatest elapsed time yielded average frictional resistance about 12 per cent greater than those tested soon after casting.

Data from load tests on drilled shafts installed with drilling mud are very scarce. However, Burland (1963) reported the results of axial load tests on two identical, uninstrumented I.C.O.S. diaphragm wall, load-bearing elements, similar to drilled shafts. Each was 4 feet by 1.6 feet in plan and embedded 40 feet in the London Clay. One of the elements was installed in the dry, while the other was installed using bentonite-water slurry. Each element took about two days to install. The load tests were conducted three weeks after casting.

Prior to the tests, it was assumed that the bentonite coating, which would be trapped between the concrete and the undisturbed soil, would diminish the side shear capacity. The load settlement curves from both elements, however, were almost identical, indicating that the element installed with mud developed as much side shear (and at the same rate) as the element installed in the dry.

Several months after the tests were concluded, shafts were sunk near the test elements and moisture samples were taken in the soil immediately adjacent to the walls of the elements. Radial moisture profiles were obtained at two levels. Both the dry element and the bentonite-coated element showed moisture increases of about three per cent in the two inches nearest the element, with little difference in the profiles between the

two elements. These tests, although certainly not conclusive in themselves, indicate that the use of bentonite mud may not adversely affect the side shear capacity of drilled shafts in stiff clay for the particular case when the I.C.O.S. procedure is used (displacing bentonite slurry directly with pumped tremie concrete without using casing).

Studies in Texas Soils

The state of Texas has long been an area in which drilled shaft construction has been popular. The soils in which drilled shafts are frequently used range from the stiff, fissured Beaumont Clay along the Gulf coastal plain, to the hard, expansive weathered shales and clay-shales of the central Texas region, to cemented sand formations in southern and western Texas. Quite naturally, some field research into the behavior of drilled shafts in various soil profiles has been attempted. One of the first reported studies was undertaken by Harris (1951) in the Beaumont Clay in Houston. One uninstrumented test shaft installed through water-bearing soil with casing but without mud was constructed and load tested. Side shear factors were not given by Harris, but α was probably not less than 0.5. This fact can be deduced from shear strengths implied from allowable load calculations given by Harris and by deducting ultimate base load using the bearing capacity equation (taking $N_c = 9$). The borehole reportedly had a ragged side and about four feet of water in the bottom at the time concrete was poured. Results of this test indicated that side friction could be of major importance in drilled shafts in Beaumont Clay, even when construction conditions were somewhat adverse.

Under sponsorship of the Texas Highway Department, the Texas
Transportation Institute undertook a study of the behavior of model and
intermediate-sized drilled shafts in College Station, Texas, in a clayey
soil of the Claiborne Group, an Eocene deposit exposed in a band parallel
to and about 125 miles from the Texas Gulf Coast (DuBose, 1956). The
results of test loadings of 35 straight and belled shafts were reported.
All shafts were installed in the dry. The investigation showed that
failure in side shear actually occurred in the soil and not by slippage
at the interface of the soil and concrete. DuBose tested straight shafts
with electrical load cells at the base and obtained an average indicated
maximum side shear stress approximately equal to the shear strength of the
soil. Similar results were indicated for underreamed shafts. He also
mounted strain gages on the reinforcing steel and measured load distribu-
tion in one shaft. This was evidently the first time this feat was suc-
cessfully performed for a drilled shaft, although Evans (1952) mentioned
installing strain gages on reinforcing bars for some load tests on shafts
in sand. A linear distribution of load was obtained for one loading
shown by DuBose.

Accompanying laboratory studies with model shafts installed in a
remolded CL material (Miller Clay) indicated that no moisture increase
occurred in the soil adjacent to the model concrete shafts, except when
the soil was placed at a very low moisture content. The water-cement ratio
seemed to have little effect on water migration in these tests.

DuBose measured the bearing capacity factor, N_c , in several shafts.
The value was found to be about 12. He also observed a nearly direct
increase in shaft capacity with length for straight shafts of the same

diameter. Settlements required to produce side shear failure increased with increasing stem diameter, with full-sized shafts of 12-to-24-inch diameters requiring about 0.1 inches displacement to produce failure.

Turner (1962) reported results of pullout tests on short, straight drilled shafts in stiff clay above the water table in the Houston, Texas, area. He concluded that the entire undisturbed shear strength of the soil was effective in side resistance, as long as the shaft was installed in the dry above the water table.

The U.S. Army Engineer District, Fort Worth, Texas (1968), conducted tests on seven straight and belled drilled shafts in the clay-shales of the upper Midway Group in San Antonio. The soil at the test site was composed of several feet of CH and GC overburden above a moisture-deficient jointed clay-shale. The purpose of the tests was to investigate the effects of soil moisture changes on shaft capacity and on the vertical movement characteristics of loaded and unloaded shafts, and to ascertain the distribution of load between sides and base of a drilled shaft in clay-shale. The shafts were instrumented with strain gages on the reinforcing bars and with Carlson earth pressure cells along the borehole wall to measure horizontal pressure changes as the soil expanded and contracted. Some stability problems were experienced with the strain gages over the long periods of time required to affect, artificially, moisture changes in the soil.

Results of long-term testing in San Antonio have already been mentioned in the previous chapter in the discussion of design of drilled shafts in expansive soils. Stress changes were very pronounced in the shafts as the soil expanded. Tensile forces were measured near the bottom of the stem

of a shaft embedded in the clay-shale under a sustained compressive load

of 120 tons after water was made available to the clay-shale by ponding

the test site. Short-term load tests were also conducted. The results

indicate short-term α factors in the order of 0.2 to 0.6 for the over-

burden and 0.3 to 0.5 for the clay-shale.

In 1965, the Center for Highway Research (CFHR) of The University of

Texas at Austin embarked on a program of installing and testing full-

scale instrumented drilled shafts in different parts of Texas to investi-

gate various aspects of behavior under load. Locations and dates of tests

conducted to the present (1970) in the series have been chronicled by

Barker and Reese (1970). Reese and Hudson (1968) reported short-term

test results on a small prototype shaft in a lean, calcareous, overburden

clay in Austin, in which α was measured to be 0.55 and N_c about 9.2

with respect to shear strength obtained from unconfined compression tests.

Tests of an instrumented drilled shaft installed through a very stiff

clay into a clay-shale containing inclusions of shells and sandstone in

San Antonio were also conducted by CFHR (Vijayvergiya, Hudson, and Reese,

1969; Reese, Hudson, and Vijayvergiya, 1969). The nature of the soil was

such that it could not be sampled. However, load transfer curves were

calculated from the output of electrical concrete embedment gages and

from strain rods, which had been placed at several levels in the shaft.

The maximum load transfer at various levels in the shaft was thus obtained.

These values were correlated with the T.H.D. dynamic cone penetrometer

(described by Vijayvergiya, Hudson, and Reese, 1969). This correlation

indicated that the maximum unit side resistance was equal to the quotient

of the number of penetrometer blows per foot divided by 35 and that the

ultimate unit base resistance was approximately equal to the number of blows per foot divided by 4. The correlation for side resistance was accurate over a rather wide range of penetrometer values.

Attempts were made to measure the shear strength profile at the Austin and San Antonio sites by using a borehole in situ shear strength device (Campbell and Hudson, 1969). However, reliable results could not be obtained.

A number of load tests were performed on the San Antonio test shaft over a period of several months. The side resistance in the top 10 to 15 feet was observed to fluctuate, and, on occasion, no side resistance was mobilized at all in that zone. This fact reflects the expansive nature of the overburden soil and the highly variable rainfall conditions at the test site.

Other Studies

Investigations of behavior of drilled shafts have been undertaken in other parts of the world. Mohan and Jain (1961) and Mohan and Chandra (1961) presented results of load tests on numerous uninstrumented shafts of varying geometry in the plastic black cotton clay soils of India. These investigators deduced approximately the same values for shear strength reduction factors (0.3 - 0.6) as those obtained in London Clay by conducting pullout tests and tests on shafts with false bottoms. The reported unconfined strengths for soil at the several black cotton soil test sites are near those for typical London Clay. Mohan and Jain report that sustained loads of one-third ultimate did not produce significant creep settlement over a two-year period. Mohan and Chandra report that increases in moisture content of two to three per cent were observed in the soil adjacent to the shafts. Mohan and Chandra also state that

retesting of shafts one year after initial loading did not yield increases in frictional resistance. This fact indicates that no increase in lateral earth pressure occurred during the period between the two tests, although the elapsed time between casting and initial testing was not reported.

Deb and Chandra (1964) gave results of further load tests in black cotton soils. They conducted long-term tests on normal belled shafts and belled shafts with side friction destroyed and concluded that the side friction was nearly constant with time at applied loads in the order of 30 to 60 per cent of ultimate. Hence, load shedding, or transferral of load from sides to base due to shear relaxation in the soil over a period of time, was shown to be quite minimal in black cotton soils at the magnitude of loading imposed. Long-term settlements were about 0.05 inches greater than short-term settlements at 30 per cent of ultimate load. The test shafts were located such that the stems were in a zone of expansion, while the bases were founded in soil below the depth of seasonal moisture change.

Komornik and Wiseman (1967) reported test results for shafts in layered sandy clay and sand. For the one shaft installed in the dry, complete failure was not achieved, but an α factor of at least 0.15 was indicated. The maximum factor would probably have been higher. The authors also discussed results of tests on a shaft installed with a bentonite slurry, which indicated that side friction was significant.

Fernandez-Renau (1965) gave results of pullout tests of shafts installed in sand by driving casing ahead of excavation, both with and without bentonite slurry in the hole. The shaft installed with bentonite actually yielded a higher pullout resistance than the shaft installed without bentonite.

The subject of the effect of using bentonite slurry on the side capacity of drilled shafts is treated in detail by Barker and Reese (1970). Although little is known about the effect of mud, the tests of Burland (1963), Komornik and Wiseman (1967), Fernandez-Renau (1965), and Chadeisson (1961) infer that it produces little, if any, maximum side shear reduction in either clay or sand when shafts are constructed in a manner which insures that the mud is completely displaced by the fluid concrete.

Woodward, Lundgren, and Boitano (1961) compared the ultimate side resistance of drilled shafts and driven piles in stiff, silty and sandy clay. They determined that driven pipe piles developed about 20 per cent greater side resistance than did drilled shafts, which had been installed with casing, but without mud. The α factors were near 0.5 for the drilled shafts.

Other informative test results were reported by Van Doren, et al., (1967) and Matich and Kozicki (1967). In the former tests, test shafts in weathered shale and overburden in Wichita, Kansas, were instrumented with hydraulic pressure cells in order to obtain load distribution. While complete failure was not achieved and some difficulty was experienced with the instrumentation, good bond was indicated in the shale and overburden, with a mobilized shear stress of at least 2000 psf indicated in the over-burden. In the latter tests, stems were rifled by cutting helical grooves in the sides of the boreholes, which were located in glacial till and in shale. The rifling was found to increase significantly the side resistance in both soils.

Bhanot (1968) conducted load tests on both model and full-sized drilled shafts in stiff, silty clay and glacial till. Model tests in the silt

indicated an α factor of about 0.8, while average values of 0.43 and 0.65 were found in clay and till, respectively, for full-sized shafts. In these tests, the various shafts were instrumented with bottomhole load cells similar to those used by Whitaker and Cooke (1966).

Bhanot characterized the α factor by the equation:

$$\alpha = \alpha_1 \alpha_2 \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5.2)$$

in which

α_1 = ratio of shear strength of soil around shaft after placing
concrete to that existing before placing concrete

α_2 = adhesion coefficient.

Bhanot quoted α_2 values of near unity for model shafts in compacted silt with a high degree of saturation (about 85 per cent). Skempton (1959), on the other hand, suggested that α_2 is approximately 0.8 for London Clay. Bhanot found in the tests in silty soil that α_1 ranged from near unity with an increase in moisture content immediately adjacent to the shaft of less than three per cent to about 0.75 with an increase of eight to nine per cent. The concrete used in the model test shafts had an average water-cement ratio of about 0.7. The actual concrete slump reportedly varied from shaft to shaft, with the lowest slump concrete producing the smallest moisture content increase in the soil. The load tests were con-ducted seven days after casting.

Bhanot also measured N_c values of 6.0 to 6.25 in the silt instead of the typical value of near 9 for saturated clays.

Watt, Kurfurst, and Zeman (1969) conducted load tests on full-sized shafts with false bottoms in plastic clay, silty clay, and glacial till. The α factors were about 0.3 in the plastic clay, and 1.0 in the silty clay and till with respect to the average of direct shear and torvane shear tests. In addition, soil shear strength was measured with a downhole shear device, which was affixed to the kelly bar of the drilling rig. The device was composed of parallel vertical concrete plates which were pressed against the sides of the borehole under a known pressure as soon as the borehole was excavated. The plates were then displaced vertically to obtain shear stress-displacement curves. The shearing resistance developed on the plates in all three strata depended almost directly on normal pressure applied. The peak stress for a normal stress of 10 psi most nearly correlated with the average peak stress measured in the load tests in all strata. This fact would imply that the average lateral pressure between concrete and soil was about 10 psi at the time of the load tests.

Tests in Sands and Silts

There is a notable lack of reported results of definitive field tests on drilled shafts in sands and silts. However, Martins (1963) reported results of compression tests of uninstrumented drilled shafts in sandy soil, from which he inferred that the side resistance is produced by soil in a fully active state along the periphery of the shaft. Another study has indicated that the average side resistance of short drilled shafts in partially saturated sandy silt in uplift is about one-half the product of the effective overburden pressure and the tangent of the angle of internal friction of the soil (Horner, 1969).

Summary

The test results outlined in this chapter reveal several important points concerning the behavior of axially loaded drilled shafts in stiff clay.

1. On the average, an α factor of 0.45 and bearing capacity factor of 9, determined with respect to the average UU tri-axial shear strength profile, appear to be valid for design, at least for short-term loading. Large variations of both factors have been observed.

2. Reduction of load transfer below a value equal to the shear strength of the soil is probably accompanied by an increase in moisture content in the soil adjacent to the shaft.

3. Little information is available concerning the effect of wet drilling. The few studies reported indicate little, if any, reduction in load transfer due to using drilling mud.

4. Settlements on the order of one-fourth inch are required to develop maximum load transfer in shafts two to four feet in diameter. Settlements of 5 to 20 per cent of the base diameter are required for full mobilization of base capacity. This implies that, at design load, the sides of a belled shaft may be in a failed condition. It also implies that the use of long, straight shafts, rather than shorter, belled shafts, is more effective in controlling initial settlement.

5. No useable information is available concerning variation in load transfer with depth. Knowledge of this variation would be helpful in determining effects of shaft geometry and variation in soil characteristics on the manner in which side shear is mobilized.

6. Little useable information is available on long-term behavior (for example, load shedding, or consolidation settlement). In-service shafts are usually so lightly loaded as to yield very little data on most aspects of long-term loading that may be of concern to the designer.