BEHAVIOR OF AXIALLY LOADED DRILLED SHAFTS IN BEAUMONT CLAY

PART ONE - STATE OF THE ART

by

Michael W. O'Neill
Lymon C. Reese

Research Report Number 89-8

Soil Properties as Related to Load Transfer
Characteristics of Drilled Shafts

Research Project 3-5-65-89

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by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

December, 1970
The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.
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
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 concisely 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
are also due to Miss Pamela Terwelp, Miss Cheryl Johnson, and Mrs. Eddie B. Hudepohl for their assistance in preparing the report. The authors also acknowledge the valuable assistance and advice given by Mr. Horace Hoy, Mr. H. D. Butler, and Mr. Gaston Berthelot, all of the Texas Highway Department, and by the maintenance personnel of District 12.

Michael W. O'Neill
Lyman C. Reese

December 1970
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LIST OF REPORTS

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


Report No. 89-3, "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. 89-5, "Load Distribution for a Drilled Shaft in Clay Shale," by Vasant N. Vijayvergiya, W. Ronald Hudson, and Lymon C. Reese, describes the development of instrumentation capable of measuring axial load distribution along a drilled shaft, the development, with the aid of full-scale load testing, of a technique of analysis of observed data, and the correlation of observed data with the Texas Highway Department cone penetration test.

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

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

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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 conducted 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 unconsolidated, 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.
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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 processed 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.
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<td>S1T2</td>
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<td>S1T3</td>
<td></td>
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<td>S2T2</td>
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<td>S4T1</td>
<td></td>
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<td>S4T2</td>
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<tr>
<td>S4T3</td>
<td></td>
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</tr>
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<td>S1T1</td>
<td></td>
</tr>
<tr>
<td>S1T2</td>
<td></td>
</tr>
<tr>
<td>S1T3</td>
<td></td>
</tr>
<tr>
<td>Test Shaft No. 2</td>
<td></td>
</tr>
<tr>
<td>S2T1</td>
<td></td>
</tr>
<tr>
<td>S2T2</td>
<td></td>
</tr>
<tr>
<td>Test Shaft No. 3</td>
<td></td>
</tr>
<tr>
<td>S3T1L1</td>
<td></td>
</tr>
<tr>
<td>S3T1L2</td>
<td></td>
</tr>
<tr>
<td>S3T1L3</td>
<td></td>
</tr>
<tr>
<td>Test Shaft No. 4</td>
<td></td>
</tr>
<tr>
<td>S4T1</td>
<td></td>
</tr>
<tr>
<td>S4T2</td>
<td></td>
</tr>
<tr>
<td>S4T3</td>
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Comparison of Test Results

Field Inspection and Moisture Migration Studies

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<tbody>
<tr>
<td>( A_B )</td>
<td>area of base</td>
</tr>
<tr>
<td>( A_c )</td>
<td>transformed cross-sectional area of stem (including effects of reinforcing steel)</td>
</tr>
<tr>
<td>( A_S )</td>
<td>peripheral area of stem</td>
</tr>
<tr>
<td>( A_S' )</td>
<td>nominal peripheral area of the stem excluding sections at the top and bottom, each equal in height to twice the stem diameter</td>
</tr>
<tr>
<td>( B )</td>
<td>diameter of loaded area</td>
</tr>
<tr>
<td>( B )</td>
<td>width of group of piles or shafts</td>
</tr>
<tr>
<td>( C' )</td>
<td>change in void ratio for increment of applied load</td>
</tr>
<tr>
<td>( C_c )</td>
<td>compression index</td>
</tr>
<tr>
<td>( C_e )</td>
<td>expansion index</td>
</tr>
<tr>
<td>( c' )</td>
<td>effective cohesion</td>
</tr>
<tr>
<td>( c_{base} )</td>
<td>average undrained cohesion of clay beneath base of shaft</td>
</tr>
<tr>
<td>( c_{mean} )</td>
<td>average undrained soil cohesion for fissured soil</td>
</tr>
<tr>
<td>( c_{sides} )</td>
<td>average undrained cohesion of clay along sides of shaft</td>
</tr>
<tr>
<td>( c_u )</td>
<td>undrained cohesion</td>
</tr>
<tr>
<td>( c_v )</td>
<td>coefficient of consolidation</td>
</tr>
<tr>
<td>( D_r )</td>
<td>relative density</td>
</tr>
<tr>
<td>( d )</td>
<td>diameter of shaft or pile</td>
</tr>
<tr>
<td>( d_{stem} )</td>
<td>diameter of stem</td>
</tr>
<tr>
<td>( E_c )</td>
<td>Young's modulus of concrete</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$E_o$</td>
<td>slope of initial tangent to nonlinear soil stress-strain curve; circuit output</td>
</tr>
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</table>
| $E_o$/
<p>| $c_u$  | ratio of $E_o$ to half of maximum indicated undrained stress difference of clay |
| $e_{\text{corrected}}$ | 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, $p_o$ |
| $e'<em>o$ | void ratio after load increased to preconsolidation pressure, then decreased to overburden pressure in consolidation test |
| $e</em>{50}$ | 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 |
| $I_\rho$ | 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 |</p>
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<tr>
<td>l</td>
<td>length of shaft or pile</td>
</tr>
<tr>
<td>N</td>
<td>number of blows per foot for T.H.D. penetrometer</td>
</tr>
<tr>
<td>N&lt;sub&gt;c&lt;/sub&gt;, N&lt;sub&gt;q&lt;/sub&gt;, N&lt;sub&gt;γ&lt;/sub&gt;</td>
<td>bearing capacity factors</td>
</tr>
<tr>
<td>N&lt;sub&gt;q&lt;/sub&gt;</td>
<td>bearing capacity factor for sands</td>
</tr>
<tr>
<td>NMC</td>
<td>natural moisture content</td>
</tr>
<tr>
<td>P&lt;sub&gt;i&lt;/sub&gt;</td>
<td>point at center of i&lt;sup&gt;th&lt;/sup&gt; layer at which consolidation settlement is computed</td>
</tr>
<tr>
<td>p</td>
<td>factor relating penetrometer results to maximum unit side resistance</td>
</tr>
<tr>
<td>Δp</td>
<td>increment of applied pressure causing consolidation</td>
</tr>
<tr>
<td>p'</td>
<td>factor relating penetrometer results to unit base capacity</td>
</tr>
<tr>
<td>P&lt;sub&gt;c&lt;/sub&gt;</td>
<td>preconsolidation pressure</td>
</tr>
<tr>
<td>P&lt;sub&gt;i&lt;/sub&gt;</td>
<td>i&lt;sup&gt;th&lt;/sup&gt; point on load transfer or load distribution curve</td>
</tr>
<tr>
<td>P&lt;sub&gt;o&lt;/sub&gt;</td>
<td>overburden pressure, or initial effective vertical pressure at the center of the compressible layer</td>
</tr>
<tr>
<td>O(z)</td>
<td>function relating load in the shaft to depth</td>
</tr>
<tr>
<td>Q&lt;sub&gt;B&lt;/sub&gt;</td>
<td>total amount of load taken by the base</td>
</tr>
<tr>
<td>Q&lt;sub&gt;S&lt;/sub&gt;</td>
<td>total amount of load removed by the sides in shear</td>
</tr>
<tr>
<td>Q&lt;sub&gt;T&lt;/sub&gt;</td>
<td>applied load</td>
</tr>
<tr>
<td>(Q&lt;sub&gt;B&lt;/sub&gt;)&lt;sub&gt;ult&lt;/sub&gt;</td>
<td>ultimate base load</td>
</tr>
<tr>
<td>(Q&lt;sub&gt;S&lt;/sub&gt;)&lt;sub&gt;ult&lt;/sub&gt;</td>
<td>ultimate side load</td>
</tr>
<tr>
<td>(Q&lt;sub&gt;T&lt;/sub&gt;)&lt;sub&gt;ult&lt;/sub&gt;</td>
<td>ultimate load at top of pile or shaft</td>
</tr>
<tr>
<td>q</td>
<td>contact pressure</td>
</tr>
<tr>
<td>(q&lt;sub&gt;B&lt;/sub&gt;)&lt;sub&gt;ult&lt;/sub&gt;</td>
<td>unit ultimate bearing stress on the base</td>
</tr>
</tbody>
</table>
### 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
- \( S1T1, \text{etc.} \): 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
- \( w_z \): downward displacement at depth \( z \)
- \( z \): depth coordinate
- \( z \): generic depth
- \( \alpha \): shear strength reduction factor
- \( \alpha_{avg} \): average shear strength reduction factor over a specified length of shaft
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_{\text{min}} )</td>
<td>minimum shear strength reduction factor from a laboratory test series</td>
</tr>
<tr>
<td>( \alpha_{\text{peak}} )</td>
<td>( \alpha ) corresponding to peak side load avg</td>
</tr>
<tr>
<td>( \alpha_{\text{ult}} )</td>
<td>( \alpha ) corresponding to ultimate load avg</td>
</tr>
<tr>
<td>( \alpha_z )</td>
<td>shear strength reduction factor at depth ( z )</td>
</tr>
<tr>
<td>( \alpha_1 )</td>
<td>ratio of shear strength of soil around shaft after placing concrete to that existing before placing concrete</td>
</tr>
<tr>
<td>( \alpha_{11} )</td>
<td>that part of ( \alpha_1 ) due to softening because of migration of water from concrete into soil</td>
</tr>
<tr>
<td>( \alpha_{12} )</td>
<td>that part of ( \alpha_1 ) due to the shear strength reduction not accompanied by moisture migration (remolding, opening of surface fissures)</td>
</tr>
<tr>
<td>( \alpha_{13} )</td>
<td>that part of ( \alpha_1 ) due to surface effects and base-side mechanical interference</td>
</tr>
<tr>
<td>( \alpha_2 )</td>
<td>adhesion coefficient</td>
</tr>
<tr>
<td>( \bar{\alpha} )</td>
<td>average shear strength reduction factor over entire stem excluding top and bottom two diameters</td>
</tr>
<tr>
<td>( \beta )</td>
<td>settlement correlation coefficient, settlement interaction factor</td>
</tr>
<tr>
<td>( \gamma' )</td>
<td>effective unit of weight of soil</td>
</tr>
<tr>
<td>( \delta )</td>
<td>angle of friction between the soil and concrete</td>
</tr>
<tr>
<td>( \delta_S )</td>
<td>elastic compression of stem</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>strain, general</td>
</tr>
<tr>
<td>( \varepsilon_{\text{circuit}} )</td>
<td>circuit strain</td>
</tr>
<tr>
<td>( \varepsilon_1 )</td>
<td>axial strain in triaxial or unconfined compression test</td>
</tr>
<tr>
<td>( \varepsilon_{\text{steel}} )</td>
<td>strain in steel in longitudinal direction</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
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<tr>
<td>-----------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$\varepsilon_{2_{\text{steel}}}$</td>
<td>strain in steel in transverse direction</td>
</tr>
<tr>
<td>$\varepsilon_{50}$</td>
<td>strain corresponding to one-half of the principal stress difference at failure</td>
</tr>
<tr>
<td>$\mu \nu$</td>
<td>abbreviation for microvolts</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>$\xi$</td>
<td>settlement ratio</td>
</tr>
<tr>
<td>$\rho_B$</td>
<td>average settlement beneath loaded area</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>total compression of compressible layer</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>normal stress</td>
</tr>
<tr>
<td>$\sigma_v'$</td>
<td>vertical effective stress in the soil adjacent to the shaft</td>
</tr>
<tr>
<td>$\sigma_\Delta$</td>
<td>principal stress difference in a triaxial or unconfined compression test</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>maximum principal stress</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>minimum principal stress</td>
</tr>
<tr>
<td>$\phi$</td>
<td>angle of internal friction</td>
</tr>
<tr>
<td>$\phi'$ ($=\phi_d$)</td>
<td>effective angle of internal friction</td>
</tr>
<tr>
<td>$\phi_u$</td>
<td>undrained angle of internal friction</td>
</tr>
<tr>
<td>$\psi$</td>
<td>additional shear strength reduction factor for shafts installed in a processed hole</td>
</tr>
<tr>
<td>$\omega$</td>
<td>bearing capacity reduction factor for fissured clay</td>
</tr>
</tbody>
</table>
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.
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 compres­
sion 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 diam­
eters 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 com­
monly 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 con­
sequence of placing wet concrete against the soil and the potential
Axial Load

Diameter
18-36 inches
Typical

Reinforcing Steel

Side Resistance

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

Base Resistance

Depth Varies With Soil Conditions

Fig. 1.1. Typical Drilled Shaft
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 capacities 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, particularly 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 placement. 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 conditions 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) describes 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. Crab-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 instrumented 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.
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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.
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
Fig. 2.2. Belling Tool

a. Belling Tool in Open Position

b. Belling Tool in Closed Position for Lowering Into Borehole
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...
Cohesive Soil

a. Dry Drilling Through Cohesive Soil

b. Drilling Through Caving Soil Under Mud

c. Setting Casing

Fig. 2.3. Steps in Wet Drilling
d. Bailing Mud from Inside Casing

e. Dry Augering Below Cased Section

f. Belling

Fig. 2.3. Continued
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 counteracts 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 diameter 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 specified, 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 concrete 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 underreamed 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 concrete 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 placement of the concrete.

Concrete for drilled shafts should be of good quality, with a minimum 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 reinforcing 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.
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 displaces 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
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 concisely below.

**Advantages:**

1. One drilled shaft can be used in place of a pile group because the capacity of a single shaft may be equivalent 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 corrective 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 surrounding 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 distribution 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 $[(Q_T)_3 - (Q_T)_2]$ 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 \[ (Q_T)_3 - (Q_T)_2 \] 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 \( z \) 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

\[
w_{\bar{z}} = w_T - \int_{z=0}^{z=\bar{z}} \frac{Q(z)}{A_E} dz \quad \ldots \ldots \ldots \ldots \ldots \ldots (3.1)\]

in which

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

\( w_T = \) the downward displacement of the butt
\[ Q(z) = \text{the function relating load in the shaft to depth} \]

\[ \frac{AE}{c} = \text{the product of shaft cross-sectional area and modulus of elasticity} \]

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 \( z \) 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," $\alpha$. This factor is an important parameter which must be known in order to compute the frictional capacity of the shaft. The parameter $\alpha$ 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 $\alpha$ 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 $\alpha$ 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 maximum 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 supported 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 hypothetical 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
\[ Q_T = Q_S + Q_B \] \hspace{1cm} (3.2)

In which

\begin{align*}
Q_S & = \text{side resistance} \\
Q_B & = \text{base resistance}
\end{align*}

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

\[ (Q_T)_{\text{working load}} = \frac{(Q_T)_{\text{ult}}}{(F.S.)_{\text{shaft}}} = \frac{(Q_S)_{\text{ult}}}{(F.S.)_{\text{sides}}} + \frac{(Q_B)_{\text{ult}}}{(F.S.)_{\text{base}}} \] \hspace{1cm} (3.3)

In which

\begin{align*}
F.S. & = \text{factor of safety at working load} \\
ult & = \text{ultimate resistance values.}
\end{align*}
It is suggested that the working load be computed in two ways. First, $(Q_{T,\text{ult}})$ is determined from considerations of limiting equilibrium, discussed in the next chapter, and an overall $(F.S.)_{\text{shaft}}$ of 2 to 3 is applied to $(Q_{T,\text{ult}})$. Second, separate factors of safety of 1 and 3 are applied to $(Q_{S,\text{ult}})$ and $(Q_{B,\text{ult}})$, 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 $(Q_{S,\text{ult}})$ to be the residual side resistance rather than the peak resistance.

Methods for calculating $(Q_{S,\text{ult}})$ and $(Q_{B,\text{ult}})$ 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 algorithm 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 magnitude 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 information with the fewest assumptions regarding soil behavior.
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
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:

\[
(Q_T)_{ult} = (Q_S)_{ult} + (Q_B)_{ult}
\]  

(4.1)

in which

\[
(Q_T)_{ult} = \text{ultimate load at top of pile or shaft}
\]
**TABLE 4.1. REPRESENTATIVE ALLOWABLE BEARING AND FRICTION VALUES FOR DRILLED SHAFTS**

<table>
<thead>
<tr>
<th>Material</th>
<th>Allowable Stress (tsf)</th>
<th>Source of Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Sound Limestone (Chicago)</td>
<td>up to 200 (bearing)</td>
<td>Osterberg, 1968</td>
</tr>
<tr>
<td>Sound Limestone (Chicago)</td>
<td>120 (bearing)</td>
<td>White, 1967</td>
</tr>
<tr>
<td>Hardpan (Till) (Chicago)</td>
<td>6 - 12 (bearing)</td>
<td>Osterberg, 1968</td>
</tr>
<tr>
<td>Hardpan (Till) (Detroit)</td>
<td>25 (bearing)</td>
<td>Housel, 1969</td>
</tr>
<tr>
<td>Shale (Texas)</td>
<td>5 - 30 (bearing)</td>
<td>Texas Highway Department, 1964</td>
</tr>
<tr>
<td>Expansive Shales and Clay-Shales (Texas)</td>
<td>6 (bearing)</td>
<td>U.S. Army Engineer District, Fort Worth, Texas, 1968</td>
</tr>
<tr>
<td>Clay (Chicago)</td>
<td>Unconfined compressive strength (bearing)</td>
<td>Osterberg, 1968</td>
</tr>
<tr>
<td>Sound Limestone</td>
<td>18 (shear)</td>
<td>White, 1967</td>
</tr>
<tr>
<td>Average Limestone</td>
<td>14 (shear)</td>
<td>White, 1967</td>
</tr>
<tr>
<td>Poor Limestone</td>
<td>7 (shear)</td>
<td>White, 1967</td>
</tr>
<tr>
<td>Shale</td>
<td>0.8 - 3.0 (shear)</td>
<td>Texas Highway Department, 1964</td>
</tr>
</tbody>
</table>
\((Q_s)_{ult} = \) 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 individual factors of safety against side and base failure will be different because ultimate side and base loads are mobilized at different displacements.

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:

\[
(q_s)_{ult} = \alpha c_u + K_o \sigma_v' \tan \delta \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.2)
\]

in which

\(\alpha\) = cohesion reduction factor

\(c_u\) = undrained cohesion of the soil along the side of the shaft

\(\sigma'_v\) = 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

\(\delta\) = angle of friction between the soil and concrete
The determination of values for $\alpha$ 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 $\alpha$ can be taken to be approximately 0.45 in deep shafts in stiff clay with the understanding that the product $\alpha u$ should not exceed some set value in the range of 1500 to 2000 psf. The variation of $\alpha$ 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 fluctuation (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 $\alpha$ can approach zero. This fact must be taken into account when computing side resistance for design. Uncertainties regarding the depth of this effect have resulted in low allowables for $\alpha$ in short shafts. In many areas having highly active soils and unfavorable rainfall situations, $\alpha$ 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 granular 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 \( \sigma_v' \) 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 \( \sigma_v' \) to be the overburden pressure present at a depth of a few diameters.
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, \( \sigma_v \) 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.

\[
(q_s)_{ult} = (0.025)(10)^{1.5D} r
\]  

(4.3)
in which

\[ D_r = \text{relative density of the sand} \]

\[ (q_S)_{ult} = \text{ultimate unit side resistance in tsf} \]

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

The angle of skin friction \( \delta \) is taken as zero in designing shafts in clay. Experimentally, \( \delta \) has been found to be approximately equal to \( \phi' \), 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 \( (q_S)_{ult} \):

\[ (q_S)_{ult} = c' + K \sigma_v' \tan \phi' \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (4.4) \]

in which

\[ c' = \text{effective cohesion of the clay} \]

\[ \sigma_v' = \text{effective vertical stress due to overburden} \]

\[ \phi' = \text{effective angle of internal friction of the clay} \]

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:

\[
(q_B)_{ult} = f_1 c_u N_c + \sigma_v' N_q + f_2 \gamma' \frac{B}{2} \gamma \n\]

in which

- \((q_B)_{ult}\) = ultimate unit bearing stress on the base
- \(c_u\) = average undrained cohesion in the soil beneath the base
- \(\sigma_v'\) = effective vertical stress in the soil on the horizontal plane passing through the base
- \(\gamma'\) = effective unit weight of the soil
- \(B\) = diameter of the base
- \(f_1, f_2\) = base shape factors
- \(N_c, N_q, N_{\gamma}\) = 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 \(N_{\gamma}\) 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 \(\sigma_v'\) 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

\[
(q_B)_{ult} = c_u N_c + h\gamma' \n\]

(4.6)
The value of the base pressure at failure due to the weight of the shaft is approximately equal to \( hY' \), where \( h \) is the depth of the base. Thus the net bearing capacity is given by

\[
(q_B)_{\text{ult,net}} = c \frac{N}{u} \text{ c u} \quad (4.7)
\]

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 \( N_c \):

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

in which \( \frac{E_o}{c_u} \) 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 \( \frac{E_o}{c_u} \) 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:

\[
(q_B)_{\text{ult,net}} = 9 \frac{c_u}{u} \quad (4.7a)
\]

and

\[
(q_B)_{\text{ult}} = 9 \frac{c_u A_B}{u} \quad (4.7b)
\]

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 introducing a single factor $N_q^*$ which incorporates the shape factors, so that

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

and

$$(Q_B)_\text{ult} = \sigma_v' N_q^* A_B \quad (4.9a)$$
The quantity $\sigma'_v$ is the effective vertical stress at the base of the shaft. Emphasis should be placed on the fact that $\sigma'_v$ 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 $\sigma'_v$, 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 $\phi$ 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

\[
(Q_T)_{ULT} = A_S \alpha c_{SIDES} + 2\pi r K_0 \tan \delta \int_0^Z (\sigma'_V)_{SIDES} \, dz + A_B \left\{ c_{BASE} N_c + (\sigma'_V)_{BASE} N_q^* \right\}
\]

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

**SPECIAL FORMS**

<table>
<thead>
<tr>
<th>In Clay (Undrained Conditions)</th>
<th>In Sand (Drained Conditions, Straight Shafts*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( (Q_T)<em>{ULT} = A_S \alpha c</em>{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 \text{ psf} )</td>
<td>( (Q_T)_{ULT} = 2\pi r K_0 \tan \delta \int_0^Z (\sigma'_V) , dz + A_B (\sigma'<em>V)</em>{BASE} N_q^* ) where ( K_0 = 0.2 - 0.4 ) ( \delta = \phi_d ) ( (\sigma'_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'<em>V)</em>{BASE} ) = Same definition as above. ( N_q^* ) = Bearing capacity factor, function of ( \phi_d ). Numerical values given by Vesić (1963; 1967).</td>
</tr>
</tbody>
</table>

*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-Ø 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 unaffected 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>
<thead>
<tr>
<th>Procedure</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quick Load (QL)</td>
<td>Texas Highway Department (1962) Specification Item 405</td>
</tr>
<tr>
<td>Constant Rate of Penetration (CRP)</td>
<td>Texas Highway Department (1965) Special Provision to Specification Item 405</td>
</tr>
<tr>
<td>Cyclic Method</td>
<td>Fuller and Hoy (1970)</td>
</tr>
<tr>
<td>Equilibrium Method</td>
<td>Whitaker and Cooke (1961)</td>
</tr>
<tr>
<td>Pullout Tests</td>
<td>Van Weele (1957)</td>
</tr>
<tr>
<td></td>
<td>Mohan, Jain and Jain (1967)</td>
</tr>
<tr>
<td></td>
<td>May be conducted according to any of the above five procedures.</td>
</tr>
</tbody>
</table>
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 structural 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
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 identi-
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 (After Whitaker and Cooke, 1966)
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:

\[ \rho_B = \frac{qB}{Eo} \left(1 - \nu^2\right) I_\rho \]  

in which

- \( \rho_B \) = average settlement beneath loaded area
- \( q \) = contact pressure
- \( B \) = diameter of loaded area
- \( E_o \) = Young's modulus of soil
- \( \nu \) = Poisson's ratio of soil
- \( I_\rho \) = 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 \( I_\rho \) 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:

\[ \rho_B = I_\rho \frac{q}{(q_B)_{ult}} \frac{(q_B)_{ult}}{c_u} \frac{3B}{4 \left(\frac{E_o}{c_u}\right)} \]  

(4.10a)
by observing that $v = 0.5$ 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 $\frac{(q_B)^{ult}}{c_u}$ increases and $I_\rho$ decreases with depth, but that their product remains nearly constant at about 5.35. He also noted from the laboratory stress-strain curve that:

$$\frac{E_o}{c_u} = \frac{\Delta}{(\Delta)^{failure}} \left(\frac{(\Delta)^{failure}}{c_u}\right) = \frac{1}{\varepsilon} = \frac{2}{\Delta} \left(\frac{\Delta}{(\Delta)^{failure}}\right)$$

(4.11)

in which $\sigma_\Delta$ is the principal stress difference in a triaxial or unconfined compression test.

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

$$\frac{\sigma_\Delta}{(\Delta)^{failure}} = \frac{q}{(q_B)^{ult}}$$

(4.11a)

the following simple expression for settlement is obtained:

$$\rho_B = 2B\varepsilon$$

(4.10b)

Skempton further observed that the strain corresponding to one-half of the principal stress difference at failure, $\varepsilon_{50}$, 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:

\[
\rho_B = 4 \frac{Q_B}{(Q_B)_{ult}} B \varepsilon_{50}
\]

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 \( \varepsilon_{50} \) of 0.01.

Other investigators have used the theory of elasticity approach to arrive at expressions for \( \rho_B \) 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 Duncan (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, Vesic (1967) noted that:

\[
1 - \frac{v^2}{E_0} = \frac{1}{\beta(q_B)_{ult}}
\]

in which \( E_0 \) is the initial Young's modulus of sand in a triaxial compression test with appropriate confining pressure, and \( \beta \) is a settlement correlation coefficient. Experimentally, Vesic observed \( \beta \) 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:

\[
\rho_B = \frac{0.18 Q_B}{(1+D_r^2)^B (q_B)^{ult}} = \frac{0.14 Q_B}{(1+D_r^2)^B (q_B)^{ult}} \quad \ldots \ldots \ldots \quad (4.10d)
\]

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 (3-to-1 bell) 100 per cent
b. Long belled shaft (3-to-1 bell, bell diameter < 4 to 6 times depth of base) 60 to 90 per cent depending upon depth of bell
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, \( Q_{Sul} \), is 100 tons, and the base capacity, \( Q_{Bult} \), 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:

\[ \delta_s = \frac{Q_T + Q_B}{2A_E L_S} \]  

(4.13)

in which

\[ \delta_s \] = 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:

\[ \rho_c = \frac{C'}{1 + e_o} H \log_{10} \frac{p_o + \Delta p}{p_o} \]  

in which

- \( \rho_c \) = total compression of the layer
- \( H \) = thickness of the compressible layer
- \( p_o \) = initial effective vertical pressure at the center of the layer
- \( \Delta p \) = 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 application 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, \( p_0 \), 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, \( \Delta p \), 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 $e_0$ 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 $p_0$ (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 $e'_0$ should be used in place of $e_0$ 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):

$$T_z = \frac{\pi}{2} (d_{stem}) (2c'z + K_o \gamma z^2 \tan \phi')$$  \hspace{1cm} (4.15)
in which

\[ z = \text{distance below top of expansive layer} \]
\[ d_{\text{stem}} = \text{diameter of stem of drilled shaft,} \]

and all other terms are as previously defined.

Collins suggested a value of \( K_0 \) 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 con­
crete 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 founda­
tion 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 information, 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 capacity 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):

\[ \bar{\xi} = \sqrt{\frac{B}{B}} \] ............................ (4.16)

in which

\( \bar{\xi} = \text{settlement ratio} \)

\( B = \text{width of group} \)

\( b = \text{diameter of single shaft} \)
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 proportion 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 procedure 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, \( \beta \), 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

Load = \( p \)

\[ \text{Poisson's Ratio} = 0.5 \]

(After Poulos, 1968)

\[ \frac{s}{d} = \frac{\text{spacing}}{\text{diameter}} \]

\[ \frac{l}{d} = \frac{\text{length}}{\text{diameter}} \]
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 $\beta$ 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 settlement 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 insufficient 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 5.1. SUMMARY OF FULL SCALE LOAD TEST RESULTS

<table>
<thead>
<tr>
<th></th>
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<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>LOCATION</td>
<td>Southall and Barnet, London Area, England</td>
<td>Horsham, Sussex, England</td>
<td>Boston, Massachusetts, USA</td>
<td>Boston, Massachusetts, USA</td>
<td>College Station, Texas</td>
</tr>
<tr>
<td>TESTING METHOD</td>
<td>Quick; Loads Applied in Failure within Several Hours</td>
<td>Cyclic; Loads Maintained until Equilibrium Reached in Each Cycle; Tests Required 3 Days per Shaft.</td>
<td>Cyclic; Loads Maintained Several Days Each Cycle</td>
<td>Statics; Long-term Design Loads Maintained 5 Years; 8 Months; Also Short-term W.L.</td>
<td>Maintained Load Method; Tests Required 4.5 Days. Also a Few Pullout and Quick Tests.</td>
</tr>
<tr>
<td>SOIL DESCRIPTION</td>
<td>London Clay, Venezolana, Sandy Clay, Mixed Material; No Ground Water; Shear Strength Varies from 0.5 to 1.5 psi at 0° to 25° - 30° Constant Below 30°.</td>
<td>London Clay; Topsoil; Shear Strength Varies from 0.4 to 1.5 psi at 0° to 5°.</td>
<td>London Clay; Topsoil; Shear Strength Varies from 0.4 to 1.5 psi at 0° to 5°.</td>
<td>London Clay; Typical; Shear Strength about 1.1 psi in Test Zone Measured on Uncrushed Core Samples.</td>
<td>London Clay Clays (not Given). Shear Strength about 0.7 psi at 0°. Avg. Shear Strength of 0.3 psi along sides.</td>
</tr>
<tr>
<td>NUMBER AND SIZE OF SHAFTS</td>
<td>Straight and Bellied</td>
<td>Straight and Bellied</td>
<td>Straight and Bellied</td>
<td>Bellied</td>
<td>Bellied</td>
</tr>
<tr>
<td>BELLED</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>CONSTRUCTION PROCEDURE</td>
<td>Borroughs Drill with Both Derrick-Type Hand Augers and Hansford Power Augers, Dry Process; Required 1 to 3 Days to Install Each Shaft with Hand Augers. 45 Minutes - 4 Hours with Power Augers.</td>
<td>Presumably with Power Augers. Plate Loading Tests Conducted on Several Shafts in Shores. Hence, Shear Presumably 1.2 psi (1.0 psi) for Periods of Time Longer than Normal.</td>
<td>Power Auger with Clamping No. 1 Auger. Hand Auger to Finished Depth (10') Three Shafts Overlapped in Wet Holes.</td>
<td>Mechanical Auger to B. No Mud. Side of Hole Not Considerable Strength. 4' Water in Shore of Borehole. No Information Presumably Completed in One Cycle.</td>
<td>All 6' and 7' Shafts Head Augered. Others Installed with Power Augers Dry Process in Each Case.</td>
</tr>
<tr>
<td>AVERAGE SHEAR STRENGTH REDUCTION FACTOR AT ULTIMATE LOAD (SOL TEST TO WHICH REFERRED IN PARENTHESES)</td>
<td>0.21 - 0.42 at Southall Water - Current Rule: 0.4 - 0.46 at Boston Water - Current Rule: 0.5 - 0.6. (Average of 12 Tests and Uncrushed Tests)</td>
<td>0.64 - 0.74 as referred to Average Soil Strength. 0.64 at Results for Equivalence of Minimum Soil Strengths (Triaxial Compression Tests)</td>
<td>0.6 at 0.5 psi cross-strength computed by Harris. Probably high, but with low values for Short-Term Values.</td>
<td>Not Measured.</td>
<td>Not Measured.</td>
</tr>
<tr>
<td>BEARING CAPACITY FACTOR N, (SOL TEST TO WHICH REFERRED IN PARENTHESES)</td>
<td>Not Measured in Shafts; but 0.97 Obtained for Plate Loading Tests in Benchmark. (Average of 12 Tests and Uncrushed Tests)</td>
<td>Not Measured in Shafts; but Factors Varying from 0.97 to 1.6. Measured in Plate Loading Tests in Lined Boreholes. (Average of Triaxial Tests Benchmark Plates)</td>
<td>Not Measured</td>
<td>Not Measured</td>
<td>Not Measured</td>
</tr>
<tr>
<td>LOAD DISTRIBUTION INFORMATION</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
</tr>
<tr>
<td>TIME EFFECT</td>
<td>No Increase in Resistance with Time up to 10 Months</td>
<td>No Increase in Resistance with Time up to 10 Months</td>
<td>No Increase in Resistance with Time up to 10 Months</td>
<td>No Increase in Resistance with Time up to 10 Months</td>
<td>No Increase in Resistance with Time up to 10 Months</td>
</tr>
<tr>
<td>MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.</td>
<td>4.3 (2%) to 7% Increase in Moisture Content in Soil within 2 inches of Shaft - Soil Interface of Southall Higher Increases of Greater Depths. 2.1 Also Tested Driven Plates of Horsham Site. Great Higher Residual Values by Factor of 2. Lower Values in Tests Attributed to Sinking of Soil by Migration of Water from Concrete.</td>
<td>These Tests Included in Skempton (1955) Review. Stemplen Standards Shear Strength Values Used by Investigators for Low Knauf Green AV Values Replaced by Skempton 0.56 - 0.60.</td>
<td>After 3 Years 8 Months, All Shafts Loaded in Approximately Twice Design Load. The Measured Settlement to this Loading Increment was Very Consistent in Corresponding Settlement of Short-Term Shaft Tests. Indicate Pronounced Loading Not Detrimental to Load-Settlement Behavior.</td>
<td>1) Good Agreement Between Quick and W.L. Tests for Small Diameter Shafts. Not so Good for Large Diameter Shafts. 2) Extracted Shafts Held 1/2 to 2/3 Soil Adhering to Sides. 3) In Model Studies, the Moisture Content Increase in Soil Adjacent to Shafts Except at Very Low Initial Moisture Content. 4) Pullout Capacity was 40%-70% of Compressive Capacity.</td>
<td>1) Good Agreement Between Quick and W.L. Tests for Small Diameter Shafts. Not so Good for Large Diameter Shafts. 2) Extracted Shafts Held 1/2 to 2/3 Soil Adhering to Sides. 3) In Model Studies, the Moisture Content Increase in Soil Adjacent to Shafts Except at Very Low Initial Moisture Content. 4) Pullout Capacity was 40%-70% of Compressive Capacity.</td>
</tr>
</tbody>
</table>
TABLE 5.1. (Continued)

<table>
<thead>
<tr>
<th>INVESTIGATOR(S) &amp; REFERENCE</th>
<th>LOCATION</th>
<th>DATE OF TEST(S)</th>
<th>TESTING METHOD</th>
<th>SOIL DESCRIPTION</th>
<th>NUMBER AND SIZE OF SHAFTS</th>
<th>CONSTRUCTION PROCEDURE</th>
<th>INSTRUMENTED IF SO, HOW?</th>
<th>AVERAGE SHEAR STRENGTH REDUCTION FACTOR @ULTIMATE LOAD</th>
<th>BEARING CAPACITY FACTOR Nq (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)</th>
<th>SETTLEMENT TO PRODUCE SIDE FAILURE</th>
<th>LOAD DISTRIBUTION INFORMATION</th>
<th>TIME EFFECT</th>
<th>MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mohan B Join (1961)</td>
<td>Jodhpur, India</td>
<td>1957 - 1958</td>
<td>Short-term Pullout and Compression Time Intervals for Loading not Specified</td>
<td>Black Cotton Soil; Shear Strength about 1 - 2 ft</td>
<td>12'-9&quot; to 12'-15&quot; dia., 6'-12' depth</td>
<td>Used Hand Spud Augers and Heradel Hand Dredgers</td>
<td>No</td>
<td>0.3 (Average of Unconfined Compression Tests)</td>
<td>8 - 9 for Straight Shafts, 7 - 4 for Bellied Shafts</td>
<td>Approximately 0.25&quot;</td>
<td>None</td>
<td>None</td>
<td>No Long-term Settlement on Shaft Loaded to Vc Ultimate Capacity for Two Years</td>
</tr>
<tr>
<td>Mohan B Chandra (1961)</td>
<td>Pune, Bombay, Surat and Jodhpur, India</td>
<td>Late 1950's</td>
<td>Short-term Pullout, Compression and Cyclic Time Intervals for Loading not Specified</td>
<td>Black Cotton Soil; Shear Strength about 0.7 - 1.6 ft</td>
<td>6' to 12' dia., 9' to 12' depth</td>
<td>Not Specified</td>
<td>No</td>
<td>0.45 - 0.54 (Average of Unconfined Compression Tests)</td>
<td>0.3 - 0.6, with 0.45 Average</td>
<td>Not Given</td>
<td>Not Given</td>
<td>0.4&quot; for two Shafts</td>
<td>No Significant Change in Capacity on Retesting Several Shafts</td>
</tr>
<tr>
<td>Skempton (1959)</td>
<td>Ten Sites in London Area, England</td>
<td>1950 - 1959</td>
<td>Varied</td>
<td>London Clay Typical; Shear Strength Varies from 0.4 ton of Top of Clay to 2.5 ton of 50 ft depth</td>
<td>12'-9&quot; to 12'-15&quot; dia., 6'-12' depth</td>
<td>Not Specified</td>
<td>No</td>
<td>0.49 - 0.52 (Average of Triaxial Tests)</td>
<td>0.8, Peak</td>
<td>Not Given</td>
<td>Not Given</td>
<td>Not Given</td>
<td>Not Given</td>
</tr>
<tr>
<td>Woodward, Lundgren &amp; Baidoo (1961)</td>
<td>London, UK</td>
<td>Not Reported</td>
<td>Short-term M.L. to 1.5 Times Working Load Followed by C.R.P. To Failure.</td>
<td>London Clay Typical; Shear Strength about 1 ton of 50 ft</td>
<td>12'-9&quot; to 12'-15&quot; dia., 6'-12' depth</td>
<td>Not Specified</td>
<td>No</td>
<td>0.49 - 0.52 (Average of Unconfined Compression Tests)</td>
<td>0.8, Peak</td>
<td>Not Given</td>
<td>Not Given</td>
<td>Approximately 0.25&quot;</td>
<td>No Significant Change in Capacity on Retesting Several Shafts</td>
</tr>
<tr>
<td>Burdon, Butter B &amp; Duneman (1966)</td>
<td>Westfield, London Area, England</td>
<td>Not Reported</td>
<td>Short-term M.L. to 1.5 Times Working Load Followed by C.R.P. To Failure.</td>
<td>London Clay Typical; Shear Strength about 1 ton of 50 ft</td>
<td>12'-9&quot; to 12'-15&quot; dia., 6'-12' depth</td>
<td>Not Specified</td>
<td>No</td>
<td>0.49 - 0.52 (Average of Unconfined Compression Tests)</td>
<td>0.8, Peak</td>
<td>Not Given</td>
<td>Not Given</td>
<td>Not Given</td>
<td>No Significant Change in Capacity on Retesting Several Shafts</td>
</tr>
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<tr>
<td>DATE OF TEST (S)</td>
<td>Not Reported</td>
<td>Not Reported</td>
<td>Not Reported</td>
<td>1962 - 1963</td>
<td>1962 - 1963</td>
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<tr>
<td>SOIL DESCRIPTION</td>
<td>London Clay. Shear Strength Profile Not Given.</td>
<td>London Clay. Maximum Shear Strength along sides = 0.12 ton (Blackfriars), 1.1 ton (St Giles). Avg. Shear Strength Required Base = 20 ton (Blackfriars), 26 ton (St Giles).</td>
<td>London Clay. Typical Strength Profile. Minimum Strength Envelope varies from 5 to 8 ft.</td>
<td>London Clay. Typical. Shear Strength Variation. 0.7 ton of Surface of Clay to 18 ton of 60 ft.</td>
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<tr>
<td>NUMBER AND SIZE OF SHAFTS</td>
<td>2; (Stephenson) 4' x 6', 40 ft Depth</td>
<td>2; 4' x 6' and 5' x 6' shafts, 5' x 6' and 6' x 6' shafts, 50% penetration of clay.</td>
<td>7; 3.5' x 6' shafts, 4' x 6', 25' - 40' penetration of clay.</td>
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<tr>
<td>STRAIGHT BHELLED</td>
<td>0</td>
<td>0</td>
<td>See Remarks</td>
<td>5; 2.5' x 4', 10.5' x 50' penetration of clay.</td>
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<tr>
<td>AVERAGE SHEAR STRENGTH REDUCTION FACTOR Nc (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)</td>
<td>Not Given. Element Installed with Benche's Bridge and Nearly Identical Load Settle- ment Curve as Dry Element. Approximately Same De Fact or Implied for Each.</td>
<td>Approximately 0.2 of Blackfriars and 0.5 of St Giles. Low Shear Blackfriars are Attributed to Higher Soil Strength.</td>
<td>Not Tested to Failure, but De Probably about 0.6.</td>
<td>0.6 when Side of Benche's are Dry.</td>
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<tr>
<td>BEARING CAPACITY FACTOR Nq (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)</td>
<td>Not Given.</td>
<td>Not Tested at Blackfriars. St Giles Shaft not Completely Failed.</td>
<td>Not Given.</td>
<td>0.44. All Shafts. Little Variation with Depth or with Size of Base.</td>
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<tr>
<td>SETTLEMENT TO PRODUCE SIDE FAILURE</td>
<td>Approximately 0.2% for both Dry and Benche's coated Elements.</td>
<td>Not Given.</td>
<td>Not Given.</td>
<td>0.75 - All Shafts Maximum at Settlement at 10% - 20% of Base Diameter. Average UU Triaxial 9.0.</td>
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</tr>
<tr>
<td>LOAD DISTRIBUTION INFORMATION</td>
<td>None</td>
<td>Not Reported</td>
<td>None</td>
<td>Minimum Envelope of UU Triaxial Tests</td>
<td>Minimum Envelope of UU Triaxial Tests</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.</td>
<td>1) Generally, 5% - 15% increase in Moisture Content in Soil next to both Elements. Extent of Moisture Migration was 2 - 3 inches. 2) Using St. Giles' Shear Strength Profile for London Clay and Bearing Capacity Cotnent. G6 was Approximately 0.8.</td>
<td>Stron Gages Experienced Some Instability, but did Show Decreasing Load with Depth. Numerical Results Not Given.</td>
<td>1) Jacked between base blocks and sections of Precast Con- crete Shaft Liner Sealed to Soil to Obtain Frictional Resis- tance and Base Capacity at Several Levels. 2) Immediate Settlement Achieved in 2 Minutes for the Lower.</td>
<td>1) Side resistance developed was greater than fully loaded but reduced Shear Strength of Soil. 2) Proposed design methods not proven.</td>
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TABLE 5.1. (Continued)
### TABLE 5.1. (Continued)

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<tr>
<td>LOCATION</td>
<td>Jodhpur, Ujjar, Parse and Indore, India</td>
<td>Bradford, Nova Scotia</td>
<td>North Terri Ave., Idaho</td>
<td>Michigan, Kansas</td>
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<tr>
<td>DATE OF TEST ($)</td>
<td>Not Reported</td>
<td>1964</td>
<td>Not Reported</td>
<td>1966</td>
</tr>
<tr>
<td>TESTING METHOD</td>
<td>Long-term (2 1/2 years)</td>
<td>Measured Load and Short Term Measured Load</td>
<td>Cyclic</td>
<td>Measured Load</td>
</tr>
<tr>
<td>SOIL DESCRIPTION</td>
<td>Black Cotton Soil</td>
<td>Dense Glacial Till (CGL)</td>
<td>Slightly Sandy Clay</td>
<td>Soft, Sand and Silty Clay</td>
</tr>
<tr>
<td></td>
<td>Highly Expansive Clay</td>
<td>Approximately 30% Thick Overlying Weathered</td>
<td>Sides Some Sand Depos</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td></td>
<td>Shear Strength about 1/2 ft. Zone of Seasonal</td>
<td>Share Average Shear Strength, Not Till, 40% till;</td>
<td>Overlying Weathered to</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td></td>
<td>Moisture Change to 12 Depth</td>
<td>for Shafts, 2.5 ft.</td>
<td>Sides Some Sand Depos</td>
<td>Soft, Sand and Silty Clay</td>
</tr>
<tr>
<td>NUMBER AND SIZE OF SHAFTS</td>
<td>6: 9&quot;-13', 6'-12'</td>
<td>4.105 m, 12&quot;, 40'</td>
<td>Depth in Soil and Sand</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2' Width, 20&quot; Dia</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10' Width, 15% Dia</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15' Depth, One Double Bell</td>
<td>Overburden 15 Feet Thick</td>
</tr>
<tr>
<td>CONSTRUCTION PROCEDURE</td>
<td>Method Not Specified</td>
<td>Mechanical Boring, Two Shafts</td>
<td>Mechanical Auger Boring</td>
<td>Mechanical Auger Boring</td>
</tr>
<tr>
<td></td>
<td>Presumably Portable Hand Auger</td>
<td>Measured Length, Two Shafts and Riffled Borrows with a</td>
<td>Through Overburden Procedure</td>
<td>Through Overburden Procedure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Specially Grooved 9&quot; Deep with</td>
<td>with Basaltic Shelly, Clay, Basalt</td>
<td>Through Overburden Procedure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1&quot; Pipe</td>
<td>and Silt, Est.</td>
<td>Basalt, Overburden 15 Feet Thick</td>
</tr>
<tr>
<td>INSTRUMENTED? IF SO, HOW?</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes Hydraulically Pressured Cells</td>
</tr>
<tr>
<td>AVERAGE SHEAR STRENGTH REDUCTION FACTOR δζ ULTIMATE LOAD (SOIL TEST TO WHICH REFERRED IN PARENTHESES)</td>
<td>Approximately 0.5: All Shafts</td>
<td>Certified: Tests Not Conducted to Failure, but δζ &lt; 0.3</td>
<td>Complete Failure Not Achieved</td>
<td>Complete Failure Not Achieved, but of Least 500 psi Developed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>in Till Based on Basis of</td>
<td>in Soil Fracture in Clay Than</td>
<td>in Soil Fracture in Clay Than</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Load Settlement Curve</td>
<td>δζ &gt; 0.17</td>
<td>δζ &gt; 0.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Riffled: Approximately 1.0</td>
<td>Complete Failure Not Achieved</td>
<td>Complete Failure Not Achieved, but δζ &lt; 0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m of Loads at 25&quot; Dirt Dia.</td>
<td>in Soil Fracture in Clay Than</td>
<td>in Soil Fracture in Clay Than</td>
</tr>
<tr>
<td></td>
<td></td>
<td>in Bore, 1/2&quot; Dia</td>
<td>δζ &gt; 0.17</td>
<td>δζ &gt; 0.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average of 20 Unfractured and 3 Fractured Tests</td>
<td>Average of 30 Unfractured and Fractured Tests</td>
<td>Average of 30 Unfractured and Fractured Tests</td>
</tr>
<tr>
<td>BEARING CAPACITY FACTOR Nc (SOIL TEST TO WHICH REFERRED IN PARENTHESES)</td>
<td>Not Given</td>
<td>Not Obscured in Tests</td>
<td>Not Obscured in Tests</td>
<td>Not Obscured in Tests</td>
</tr>
<tr>
<td>SETTLEMENT TO PRODUCE SIDE FAILURE</td>
<td>Not Given</td>
<td>Complete Failure Not Achieved in Normal Tests in Riffled Shafts of 0.8&quot;-1.0&quot; Diameter</td>
<td>About 0.1&quot; Implied From Load Settlement Curve</td>
<td>Not Given</td>
</tr>
<tr>
<td>LOAD DISTRIBUTION INFORMATION</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>Load Decreased Slightly with Depth</td>
</tr>
<tr>
<td>TIME EFFECT</td>
<td>Settlements Stabilized in 1-3 months at 10-15 Times</td>
<td>Not Given</td>
<td>Not Given</td>
<td>Not Given</td>
</tr>
<tr>
<td></td>
<td>Working Load Long-term Speck only at About 0.09 Inches at 0.3 Times Ultimate Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS</td>
<td>Comparison of Normal Riffled Shafts with Bored Shafts with Side Friction Destroyed Indicating that Side Friction is Essentially Constant with Time at Loads of 0.3-0.6 of Ultimate or Long Term Tests</td>
<td>Author Discusses Initial and Final Settling of Nearby Site with Basaltic Shelly Technique</td>
<td>Test Results Indicate High Side Resistance</td>
<td>1) No Evidence of Plane of Weakness Between Shaft and Soil in Overburden where Shelly Was Used</td>
</tr>
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<thead>
<tr>
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<tbody>
<tr>
<td>LOCATION</td>
<td>Laskin Air Force Base, San Antonio, Texas</td>
<td>Austin, Texas</td>
<td>Edmonton, Alberta</td>
<td>Sacramento, San Francisco</td>
<td>San Antonio, Texas</td>
</tr>
<tr>
<td>TESTING METHOD</td>
<td>Short and Long Term Instrumented Load</td>
<td>Quick Tests</td>
<td>Monitored Load</td>
<td>Equilibrium Method</td>
<td>Quick Tests</td>
</tr>
<tr>
<td>SOIL DESCRIPTION</td>
<td>13 Test of 15 and 60 Natural Clayey and Clayey Sand and Overburden High Expansive Contain 25% Water in Rem. Average Soil Strength of Overburden 1 Ton at Initial Load 7 Ton.</td>
<td>1st Clay with Calcium Material and 6% Under Clay to 5% Over Clay. Overburden Soil Strength Devitrifying 2.5 Ton. No Water.</td>
<td>Top 20 Abundant Silty Clay Shear Strength 1 Ton; Under Clay 5% of Soil. Glacial Till Shear Strength 1 Ton, Except Stir.</td>
<td>60% Plastic Clay 15% - 15% with Shear Strength 1 Ton 30% with Shear Strength 0.5 Ton. Glacial Till 18% - 28% with Shear Strength 1.0 Ton.</td>
<td>Clayey Silt 15% - 20% Under Clay 30% - 40% with Shear Strength 0.5 Ton. Under Clay 40% - 50% with Shear Strength 0.5 Ton.</td>
</tr>
<tr>
<td>NUMBER AND SIZE OF SHAFTS</td>
<td>STRAIGHT 18±5' φ, 15' Depth, Base Reinforced Destroyed</td>
<td>1: 24' φ, 12' Depth 1: 24' φ, 12' Depth</td>
<td>1: 24' φ, 12' Depth 1: 24' φ, 12' Depth</td>
<td>1: 24' φ, 12' Depth 1: 24' φ, 12' Depth</td>
<td>1: 24' φ, 12' Depth 1: 24' φ, 12' Depth</td>
</tr>
<tr>
<td>BELLED</td>
<td>4 ± 10φ φ, 36' φ, 48' φ</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>CONSTRUCTION PROCEDURE</td>
<td>Power August with Coating Dry Process After Initial Load Testing, Site was Paved for One Year to Produce Soil Expansion.</td>
<td>Mechanical August Installed in the Dry or in Several Hours.</td>
<td>Mechanical August Installed in the Dry or in Several Hours.</td>
<td>Mechanical August: Drilled Below Water Table Without Use of Mud. Water in Longest Shaft: installation Required 4 Hours Per Shaft.</td>
<td>Power August: One Procedure, Infiltrated in One Day.</td>
</tr>
<tr>
<td>INSTRUMENTED SHALE TO WHICH REFERRED IN PARENTHESES</td>
<td>Yes, Bonded Strain Gages among Reinforcing Steel. Typical Strain Gages Installed Along Bore in Measure Lateral Pressure.</td>
<td>Yes, Concrete Embedment Gages, Gage Rods, Lateral Earth Pressure Cells at Several Levels.</td>
<td>Yes, All Shales Had Electronic Load Cells Near Beam, Generally Reliable Results, Also Used Stress Rods.</td>
<td>Not Reported.</td>
<td>Yes, Concrete Embedment Gages, Stress Rods, Lateral Pressure Gages.</td>
</tr>
<tr>
<td>AVERAGE SHEAR STRENGTH REDUCTION FACTOR AT ULTIMATE LOAD (SOIL TEST TO WHICH REFERRED IN PARENTHESES)</td>
<td>About 0.2 to 0.4 for Shale Shocks in Overburden, on the Order of 0.5 - 0.3 for Shale. Based on Short Term Testing. Average of UU Triaxial Tests.</td>
<td>0.55</td>
<td>0.45</td>
<td>0.3</td>
<td>Maximum of Approximately 800 Tons Transferred in Side Friction - 80% Not Computed. Since Undisturbed Samples Could Not Be Recovered for Entire Length of Embankment.</td>
</tr>
<tr>
<td>BEARING CAPACITY FACTOR</td>
<td>Ultimate Factor Not Clearly Defined.</td>
<td>9.2</td>
<td>Concrete Bearing Capacity 1.5 times Recommended.</td>
<td>None</td>
<td>None.</td>
</tr>
<tr>
<td>ISOLATED TEST TO WHICH REFERRED IN PARENTHESES</td>
<td>Lateral Earth Pressure Cells at Several Levels.</td>
<td>Average of Undrained Triaxial and Undrained Tests)</td>
<td>Lateral Earth Pressure Cells at Several Levels.</td>
<td>Not Given.</td>
<td>Not Given.</td>
</tr>
<tr>
<td>SETTLEMENT TO PRODUCE SIDE FAILURE</td>
<td>0.05 ± 0.1' in Shale Slides in Overburden. Not Clearly Defined for Shale.</td>
<td>Approximately 0.1'</td>
<td>0.025 ± 0.1' in Clay, Vial in Test 0.8 ± 0.15</td>
<td>Approximately 0.15 for Shell Embodied in Plastic Clay and Silty Clay. Not Clearly Defined for Others.</td>
<td>Very Little Load Transfer in Top of Shaft; Highest Load Transfer 1.4 φ in Clay - Shale Near Bore.</td>
</tr>
<tr>
<td>LOAD DISTRIBUTION INFORMATION</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
<td>None.</td>
</tr>
<tr>
<td>MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS, SPECIAL FEATURES, OTHER REMARKS.</td>
<td>1.0 to 1.5% Gases Indicated Tension During Curing. 2.5 Tons Tonnage on Only Four Shafts. 3. Tensile Stress in Order of 20% of Maximum Shear Produced During Consolidation Testing (Collated in Table). 4. Lateral Pressure Cells Indicated Lateral Pressure Coefficient Greater Than 1.0 for Shaft One Year After Pounding Site.</td>
<td>Tests Conducted Primarily to Evaluate Methods of Instrumentation.</td>
<td>1.1 Presents Results of Model Tests of Shafts in Soil. Rg About 0.6, ED About 0.5. 2.1 24 Hour Settlemnet Measurements Became &quot;Insignificant&quot; from Immediate Settlements of About 0.5&quot; of Plugging Load.</td>
<td>Borehole Direct Shear Device Used to Obtain Average Of Shear Stress for Various Normal Pressures against Walls of Borehole From Results. Authors Deduced Maximum Pressure of 1400 psi Acting Laterally During Actual Load Test.</td>
<td>Load Transfer and Load - Settlement Correlation Modeled on Data of Texas Highway Department Core Parameter Tests.</td>
</tr>
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</table>
(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 $\alpha$, the average side shear strength reduction factor. Before $\alpha$ 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 $\alpha$ 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 $\alpha$ 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 preceding 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 $\alpha$ 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 \( \alpha \) 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 \( \alpha \) 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 \( \alpha \) factors and \( N_c \) values reported in Table 5.1 were obtained by investigators who employed inconsistent testing methods, and who determined \( \alpha \) 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 uninstrumented drilled shafts at Southall and Barnet in London Clay in which they determined that \( \alpha \) 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 $\alpha$ was the water-cement ratio of the concrete mix: the higher the ratio, the lower the $\alpha$ value. The unusually low $\alpha$ 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 $\alpha$ 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 $\alpha$ 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 $\alpha$ of 0.45, unless the product of $\alpha$ 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 $\alpha$ 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 conditions 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 opposition 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 thereby have less influence than if the sides were drier. It was generally observed by Skempton that lower values of $\alpha$ 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 $\alpha$. 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 $\alpha$ 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
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 \( \alpha \) 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 \( \alpha \) 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 $\alpha$ 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 $\alpha$ 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 $\alpha$ 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 \( \alpha \) 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 \( \alpha \) values were calculated from this profile. They observed that the average \( \alpha \) 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" $\alpha$ factor should be used in the design of belled shafts.

A comprehensive series of load tests on instrumented shafts was conducted 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 $\alpha$ value of 0.45 which Skempton had proposed for design applications in London Clay. Whitaker and Cooke measured an average $\alpha$ 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 $\alpha$ factors might have been slightly higher had the tests been carried to failure under maintained load, but the differences would probably have been insignificant. 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:

\[
(Q_B)_{ult} = 9 \cdot \frac{A_B \cdot c_{\text{mean}}}{\omega}
\]  

(5.1)

in which

\[ \omega = \text{bearing capacity reduction factor for fissured clay} \]

(varies according to soil formation)

\[ A_B = \text{area of base} \]

\[ c_{\text{mean}} = \text{average soil cohesion beneath the base.} \]

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 \( \omega \). 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 \( \omega \) appropriate for the soil in question, would be a more reliable procedure. They suggested a factor of 0.75 for \( \omega \) 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 \( \alpha \) 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 distribution in one shaft. This was evidently the first time this feat was successfully 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 $\alpha$ factors in the order of 0.2 to 0.6 for the overburden 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 investigate 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 $\alpha$ 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.

Komorník 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 $\alpha$ 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 $\alpha$ 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 overburden. 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 \( \alpha \) 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 \( \alpha \) factor by the equation:

\[
\alpha = \alpha_1 \alpha_2
\]

(5.2)

in which

\[
\begin{align*}
\alpha_1 &= \text{ratio of shear strength of soil around shaft after placing concrete to that existing before placing concrete} \\
\alpha_2 &= \text{adhesion coefficient}.
\end{align*}
\]

Bhanot quoted \( \alpha_2 \) 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 \( \alpha_2 \) is approximately 0.8 for London Clay. Bhanot found in the tests in silty soil that \( \alpha_1 \) 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 conducted 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 $\alpha$ 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 down-hole 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 $\alpha$ factor of 0.45 and bearing capacity factor of 9, determined with respect to the average UU triaxial 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 usable 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.