Experimental Study of Drained Behavior of Drilled Shafts During Static Inclined Loading

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Experimental Study of Drained Behavior of Drilled Shafts During Static Inclined Loading

This report presents the results of an experimental study of the behavior of model drilled-shaft foundations in cohesionless soils under static inclined loading. It offers criteria for evaluating the influence of inclined loading on shaft capacity and displacements, as well as guidelines for drilled-shaft foundation design.

BACKGROUND Foundations for transmission line structures are commonly loaded at orientations ranging from inclined uplift to inclined compression. However, most knowledge of foundation behavior is restricted to axial uplift/compression and lateral/moment loading. Available solutions for inclined loads are semi-empirical and have been developed from laboratory model tests on jacked piles. Because little was known about the design of drilled-shaft foundations under inclined loading, design has had to be conservative to address this situation.

OBJECTIVES To conduct a comprehensive laboratory test program to examine the influence of inclined static loads on drilled-shaft foundations in cohesionless soils; to use these results to develop design recommendations.

APPROACH Researchers constructed model drilled-shaft foundations in three depth-to-diameter ratios in uniform cohesionless soils ranging from loose to dense. They used seven load inclinations, ranging from axial uplift to axial compression. All of the tests were loaded monotonically to failure and beyond. For convenience, the results were normalized by the test results in axial uplift.

RESULTS Test results showed that the capacity of drilled-shaft foundations is influenced significantly by load inclination. The normalized capacity generally increases with increasing shaft depth and soil density. On the basis of these results, design guidelines were developed to address loading combinations.

EPRI PERSPECTIVE These results will assist the foundation designer in analyzing and designing drilled-shaft foundations in cohesionless soils for static inclined loads. They will also assist designers in evaluating existing foundations for upgrading studies. Basic EPRI research on drilled-shaft foundations has been published in EPRI reports EL-2870 and EL-3771. Recent EPRI research on drilled-shaft foundations in undrained soils has been published in EPRI reports TR-100221 and TR-100223.
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Experimental Study of Drained Behavior of Drilled Shafts During Static Inclined Loading

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ABSTRACT

Drilled shafts are used commonly as the foundations for transmission line structures. These structures often impose inclined uplift and compression loads on their foundations. The available solutions for these inclined loads are semi-empirical and have been developed from laboratory model tests on jacked piles. Therefore, this study was conducted to investigate, through laboratory model testing, the response of straight, vertical drilled shafts to static inclined loads under drained conditions.

Fifty-eight laboratory tests on model drilled shafts were conducted as part of a parametric study of the following variables: (a) shaft geometry, (b) initial soil density, and (c) load inclination. Shafts were prefabricated and reusable with a diameter (B) of 52 mm (2.05 in) and depth to diameter (D/B) ratios of 3, 6, and 9. They were tested in uniform deposits of loose, medium dense, and dense sand. Loading modes included axial uplift and compression, lateral, and inclined uplift and compression. A total of seven load inclinations were investigated.

Loads were applied to the shafts at a constant displacement rate using a hydraulic actuator controlled by a closed-loop, electro-hydraulic control system. All tests were fully automatic and controlled by a mini-computer. Measurements of load, displacement, and rotation were taken automatically with precision electronic transducers linked to a computer-controlled, high-speed data acquisition system.

The drilled shaft load versus displacement response in inclined uplift was smooth and nonlinear. Increasing soil density and shaft depth both resulted in higher initial stiffness and higher loads for the same displacement. When the applied load deviated less than 45 degrees from the vertical, the load-displacement curves had a clearly defined peak followed by post-peak softening. As the load inclination increased, the curves became flatter. In dense sand, however, the response was characterized by post-peak softening regardless of the load
inclination. Shaft geometry had no apparent influence on the point of rotation.

For inclined compression, increasing soil density resulted in higher initial stiffness and higher loads for the same displacement, as well as a higher point of rotation. The load-displacement curves were smooth and nonlinear, with no apparent yielding, except in dense sand where a peak was followed by post-peak softening. The less the applied load deviated from the vertical, the lower was the point of rotation. Shaft geometry had no apparent influence on the rotational pattern.

The lateral/moment limit method was used to interpret the failure loads from inclined load tests. These interpreted failure loads were normalized by the corresponding axial uplift failure loads. With this normalization, very good correlations were established between the normalized interpreted failure load and load inclination. These correlations were nonlinear, but the linearity of the fit increased with increasing depth to diameter (D/B) ratio. To interpolate the results for shaft geometries other than those analyzed in this study, the correlations between the normalized interpreted failure load and D/B were provided for various load inclinations.
ACKNOWLEDGMENTS

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SYMBOLS

ENGLISH LETTERS - UPPER CASE

\( A_t \) - tip area

\( \text{ADOR} \) - apparent depth of rotation

\( AC \) - axial compression

\( AU \) - axial uplift

\( B \) - shaft diameter

\( \text{CID} \) - consolidated isotropic drained triaxial test

\( \text{COV} \) - coefficient of variation (standard deviation divided by mean)

\( D \) - shaft depth

\( D_r \) - relative density

\( \text{DCDT} \) - direct current differential transducer

\( DS \) - direct shear

\( DSS \) - direct simple shear

\( H \) - lateral load

\( H_x, H_y \) - horizontal loads along \( x \) and \( y \) axes

\( H_u \) - lateral capacity

\( \text{IC} \) - inclined compression

\( \text{IU} \) - inclined uplift

\( K_o \) - in-situ coefficient of horizontal soil stress

\( K_{\text{onc}} \) - \( K_o \) for normally consolidated soil

\( K_{\text{opu}} \) - \( K_o \) for primary unloading

\( K_{\text{opr}} \) - \( K_o \) for primary reloading

\( K_{\text{bu}}, K_{\text{cu}} \) - inclined uplift coefficients

\( K_c, K_q \) - yield stress coefficients for lateral loading
$K_p$ - Rankine coefficient of maximum passive soil stress
$L$ - lateral
$L_1$ - elastic limit load
$L_2$ - failure threshold load
$LVDT$ - linear variable differential transducer
$M$ - moment
$M_x, M_y$ - moments with respect to $x$ and $y$ axes
$M_t$ - torsional moment
$N_q$ - bearing capacity factor
$N_{q\omega}$ - bearing capacity factor at $\omega$
$OCR$ - overconsolidation ratio
$OCR_{max}$ - maximum overconsolidation ratio
$P$ - applied load
$P_0$ - axial uplift load; vertical uplift component of inclined load
$P_{0f}$ - interpreted axial uplift failure load
$P_{15f}$ - interpreted net inclined resultant failure load for $\psi = 15$ degrees
$P_{45f}$ - interpreted net inclined resultant failure load for $\psi = 45$ degrees
$P_{90}$ - horizontal load; horizontal component of inclined load
$P_{90f}$ - interpreted lateral failure load
$P_{135}$ - inclined compression load for $\psi = 135$ degrees
$P_{135f}$ - interpreted inclined compression failure load for $\psi = 135$ degrees
$P_{155}$ - inclined compression load for $\psi = 155$ degrees
$P_{165}$ - inclined compression load for $\psi = 165$ degrees
$P_{165f}$ - interpreted inclined compression failure load for $\psi = 165$ degrees
$P_{180}$ - axial compression load; vertical compression component of inclined load
$P_{180f}$ - interpreted axial compression failure load
$P_{\psi}$ - total applied inclined load
$P_{\psi f}$ - interpreted net resultant inclined uplift failure load; interpreted inclined compression failure load
$P_{\psi h}$ - horizontal component of inclined load

$P_{\psi n}$ - net resultant inclined uplift load

$P_{\psi v}$ - total vertical component of inclined uplift load

$P_{\psi VN}$ - net vertical component of inclined uplift load

$P_{\psi u}$ - inclined capacity

$P_{\psi uh}$ - horizontal component of inclined capacity

$P_{\psi uv}$ - vertical component of inclined capacity

PSC - plane strain compression

PSE - plane strain extension

$Q_c$ - capacity in axial compression

$Q_s$ - side resistance

$Q_{sc}$ - side resistance in compression

$Q_{su}$ - side resistance in uplift

$Q_{s6}$ - side resistance for inclined loading

$Q_t$ - tip resistance

$Q_{tc}$ - tip resistance in compression

$Q_{tu}$ - tip resistance in uplift

$Q_{tw}$ - tip load

$Q_{twv}$ - tip load vertical component

$Q_u$ - capacity in axial uplift

$\Delta Q_u$ - increase in axial uplift capacity because of inclined loading

R - concentrated tip load during inclined loading

S.D. - standard deviation

T - resultant of soil lateral stresses because of inclined loading

TC - triaxial compression

TE - triaxial extension

V - vertical load; output voltage

W - shaft weight
ENGLISH LETTERS - LOWER CASE

a - g  - regression coefficients
h     - sand drop height
i_\psi - pile inclination factor
m     - passive stress mobilization factor; regression exponent
n     - number of samples
p     - surface load (ground surcharge)
P_u   - soil yield stress
r_2   - coefficient of determination
t     - vibration time
w     - weight of soil in density scoop
x,y   - coordinate axes; transformed linearized regression variables
z     - depth
z_r   - depth of point of rotation

GREEK LETTERS  [Note that all angles are in degrees.]

\gamma - soil unit weight
\delta - displacement; interface friction angle
\delta_0 - vertical uplift displacement; vertical uplift component of inclined displacement
\delta_{90} - horizontal displacement; horizontal component of inclined displacement
\delta_{135} - resultant displacement for initial load inclination \psi = 135 degrees
\delta_{165} - resultant displacement for initial load inclination \psi = 165 degrees
\delta_{180} - vertical compression displacement; vertical compression component of inclined displacement
\delta_\psi - resultant displacement for initial load inclination \psi
\phi    - effective stress friction angle
\phi_{DS} - effective stress friction angle in direct shear
\phi_{TC} - effective stress friction angle in triaxial compression
\phi_{TE} - effective stress friction angle in triaxial extension
\sigma_h - horizontal effective stress

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\( \sigma_v \) - vertical effective stress
\( \sigma_{vo} \) - effective overburden stress
\( \theta \) - shaft rotation
\( \tau_f \) - shear stress on Rankine wedge plane
\( \psi \) - applied load inclination measured from upward vertical axis
\( \omega \) - tip load inclination
Section 1

INTRODUCTION

Drilled shafts are used commonly as the foundations for transmission line structures. These structures often impose inclined loads on their foundations. However, foundation response to these loads is not well understood. Current knowledge is focused on simple coupling of the two limit cases, axial and lateral, while some research efforts are aimed at the development of semi-empirical design procedures. In addition, the existing data base has been derived from model tests on flexible piles and may not be readily applicable to drilled shafts. Therefore, this study was conducted to investigate, through model testing, the response of straight, rigid drilled shafts to static inclined loads under drained conditions.

This section presents a brief description of drilled shaft foundations and their response to axial and lateral loads, with emphasis on transmission line structures. General loading characteristics are included to assess the relative importance of inclined loading. Then the scope of this study is presented.

DRILLED SHAFT FOUNDATIONS

Drilled shafts are constructed by augering a cylindrical excavation and placing concrete in the excavation. The shaft may be reinforced or unreinforced. Typical sizes for transmission line structures range from 0.6 to 2.5 m (2 to 8 ft) in diameter with depth to diameter (D/B) ratios of 3 to 10. The shafts also may be belled in some conditions. However, in this study, straight-sided shafts are addressed. Techniques for drilled shaft construction, as well as appropriate equipment, are described elsewhere (1).

Axial Capacity

Many models have been proposed during the past thirty years for evaluating the capacity of drilled shafts under axial loads. Research at Cornell University (2) utilized a fundamental approach that resulted in reliable and general design procedures. In this approach, the capacity of a drilled shaft includes the side resistance \( Q_s \), tip resistance \( Q_t \), and weight \( W \), as illustrated in Figure 1-1.
1-1. The compression capacity ($Q_c$) is:

$$Q_c = Q_{sc} + Q_{tc} - W$$  \hspace{1cm} (1-1)

in which the subscript $c$ refers to compression. The uplift capacity ($Q_u$) is:

$$Q_u = Q_{su} + Q_{tu} + W$$  \hspace{1cm} (1-2)

in which the subscript $u$ refers to uplift. Methods for predicting the correct failure mode and the side and tip resistances are given elsewhere (e.g., 2, 3).

Lateral Capacity

A recent study (4) has provided a detailed summary of the available analytical methods for predicting the ultimate lateral capacity under drained loading. These methods represent upper bound solutions resulting from static equilibrium of the foundation for an assumed soil stress profile at failure. Failure refers to either complete soil yield or structural failure of the shaft.

The yield stress of sand ($p_u$) can be defined as the maximum average horizontal soil resistance at the foundation-soil interface. According to Hansen (5), soil yield stresses develop differently at shallow, moderate, and large depths. For shallow depths, Hansen considered the difference between the passive and active stress coefficients corresponding to a rough wall that is being translated horizontally. At moderate depths, the soil yield stress is calculated from the static equilibrium of a Rankine passive wedge bounded by two vertical planes along which at-rest soil stresses develop (Figure 1-2). At large depths, the

![Diagram](image)

**Figure 1-1. Equilibrium Forces Acting on Drilled Shaft Foundations**

Source: Kulhawy, et al. (2), pp. 8-2 and 9-2.
yield stresses are calculated assuming a horizontal failure plane.

Hansen proposed a single general equation that can be used to calculate the resultant (passive minus active) stress per unit projected area of the shaft for all three depth models:

\[ p_u = \sigma_{vo} K_q + c K_c \]  

(1-3)

in which \( \sigma_{vo} \) = effective overburden stress, \( c \) = soil cohesion, and \( K_q \) and \( K_c \) = factors in Figure 1-3. For a rigid shaft and the general yield stress distribution given in Figure 1-4, the ultimate capacity \( (H_u) \) can be calculated from the static equilibrium of forces acting on the shaft, as follows:

\[ H_u = \int_{0}^{z_r} p_u B \, dz - \int_{z_r}^{D} p_u B \, dz \]  

(1-4)

in which \( z_r \) = point of zero stress (point of rotation), \( B \) = shaft diameter, and \( D \) = shaft depth.

Agaiby, et al. (4) showed that the ultimate capacity calculated using Hansen's method compares well with the hyperbolic capacity, which can be obtained by fitting a hyperbola to the load-displacement curve, as discussed in Section 5. The comparison yielded an overall ultimate to hyperbolic capacity ratio of 1.02.
Figure 1-3. Hansen’s Yield Stress Coefficients

Figure 1-4. Yield Stress Distribution for Calculation of Ultimate Capacity

Interpretation of Failure Load from Axial and Lateral Load Tests
Recent research (6, 7) has shown that there are at least 41 different methods for interpreting the results of axial compression tests on deep foundations. These methods fall into three broad categories: settlement limitations,
graphical construction, and mathematical models. Based on evaluation of load test results, Hirany and Kulhawy (6, 7) proposed a new interpretation method for drilled shafts and compared it against selected existing methods. By analyzing the load-displacement response in axial compression, as given in Figure 1-5, they proposed that the failure threshold, i.e., the load at the initiation of the final linear region ($L_2$), be adopted as the failure load. Using a database derived from full-scale field load tests, they found that $L_2$ typically occurs in compression at a displacement equal to about four percent of the foundation diameter. This improved method is independent of individual scale and judgment, is not based on extrapolation, accounts for foundation diameter, and takes into consideration the shape of the load-displacement curve and ratio of change in load to change in displacement.

The general shapes of load-displacement curves for axial uplift and compression tests are similar. However, since the maximum uplift resistance of deep foundations generally is independent of the foundation diameter (2, 8), the method proposed for interpreting failure loads for axial compression tests can not be applied for axial uplift tests. The failure load for axial uplift tests can be interpreted readily for load-displacement curves of type A or B in Figure 1-6 but, in the case of a type C curve, the maximum load is difficult to evaluate.

![Diagram of Load-Displacement Curve](Image)

Figure 1-5. Simplified Regions of Load-Displacement Curve

Using an $L_2$ type of criterion, Hirany and Kulhawy (6, 2) proposed a new interpretation method that is based on observations from drilled shaft load tests. With this method, the load corresponding to a displacement equal to 12.7 mm (0.5 in) is interpreted as the failure load.

For lateral load test interpretation, Hirany and Kulhawy (6, 10) identified twelve different methods. Eleven of them can be grouped under displacement (including rotation) limit and graphical construction methods. Interpretation methods that utilize a displacement limit are based either on an absolute displacement or a certain percentage of the foundation diameter (B), as shown in Figure 1-7. There is no fundamental justification for these methods, and they generally are based on the experience and preferences of their authors. The new method proposed by Hirany and Kulhawy (6, 10) considers both the shaft rotation and displacement. By analyzing the relationship between the apparent depth of rotation and the applied lateral load, they introduced the lateral/moment limit as the interpreted failure load.

The apparent depth of rotation is defined as the ratio of the butt (top or head) lateral displacement to tangent of the butt slope, as shown in Figure 1-8.
Figure 1-7. Comparison of Lateral Load Interpretation Criteria


Figure 1-8. Definition of Apparent Point of Rotation


Depending on the geotechnical conditions, the apparent depth of rotation can remain constant, increase, or decrease with applied load, as shown in Figure 1-9. Each of these cases represents different overall foundation behavior and
allows for the lateral/moment limit to be defined. The method is summarized in Figure 1-10.

LOADS ACTING ON TRANSMISSION LINE STRUCTURE FOUNDATIONS

Transmission line structures are subjected to many different sources of loading, including: (a) steady-state loads, such as the line-angle tension and the dead weight of the structure, conductors, shield wires, and associated hardware, (b) transient loads, such as those from wind, ice, earthquake, and broken lines, (c) construction loads, and (d) maintenance loads, including unbalanced loads during temporary removal of lines (2). Depending on the type of structure, different combinations of loads will be transmitted to the foundations. Figure 1-11 shows the loads that can be transmitted to the foundations of single pole, two-legged, and four-legged structures.

The foundation of a single pole structure is subjected to sequential loading. Lateral and torsion loads from wind, line angles, or unbalanced line loads are added to the already present sustained axial compression load from the structure weight. These structures must be designed for vertical, horizontal, and
Figure 1-10. Typical Variation of Apparent Depth of Rotation with Load


Figure 1-11. Loads Transmitted to Foundations of Transmission Line Structures

torsional loads, as well as large overturning moments (11).

However, for lattice towers and H-frames, the lateral load can induce additional compression or uplift load to the foundation. Lattice tower foundations must be
designed for vertical tension or compression and horizontal shear components, while H-frame foundations must be designed for vertical and horizontal loads, in addition to overturning moments (11).

Table 1-1 lists the possible load combinations for transmission line structures. For a single pole structure, four combinations are possible. However, only the first two combinations are common. The other two require heaving, in which case lower factors of safety may be warranted for design loads. A general observation for loads transmitted to the foundation of a single pole structure is that lateral load and moment go together.

Table 1-1 also lists five possible modes of combined loading for a two-legged (H-frame) structure with X-bracing. Combinations 1 through 3 are encountered often. Combinations 4 and 5, which require line breakage with lateral forces from wind acting roughly perpendicular to the line direction, are less common. Foundations for two-legged structures are not subjected to torsion, because the torsional load is resolved into two parallel horizontal load components acting in opposite directions on each of the shafts. Pinned two-legged structures without X-bracing will behave similarly to single pole structures.

Table 1-1
LOAD COMBINATIONS FOR FOUNDATIONS OF TRANSMISSION LINE STRUCTURES

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Load Combination</th>
<th>Uplift</th>
<th>Compression</th>
<th>Lateral</th>
<th>Moment</th>
<th>Torsion</th>
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<td>1</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<tr>
<td></td>
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<td>x</td>
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<td>x</td>
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<tr>
<td></td>
<td>4</td>
<td>x</td>
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<td>Two-Legged</td>
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<td>3</td>
<td></td>
<td>x</td>
<td>x</td>
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<td></td>
<td>4</td>
<td>x</td>
<td></td>
<td>x</td>
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<td>x</td>
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<td>2</td>
<td>x</td>
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1-10
The two load combinations transmitted to the foundations of four-legged (lattice towers) structures also are given in Table 1-1. Both combinations are common. Moment and torsion cannot be transmitted to the foundations of lattice towers because they are resolved into vertical and horizontal load components.

When a two-legged or four-legged transmission line structure is subjected to lateral loads, the resultant load is inclined. The load inclination angle is controlled by the structure geometry and is independent of the change in magnitude of the applied lateral load. Lateral loads acting on these structures result in lateral load and moment when they are brought to ground level. Moment is proportional to the applied lateral load, and it depends on the magnitude of lateral load and the structure geometry. As indicated previously, moment is resolved into axial tension and compression, and they in turn are proportional to the applied lateral load and are dependent on the structure geometry. Therefore, foundations will be subjected to either inclined tension or inclined compression, depending on how the moment is resolved. In either case, the tangent of the angle of the resultant inclined load will be equal to the ratio of lateral to axial load component. Since the axial load component is directly proportional to the lateral load component, the angle of the resultant inclined load will depend only on the structure geometry. For most four-legged transmission line structures, this angle ranges from about 10 to 15 degrees.

SCOPE OF STUDY

This study focuses on the drained behavior of vertical drilled shafts subjected to inclined loading. An extensive laboratory testing program was conducted to facilitate a parametric study on model drilled shafts tested in uniform sand deposits. The following parameters were evaluated: (a) drilled shaft geometry, (b) initial soil conditions, and (c) angle of applied load. The range of values selected for these parameters reflects conditions that are relevant to typical transmission line structure applications. Laboratory testing was conducted in a controlled environment using fully-automated, computer-operated testing equipment. When evaluating the effect of one particular parameter on drilled shaft response, all other parameters were held constant.

Section 2 reviews the available analysis and design methods for inclined loading of piles and drilled shafts. All of the procedures currently available have been derived from model tests on piles, with very little verification on drilled shafts. The potential for application of these methods to drilled shafts is
examined. Sections 3 and 4 present the procedures and equipment used to prepare sand deposits and to conduct tests on model drilled shafts. The results of the axial and lateral tests are evaluated in Section 5. The inclined test results are evaluated in Sections 6 and 7. Section 8 focuses on the relationships among the axial, lateral, and inclined responses of drilled shafts and provides guidelines for the application of the model test results. A summary of this study and design recommendations are presented in Section 9. Detailed test results are provided in the Appendix.

REFERENCES


Section 2

ANALYSIS AND DESIGN METHODS FOR INCLINED LOADING

This section reviews available analysis and design methods for piles and drilled shafts subjected to inclined loads during drained conditions. The theoretical determination of capacity under inclined load is complex because of soil nonlinearity and the three-dimensional, nonsymmetric nature of the problem. Approximate theoretical and semi-empirical solutions have been proposed by several authors and are discussed briefly. The semi-empirical solutions have been developed from laboratory model tests conducted on jacked piles and may not be applicable to drilled shafts. In addition, the problems of interpreting the failure load from inclined load-displacement curves are presented.

CAPACITY UNDER INCLINED COMPRESSION LOADING

Early work (1, 2, 3) in evaluating inclined compression was done using a well-known lower bound solution for pile axial compression capacity. To account for load inclination, Meyerhof and Ranjan (2) modified the bearing capacity factor \( N_q \), which is used to calculate the pile tip resistance. Figure 2-1 shows the assumed loading and soil stress conditions. The vertical component of the tip load \( Q_{tuv} \) was expressed as:

![Diagram of forces on pile under various load inclinations](image)

*Figure 2-1. Forces on Pile Under Various Load Inclinations*

Source: Based on Meyerhof and Ranjan (2), p. 438.
\[ Q_{t \omega} = Q_{t \omega} \cos \omega = \gamma D N_{q \omega} A_t \]  

(2-1)

In which \( Q_{t \omega} = \) tip load, \( \omega = \) inclination of tip load, \( \gamma = \) soil unit weight, \( D = \) pile depth, \( N_{q \omega} = \) bearing capacity factor at \( \omega \), and \( A_t = \) tip area. For a particular value of effective stress friction angle \( (\phi) \), the bearing capacity factors \( (N_{q \omega}) \) are given for different values of tip load inclination \( (\omega) \) in Figure 2-2. The original values of \( N_q \) for axially loaded piles (2) were modified proportionally for Figure 2-2 to account for load inclination. The modification was done so that, for \( \psi = 180 \) degrees, the \( N_{q \omega} \) values are the same as for axially loaded piles.

Meyerhof and Ranjan (3) related the tip load angle \( (\omega) \) to the applied load angle \( (\psi) \) by experimenting with a number of relationships. For the limits of \( \psi = 180 \) and 90 degrees, \( \omega = 0 \) and 90 degrees, respectively. The chosen relationship, given in Figure 2-3, is nonlinear and maintains the interface friction angle below the sand friction angle for any \( \psi \). For a given \( \psi \), the corresponding tip load angle \( (\omega) \) can be obtained from Figure 2-3. Once \( \omega \) is known, Equation 2-1

![Diagram](image)

Figure 2-2. Bearing Capacity Factors for Vertical Piles Under Inclined Loads

Source: Based on Meyerhof and Ranjan (3), p. 438.
can be used to calculate both the tip load and its vertical component.

To calculate the side resistance \( Q_{s\delta} \), Meyerhof and Ranjan (2) extended Hansen's method (4) to account for load inclination, as follows:

\[
Q_{s\delta} = m H_u
\]  

\[ (2-2) \]

in which \( H_u \) = lateral resistance under horizontal load and \( m \) = passive stress mobilization factor, given in figures as a function of \( \psi \) and D/B (2). These figures were obtained experimentally by dividing the horizontal component of the observed inclined capacity by the theoretical lateral capacity. Once the tip and side loads are calculated, the inclined capacity \( P_{\psi u} \) can be obtained from the static equilibrium of horizontal forces:

\[
P_{\psi u} = (Q_{s\delta} \cos \delta - Q_{t\omega} \sin \omega) / \sin \psi
\]  

\[ (2-3) \]

in which \( Q_{s\delta} \) = side load, \( \delta \) = interface friction angle, \( Q_{t\omega} \) = tip load, \( \omega \) = tip load inclination, and \( \psi \) = applied load inclination. Equation 2-3 was not verified using experimental results.

Subsequent work (5) concluded that pile behavior under inclined loads depends significantly on the deformation characteristics of both the pile and
surrounding soil. The failure mechanism that corresponds to the ultimate load is very complex and difficult to analyze using conventional theoretical procedures. Therefore, Chari and Meyerhof (5) used an available semi-empirical solution (6, 7), which states that the pile capacity under inclined load \( P_{\psi u} \) can be calculated from the following:

\[
(P_{\psi u} \cos \psi/Q_c)^2 + (P_{\psi u} \sin \psi/H_u)^2 = 1
\]  

(2-4)

in which \( \psi \) = applied load inclination (measured from the upward vertical axis), \( Q_c \) = axial compression capacity, and \( H_u \) = lateral capacity. Equation 2-4 is a modification of the well-known structural interaction formula and predicts a reduction of pile capacity as the applied load deviates from the vertical.

Equation 2-4 is represented by the dashed line on the polar diagram given as Figure 2-4, which was used for comparing the equation with experimental data (8). The axial and lateral capacities for the interaction equation were calculated from available theories (8, 9). By re-plotting as in Figure 2-5, it can be seen that the pile capacity is reduced as the applied load deviates from the vertical. The reduction is nearly 30 percent for a 150 degree inclination and about 50 percent when the inclination is 120 degrees (5). The relatively small lateral resistance of a rigid pile compared with its large axial compression

---

**Figure 2-4. Polar Capacity Diagram**

Source: Based on Chari and Meyerhof (5), p. 853.
capacity is the primary reason for this reduction.

By extending the theory for shallow strip footings subjected to inclined loads, Koumoto, et al. (10) proposed that inclination factors be used to predict the pile inclined capacity. For a given load inclination and axial compression capacity, the pile inclined capacity can be calculated as:

\[ P_{\psi u} = i_{\psi} Q_c \cos (180^\circ - \psi) \tag{2-5} \]

and

\[ i_{\psi} = \cos (180^\circ - \psi) \left[ 1 - (1 - H_u/Q_c) \sin \psi \right] \tag{2-6} \]

in which \( \psi \) = applied load inclination (measured from the upward vertical axis), \( i_{\psi} \) = inclination factor, \( Q_c \) = axial compression capacity, \( H_u \) = lateral capacity, and \( P_{\psi u} \) = inclined capacity. They also analyzed the following approximate equation for shallow foundations subjected to inclined loads, given by Meyerhof (2):

\[ i_{\psi} = \left[ 1 - (180^\circ - \psi)/90^\circ \right]^2 \tag{2-7} \]

Equations 2-6 and 2-7 agreed well with observed values of \( i_{\psi} \) in dense sand; in loose sand, the observed values of \( i_{\psi} \) were greater than the computed.

The research of Meyerhof and his co-workers has been based on laboratory tests using model steel piles. Therefore, the application of these results to drilled shafts can not be warranted without additional investigation. Recent research
by Yalcin and Meyerhof (11) showed pronounced differences in response to inclined loading between driven and buried piles. Equation 2-7 was recommended for driven piles, while no comparisons or recommendations were given for buried piles. Furthermore, Equation 2-4 yielded conservative estimates of pile inclined capacity (11), which is not shown in Figure 2-4.

CAPACITY UNDER INCLINED UPLIFT LOADING

Early work in evaluating inclined uplift was done by Yoshimi (12), who tested both vertical and battered piles. The loading angle was 30 degrees from the vertical, while the initial pile inclination varied from +30 to -30 degrees from the vertical in increments of 15 degrees. The capacity was determined by assuming that the axial uplift capacity is independent of the lateral load component and that the lateral capacity is independent of the axial load component. However, no explanation was given as to how the capacities were determined from the load-displacement curves.

Broms (13) modified Yoshimi's method to include the effect of lateral soil stress, resulting from inclined uplift loading, on the axial uplift capacity. He noted that the pile behavior under inclined uplift loading is influenced by the load inclination. For small inclinations, failure will occur by pullout. However, lateral failure will occur for a large load inclination.

The change in stress distribution with load inclination is illustrated in Figure 2-6. The largest change will occur near the tip and butt. The resulting stress distribution can be calculated from an elastic analysis. Because the shear strength of sand is proportional to the confining stress, the lateral stress will increase linearly with depth below the ground surface as failure occurs in the soil and, according to Broms, this stress will be equal to approximately three to nine times the Rankine maximum passive stress.

Broms assumed that the lateral soil stress is equal to five times the Rankine maximum passive stress to a depth z below the ground surface and that the high lateral stress at the tip can be replaced by a concentrated load, as shown in Figure 2-7. The vertical component of the inclined capacity \( P_{\text{uv}} \) can be calculated from:

\[
P_{\text{uv}} = Q_u + \Delta Q_u
\]  

(2-8)
in which $Q_u = \text{uplift capacity under axial load}$, and $\Delta Q_u = \text{increase in uplift capacity caused by the lateral forces } T \text{ and } R$. The capacity increase then is:

$$\Delta Q_u = (T + R) \tan \delta$$ \hspace{1cm} (2-9)
in which $\delta$ = interface friction angle. The lateral force ($T$) can be calculated from the shaded area of the triangular stress distribution as:

$$T = 5K_p \gamma z^2 B/2$$

(2-10)

in which $K_p$ = Rankine coefficient of maximum passive soil stress, $\gamma$ = soil unit weight, and $B$ = diameter. The depth $z$ depends on the load inclination angle and would have to be assumed. The concentrated tip force ($R$) can be calculated from moment equilibrium as:

$$R = T (2z/3 + e)/(D + e)$$

(2-11)

in which $e$ = load eccentricity and $D$ = depth. The resulting horizontal load component ($P_{\psiuh}$) then can be calculated as:

$$P_{\psiuh} = T (D - 2z/3)/(D + e)$$

(2-12)

The inclined capacity ($P_{\psiu}$) then is given by:

$$P_{\psiu} = (P_{\psiuv}^2 + P_{\psiuh}^2)^{0.5}$$

(2-13)

According to Broms (13), the capacity obtained this way corresponds to "tension failure". Therefore, it was assumed that failure occurs when the pile is pulled out of the ground and moves parallel with the pile axis. In the case of "lateral failure", the capacity was calculated from:

$$P_{\psiu} = H_u / \sin \psi$$

(2-14)

in which $\psi$ = applied load inclination (measured from the upward vertical axis) and $H_u$ = lateral capacity calculated as (13):

$$H_u = (5/6)K_p \gamma B D^2$$

(2-15)

Like Yoshimi (12), Broms assumed that the vertical component of the applied load does not affect the lateral resistance.

Broms attempted to verify his method through comparison with Yoshimi's data. It is not clear how the depth $z$ of the assumed soil stress distribution was
determined, and that is critical for the calculation of the inclined load. Furthermore, the Broms method yields the net inclined capacity (independent of pile weight), since he calculated \( Q_u \) as the net resistance of a vertical pile. Yoshimi, on the other hand, plotted gross load-displacement curves, but he provided no data regarding the weight of the model piles.

The response of a drilled shaft to inclined loading is a complex problem that cannot be decoupled simply into axial and lateral modes. The key to understanding the behavior of a drilled shaft, as will be shown in Section 7, is the shaft rotation under inclined loading, which is fundamental to the shaft response regardless of the load inclination. In addition, plotting the inclined uplift resultant load versus the corresponding displacement for model piles is a problem because the plots are affected significantly by the model weight.

Meyerhof (\textsuperscript{14}) proposed that the analysis for anchor walls under inclined uplift loading can be adapted for rigid piles by multiplying the uplift coefficients for walls by shape factors to obtain the corresponding coefficients for piles. The basic theoretical equation for anchor walls that had been derived from force polygons remained the same. The inclined capacity \( F_{\psi u} \) for rigid piles was obtained by substituting the appropriate coefficients for piles into that equation:

\[
P_{\psi u} = (c K_{cu} D + \gamma b^2 K_{bu}/2) B + W \cos \psi
\]

(2.16)

in which \( c = \) cohesion, \( K_{cu} \) and \( K_{bu} \) = uplift coefficients, \( D = \) depth, \( \gamma = \) soil unit weight, \( B = \) diameter, \( W = \) pile weight, and \( \psi = \) applied load inclination (measured from the upward vertical axis). The first term is zero for drained analyses. The \( K_{bu} \) uplift coefficient for rough circular piles is shown in Figure 2-8 for axial uplift and lateral loading, and values of \( K_{bu} \) for intermediate load inclinations can be interpolated, as for vertical walls. Comparison of the two charts in Figure 2-8 shows that, for a given effective stress friction angle, the value of \( K_{bu} \) for short piles decreases as the load inclination (\( \psi \)) increases while, for long piles, \( K_{bu} \) increases with increasing \( \psi \).

Based on model tests in both clay and sand, Meyerhof (\textsuperscript{14}) proposed an approximate parabolic relationship between the inclined capacity \( F_{\psi u} \) and the limits of vertical \( (Q_u) \) and lateral capacity \( (H_u) \), as given below:

\[
P_{\psi u} \cos \psi/Q_u + (P_{\psi u} \sin \psi/H_u)^2 = 1
\]

(2.17)
in which $\phi = \text{applied load inclination (measured from the upward vertical axis)}$. The inclined and vertical capacities in Equation 2-17 denote the gross capacities that are dependent on the pile weight. It is unfortunate that the comparison plots of theoretical versus observed experimental capacities for various load inclination angles are difficult to decipher. However, it is obvious that the results are affected seriously by the model weights, which is perhaps why Meyerhof (14) states that, for short piles, the capacity decreases from a maximum for vertical uplift to a minimum for lateral load; for long piles, the capacity increases as the load inclination from the vertical increases. Such inconsistencies should not exist.

Das, et al. (15) remarked that the unsymmetrical failure of soil associated with piles subjected to oblique loading makes the theoretical analysis difficult. In general, they verified Equation 2-17 by conducting laboratory tests on buried model piles. The piles were made of wood and then coated with glue and rolled over the sand used for the tests. Good agreement was obtained, as shown in Figure 2-9. Since the model piles were very lightweight, their weight did not influence the capacity to the extent noted by Meyerhof (14). Therefore, the capacity was found to increase with increasing load inclination, regardless of the pile geometry. To calculate the axial uplift and lateral capacities, Das, et al. (15) used the Meyerhof (16) and Brongs (17) methods, respectively.
Figure 2-9. Variation of Inclined Capacity with Load Inclination


INTERPRETATION OF FAILURE LOAD FROM INCLINED LOAD TESTS

There is no generally accepted methodology for the interpretation of load-displacement curves from inclined load tests. Different researchers have employed different approaches that have led to certain difficulties when comparing various results. Observations and recommendations made by Hirany and Kulhawy (18) should be followed when interpreting inclined load tests.

Awad and Petrasovits (19) used an arbitrarily-defined critical displacement, expressed as a percentage of pile depth, to define the failure load. Since only axial and lateral displacement can be measured effectively, they chose the one that first reached the predetermined critical value.

Chari and Meyerhof (5) adopted the point where the portion of the load-displacement curve becomes straight or substantially straight to be indicative of failure. Similarly, Yalcin and Meyerhof (11) defined the failure load as that corresponding to the beginning of the final linear portion of the load-displacement curve and determined it graphically. This method is necessarily dependent on individual judgment and scale. Familiarity, experience, and judgment are required for consistent and reliable interpretation of the test results (5).

Subsequent work by Meyerhof and Sastry (20) defined the failure load as that
where the increase in the displacement rate first reaches its maximum value. For piles driven into sand, the failure load corresponded to a resultant displacement of about 0.5 to 3.5 percent of the pile depth and occurred at a rotation of about 1 to 2 degrees. Recommendations made by Hirany and Kulhawy (21) for the interpretation methods based on strain rate also are valid here. For strain-hardening load-displacement curves, the peak in the displacement rate may never be achieved.

Das, et al. (15) determined the failure load from the load-displacement curves by evaluating the region where sudden failure occurred or a large displacement was obtained for a small increment of applied load. They indicated that sudden failure occurred where the load inclination from vertical was less than 30 degrees, while large displacement occurred for inclinations greater than 30 degrees. However, it should be noted that such an approach depends on individual judgment and scale. Furthermore, depending on geotechnical conditions, load-displacement curves for inclined load tests may not conform to either one of these two failure type patterns.

SUMMARY

A review of available methods for the analysis of piles and drilled shafts under inclined loading has been presented. Two loading modes were identified: (a) inclined compression and (b) inclined uplift. These modes also have been called combined loading, but this notation is ambiguous and should be discouraged.

The capacity for inclined compression was calculated from a theory developed for piles subjected to axial compression, including modified bearing capacity factors to account for load inclination. For inclined uplift, the capacity was calculated using an approximate theoretical procedure based on a simplified, two-dimensional representation with an assumed soil stress profile at failure. In both cases, verification by laboratory model tests has been inadequate or virtually nonexistent. Furthermore, it has been widely accepted that theoretical procedures for piles subjected to inclined loading are very difficult to develop because of soil nonlinearity and the three-dimensional, nonsymmetrical nature of the problem.

The semi-empirical procedures rely on experimentally-determined interaction equations that relate the inclined capacity to the axial and lateral capacities for both inclined compression and uplift. In addition, the inclination factors
have been developed for calculating the inclined compression capacity from the axial compression capacity and load inclination. Little has been done to verify the applicability of these semi-empirical methods to drilled shafts. As will be shown in Section 8, the semi-empirical equations apply only to drilled shafts subjected to inclined compression; for inclined uplift, the applicability is limited and influenced by sand density. Moreover, the data obtained in this study show that inclination factors do not apply to drilled shafts.

Finally, a summary of proposed methods for the interpretation of load-displacement curves for inclined tests was presented. The available methods are based on displacement limits, strain rate limits, or graphical construction, and they are incompatible with each other. This situation further complicates any comparison between different research results. As will be shown in Sections 6 and 7, the lateral/moment limit interpretation method developed by Hirany and Kulhawy (18, 22) was found to be applicable to both inclined uplift and compression. This method enables consistent and reliable interpretation of the inclined uplift and compression load-displacement curves.

REFERENCES


Section 3
PREPARATION OF LABORATORY TEST DEPOSITS

The soil handling facilities and test preparation procedures are described in this section. Tests were performed in deposits of dry sand prepared in testing chambers. Three sand densities (loose, medium dense, and dense) and two stress histories (normally consolidated and overconsolidated) were included.

SOIL TYPE

The soil used throughout this testing program was a processed uniform sand referred to as filter sand. Previous use of this soil at Cornell University has provided information on its properties. The sand consists of subangular grains of limestone (38%), quartz (20%), siltstone/sandstone fragments (20%), quartzite (12%), and trace quantities of other materials. The specific gravity of solids is 2.74, the minimum dry density is 14.8 kN/m$^3$ (94.3 pcf), and the maximum dry density is 18.3 kN/m$^3$ (116.6 pcf). The relationship between the effective stress friction angle and unit weight has been obtained from several different laboratory tests and is presented later. A detailed description of the index and strength properties of the filter sand is given elsewhere (1).

TESTING CHAMBERS

All tests were conducted in deposits of filter sand prepared in two identical rectangular testing chambers, shown in Figure 3-1. These chambers were constructed of 19 mm (0.75 in) thick plywood that was stiffened with steel angles along the edges and sides. All the pieces were fastened together with 6.35 mm (0.25 in) steel bolts. The inside dimensions of the chambers were 864 mm (34 in) long by 457 mm (18 in) wide by 762 mm (30 in) deep. These chambers were fitted with casters to make them maneuverable by one person.

SOIL PLACEMENT

Test deposit preparation was planned as a one-person operation for efficiency. The turn-around time per test was four to five hours, depending on the desired density of the deposit.
The two chambers were used interchangeably as sand hoppers and testing chambers. A narrow rectangular slot measuring 63.5 x 19.1 mm (2.5 x 0.75 in) was cut through the bottom plate of the chambers, and a special valve was attached to the bottom of the chambers over the slot. This valve was used to regulate sand flow through the slot. Deposit preparation would start with an empty chamber positioned directly beneath the full chamber that was suspended from the reaction frame, as shown in Figure 3-2. The height of the upper chamber (sand hopper, in this case) and its position relative to the testing chamber on the floor were controlled by an electric winch and a set of two pulleys attached to a trolley. An all-welded steel cross-bracing with an eye-bolt in the middle was bolted to the top of the sand hopper. A special hook assembly and aircraft wire cable were used to lift the chamber. In addition, a heavy-duty steel alloy chain was attached between the trolley and the eye-bolt of the cross-bracing to provide safety redundancy (Figure 3-2).

For loose and medium dense deposits, sand placement was accomplished by sand raining (pluviation) through a flexible 102 mm (4 in) diameter hose attached to the bottom of the sand hopper. The flow of sand was controlled by a valve. Details of the raining procedure are given elsewhere (1, 2).

As the sand was rained through the hose, a cone that was created by the sand sloping at its angle of repose would form inside the hopper. Since it was not
possible to empty the hopper completely by raining, the hopper was used with an extra 305 mm (1 ft) deep rectangular box attached to the top of the hopper between the cross-bracing and the sides of the hopper. With this addition, the hopper effectively had enough volume to fill 4/3 of the testing chamber below, and it provided for an uninterrupted raining procedure. Upon completion of raining, the sand surface in the testing chamber was leveled, and excess sand was removed with an aluminum straight edge.

For dense deposits, a VC-35 electric vibrator (Syntron Division, FMC Corp., Homer City, PA) was used to compact the sand beyond the level possible with pluviation. The vibrator provided 60 Hz vibrations with a variable peak to peak amplitude controlled by a rheostat that ranged from 0.4 to 1.6 mm (0.016 to 0.063 in). The vibrator was bolted to a 19 mm (0.75 in) thick plywood plate to which steel weights were added (Figure 3-3). The total weight of the vibrator assembly was 302.5 N (68 lb), and no additional pressure was exerted on the vibrator by the operator during deposit preparation. The bottom plate of the
vibrator assembly measured 432 x 432 mm (17 x 17 in) and was contoured so that it fitted around the shaft and covered one half of the chamber (Figure 3-3).

UNIT WEIGHT MEASUREMENT

As mentioned previously, loose and medium dense deposits were obtained by raining sand through a 102 mm (4 in) diameter flexible hose. With this method, the sand unit weight depends primarily on the velocity and intensity of the sand particle drop (1, 2). The placement unit weight increases with increasing velocity, which can be obtained by increasing the drop height. The intensity of sand raining refers to the rate of sand discharge through the flexible hose. By installing a set of wire mesh screens at the end of the hose, the rate of sand discharge is reduced, which improves the packing of sand grains and therefore increases the placement unit weight.

For loose deposits, the hoses were fitted with plastic end caps, perforated by equally-spaced, 9.5 mm (0.375 in) diameter drilled holes. The sand compaction and unit weight depended on the drop height. Using density cups (4), the unit weight (γ) was correlated to the drop height (h), as shown in Figure 3-4. A drop height of 50 to 76 mm (2 to 3 in) was maintained throughout the raining procedure to obtain the loose sand deposits.

Figure 3-4 shows that a drop height of 400 mm (15.7 in) is necessary to obtain a unit weight of approximately 16.2 kN/m³ (103.2 pcf) when the perforated plastic end caps are used. This height is not convenient in the laboratory, and therefore an alternative was adopted for the medium dense deposits. A set of two
coarse-meshed screens with 6.25 mm (0.25 in) openings was used instead of the perforated end cap. These screens were rotated 45 degrees with respect to each other, spaced at 12.7 mm (0.5 in), and installed at the end of the hose, as shown in Figure 3-5. The wire mesh attachment allowed the sand to fall in a more diffused state, thereby reducing the rate of discharge. Using density cups, the deposit unit weight (γ) was correlated to the drop height (h), as shown in Figure 3-6. With wire mesh screens at the end of the raining hose, the unit weight was less dependent on the drop height, and higher unit weights were possible with smaller drop heights. For the medium dense deposits, the drop
height was maintained between 200 mm (8 in) and 300 mm (12 in).

Sand raining was unsuitable to achieve dense deposits because unit weights greater than 16.5 kN/m³ (105 pcf) could not be obtained. Therefore, a vibrator was used to compact the sand deposits. Sand was placed in the testing chamber in 102 mm (4 in) thick layers, and each layer was vibrated with a VC-35 electric vibrator at an amplitude of 1.6 mm (0.063 in). Figure 3-7 shows the correlation between the layer unit weight (γ) and vibration time (t). Measurements of unit weight were obtained with the density scoop because density cups are less reliable in dense sand (Δ). The relationship between the weight of sand in the density scoop and the sand unit weight is given in Figure 3-8. To examine whether the unit weight depended on the layer depth within the testing chamber, unit weight measurements were taken from layers placed at two different depths (Figure 3-7). Similar trends were obtained from both layers.

Based on Figure 3-7, a vibration time of 45 seconds was selected for the dense deposits. The vibrator covered only one-half of the chamber, as noted previously. Therefore, each 102 mm (4 in) thick layer first was leveled with a straight edge, and then one part of the layer was vibrated for 45 seconds with the other part vibrated immediately afterwards for 45 seconds.

The procedures for obtaining loose, medium dense, and dense deposits ensured
Figure 3-7. Unit Weight versus Vibration Time at 60 Hz Frequency and 1.6 mm Peak-to-Peak Amplitude

Figure 3-8. Calibration for Density Scoop in Filter Sand


uniform and consistent unit weights throughout the testing program. Final determination of unit weight for each deposit was carried out by dividing the net weight of sand in the testing chamber by the volume of the chamber. Table
3-1 summarizes the unit weight measurements. The mean values were 15.44 kN/m$^3$ (98.4 pcf) for the loose, 16.21 kN/m$^3$ (103.3 pcf) for the medium, and 17.54 kN/m$^3$ (111.7 pcf) for the dense deposits. The mean relative densities ($D_r$) were 17.1, 40.0, and 77.1 percent for the loose, medium, and dense sand, respectively.

**STRESS CONDITIONS**

The initial soil stresses were evaluated from the stress history of the

**Table 3-1**

<table>
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<th>Loose Test</th>
<th>Unit Weight (kN/m$^3$)</th>
<th>Medium Dense Test</th>
<th>Unit Weight (kN/m$^3$)</th>
<th>Dense Test</th>
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<td>32</td>
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<tr>
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<td>34</td>
<td>16.11</td>
<td>63</td>
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<td>48</td>
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<td>15.95</td>
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<td>45</td>
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</tr>
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<td>66</td>
<td>N.A.</td>
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<td>16.29</td>
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<td></td>
</tr>
<tr>
<td>67</td>
<td>15.58</td>
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</tr>
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<td></td>
<td></td>
<td>55</td>
<td>16.20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Mean** = 15.44
**S.D.** = 0.11
**COV** = 0.71%

1 kN/m$^3$ = 6.37 pcf
deposits. An important stress parameter is the in-situ coefficient of horizontal soil stress \((K_o)\), which relates the horizontal effective stress \((\sigma_h)\) and the vertical effective stress \((\sigma_v)\):

\[
\sigma_h = K_o \sigma_v
\]  \(3-1\)

For normally consolidated soil, \(K_o\) can be best represented by (e.g., \(\phi\)):

\[
K_{onec} = 1 - \sin \phi
\]  \(3-2\)

in which \(\phi\) = effective stress friction angle. Figure 3-9 shows the general relationship between the effective horizontal and vertical stress, and therefore \(K_o\), for three simple stress histories. For primary unloading \((K_{opu})\) and reloading \((K_{opr})\), \(K_o\) is given by \((5)\):

\[
K_{opu} = (1 - \sin \phi) \text{OCRsin} \phi
\]  \(3-3\)

\[
K_{opr} = (1 - \sin \phi)((\text{OCR}/\text{OCR}_{max})^{1-\sin \phi}) + [0.75 (1 - \text{OCR}/\text{OCR}_{max})]
\]  \(3-4\)

in which \(\text{OCR} = \) overconsolidation ratio \((\sigma_v/\sigma_v)\) and \(\text{OCR}_{max} = \) maximum overconsolidation ratio. Figure 3-9 also shows that different values of \(K_o\) can exist for the same value of effective vertical stress, depending on the soil stress history. An upper bound for \(K_o\) is the Rankine coefficient of maximum passive soil stress, given as:

\[
K_p = \frac{1 + \sin \phi}{1 - \sin \phi}
\]  \(3-5\)

![Figure 3-9. Stress Paths for Simple Stress Histories](image)

Loose and medium dense deposits were prepared by raining sand into the testing chamber, which results in normally consolidated deposits (1). Therefore, the in-situ coefficient of horizontal soil stress can be calculated using Equation 3-2. Additional research (2) has verified that sand deposits constructed by pluviation are normally consolidated and that use of Equation 3-2 for calculating $K_0$ is appropriate. Evaluation of the soil effective stress friction angle is discussed later in this section.

Dense deposits were constructed by placing the sand in 102 mm (4 in) thick layers and vibrating each layer for 45 seconds prior to placement of a subsequent layer. The effective surcharge stress caused by the vibrator assembly can be calculated (e.g., 2) by dividing the total weight of the vibrator assembly by the contact area and is equal to 1.6 kN/m² (33.8 psf), assuming dynamic effects to be negligible. This procedure results in an overconsolidated deposit because of sequential loading, unloading, and reloading. The stress paths corresponding to layer placement are shown in Figure 3-10, and the resulting OCR profile is given in Figure 3-11. Each increment of vertical effective stress was obtained by summing the effective stress from the weight of soil layer and effective stress surcharge exerted by the vibrator assembly. For dense deposits, Equation 3-3 was used to calculate $K_0$.

DETERMINATION OF EFFECTIVE STRESS FRICTION ANGLE

Depending on the type of foundation problem, different laboratory tests should be used to evaluate the soil strength parameters, as shown in Figure 3-12. For

![Figure 3-10. Load-Unload Cycles in Dense Sand Deposits](Modified After: Agaiby, et al. (2), p. 3-20.)
a drilled shaft in compression, the tip resistance is evaluated from an average of triaxial compression, direct simple shear (or direct shear for sand), and triaxial extension tests. The side resistance is modeled by the direct shear test and is the same in compression and uplift. For laterally loaded drilled shafts, triaxial extension is more appropriate.

The relationship between the effective stress friction angle and sand unit weight used in this study is given in Figure 3-13. The direct shear tests of Trautmann and O'Rourke (7) are most appropriate to obtain values of $\phi_{DS}$. For laterally loaded drilled shafts, values of effective stress friction angle from triaxial extension are needed. Correlations between the different effective stress friction angles have been given by Kulhawy and Mayne (6). Table 3-2 summarizes the values of effective stress friction angles calculated for the range of sand unit weights obtained in this study.

As the loading mode becomes more complex, such as with inclined loading, the choice of the appropriate laboratory test to evaluate the effective stress friction angle becomes more difficult. At present, no single laboratory test can be used to model the response of a drilled shaft under inclined loading because it is a combination of axial and lateral loading. The correct value of effective stress friction angle for inclined uplift is between the values obtained from the DS and TE tests. Since the difference between $\phi_{DS}$ and $\phi_{TE}$ is not that
Figure 3-12. Relevance of Laboratory Strength Tests to Field Conditions

Source: Kulhawy and Mayne (6), p. 4-6.

great, the effective stress friction angle for inclined uplift is taken as the average of $\phi_{DS}$ and $\phi_{TE}$. Similarly, the effective stress friction angle for inclined compression can be obtained as the average of the effective stress friction angles corresponding to axial compression and lateral loading modes. Table 3-3 summarizes the values used in this study.
Figure 3-13. Effective Stress Friction Angle versus Unit Weight for Filter Sand


Table 3-2

EFFECTIVE STRESS FRICTION ANGLES FOR DIFFERENT SAND DENSITIES

<table>
<thead>
<tr>
<th>Density</th>
<th>Average Unit Weight (kN/m³)</th>
<th>Effective Stress Friction Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DS</td>
</tr>
<tr>
<td>Loose</td>
<td>15.44</td>
<td>31.0</td>
</tr>
<tr>
<td>Medium</td>
<td>16.21</td>
<td>35.4</td>
</tr>
<tr>
<td>Dense</td>
<td>17.54</td>
<td>44.2</td>
</tr>
</tbody>
</table>

a - DS = direct shear; TC = triaxial compression; TE = triaxial extension

1 kN/m³ = 6.37 pcf

SUMMARY

Laboratory test deposits were prepared from dry uniform sand known as Cornell...
Table 3-3

EFFECTIVE STRESS FRICTION ANGLES FOR VARIOUS LOADING MODES

<table>
<thead>
<tr>
<th>Sand Density</th>
<th>φ for Loading Mode (degrees)</th>
<th>Axial Uplift</th>
<th>Inclined Uplift</th>
<th>Lateral Compression</th>
<th>Inclined Compression</th>
<th>Axial Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>31.0</td>
<td>33.3</td>
<td>35.6</td>
<td>34.2</td>
<td>32.8</td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>35.4</td>
<td>37.8</td>
<td>40.2</td>
<td>38.7</td>
<td>37.2</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>44.2</td>
<td>46.7</td>
<td>49.2</td>
<td>47.5</td>
<td>45.8</td>
<td></td>
</tr>
</tbody>
</table>

filter sand. The procedures and equipment used for soil deposit preparation were presented for loose, medium dense, and dense soil deposits. The sand stress conditions were evaluated, indicating the loose and medium dense deposits to be normally consolidated and the dense deposits to be overconsolidated. Finally, the relevant sand strength parameters were addressed.

REFERENCES


Section 4
TESTING OF MODEL DRILLED SHAFTS

This section describes the overall laboratory testing program. The model drilled shafts were constructed with a diameter of 52 mm (2.05 in) and depth to diameter ratios (D/B) of 3, 6, and 9. The testing system is discussed, and a summary of the tests is provided.

MODEL DRILLED SHAFTS

Three test models were constructed from stainless steel tubes by cutting the desired length and welding circular steel plates to the ends of each tube. A hole was tapped in the center of the top end-plate, and a 12.7 mm (0.5 in) diameter stainless steel threaded rod was fitted through that hole. The rod was tightened to the plate from the inside of the shaft by a nut and was permanently secured in place by a small amount of concrete that was poured into the shaft. When the shaft was assembled, it was a cylinder with 6.4 mm (0.25 in) of threaded rod protruding from the center of the top end-plate. The 12.7 mm (0.5 in) diameter threaded rod was used to attach a special swivel assembly that enabled coupling of the shaft to the loading system, as shown in Figure 4-1. The shank

![Diagram of Shaft Swivel Assembly]

Figure 4-1. Shaft Swivel Assembly
of the swivel was threaded and had a diameter of 9.5 mm (0.375 in).

The model shafts were coated with epoxy adhesive and sprinkled with washed and oven-dried filter sand. This procedure resulted in a rough shaft surface that was comparable to that of a cast-in-place shaft. After coating, the three models had a diameter of 52 mm (2.05 in) and depth to diameter (D/B) ratios of 3, 6, and 9.

The decision to use prefabricated instead of cast-in-place models was based primarily on the large number of tests necessary for the parametric study. The number of variables addressed in this study required 63 tests to fill the parametric matrix, as discussed later. The prefabricated and reusable models improved considerably the turn-around by eliminating the time required for concrete curing. Given the small scale of the models, construction simulation probably would have resulted in disturbance of the shaft-soil interface. Furthermore, the prefabricated models eliminated any variations in shaft geometry.

LOADING SYSTEM AND TESTING FACILITIES

Actuator

A MTS model 242.02 hydraulic actuator, with closed-housing LVDT (linear variable differential transformer) and pedestal base, was used throughout the testing program. The actuator was equipped with a MTS model 252.21 servo valve and a MTS model 661.19 load cell. The actuator had a 9.79 kN (2.2 kip) capacity and a 152 mm (6 in) stroke.

Reaction Frame

The same reaction frame used during soil deposit preparation, as described in Section 3, was utilized to support the actuator during loading. For the lateral tests, the actuator was placed horizontally and was attached rigidly to the vertical steel column at the desired height. The attachment was made with removable C-clamps so that it was possible to pinpoint the position that ensured horizontal shaft loading. For the axial tests, the actuator was oriented vertically and bolted to a special 1.08 m (42.8 in) long steel extension beam that was attached to the horizontal beam of the reaction frame and left in place permanently.

Inclined loading was made possible by a specially-machined steel base that was clamped to the steel column of the reaction frame. The base mechanism
facilitated actuator orientation at virtually any desired angle. Changing the angle of loading was a simple one-person operation. For any chosen angle of loading from lateral to axial, the actuator always was attached rigidly to the reaction frame. Therefore, the weight of the actuator did not influence the load applied to the model shafts. The weight of rods used for hook-up and the weight of the shaft instrumentation were recorded for every test and later taken into account when analyzing the data.

Testing Facilities

An electro-hydraulic test system with a servo-control loop was used to control the actuator. The test system was manufactured by MTS and was linked to a Hewlett-Packard HP 1000, A900 series mini-computer that has extensive capabilities for real-time operation. Peripherals were interfaced to the computer to facilitate an automated testing procedure. Figures 4-2 and 4-3 show schematics of the testing facilities and MTS system that are described in detail elsewhere.  

![Diagram of testing facility](image)

Figure 4-2. Model Drilled Shaft Testing Facility

Source: McManus and Kulhawy (1), p. 4-16.
INSTRUMENTATION

Measurements of shaft rotation were obtained with a Schaevitz model LSOC-30 inclinometer. This sophisticated and accurate (0.1 arc-second) device also was rugged and easy to use. It was self-calibrated within its operating range of ± 30 degrees and provided output voltage that was directly proportional to the sine of the tilt angle. The output voltage changed linearly from zero at perfectly horizontal position to ± 5 volts at the ends of the thirty degree range with the sign of output voltage dependent on the sense of rotation.

Two RDP Electronics model D2/1000E direct current differential transformers (DCDTs) were used in conjunction with the inclinometer to infer the movement of the shaft butt (Figure 4-4). DCDTs had miniature precision swivels at each end to facilitate their hook-up and enable their rotation. The instrumentation assembly was attached securely to the testing chamber and was installed in the exact same way so that, prior to the beginning of a test, the angle between two DCDTs was 90 degrees with one vertical and the other horizontal. The DCDTs had a stroke of 76.2 mm (3 in) and were calibrated prior to conducting the testing program. Figure 4-5 shows calibration curves for two DCDTs with displacement correlated to output voltage. The fit was linear and provided a stroke of about
63.5 mm (2.5 in). The DCDT performance was checked often during the testing program, and the calibration was verified after the last test. They performed consistently and reliably without any zero voltage drift.

TEST PROCEDURE

Shaft Installation

Model drilled shafts were installed during deposit preparation. Sand raking was terminated temporarily when the desired depth for shaft installation was reached. The swivel assembly was screwed onto the shaft and covered with a rectangular aluminum box that protected it from dust. The shaft was clamped to a piece of steel angle that then was bolted to the side stiffener of the testing chamber. Therefore, the shaft was suspended rigidly in the vertical position in the center of the testing chamber. The top end plate of the shaft was flush with the edges of the testing chamber. With a bubble level to check the shaft orientation, and alignment marks inscribed in the chamber sides to check the shaft position, it was possible to install shafts quickly and be consistent with the installation procedure.
Once the shaft was installed, the sand raining resumed. The flexible hose used for sand raining was moved inside the chamber and around the shaft. Care was taken not to touch the shaft during sand raining because that would disturb the shaft-sand interface. It was mentioned previously that the unit weight of the deposit depended on the drop height. Once the shaft was installed, it was no longer possible to bring the hose directly above the area of sand located beneath the shaft bottom. Therefore, if there had been a gap between the shaft bottom and the sand surface after the shaft installation, it would not have been possible to fill that gap with sand of equal density to that of the surrounding
sand. This problem would affect the load-displacement curves for compression tests in particular, because the side resistance would mobilize before any contribution from the tip developed. Several axial compression tests, which were repeated later, revealed this peculiar behavior. Consequently, it was important to install the shafts carefully so that the tip of the shaft was in close contact with the underlying sand before resuming the raining procedure.

For loose and medium dense deposits, the attachments used to keep the shaft in place (the clamp and steel angle) were removed when the chamber was filled completely with sand and prior to leveling the sand surface. For dense deposits, the attachments were removed once two-thirds of the shaft depth was embedded.

Hook-up Procedure

After the sand surface in the testing chamber was leveled, excess sand was removed, and the area was cleaned, the shaft was ready for hook-up. The chamber, which was fitted with casters, was moved slowly with a crowbar into the testing position and was aligned carefully. It was important not to shake the chamber during this operation because the deposit would be disturbed. Before hook-up, the chamber was lifted off the ground with a crowbar, and two pieces of 19 mm (0.75 in) thick plywood were inserted underneath the chamber steel I-beam legs to lift the casters off the floor and make the chamber immovable.

Figure 4-6 illustrates the hook-up procedure. A precision MTS swivel was attached to the load cell, and a 50.8 mm (2 in) piece of square steel structural tubing was bolted to the load cell swivel. A 12.7 mm (0.5 in) diameter hole was drilled in the side of the tubing opposite to the swivel. A 12.7 mm (0.5 in) diameter stainless steel threaded rod cut to desired length was used to hook up the load cell to the shaft. The rod had a hole tapped at one end, and it was inserted first through the hole in the tubing and then screwed carefully onto the threaded shank of the shaft swivel via its tapped hole. Once this process was accomplished, the rod was fixed at the other end with two nuts that were finger-tightened carefully, because it was important to avoid preloading the shaft. The loads exerted on the shaft during the hook-up operation were monitored constantly. Since the instrumentation assembly (the inclinometer and two DCDTs) was attached to the shaft prior to the hook-up, it also was possible to calculate shaft movements that resulted from the hook-up procedure. These movements typically were a fraction of a millimeter (less than 0.02 in) and were considered to be within instrumentation accuracy.
Figure 4-6. Hook-up of Actuator to Shaft

Test Conduct
Prior to running a test and before the final hook-up, the hydraulic power was applied to the actuator, and the hydraulic oil was warmed by leaving the power on for approximately thirty minutes, during which time the actuator was cycled manually from the main control panel. This preconditioning was conducted to ensure smooth piston rod movement.

Before the final hook-up, the instrumentation was connected to the power supplies and signal channels. The initial readings from both DCDTs and the inclinometer were recorded. Once the shaft was attached to the actuator, the testing commenced.

Running the test was a relatively simple and straightforward procedure using software developed on the HP 1000. The necessary test parameters were input interactively from the keyboard. All of the other information (transducer configuration, channels to be scanned, calibration factors, and offsets) was stored on the hard disk and was accessed by the program. After the test was completed, the actuator was disconnected, and the hydraulic power was shut down.
All of the tests were conducted in the displacement control mode under constant rate of displacement. This mode is different from most full-scale field tests that are conducted in force control. The unavailability of sophisticated testing equipment on site commonly is the reason that precludes displacement-controlled field tests.

The loading rate was chosen so that the test could be run conveniently. For axial uplift tests, the rate was 1.1 mm (0.04 in)/min, which enabled a sufficient number of readings to be taken in the critical peak zone. For more monotonic tests such as a lateral test, the rate was increased to 9.1 mm (0.36 in)/min, while the inclined and axial compression tests were conducted at 3.1 mm (0.12 in)/min, on the average. For drained analysis, the rate of loading does not affect significantly the response of drilled shafts. One study (2) determined that a change of one order of magnitude in the loading rate did not influence the load-displacement curve for laterally loaded shafts.

DATA ACQUISITION AND REDUCTION

Data acquisition was automated. The data file was stored on the HP 1000 hard disk and later was transferred to a PC-AT compatible computer for manipulation. To infer horizontal and vertical movement of the shaft butt, the data were reduced using a computer program.

Dashed lines in Figure 4-7 show the shaft orientation following exaggerated movement in an inclined uplift test. By applying the calibration and offset factors to the output voltage of a DCDT, its readings were converted to a distance in millimeters between the two end swivels of the DCDT. Therefore, at the beginning of the test, the initial readings from the two DCDTs were treated as the radii of two circles that had centers aligned on the two coordinate axes and an intersection at the origin of the coordinate system that coincided with the pin dowel of the shaft swivel. Knowing the initial and subsequent radii of the two circles, it was possible to calculate the X and Y coordinates of their intersection (i.e., the pin dowel) at any instance during the test. The calculation required solving two simultaneous quadratic equations with two unknowns (equations of two circles with unknown X and Y coordinates of their intersection). Once these coordinates were obtained, the actual shaft groundline horizontal and vertical displacements, as well as the depth of the point of rotation, were calculated from the shaft geometry and rotation. The reduced data were stored on a hard disk and were manipulated using a spreadsheet. The plots
were created using CoPlot by CoHort Software, Berkeley, CA.

TESTING PROGRAM

The testing program included a parametric study that investigated a matrix of different variables. The parameters examined were the shaft geometry (3), initial soil conditions (3), and angle of load application (7). The $3 \times 3 \times 7$ matrix was not filled completely because of the excessive boundary effects in dense sand deposits that precluded some of the tests. Boundary conditions in these test deposits are described in Section 5. Of the 67 tests performed in this study, 58 were analyzed, including the four replicate tests. The nine tests not reported included the shakedown tests and the tests where instrumentation and loading problems were encountered.

The model drilled shafts had a diameter of 52 mm (2.05 in) and depth to diameter (D/B) ratios of 3, 6, and 9. Shafts with D/B of 6 and 9 were felt to provide more reliable response in loose deposits because of greater resistance. The axial uplift test in loose sand with D/B of 3 was disregarded because the soil resistance was too low to be measured reliably. On the other hand, the shaft with D/B of 3 was less affected by boundary conditions in dense deposits. The range of geometries adopted for this study was based on the typical geometries
of drilled shafts used in the utility industry.

Three sand densities were investigated: loose, medium dense, and dense. Of the 58 tests reported in this study, 21 were in loose sand, 24 in medium dense sand, and 13 in dense sand. The loose and medium dense deposits were constructed similarly by pluviation, and both were normally consolidated. In general, the tests in loose and medium dense deposits responded similarly, showing strain hardening for lateral and inclined compression loading modes. The tests in dense sand exhibited different post-peak behavior, with clearly evident failure mechanisms that are discussed in detail in Sections 5, 6, and 7.

Seven loading angles were applied: 0, 15, 45, 90, 135, 165, and 180 degrees. All angles are measured from the upward vertical axis. Load inclination angles of 0, 90, and 180 degrees define axial uplift, lateral, and axial compression tests, respectively. These tests, although strictly not part of the inclined testing program, were conducted to establish a relationship between the inclined response and the corresponding limit cases. Angles of 15 and 45 degrees denote inclined uplift tests and angles of 135 and 165 degrees refer to inclined compression tests.

The test designations included the shaft geometry, sand density, and angle of loading. A summary of the testing program is given in Table A-1 on page A-2.

SUMMARY

Prefabricated and reusable model drilled shafts were tested in the laboratory using a computer-controlled, electro-hydraulic testing system. The loading equipment, instrumentation, and test set-up were presented followed by a description of the test procedure. Test data were reduced by computer. The procedure used to calculate shaft displacements and rotation was described. The testing program addressed a matrix of variables including the shaft geometry, sand density, and angle of load inclination.

REFERENCES


4-11
Section 5
RESPONSE OF MODEL DRILLED SHAFTS TO AXIAL AND LATERAL LOADS

An evaluation of the response of model drilled shafts to axial and lateral loading is presented in this section, as a prelude to evaluating inclined loading. Recent research (e.g., 1, 2) has provided in-depth analysis of drilled shaft behavior under axial and lateral loads. Consequently, this section presents only a brief summary of the load tests.

Boundary conditions in the sand deposits are discussed first, followed by an evaluation of test repeatability. The parameters investigated included the shaft geometry and initial soil density. The effect of these parameters on drilled shaft response is assessed. Then the interpretation of the failure load from the load test results is discussed. The individual test results are presented in Appendix A.

BOUNDARY CONDITIONS IN TEST DEPOSITS

Model drilled shafts were centered in the testing chamber, and only one shaft was tested per deposit. For the lateral tests, the loading direction always was parallel with the longer side of the chamber. The chamber design had to accommodate two opposing requirements: minimum boundary effects and small deposit size. The former determines the usefulness of the testing program, while the latter influences the turn-around rate and testing program efficiency. The chambers were designed using boundary condition assessments established for axially and laterally loaded model drilled shafts (e.g., 1, 2).

Figure 5-1 shows the shaft position relative to the chamber boundaries for the deepest model drilled shaft with a diameter (D) of 52 mm (2.05 in) and depth to diameter (D/B) ratio of 9. For laterally loaded shafts, the boundary was eight diameters from the shaft in the loading direction and four diameters from the shaft normal to the loading direction. For axially loaded shafts, the bottom of the chamber was six diameters from the tip of the deepest shaft.

Agaiby, et al. (2) positioned stress cells at various distances from laterally
loaded drilled shafts to assess the horizontal stress attenuation with distance from the shaft. The average unit weights in their study for loose, medium, and dense deposits were virtually identical to those in this study. However, the drilled shafts were larger than those herein, with diameters of 76.2 mm (3 in) and 152 mm (6 in) and depth to diameter ratios of 3, 6, and 9. For loose and medium deposits, the stress cells showed hardly any change in the horizontal stresses at approximately six diameters from the shaft at failure. For the dense sand, the horizontal stress increase at 4.6 diameters from the shaft was equal to 12.5 percent of the increase at one diameter from the shaft for a horizontal load of 2400 N (535 lb). In this study, the boundary is eight diameters from the shaft, and therefore the boundary influence is minor.

Turner and Kulhawy (1) reported a number of problems with model drilled shafts tested in dense sand under axial compression, including: failure of stress cells, structural failure of model drilled shafts, large deflections and buckling of the reaction frame, and pull-out of the reaction frame from the concrete foundations. Consequently, model testing in dense sand under axial compression was avoided in this study. Only one attempt was made for a shaft with D/B of 3,
but this test was terminated when the measured load exceeded 70 percent of the actuator capacity and the shaft rotated from the initial vertical position.

IMPORTANCE OF NET LOADS

When conducting a laboratory testing program using model drilled shafts, care must be taken to differentiate between the capacity resulting from soil resistance and the capacity resulting from the weight of the shaft, hook-up equipment, and attached instrumentation. This point is particularly important for axial and inclined uplift tests. With full-scale field tests, the weight of the shaft is small, and the weight of the attached equipment used for hook-up and instrumentation is negligible, compared to the soil resistance. The added weight is seldom recorded, which means that most uplift field tests are documented by gross load-displacement curves. However, utilizing the same practice in the laboratory could alter the load-displacement curves significantly. Depending on the soil conditions and shaft geometry, the weight of the shaft and attachments could be as much as 70 percent of the total gross uplift resistance.

In addition, direct comparisons between uplift tests on different model shafts (e.g., one of reinforced concrete and the other of wood) are not possible if only the gross load-displacement curves are reported. Unfortunately, this is commonly the case with the available laboratory tests on model drilled shafts (piles) subjected to inclined loading.

In this study, only the net load-displacement curves are reported for all axial and inclined uplift tests. Therefore, the measured load has been adjusted by subtracting the weight of the shaft and attachments. For all lateral and compression tests, the measured loads are net loads (i.e., independent of the weight of the shaft and attachments), because the shaft is in equilibrium under the weight of the shaft and attachments prior to loading, and these weights are not part of the subsequent loading behavior.

RESULTS OF AXIAL AND LATERAL LOAD TESTS

Repeatability of Test Results

Repeatability of the test results was ensured through controlled geometry and initial soil conditions. Attaching reusable model shafts in the same position inside the testing chamber reduced the variability within the same test group resulting from different loading angle, eccentricity, shaft geometry, and boundary conditions. The unit weight measurements discussed in Section 3 showed
good repeatability and consistency throughout the testing program.

Test repeatability was verified by replicate testing in conditions that were considered most likely to be affected by sand deposit preparation and test procedure. This situation refers primarily to the medium dense deposits, in which the unit weight showed more scatter than the others, and small shaft geometry. Figure 5-2 shows the results from four lateral load tests with a D/B of 3 in medium dense sand. For Test 4, the stainless steel threaded rod that normally was used for the hook-up was substituted for a much lighter aluminum rod to investigate the effect of added weight on drilled shaft response. Similar agreement was obtained for replicate tests in inclined compression, as can be seen by comparing Tests 50 and 67 (3L135 and 3L135-1) in Appendix A.

**Effect of Sand Density**

Loose, medium dense, and dense sand deposits were investigated. As discussed in Section 3, the loose and medium dense deposits were normally consolidated, and the dense deposits were overconsolidated. The load-displacement curves for all axial uplift tests had a similar shape, with relatively steep initial response, clearly defined peak, and post-peak softening. Figure 5-3 illustrates the results for loose and medium dense sand. The response in dense sand was similar. These results are not shown because much higher peak loads developed (558 N for D/B = 6 and 1884 N for D/B = 9), and therefore they could not be plotted conveniently with the other results.

![Figure 5-2. Repeatability of Test Results](image-url)
Kulhawy, et al. (3) showed that a drilled shaft in uplift, with low D/B ratio in soil with high strength and therefore high in-situ stresses, can develop a composite failure surface, consisting of a cone of soil near the ground surface and a cylindrical shear below. This type of failure also occurred in this study, as shown in Figure 5-4 by the ground surface relief for the shortest drilled shaft (D/B = 3) in dense sand. The surface of the cone of soil extended to a distance of approximately one shaft diameter out from the test shaft. A detailed discussion of this failure type is given elsewhere (3).

Axial compression tests in loose and medium dense sand resulted in smooth,
nonlinear load-displacement curves, as shown in Figure 5-5. These load-displacement curves consist of an initial linear region, a transition zone, and a final linear region. The curve in medium dense sand is both stiffer and stronger. As noted previously, axial compression tests were not conducted in dense sand.

Typical lateral tests on drilled shafts are shown in Figure 5-6. The load-displacement curves for the loose and medium dense sands were smooth, nonlinear, and without a clearly defined failure zone. However, for the dense sand, the load-displacement curves were much stiffer, with a clearly-defined peak followed by post-peak softening. Detailed analysis of this behavior is given by Agaiby, et al. (2).

Lateral tests on drilled shafts in dense sand resulted in failure wedges that formed in front of and behind the shafts, as shown in Figure 5-7 for D/B = 6. The wedge behind the shaft is larger than in front and extends approximately six shaft diameters out from the shaft. This wedge apparently resulted from the kick-out of soil around the shaft tip (base) following shaft rotation. Similar wedges were observed for D/B = 3. Failure wedges behind the shaft have not been reported in the literature. However, Agaiby, et al. (2) recorded failure wedges in front of laterally loaded shafts in dense sand with a plaster cast and provided accurate measurements of their geometry.
Figure 5-5. Effect of Sand Density on Axial Compression Response
Figure 5-6. Effect of Sand Density on Lateral Response
Effect of Shaft Geometry

As described in Section 4, the model drilled shafts had a diameter (B) of 52 mm (2.05 in) and depth to diameter (D/B) ratios of 3, 6, and 9. The effect of geometry on drilled shaft response was evaluated by comparing the test results for all D/B ratios under comparable test conditions.

Figure 5-8 shows the load-displacement curves for axial uplift tests in loose, medium dense, and dense sand. Stiffer responses and higher capacities were obtained for the deeper shafts, and post-peak softening occurred for all tests.

Axial compression tests were conducted in loose and medium dense sand, as shown in Figure 5-9. Again, as the shaft becomes deeper, the response stiffens and higher loads develop. However, the increase in load with increasing depth is not as pronounced as with axial uplift. Furthermore, the shape of the load-displacement curves is different, with a final linear region and no post-peak softening.

The results for laterally loaded shafts in loose, medium dense, and dense sand are shown in Figure 5-10. Again, a stiffer response developed with deeper shafts.
Figure 5-8. Effect of Shaft Geometry on Axial Uplift Response
Figure 5-9. Effect of Shaft Geometry on Axial Compression Response

Interpretation of Failure Load

Methods used for interpreting failure loads from axial uplift, axial compression, and lateral load-displacement curves were presented in Section 1. These methods are based on the recommendations made by Hirany and Kulhawy (4 - 7). Detailed comparisons between interpreted failure loads and theoretical solutions for axial and lateral load tests are given by Turner and Kulhawy (1) and Agaiby, et al. (2).

To differentiate among the load inclinations, the applied loads are denoted by
Figure 5-10. Effect of Shaft Geometry on Lateral Response
the letter P, followed by the load inclination angle given in subscript. Moreover, to distinguish between the computed shaft capacity (described in Sections 1 and 2) and the interpreted failure load from the load tests, a subscript f is used in Sections 5 through 8 to denote the interpreted failure load.

The failure loads in axial uplift were determined as the peak loads from the test results, as shown in Figure 5-11. The peak loads were defined clearly for all of the axial uplift load-displacement curves that are given in Appendix A and are summarized in Table 5-1. These curves show post-peak softening, which is normally not the case with field axial uplift tests. The load tests in this study were conducted under constant rate of displacement. In this mode, it is easy to determine the peak or failure load from a load-displacement curve. However, the majority of field tests are conducted in force control. In this mode, the load-displacement curves do not show a peak or post-peak softening; instead, they are monotonic and look like the load-displacement curves for axial compression.

Axial uplift failure loads were used in Section 8 to normalize the failure loads interpreted for other load inclinations. Because of the very low soil resistance, it was not possible to measure reliably the axial uplift response of the shortest test shaft with D/B = 3 in loose sand. Therefore, this failure load was obtained from a nonlinear regression fit of normalized uplift failure load versus D/B, as shown in Figure 5-12. Interpreted failure loads for each soil density were normalized by a reference failure load that was chosen at D/B = 6.
<table>
<thead>
<tr>
<th>Test Designation</th>
<th>D/B</th>
<th>Sand Density</th>
<th>Applied Load Inclination&lt;sup&gt;a&lt;/sup&gt; (degrees)</th>
<th>Test Type&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Test Number</th>
<th>Interpreted Failure Load (N)</th>
</tr>
</thead>
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<tr>
<td>3L60</td>
<td>3</td>
<td>Loose</td>
<td>0</td>
<td>AU</td>
<td>N.A.</td>
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<td>AU</td>
<td>14</td>
<td>31.0</td>
</tr>
<tr>
<td>9L60</td>
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<td>AU</td>
<td>13</td>
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<td>AU</td>
<td>17</td>
<td>6.5</td>
</tr>
<tr>
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<td>AU</td>
<td>12</td>
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<td>L</td>
<td>6</td>
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<td>90</td>
<td>L</td>
<td>9</td>
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<tr>
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<td>Medium</td>
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<td>L</td>
<td>3</td>
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<tr>
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<td>126.7</td>
</tr>
<tr>
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<td>24</td>
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<td>AC</td>
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<td>518.6</td>
</tr>
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<td>AC</td>
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</tr>
<tr>
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<td>AC</td>
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<td>Medium</td>
<td>180</td>
<td>AC</td>
<td>22</td>
<td>1571.0</td>
</tr>
</tbody>
</table>

<sup>a</sup> - angle measured from upward vertical axis
<sup>b</sup> - AU = axial uplift; L = lateral; AC = axial compression
N.A. = not available (Equation 5-1 used to calculate the failure load)
1 N = 0.225 lb

because it represents an intermediate geometry for the range studied. With this normalization, the following correlation was obtained:

\[ \frac{P_{0f}}{P_{0f \text{ at } D/B = 6}} = 0.008 \left( \frac{D/B}{6} \right)^{2.70} \] (5-1)

in which \( P_{0f} \) = interpreted axial uplift failure load. The statistics given in Figure 5-12 were obtained from a least-squares regression analysis on the transformed (linearized) variables. As shown later, the \( P_{0f} \) for the shaft with \( D/B = 6 \) in loose sand was interpreted as 31 N (7 lb). Using Equation 5-1, the \( P_{0f} \) for
the shortest shaft with \( D/B = 3 \) in loose sand was calculated as \( 4.8 \) N (1.1 lb).

Figure 5-13 shows a typical load-displacement curve for an axial compression test. The failure load was interpreted as the failure threshold, i.e., the load at the initiation of the final linear region of the curve. This final linear region was displayed prominently for all of the axial compression load-displacement curves that are given in Appendix A. The interpreted failure loads are summarized in Table 5-1.

The lateral/moment limit method (4, 6) for interpreting lateral load tests could not be applied to some of the tests in this study. Figure 5-14 shows a typical relationship between the normalized apparent depth of rotation and the applied load for a lateral test. The initial part of the normalized ADOR plot shows some noise (test scatter) or erratic spikes because the shaft rotation was very close to zero at the beginning of the test. However, for the most part, the relationship is a straight horizontal line that does not conform to any of the interpretation shapes established by Hirany and Kulhawy (4, 6). This departure might result from scaling influences on the response of drilled shafts tested in the laboratory. The lateral/moment limit method was developed for full-scale field tests and does not account for scaling effects.

Therefore, the correlation between the lateral/moment limit and the hyperbolic
Figure 5-13. Interpreted Failure Load for Typical Axial Compression Test

Figure 5-14. Typical Relationship Between Normalized Apparent Depth of Rotation and Applied Load for Laterally Loaded Shaft

capacity given by Agaiby, et al. (2) was used to determine the failure loads from the lateral load tests. The hyperbolic capacity (8) can be obtained by establishing a linear relationship between the horizontal groundline displacement (δ) and the lateral load (H) in the following form:

\[
\frac{\delta}{H} = a + b\delta
\]  

(5-2)
in which \( a = \) intercept of the \( \delta/H \) versus \( \delta \) curve and \( b = \) slope of the \( \delta/H \) versus \( \delta \) curve. The hyperbolic capacity can be calculated as \( 1/b \), while the initial stiffness of the hyperbolic fit is given by \( 1/a \). According to Agaiby, et al. (2), the lateral/moment limit is, on average, equal to 0.7 times the hyperbolic capacity. Consequently, the failure load for the lateral tests in loose and medium dense sand was obtained as 70 percent of the calculated hyperbolic capacity. For the lateral tests in dense sand, the failure loads were interpreted from the load-displacement curves in Appendix A as the peak loads. The interpreted failure loads are summarized in Table 5-1.

**SUMMARY**

Axial uplift, axial compression, and lateral load tests were performed to investigate the effect of shaft geometry and soil density on the response of model drilled shafts. The results were used to establish the limiting conditions for the inclined load tests that are discussed in Sections 6 and 7. Typical load-displacement curves for the three loading modes were presented. The methods available for the interpretation of failure loads from load tests were analyzed. Graphical procedures were used for the axial uplift and compression and dense lateral tests while, for the loose and medium dense lateral tests, the failure load was obtained as 70 percent of the shaft hyperbolic capacity.

**REFERENCES**


1160-1172.


Section 6

RESPONSE OF MODEL DRILLED SHAFTS TO INCLINED COMPRESSION

A comprehensive parametric study was conducted to develop a basic understanding of drilled shaft behavior under inclined compression loading. Parameters investigated included the initial soil conditions, shaft geometry, and load inclination. The laboratory tests were conducted to isolate the influence of each parameter on drilled shaft performance. The results of this study are presented in this section.

A qualitative assessment of behavior was developed through multi-level comparison plots given in parallel form for each of the influential variables. The individual test results of load versus displacement, horizontal load component versus rotation, normalized apparent depth of rotation versus horizontal load component, and normalized apparent depth of rotation versus horizontal displacement are presented in Appendix A.

RESULTS OF INCLINED COMPRESSION TESTS

Laboratory tests were conducted using the fully-automated testing system described in Section 4. With the two swivels placed between the MTS load cell and the test shaft, the shafts were unrestrained, and the moment resulting from soil resistance to the load cell was eliminated. As described in Section 5, the measured loads for the compression tests were net loads. For the inclined tests, the vertical and horizontal load components were plotted versus the corresponding displacements, and the resultant inclined load was plotted versus the corresponding resultant displacement.

Data reduction provided the vertical and horizontal displacement components for each test, from which the resultant displacement was calculated as the square root of the sum of the squares. This resultant displacement did not coincide with the direction of loading because the model drilled shafts were unrestrained. The controlling factor for shaft movement is the soil resistance and therefore, as the shaft was displaced, it always followed the path of least resistance. For the tests herein, the resistance to axial compression was much
higher than to lateral loading. Therefore, for the same magnitude of load components in inclined compression, the shaft vertical movement always was smaller than the horizontal.

To represent the load-displacement behavior, either resultant or component curves can be used. For simplicity, the resultant load versus resultant displacement was chosen for this section to evaluate the load-displacement response. However, both resultant and component load-displacement curves are given in Appendix A.

**Effect of Sand Density**

Loose, medium dense, and dense sand deposits were investigated. However, boundary conditions precluded some tests in dense sand, as described subsequently. The loose and medium dense deposits were normally consolidated and resulted in overall similar test response. The dense deposits were overconsolidated, and their response was markedly different.

Figures 6-1 and 6-2 show the effect of sand density on drilled shaft response. For all load inclinations, the tests in medium dense sand showed higher initial stiffness and loads than the tests in loose sand. The load-displacement curves were monotonic and without a clearly defined maximum value. The response in dense sand was much stiffer, and the load-displacement curves had a clearly defined peak followed by post-peak softening.

A key parameter for understanding the overall response is the shaft rotation. Figures 6-3 and 6-4 show plots of normalized apparent depth of rotation (as described in Section 1) versus the horizontal displacement component for the same conditions as Figures 6-1 and 6-2. In these figures, the normalized apparent depth of rotation curves for loose and medium dense sand were similar, and the shafts rotated closer to the shaft tip in loose sand. This behavior was expected because loose sand has lower strength, and therefore a larger mass of soil must be mobilized in front of the shaft to resist the same horizontal load component. As the soil strength increased with medium dense and then dense sand, the static equilibrium requirement forced the point of rotation upward to give a lower normalized apparent depth of rotation. The rotation/displacement pattern that facilitated this shift in the point of rotation was governed by the loading mode. In inclined compression, drilled shafts were forced to displace horizontally and mobilize soil lateral resistance because the resistance to
Figure 6-1. Effect of Sand Density on Inclined Compression Response for $\psi$ – 135 Degrees
Figure 6-2. Effect of Sand Density on Inclined Compression Response for $\psi = 165^\circ$ Degrees
Figure 6-3. Effect of Sand Density on ADOR for $\psi = 135^\circ$ Degrees
Figure 6-4. Effect of Sand Density on ADOR for $\psi = 165$ Degrees
axial loading was much higher than to lateral loading. As will be shown in Section 7, this situation can reverse in inclined uplift, depending on the applied load inclination.

In dense sand, the apparent depth of rotation decreased once the displacement corresponding to the peak load on the load-displacement diagram had been reached. At peak load, soil yielding occurred and was accompanied by the development of failure wedges in front of and behind the shaft. No additional soil resistance could be mobilized past failure. Instead, post-peak softening developed. The upward displacement of the failure wedges also moved the point of shaft rotation upward to maintain the moment balance of the resisting horizontal forces acting on the shaft above and below the point of rotation.

Another interesting phenomenon was noted for inclined compression tests conducted in dense sand, as shown in Figure 6-5 for tests at 135 degrees with D/B of 3 and 6. Solid lines in these figures denote applied load versus resultant displacement, while dashed lines are used to represent the component loads versus their corresponding component displacements. Even though the shafts were loaded in compression and were pushed into the sand at the angle of 135 degrees, the initial vertical movement of the shafts was negative, indicating that the shafts actually moved upward.

Although the vertical and horizontal components of the applied inclined load were equal in these tests, the shaft moved horizontally much more than vertically because of lower resistance to lateral loads. As a result of shaft rotation, the shaft tip rotated also and, because of the tip resistance to compression, the shaft was displaced vertically upward.

The shaft upward vertical movement has been observed in the lateral tests where the horizontal displacement and rotation are especially pronounced (Figure 6-6). The tip resistance to tilting is the primary mechanism that displaces the shaft upward. However, even with this mechanism removed (e.g., by having a cone-shaped tip), the shaft still is displaced vertically upward because of the friction that develops along the contact of the rotated shaft and the surrounding soil. In dense sand, it is easy to visualize this mechanism because of the upward displacement of the frontal failure wedge that mobilizes the friction resistance along the soil-shaft interface, as described later in this section.

Consequently, this predominant lateral movement and rotation that occurred
Figure 6-5. Load versus Displacement for Tests 3D135 and 6D135

during the test caused the shaft to move upward. However, this behavior was not observed for tests conducted at 165 degrees, as given in Figure 6-7. In this case, the horizontal load component was much smaller than the vertical. Therefore, the shaft upward vertical movement caused by the tip rotation and the wedge-mobilized friction was smaller than the shaft settlement under the vertical load component.

Inclined compression tests in dense sand resulted in failure wedges that formed in front of and behind the shafts, as shown in Figure 6-8. Figure 6-8c suggests some boundary effects as described later. The wedge in front of the shaft
Figure 6-6. Typical Load versus Displacement for Laterally Loaded Shaft in Dense Sand

Figure 6-7. Load versus Displacement for Test 3D165

resulted from the upward displacement of soil within the failure zone during shaft rotation. The wedge behind the shaft resulted from the kick-out of soil around the shaft tip during shaft rotation and was noticeably larger than the front wedge in all cases.

Inclined compression tests can cause substantial loads to be transmitted to the shaft tip, and therefore a larger mass of soil can be mobilized at failure than
a) Inclined compression test, \( \psi = 135^\circ \)
Dense sand
\( B = 52 \text{ mm}, \ D/B = 3 \)
\( 1 \text{ mm} = 0.039 \text{ in} \)

b) Inclined compression test, \( \psi = 165^\circ \)
Dense sand
\( B = 52 \text{ mm}, \ D/B = 3 \)
\( 1 \text{ mm} = 0.039 \text{ in} \)

c) Inclined compression test, \( \psi = 135^\circ \)
Dense sand
\( B = 52 \text{ mm}, \ D/B = 6 \)
\( 1 \text{ mm} = 0.039 \text{ in} \)

Figure 6-8. Failure Wedges in Dense Sand During Inclined Compression Loading
in a lateral test. Subsequently, when the shaft tip rotated, the failure wedge that was displaced behind the shaft was larger for inclined compression than for lateral loading, as can be seen by comparing Figures 5-7 and 6-8c. For the same reason, when comparing inclined compression tests conducted at different load inclinations, a larger failure wedge developed behind the shaft for the test where the applied load deviated less from the vertical, thereby causing higher loads to be transmitted to the shaft tip. This was the case with tests 3D135 and 3D165, which had failure wedges that are presented in Figures 6-8a and 6-8b, respectively. However, it is important to note that shaft rotation is necessary to create a failure wedge through kick-out of the soil around the shaft tip. Because of the absence of shaft rotation, failure wedges can not develop in axial compression, even though this loading mode mobilizes the largest amount of soil around the shaft tip at failure.

Figure 6-8a shows further that two concentric wedges developed in front of the shaft, instead of the commonly found single failure wedge. Similar results were reported by Agaiby, et al. (1) for lateral tests conducted in dense sand. It is important to note that the outer wedge developed first, followed by the inner wedge. In addition, this behavior was observed only for the test conducted at 135 degrees on the shortest shaft with D/B of 3 (Test 3D135). In this test, very high shaft rotation that exceeded 12 degrees had occurred when the inner wedge formed. It is believed that this high rotation forced the inner wedge out of the already displaced outer wedge through localized failure.

In dense sand, deeper shafts produced failure wedges that extended farther away from the shaft because a larger mass of soil was mobilized at failure, as can be seen by comparing Figures 6-8a and 6-8c. Figure 6-8c also suggests that the shape of the failure wedge probably was affected by the boundary; otherwise, the failure wedge would have had the same rounded shape as that in Figure 6-8a. Because of this boundary influence, test 9D135 was not conducted. Since test 3D165 (Figure 6-8b) showed a larger wedge than test 3D135, the boundary influences would be large for tests 6D165 and 9D165, and therefore they were not conducted.

No failure wedges developed in either loose or medium dense sand deposits.

**Effect of Shaft Geometry**

The effect of geometry on drilled shaft response was evaluated by comparing the

6-11
test results for all D/B ratios under comparable initial load inclinations and soil densities, as given in Figures 6-9 and 6-10. In all cases, deeper shafts resulted in stiffer response and higher loads.

Figures 6-11 and 6-12 show the normalized apparent depth of rotation (ADOR) versus the horizontal displacement component for the same conditions as Figures 6-9 and 6-10. These figures show that the normalized ADOR curves were not influenced significantly by the shaft geometry, except for tests 9L165 (Figure 6-12a) and 3M165 (Figure 6-12b). These inconsistencies are attributed to test error or scatter, because all other results in loose and medium dense sand indicated conclusively that, for the same test conditions, shaft geometry does not influence the normalized ADOR.

In dense sand, the normalized ADOR versus horizontal displacement curves had a different shape, characterized by a noticeable drop in the normalized ADOR once the displacement corresponding to the peak load was reached (Figure 6-11c). This reduction occurred during post-peak softening. However, during the initial part of the test and prior to failure, the normalized ADORs were approximately the same for both shaft geometries. As expected, larger displacement at the peak load developed for the deeper shaft, and therefore the normalized ADOR curve for D/B of 6 extended farther from the origin than that for D/B of 3. Nevertheless, following the post-peak softening, the normalized ADORs for both geometries remained constant and approximately the same.

**Effect of Load Inclination**

The effect of load inclination on drilled shaft response was evaluated by comparing the test results at comparable shaft geometries and initial soil conditions, as given in Figures 6-13 through 6-15. In these figures, the corresponding axial compression and lateral test results are added for completeness.

These figures clearly show a stiffer response and higher load as the loading inclination increased from 90 to 180 degrees. This behavior was expected because smaller deviation of the applied load from the vertical resulted in greater mobilization of the shaft tip resistance.

Figures 6-16 through 6-18 show the normalized ADOR versus the horizontal displacement component for the same conditions as Figures 6-13 through 6-15. In all cases, the normalized ADOR increased as the load inclination increased from
Figure 6-9. Effect of Shaft Geometry on Inclined Compression Response for $\psi = 135$ Degrees
90 to 165 degrees, which means that the point of shaft rotation moved closer to the shaft tip as the applied load deviated less from the vertical.

Under purely lateral load, the horizontal soil resistance is mobilized above and below the point of rotation. The moments produced by these forces with respect to the point of rotation add up to counterbalance the moment created by the applied lateral load with respect to the point of rotation. Chari and Meyerhof (2) and Agaiby, et al. (1) measured the horizontal soil stresses that developed above and below the point of rotation for laterally loaded shafts. These soil
Figure 6-11. Effect of Shaft Geometry on ADOR for $\psi = 135$ Degrees
stresses profiles imply that the moment created by the forces above the point of rotation is much higher than the moment resulting from the forces below. Therefore, lowering the point of rotation and increasing the magnitude and moment arm for the forces above the point of rotation will increase the total resisting moment.

Furthermore, when the applied load acts in axial compression, the shaft tip resistance does not produce a moment with respect to any point on the shaft axis. However, under inclined compression, the shaft rotates and the tip resistance produces a moment with respect to the point of rotation that has the same sense.
Figure 6.13. Effect of Load Inclination on Inclined Compression Response in Loose Sand
Figure 6-14. Effect of Load Inclination on Inclined Compression Response in Medium Dense Sand
of rotation as the moment produced by the applied inclined load. The only way to compensate for this added moment is to increase the counteracting moment resulting from the soil lateral restraint by lowering the point of rotation. Therefore, for the same magnitude of the horizontal load component, an inclined compression test will result in a lower point of rotation, when compared to a pure lateral test, so that static equilibrium can be satisfied. Similarly, when comparing two inclined compression tests, the less the applied inclined load deviates from the vertical, the higher the tip resistance will be for the same shaft rotation which, in turn, will result in a lower point of rotation.

Figures 6-16 and 6-17 also show that the shape of the normalized ADOR curves in loose and medium dense sand depends on the applied load inclination. For 165 degrees, the normalized ADOR increased initially and then remained essentially constant with increasing horizontal displacement. For 135 degrees, the normalized ADOR increased initially and then decreased with increasing horizontal displacement. Finally, for the lateral tests, the curves were essentially parallel with the x-axis, indicating no change in the ADOR.

In dense sand (Figure 6-18), the shape of the normalized ADOR curves was influenced by the post-peak softening. For 90 and 135 degrees, the normalized ADOR was approximately constant initially, and then it started to decrease once the displacement at the peak load had been reached. During post-peak softening, it
Figure 6-16. Effect of Load Inclination on ADOR in Loose Sand
Figure 6-17. Effect of Load Inclination on ADOR in Medium Dense Sand
remained constant with increasing horizontal displacement. A different shape was observed for 165 degrees. According to Turner and Kulhawy (3), the shaft tip resistance mobilized during axial compression in dense sand can be very high. The applied inclined compression load for test 3D165 deviated only 15 degrees from the vertical. Therefore, the substantial vertical load component in this test generated high tip resistance that forced the point of rotation closer to the shaft tip, as described previously.

A summary of the average normalized ADOR measurements is presented in Table 6-1. Since the shaft geometry did not influence the ADOR, the average normalized ADOR for a sand density and load inclination was obtained by averaging the normalized ADORs from the pertinent tests. For the lateral tests (90 degrees) in loose and medium dense sand, the normalized ADOR did not change throughout the test. For the 135 degree tests in loose and medium dense sand, the normalized ADORs in Table 6-1 were the peak values from the normalized ADOR versus horizontal displacement plots. For the 165 degree tests in loose and medium dense sand, the normalized ADOR was given by the flat portion of the normalized ADOR versus horizontal displacement plots. Finally, for the tests in dense sand, the normalized ADOR values correspond to the horizontal displacement at the peak load.

The relationship between the normalized ADOR and load inclination for the three sand densities is shown in Figure 6-19.
Table 6-1
AVERAGE NORMALIZED ADOR FOR INCLINED COMPRESSION TESTS

<table>
<thead>
<tr>
<th>Sand Density</th>
<th>Applied Load Inclination(^a) (degrees)</th>
<th>Average ADOR/Shaft Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>90</td>
<td>0.74</td>
</tr>
<tr>
<td>Loose</td>
<td>135</td>
<td>0.84</td>
</tr>
<tr>
<td>Loose</td>
<td>165</td>
<td>0.91</td>
</tr>
<tr>
<td>Medium</td>
<td>90</td>
<td>0.73</td>
</tr>
<tr>
<td>Medium</td>
<td>135</td>
<td>0.78</td>
</tr>
<tr>
<td>Medium</td>
<td>165</td>
<td>0.87</td>
</tr>
<tr>
<td>Dense</td>
<td>90</td>
<td>0.69</td>
</tr>
<tr>
<td>Dense</td>
<td>135</td>
<td>0.70</td>
</tr>
<tr>
<td>Dense</td>
<td>165</td>
<td>0.78</td>
</tr>
</tbody>
</table>

\(^a\) - angle measured from upward vertical axis

Figure 6-19. Average Normalized ADOR versus Applied Load Inclination for Inclined Compression

INTERPRETATION OF FAILURE LOAD

The lateral/moment limit method (4), described in Section 1, was used to interpret the failure loads from the inclined compression tests. This method was derived from analysis of full-scale field lateral load tests, and it was suggested (4) that it could be used to interpret inclined load tests.
Figure 6-20 shows a typical plot of the normalized ADOR versus the horizontal load component for an inclined compression test. According to the lateral/moment limit method, this curve is type B, indicating failure by kick-out of the shaft tip. This condition occurs if the soil behind the foundation yields (4, 5), as was observed herein. In this case, the failure load corresponds to the peak in the normalized ADOR versus the horizontal load component curve. Once this horizontal component was determined, the vertical component and then the resultant applied inclined load at failure were back-calculated from trigonometry. Figure 6-21 shows the projection of the interpreted failure loads on the corresponding load-displacement curves.

Table 6-2 summarizes the interpreted failure loads for the inclined compression tests. These loads are the resultant inclined compression loads at failure and are plotted versus D/B in Figures 6-22 and 6-23. In these figures, the interpreted failure loads were normalized by a reference failure load for each sand density, chosen as that at D/B = 6 because it represents an intermediate geometry for the range examined. With this normalization, the following correlations were obtained:

\[
P_{135f}/(P_{135f \text{ at } D/B = 6}) = 0.023 \ (D/B)^{2.08} \quad (6-1)
\]

\[
P_{165f}/(P_{165f \text{ at } D/B = 6}) = 0.055 \ (D/B)^{1.59} \quad (6-2)
\]

![Diagram](attachment:image.png)

**Figure 6-20.** Interpreted Failure Load for Typical Inclined Compression Test
Figure 6-21. Projection of Interpreted Failure Load on Load versus Displacement Curves for Inclined Compression

Table 6-2

INTERPRETED FAILURE LOADS FOR INCLINED COMPRESSION TESTS

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>D/B</th>
<th>Sand Density</th>
<th>Applied Load Inclination(^a) (degrees)</th>
<th>Test Number</th>
<th>Interpreted Failure Load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3L135</td>
<td>3</td>
<td>Loose</td>
<td>135</td>
<td>50</td>
<td>26.7</td>
</tr>
<tr>
<td>6L135</td>
<td>6</td>
<td>Loose</td>
<td>135</td>
<td>49</td>
<td>116.0</td>
</tr>
<tr>
<td>9L135</td>
<td>9</td>
<td>Loose</td>
<td>135</td>
<td>48</td>
<td>243.4</td>
</tr>
<tr>
<td>3M135</td>
<td>3</td>
<td>Medium</td>
<td>135</td>
<td>45</td>
<td>62.3</td>
</tr>
<tr>
<td>6M135</td>
<td>6</td>
<td>Medium</td>
<td>135</td>
<td>46</td>
<td>230.7</td>
</tr>
<tr>
<td>9M135</td>
<td>9</td>
<td>Medium</td>
<td>135</td>
<td>47</td>
<td>463.7</td>
</tr>
<tr>
<td>3D135</td>
<td>3</td>
<td>Dense</td>
<td>135</td>
<td>58</td>
<td>305.4</td>
</tr>
<tr>
<td>6D135</td>
<td>6</td>
<td>Dense</td>
<td>135</td>
<td>59</td>
<td>1856.0</td>
</tr>
<tr>
<td>3L165</td>
<td>3</td>
<td>Loose</td>
<td>165</td>
<td>42</td>
<td>99.4</td>
</tr>
<tr>
<td>6L165</td>
<td>6</td>
<td>Loose</td>
<td>165</td>
<td>44</td>
<td>305.2</td>
</tr>
<tr>
<td>9L165</td>
<td>9</td>
<td>Loose</td>
<td>165</td>
<td>43</td>
<td>504.3</td>
</tr>
<tr>
<td>3M165</td>
<td>3</td>
<td>Medium</td>
<td>165</td>
<td>39</td>
<td>175.8</td>
</tr>
<tr>
<td>6M165</td>
<td>6</td>
<td>Medium</td>
<td>165</td>
<td>40</td>
<td>600.1</td>
</tr>
<tr>
<td>9M165</td>
<td>9</td>
<td>Medium</td>
<td>165</td>
<td>41</td>
<td>1005.0</td>
</tr>
<tr>
<td>3D165</td>
<td>3</td>
<td>Dense</td>
<td>165</td>
<td>60</td>
<td>1420.0</td>
</tr>
</tbody>
</table>

\(^a\) - angle measured from upward vertical axis
1 N = 0.225 lb
in which \( P_{135f} \) and \( P_{165f} \) = interpreted inclined failure loads for \( \psi = 135 \) and 165 degrees. The statistics in Figures 6-22 and 6-23 were obtained from least-squares regression analyses on the transformed (linearized) variables. As can be seen, a good nonlinear fit exists between the normalized interpreted failure load and D/B.
SUMMARY

Laboratory tests were performed on model drilled shafts to investigate the effects of various parameters that control the shaft response to inclined compression loading. The parameters included the initial soil conditions, shaft geometry, and load inclination. The test results analyzed were the load versus displacement and the normalized apparent depth of rotation (ADOR) versus horizontal displacement component.

Increasing soil density resulted in stiffer initial response and higher loads, but lower normalized ADOR. Depending on the sand density and load inclination, drilled shafts did not necessarily settle under inclined compression. In dense sand, drilled shafts actually moved upward during the initial part of an inclined compression test at 135 degrees. For the same soil density and load inclination, deeper shafts resulted in stiffer initial response and higher loads, and the shaft geometry had no apparent influence on the ADOR. The load inclination had a pronounced effect on the shaft rotational pattern. As the applied load deviated less from the vertical, a higher normalized ADOR was obtained.

The lateral/moment limit method was used to interpret the failure loads. These interpreted failure loads were found to increase with increasing D/B and load inclination from 90 to 180 degrees.

REFERENCES


Section 7
RESPONSE OF MODEL DRILLED SHAFTS TO INCLINED UPLIFT

A comprehensive parametric study was conducted to develop a basic understanding of drilled shaft behavior under inclined uplift loading. Parameters investigated included the initial soil conditions, shaft geometry, and load inclination. The laboratory tests were conducted to isolate the influence of each parameter on drilled shaft performance. The results of this study are presented in this section.

The importance of net loads is discussed first, followed by a qualitative assessment of behavior for each of the influential parameters. The comparisons are given in parallel form through multi-level comparison plots. The individual test results of load versus displacement, horizontal load component versus rotation, normalized apparent depth of rotation versus horizontal load component, and normalized apparent depth of rotation versus horizontal displacement are presented in Appendix A.

RESULTS OF INCLINED UPLIFT TESTS

Importance of Net Loads

As described in Section 5, measured loads for the axial and inclined uplift tests included the soil resistance and the weight of the shaft and attachments used for hook-up and instrumentation. These weights are given in Appendix A, and they varied because of different shaft geometries and test configurations. If not accounted for, this added weight could contribute significantly to the total measured inclined uplift load in loose and medium dense sand and for small D/B, thereby influencing the shape of the load-displacement curves. At field scale, this problem is minimal because the foundation weight is relatively small compared to the axial or inclined uplift resistance.

To compare the test results properly, the applied inclined load was resolved into horizontal and vertical load components, and then the added weight was subtracted from the vertical load component. To cause vertical movement, the vertical load component had to exceed the weight of the shaft and attachments.
making this adjustment, the vertical load component became a true net load, and the load-displacement curves depended on the soil resistance alone. Therefore, the load-displacement curves for the inclined uplift tests consist only of the vertical and horizontal load components versus the corresponding displacement components. There is no resultant inclined load plotted versus inclined displacement because of the above issues related to the weights and the fact that shafts do not move in the direction of loading under inclined uplift.

Figure 7-1 shows the inclined load and its components. The total applied inclined load \( (P_\psi) \) is given by:

\[
P_\psi = (P_{\psi v}^2 + P_{\psi h}^2)^{0.5}
\]  

(7-1)

in which \( P_{\psi v} \) = total vertical load component and \( P_{\psi h} \) = horizontal load component. Furthermore, the total vertical load component can be expressed as:

\[
P_{\psi v} = P_\psi \cos \psi = P_{\psi v n} + W
\]  

(7-2)

in which \( \psi \) = applied load inclination (measured from the upward vertical axis), \( P_{\psi v n} \) = net vertical load component, and \( W \) = weight of shaft and attachments. For this study, the net inclined resultant load \( (P_{\psi n}) \) is defined as:

\[
P_{\psi n} = (P_{\psi v n}^2 + P_{\psi h}^2)^{0.5}
\]  

(7-3)

Substituting Equation 7-3 for \( P_{\psi h} \) into Equation 7-1 results in:

Figure 7-1. Applied Inclined Uplift Load and Its Components
\[ P_V = (P_{\psi V}^2 + P_{\psi n}^2 - P_{\psi VN}^2)^{0.5} \]  \hfill (7-4)

Similarly, substituting Equation 7-2 for \( P_{\psi V} \) and \( P_{\psi VN} \) into Equation 7-4 results in:

\[ P_V = \left( (P_{\psi} \cos \psi)^2 + P_{\psi n}^2 - (P_{\psi} \cos \psi - W)^2 \right)^{0.5} \]
\[ - \left( P_{\psi n}^2 + 2 P_{\psi} W \cos \psi - W^2 \right)^{0.5} \]  \hfill (7-5)

which is a quadratic equation with an unknown \( P_{\psi} \). Solving for \( P_{\psi} \) and disregarding the physically impossible solution results in:

\[ P_{\psi} = W \cos \psi + (P_{\psi n}^2 - W^2 \sin^2 \psi)^{0.5} \]  \hfill (7-6)

Equation 7-6 can be used to back-calculate the total applied inclined uplift load, and the net inclined resultant load can be calculated from Equation 7-3. It should be noted that Equation 7-6 is intended for use in inclined uplift. It is not valid for lateral loading with \( \psi = 90 \) degrees.

**Effect of Sand Density**

Loose, medium dense, and dense sand deposits were investigated. However, boundary conditions precluded some inclined uplift tests in dense sand, as described later. The loose and medium dense deposits were normally consolidated and resulted in overall similar test response. The dense deposits were overconsolidated, and their response was much stiffer than that in loose and medium dense sand.

Figures 7-2 and 7-3 show the effect of sand density on drilled shaft response. For all load inclinations, the tests in medium dense sand showed higher initial stiffness and loads than the tests in loose sand. At 15 degrees, the load-displacement curves in loose and medium dense sand had a clearly defined peak followed by post-peak softening that strongly resembled the corresponding axial uplift curves. At 45 degrees, the curves were flatter, having a shape more typical of the corresponding lateral tests. In Figure 7-3a, only the horizontal load component versus the horizontal displacement component is shown for test 3La5, because the vertical load component was too low to be measured. Similar measurement problems because of low soil resistance were reported previously for test 3L0.
Figure 7-2. Effect of Sand Density on Inclined Uplift Response for $\psi = 15$ Degrees
Figure 7-3. Effect of Sand Density on Inclined Uplift Response for $\psi = 45$ Degrees
In dense sand, the initial stiffness and load were much higher than those for loose or medium dense sand. All of the load-displacement curves had a clearly defined peak followed by post-peak softening.

The rotation pattern in inclined uplift differed somewhat from that in inclined compression. Figures 7-4 and 7-5 show the normalized apparent depth of rotation (ADOR) versus the horizontal displacement component for the same conditions as Figures 7-2 and 7-3. At 45 degrees, the effect of soil density on the normalized ADOR was similar to that for the inclined compression and lateral tests. For the lateral tests, loose sand resulted in a lower point of shaft rotation than medium dense or dense sand, because a larger mass of soil must be mobilized in front of the shaft to sustain the same applied load. The same pattern is evident for the inclined uplift tests conducted at 45 degrees. As can be seen from the corresponding load-displacement curves, the shaft horizontal movement was much larger than the vertical movement. Therefore, the mobilized soil lateral resistance controlled the point of rotation in much the same way it did for the inclined compression or lateral tests. However, the differences in the normalized ADOR response for varying soil densities were less pronounced for the inclined uplift tests.

As the load deviated more from the horizontal, the effect of sand density on the normalized ADOR changed. At 15 degrees, the point of rotation was higher for the loose sand than for the medium dense or dense sand, as can be seen from the lower normalized ADOR. When the applied inclined uplift load was close to the vertical, the axial uplift loading mode dominated the displacement/rotation pattern, forcing the shafts to move more vertically than horizontally. Moreover, the shaft rotation was very small as the shafts moved upward and out of the soil. Consequently, the small horizontal movement of the shafts was insufficient to produce the soil lateral resistance that would control the point of rotation in the same way as for the inclined compression or lateral tests.

In addition, the reduction of the mobilized soil lateral stresses with decreasing load inclination, such as for $\psi = 15$ degrees, did not occur proportionally for all three densities. As can be seen from Table 5-1, the ratio of lateral to axial uplift resistance decreased with increasing soil density. For the loose sand, this ratio exceeded 3.0 while, for the dense sand, it decreased to below 1.5. Therefore, for $\psi = 15$ degrees and given load level (e.g., the peak load), the ratio of shaft lateral to axial uplift displacement increased with increasing soil density (Figure 7-2c). Consequently, a larger mass of soil was
Figure 7-4. Effect of Sand Density on ADOR for $\psi = 15$ Degrees
Figure 7-5. Effect of Sand Density on ADOR for $\psi = 45$ Degrees
mobilized in front of the shaft with increasing soil density that, in turn, lowered the point of rotation.

Inclined uplift tests in dense sand resulted in failure wedges that formed in front of the shafts in the direction of loading. Figure 7-6 shows the wedges in tests with $\psi = 15$ degrees that resulted from the upward displacement of soil within the failure zone during shaft rotation. The mechanism involved with the failure wedge development essentially was similar to that for lateral loading, as described in Section 5. However, for inclined uplift, failure also was influenced by the vertical load component and the corresponding vertical displacement. Therefore, the mass of the mobilized soil in front of the shaft at failure was smaller for inclined uplift than for lateral loading, as can be seen by comparing Figures 5-7 and 7-6b.

Figure 7-7 shows the wedges that developed in tests with $\psi = 45$ degrees. In these tests, the horizontal load component was higher than that for the tests conducted with $\psi = 15$ degrees, and this difference resulted in greater shaft horizontal displacement and larger failure wedges. This increase in wedge size with increasing $\psi$ precluded test 9D45. Figure 7-6c shows that the failure wedge for test 9D15 already extended almost to the boundary, and therefore the boundary influence would have been very large for test 9D45.

No failure wedges developed in either loose or medium dense sand deposits.

Effect of Shaft Geometry

The effect of geometry on drilled shaft response was evaluated by comparing the test results at comparable initial load inclinations and sand densities, as given in Figures 7-8 and 7-9. In all cases, deeper shafts resulted in stiffer response and higher loads. In Figure 7-9a, only the horizontal load component is shown for test 3L45, because the vertical load component was too low to be measured.

Figures 7-10 and 7-11 show the normalized apparent depth of rotation (ADOR) versus the horizontal displacement component for the same conditions as Figures 7-8 and 7-9. These figures show that the normalized ADOR curves were not influenced significantly by the shaft geometry, except for test 3L15 (Figure 7-10a). However, as mentioned previously, the soil resistance for uplift tests in loose sand on the shortest shaft was generally too low to be measured reliably, and
a) Inclined uplift test, $\psi=15^\circ$
Dense sand
B = 52 mm, D/B = 3
1 mm = 0.039 in

b) Inclined uplift test, $\psi=15^\circ$
Dense sand
B = 52 mm, D/B = 6
1 mm = 0.039 in

c) Inclined uplift test, $\psi=15^\circ$
Dense sand
B = 52 mm, D/B = 9
1 mm = 0.039 in

Figure 7-6. Failure Wedges in Dense Sand for $\psi$ = 15 Degrees
therefore these results may not be representative of the true shaft behavior. In addition, a noticeable drop in the normalized ADOR for tests 3145 and 3M45 occurred toward the end of the tests when the shafts experienced substantial rotations. However, during the initial part of the tests and for a comparable range of rotations, the normalized ADOR was essentially the same for all D/B values.

In dense sand for $\psi = 45$ degrees (Figure 7-11c), the normalized ADOR decreased after the displacement at peak load had been reached. However, prior to post-peak softening, the normalized ADOR was approximately the same for D/B = 3 and
Figure 7-8. Effect of Shaft Geometry on Inclined Uplift Response for $\psi = 15$ Degrees
Figure 7-9. Effect of Shaft Geometry on Inclined Uplift Response for $\psi = 45$ Degrees
Figure 7-10. Effect of Shaft Geometry on ADOR for $\psi = 15$ Degrees
Figure 7-11. Effect of Shaft Geometry on ADOR for $\psi = 45$ Degrees
6. Similar behavior was described for inclined compression in Section 6.

Effect of Load Inclination

The effect of load inclination on drilled shaft response was evaluated by comparing the test results at comparable shaft geometries and initial soil conditions, as given in Figures 7-12 through 7-14. In these figures, the corresponding axial uplift and lateral test results were added for completeness.

The shape of the load-displacement curves depends significantly on the loading inclination. For $\psi = 15$ degrees, the vertical response closely resembled the corresponding axial uplift test. However, higher loads were obtained in inclined uplift because of the mobilized lateral soil resistance, as should be expected (e.g., 1). For $\psi = 45$ degrees, the results were influenced more by the corresponding lateral tests, because the loads were higher and the post-peak softening was less pronounced. In Figure 7-12a, only the horizontal load component is shown for test 3L45, because the vertical load component was too low to be measured.

The response in inclined uplift was controlled by the load transfer. When the applied load was close to vertical, such as at 15 degrees, the upward vertical movement was characteristic of the axial uplift response. As the applied load deviated more from the vertical, such as at 45 degrees, higher lateral soil resistance was mobilized through pronounced rotation that was characteristic of the lateral response.

In dense sand, the response was similar for both $\psi = 15$ and 45 degrees, with a clearly defined peak followed by post-peak softening. This response was discussed previously in this section.

Figures 7-15 through 7-17 show the normalized ADOR versus the horizontal displacement component for the same conditions as Figures 7-12 through 7-14. The results from the corresponding lateral tests were added for completeness. In loose and medium dense sand, the normalized ADOR had a clearly defined peak for $\psi = 15$ degrees while, for $\psi = 45$ degrees, the normalized ADOR was either constant throughout the test or decreased to a constant value at the beginning of the test. As with the lateral tests ($\psi = 90$ degrees) in loose and medium dense sand described in Section 5, the normalized ADOR was essentially constant throughout the test.
Figure 7-12. Effect of Load Inclination on Inclined Uplift Response in Loose Sand
Figure 7-13. Effect of Load Inclination on Inclined Uplift Response in Medium Dense Sand
Figure 7-14. Effect of Load Inclination on Inclined Uplift Response in Dense Sand

In dense sand, the normalized ADOR initially was approximately constant, and then it decreased once the displacement at peak load had been reached. In addition, the normalized ADOR increased with decreasing ψ.

A summary of the average normalized ADOR measurements is presented in Table 7-1. Since the shaft geometry did not influence the ADOR, the average normalized ADOR for a sand density and load inclination was obtained by averaging the normalized ADORs from the pertinent tests. For ψ = 15 degrees in loose and medium dense sand, the normalized ADORs in Table 7-1 were the peak values from the normalized
Figure 7-15. Effect of Load Inclination on ADOR in Loose Sand
Figure 7-16. Effect of Load Inclination on ADOR in Medium Dense Sand
ADOR versus horizontal displacement plots. The ADOR selected this way coincided with the ADOR corresponding to the horizontal displacement at peak load. For $\psi = 45$ and 90 degrees in loose and medium dense sand, the normalized ADOR essentially was constant. Finally, for all tests in dense sand, the normalized ADORs in Table 7-1 corresponded to the horizontal displacement at peak load. The relationship between the normalized ADOR and load inclination for the three sand densities is shown in Figure 7-18.

INTERPRETATION OF FAILURE LOAD
The lateral/moment limit method (2), described in Section 1, was used to
Table 7-1
AVERAGE NORMALIZED ADOR FOR INCLINED UPLIFT TESTS

<table>
<thead>
<tr>
<th>Sand Density</th>
<th>Applied Load Inclination(^{\circ})</th>
<th>Average ADOR/Shaft Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>15</td>
<td>0.67</td>
</tr>
<tr>
<td>Loose</td>
<td>45</td>
<td>0.74</td>
</tr>
<tr>
<td>Loose</td>
<td>90</td>
<td>0.74</td>
</tr>
<tr>
<td>Medium</td>
<td>15</td>
<td>0.74</td>
</tr>
<tr>
<td>Medium</td>
<td>45</td>
<td>0.74</td>
</tr>
<tr>
<td>Medium</td>
<td>90</td>
<td>0.73</td>
</tr>
<tr>
<td>Dense</td>
<td>15</td>
<td>0.81</td>
</tr>
<tr>
<td>Dense</td>
<td>45</td>
<td>0.74</td>
</tr>
<tr>
<td>Dense</td>
<td>90</td>
<td>0.69</td>
</tr>
</tbody>
</table>

\(^{a}\) angle measured from upward vertical axis

Figure 7-18. Average Normalized ADOR versus Applied Load Inclination for Inclined Uplift

Interpret the failure loads from the inclined uplift tests. This method was derived from analysis of full-scale field lateral load tests, and it was suggested (2) that it could be used to interpret inclined load tests.

Figure 7-19 shows a typical plot of the normalized ADOR versus the horizontal
load component for the inclined uplift tests. Unlike inclined compression, this curve does not fit the lateral/moment limit classification, primarily because the type of failure observed in inclined uplift does not have a physical meaning in this classification. In inclined uplift, failure corresponds to the peak load, which means that the interpreted failure load can be obtained from the load-displacement curves just as well. However, the lateral/moment limit method still is preferred for the inclined uplift tests, because the failure load can be determined more precisely. This factor becomes particularly important for the load-displacement curves where the peak is not well-pronounced, which can be the case for $\psi = 45$ degrees. Consequently, the lateral/moment limit method was used herein. Once the horizontal component at failure was determined, as shown in Figure 7-19, the corresponding net vertical component was inferred directly from the data. Equation 7-3 then was used to calculate the net resultant load at failure. Figure 7-20 shows the projection of the interpreted failure loads on the load-displacement curves. The failure loads correspond to the peaks of the curves but, since the horizontal and vertical displacements did not accumulate proportionally, the two peaks do not coincide.

Table 7-2 summarizes the interpreted failure loads for the inclined uplift tests. These loads are the net resultant inclined uplift loads at failure calculated from the previously determined horizontal and vertical load components using Equation 7-3. The total applied inclined uplift loads at failure can be
Figure 7-20. Projection of Interpreted Failure Load on Load versus Displacement Curves for Inclined Uplift

Table 7-2

INTERPRETED FAILURE LOADS FOR INCLINED UPLIFT TESTS

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>D/B</th>
<th>Sand Density</th>
<th>Applied Load Inclination^a</th>
<th>Test Number</th>
<th>Interpreted Failure Load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3L15</td>
<td>3</td>
<td>Loose</td>
<td>15</td>
<td>65</td>
<td>13.5</td>
</tr>
<tr>
<td>6L15</td>
<td>6</td>
<td>Loose</td>
<td>15</td>
<td>32</td>
<td>48.0</td>
</tr>
<tr>
<td>9L15</td>
<td>9</td>
<td>Loose</td>
<td>15</td>
<td>33</td>
<td>89.7</td>
</tr>
<tr>
<td>3M15</td>
<td>3</td>
<td>Medium</td>
<td>15</td>
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<td>19.1</td>
</tr>
<tr>
<td>6M15</td>
<td>6</td>
<td>Medium</td>
<td>15</td>
<td>36</td>
<td>69.6</td>
</tr>
<tr>
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<td>15</td>
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<td>6</td>
<td>Dense</td>
<td>15</td>
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<td>6</td>
<td>Dense</td>
<td>45</td>
<td>56</td>
<td>587.3</td>
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</tbody>
</table>

^a - angle measured from upward vertical axis
1 N = 0.225 lb
back-calculated using Equation 7-6.

The interpreted failure loads are plotted in Figures 7-21 and 7-22. In these figures, the interpreted failure loads were normalized by a reference failure load for each sand density, chosen as that at D/B = 6 because it represents an intermediate geometry for the range examined. With this normalization, the following correlations were obtained:

\[
P_{15f}/(P_{15f} \text{ at } D/B = 6) = 0.023 \ (D/B)^{2.10} \tag{7-7}
\]

\[
P_{45f}/(P_{45f} \text{ at } D/B = 6) = 0.026 \ (D/B)^{2.03} \tag{7-8}
\]

in which \(P_{15f}\) and \(P_{45f}\) = interpreted net inclined resultant failure loads for \(\psi = 15\) and 45 degrees. The statistics in Figures 7-21 and 7-22 were obtained from least-squares regression analyses on the transformed (linearized) variables. As can be seen, a good nonlinear fit exists between the normalized interpreted failure load and D/B.

**SUMMARY**

Laboratory tests were performed on model drilled shafts to investigate the effects of various parameters that control the shaft response to inclined uplift loading. The parameters included the initial soil conditions, shaft geometry, and load inclination. The test results analyzed were the load versus

![Figure 7-21. Normalized Inclined Uplift Failure Load at \(\psi = 15\) Degrees versus D/B](image)
displacement and the normalized apparent depth of rotation (ADOR) versus horizontal displacement component.

Increasing soil density resulted in stiffer initial response and higher loads. The normalized ADOR increased with increasing soil density for $\psi = 15$ degrees while, for $\psi = 45$ degrees, the normalized ADOR decreased with increasing soil density. For the same soil density and load inclination, deeper shafts resulted in stiffer initial response and higher loads, and the shaft geometry had no apparent influence on the ADOR. The load inclination had a pronounced effect on both the ADOR and the shape of the load-displacement curves, depending on the relative contribution of the axial uplift and lateral loading modes to overall shaft response.

The lateral/moment limit method was used to interpret the failure loads. These interpreted failure loads were found to increase with increasing D/B and load inclination from 0 to 90 degrees.

REFERENCES


2. Hirany, A. and Kulhawy, F.H., "Conduct and Interpretation of Load Tests on
Section 8

APPLICATION OF MODEL TEST RESULTS

The methods that can be used to predict the response of drilled shafts subjected to inclined loading are presented in this section. The interpreted failure loads were normalized and correlated to the applied load inclination for the range of shaft geometries and sand densities analyzed. Because different trends were observed for inclined uplift and compression, the two loading modes are analyzed separately. In addition, the relationships among the axial, lateral, and inclined capacities that were described in Section 2 are evaluated using the test results from this study.

EFFECT OF LOAD INCLINATION ON DRILLED SHAFT RESPONSE

The change in the interpreted failure load with load inclination can be evaluated by combining the results from Tables 5-1 for the axial and lateral tests, 6-2 for the inclined compression tests, and 7-2 for the inclined uplift tests. It is important to note that the interpreted failure loads, not the ultimate capacities, are analyzed in this section. The first term states that the failure load is an interpreted value from the corresponding load test, while the second term lacks a universally accepted definition (e.g., $L$). Calculation of ultimate capacity for inclined loading requires an assumption of the soil stress distribution at failure and an adequate theoretical procedure. It generally has been recognized that the theoretical analysis of drilled shaft response to inclined loading represents a complex and difficult task because of the inherent soil nonlinearity and nonsymmetrical, three-dimensional nature of the problem. Development of such a theory is beyond the scope of this study, and therefore the correlations presented in this section are confined to the interpreted failure loads.

It was shown in Sections 5 through 7 that higher soil density resulted in higher interpreted failure loads for the same shaft geometry and load inclination. When evaluating the effect of load inclination on the interpreted failure load, the influence of soil density can be reduced or eliminated by normalization. It was found that, for a given shaft geometry and soil density, normalizing the
inclined failure loads by the net failure load in axial uplift resulted in very high quality correlations. Moreover, axial uplift has been investigated thoroughly, and it may be the best understood loading mode for drilled shafts.

**Inclined Uplift**

Interpreted failure loads for the inclined uplift tests were the net inclined resultant failure loads, calculated using Equation 7-3, that were independent of the weight of the shaft and attachments and were dependent only on the soil resistance. To calculate the total applied inclined uplift load at failure, Equation 7-6 can be used.

Figure 8-1 shows the relationships between the normalized interpreted failure load and load inclination in loose and medium dense sand. The symbols denote the experimental results, while the lines indicate the best fit from least-squares regression analyses on the transformed (linearized) variables. The accompanying statistics consist of the coefficient of determination ($r^2$), standard deviation (S.D.), and number of observations (n), and pertain to the transformed (linearized) variables. These statements also pertain to all of the other regressions presented in this section.

As can be seen, the linearity of the correlation increases with increasing D/B. A power fit seemed appropriate for D/B of 3 and 6 while, for D/B of 9, a linear fit (power fit of 1) was more suitable. The normalized results for the loose and medium dense sand followed approximately the same trend. The regression lines for the three shaft geometries are given below:

\[ P_{\psi f}/P_{0f} = 1 + 5.14 (\psi/90^\circ)^{0.550} \quad \text{[for D/B = 3]} \quad (8-1) \]

\[ P_{\psi f}/P_{0f} = 1 + 2.36 (\psi/90^\circ)^{0.799} \quad \text{[for D/B = 6]} \quad (8-2) \]

\[ P_{\psi f}/P_{0f} = 1 + 2.01 (\psi/90^\circ) \quad \text{[for D/B = 9]} \quad (8-3) \]

in which $\psi$ = applied load inclination (in degrees) measured from the upward vertical axis, $P_{\psi f}$ = interpreted net resultant inclined uplift failure load, and $P_{0f}$ = interpreted net axial uplift failure load.

As described in Sections 5 through 7, the tests in dense sand resulted in a different overall response, and therefore these results are presented separately in
Figure 8-1. Normalized Inclined Uplift Failure Load versus Load Inclination in Loose and Medium Dense Sand
Figure 8-2. Regardless of the load inclination, the load displacement curves showed a clearly defined peak followed by post-peak softening. The normalized interpreted failure loads for different D/B were correlated together versus the load inclination, because D/B did not influence the overall trend. The following was obtained from least-squares regression analysis:

\[ \frac{P_{\psi f}}{P_{DF}} = 1 + 0.477 \left( \frac{\psi}{90^\circ} \right)^{3.11} \]  

with the same symbols as before.

**Inclined Compression**

Interpreted failure loads from the inclined compression tests also were independent of the weight of shaft and attachments, as described in Section 6. Therefore, the results were comparable to those from inclined uplift tests. The net interpreted axial uplift failure loads also were used to normalize the inclined compression interpreted failure loads.

Figure 8-3 shows the relationships between the normalized interpreted failure load and load inclination in loose and medium dense sand. Again, symbols denote the experimental results, while lines indicate the best correlation fits. The relationships were nonlinear and required an exponential power fit to model the data, using the following general form:

\[ \frac{P_{\psi f}}{P_{DF}} = 1 + 0.477 \left( \frac{\psi}{90^\circ} \right)^{3.11} \]  

Dense sand \[ r^2 = 0.956 \]  
O \[ D/B = 3 \]  
\[ \text{S.D.} = 0.05 \]  
\[ n = 8 \]  

1 mm = 0.039 in

Figure 8-2. Normalized Inclined Uplift Failure Load versus Load Inclination in Dense Sand
Figure 8-3. Normalized Inclined Compression Failure Load versus Load Inclination in Loose and Medium Dense Sand
\[ P_{\psi f}/P_{0f} = \exp [e + f (\psi/180^\circ)^g] \]  \hspace{1cm} (8-5)

in which \( \psi \) = applied load inclination (in degrees) measured from the upward vertical axis, \( P_{\psi f} \) = interpreted inclined compression failure load, \( P_{0f} \) = interpreted net axial uplift failure load, \( e \) and \( f \) = regression coefficients, and \( g \) = exponent that depends on the nonlinearity of the best fit and must be assumed prior to regression. By taking the natural logarithm of Equation 8-5, the following linear fit can be established:

\[ y = fx + e \]  \hspace{1cm} (8-6)

in which \( y = \ln (P_{\psi f}/P_{0f}) \) and \( x = (\psi/180^\circ)^g \). The coefficients \( e \) and \( f \) then can be determined from least-squares regression analysis. Dividing the angle \( \psi \) by 180 degrees makes the equation dimensionally correct and keeps the coefficients \( e \) and \( f \) within convenient limits. However, selection of the exponent \( (g) \) directly affects the coefficient of determination \( (r^2) \), because it determines how steeply the correlation curve rises at high \( \psi \) values. The exponent \( (g) \) was obtained from a parametric correlation study as the value that yielded the highest \( r^2 \).

Figure 8-3 shows that increasing load inclination resulted in nonlinearly increasing normalized interpreted failure load, and the nonlinearity increased with decreasing \( D/B \). Normalizing the loads reduced the effect of soil density, but it did not eliminate it completely, as with inclined uplift. This effect occurs primarily because of the nonlinear mobilized tip resistance.

For the loose sand, the regression lines for the three shaft geometries are given below:

\[ P_{\psi f}/P_{0f} = \exp [1.67 + 2.69 (\psi/180^\circ)^{8.70}] \]  \hspace{1cm} [for D/B = 3]  \hspace{1cm} (8-7)

\[ P_{\psi f}/P_{0f} = \exp [1.16 + 1.70 (\psi/180^\circ)^{6.10}] \]  \hspace{1cm} [for D/B = 6]  \hspace{1cm} (8-8)

\[ P_{\psi f}/P_{0f} = \exp [1.07 + 1.04 (\psi/180^\circ)^{5.80}] \]  \hspace{1cm} [for D/B = 9]  \hspace{1cm} (8-9)

For the medium dense sand, the regression lines for the three shaft geometries are given below:

\[ P_{\psi f}/P_{0f} = \exp [1.78 + 2.79 (\psi/180^\circ)^{6.60}] \]  \hspace{1cm} [for D/B = 3]  \hspace{1cm} (8-10)
\[ P_{\psi f}/P_{0f} = \exp [1.11 + 2.37 (\psi/180^\circ)^{5.20}] \quad [\text{for } D/B = 6] \quad (8-11) \]

\[ P_{\psi f}/P_{0f} = \exp [0.988 + 1.62 (\psi/180^\circ)^{4.60}] \quad [\text{for } D/B = 9] \quad (8-12) \]

Tests in dense sand showed that the shaft geometry did not influence the relationship between the normalized interpreted failure load and load inclination, as shown in Figure 8-4. The same conclusion was reached for inclined uplift. However, for inclined compression, boundary effects precluded some of the tests. Therefore, no test data were available for \( \psi = 180 \) degrees and, for 165 degrees, only the shortest shaft (D/B = 3) was tested. Using Kulhawy's method (2), the axial compression (\( \psi = 180 \) degrees) capacities for D/B = 3 and 6 were calculated as 2.48 kN (0.558 kips) and 4.91 kN (1.10 kips), giving computed \( P_{\psi f}/P_{0f} = 30.2 \) for D/B = 3 and 8.80 for D/B = 6. These computed points are not plotted in Figure 8-4, but they were accounted for in the regression analysis. The best fit was nonlinear, as given by the following for D/B = 3 and 6:

\[ P_{\psi f}/P_{0f} = \exp [0.035 + 2.94 (\psi/180^\circ)^{2.90}] \quad (8-13) \]

The solid line corresponds to the range of load inclinations within which experimental data were available, while the dashed line extends the trend to include the calculated values for \( \psi = 180 \) degrees.

The regression coefficients obtained herein can not be linked to a specific

![Figure 8-4. Normalized Inclined Compression Failure Load versus Load Inclination in Dense Sand](image)
geotechnical property because of the normalization used. The interpreted inclined failure loads were normalized by the corresponding net axial uplift failure load, which is influenced by several geotechnical properties including the soil density, in-situ coefficient of horizontal soil stress, effective stress friction angle, interface friction angle, and shaft geometry. Therefore, there is only limited justification for isolating any one of these properties to establish a link with the regression coefficients. Attempts to normalize the interpreted failure loads by a product of density and geometry (e.g., \( \gamma B D^2 \), \( \gamma B^2 D \), or \( \gamma B^3 \)) and other geotechnical parameters did not result in improved correlations over those given herein.

**EFFECT OF DEPTH ON INTERPRETED FAILURE LOAD**

Figures 8-5 and 8-6 show the relationships between the normalized interpreted failure load and \( D/B \) for load inclinations (\( \psi \)) ranging from 15 to 180 degrees. The best regression correlations were obtained using the inverse exponential power fit given below:

\[
P_{\psi f}/P_{0f} = c \exp \left[ d/(D/B) \right]
\]  

(8-14)

in which \( c \) and \( d \) = regression coefficients, and all other terms are as defined previously.

Sand density did not appear to affect the normalized interpreted failure load in inclined uplift. Therefore, for \( \psi = 15 \) to 90 degrees, Figure 8-5 shows good correlations for the loose and medium dense sand combined, as given below:

\[
P_{15 f}/P_{0f} = 0.809 \exp \left[ 3.82/(D/B) \right]
\]  

(8-15)

\[
P_{45 f}/P_{0f} = 1.22 \exp \left[ 4.04/(D/B) \right]
\]  

(8-16)

\[
P_{90 f}/P_{0f} = 2.09 \exp \left[ 3.10/(D/B) \right]
\]  

(8-17)

For \( \psi = 135 \) to 180 degrees, the sand density influenced the normalized interpreted failure load, and therefore separate curves are provided for the loose and medium dense sand in Figure 8-6. As mentioned previously, this divergence occurs primarily because of the nonlinear mobilized tip resistance. For the loose sand, the correlations are given below:
Figure 8-5. Normalized Inclined Uplift and Lateral Failure Load versus D/B in Loose and Medium Dense Sand
Figure 8-6. Normalized Inclined and Axial Compression Failure Load versus D/B in Loose and Medium Dense Sand
\[ \frac{P_{135f}}{P_{0f}} = 2.33 \exp \left[ \frac{2.64}{(D/B)} \right] \]  
\[ \frac{P_{165f}}{P_{0f}} = 3.81 \exp \left[ \frac{5.15}{(D/B)} \right] \]  
\[ \frac{P_{180f}}{P_{0f}} = 2.82 \exp \left[ \frac{9.98}{(D/B)} \right] \]  

For the medium dense sand, the correlations are given below:

\[ \frac{P_{135f}}{P_{0f}} = 2.63 \exp \left[ \frac{3.91}{(D/B)} \right] \]  
\[ \frac{P_{165f}}{P_{0f}} = 5.30 \exp \left[ \frac{4.98}{(D/B)} \right] \]  
\[ \frac{P_{180f}}{P_{0f}} = 6.20 \exp \left[ \frac{8.49}{(D/B)} \right] \]  

In dense sand, the normalized interpreted failure load was not influenced by the shaft geometry for a load inclination (\( \phi \)) from 15 to 180 degrees, as can be seen in Figures 8-2 and 8-4. Therefore, interpolation curves are not needed.

It is important to note that Figures 8-5 and 8-6, and the corresponding correlations given in Equations 8-15 through 8-23, are intended for interpolation only for D/B between 3 and 9. Use of the correlations beyond this range (extrapolation) must be avoided.

RELATIONSHIP AMONG AXIAL, LATERAL, AND INCLINED RESPONSES OF DRILLED SHAFTS

In Section 2, several relationships were described that have been proposed to interrelate the axial, lateral, and inclined capacities. These relationships are compared to the test results from this study in the following paragraphs.

For inclined uplift, Equation 2-17 was suggested as an "approximate parabolic interaction relationship between the ultimate inclined pull-out resistance and the limits of vertical and horizontal pulling resistance" (3). This equation was found to be only partially useful for drilled shafts. In loose and medium dense sand, Equation 2-17 proved unsuitable because it greatly underpredicted the experimental results, as shown in Figures 8-7 and 8-8. In dense sand, however, Equation 2-17 provided a fair agreement, as shown in Figure 8-9. It should be noted that Equation 2-17 was proposed to interrelate the gross ultimate axial uplift, inclined uplift, and lateral resistances, while only the net interpreted failure loads are addressed herein. The difficulties and
Figure 8-7. Comparison of Experimental Results and Equation 2-17 in Loose Sand
Figure 8-8. Comparison of Experimental Results and Equation 2-17 in Medium Dense Sand
inconsistencies associated with comparison of gross inclined uplift loads were described in Section 7.

For inclined compression, Equation 2-4 was suggested as a modification of the well-known structural interaction formula (4, 5). This equation gave good agreement with the experimental results for the loose and medium dense sand, as shown in Figures 8-10 and 8-11. However, in dense sand, Equation 2-4 underpredicted the experimental results, as shown in Figure 8-12. As mentioned previously, axial compression tests, as well as some inclined compression tests, were not conducted in dense sand because of boundary effects. The axial compression
Figure 8-10. Comparison of Experimental Results and Equation 2-4 in Loose Sand

1 N = 0.225 lb
1 mm = 0.039 in
Figure 8-11. Comparison of Experimental Results and Equation 2-4 in Medium Dense Sand
capacity for D/B = 3 in dense sand therefore was calculated from available theory.

The pile inclination factor given as Equation 2-6 (6) proved unsuitable for the drilled shafts analyzed in this study. The same goes for the pile inclination factor that is given as Equation 2-7 (7). These factors greatly overpredicted the experimental results, as shown in Figures 8-13 and 8-14. The response of drilled shafts to inclined loading is a complex problem, and the simplified concept of inclination factors does not provide an adequate solution at this time.

The disadvantage of interaction equations, such as Equations 2-4 and 2-17, compared to Equations 8-1 through 8-4 and 8-7 through 8-13, is that both the axial and lateral capacities are needed to estimate an inclined capacity. Therefore, the interaction equations involve necessarily more uncertainty than the correlations proposed in this study, which require only one parameter (axial uplift capacity) that can be calculated readily or can be interpreted from a commonly conducted field load test. Accordingly, the correlations obtained herein should be a first choice for the design of drilled shafts subjected to inclined loading. Equation 2-4 can be used as a second estimate.

SUMMARY

Interpreted failure loads presented in Sections 5 through 7 for various load
Figure 8-13. Comparison of Experimental Results and Equations 2-6 and 2-7 in Loose Sand
Figure 8-14. Comparison of Experimental Results and Equations 2-6 and 2-7 in Medium Dense Sand
inclinations were normalized by the corresponding axial uplift interpreted failure loads. With this normalization, good correlations were established between the normalized interpreted failure load and load inclination.

For inclined uplift, density did not influence the normalized interpreted failure load for the loose and medium dense sands. Therefore, the correlations between the interpreted failure load and load inclination for D/B ranging from 3 to 9 were established for the loose and medium dense sand combined. For D/B = 3 and 6, a power fit was adopted while, for D/B = 9, a linear fit seemed more appropriate. In dense sand, D/B did not affect the normalized failure load, and a correlation based on a power fit was chosen for D/B = 3 and 6.

For inclined compression, sand density influenced the normalized interpreted failure load. Therefore, correlations between the interpreted failure load and load inclination for D/B = 3 to 9 were obtained for both loose and medium dense sand. In dense sand, D/B did not affect the normalized interpreted failure load, and the corresponding correlation included both D/B = 3 and 6. An exponential power correlation provided the best match with the experimental results.

A number of interrelationships among the axial, inclined, and lateral capacities, described in Section 2, were evaluated using the results from this study. These relationships were derived from laboratory tests on steel piles and were found to be only partially applicable for drilled shafts. However, the pile inclination factors were not verified for drilled shafts, and therefore they can not be recommended.

Design of drilled shafts subjected to inclined loading should be based on Equations 8-1 through 8-4 and 8-7 through 8-13, provided that additional verification is obtained through full-scale field testing. These equations require only one input parameter (axial uplift failure load), and they result in good correlations with the experimental results. For inclined compression, a second estimate can be obtained from Equation 2-4, for which both the axial compression and lateral failure loads are needed.

REFERENCES


Drilled shaft foundations are used commonly to support transmission line structures. In the case of two- and four-legged structures, the lateral loads from wind and line tension result in combined axial and lateral loading of the foundations that can range from inclined uplift to inclined compression. The response of drilled shafts to these inclined loadings is not well understood. Current knowledge is based largely on the results of laboratory tests conducted on jacked steel model piles. Additional research was necessary to investigate whether the existing methodologies are applicable to drilled shafts. This research was performed to evaluate, through a parametric study, the response of drilled shafts to static inclined loading under drained conditions.

LABORATORY TESTING SUMMARY

Fifty-eight model drilled shafts were tested in the laboratory to evaluate the effect of the following parameters on drilled shaft response:

(a) **Shaft Geometry.** The model shafts in this study had a diameter (b) of 52 mm (2.05 in) and depth to diameter (D/b) ratios of 3, 6, and 9, and they were prefabricated and reusable. They were made of stainless steel tubes, coated with epoxy adhesive, and sprinkled with sand to achieve a rough surface. The selected D/b values are representative of those used in the utility industry.

(b) **Soil Conditions.** The shafts were tested in uniform deposits of loose, medium dense, and dense sand, with relative densities (D_r) equal to 17.1, 40.0, and 77.1 percent, respectively. The loose and medium dense sands were obtained by pluviation and were normally consolidated. Dense deposits were constructed by vibrating in layers and were overconsolidated.

(c) **Load Inclination.** Seven load inclinations were investigated, including 0, 15, 45, 90, 135, 165, and 180 degrees, measured from the upward vertical axis. A summary of the testing program is given in Section 4 and Appendix A.

Loads were applied to the model shafts by a hydraulic actuator coupled to a closed-loop, electro-hydraulic control system. All tests were fully automatic.
and controlled by a mini-computer. Measurements of load, displacement, and rotation were taken automatically with electronic transducers linked to a computer-controlled data acquisition system.

For axial uplift and compression, the test results consisted of load versus displacement. For inclined uplift, lateral, and inclined compression, the test results included component loads versus the corresponding displacement components, horizontal load component versus rotation, normalized apparent depth of rotation (ADOR) versus horizontal load component, and normalized ADOR versus horizontal displacement component.

The load versus displacement response in inclined uplift was smooth and nonlinear, and increasing sand density resulted in higher initial stiffness and higher loads. The shape of the load-displacement curves was influenced by the load inclination. At 15 degrees, the load-displacement curves in loose and medium dense sand had a clearly defined peak, followed by post-peak softening that strongly resembled the corresponding axial uplift tests. At 45 degrees, the curves were flatter, having a shape similar to the corresponding lateral tests. In dense sand, however, the shape of the load-displacement curves was characterized by a clearly defined peak, followed by post-peak softening, regardless of the load inclination.

For inclined uplift, increasing soil density resulted in decreasing normalized ADOR only when the lateral loading mode was dominant, e.g., for a load inclination of 45 degrees. When the applied load deviated slightly from the vertical, higher soil density resulted in a lower point of rotation (higher normalized ADOR). Shaft geometry did not influence the shape of the load-displacement curves or the normalized ADOR. However, load inclination influenced the shape of the load-displacement curves and the normalized ADOR, as discussed in detail in Section 7. The lateral/moment limit method (1, 2) was used to interpret the failure loads from inclined uplift tests. Interpreted failure loads increased nonlinearly with increasing load inclination. However, the nonlinearity of this relationship decreased with increasing depth to diameter (D/B) ratio.

For inclined compression, increasing soil density resulted in higher initial stiffness and higher loads, but lower normalized ADOR. The load-displacement curves were smooth and nonlinear, with no apparent yielding, except in dense sand where a peak followed by post-peak softening could be identified. Shaft geometry did not influence the shape of the load-displacement curves or the
normalized ADOR. However, load inclination influenced the rotational pattern of the shafts. As the applied load deviated less from the vertical, the normalized ADOR was higher. The lateral/moment limit method was used to interpret failure loads from the inclined compression tests. The increase in the interpreted failure load with load inclination was highly nonlinear, and this nonlinearity increased with decreasing D/B.

DESIGN RECOMMENDATIONS

As discussed by McManus and Kulhawy (3), laws of dimensional analysis and similarity are not satisfied easily by model drilled shafts constructed in soil. Therefore, the approach adopted in this study was to: (a) consider the model drilled shafts as small prototypes and assess their response qualitatively, and (b) establish quantitative assessment on the basis of model drilled shaft normalized response.

Previous research (e.g., 4, 5, 6) has shown that the results from model testing of drilled shafts can be applied to the design of full-scale foundations. The approach requires the appropriate soil-stress profile and soil properties, including the soil strength, stress history, and deformation characteristics. These properties can be evaluated from laboratory tests, in-situ tests, and local experience while, for important structures, field tests may be warranted.

As mentioned previously, the model shafts were prefabricated and reusable, and therefore the field construction procedure was not followed. It was believed that, given the small scale of the models, construction simulation probably would have resulted in an excessive disturbance of the shaft-soil interface. However, based on extensive research on construction procedure effects (e.g., 4, 7), the shaft response obtained herein can be considered comparable to that of cast-in-place drilled shafts.

Sections 6 and 7 contain detailed analyses of drilled shaft behavior under the various controlling parameters. The shaft rotation under inclined loading was found to be fundamental to its response, because it governed the change in the apparent depth of rotation (ADOR) and played a major role in interpreting the failure load, as discussed later. The ADOR is the key parameter required for the analysis of mobilized soil stresses along a drilled shaft subjected to inclined loading, and it was established that the normalized ADOR is independent of shaft geometry. Figure 9-1 can be used to estimate the point of shaft...
Figure 9-1. Average Normalized ADOR versus Applied Load Inclination

rotation at failure under inclined loading.

To interpret the failure loads from the inclined uplift and compression tests, the lateral/moment limit method (1, 2) was used. The interpreted inclined failure loads then were normalized by the corresponding interpreted axial uplift failure loads, which are reasonably well understood (6, 2). Once the axial uplift failure load is known, the net inclined uplift failure load ($P_{\psi_{nf}}$) can be computed as below:

$$P_{\psi_{nf}}/P_{0f} = 1 + a (\psi/90^\circ)^b$$

(9-1)

in which $P_{0f}$ = net axial uplift failure load, $\psi$ = applied load inclination measured from the upward vertical axis, and $a$ and $b$ = regression coefficients given in Table 9-1. This equation is valid from $\psi = 0$ to 90 degrees. The total inclined uplift failure load ($P_{\psi_{f}}$) then is given by:

$$P_{\psi_{f}} = W \cos \psi + \left( P_{\psi_{nf}}^2 - W^2 \sin^2 \psi \right)^{0.5}$$

(9-2)

in which $W$ = foundation weight. This equation is intended for use in inclined uplift. It is not valid for lateral loading with $\psi = 90$ degrees. For inclined compression, the total inclined compression failure load ($P_{\psi_{c}}$) is given by:
Table 9-1
RECOMMENDED INCLINED LOADING COEFFICIENTS

<table>
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<tr>
<th>$\text{D/B}$</th>
<th>$\text{Sand Density}$</th>
<th>$\text{Coefficients in Equations 9-1 and 9-3}$</th>
<th>$a$</th>
<th>$b$</th>
<th>$c$</th>
<th>$f$</th>
<th>$g$</th>
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</thead>
<tbody>
<tr>
<td>3</td>
<td>Loose</td>
<td>5.14</td>
<td>0.550</td>
<td>1.67</td>
<td>2.69</td>
<td>8.70</td>
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</tr>
<tr>
<td>6</td>
<td>Loose</td>
<td>2.36</td>
<td>0.799</td>
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Note: Loose and medium dense deposits were normally consolidated; dense deposits were overconsolidated

Unit weights: loose $\gamma = 15.4 \text{ kN/m}^3$ (98.4 pcf), $D_r = 17.1\%$
medium $\gamma = 16.2 \text{ kN/m}^3$ (103.3 pcf), $D_r = 40.0\%$
dense $\gamma = 17.5 \text{ kN/m}^3$ (111.7 pcf), $D_r = 77.1\%$

\[
P_{\psi f}/P_{0f} = \exp\{c + f (\psi/180)\theta\}
\]
(9-3)
in which $e$, $f$, and $g$ — regression coefficients given in Table 9-1. This equation is appropriate from $\psi = 90$ degrees (lateral) to $\psi = 180$ degrees (axial compression).

Figure 9-2 shows a summary of the test results and Equations 9-1 and 9-3 plotted through the data. As can be seen, the agreements are good, and the relationships differ significantly between inclined uplift and compression. Because of the nature of the different relationships, a small discontinuity exists where the relationships overlap at 90 degrees. Interpolations can be done based on this figure and the corresponding equations.

As mentioned in Section 4, all of the tests in this study were conducted using a constant rate of displacement. This loading mode was chosen because one of the primary objectives was to evaluate the effect of the applied inclination on the shaft failure load. With a constant displacement rate, a peak followed by post-peak softening could be identified easily for the majority of the axial and inclined uplift tests.
Figure 9.2. Normalized Failure Load versus Load Inclination for All Test Conditions.
However, in the field, the loading for drilled shafts is force-controlled, and the displacement rate is variable. In such cases, the load-displacement curves for the axial and inclined uplift tests may have higher initial stiffness and overall flatter shape. Therefore, the load versus displacement response should be evaluated from the tests conducted in the force control mode under constant rate of force increase. A continuation of this study will address the response of drilled shafts under repeated inclined loading, based on laboratory tests conducted in the force control mode. A comprehensive solution to inclined load-displacement prediction can be obtained only after detailed evaluation of both the static and cyclic responses.

All tests analyzed in this study have been conducted in uniform sand deposits. No attempts have been made to test drilled shafts in multi-layered soil deposits, because it is difficult to ensure the repeatability of such deposits in the laboratory. However, available research results on axial response (10) and lateral response (11, 12) in layered soils can be applied. It is believed that similar methodology can be utilized for inclined loading.

As with all model testing, verification of the proposed design equations should be obtained through full-scale field testing. The full-scale test conduct, instrumentation, and data interpretation should follow the guidelines presented in this study.

REFERENCES


6. Agaiby, S.W., Kulhawy, F.H., and Trautmann, C.H., "Experimental Study of


Appendix A

LOAD TEST RESULTS

The results from 58 load tests on model drilled shafts are presented in this appendix. These tests are identified by designations that describe the shaft geometry, sand conditions, and load inclination, as given in Table A-1. The shafts had a diameter \( (D) \) of 52 mm (2.05 in) and depth to diameter \( (D/B) \) ratios of 3, 6, and 9. They were tested in uniform deposits of loose (L), medium (M), and dense (D) sand. Seven different load inclination angles were investigated, as measured from the upward vertical axis. Angles of 0, 90, and 180 degrees define axial uplift, lateral, and axial compression tests, respectively. Angles of 15 and 45 degrees denote inclined uplift tests, and angles of 135 and 165 degrees refer to inclined compression tests. The order of testing is given by the test number. Of 67 tests performed in this study, 58 are reported herein, including four replicate tests. The remaining nine tests were not analyzed because they were shakedown tests or tests where instrumentation and/or loading problems were encountered.

As mentioned in Sections 5 through 7, the weight of the model drilled shafts, together with the hook-up and instrumentation attachments used, were recorded and are presented in Table A-1 as added weights. The applied axial uplift load or the vertical component of the applied inclined uplift load had to exceed the weight of shaft and attachments before any vertical movement of the shaft could occur.

For the axial uplift and compression tests, the load test results consist of the applied load versus corresponding displacement. As described in Section 5, the measured loads for the axial uplift tests have been adjusted by subtracting the added weight. Therefore, all of the axial uplift load-displacement curves reported in this appendix are net load-displacement curves that are independent of the weight of the model drilled shaft and attachments. For the axial compression tests, the measured loads are net loads, because the shafts settled under their own weight prior to hook-up.
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\(^a\) angle measured from upward vertical axis
\(^b\) AU = axial uplift; IU = inclined uplift; L = lateral; IC = inclined compression; AC = axial compression
\(^c\) weight of shaft and attachments; shaft weight = 8.5 N for D/B = 3, 16.4 N for D/B = 6, and 22.1 N for D/B = 9
\(^d\) repeatability test
1 N = 0.225 lb

For the lateral and inclined uplift and compression tests, the results include load versus displacement, horizontal load component versus rotation, apparent depth of rotation versus horizontal load component, and apparent depth of rotation versus horizontal displacement component. For each test, these four data sets are reported as part of the same figure.

For the lateral tests, the measured applied load was plotted versus the horizontal displacement component. As described in Section 5, laterally loaded shafts experience a small vertical uplift in addition to the dominant horizontal displacement. However, only the horizontal displacement components are reported in this study for the laterally loaded shafts.

For the inclined compression tests, the applied inclined load was plotted versus the shaft resultant displacement (not in the direction of loading), in addition
to the horizontal and vertical load components plotted versus their corresponding displacement components. As discussed in Section 6, the measured loads already were net loads, since the model drilled shafts settled under their own weight prior to hook-up.

For the inclined uplift tests, the horizontal and vertical load components were plotted versus their corresponding displacements. As described in Section 7, there is no inclined resultant load plotted versus inclined displacement. Moreover, the weight of shaft and attachments (added weight) has been subtracted from the vertical load component to make it a net load.

The apparent depth of rotation (ADOR) was described in Section 1. All of the figures show ADOR normalized by the depth of the corresponding model drilled shaft. The initial parts of the normalized ADOR plots show some noise (test scatter) or erratic spikes because the shaft rotations were very close to zero at the beginning of a test. However, once the readings stabilized, the normalized ADOR was less than 1.0 in all cases, which means that the shafts rotated about a point located above the shaft tip (base or bottom).

The load test results are presented in Figures A-1 through A-58 in the order given in Table A-1.
Figure A-1. Results of Test 610

Figure A-2. Results of Test 910

Figure A-3. Results of Test 360

Figure A-4. Results of Test 640
Figure A-10. Results of Test 6L15
Figure A-12. Results of Test 3ML5

Net Horizontal Load, $P_{h0}$ (N)
Figure A.13. Results of Test 6N15.
Figure A-14. Results of Test 9M15

- Inclined uplift, $\psi = 15^\circ$
- Medium dense sand
- $B = 52$ mm, $D/B = 9$

1 N = 0.225 lb
1 mm = 0.039 in

Displacement, $\delta_{90^\circ}$ and $\delta_{90^\circ}$ (mm)

Net Horizontal Load, $P_{90^\circ}$ (N)

Rotation, $\theta$ (degrees)

Net Horizontal Load, $P_{90^\circ}$ (N)

Horizontal Displacement, $\delta_{90^\circ}$ (mm)
Figure A-15. Results of Test 3D15

Inclined uplift, $\Psi = 15^\circ$

Dense sand
B = 52 mm, D/B = 3

1 N = 0.225 lb
1 mm = 0.039 in

Rotation, $\theta$ (degrees)

Horizontal Displacement, $\delta_{90^\circ}$ (mm)

Net Horizontal Load, $P_{90^\circ}$ (N)

Ador / Shell Depth

Displacement, $\delta_0$ and $\delta_{90^\circ}$ (mm)

Net Load, $P_0$ and $P_{90^\circ}$ (N)

Ador / Shell Depth
Figure A-18. Results of Test 3L45
Figure A-22. Results of Test 6M45
Figure A-23. Results of Test 9M45
Figure A-24. Results of Test 3D45.
Figure A-25. Results of Test 6D45
Figure A-26. Results of Test 3L90
Figure A-27. Results of Test 6L90
Figure A-29. Results of Test 3290
Figure A-30. Results of Test 3M90-2
Figure A-34. Results of Test 9M90
Figure A-35. Results of Test 3D90
Figure A-35. Results of Test 6D90
Figure A-38. Results of Test 3L135-2

Inclined compression, $\psi = 135^\circ$
Loose sand
$B = 52$ mm, $D/B = 3$

Net Load, $P_{90^\circ}$, $P_{135^\circ}$, and $P_{180^\circ}$ (N)

Displacement, $\delta_{90^\circ}$, $\delta_{135^\circ}$, and $\delta_{180^\circ}$ (mm)

Net Horizontal Load, $P_{90^\circ}$ (N)

Rotation, $\theta$ (degrees)

$1$ N = 0.225 lb
$1$ mm = 0.039 in

Inclined compression, $\psi = 135^\circ$
Loose sand
$B = 52$ mm, $D/B = 3$

Net Horizontal Load, $P_{90^\circ}$ (N)

Horizontal Displacement, $\delta_{90^\circ}$ (mm)

$1$ N = 0.225 lb
$1$ mm = 0.039 in

ADOH/Shaft Depth

Net Horizontal Load, $P_{90^\circ}$ (N)

ADOH/Shaft Depth

$1$ N = 0.225 lb
$1$ mm = 0.039 in
Figure A-40. Results of Test 9L135
Figure A-41. Results of Test 3N135
Figure A-42. Results of Test 6M135
Figure A-44. Results of Test 3D135
Figure A-47. Results of Test 6L165
Figure A-48 Results of Test 9165

Inclined compression, \( \psi = 185^\circ \)
Loose sand
B = 52 mm, D/B = 9

Net Horizontal Load, \( P_{g0}^\circ \) (N)

Rotation, \( \theta \) (degrees)

Horizontal Displacement, \( \delta_{g0}^\circ \) (mm)

Displacement, \( \delta_{g0}^\circ \), \( \delta_{g30}^\circ \), and \( \delta_{g60}^\circ \) (mm)

Net Load, \( P_{g0}^\circ \) and \( P_{g30}^\circ \)
0 10 20 30 40 50
1 0.225 lb
1 mm = 0.039 in

0 500 1000 1500 2000
1 N = 0.225 lb
1 mm = 0.039 in

0 1 0.5 1.5
0 10 20 30 40 50
1 mm = 0.039 in

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Figure A-49. Results of Test 3M165
Figure A-50. Results of Test 6M165
Figure A-51. Results of Test 9M165
Figure A-58. Results of Test 94180

Figure A-57. Results of Test 65180

Axial compression
Medium dense sand
B = 52 mm, D/B = 9

Vertical Displacement, δ_{90\text{o}} (mm)

Net Vertical Load, P_{180}(N)

1 N = 0.225 lb
1 mm = 0.039 in
About EPRI

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