BEHAVIOUR OF LARGE SCALE RIGID MODEL
PILES UNDER INCLINED LOADS IN SAND

CENTRE FOR NEWFOUNDLAND STUDIES

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JAE SHIK JOO
BEHAVIOUR OF LARGE SCALE RIGID MODEL PILES
UNDER INCLINED LOADS IN SAND

BY

© JAE SHIK JOO, B.Eng.

A thesis submitted to the School of Graduate Studies in partial fulfillment of the requirements for the degree of
Master of Engineering

Faculty of Engineering & Applied Science
Memorial University of Newfoundland

April 1985

St. John's Newfoundland Canada
To my wife and mother
ABSTRACT

Attention has been directed towards the stability of offshore structures since the discovery of oil under the sea bed in the 1920's. Especially important in this field of engineering are the large lateral loads from wind, waves, and currents in conjunction with vertical loads. This combination of loads creates the need to analyze systems exposed to large inclined loads.

The scope of this research is to understand the behaviour of a vertical rigid short pile under inclined loads in dense sand. The pile behaviour under inclined loads has been examined in the laboratory using relatively large circular model piles of 75 mm, 90 mm, and 102 mm diameters and a square pile of 73 mm width. These model piles were instrumented with pressure transducers and load cells in order to measure soil pressures. The piles were tested with vertical, inclined, and horizontal loads using a computerized data acquisition system. For these model pile tests a suitable laboratory test frame and a circular steel soil container were designed and assembled.

As part of the comprehensive test program, the piles were first subjected to vertical loads. The bearing capacity factor $N_q$ was found to be constant with depth and consistently smaller than that predicted by various existing theories. For a smooth circular pile the pull out resistance can be estimated as the sum of one half the downward skin friction plus the weight of the pile.

For computing the ultimate lateral load on circular piles, modification of existing theories is necessary to take into account the parabolic soil pressure
variation across the projected pile diameter, rather than the rectangular distribution which is conventionally assumed.

The ultimate load capacity under inclined loads does not decrease uniformly with load inclination. For angles up to about $35^\circ$, the ultimate load capacity is larger than the vertical load capacity.
ACKNOWLEDGEMENTS

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The author appreciates very much the advice, guidance, and careful review of the manuscript by his supervisor Dr. T. R. Chari, Professor and Associate Dean of Engineering. Thanks are due to Dr. G. R. Peters, Dean of Engineering for facilities provided and to Dr. F. A. Aldrich, Dean of Graduate Studies for his help and encouragement.

The author expresses his special appreciation to Mr. C. Roy Dawe and Mr. K. S. R. Prasad for advice and encouragement and to Mr. M. Ruprai for his help in the experiments. Thanks are due to Mr. P. Robinson, Mr. J. Andrews and their colleagues at the University Technical Services for assembling the experimental facilities. The help of Mr. A. Bursey and Mr. C. Ward is much appreciated.
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The symbols used in this thesis conform generally to the recommendation of the Canadian Geotechnical Society (Barsvary et al. 1980). They are also defined where they first appear in the text of the thesis.

<table>
<thead>
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<th>Symbol</th>
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<tr>
<td>( A_p )</td>
<td>Area of pile base ((L^2))</td>
</tr>
<tr>
<td>( A_s )</td>
<td>Area of pile shaft embedded in the soil ((L^2))</td>
</tr>
<tr>
<td>( B )</td>
<td>Diameter or width of pile ((L))</td>
</tr>
<tr>
<td>( C_U )</td>
<td>Uniformity coefficient ((\text{dimensionless}))</td>
</tr>
<tr>
<td>( D )</td>
<td>Depth of pile beneath ground ((L))</td>
</tr>
<tr>
<td>( D_n )</td>
<td>(n) percent grain size ((L))</td>
</tr>
<tr>
<td>( D_R )</td>
<td>Relative density ((\text{dimensionless})) (formerly called specific gravity)</td>
</tr>
<tr>
<td>( E )</td>
<td>Modulus of linear deformation ((FL^{-2})) (modulus of elasticity)</td>
</tr>
<tr>
<td>( e )</td>
<td>Eccentricity ((L))</td>
</tr>
<tr>
<td>( F )</td>
<td>Factor of safety ((\text{dimensionless}))</td>
</tr>
<tr>
<td>( H )</td>
<td>Lateral force applied to a pile ((F))</td>
</tr>
<tr>
<td>( I )</td>
<td>Moment of inertia ((L^4))</td>
</tr>
<tr>
<td>( I_D )</td>
<td>Density index (%)</td>
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<td>( K_a )</td>
<td>Coefficient of active earth pressure ((\text{dimensionless}))</td>
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<td>( K_o )</td>
<td>Coefficient of earth pressure at rest ((\text{dimensionless}))</td>
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<td>( K_{s} )</td>
<td>Average coefficient of earth pressure on the</td>
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pile shaft (dimensionless)

$L$ Length of pile (L)

$N$ S. P. T. blow count (Blows/0.3 m)

$N_c$, $N_q$, $N_r$ Bearing capacity factors (dimensionless)

$n_h$ Horizontal coefficient of subgrade reaction (FL$^{-3}$)

$Q$ Applied axial load (F)

$Q_{a}$ Ultimate axial load (F)

$Q_{n}$ Ultimate lateral load (F)

$Q_{u}$ Ultimate inclined load (F)

$Q_p$ Point resistance force (F)

$Q_s$ Total shaft resistance (F)

$Q_s'$ Total pull out resistance (F)

$q_c$ Static cone point resistance (FL$^{-2}$)

$q_p$ Point resistance pressure (FL$^{-2}$)

$q_{pn}$ Net point resistance pressure (FL$^{-2}$)

$q_s$ Unit shaft resistance (FL$^{-2}$)

$T$ Stiffness factor (L)

$w$ Water content (%)

$\alpha$ Inclination of load (°(deg))

$\gamma$ Unit weight (FL$^{-3}$)

$\gamma_d$ Dry unit weight (FL$^{-3}$)
\[ \delta \] Angle of wall friction (°(deg))

\[ \rho \] Density (ML\(^{-3}\))

\[ \rho_d^{(\text{max})} \] Maximum dry density (ML\(^{-3}\))

\[ \rho_d^{(\text{min})} \] Minimum dry density (ML\(^{-3}\))

\[ \sigma \] Total normal stress (FL\(^{-2}\))

\[ \sigma' \] Effective normal stress (FL\(^{-2}\))

\[ \tau \] Shear strength (FL\(^{-2}\))

\[ \phi \] Apparent angle of internal friction (°(deg))
Chapter 1
INTRODUCTION

1.1. General

Piles have been commonly used to transfer structural loads through weak soil strata to a more suitable soil stratum at greater depths. The loads on structures could be vertical, lateral or a combination of these. Generally for most buildings, the vertical load is predominant. However, lateral load is an important factor in the design of tall structures, piers, bridge abutments, poles with large sign boards, retaining walls on land and for deep water ports, light stations, offshore structures, nuclear power plants, and harbour facilities. Lateral forces may be caused by the wind, waves, currents, ice movement, berthing ships, earthquake loading, etc.

Depending on the inclination of the pile axis to the plumb line, piles can be classified as vertical or batter piles. Often, vertical piles are used to resist vertical loads, and batter piles are separately designed for the lateral loads. However, with the advent of offshore structures subjected to large lateral loads there is an increased need for the design of vertical piles subjected to inclined loads.

One of the early field tests of vertical piles subjected to inclined loads was
conducted by Evans (1953). The piles were subjected to a constant vertical load with increasing horizontal loads at the site of Sepulveda dam, California. Since then, investigation of piles subjected to inclined loads has been mostly laboratory research in which small diameter model piles have been examined (Awad and Petrasovits 1968, Meyerhof and Ranjan 1972, Meyerhof et al. 1981, 1983). Meyerhof et al. (1981) have proposed an interaction equation for estimating the ultimate load under inclined loads based on test results of a 12.5 mm diameter model pile. This equation was verified subsequently using a 75 mm diameter instrumented pile (Chari and Meyerhof 1983).

1.2. Scope of the investigation

This investigation is a continuation of the earlier efforts to understand the behaviour of short piles subjected to inclined loads, using model piles of larger diameter.

The laboratory facilities were designed and assembled for conducting the model pile tests. A galvanized corrugated steel tank 1.8 m diameter and 2 m high was suitably modified to enable the preparation of samples of sand with different densities. Instrumented model piles of 73 mm, 90 mm, 102 mm diameters, and 73 mm square were used and the piles were loaded to the ultimate bearing capacity of the soil with vertical, inclined, and horizontal loads. The inclinations of loads were at 30, 45, and 60 degrees.

The objectives of this investigation are:

(1) to compare the predicted ultimate bearing capacity of piles with the measured value,
(2) to study the variation of the ultimate bearing capacity of piles with
the inclination of loads, and

(3) to analyze the results in the light of available theoretical and empirical
methods of prediction.

A brief review of literature is presented in Chapter 2. The details of the
experimental set up and test procedures are given in chapter 3. Chapter 4 deals
with experimental results and discussion. Chapter 5 gives the summary,
conclusions, and recommendations for further research relevant to this study.
Chapter 2

REVIEW OF LITERATURE

2.1. General

Piles are classified in a number of ways depending on their function, composition, and method of installation. A definition diagram showing the commonly used nomenclature for pile foundations is given in Figure 1.

A vertical pile has its axis coinciding with the plumb line while a batter pile has its axis inclined to the plumb line. Vertical piles are usually used to resist dead and live loads, uplift due to swelling and frost expansion of soil, and forces due to hydrostatic pressure beneath the base of a structure. Batter piles are commonly used to resist inclined or large lateral loads.

A short pile is relatively rigid and rotates as one unit under lateral loads while a long pile is relatively flexible and acts like a beam under lateral loads. The criteria for the classification of short and long piles is given in Figure 1. The design length of a pile mainly depends on the profile of the subsoil and the type and magnitude of loading.

There is an extensive amount of available literature on axially and laterally loaded piles. Generally, the vertical capacity of a pile is dictated by the ultimate
(a) A vertical pile under vertical loads

(b) A batter pile under inclined or lateral loads

(c) A short rigid pile

(d) A long flexible pile

\[ D \leq 2T \]  
\[ D \geq 4T \]

Figure 1: Types of piles
bearing capacity of the soil. The ultimate bearing capacity in turn may be defined as the maximum load which the pile can support without undergoing significant settlements. The ultimate bearing capacity of vertical piles under axial loads in sands is generally evaluated using soil properties such as its density and the angle of shear resistance (Terzaghi 1943, Meyerhof 1951, Vesic 1963). The ultimate lateral load of vertical short rigid piles is generally computed based on lateral earth pressure theories (Brinch Hansen 1961, Broms 1964, Petrasovits et al. 1972, Meyerhof et al. 1981), and the ultimate lateral load of a long flexible pile can be evaluated using the theory of elasticity (Rowe 1955, Matlock and Reese 1962, Broms 1964, Poulos 1971).

Literature on the ultimate capacity of vertical rigid piles subjected to inclined loads is somewhat limited. One of the present methods to compute the ultimate capacity under inclined loads is to use an interaction equation (Meyerhof 1981). A brief review of the existing theories of vertical and lateral ultimate capacity of short rigid piles is presented here.

2.2. Vertical piles under axial loads

The ultimate bearing capacity \( Q \), of a vertical pile under an axial load (Figure 1a) is generally expressed as the sum of point resistance force \( Q_p \), and total shaft resistance \( Q_s \), as follows:

\[
Q = Q_p + Q_s
\]

\[
= q_p A_p + q_s A_s
\]  

(1)
where $q_p$ is the point resistance pressure,

$q_s$ denotes the average unit shaft resistance,

$A_p$ is the area of pile base, and

$A_s$ is the area of embedded pile shaft.

The point resistance pressure $q_p$, and the average unit shaft resistance $q_s$, are functions of several parameters but mainly depend on the type of soil, the density of soil, the angle of friction, and the physical properties of the pile. For practical purposes, Equation 1 is formulated on the premise that the two components $q_q$ and $q_s$ are independent of each other. In fact, for piles driven into cohesionless soils there is some interdependence between the two components (Kezdi 1957), but this small influence is generally neglected (Broms 1966). The magnitudes of the two components $Q_p$ and $Q_s$ in cohesionless soil may be intuitively expected to be proportional to the embedded depth, but according to laboratory and field test results, the proportionality cannot be satisfied beyond the critical depth below which the ultimate load remains relatively constant (Kerisel 1964, Vesic 1963, Vesic 1964, Tavenas 1970). The relative magnitudes of $Q_p$ and $Q_s$ depend on the type of soil and the method of installation of the pile.

Based on the method of placement, vertical piles may be classified into two broad categories. A pile driven into the soil is classified as a displacement pile. A pile which is placed by removing an equal volume of the soil is generally called non-displacement pile (sands) or a bored pile (clays). The capacity of the pile is predominantly the end bearing resistance for a non-displacement (bored) pile,
while it is the sum of the end bearing and side frictional resistance for a displacement (driven) pile.

The ultimate bearing capacity of a pile $Q$, can be estimated by several methods and the most commonly used are:

(1) based upon bearing capacity theories,

(2) from the results of in-situ tests, and

(3) prototype pile load tests.

The first two methods which are relevant to this thesis will be reviewed in the following sections.

2.2.1. Estimation of $Q$ based on bearing capacity theories

Point resistance force, $Q_p$

Most of the present solutions for the point resistance force of pile foundations are derived using Prandtl's (1920) and Reissner's (1924) general bearing capacity theories based on the assumption of weightless material, and Ohde's (1938) theory considering the weight of the material. The resulting point resistance pressure is expressed by the following general equation.

$$ q_p = (c N_c + \gamma D N_q + \frac{B}{2} \gamma N_\gamma) $$

(2)

where $q_p$ is the point resistance pressure of the cross-section area of pile,
$N_c$, $N_q$, $N_\gamma$ are the bearing capacity factors,

$B$ is the diameter of pile,

$\gamma$ is the effective unit weight of soil at the level of pile tip,

$c$ is the cohesive strength of soil, and

$D$ is the vertical distance between the ground surface and the level of pile tip.

For deep foundations in cohesionless soils, the first term will be zero and the third term is negligibly small in comparison to the second term. Hence, Equation 2 can be simplified as:

$$q_p = \gamma D N_q$$

(3)

When it is necessary to consider the weight of the pile, the net point resistance pressure $q_{pn}$, of a pile can be determined based on the assumption that the unit weight of pile material is equal to that of soil.

$$q_{pn} = \gamma D (N_q - 1)$$

(4)

Equations 3 and 4 indicate that the bearing capacity of a pile varies with the bearing capacity factor $N_q$, which depends on the deformation characteristics of the soil. Vesic (1967, 1977) has summarized the various theoretical approaches to simulate the failure mechanism of soil as shown in Figure 2. The corresponding $N_q$ values in sand as suggested by various investigators are reproduced in Figure 3 and Table 1.
Figure 2: Assumed failure patterns of soil

Berezantsev and Yaroshenko (1962)  
Bishop, Hill & Mott (1945)  
Vesic (1977)  

Prandtl (1921)  
Reissner (1924)  
Caquot (1934)  
Buisman (1935)  
Terzaghi (1943)  

DeBeer (1945)  
Jaky (1948)  
Meyerhof (1951)  

Bishop, Hill & Mott (1945)  
Skemton, Yassin & Gibson (1953)  

Vesic (1963)
Figure 3: Bearing capacity factors for circular deep foundations after Vesic (1977)
# TABLE 1

Experimental values $N_q$ in sand

<table>
<thead>
<tr>
<th>SAND COMPACTNESS</th>
<th>DENSITY INDEX (%)</th>
<th>$N_q$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>DRIVEN PILES</td>
</tr>
<tr>
<td>Very dense</td>
<td>$&gt;80$</td>
<td>60-200</td>
</tr>
<tr>
<td>Dense</td>
<td>60-80</td>
<td>40-80</td>
</tr>
<tr>
<td>Medium</td>
<td>40-60</td>
<td>25-60</td>
</tr>
<tr>
<td>Loose</td>
<td>$&lt;40$</td>
<td>20-30</td>
</tr>
</tbody>
</table>

After Vesic (1977), higher values apply to shorter piles.
It is reported (Norlund 1963, Broms 1966, Vesic 1964, 1967) that in practice, the $N_q$ values of Berezantzev are found to correlate well with the measured values. However, Coyle and Castello (1979) suggested that Terzaghi's $N_q$ values for general shear failure were found to fit their experimental results.

The point resistance pressure $q_p$, has been normally found to increase up to a certain depth beyond which any increase in $D$ does not result in significant increase of $q_p$. This depth has been normally designated as the critical depth. Kerisel (1964), and Meyerhof (1976) reported that the value of $N_q$ in sand increases with depth and reaches its maximum value at less than half of the critical depth. While Berzantsev et al. (1961), and Drugunoglu & Mitchell (1973) found that $N_q$ decreases with increasing $D/B$ ratio, Vesic (1977) concluded that $N_q$ is a constant, independent of the depth.

In addition to the depth, $N_q$ depends on many factors such as density of the soil, overburden pressure, shape of the pile and method of installation. For driven piles the change of density of soil due to driving a pile has to be taken into account to evaluate the ultimate load capacity of the pile. However, as mentioned earlier, the available theories are based on the assumption that the soil density during pile driving is not changed. In fact, the density index increases for driven piles in sand except in very dense sand, and therefore the angle of internal friction $\phi_2$, after driving the piles is larger than the initial internal angle of friction $\phi_1$. The relationship between $\phi_1$ and $\phi_2$ in sand has been suggested as follows (Kishida and Meyerhof 1965):
\[
\phi_2 = \frac{(\phi_1 + 40)}{2}
\]

Equation 5 implies that there is no change in density index for soils with an internal friction angle of 40°.

Based on the failure mechanism shown in Figure 2e, Vesic (1977) has given the following equation for \( q_p \):

\[
q_p = \sigma_0 N_\sigma
\]

\[
= \frac{(1 + 2K_o) \gamma D}{3} N_\sigma
\]

where \( K_o \) is the coefficient of lateral earth pressure at rest.
\( \sigma_0 \) is the mean normal stress at pile tip,
\( \gamma \) is the unit weight of soil,
\( D \) is the embedded pile length, and
\( N_\sigma \) is the bearing capacity factor for mean normal stress term and is a function of compressibility as well as internal friction angle of soil.

The point resistance force \( Q_p \) can thus be computed as the product of \( q_p \) and the area of the pile base \( A_p \).

*Total shaft resistance, \( Q_s \)*

For deep foundations the total shaft resistance \( Q_s \) can be defined as the
resistance to the sliding of a rigid body relative to the surrounding soil and is
generally expressed by two components: (1) adhesion, and (2) friction, dependent
on normal stresses.

The unit shaft resistance of driven piles $q_s$, at any depth $Z$ below the ground
surface can be calculated from Mohr-Coulomb's theory of rupture as follows:

$$q_s = c_a + K_s \sigma' \tan \delta$$ (7)

where $c_a$ is the undrained pile-soil cohesion,

$K_s$ denotes the coefficient of earth pressure on the pile shaft,

$\delta$ is the angle of shaft friction between soil and pile material,

$\sigma'$ is the average effective overburden pressure at any point and
defined as the product of $\gamma'$ and $Z$,

$\gamma'$ is the effective unit weight of soil,

For piles in cohesionless soils the value of $c_a$ is zero. Equation 7 can be
rewritten integrating along the embedded pile length for the total shaft resistance
$Q_s$, as follows (Dorr 1922, Meyerhof 1951, Norlund 1963).

$$Q_s = \frac{1}{2} K_s \gamma' D \tan \delta A_s$$ (8)

where $A_s$ is the total area of embedded pile shaft,

$D$ is the embedded depth of pile, and

$K_s$ is the average coefficient of earth pressure on the pile shaft.
The magnitude of coefficient $K_s$ in Equation 8 depends mainly on the initial relative density, the displacement volume of the pile, the shape of pile, and the method of pile installation. However, for practical purposes the averaged values of $K_s$ can be taken for piles driven into cohesionless soil. The coefficient $K_s$ for driven steel piles has been suggested as 1.0 for dense sand and 0.5 for loose sand regardless of pile type and roughness of the pile surface (Meyerhof 1951, Broms 1966, Coyle and Castello 1979).

The angle of friction $\delta$, between the soil and the shaft has been suggested based on experimental data as 0.54 $\phi$ for smooth steel piles and 0.76 $\phi$ for rusted steel piles where $\phi$ represents the angle of internal friction for the soil (Pontyondy, 1961). Other researchers have given the skin friction angle $\delta$ as 20° for steel piles assuming that the value of $\delta$ is independent of the density index of the soil surrounding the pile (Broms 1966, Craig 1978, Tomlinson 1981).

### 2.2.2. Estimation of $Q$ based on in-situ tests

The vertical capacity of piles can be also estimated based on in-situ tests such as the standard penetration test (SPT) and the cone penetrometer test (CPT). Both types of tests are routinely done as part of site investigations.

The standard penetration test can be used to determine the ultimate bearing capacity of piles in cohesionless soils. This ultimate bearing capacity $Q$, in sands has been expressed as (Meyerhof 1956, 1976):

$$ Q = \{ 4 NA_p + \frac{NA_s}{50} \} \ (100 \ kN/m^2) $$

(9)
where \( N \) denotes the average penetration resistance near the pile tip (blows/0.3 m),

\( A_p \) represents the area of the pile base, and

\( A_s \) denotes the area of embedded pile shaft.

It should be noted that the accuracy of the above estimate depends on the reliability of the blow count \( N \). As is common knowledge, the standard penetration test is not generally used for cohesive soils and Equation 9 is valid only for cohesionless materials.

If the ratio of depth to diameter of the pile is less than 10, the point resistance pressure \( q_p \), can be expressed as (Meyerhof 1956):

\[
q_p = \frac{4ND}{10B} \text{ (100 kN/m}^2) \quad \text{(10)}
\]

The cone penetrometer test in cohesive and cohesionless soils has been correlated with the ultimate bearing capacity of piles. The ultimate bearing capacity \( Q \), of piles in cohesionless soil has been given as (Meyerhof 1956):

\[
Q = q_c A_p + 0.005 q_c A_s \quad \text{(11)}
\]

where \( q_c \) denotes the average static cone point resistance, and

\( A_p \) is the area of pile, and

\( A_s \) is the area of embedded pile shaft.

Equation 11 has been derived on the assumption that the point resistance pressure of the pile is equal to the average static cone point resistance \( q_c \), over a
depth of 4 pile diameters above and one pile diameter below the anticipated depth of the pile tip (Meyerhof 1956, Menzenbach 1961). It is also assumed that the unit shaft resistance is equal to 0.5 % of the average static cone point resistance (Meyerhof 1956, 1976).

If the depth of foundation is less than 10 times the pile diameter, the point resistance pressure \( q_p \), can be expressed as:

\[
q_p = \frac{q_c D}{10B}
\]  
(12)

where \( D \) is the embedded pile length, and

\( B \) represents the width or diameter of the pile.

Subsequent work shows good correlations for pile diameters less than 50 cm (Kerisel 1961). However, the total shaft resistance on concrete piles was found to be greater than that given by Equation 13 (Mohan et al. 1963). Tomlinson (1977) has given a slightly different approach in which the values of \( K_s \) and \( \phi \) can be estimated from cone penetrometer tests. The suggested values are given in Table 2, and from these, the ultimate bearing capacity \( Q \), can be estimated as:

\[
Q = q_c A_p + 0.5 K_s \gamma D \tan \delta A_s
\]  
(13)
### TABLE 2

*$\phi$ and $K$, corresponding to the various values, $q_c$.

<table>
<thead>
<tr>
<th>$q_c$ (g/cm$^2$)</th>
<th>$\phi$ (deg.)</th>
<th>$K$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-50</td>
<td>28-30</td>
<td>Low relative density</td>
</tr>
<tr>
<td>50-100</td>
<td>30-36</td>
<td>Medium relative density</td>
</tr>
<tr>
<td>$&gt; 100$</td>
<td>$&gt;36$</td>
<td>High relative density</td>
</tr>
</tbody>
</table>

*After Tomlinson (1977)*
2.3. Vertical Rigid Piles under Lateral Loads

Piles are generally classified as short and long based on two criteria. A short free-headed pile having a depth/diameter (D/B) ratio of 10-12 will fail by rotation developing passive resistance on opposite faces above and below the point of rotation. The pile rigidity is also related to a stiffness factor $T$ which is expressed as (Davison and Prakash 1963, Broms 1964, Tomlinson 1977):

$$ T = \left( \frac{EI}{n_h} \right)^{1/5} \quad (14) $$

where $EI$ is the stiffness of the pile, and $n_h$ is the coefficient of horizontal subgrade reaction.

The length of the pile is to be less than about $2T$ for behaviour as a short rigid pile and greater than $4T$ for behaviour as a long elastic pile. Theoretical analyses of the behaviour of short rigid piles have been presented in published literature (Brinch Hansen 1961, Christensen 1961, Broms 1964, Petrasovits and Awad 1972, Adams and Radhakrishna 1973, Meyerhof et al. 1976, 1981). At failure, it is assumed that a short rigid pile rotates as a unit body, and that the ultimate lateral resistance of the soil will be reached before a structural failure of the pile.

The exact pressure distribution on a rigid pile subjected to lateral loads is nonlinear. However, presently available analytical methods are based on a simplified assumption of the true pressure distribution as shown in Figure 4.

Brinch Hansen (1961) has suggested an equation for the lateral earth
Figure 4. Assumed soil pressure distribution under lateral loads
Figure 5. Calculation of $Q_n$. 

(a) Brinch Hansen (1961)  
(b) Broms (1964)  
(c) Petrasovits et al. (1972)
pressure \( p_x \) at depth \( x \) based on the assumption of nonlinear soil pressure distribution as shown in Figure 4a. The lateral earth pressure \( p_x' \) at depth \( x \) is expressed as:

\[
p_x = c K_c + \gamma x K_q
\]

where \( c \) is the cohesive strength of soil,
\( \gamma \) is the unit weight of soil,
\( x \) is the arbitrary depth below the soil surface, and

\( K_c, K_q \) are the earth pressure coefficients dependent on \( \phi \) and the ratio of embedded pile depth to pile diameter (\( D/B \)).

For driven piles in cohesionless soil, Equation 15 can be simplified by taking the value of \( c \) as zero:

\[
p_x = \gamma x K_q
\]

Assuming the earth pressure distribution as shown in Figure 5a, the depth of rotation point \( D_o' \), can be found by trial and error by taking moments about the line of application of the load as follows:

\[
\sum M(=0) = \sum_{x=0}^{D_o} P_x \frac{D}{n} (e+x)B - \sum_{x=D_o}^{D} P_x \frac{D}{n} (e+x)B
\]

Knowing \( D_o' \), the ultimate lateral load \( Q_n' \), can be calculated taking moments about \( D_o' \):

\[
Q_n (e + D_o) = \sum_{x=0}^{D_o} P_x \frac{D}{n} B(D_o-x) + \sum_{x=D_o}^{D} P_o \frac{D}{n} B(x - D_o)
\]
where \( n \) is the convenient number of horizontal elements of embedded pile length,

\( e \) is the eccentricity of the applied load above the soil surface, and

\( B \) is the diameter or width of pile.

Another approach has been given by Broms (1964) who suggested a simple equation based on the following assumptions for calculation of ultimate lateral resistance of rigid vertical free headed piles in cohesionless soil:

1. Maximum lateral earth pressure is equal to three times the Rankine passive earth pressure at failure assuming that the pile surface is frictionless.

2. The active pressure along the pile is negligible.

3. The shape of the pile cross-section has little effect on the earth pressure distribution.

4. A large lateral reaction develops at the pile tip in the same direction as the applied load.

Assuming a lateral earth pressure distribution as shown in Figure 5b, the ultimate lateral resistance \( Q_n \), can be expressed by employing equilibrium conditions.

By taking moments about the tip of pile,
\[ Q_n = \frac{(0.5 \gamma D^2 K_p B)}{(e + D)} \] (19)

where \( e \) is the eccentricity of the applied load above the soil surface,

\( D \) is the embedded pile length,

\( \gamma \) is the unit weight of soil,

\( K_p \) is the coefficient of the Rankine passive earth pressure defined as \( K_p = \frac{(1+\sin(\phi))/(1-\sin(\phi))}{\gamma} \)

\( \phi \) is the internal friction angle of soil, and

\( B \) is the diameter or width of pile.

In the above theory, the assumed triangular earth pressure distribution is quite different from the actual pressure distribution and gives relatively higher values than published experimental results (Poulos 1978).

Petrasovits and Awad (1972) extended Broms' method for rigid piles in cohesionless soil by assuming that the rotation point occurs along the pile rather than at the pile tip. They assumed that at the back side of the pile the earth pressure is equal to the Rankine active earth pressure (Rowe 1956) and the full passive earth pressure is equal to 3.7 times the Rankine passive earth pressure. Based on the assumed earth pressure distribution as shown in Figure 5c, the ultimate lateral resistance \( Q_n \), can be calculated using the horizontal force equilibrium, and moment equilibrium:

\[ Q_n = \frac{1}{2} (3.7 K_p - K_a) \gamma B D^2 (2 R^2 - 1) \] (20)
The following equation can be derived from Equations (20) and (21):

\[
Q_n = \frac{1}{3} (3.7K_p - K_a) \gamma BD^3 (1 - 2R^3)/e
\]

(21)

The depth of rotation point \(D_o\) is obtained by trial and error. When \(D_o\) is found, \(Q_n\) can be calculated by Equations 20 and 21. Petrasovits and Awads (1972) indicate that the ratio \(D/e\) has little influence on the depth of the point of rotation. Results by the same authors show that this method is more suitable for short rigid piles rather than for long piles.

Meyerhof et al. (1981) have extended the theory of the ultimate lateral resistance of rigid vertical walls in layered soil to rigid vertical piles considering a shape factor for laterally loaded vertical piles. They suggest an equation for calculation of ultimate lateral resistance \(Q_n\) based on the assumed earth pressure distribution as observed on rigid vertical walls as shown in Figure 4d.
From the equilibrium equation of moment about the point at ground surface and the equilibrium equation of lateral forces, the ultimate lateral resistance $Q_n'$ is approximated by Meyerhof et al. (1981) as follows:

$$Q_n = B \gamma D^2 F_b K_b r_b S_{bu}$$  \hspace{1cm} (23)

where $\gamma$ is the unit weight of soil,

$D$ is the embedded pile length,

$K_b$ is the coefficient of earth pressure defined as:

$$\tan^2(45 + \phi/2) - \tan^2(45 - \phi/2),$$

$\phi$ is the internal friction angle of soil,

$F_b$ is the lateral resistance factor given by Meyerhof et al. (1981),

$r_b$ is the reduction factor due to the moment $Q_n$ e which is defined as $r_b = 1 / (1 + 1.4 \frac{e}{D})$,

$B$ is the diameter or width of pile, and

$S_{bu}$ is the shape factor given by Meyerhof et al. (1981).

It may thus be observed that there are a number of theories for the prediction of lateral resistance of rigid piles and one would expect a variation in the results of these theoretical computations. Four of the available theories are used for the prediction purpose. In this thesis, a relative comparison will be made with actual measurements.
2.4. Inclined loads on piles

Two types of inclined loads on piles are discussed in the literature,

(1) piles subjected to pull out tests (Yoshimi 1965, Broms 1965, Awad and Petrasovits 1968, Meyerhof 1972), and


The latter types of test which are relevant to this thesis are discussed below.

One of the early contributions to research on piles subjected to inclined loads is the work of Evans (1953) in which field tests were done under constant vertical load and increasing horizontal loads for vertical and batter piles. However, published data were not enough to study the behaviour of a pile under inclined loads in terms of the ultimate bearing capacity of a pile.

For the behaviour of vertical rigid piles under inclined loads some tests were performed taking into account vertical eccentricity on three piles of different diameters ranging from 13mm to 35mm (Awad and Petrasovits 1968). According to the experimental results, for a load inclination of 22.5° the ultimate bearing capacity of three piles driven in uniform sand was a maximum and 16 to 35 % higher than the ultimate vertical bearing capacity. In these tests the ratio of vertical eccentricity to embedded pile length was 0.3, the density index \( I_D \) of the soil was 80\%.
Meyerhof and Ranjan (1972) studied the behaviour of piles under inclined loads both theoretically and experimentally in uniform sand. Their investigation showed that the pile rotates only when the inclination of the load is more than 45°. It has been reported that the ultimate bearing capacity of vertical rigid piles under inclined loads decreased with the inclination of loads. The results are based on the experiments with a 12.7 mm diameter pile pushed into uniform dense sand. Meyerhof and Ranjan (1972) have reported an equation for the estimation of the point resistance force $Q_{pv}$, under inclined loads as follows:

$$Q_{pv} = \gamma DN_q 'A_b$$  \hspace{1cm} (24)$$

where $\gamma$ is the unit weight of the soil,

D is the embedded pile length,

$N_q '$ is the bearing capacity factor relevant to load inclination given by Meyerhof et al. (1972), and

$A_b$ is the area of the pile base.

Equation 24 implies that the point resistance force decreases with the load inclination. Meyerhof et al. (1981) have reported that the ultimate bearing capacity under inclined loads decreases with increasing inclination of loads to the vertical and have proposed an interaction equation for the determination of ultimate bearing capacity as follows:

$$
\left( \frac{Q_u \cos(\alpha)}{Q_d} \right)^2 + \left( \frac{Q_u \sin(\alpha)}{Q_n} \right)^2 = 1
$$

\hspace{1cm} (25)$$

where $Q_u$ represents the ultimate bearing capacity of the pile under
inclined loads,

\[ Q_a \] denotes the ultimate axial load of the pile,

\[ Q_n \] is the ultimate lateral load of the pile, and

\[ \alpha \] is the inclination of applied loads to vertical in degrees.

The experimental results with a 12.7mm diameter pile in layered uniform sand confirmed that the ultimate bearing capacity under inclined loads decreases with load inclination (Meyerhof et al. 1981). Earlier research showed that the ultimate capacity of a pile under inclined loads with a buried pile in compact sand did not decrease continuously with increasing load inclination (Meyerhof and Ranjan, 1973).

Chari and Meyerhof (1983) conducted laboratory tests with a relatively larger pile of 75mm diameter, and considered the depth of the point of rotation and the lateral earth pressure distribution under inclined loads in uniform dense sand: They compared the experimental results with the predicted values by the empirical interaction equation using Broms' theory for calculation of ultimate lateral resistance. The results indicated that there was good agreement between predicted and experimental results, and that the ultimate bearing capacity of the pile under inclined loads decreased continuously with increasing inclination of load.

A review of literature shows that there is a divergence of results in the literature. No agreement exists on the variation of ultimate capacity with inclination of load among researchers. This aspect is examined in some detail in this work and the results thereof presented in Chapter 4.
Chapter 3

EXPERIMENTAL FACILITIES
and PROCEDURES

3.1. General

Most of the model tests reported in the literature have been conducted with piles of 12.5 to 35 mm in diameters. Test results with large diameter piles are sparse. Similarly there is not much data on test piles instrumented with pressure and load cells. In this study, circular piles of 73 mm, 90 mm, and 102 mm and a square pile of 73 mm were tested under vertical and inclined loads in sand. These piles were instrumented with pressure transducers and load cells.

The objectives of this experimental research are:

(1) study the variation of bearing capacity factor $N_q'$ with relative depths,

(2) evaluate the pull out resistance of a vertical pile,

(3) study the variation of lateral earth pressure along the pile in order to evaluate the ultimate lateral load, and
study the variation of the ultimate bearing capacity of a pile under vertical, inclined, and lateral loads and compare with theoretical computations.

To accommodate the physical size of the piles and the associated large forces, the soil container and the loading frame as shown in Figure 6 had to be suitably designed. Two screw jacks, one with a capacity of 178 kN and the other with 44.5 kN, were used in this study. The initial placement of the pile in the sand was done by pushing the pile vertically down using the jack of higher capacity. After pushing to the required depth, testing of the piles was done using the smaller jack with a swivel joint as shown in Figure 7. For all the different pile sizes, their corresponding lengths of embedment were chosen to ensure that the piles behaved as rigid piles.

A sketch of the different types of test piles is given in Figure 8. While only one length of embedment was used for the 73 mm and 90 mm diameter circular piles and the square pile, the 102 mm diameter pile was tested with three different lengths of embedment.

A total of 25 different types of tests were conducted using these piles as shown in Figure 8. A description of the test facilities and the experimental procedures is given below.
3.2. Experimental facilities

3.2.1. Soil container

The soil container is made out of a galvanized corrugated steel pipe (1.83 m dia. x 2.8 mm thick x 2 m high) as shown in Figure 6. The length and diameter of the container are governed by the anticipated zone of influence of a pile pushed into soil. This diameter should be large enough to avoid end effects of the container with a reasonable clearance. Figure 9 shows a typical pile pushed into sand and the type of densification that normally occurs around it. Table 3 is a summary of the published data on this phenomenon. The magnitude of dimensions $a$ and $b$ is dependent on the diameter of pile and the density of the sand.

In designing the size of the container, consideration was given to provide an adequate clearance between the walls of the container and the zone of soil densification. This extra clearance prevents confining effects of the walls on the test results and allows cone penetrometer testing of the relatively undisturbed soil after the pile is tested. The soil container used is large enough to test piles of up to 120 mm diameter with a 1400 mm embedded length. The structural strength of the container was also checked to verify the hoop tension due to the soil pressure inside. The soil container has two side openings with a chute and metal sliding doors, one near the bottom and the other 1 m above the base.

These openings facilitate easy removal of the soil after testing. A 2 mm thick steel plate is welded to the bottom of the container and the container rests
Figure 6: General experimental set up
Figure 7: Configuration of loading system
Figure 8: Types of tests
Figure 9: Influence zone of densification during placement of a pile
### TABLE 3

Densification influence zone for driven pile in sand

<table>
<thead>
<tr>
<th>INVESTIGATOR</th>
<th>DENSITY</th>
<th>INFLUENCE ZONE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhof (1959)</td>
<td>loose</td>
<td>6B' - 8B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5B</td>
</tr>
<tr>
<td>Kerisel (1961)</td>
<td>dense</td>
<td>5B</td>
</tr>
<tr>
<td></td>
<td>General</td>
<td>3B</td>
</tr>
<tr>
<td>Robinsky &amp; Morrison (1963)</td>
<td>loose</td>
<td>7B - 9B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.5B - 3.5B</td>
</tr>
<tr>
<td></td>
<td>medium</td>
<td>10B - 12B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3B - 4.5B</td>
</tr>
<tr>
<td>Kishida (1963, 1967)</td>
<td>loose</td>
<td>6B - 8B</td>
</tr>
<tr>
<td></td>
<td>General</td>
<td>5B</td>
</tr>
<tr>
<td>Broms (1965)</td>
<td>General</td>
<td>7B - 12B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3B - 5B</td>
</tr>
<tr>
<td>Lamb &amp; Whitman (1969)</td>
<td>General</td>
<td>16B</td>
</tr>
</tbody>
</table>

*a* represents the width of densification zone.

*b* denotes the depth of densification zone below the tip.

*B* is the diameter of pile.
on a reinforced concrete floor. Adequate facilities are made so that the container can be lifted using an overhead crane and properly positioned relative to the loading frame.

A loading frame was designed and fabricated using two W 250 x 115 H sections for columns and a horizontal member made out of two C 310 x 31 channel sections as shown in Figure 6. The overall size of loading frame is 5.48 m high x 3.95 m wide. This frame is capable of withstanding vertical loads of 653 kN with a safety factor of 2 and horizontal loads of 16 kN applied at 2.1 m from the base of the frame.

3.2.2. Model piles

All the model piles are fabricated from standard, extra heavy black steel pipes. The pipes were split longitudinally and reassembled using suitably designed internal connecting rings to fasten the two halves. Pressure transducers were fitted in drilled holes and connected to electrical cables going through the center of the pile and finally coming out from the side at the top of the pile. The piles are pushed into sand manually using the jack of higher capacity which has a stroke of 1500 mm. Figures 10 and 11 show the model piles and some of the details of their dimensions. The physical properties of the piles are listed in Table 4.
3.2.3. Instrumentation and recording system

Lateral soil pressures on the piles were measured by two rows of diaphragm pressure transducers which were mounted flush with the pile wall. A detailed specification of the pressure transducers is given in Table 5. The total applied load was measured using a commercially available load cell located at the pile top. The point resistance force of each pile was measured using a full bridge strain gauge type load cell fabricated in-house. Figure 12 and 13 give the details of these load cells. Displacements were measured using dial gauges and linear variable differential transformers (LVDT). The output from the pressure transducers, load cells, and LVDTs were recorded on magnetic discs through an HP 86 micro computer and an HP 3497A Data Acquisition/Control Unit. The required computer programs were developed for subsequent plotting and analysis of data. The major computer programs are listed in the Appendix.

3.3. Soil properties

The soil used was commercially available dry coarse sillica sand with a maximum dry density of 1,570 kg/m$^3$, a minimum dry density of 1,340 kg/m$^3$ and uniformity coefficient of 1.4. The sand bed used in the tests had a density of 1,510 kg/m$^3$, a density index of 0.77, and an internal angle of friction of 41.2°. The mechanical properties of the soil are listed in Table 6. The grain size distribution is shown in Figure 14. The shear strength of the soil was determined by direct shear tests and triaxial tests. The results are shown in Figures 15 and 16 and summarized in Table 6.

In order to obtain reproducible laboratory samples of the soil, the test bed
Figure 10: Model piles
Figure 11: Details and dimensions (in mm) of piles
### TABLE 4

Specification of model piles

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>CIRCULAR PILE</th>
<th>SQUARE PILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plie width, B (mm)</td>
<td>73</td>
<td>90</td>
</tr>
<tr>
<td>Length, L (mm)</td>
<td>900</td>
<td>1050</td>
</tr>
<tr>
<td>Thickness, t (mm)</td>
<td>5.3</td>
<td>6.1</td>
</tr>
<tr>
<td>Moment of Inertia, I (m^4)</td>
<td>6.4x10^{-7}</td>
<td>1.42x10^{-6}</td>
</tr>
<tr>
<td>Elastic modulus, E (GPa)</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Hor. coeff. of subgrade reaction for dense sand, n_h (MN/m²)</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Max. embedded length for a short pile, L (mm)</td>
<td>730</td>
<td>854</td>
</tr>
<tr>
<td>Embedded length, D (mm)</td>
<td>730</td>
<td>800</td>
</tr>
</tbody>
</table>
**TABLE 5**

Specification of pressure transducers

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>MODEL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rated pressure</strong></td>
<td>3500 kPa</td>
</tr>
<tr>
<td></td>
<td>1750 kPa</td>
</tr>
<tr>
<td><strong>Max. pressure</strong></td>
<td>7000 kPa</td>
</tr>
<tr>
<td></td>
<td>3500 kPa</td>
</tr>
<tr>
<td><strong>Rated excitation</strong></td>
<td>10 V/DC</td>
</tr>
<tr>
<td></td>
<td>10 V/DC</td>
</tr>
<tr>
<td><strong>Max. excitation</strong></td>
<td>12 V/DC</td>
</tr>
<tr>
<td></td>
<td>12 V/DC</td>
</tr>
<tr>
<td><strong>Sensitivity</strong></td>
<td>0.028 mV/kPa</td>
</tr>
<tr>
<td></td>
<td>0.056 mV/kPa</td>
</tr>
<tr>
<td><strong>Full scale output (FSO)</strong></td>
<td>100 mV</td>
</tr>
<tr>
<td></td>
<td>100 mV</td>
</tr>
<tr>
<td><strong>Thermal sensitivity</strong></td>
<td>2 % FSO/ 55°C</td>
</tr>
<tr>
<td></td>
<td>2 % FSO/ 55°C</td>
</tr>
<tr>
<td><strong>Comp. temperature</strong></td>
<td>27°C to 80°C</td>
</tr>
<tr>
<td></td>
<td>27°C to 80°C</td>
</tr>
<tr>
<td><strong>Diameter</strong></td>
<td>19 mm</td>
</tr>
<tr>
<td></td>
<td>19 mm</td>
</tr>
</tbody>
</table>
Figure 12: Load cells
Figure 13: Strain gauge assembly for load cells
TABLE 6

Properties of the soil used

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum dry density, $\rho_{d(max)}$</td>
<td>1570 kg/m$^3$</td>
</tr>
<tr>
<td>Minimum dry density, $\rho_{d(min)}$</td>
<td>1340 kg/m$^3$</td>
</tr>
<tr>
<td>Apparent density, $\rho$</td>
<td>1510 kg/m$^3$</td>
</tr>
<tr>
<td>Density index, $I_D$</td>
<td>77 %</td>
</tr>
<tr>
<td>Apparent angle of internal friction, $\phi$</td>
<td>41.2°</td>
</tr>
<tr>
<td>Effective grain size, $D_{10}$</td>
<td>1.45 mm</td>
</tr>
<tr>
<td>Uniformity coefficient, $Cu$</td>
<td>1.4</td>
</tr>
<tr>
<td>Relative density, $D_R$</td>
<td>2.64</td>
</tr>
<tr>
<td>Water content, $\omega$</td>
<td>0.02 %</td>
</tr>
</tbody>
</table>
DIRECT SHEAR TESTS
SOIL DENSITY = 1510 kg/m³

Figure 15: Results of direct shear tests
TRIAXIAL TESTS
SOIL DENSITY = 1510 kg/m³

Figure 16: Results of triaxial tests
was prepared by the raining technique using a hopper shown in Figure 17. The sand was allowed to drop from the hopper through the flexible corrugated plastic hose (50 mm dia.) with a 38 mm diameter 510 mm long straight pipe at the open end. The free fall height of the sand was kept constant at about 100 mm and the sand was laid in layers of 25 mm thickness to obtain the desired density of sand (1,510 kg/m$^3$). Each hopper load of the soil was weighed every time the test sand bed was prepared and the actual density of the deposited sand was computed by measuring the height of the soil in the container by means of 4 measuring scales which were located at the ends of two perpendicular diameters on the inside. The total density of sand in the container for each pour was computed to ensure that the density was uniform. The uniformity of density over the entire depth of the soil was verified by cone penetrometer tests and also confirmed by point resistance force during pile pushing.

3.4. Test procedures

The piles were tested under vertical, lateral, and inclined loads. The inclinations were at 30, 45, and 60 degrees as shown in Figure 8. Pull out tests were also conducted for vertically loaded piles after the completion of the axial testing.

The following is the general procedure adopted for all the above tests in the preparation of the sand bed, loading, and data logging.

First, the test sand bed was prepared by the raining technique described earlier. The density of the sand bed for each pour was checked. If the density of
soil was less than 1,500 kg/m³ or more than 1,520 kg/m³, the test was abandoned and a new test bed was prepared.

After the soil was placed and the density was determined to be within the acceptable range, the recording equipment was checked using the computer program to be used. The pile and the 178 kN screw jack were mounted and made ready for pushing the pile as shown in Figure 6. The pile was lowered to touch the soil. The recording equipment was rechecked manually using the data acquisition unit.

The test pile was then pushed into sand vertically in 50 mm increments at a speed of 0.8 mm/s using the manually operated 178 kN screw jack. At each 50 mm increment, the pile penetration was stopped for about 3 seconds to let the soil and equipment stabilize before readings were taken. Then 10 readings were taken on each reading device, averaged, and recorded on a magnetic disc. Penetration was then continued to the next predetermined depth up to the final depth.

After the test pile was pushed to the predetermined depth, for axial loading tests the load was removed and then reapplied measuring vertical displacements of pile by counting the number of turns of the screw jack crank, while for the inclined and horizontal load tests the 178 kN screw jack was removed and the 44.5 kN screw jack with a swivel joint was installed and set for desired inclination (30, 45, 60, and 90 degrees) as shown in Figure 7. Two LVDTs and dial gauges with a precision of 0.001 mm/div. were mounted to measure displacements of pile in the horizontal direction and the direction of the load as shown in Figure 18.
Figure 17: Hopper and hose
Figure 18: General experimental set up for inclined loads
The load was then applied from 0 to the failure load at a strain rate of about 0.2 mm/s. Data from the pressure transducers, load cells, and LVDTs were sampled, averaged, and recorded in a similar fashion to that described earlier. The test was terminated either based on the load-settlement curve or at a displacement corresponding to half the diameter of the pile.

Before removing the sand or after pushing the pile into the sand, the density of the test sand bed was periodically verified using the Fugro-type cone penetrometer; care was taken to perform the test beyond the zone of the densification influence around the pile. The cone penetrometer was pushed into the sand in increments of 50 mm at a rate of 0.8 mm/s using the 178 kN screw jack. The cone resistance was recorded using the data acquisition unit.

Pull out tests were performed on the vertically loaded piles to find the ultimate pull out resistance.

At the end of the test, the sand was removed from the soil container by opening the doors on the side of the container.

The results of the tests are presented and discussed in the following chapter along with the various theoretical predictions where such theories are available.
Chapter 4

TEST RESULTS and DISCUSSION

4.1. General

The test results and discussion have been organized under the following broad categories:

(1) Evaluation of the sand bed preparation, uniformity of test conditions, and cone penetration tests.

(2) Axial loading of the piles, evaluation of $N_q$, and pull out resistance.

(3) Lateral loading of piles, ultimate lateral loads, and comparison with various theoretical predictions.

(4) Piles under inclined loads, evaluation of the end resistance, and correlation with theoretical calculations.

The interrelation between the above different loading conditions is discussed at the end of the chapter.
4.2. Cone penetration tests and uniformity of test conditions

In order to obtain reproducible test conditions, the raining technique described earlier was used for the preparation of the sand bed. Densities were computed for each hopper load deposited into the container and the density achieved was $1510 \pm 10 \text{ kg/m}^3$ for all tests. A further verification was made of the uniformity of the test bed using the static Fugro type cone penetrometer.

Figure 19a shows the variation of the static cone pressure with depth for five different tests. It may be observed that the results are scattered within $\pm 6\%$. The cone pressure increases linearly, and the soil sample prepared is consistently uniform.

In the bearing capacity formulation for pile foundations in sand, the point resistance force is given by Equation (3) in the form;

$$q_p = \gamma D N_q$$  \hspace{1cm} (3)

where $q_p$ is the point resistance pressure, $\gamma$ is the effective unit weight of soil, $D$ is the depth of pile foundation, and $N_q$ is the bearing capacity factor.

However, beyond a certain critical depth ($D_c$) the point resistance does not increase significantly with depth and thus $q_p$ tends to become constant. This critical depth ($D_c$) is generally believed to be in the range of 10 to 20 times the pile diameter (Kerisel 1964, Vesic 1970, Tavenas 1971). Parameters such as the width of the pile foundation, the density, and the type of the soil, influence the critical depth $D_c$. From the cone penetrometer results in Figures 19a and 19b it
Figure 10: The variation of cone pressure distribution with penetration depth for five typical tests
can be seen that the cone pressure $q_p$ tends to become constant below a depth of 16.5 times the diameter (16.5 $B$) which is taken as the critical depth for the soil tested.

While monitoring the cone penetration resistance, the total load on the cone penetrometer was also constantly measured at the top. The difference between the cone resistance and the total applied load was taken as the resistance due to the skin friction. The total friction as well as the unit frictional stress along the length of cone penetrometer (expressed as the average skin friction), were computed and shown in Table 7 and Figure 20. It may be seen that the frictional force also tends to reach a nearly constant value at a depth of about 19 times the diameter. The frictional stress was also measured by the friction sleeve of the penetrometer. The variation of the unit sleeve friction with depth is shown in Figure 21. It may be seen in Figure 21b that there is a reasonably good correlation between the total skin friction computed from sleeve measurement and that obtained from the measured total force on the penetrometer.

Cone penetration tests show that the soil sample was uniform and repeatable test conditions were obtained for each tests.

### 4.3. Axial load tests on model piles

The tests on the model piles are classified under three broad categories. In the first category discussed in this section, the pile was axially loaded to its ultimate bearing capacity and was also subsequently subjected to pull out tests.

Soon after the test bed was prepared, the pile was pushed into the sand
### TABLE 7

The averaged results of five typical cone penetrometer tests given in Figure 19

<table>
<thead>
<tr>
<th>DEPTH (cm)</th>
<th>( Q_T ) (N)</th>
<th>( Q_P ) (N)</th>
<th>( Q_T - Q_P = Q_s ) (N)</th>
<th>( q_p ) (kPa)</th>
<th>( q_s ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>171</td>
<td>163</td>
<td>8</td>
<td>163</td>
<td>0.7</td>
</tr>
<tr>
<td>30</td>
<td>254</td>
<td>242</td>
<td>12</td>
<td>242</td>
<td>0.72</td>
</tr>
<tr>
<td>40</td>
<td>347</td>
<td>326</td>
<td>21</td>
<td>326</td>
<td>0.94</td>
</tr>
<tr>
<td>50</td>
<td>467</td>
<td>432</td>
<td>35</td>
<td>432</td>
<td>1.24</td>
</tr>
<tr>
<td>60</td>
<td>523</td>
<td>481</td>
<td>42</td>
<td>481</td>
<td>1.26</td>
</tr>
<tr>
<td>70</td>
<td>529</td>
<td>478</td>
<td>51</td>
<td>478</td>
<td>1.3</td>
</tr>
<tr>
<td>80</td>
<td>555</td>
<td>504</td>
<td>51</td>
<td>504</td>
<td>1.12</td>
</tr>
<tr>
<td>90</td>
<td>560</td>
<td>502</td>
<td>58</td>
<td>502</td>
<td>1.14</td>
</tr>
</tbody>
</table>

\( q_p = Q_p / A_s \)

\( q_s = Q_s / (0.5xA_s) \) at a given depth above critical depth
Figure 20: The variation of shaft resistance with penetration depth for six typical cone penetrometer tests.
Figure 21: The averaged variation of unit sleeve friction and shaft resistance with penetration depth
slowly at 0.8 mm/s to the predetermined depth. Once the desired depth of foundation was reached, the load was removed and the pile was allowed to set.

The total resistance of the pile to penetration was measured by the load cell on the top of the pile, while the end resistance was measured by the load cell at the tip. The difference between the two is the shaft resistance due to skin friction. Typical results of the point resistance as the pile penetrated the soil are given in Figures 22, 23, and 24 for the piles of 73 mm, 90 mm, and 102 mm diameters respectively. The results for the three different piles are compared in Figure 25. The average unit cone resistance obtained from the cone penetrometer tests are also shown in this Figure.

From the cone penetrometer tests the critical depth for $q_p$ for this material was found to be 16.5 times the diameter of the pile. For a 73 mm diameter pile the critical depth will therefore be in the order of 1.2 m. However, the maximum depth of penetration for the model piles was 90 cm which is less than the critical depth. It can be seen in Figure 25 that while the critical depth was reached for the cone penetrometer, the pile penetration is still less than the critical depth for all the three piles.

4.3.1. Load tests, Base resistance, and $N_q$

Load tests on pile foundations fall under two broad categories. In the load-controlled method, the load is applied in increments of the design load and maintained until the settlement ceases. In the displacement-controlled mode, small increments of settlement are imposed and maintained until the load reaches
Figure 22: The variation of point resistance with relative depth (73 mm diameter)
Figure 23: The variation of point resistance with relative depth (90 mm diameter)
Figure 24: The variation of point resistance with relative depth (102 mm diameter)
Figure 25: A comparison of average point pressure for model piles and cone penetrometer with relative depth
equilibrium. Although the process of initially pushing the pile to the required depth is a variety of load test under a vertical load, the rate and magnitude of loading, it is, however, to be noted that the resulting displacements are very large by normal standards for load tests. Nevertheless, the measured loads represent the ultimate axial bearing capacity at each depth during the process of pile penetration.

Load tests were conducted after the piles were pushed to the required depths and the loads were applied monotonically in the vertical, horizontal, or inclined directions, as required. In the case of axially loaded vertical piles, these load tests supplement the information already obtained while pushing the piles into the soil. In the case of piles under inclined and lateral loads, load tests were necessary to evaluate the ultimate load in the required direction.

The load-settlement curves for the axially loaded vertical piles are shown in Figure 26. The three sizes, 73 mm, 90 mm, and 102 mm were first pushed to a depth such that the D/B ratio was about 10 in each case. In addition, the 102 mm diameter pile alone was tested at three different D/B ratios and those load-settlement curves are shown in Figure 27. The criterion for establishing the ultimate load from load-settlement diagrams has been discussed by Whitaker (1957, 1963), Berezantzev (1965), Vesic (1967), and Poulos and Davis (1980). The point where the portion of the load-settlement curve becomes straight or substantially straight is generally taken as the failure load. These are so identified in Figures 26, and 27. It is, however, to noted that a consistent and reliable interpretation of the test results requires some familiarity, experience, and judgement.
Figure 26: Load-settlement curves for model piles under vertical loads
Figure 27: The load-settlement curves for a 102 mm diameter pile under vertical loads
The various theories available to determine the end bearing capacity were described in Chapter 2. The theories of Terzaghi (1943), Brinch Hansen (1951), Berezantzev (1961), Durgunoglu and Mitchell (1973), Meyerhof (1976), and Vesic (1977) were used to compute the end bearing resistance. The theoretical and experimental results are tabulated in Table 8 and also compared in Figures 28, 29, and 30 for the piles of 73 mm, 90 mm, and 102 mm diameters. The general equation for the end bearing resistance is given by:

\[ q_p = \gamma D N_q \]  

(3)

Thus for a given soil at a particular depth the end resistance depends on the assumed value of \( N_q \). The various theories described above differ from one another in the assumed soil failure mode and hence the value of \( N_q \) varies from one theory to the other. This variation is shown in Figure 31 together with the values of \( N_q \) computed from the measured bearing capacity. It is seen that the experimental results are closest to the theoretical values of Vesic (1977). In the theoretical computation of \( N_q \), Meyerhof (1976) suggested that the \( N_q \) increases with depth. A somewhat similar increase was suggested by Durgunoglu and Mitchell (1973) up to a certain depth while Berezantzev (1961) indicated a decrease of \( N_q \) with depth for deep foundations in sand. Vesic (1977) proposed a constant value of \( N_q \). The variation of \( N_q \) obtained from the present tests (Figure 31) show an agreement with the conclusions of Vesic (1977). In fact there is a slight decrease in \( N_q \) with depth.
### TABLE 8
Comparison of theoretical and measured ultimate bearing loads (kN)

<table>
<thead>
<tr>
<th>METHODS</th>
<th>PILE DIAMETER</th>
<th>73(mm)</th>
<th>90(mm)</th>
<th>102(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>Q_p</td>
<td>Q_s</td>
<td>Q_u</td>
</tr>
<tr>
<td>Terzaghi (1943)</td>
<td>730</td>
<td>5.33</td>
<td>0.37</td>
<td>5.7</td>
</tr>
<tr>
<td>Brinch Hansen (1951)</td>
<td>730</td>
<td>9.27</td>
<td>0.37</td>
<td>9.6</td>
</tr>
<tr>
<td>Berezantsev (1961)</td>
<td>730</td>
<td>7.77</td>
<td>0.37</td>
<td>8.14</td>
</tr>
<tr>
<td>Mitchell (1973)</td>
<td>730</td>
<td>4.30</td>
<td>0.37</td>
<td>4.7</td>
</tr>
<tr>
<td>Meyerhof (1976)</td>
<td>730</td>
<td>15.82</td>
<td>0.37</td>
<td>16.2</td>
</tr>
<tr>
<td>Vesic (1977)</td>
<td>730</td>
<td>3.62</td>
<td>0.37</td>
<td>4.0</td>
</tr>
<tr>
<td>Experiment</td>
<td>730</td>
<td>2.9</td>
<td>0.2</td>
<td>3.1</td>
</tr>
</tbody>
</table>
Figure 28: A comparison of theoretical and experimental point resistance pressures with relative depth for a 73 mm diameter pile.
Figure 29: A comparison of theoretical and experimental point resistance pressures with relative depth for a 90 mm diameter pile
Figure 30: A comparison of theoretical and experimental point resistance pressures with relative depth for a 102 mm diameter pile
Figure 31: The variation of bearing capacity factor $N_q$ with relative depth
4.3.2. Skin friction

The ultimate bearing capacity of a pile is the sum of the point (base) resistance force and the shaft friction. The point resistance force which is the primary component in cohesionless soils was discussed in the preceding section. The averaged values of the point and shaft resistance, during pile penetration, are shown in Table 9 and Figure 32. The shaft resistance is in the order of 5 - 12% of the total ultimate resistance and can be considered as not significant, consistent with the normal practice for piles in cohesionless soils. However, during the review of the fairly extensive literature on shaft friction of piles, it was observed that there are still several uncertainties in the computation of the frictional resistance. Although the evaluation of skin friction is not a major topic in this research, some of the problems in the determination of the shaft resistance will be briefly discussed below.

The shaft resistance of a pile in sand is given by

\[ Q_s = q_s A_s \]  

(26)

where \( A_s \) is the area of the pile shaft and \( q_s \) is the unit shaft resistance.

\[ q_s = q_n \tan \delta \]  

(27)

where \( q_n \) is the normal stress acting on the foundation shaft and \( \delta \) is the angle of friction between the pile material and soil.

\[ q_n = K_s q_v \]  

(28)
### TABLE 9

Values of measured point resistance force and shaft resistance

<table>
<thead>
<tr>
<th>DEPTH (cm)</th>
<th>PILE DIAMETER</th>
<th>73 mm</th>
<th>90 mm</th>
<th>102 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q&lt;sub&gt;T&lt;/sub&gt;</td>
<td>Q&lt;sub&gt;p&lt;/sub&gt;</td>
<td>Q&lt;sub&gt;r&lt;/sub&gt;</td>
<td>Q&lt;sub&gt;T&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td>(kN)</td>
<td>(kN)</td>
<td>(kN)</td>
<td>(kN)</td>
</tr>
<tr>
<td>20</td>
<td>0.92</td>
<td>0.88</td>
<td>0.04</td>
<td>1.18</td>
</tr>
<tr>
<td>30</td>
<td>1.31</td>
<td>1.24</td>
<td>0.07</td>
<td>1.85</td>
</tr>
<tr>
<td>40</td>
<td>1.84</td>
<td>1.70</td>
<td>0.14</td>
<td>2.28</td>
</tr>
<tr>
<td>50</td>
<td>2.27</td>
<td>2.05</td>
<td>0.22</td>
<td>2.82</td>
</tr>
<tr>
<td>60</td>
<td>2.61</td>
<td>2.37</td>
<td>0.24</td>
<td>3.37</td>
</tr>
<tr>
<td>70</td>
<td>3.00</td>
<td>2.70</td>
<td>0.30</td>
<td>3.75</td>
</tr>
<tr>
<td>80</td>
<td>3.72</td>
<td>3.74</td>
<td>0.54</td>
<td>5.51</td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 32: The variation of point resistance and shaft resistance with depth
where $K_s$ is the coefficient of earth pressure and $q_v$ is the effective vertical ground stress.

The computation of $q_n$ is not simple. The coefficient of skin friction $K_s$ is not uniform and varies with depth from the passive to active pressure range (Coyle et al. 1979). The effective ground stress reaches a critical value due to arching action. Vesic (1977) has suggested several theoretical load transfer models for evaluating $Q_s$, but no experimental work appears to have been conducted to verify any of those models. Cone penetrometer tests show that the shaft resistance $Q_s$ reaches a constant value (Figure 21) somewhat similar to the critical value for the point resistance force $Q_p$. Even if the shaft resistance may be a fraction of the base resistance, the possibility that it could reach a critical value has to be examined as a separate research topic.

4.3.3. Pull out resistance

The pull out resistance of piles is an important parameter in the design of offshore structures. Determination of the pull out resistance of a vertical pile is also generally important in the design of tall structures against overturning moments, buoyant structures against uplift forces, and structures against frost expansion of soil, etc.

Although the resistance to pull out is the result of soil-pile friction, the pull out resistance generally tends to be less than the shaft friction discussed in the
previous section. Pull out tests were conducted on the piles and load-deflection curves similar to the load tests discussed for the downward loads were obtained. The results are shown in Figure 33 and the pull out loads are identified.

Several theories have been proposed to compute the pull out resistance of piles in sand (Meyerhof 1973, Poulos 1980, Levacher and Sieffert 1984). Table 10 shows the computed pull out resistance, measured shaft friction, and measured pull out resistance. It is seen that there is a considerable variation between the measured and the computed pull out resistance. A comparison of the measured shaft friction and pull out resistance shows that a good correlation can be obtained by expressing the pull out resistance $Q_s'$ as:

$$Q_s' = 0.5 Q_s + W$$

where $W$ is the weight of pile.

The correlation of the results using the above expression with the measured values is also shown in Table 10.

Axial loading of vertical piles shows that the existing theories generally overestimate the bearing capacity of piles in sand. Tests indicate that the shaft resistance also tends to reach a critical value similar to the end bearing resistance. This has to be investigated in some detail. The pull out resistance at the soil-pile interface is about one half of the shaft resistance.
Figure 33: Load test curves for piles subjected to pull out forces
### TABLE 10

Pull out resistance (kN)

<table>
<thead>
<tr>
<th>PILE DIA.</th>
<th>D/B</th>
<th>Qp</th>
<th>W</th>
<th>THEORY</th>
<th>EXPERIMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>73 mm</td>
<td>10</td>
<td>0.37</td>
<td>0.12</td>
<td>0.36</td>
<td>3.28</td>
</tr>
<tr>
<td>90 mm</td>
<td>9</td>
<td>0.54</td>
<td>0.18</td>
<td>0.54</td>
<td>4.4</td>
</tr>
<tr>
<td>102 mm</td>
<td>5</td>
<td>0.25</td>
<td>0.27</td>
<td>0.44</td>
<td>2.21</td>
</tr>
<tr>
<td>102 mm</td>
<td>7</td>
<td>0.49</td>
<td>0.27</td>
<td>0.8</td>
<td>4.08</td>
</tr>
<tr>
<td>102 mm</td>
<td>9</td>
<td>0.78</td>
<td>0.27</td>
<td>0.79</td>
<td>6.32</td>
</tr>
</tbody>
</table>

\[ Q_p = 0.5 K_\gamma D \tan \delta A \]

1. **POULOS** \([2/3]Q_p + w\]

2. **MEYERHOF** \([B \gamma D^2 k_b/2 + W]\) (for rough piles)

3. **LEVACHER** \([0.5 K_\gamma Pf_h^2K_{mo}]\)

4. **PROPOSED** \([Q_p/2 + w]\)

W = the weight of pile
4.4. **Vertical pile under lateral loads**

The second series of load tests consisted of a vertical pile subjected to horizontal loads at the top of the pile. Initially the piles were pushed into the sand as described in Chapter 3. The larger screw jack was then removed and the smaller screw jack with a swivel joint was mounted on the frame and set for horizontal loads. The horizontal deflection of the pile was measured by gradually increasing lateral loads. Load-deflection curves similar to that already described were obtained. Typical curves for the piles of 73 mm, 90 mm, and 102 mm diameters are shown in Figures 34, 35, and 36. The ultimate lateral resistance of each pile was obtained as already described, from the load-deflection curves.

The theoretical lateral load capacity of a circular pile can also be obtained considering the pressure distribution along the length of the pile. The formulations developed by Brinch Hansen (1961), Broms (1961), Petrasovits and Adams (1972), and Meyerhof et al. (1976, 1981) can be used to find the total lateral soil resistance. The details of these theories were discussed in Chapter 2. All the above theories take into account the effect of eccentricity \( e \), which is the distance between the point of application of load and the soil surface. As the value of \( e \) becomes greater relative to the pile length below the ground level, there is a corresponding reduction in the ultimate lateral capacity. This effect is shown in Figures 37, 38, and 39 for the various theories and for the different pile diameters. The measured value of the ultimate lateral load is also shown in these figures. It may be seen that the values predicted by all theoretical methods are higher than the measured values. Meyerhof's (1981) theory is the closest to the
Figure 34: Load-curves for a 73 mm diameter pile under inclined loads
Figure 35: Load-deflection curves for a 90 mm diameter pile under inclined loads
Figure 36: Load-deflection curves for a 102 mm diameter pile under inclined loads
Figure 37: The variation of ultimate lateral load with the ratio $e/D$ for a 73 mm diameter pile
Figure 38: The variation of ultimate lateral load with the ratio e/D for a 90 mm diameter pile
Figure 39: The variation of ultimate lateral load with the ratio $e/D$ for a 102 mm diameter pile.
experimental values. In all the above theories, it is assumed that the pressure acts uniformly on the projected width of the pile which for a circular pile is its diameter. If $p$ is the pressure at any point, the lateral force $Q_n$ is given by

$$Q_n = \sum_{x=0}^{D} p x B$$

$$= \int_{x=0}^{D} p x B dx$$

(30)

where $B$ is the pile diameter.

A nomenclature diagram explaining the above concept is given in Figure 40.

For a circular pile, it is inappropriate to assume that the pressure will be a constant across the diameter. In fact, at the two ends, it is most probable that the pressure is zero or nearly so while at the center where the curvature is a maximum relative to the direction of the pile movement, the pressure will be a peak value. This concept is also shown in Figure 40. Pressures which are measured experimentally are these maximum pressures. Using this approach, the measured ultimate lateral loads were compared with theoretical computations as shown in Table 11. It may be seen that the measured values tend to be closer to computed values when the pressure across the pile diameter is assumed to be parabolic instead of a rectangular distribution. In order to further verify the above assumption, a 73 mm square pile was fabricated and tested under lateral loads. The load test results are shown in Figure 41 and the ultimate load is compared with that for the circular pile in Table 11. It may be seen that there is a better agreement between the various theories and the result from the square
Figure 40: Lateral earth pressure distribution along the pile length and across the pile width
TABLE 11

Computed and measured ultimate lateral resistance

<table>
<thead>
<tr>
<th>DIAMETER</th>
<th>THEORY</th>
<th>LATERAL RESISTANCE</th>
<th></th>
<th></th>
</tr>
</thead>
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<td></td>
<td></td>
<td>CALCULATED</td>
<td>EXPERIMENTAL</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\sum P_x B$</td>
<td>$\frac{2}{3} \sum P_x B$</td>
<td>CIRCULAR</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(kN)</td>
<td>(kN)</td>
<td>(kN)</td>
</tr>
<tr>
<td>73 mm</td>
<td>Brinch Hansen</td>
<td>1.3</td>
<td>0.87</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Broms</td>
<td>1.1</td>
<td>0.73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Petrasovits</td>
<td>1</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Meyerhof</td>
<td>0.89</td>
<td>0.59</td>
<td></td>
</tr>
<tr>
<td>90 mm</td>
<td>Brinch Hansen</td>
<td>1.7</td>
<td>1.13</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Broms</td>
<td>1.54</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Petrasovits</td>
<td>1.37</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Meyerhof</td>
<td>1.13</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>102 mm</td>
<td>Brinch Hansen</td>
<td>2.41</td>
<td>1.6</td>
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<td></td>
<td>Broms</td>
<td>2.2</td>
<td>1.47</td>
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</tr>
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<td></td>
<td>Petrasovits</td>
<td>1.96</td>
<td>1.3</td>
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<tr>
<td></td>
<td>Meyerhof</td>
<td>1.63</td>
<td>1.09</td>
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</tr>
</tbody>
</table>
Figure 41: Load-deflection curve for a square pile of 73 mm under lateral loads
pile test. Assumption of a uniform pressure across the diameter will lead to an overestimation of the ultimate capacity of circular piles.

It is also seen from Table 11 that the computations using the theories of Broms and Petrasovits are closer to the measured lateral capacities. One of the reasons for the differing theoretical estimates between the various theories in Table 11 is the assumed pressure distribution along the length of the pile. Those pressure distributions are shown in Figures 42, 43, and 44 for the different pile diameters together with the measured pressure distribution at failure superimposed therein. It is seen that the actual pressure distribution curve is entirely different from all the theoretical assumptions. Adams and Radhakrishna (1973) reported tests on a pile under lateral loads and obtained a lateral pressure distribution somewhat similar that obtained in this work. Chari and Meyerhof (1983) have reported a similar pressure distribution. It is reasonable to conclude that the best estimate of the lateral load capacity is obtained by considering the nonlinear pressure distribution from the experimental measurement.

The soil pressure along the length of the pile at the ultimate lateral load is shown in Figures 45, 46, 47, and 48 for the different piles. A comparison is made in Table 12 between the applied lateral load and that integrated from the pressure distribution. There is a net unbalanced force in all cases which most likely acts as a reaction at the base of the pile. No definite correlation can be made with the limited data available at present, but it is suggested that this should be examined further to quantify the pressure distribution in terms of the soil properties.
Based on the soil pressure distribution, shown in Figures 45, 46, 47, and 48, the location of the point of rotation was examined. It was found that the depth of rotation $D_o$ is not very much influenced by the magnitude of the applied load, and is located at about 0.75 D. This compares well with the results reported by Chari and Meyerhof (1983).

Pile tests under lateral loads show that the existing theories require some modifications. For circular piles, the pressure distribution across the diameter is not likely to be constant. Further work is required to quantify the reaction at the base considering the base area of a larger pile and the pressure distribution along the length of the pile.

4.5. Vertical pile under inclined loads

In the last series of tests, the behaviour of vertical piles under inclined loads was studied. Presently available data on the behaviour of piles subjected to inclined loads are somewhat limited.

The test bed was prepared similar to the other tests and the pile was pushed into the soil as described earlier. Inclined loads were applied using the small jack and the inclination of the load was facilitated by means of a swivel joint on top of the jack. The load on the pile, the lateral pressures, the end resistance at the tip of the pile, and deflections were measured as described earlier for the lateral load tests and recorded using the data logging system. The ultimate inclined load in each case was experimentally determined from load-deflection curves as already described and shown in Figures 34, 35, and 36. Figure 49 shows the results of
Figure 42: A comparison of assumed and actual soil pressure distribution along the pile length under lateral loads for a 73 mm diameter pile
Figure 43: A comparison of assumed and actual soil pressure distribution along the pile length under lateral loads for a 90 mm diameter pile.
Figure 44: A comparison of assumed and actual soil pressure distribution along the pile length under lateral loads for a 102 mm diameter pile.
Figure 45: The measured soil pressure distribution along the length of a 73 mm diameter pile under lateral loads.
Figure 46: The measured soil pressure distribution along the length of a 90 mm diameter pile under lateral loads.
Figure 47: The measured soil pressure distribution along the length of a 102 mm diameter pile under lateral loads.
Figure 48: The measured soil pressure distribution along the length of a square pile of 73 mm under lateral loads
TABLE 12

Computed and measured lateral resistances based on the actual pressure distribution

<table>
<thead>
<tr>
<th>CIRCULAR PILE DIA. (mm)</th>
<th>SQUARE PILE WIDTH (mm)</th>
<th>D (mm)</th>
<th>e/D</th>
<th>ASSUMPTIONS</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Q&lt;sub&gt;hm&lt;/sub&gt; (kN)</td>
</tr>
<tr>
<td>73</td>
<td>730</td>
<td>0.23</td>
<td>0.76</td>
<td>0.84</td>
</tr>
<tr>
<td>90</td>
<td>800</td>
<td>0.31</td>
<td>0.8</td>
<td>0.91</td>
</tr>
<tr>
<td>102</td>
<td>900</td>
<td>0.31</td>
<td>1.4</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>73</td>
<td>730</td>
<td>0.23</td>
<td>0.92</td>
</tr>
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</table>

Q<sub>hm</sub> = Q<sub>u</sub>

Q<sub>hc</sub> = \( \frac{2}{3}B\sum P_x \) (for circular piles)

Q<sub>hc</sub> = B \( \sum P_x \) (for square piles)

R<sub>tip</sub> = Q<sub>hc</sub> - Q<sub>hm</sub>
Figure 49: Load-deflection curves for a 102 mm diameter pile at 30° load inclination at different embedment depths
inclined load tests of a 102 mm diameter pile at 30 degrees with different embedments and eccentricities.

As already noted in an earlier section, the measured ultimate axial load capacity was closest to Vesic’s (1977) theory and the ultimate lateral resistance was closest to that of Meyerhof (1981). These two theories were used to compute the ultimate axial and lateral loads. From these limiting loads and using the interaction equation of Meyerhof and Ranjan (1981), the bearing capacity at different inclinations of the load was theoretically computed and compared with the measured values. These results are shown in Figures 50, 51, and 52, and in a polar representation, in Figure 52A. It is seen that the computed results are consistently lower than the experimental values at all the load inclinations. The results are also shown in Table 13. The ultimate pile capacity under inclined loads was also computed as a percentage of ultimate vertical load capacity. The results are shown in Figure 53. It is seen that at a load inclination of 30 degrees the ultimate bearing capacity increases by 5 to 16% compared to the vertical load capacity. It can also be seen that the ultimate bearing capacity reduces rapidly when the inclination of load is between 45 to 60 degrees. These experimental results are somewhat similar to those reported by Berezantzev et al. (1961), and Awad and Petrasovits (1968).

If $Q_a$ is the ultimate bearing capacity under an axial load, $Q_n$, the ultimate lateral load capacity, and $Q_u$, the ultimate load at an inclination $\alpha$, it may be concluded
Figure 50: A comparison of theoretical and measured ultimate load capacities with load inclination for a 73 mm diameter pile

DIA. = 73 mm  D=730 mm  e/D=0.2

**---THEORY**

O---EXPERIMENT

INCLINATION OF LOAD (deg.)
Figure 51: A comparison of theoretical and measured ultimate load capacities with load inclination for a 90 mm diameter pile.
Figure 52: A comparison of theoretical and measured ultimate load capacities with load inclination for a 102 mm diameter pile.
Figure 52A: The variation of ultimate bearing capacity with load inclination for model piles
TABLE 13

Theoretical bearing capacity under inclined loads

<table>
<thead>
<tr>
<th>D(mm)</th>
<th>e/D</th>
<th>α(deg)</th>
<th>Theo.(Q_u)(kN)</th>
<th>Exp.(Q_u)(kN)</th>
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</thead>
<tbody>
<tr>
<td>73</td>
<td>0.23</td>
<td>0</td>
<td>4.0</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>1.7</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45</td>
<td>1.2</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>0.89</td>
<td>0.76</td>
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<td>90</td>
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<td>6.6</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>2.2</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45</td>
<td>1.6</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60</td>
<td>1.3</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90</td>
<td>1.13</td>
<td>0.8</td>
</tr>
<tr>
<td>102</td>
<td>0.31</td>
<td>0</td>
<td>9.5</td>
<td>8.3</td>
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<td></td>
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<td>3.1</td>
<td>8.7</td>
</tr>
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<td></td>
<td></td>
<td>45</td>
<td>2.3</td>
<td>6.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60</td>
<td>1.9</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90</td>
<td>1.63</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Figure 53: The variation of the ratio $Q_s/Q_u$ with load inclination for model piles
that \[ Q_u \cos \alpha < Q_a \] (31)

\[ Q_u \sin \alpha < Q_n \] (32)

Recalling that \( Q_n \) is the lateral capacity when the pile is not under any axial load, equation (32) can be modified and expressed as

\[ Q_u \sin \alpha < k Q_n \] (33)

where \( k \) is a factor which depends on \( \alpha \).

From (31) and (33), \( Q_u \) can be maximized and the corresponding value of \( \alpha \) may be expressed as

\[
\tan \alpha = \frac{k Q_n}{Q_u} \quad (34)
\]

The component \( Q_u \cos \alpha \) and \( Q_u \sin \alpha \) are shown in Table 14 from which the value of \( k \) may be estimated to be in the order of 3.0. Based on the simple analysis presented above the critical angle \( \alpha \) for a maximum \( Q_u \) can be estimated. While it can be shown theoretically that the ultimate load capacity will increase with the load inclination up to an angle of 30° - 35°, this is a potential area of further detailed mathematical analysis and experimental study.

The variation of lateral earth pressures along the pile length under inclined loads are plotted in Figures 54, 55, and 56 for the piles of 73mm, 90 mm, and 102 mm diameters. A summary of all the tests under inclined loads is presented in Table 15. The depth of pile rotation was examined based on the earth pressure
distribution and it is seen that $D_o$ increases initially with increasing $\alpha$ to about 45° and then decreases as shown in Table 15. This is found to be true for all the piles. It is also noticed in this table that the end resistance under inclined loads is not likely to decrease continuously with inclination of load, and that the pile diameter has little effect on the variation of the $D_o/B$ ratio with the load inclination.

Although some correlations can be established between the measured pressure distribution and the ultimate inclined load capacity similar to that attempted for the lateral load $Q_n$, it is felt that further experimental work will be necessary before any conclusions can be drawn.

Based on the test results and discussion, a set of conclusion and areas requiring further work are presented in the following chapter.
### TABLE 14

**Pile tests under inclined loads-components of the ultimate load**

<table>
<thead>
<tr>
<th>PILE DIA. (mm)</th>
<th>$\alpha$ (deg)</th>
<th>$Q_u$ (kN)</th>
<th>$Q_u \cos \alpha$ (kN)</th>
<th>$Q_u \sin \alpha$ (kN)</th>
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</thead>
<tbody>
<tr>
<td>73</td>
<td>30</td>
<td>3.6</td>
<td>3.1</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>3.0</td>
<td>2.1</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>1.4</td>
<td>0.7</td>
<td>1.2</td>
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<tr>
<td></td>
<td>90</td>
<td>0.76</td>
<td>0</td>
<td>0.76</td>
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<td>5.0</td>
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<td>1.9</td>
<td>0.95</td>
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<td>4.35</td>
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Figure 54: The measured soil pressure distribution along the length of a 73 mm diameter pile under inclined loads.
Figure 55: The measured soil pressure distribution along the length of a 90 mm diameter pile under inclined loads.
Figure 56: The measured soil pressure distribution along the length of a 102 mm diameter pile under inclined loads
### TABLE 15

**Summary of tests under inclined loads**

<table>
<thead>
<tr>
<th>B (mm)</th>
<th>D (mm)</th>
<th>D/B</th>
<th>e (mm)</th>
<th>e/D</th>
<th>α (deg)</th>
<th>$Q_u$ (kN)</th>
<th>$Q_p$ (kN)</th>
<th>$D_o$ (mm)</th>
<th>β (deg)</th>
<th>$D_o/D$</th>
<th>$D_o/B$</th>
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<tr>
<td>73</td>
<td>730</td>
<td>10</td>
<td>170</td>
<td>0.23</td>
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<td>3.1</td>
<td>2.9</td>
<td>549</td>
<td>1.88</td>
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<td></td>
<td>30</td>
<td>3.6</td>
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<td>596</td>
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<td>60</td>
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<td>674</td>
<td>1.78</td>
<td>0.75</td>
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</tr>
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</table>
Chapter 5

SUMMARY and CONCLUSIONS

Laboratory experiments were conducted to better understand the behaviour of a vertical short rigid pile under inclined loads in sand and the comparison was made of experimental and theoretical values. The following conclusions are drawn on the results of this research work.

(1) Cone penetration tests show that fairly uniform and repeatable test conditions are obtained for the soil using the raining technique. The critical depth for the soil used was found to be 16.5 B consistent with the range of values reported in the literature.

(2) The values for the bearing capacity factor $N_q$ compare well with those obtained by Vesic (1977). The value of $N_q$ does not vary with depth and is found to be nearly constant. All existing theories are found to generally overestimate the bearing capacity of piles in sand.

(3) The shaft resistance in sands is only a fraction of the total ultimate capacity. However, preliminary analysis shows that this shaft resistance also reaches a critical value. Further study is required to show how shaft resistance is affected by method of pile installation.
(4) The pressure distribution along the length of the pile is nonlinear, contrary to the assumptions made in the various existing theories. Further work is required to obtain an analytical solution to the actual pressure distribution.

(5) Pull out resistance of a vertical smooth pile is estimated as about one half of the shaft resistance; This is a modification of the presently available theoretical estimates and gives a good correlation with actual measured values.

(6) Predictions of ultimate lateral resistance for circular piles using existing theories overestimate the load capacity. This may be due to the assumption that the lateral soil pressure is uniform on the projected width of pile. For the calculation of lateral resistance of a circular pile, the shape of pressure distribution across the pile diameter is to be taken into account. A parabolic pressure distribution is suggested for better correlation.

(7) The point of rotation of the pile under lateral load was found to be about 0.75 times the embedment depth and is not much influenced by the magnitude of the applied load. This result compares well with the results reported by Chari and Meyerhof (1983).

(8) The ultimate bearing capacity of a pile under a load inclined at 30° was 5 to 16 % higher than the axial ultimate bearing capacity. The ultimate bearing capacity decreases gradually after a 30° inclination. The reduction is rapid for inclinations larger than 45°. This result compares well with the published results by Adams and Petrasovits (1968). Further theoretical work is required to examine this phenomenon in detail.
REFERENCES


APPENDIX

COMPUTER PROGRAMS
REM ************************** JMTEST ******************************
10 REM **** PROGRAM TO MEASURE PRESSURES ON ************
  ******** PRESSURE TRANSDUCERS & LOAD CELLS *****
20 DIM BDATA$(1488),CDATA$(1488),TL(11),BL(11),P1(11),
P2(11),P3(11),P4(11),P5(11),P6(11),P7(11),P8(11),
P9(11),P10(11),P11(11),P12(11),P13(11),P14(11),
MULTPLX(22)!
30 DIM TLAVG(25),BLAVG(25),PIAVG(25),P2AVG(25),P3AVG
  (25),P4AVG(25),P5AVG(25),P6AVG(25),P7AVG(25),P8AVG
  (25),P9AVG(25),P10AVG(25),P11AVG(25),P12AVG(25),
P13AVG(25),P14AVG(25),D(25)!
40 DISP "INPUT NUMBER OF LOAD READINGS TO BE TAKEN 
  (=NLR)"
50 INPUT NLR
60 REM *** NUMBER OF DATA PER CHANNEL (NDP=10) ****
70 NDP=10
80 DISP "INPUT THE NUMBER OF CHANNELS (NC=PT'S+LC'S 
  LVDT'S)"
90 INPUT NC
100 CREATE "PILE",20,256
110 ASSIGN # 1 TO "PILE"
120 REM **** START LOOP TO MEASURE PRESSURES *****
130 FOR J=1 TO NLR
140 DISP "INPUT DEPTH (cm or turns)"
150 INPUT D(J)
160 REM ****************** READ DATA ********************
170 IOBUFFER BDATA$
180 DISP "Reading data from 3497A for BDATA$"
190 CLEAR 509
200 OUTPUT 509 ;"VF2VAOVR2VT2SDO"
210 OUTPUT 509 ;"SO1VN1AF2AL11AE1AC2T02"
220 REM **** TRANSFER DATA TO FILE USING FHS *****
230 TRANSFER 509 TO BDATA$ FHS
240 LOCAL 509 @CLEAR 509 @BEEP 10,100
250 DISP "Data transfer complete"
260 IF NC<=10 THEN GOTO 370
270 DISP "NC>10"
280 IOBUFFER CDATA$
290 DISP "Reading data from 3497A for CDATA$"
300 CLEAR 509
310 OUTPUT 509 ;"VF2VAOVR2VT2SD0"
320 OUTPUT 509 ;"SO1VN1AF12AL17AE1AC12T02"
330 TRANSFER 509 TO CDATA$ FHS
340 LOCAL 509 @CLEAR 509 @BEEP 10,100
350 REM **************** UNPACKING DATA *******************
360 DISP "Unpacking data. Please wait"
370 FOR I=3 TO 3*NDP*NC STEP 3
380 IF I>3*NDP*10 THEN GOTO 570
390 A$=DTB$(NUM(BDATA$[I-2,I-2]))
400 D$=A$
410 A2=BINAND(BDT(A$[9,10],3)
420 M=10**(-6+A2) ! Range multiplier
430 IF BINAND (BDT(A$[11,11],1)=1 THEN SIGN=-1 ELSE
  SIGN=1
440 ORNG = BINAND (BTD(A$[12,12]),1) ! Overange bit
450 MSD = BINAND (BTD(A$[13,16]),15)
460 A$ = DTB$(NUM(BDATA$[I-1,I-1]))
470 B$ = A$
480 SSD = BINAND (BTD(A$[9,12]),15)
490 TSD = BINAND (BTD(A$[13,16]),15)
500 A$ = DTB(NUM(BDATA$[I,I]))
510 C$ = DTB(NUM(BDATA$[I,I]))
520 FSD = BINAND (BTD(A$[9,12]),15)
530 LSD = BINAND (BTD(A$[13,16]),15)
540 MULTPLX (I/3) = (ORNG*10**5 + MSD*10**4 + SSD*10**3 + TSD *10**2 + FSD*10 + LSD) * M*SIGN
550 NEXT I
560 GOTO 750
570 FOR I = 3*NDP*10+3 TO 3*NDP*NC+6 STEP 3
580 A$ = DTB$(NUM(CDATA$[I-2,I-2]))
590 D$ = A$
600 A2 = BINAND (BTD(A$[9,10]),3)
610 M = 10**(6+A2)
620 IF BINAND (BTD(A$[11,11]),1) = 1 THEN SIGN = -1 ELSE
630 ORNG = BINAND (BTD(A$[12,12]),1)
640 MSD = BINAND (BTD(A$[13,16]),15)
650 A$ = DTB$(NUM(CDATA$[I-1,I-1]))
660 B$ = A$
670 SSD = BINAND (BTD(A$[9,12]),15)
680 TSD = BINAND (BTD(A$[13,16]),15)
690 A$ = DTB$(NUM(CDATA$[I,I]))
700 C$ = DTB$(NUM(CDATA$[I,I]))
710 FSD = BINAND (BTD(A$[9,12]),15)
720 LSD = BINAND (BTD(A$[13,16]),15)
730 MULTPLX (I/3) = (ORNG*10**5 + MSD*10**4 + SSD*10**3 + TSD *10**2 + FSD*10 + LSD) * M*SIGN
740 NEXT I
750 CALO = 1
760 REM ******************** DEMULTPLEX ********************
770 Cl = 1
780 FOR I = 1 TO NDP
790 TL(I) = MULTPLX(C1)*CALO @ C1 = C1+1
800 BL(I) = MULTPLX(C1)*CALO @ C1 = C1+1
810 P1(I) = MULTPLX(C1)*CALO @ C1 = C1+1
820 P2(I) = MULTPLX(C1)*CALO @ C1 = C1+1
830 P3(I) = MULTPLX(C1)*CALO @ C1 = C1+1
840 P4(I) = MULTPLX(C1)*CALO @ C1 = C1+1
850 P5(I) = MULTPLX(C1)*CALO @ C1 = C1+1
860 P6(I) = MULTPLX(C1)*CALO @ C1 = C1+1
870 P7(I) = MULTPLX(C1)*CALO @ C1 = C1+1
880 P8(I) = MULTPLX(C1)*CALO @ C1 = C1+1
890 NEXT I
900 C1 = NDP*10+3
910 FOR I = 1 TO NDP
920 P9(I) = MULTPLX(C1)*CALO @ C1 = C1+1
930 P10(I) = MULTPLX(C1)*CALO @ C1 = C1+1
940 P11(I) = MULTPLX(C1)*CALO @ C1 = C1+1
130

950   P12(I)=MULTPLX(C1)*CALO @C1=C1+1
960   P13(I)=MULTPLX(C1)*CALO @C1=C1+1
970   P14(I)=MULTPLX(C1)*CALO @C1=C1+1
980   NEXT I
990   N$="NO" @TL$="TOP L" @BL$="BOTTOM L" @P1$=
   "PT1" @P2$="PT2" @P3$="PT3" @P4$="PT4" @P5$=
   "PT5" @P6$="PT6"
1000  PRINT USING 1010; N$,TL$,BL$,P1$,P2$,P3$,P4$,P5$,P6$
1010  IMAGE 4A,10A,10A,10A,10A,9A,9A,9A
1020  FOR I=1 TO NDP
1030  PRINT USING 1040;I,TL(I),BL(I),P1(I),P2(I),P3(I),P4(I),P5(I),P6(I)
1040  IMAGE 2D,2X,SD.SD,2X,SD.SD,2X,SD.SD,2X,SD.SD,2X,SD.SD,1X,SD.SD,1X,SD.SD,1X,SD.SD,1X,SD.SD
1050  NEXT I
1060  N$="NO" @P7$="PT7" @P8$="PT8" @P9$="PT9" @P10$="PT10" @P11$="PT11" @P12$="PT12" @LV1$="R. DEF" @LV2$="H. DEF"
1070  PRINT USING 1080; N$,P7$,P8$,P9$,P10$,P11$,P12$,LV1$,LV2$
1080  IMAGE 4A,10A,10A,9A,9A,9A,9A,10A,10A
1090  FOR I=1 TO NDP
1100  PRINT USING 1110;I,P7(I),P8(I),P9(I),P10(I),P11(I),P12(I),P13(I),P14(I)
1110  IMAGE 2D,2X,SD.SD,2X,SD.SD,1X,SD.SD,1X,SD.SD,1X,SD.SD,1X,SD.SD
1120  NEXT I
1130  REM *************** AVERAGE READING***************
1140  TL(0)=0 @BL(0)=0 @P1(0)=0 @P2(0)=0 @P3(0)=0,
   P4(0)=0 @P5(0)=0 @P6(0)=0 @P7(0)=0 @P8(0)=0 @P9
   (0)=0 @P10(0)=0 @P11(0)=0 @P12(0)=0 @P13(0)=0
   @P14(0)=0
1150  FOR I=1 TO NDP
1160  TL(I)=TL(I-1)+TL(I)
1170  BL(I)=BL(I-1)+BL(I)
1180  P1(I)=P1(I-1)+P1(I)
1190  P2(I)=P2(I-1)+P2(I)
1200  P3(I)=P3(I-1)+P3(I)
1210  P4(I)=P4(I-1)+P4(I)
1220  P4(I)=P4(I-1)+P4(I)
1230  P5(I)=P5(I-1)+P5(I)
1240  P6(I)=P6(I-1)+P6(I)
1250  P7(I)=P7(I-1)+P7(I)
1260  P8(I)=P8(I-1)+P8(I)
1270  P9(I)=P9(I-1)+P9(I)
1280  P10(I)=P10(I-1)+P10(I)
1290  P11(I)=P11(I-1)+P11(I)
1300  P12(I)=P12(I-1)+P12(I)
1310  P13(I)=P13(I-1)+P13(I)
1320  P14(I)=P14(I-1)+P14(I)
1330  NEXT I
1340  TLAVG(J)=TL(NDP)/NDP
1350  BLAVG(J)=BL(NDP)/NDP
<table>
<thead>
<tr>
<th>Line</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>1360</td>
<td>[ P_{1AVG}(J) = \frac{P_{1}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1370</td>
<td>[ P_{2AVG}(J) = \frac{P_{2}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1380</td>
<td>[ P_{3AVG}(J) = \frac{P_{3}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1390</td>
<td>[ P_{4AVG}(J) = \frac{P_{4}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1400</td>
<td>[ P_{5AVG}(J) = \frac{P_{5}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1410</td>
<td>[ P_{6AVG}(J) = \frac{P_{6}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1420</td>
<td>[ P_{7AVG}(J) = \frac{P_{7}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1430</td>
<td>[ P_{8AVG}(J) = \frac{P_{8}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1440</td>
<td>[ P_{9AVG}(J) = \frac{P_{9}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1450</td>
<td>[ P_{10AVG}(J) = \frac{P_{10}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1460</td>
<td>[ P_{11AVG}(J) = \frac{P_{11}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1470</td>
<td>[ P_{12AVG}(J) = \frac{P_{12}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1480</td>
<td>[ P_{13AVG}(J) = \frac{P_{13}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1490</td>
<td>[ P_{14AVG}(J) = \frac{P_{14}(NDP)}{NDP} ]</td>
</tr>
<tr>
<td>1500</td>
<td>REM ************ STORE DATA ***************</td>
</tr>
<tr>
<td>1510</td>
<td>PRINT # 1; J, D(J), TLAVG(J), BLAVG(J), P_{1AVG}(J), P_{2AVG}(J), P_{3AVG}(J), P_{4AVG}(J), P_{5AVG}(J), P_{6AVG}(J), P_{7AVG}(J), P_{8AVG}(J), P_{9AVG}(J), P_{10AVG}(J), P_{11AVG}(J), P_{12AVG}(J), P_{13AVG}(J), P_{14AVG}(J)</td>
</tr>
<tr>
<td>1520</td>
<td>NEXT J</td>
</tr>
<tr>
<td>1530</td>
<td>ASSIGN # 1 TO *</td>
</tr>
<tr>
<td>1540</td>
<td>PRINT; &quot; &quot;</td>
</tr>
<tr>
<td>1550</td>
<td>N$ = &quot;NO&quot; @ D$ = &quot;DEPTH&quot; @ TL$ = &quot;TOP L&quot; @ BL$ = &quot;BOTTOM&quot; @ P1$ = &quot;PT1&quot; @ P2$ = &quot;PT2&quot; @ P3$ = &quot;PT3&quot; @ P4$ = &quot;PT4&quot; @ P5$ = &quot;PT5&quot; @ P6$ = &quot;PT6&quot;</td>
</tr>
<tr>
<td>1560</td>
<td>PRINT USING 1570; N$, D$, TL$, BL$, P1$, P2$, P3$, P4$, P5$, P6$</td>
</tr>
<tr>
<td>1570</td>
<td>IMAGE 4A, 8A, 8A, 8A, 8A, 8A, 8A, 8A, 8A, 8A</td>
</tr>
<tr>
<td>1580</td>
<td>FOR J = 1 TO NLR</td>
</tr>
<tr>
<td>1590</td>
<td>PRINT USING 1600; J, D(J), TLAVG(J), BLAVG(J), P_{1AVG}(J), P_{2AVG}(J), P_{3AVG}(J), P_{4AVG}(J), P_{5AVG}(J), P_{6AVG}(J), IMAGE 2D, 2X, 6D, 1X, S. 5D, 1X, S. 5D, 1X, S. 5D, 1X, S. 5D, 1X, S. 5D</td>
</tr>
<tr>
<td>1610</td>
<td>NEXT J</td>
</tr>
<tr>
<td>1620</td>
<td>N$ = &quot;DEPTH&quot; @ P7$ = &quot;PT7&quot; @ P8$ = &quot;PT8&quot; @ P9$ = &quot;PT9&quot; @ P10$ = &quot;PT10&quot; @ P11$ = &quot;PT11&quot; @ P12$ = &quot;PT12&quot; @ LV1$ = &quot;R.DEF&quot; @ LV2$ = &quot;H.DEF&quot;</td>
</tr>
<tr>
<td>1630</td>
<td>PRINT USING 1640; N$, P7$, P8$, P9$, P10$, P11$, P12$, LV1$, LV2$</td>
</tr>
<tr>
<td>1650</td>
<td>FOR I = 1 TO NLR</td>
</tr>
<tr>
<td>1660</td>
<td>PRINT USING 1670; D(I), P_{7AVG}(I), P_{8AVG}(I), P_{9AVG}(I), P_{10AVG}(I), P_{11AVG}(I), P_{12AVG}(I), P_{13AVG}(I), P_{14AVG}(I), IMAGE 2D, 2X, S. 5D, 2X, S. 5D, 1X, S. 5D, 1X, S. 5D, 1X, S. 5D</td>
</tr>
<tr>
<td>1680</td>
<td>NEXT I</td>
</tr>
<tr>
<td>1690</td>
<td>END</td>
</tr>
</tbody>
</table>
10 REM ****** PROGRAM TO DRAW LATERAL PRESSURE ******
20 DIM BY(9,100),BZ(9,100),E(9),EZ(600),CY(600),
30 PLOTTER IS 505
40 DISP "IF YOU DON'T WANT TO DRAW FRAME,LIFT PEN UP"
50 PAUSE
60 DISP "PRESS (CONTINUE) TO CONTINUE"
70 LOCATE 20,120,20,90
80 SCALE -200,200,900,0
90 DEG
100 FXD 0,0
110 LGRID -10,50,0,900,10,4
120 MOVE -50,990
130 LDIR 0
140 CSIZE 3
150 LABEL "SOIL PRESSURE (kPa)"
160 MOVE -260,450
170 LDIR 90
180 LABEL "DEPTH (mm)"
190 MOVE 60,-30
200 LDIR 0
210 LABEL "B=102 mm"
220 MOVE 150,-30
230 LDIR 0
240 LABEL "0=30"
250 MOVE 150,-30
260 LDIR 0
270 LABEL "-=30"
280 MOVE -40,-30
290 LDIR 0
300 LABEL "D=900 mm"
310 FRAME
320 DISP "IF YOU WANT TO PLOT DATA,TAKE PEN DOWN"
330 PAUSE
340 DISP "PRESS (CONTINUE) TO CONTINUE"
350 DISP "ENTER NUMBER OF DATA (N)"
360 INPUT N
370 FOR J=1 TO N
380 DISP "ENTER DATA POINTS (X(J) in kPa, Y(J) in mm)"
390 INPUT X(J),Y(J)
400 NEXT J
410 FOR I=1 TO N-1
420 H(I)=X(I+1)-X(I)
430 F(I)=Y(I+1)-Y(I)
440 NEXT I
450 FOR I=2 TO N-1
460 P(I)=(F(I)/H(I)-F(I-1)/H(I-1))*6
470 NEXT I
480 P(1)=0 @P(N)=0
490 FOR I=1 TO N
500 FOR J=1 TO N
510 G(I,J)=0
133

BEGIN

FOR I=2 TO N-1
  G(I, I)=2*(H(I-1)+H(I))
  G(I, I-1)=H(I-1)
  G(I, I+1)=H(I)
  NEXT I
  G(1, 1)=H(2)
  G(1, 2)=-(H(1)+H(2))
  G(1, 3)=H(1)
  G(N, N-2)=H(N-1)
  G(N, N-1)=-(H(N-2)+H(N-1))
  G(N, N)=H(N-2)
REM ******** OBTAIN THE INVERSION MATRIX AND ********
********** THE VALUE OF Y ******************
FOR K=1 TO N
  FOR J=1 TO N
    IF J=K THEN GOTO 700
    G(K, J)=G(K, J)/G(K, K)
    NEXT J
  G(K, K)=1/G(K, K)
  FOR I=1 TO N
    IF I=K THEN GOTO 780
    FOR J=1 TO N
      IF J=K THEN GOTO 770
      G(I, J)=G(I, J)-G(K, J)*G(I, K)
    NEXT J
    FOR J=1 TO N
      IF J=K THEN GOTO 770
      G(I, J)=G(I, J)-G(K, J)*G(I, K)
    NEXT I
  NEXT K
  FOR I=1 TO N
    E(I)=0
    FOR J=1 TO N
      E(I)=E(I)+G(I, J)*(P(J)
    NEXT J
    NEXT I
  FOR I=1 TO N
    PRINT; E(I)
  NEXT I
REM ********** FIND THE COEFFICIENTS A,B,C,AND D ***
********** FOR EACH INTERVALS **********
FOR I=1 TO N-1
  B(I)=E(I)/2
  A(I)=(E(I+1)-E(I))/(6*H(I))
  C(I)=P(I)/H(I)-(2*H(I)*E(I)+H(I)*E(I+1))/6
  D(I)=Y(I)
  NEXT I
PRINT; "GIVEN DATA"
PRINT; "DEPTH (mm)", "SOIL PRESSURE (kPa)"
FOR I=1 TO N
  PRINT X(I), Y(I)
NEXT I
PRINT; " "
FOR I=1 TO N-1
SUMA=0
Z=X(I)
M=20
DZ=H(I)/M
FOR J=1 TO M
AY=A(I)*(Z-X(I))*2+B(I)*(Z-X(I))+C(I)*(Z-X(I))+D(I)
BY(I,J)=AY
BZ(I,J)=Z
Z=Z+DZ
NEXT J
NEXT I
PRINT ;" "
THE CALCULATED DATA
PRINT ;"DEPTH (mm)" , "SOIL PRESSURE (kPa)"
k=1
FOR I=1 TO N-1
FOR J=1 TO M
CZ(I)=BZ(I,J)
CY(I)=BY(I,J)
FX(K)=CZ(I)
FY(K)=CY(I)
PRINT ;FX(K),FY(K)
K=K+1
NEXT J
NEXT I
REM ****** PLOT DATA ON THE PLOTTER **********
DISP "ENTER CHARACTER STRING"
INPUT C$
LORG 5
FOR I=1 TO M*(N-1)
PLOT FY(I),FX(I)
NEXT I
DISP "DO YOU WANT TO PUT DATA POINTS ON ?
(YES/NO)"
INPUT PL$
IF PL$="YES" THEN GOTO 1420 ELSE GOTO 1460
DISP "ENTER DATA POINTS (DEPTH, PRESSURE)"
INPUT X,Y
MOVE Y,X @LABEL C$
GOTO 1390
DISP "DO YOU WANT TO LABEL ? (YES/NO)"
INPUT YN$
IF YN$="YES" THEN GOTO 1490 ELSE GOTO 1610
DISP "INPUT COORDINATES AT CENTER OF LABEL; X,Y"
INPUT X,Y
MOVE X,Y
DISP "INPUT LABEL DIRECTION IN DEGREE"
INPUT D
DEG @LDIR D
DISP "INPUT LABEL"
1560 INPUT L$
1570 DISP "INPUT CHARACTER SIZE"
1580 INPUT S
1590 CSIZE S
1600 LABEL L$
1610 DISP "DO YOU WANT TO LABEL MORE? (YES/NO)"
1620 INPUT YO$
1630 IF YO$="YES" THEN GOTO 1490 ELSE GOTO 1640
1640 DISP "DO YOU WANT TO GET ANOTHER DATA FILE? (YES/NO)"
1650 INPUT YE$
1660 IF YE$="YES" THEN GOTO 350 ELSE GOTO 1670
1670 DUMP GRAPHICS
1680 END