CENTRIFUGE MODELING OF UPLIFT BEHAVIOUR OF TAPERED PILES
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ABSTRACT
The uplift performance of tapered piles has not been fully understood. The objective of the current study is to evaluate the uplift performance of tapered piles relative to the uplift performance of straight-sided wall piles. The uplift performance of tapered piles was investigated using centrifuge model tests. Twelve one tenth-scale model tapered and cylindrical piles were installed in sand and subjected to axial loading with a load path featuring axial compression prior to uplift. Six piles were instrumented and six piles were not. The results of loading tests were presented and discussed. The results showed that the uplift capacity of tapered piles was slightly higher than the cylindrical pile capacity of the same average diameter and length. Also, the stiffness of tapered piles within the range of the design load was equal to or higher than that for cylindrical piles. This suggested that the performance of the tapered piles in the uplift mode is better than or comparable to the straight-sided wall piles.

1. INTRODUCTION
Pile foundations are generally used to support compressive loads from superstructures. Some structures such as transmission towers, tall chimneys, and jetties are constructed on pile foundations that have to resist uplift loads. Also, piles installed in expansive soils are subjected to significant uplift loads. In these situations, the uplift performance of the piles may become the governing factor for the design.

Several small model laboratory and centrifuge tests conducted on cylindrical piles with different slenderness ratios were reported in the literature (Awad and Ayoub 1976; Das 1983; Levacher and Sieffert 1984; Nunez et al. 1988). Das (1983) observed that the average skin friction reaches a constant value after a certain value of slenderness ratio and this value is dependent on the relative density of the sand. Levacher and Sieffert (1984) reported uplift test results on piles installed using different techniques such as high frequency vibro-driving and driving methods. Their results showed that the installation method has an important effect on the uplift capacity. Nunez et al. (1988) investigated the response of piles driven into sand and subjected to uplift loading in a centrifuge facility. Their results showed that the skin friction increased approximately in proportion to the depth of embedment.

Tapered piles such as MonoTube piles, used in United States and Canada, have a substantial advantage with regard to their compressive capacity. El Naggar and Sakr (2000) conducted an experimental study on the axial performance of tapered piles in compression. Their results showed that using a taper angle of about 1˚, can result in an increase in the skin friction up to 175% of that of cylindrical piles in compression mode. Several installation methods for tapered piles can be used such as jacking or driving, or pre-boring, augering or jetting when hard driving is expected as the case of driving in dense sand. Tapered piles can be either fabricated from steel tubes (MonoTube type), wooden piles, or pre-cast concrete piles. In practice, tapered piles (or tapered segments in the MonoTube piles) vary in length between 6 m and 12 m, with diameters varying between 200 mm to 350 mm and taper angles varying between 0.2˚ and 1.0˚. The use of tapered piles as a design option in foundations subjected to uplift loads necessitates exploring the uplift performance of tapered piles. El Naggar and Wei (2000) investigated the uplift behaviour of tapered piles using model piles in a laboratory setup. They found that the uplift performance of tapered piles is comparable to that of cylindrical piles.

2. SCOPE OF WORK
The objective of the current study is to evaluate the uplift performance of tapered piles relative to the uplift performance of straight-sided wall piles. For this
purpose, a centrifuge experimental investigation into the uplift performance of cylindrical and tapered piles was conducted. The performance of piles was evaluated in terms of the load-displacement behaviour, ultimate uplift pile capacity, uplift stiffness, the ratio of uplift to compressive shaft capacity of the investigated piles, load distribution curves and unit skin friction curves.

3. TEST SETUP

Twelve one-tenth-scale piles were used in this study. Six piles were instrumented, three of them were tapered with taper angle \( \alpha \) equal to 0.35°, 0.54°, and 1.02°, and three were cylindrical piles with the same average diameter and length as the tapered piles. Six uninstrumented (blank) piles with the same configuration as the instrumented ones have been used as well. The purpose of testing blank piles is to quantify the roughness effect on the pile performance due to different loading modes (i.e. compression, uplift and cyclic loading). The prototype cylindrical piles were modeled using open-ended, cold drawn steel tubing with elastic modulus \( E_P = 2.15 \times 10^5 \) MPa, and a thickness of 0.88 mm. The tapered piles were fabricated from steel sheets with the same properties and approximately the same thickness. Table 1 summarizes the geometry of the piles.

Table 1 Geometry of tested piles.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Model Pile</th>
<th>g level</th>
<th>Prototype pile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( d_a )</td>
<td>( t )</td>
<td>( l )</td>
</tr>
<tr>
<td>1.1 T1b</td>
<td>27.8</td>
<td>0.9</td>
<td>761.5</td>
</tr>
<tr>
<td>1.2 T1a</td>
<td>29.6</td>
<td>1.8</td>
<td>762.0</td>
</tr>
<tr>
<td>1.3 S1b</td>
<td>25.5</td>
<td>0.9</td>
<td>763.0</td>
</tr>
<tr>
<td>1.4 S1a</td>
<td>26.9</td>
<td>1.6</td>
<td>764.0</td>
</tr>
<tr>
<td>2.1 T2b</td>
<td>27.2</td>
<td>0.9</td>
<td>516.0</td>
</tr>
<tr>
<td>2.2 T2a</td>
<td>28.9</td>
<td>1.8</td>
<td>516.5</td>
</tr>
<tr>
<td>2.3 S2b</td>
<td>25.5</td>
<td>0.9</td>
<td>518.0</td>
</tr>
<tr>
<td>2.4 S2a</td>
<td>27.3</td>
<td>1.8</td>
<td>518.0</td>
</tr>
<tr>
<td>3.1 T3b</td>
<td>29.9</td>
<td>0.9</td>
<td>455.0</td>
</tr>
<tr>
<td>3.2 T3a</td>
<td>31.1</td>
<td>1.5</td>
<td>454.0</td>
</tr>
<tr>
<td>3.3 S3b</td>
<td>28.5</td>
<td>0.9</td>
<td>452.0</td>
</tr>
<tr>
<td>3.4 S3a</td>
<td>30.4</td>
<td>1.5</td>
<td>453.0</td>
</tr>
</tbody>
</table>

\( T \) denotes tapered pile; \( S \) denotes cylindrical pile
\( a \) denotes instrumented pile; \( b \) denotes blank pile
\( d_a \) average outside diameter of the pile shaft
\( t \) overall thickness of pile wall; \( l \) pile length; \( \alpha \) taper angle

The instrumentation of the piles consisted of eight levels of strain gauges attached to the external surface of the model pile. Two pairs of 2-element 90° rosette strain gauges were affixed at each level, constituting a full bridge, to measure the axial stresses along the pile shaft. The strain gauges were distributed over the length of the piles such that the first bridge was at 20 mm from the pile head, above the sand surface, to compare the load measured from both the load cell and the strain gauges. The rest of the strain gauges were distributed equally over the pile embedded length. They were connected to the external wire lead using solid copper wire (136-AWP). Only instrumented piles were coated using two layers of AE-10 epoxy and fine sand of 0.15 mm in diameter was sprinkled over the second coating layer in order to ensure rough pile-soil interface. The strain gauges and their leads were covered by the epoxy coating.

Three packages were prepared for centrifuge testing. Each package consisted of steel tub with a diameter of 904 mm and height of 915 mm. The sand was rained into this tub to height of 850 mm. Each package contained four piles. They were positioned at a circumference of circle of 450 mm in diameter so that the minimum distance between any two piles was 318 mm (12 pile diameters) and the minimum distance between the pile and the steel wall of the tub was 225 mm (9 pile diameters). This arrangement insured minimum interaction and boundary effects (El Naggar and Sakr 1999).

The soil used in the tests consisted of medium angular dried sand. The sand was well graded Al white Silica, with particle sizes in the range of 0.075 to 0.59mm. Sand samples were prepared using a raining (pluviation) technique to provide a relatively homogenous specimen with the desired relative density. The relative density was 25.5 ± 2.5% to model loose sand. The relative density after flight increased to 35%.

The loading setup consisted of electrical step motor jack (cone driver) with a maximum capacity of 10 kN (1000 kN in prototype) with a minimum step of 0.005mm per second to control the rate of movement. Seven linear displacement transducers were distributed over the pile heads and the sand surface in order to monitor the surface movement. Figure 1 shows the testing setup and the loading system.

![Figure 1 Oblique view for the test setup.](image-url)
3.1 Testing Procedure

For clarity, the description of the testing procedures and results in this paper will be based on a prototype scale provided that all similarity conditions were achieved. The axial compressive loading tests were performed on the piles during the flight at 10 g with a constant rate of penetration of 0.5mm per second to a maximum settlement of 270mm. After completion of the compressive loading, the upward movement with the same rate was started. The upward movement was continued till the initial position of the pile was attained. All the instrumentation was reset to zero at the beginning of compressive loading tests and the data was recorded for both compressive and tensile loading tests. This was done to facilitate tracing the residual stresses mobilized along the pile shaft and at the tip due to the compressive loading tests. It is assumed that the piles were free from residual stresses when initially tested in compression since the piles were jacked at 1g and then the loading procedures were conducted at 10g. However, the residual stresses developed during the compressive loading phase were large and would influence the pile performance during the subsequent unloading and uplift tests. The loads developed in the pile at different levels were monitored during all loading stages. The load measured was used to assess the residual loads and to correctly interpret the results of the uplift loading stage.

Since the data was measured with reference to the initial state right before the start of the compressive loading tests, it is important to distinguish between the different loading phases. Figure 2 shows schematically the different loading phases. Phase (a) represents the compressive loading test, in which the pile is moving downward and therefore, shear stresses along the pile shaft and compressive tip resistance are mobilized. The upward movement starts at phase (b) and the shear stresses around the pile shaft start to reverse themselves. The developed tip residual load during the compressive phase is relieved and pushes the pile upward. During phase (c) the load cell reading at the pile head is either less than the residual load at the tip or shows an explicit tensile load and all the stresses along the pile shaft are reversed (tensile stresses). Therefore, a rational estimate of residual loads developed at different levels along the pile shaft and at the pile tip is essential for interpreting the uplift data results.

3.2 Residual Stresses

The existence of residual stresses in driven piles due to installation or loading in compression has been recognized for a long time. Poulos and Davis (1980) emphasized the importance of incorporating the evaluation of residual stresses into the analysis of driven piles. Briaud and Tucker (1984) examined data for driven piles in sand and suggested a simplified method for estimating the residual load in these piles. Poulos (1987) suggested procedures for estimating the residual stresses due to jacking or driving processes based on a simplified boundary element analysis in which the pile is loaded to failure in compression and then unloaded to zero. Poulos (1987) concluded that the effect of residual stresses is significant for piles in sand.

3.2.1 Residual Stresses in Model Piles Due to Installation

The instrumentation was set to zero at the beginning of compressive loading in flight. The measured point loads (tip resistance) are those corresponding to that part mobilized by the compressive loading test itself, neglecting the point load mobilized due to the installation process (at 1g). The existence of residual stresses due to the installation process is recognized but these stresses have minor effects on the compressive pile response. The open-ended pile model results in a small tip resistance since the area of the pile tip is relatively small (about 9-16 % of the pile cross-section). Furthermore, the pile was jacked at 1g and the driving force was small (1% of the driving force required for the prototype pile). Therefore, the residual loads and stresses developed due to the installation process would have been small and can be neglected. The main source of residual loads and stresses in uplift tests would have been that resulted from in-flight compressive loading.

3.2.2 Estimating the Residual Stresses

The approach proposed by Briaud and Tucker (1984) was used in this study to estimate the residual forces due to the compressive loading. This approach, besides its simplicity, incorporates rational pile response parameters into the solution such as the ultimate compressive point
load and total loads, the pile length, and the relative pile-soil stiffness. These quantities can be obtained readily from the load test data. The solution involved the following assumptions:

1. The analysis considers the state of stress and load distribution of the compressive loading at failure as shown in Fig. 3a. The ultimate load at the pile head is $Q_{Tu}$; the ultimate load at the pile point is $Q_{Pu}$, and the load anywhere along the pile's embedded depth is $Q_{zu}$. The ultimate skin friction is $\tau_u$; and the ultimate point resistance is $q_u$.

2. The unloading of the stress along the pile shaft and at the pile tip is assumed to obey the linear elastic model, i.e.

$$\Delta \tau = K'_\tau \Delta w$$  \[1a\]

$$\Delta q = K'_p \Delta w_p$$  \[1b\]

where $\Delta \tau$ is the decrease in pile-soil friction stress at depth $z$, $K'_\tau$ is the unloading stiffness in friction, which is defined as the slope of the unloading stage for shaft resistance-settlement curve (Fig. 3b), and $\Delta w$ is the upward movement of the pile at depth $z$ upon unloading. $\Delta q$ is the decrease in point resistance, $K'_p$ is the unloading stiffness for the point, which is defined as the slope of the unloading stage for tip resistance-settlement curve, and $\Delta w_p$ is the upward movement of the point upon unloading. These parameters are shown schematically in Figs. 3b,c.

![Figure 3](image)

**Figure 3** Unloading process and required parameters for calculations of residual loads.

Based on the equilibrium of an elementary pile element, the constitutive equation for the pile was drawn and the solution for this equation was attained as (Briaud and Tucker, 1984):

$$Q_{Rz} = Q_z - Q_{zu} \left( \frac{E_p \Omega + K_p}{E_p \Omega + K_p} \right)^{\frac{1}{\alpha + 1}} - \left( \frac{E_p \Omega + K_p}{E_p \Omega + K_p} \right)^{\frac{1}{\alpha + 1}}$$  \[2a\]

$$\Omega = \sqrt{K_p \rho / E_p A}$$  \[2b\]

where $Q_z$ is the residual load along the pile at depth $z$; $E_p$ is the pile modulus of elasticity; $l$ is the pile length; $z$ is the embedded depth; $p$ is the pile perimeter and $A$ is the cross-sectional area of the pile.

### 3.2.3 Interpretation of Uplift Test Data

The data obtained from the data acquisition system includes the compressive loading data (phase a, i.e. from A to B on Fig. 4), the unloading data (phase b, i.e. from B to C on Fig. 4) and uplift data (phase c, i.e. from C to A which includes a peak at A' as shown in Fig. 4). It is important to decouple phase b from phase c. The beginning of the uplift loading was defined by the following observations:

1. A change in the slope of the unloading-upward movement curve was observed. As an example, this behaviour is shown in Fig. 4 for pile T3a.

2. The load measurements at different stages (strain gauges and load cell) measured almost the same load along the pile at a certain instant of time. This means there was no load transfer along the shaft at that instant and the measured load at the point represented the residual load. This specific moment could be considered the end of the unloading stage and the onset of the uplift loading.

3. The point load measured at that specific instant was in good agreement with the residual load calculated using Eq. 2, especially for cylindrical piles.

![Figure 4](image)

**Figure 4** Assessment of tip residual load from pile unloading data.

Based on these observations, it was possible to decouple the unloading and uplift stages and define the beginning of the uplift loading (point C in Fig. 4). The former observations were of great importance especially for blank piles, where there were no means of measuring the tip resistance and therefore using Eq. 2 to evaluate the residual loads was impossible. The load-displacement curves for uplift tests, load distribution along the pile, and unit skin friction curves were plotted accordingly.
4. TESTING RESULTS

4.1 Uplift Load-Displacement Curves

The load applied at the pile head and the upward movements were monitored simultaneously during the unloading and uplift tests. The results for the uplift loading were interpreted accounting for the residual stresses, employing the approach proposed by Briaud and Tucker (1984) as described previously. The results are presented in three groups: (a) group one includes the results for piles T1a and T1b with a taper angle of 0.35° and piles S1a and S1b with a diameter equal to the average embedded diameter of the tapered piles; (b) group two includes the results for piles T2a, T2b with a taper angle of 0.54° and piles S2a and S2b; (c) group three includes the results of piles T3a, T3b with a taper angle of 1.02° and piles S3a and S3b.

![Figure 5: Load displacement curves for: (a) instrumented piles S3a and T3a; and (b) blank piles S3b and T3b.](image)

As an example, the results for the third group of tests are plotted in Fig. 5 in terms of loads applied at the pile head versus the displacement ratio defined as the ratio of the pile head upward movement to the average pile diameter. Figure 5 shows that the response of tapered piles was slightly stiffer than the response of cylindrical piles right from the beginning of the uplift loading and that the tapered pile offered a higher uplift resistance than the cylindrical one at higher displacement levels. However, this trend seemed to be transient as the applied tensile load at the pile head decreased approaching an asymptote close to the load of the cylindrical pile at higher displacement values. This may be attributed to loosening of the sand around tapered piles at high upward displacements revising the densification occurred during the compressive loading phase. Similar observations could be made for the other groups of test.

4.2 Ultimate Uplift Pile Capacity

Kulhawy and Hirany (1989) reviewed the existing interpretation methods for axial uplift pile load tests and recommended that the load corresponding to a displacement equal to 12.7 mm is interpreted as the “failure” load. De Nicola and Randolph (1999) defined the ultimate tensile pile capacity as the pile head load corresponding to a pile head displacement of 5% of the pile diameter. The average diameter of piles tested in the current study was approximately 270 mm. Therefore, the failure criterion will be defined based on 5% displacement of the pile diameter, which represents 13.5 mm.

Table 2 shows the uplift capacity of the piles, $Q_{up}$, based on the load corresponding to 5% of the average pile diameter. The uplift capacity ratio, $K_V$, was used for the comparison of the uplift capacity of tapered and cylindrical piles. The uplift capacity ratio is defined as the ratio of the uplift capacity per surface area of the tapered pile to that of the equivalent cylindrical pile. The $K_V$ values were higher than 1.0 for all piles meaning that tapered pile had higher uplift capacities. This could be attributed to the densification of the sand around the piles due to the wedging effect of tapered piles during the preceding compression tests. Accordingly, the friction angle in the vicinity of the tapered pile was increased (see Nordlund 1963).

Table 2 also shows that the uplift capacity of instrumented piles (with rough surface) was higher than the corresponding blank piles (with smooth surface). This...
observation may be explained for rough piles as the shearing mechanism for rough surfaces involves dilation of the sand surrounding the pile which result in an increased shearing resistance, but for smooth piles, the shearing is confined to a very thin layer of soil and no dilatational effects

4.3 Load Distribution Curves

The forces transmitted at different locations along the pile shaft were obtained directly from the strain gauge measurements. The existence of residual loads in the soil along the shaft interface and at the tip has been discussed and the maximum residual loads have been assessed. As the uplift test progressed and the pile moved upward, the locked in residual stresses started to relieve. The residual loads at different levels were calculated using Eq. 2, which considers the residual load distribution at “failure”. On the other hand, the load-displacement curves were obtained by assigning the origin of the uplift loading to coincide with the beginning of phase (c). The only fact known is that the load measured at the pile tip during the uplift loading tests should be equal to zero. Therefore, a correction factor, $F_c$, was used to correct the residual loads at different locations along the pile and at the pile tip calculated by means of Eq. 2. The correction factor $F_c$ is defined as the ratio of the measured tip resistance at the ultimate uplift pile capacity and the calculated residual load at the pile tip, i.e.

$$ F_c = \frac{Q_{Rt_{\text{measured}}}}{Q_{Rt_{\text{calculated}}}} \quad [3] $$

where $Q_{Rt_{\text{calculated}}}$ is the calculated residual load at the pile tip and $(Q_{Rt_{\text{measured}}})$ is the measured tip resistance. Then the residual loads along the pile shaft were corrected and used to infer the load distribution curve along the pile shaft.

Figure 6 shows the load distribution curves for cylindrical pile S3a and tapered pile T3a at one third, two thirds and the full value of the ultimate load of the cylindrical pile, $Q_{S3a_{up}}$. The load distribution curve at the ultimate load of tapered pile, $Q_{T3a_{up}}$, is also shown. It may be observed from Fig. 6 that the general trend of the load distribution was almost the same for both cylindrical and tapered piles at different loading increments, although tapered piles offered a slightly higher load distribution along the shaft.

![Figure 6 Load distribution curves along pile shaft for S3a and T3a.](image)

4.4 Unit Skin Friction

The unit skin friction of the piles in the uplift mode was calculated at the ultimate uplift capacity. The difference between the corrected values of the load at two stations minus the corresponding weight of the pile was divided by the surface area of the pile between the two stations. The result represents the average unit skin friction between these two stations, i.e.

$$ \tau_u = \frac{(q_i - Q_{Rc_i}) - (q_j - Q_{Rc_j}) + W_i}{S_p} \quad [4] $$

where the subscripts $i$ and $j$ denote the stations $i$ and $j$, $q$ is the measured load reading; $W_i$ is the self weight of the pile between stations $i$ and $j$ and $Q_{Rc_i}$ is the corrected residual load at station $i$, i.e. $Q_{Rc_i} = F_c Q_{Ri}$; and $Q_{Ri}$ is the residual load at different levels calculated using Eq. 2.

Figure 7 shows the unit skin friction for: a) piles S1a and T1a, b) piles S2a and T2a, and c) piles S3a and T3a. It is noted from Fig. 7 that the tapered piles offered a slightly higher skin friction in the uplift mode than the corresponding cylindrical piles. For cohesionless soils, the shaft resistance can be calculated by

$$ Q_{wp} = \int_0^h \tau_s p dz = \int_0^h \beta \sigma' p dz \quad [5] $$

where $\tau_s$ is the shear stress along the shaft that can be calculated at different levels using Eq. 4; $p$ is the pile perimeter; $\beta$ is a combined shaft resistance factor and $\sigma'$ is the effective overburden stress. The shaft resistance factor $\beta$ is influenced by the angle of internal friction of the soil, $\phi$, the frictional angle of pile-soil interface, $\delta$, the method of installation, the original state of stress in the ground, and the material, size and shape of the pile. The coefficient $\beta$ for uniform cross-section piles
in tension generally ranges from 0.1 to 0.75 (see Canadian Foundation Engineering Manual 1992).

The value of $\beta$ was deduced from the values of the unit skin friction that were established from the experimental results. The coefficient $\beta$ was calculated as

$$\beta = \frac{\tau_s}{\sigma_v}$$  \hspace{1cm} [6]

The average shear stress along the shaft, $\tau_s$, was determined at different levels using Eq. 4. The skin friction increased approximately proportionally to depth as shown in Fig. 8. It increased at a rate of 4.40, 7.24, and 9.66 kPa/m for cylindrical piles S1a, S2a, and S3a, and at a rate of 4.71, 8.88, and 11.31 for tapered piles T1a, T2a, and T3a, respectively. Assuming that the vertical stress increased linearly with depth, the combined shaft resistance factor $\beta$ was then obtained using Eq. 6. The calculated $\beta$ values were 0.326, 0.539, and 0.714 for piles S1a, S2a, and S3a, respectively. The calculated $\beta$ values for tapered piles T1a, T2a, and T3a were 0.348, 0.657, and 0.832, respectively. The latter values were slightly higher than those for cylindrical piles.

![Figure 7](image1.png)

**Figure 7** Skin friction curves for uplift loading tests for: (a) S1a and T1a; (b) S2a and T2a; and (c) S3a and T3a.

4.5 Compressive and Tensile Skin Friction

The comparison of unit skin friction at ultimate compressive (El Naggar and Sakr, 2000) and tensile loads is shown in Fig. 8. This comparison revealed that for cylindrical piles, both the compressive and tensile skin friction increased approximately linearly with depth and the tensile friction was lower than the compressive skin friction. De Nicola and Randolph (1999) reported similar observations. For tapered piles, the results showed that the unit skin friction in tension was significantly lower than the unit skin friction in compression. This increase in skin friction in compression in tapered piles may be attributed due to wedging effect and the substantial increase in relative density around the pile.

![Figure 8](image2.png)

**Figure 8** Comparison of unit skin friction at ultimate uplift and compressive capacity of instrumented piles.

The results presented in this study showed that the pile capacity in tension was much less than the pile capacity in compression. De Nicola and Randolph (1993) showed in their study that there were sound reasons for the lower ratio of tensile to compressive shaft capacity. They attributed these differences to the following reasons:

1. Poisson’s ratio expansion and contraction of the pile shaft led to changes in the radial effective stress field in the soil around the pile.

2. Differences in total stress field where compressive loading tended to increase and tensile loading tended to decrease the mean stress level in the soil.
Moreover, for tapered piles the change in the radial stresses is a function of the taper angle that will increase the compressive capacity significantly and reduce the tensile capacity as well. This could explain the much lower bearing ratios observed for tapered piles than the corresponding cylindrical piles.

5. CONCLUSIONS

An experimental investigation of the axial uplift response of twelve steel piles with different surface roughness conditions has been conducted in a centrifuge facility. The uplift performance characteristics of the piles were investigated and the following conclusions for load path featuring axial compressive prior to uplift were drawn:

1. The uplift capacity of tapered piles was slightly higher than the cylindrical pile capacity of the same average diameter and length. Also, the stiffness of tapered piles within the range of the design load was equal to or higher than for the cylindrical piles. This suggested that the performance of the tapered piles in the uplift mode is better than or comparable to the straight-sided wall piles.

2. The ratios of uplift to compressive capacity for straight-sided piles were less than unity and were similar to those obtained by other researchers. The load distribution patterns and skin friction values were also similar to the experimental results available in the literature. The ratio was significantly less for tapered piles. This ratio varied from 0.36 to 0.55 for instrumented tapered piles and from 0.44 to 0.53 for blank piles.

3. The uplift capacity of cylindrical piles with smooth surfaces is governed by the pile-soil friction angle, while the uplift capacity of piles with rough surfaces is governed by the change in the mean stress value and the dilatation effects. Therefore, the proper modeling of the uplift capacity requires consideration of both mechanisms and the incorporation of the surface roughness effects.

Residual stresses developed in the surrounding soil during the compressive loading mode had a great effect on the pile performance in the uplift-loading mode. A theoretical model for presenting the residual loads in tapered piles to account for the residual loads developed along the pile shaft due to wedging effect is still needed.

6. ACKNOWLEDGEMENTS

The authors would like to thank Dr. Ryan Philips Director of C-CORE for his guidance and support during this research program. This research was supported by a MFA grant from NSERC and an operating grant to the first author. Both sources of support are greatly appreciated.

7. References


