Performance of Pressure Grouted Helical Piles Under Monotonic Axial and Lateral Loading

Mohamed A. Mansour
The University of Western Ontario

Supervisor
El Naggar, M. Hesham
The University of Western Ontario

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Abstract

The pressure grouted helical pile (PGHP) is an innovative pile foundation system that allows a significant increase in the helical pile capacity with some additional cost. The pile is constructed by applying pressurized grout during the installation of a closed ended helical pile through two grout nozzles welded to the hollow pile shaft. Torquing PGHP into the ground allows the grout nozzles to create a cavity around the pile shaft. The cavity then expands under the effect of pressurized grout and helix rotation. This thesis presents a comprehensive laboratory study and three-dimensional finite element analysis to investigate the effects of the different testing conditions, including grout nozzle configurations, soil properties, and grouting pressure on the shape and performance of PGHPs under monotonic axial and lateral loading.

The experimental program includes the installation of five un-grouted helical piles and seventeen PGHPs in loose, medium, and dense sand with two different grouting pressures; 480 kPa and 690 kPa. The piles were then tested under monotonic uplift, compression, and lateral loading. The results reveal a significant increase in the PGHP’s shaft resistance over the un-grouted helical pile due to the formation of a large diameter grout column, enhanced friction angle at the pile-soil interface, and increased lateral earth pressure around the pile shaft as a result of the cavity expansion (Δr) that occurred during installation. The shape and diameter of the created grout column depend on the nozzle configuration, the relative density of the surrounding sand (R.D), and the grouting pressure used during construction (Pg). An increase in the unit end-bearing resistance is also observed due to the densification of the supporting soil and the permeation of some grout into its voids during the pile construction. Moreover, PGHPs offer a significant increase in the lateral pile capacity due to the larger shaft diameter and the existence of a disturbed soil zone around the un-grouted pile shaft.

A three-dimensional (3D) finite element model was developed using ABAQUS software. The model was calibrated and verified using the experimental data set. A strong relationship was observed between Δr, Pg, and R.D. Finally, two equations are proposed to calculate the placement method coefficient (kmo) for the design of PGHPs in order to account for the influence of the novel installation technique (i.e. cavity expansion) on the surrounding soil.
Keywords

Pressure grouted helical pile, laboratory pile load tests, finite element analysis, cavity expansion, radial stresses, axial loading, placement method coefficient, and lateral loading.
Lay audience summary

Pressure grouted helical pile (PGHP) is a new deep foundation system that involves grout injection under high pressure during the installation of a closed-ended helical pile with a hollow pipe shaft. The grout is injected into the surrounding soil through two grout nozzles welded to the pile shaft. Although PGHP is expected to be successful in many engineering applications with different soil conditions, it is not used in practice due to the lack of knowledge regarding the shape of the created grout column around the pile and its performance under different loading conditions.

A comprehensive investigation program was designed and implemented that included laboratory experiments and three-dimensional finite element (FE) modelling. The laboratory experiments comprised the installation of 5 small helical piles and 17 model PGHPs into cylindrical sand beds with different relative densities to represent loose, medium, and dense soil conditions. The PGHPs were installed with two different grouting pressures; 70 psi (480 kPa) and 100 psi (690 kPa). The piles were subjected to monotonic uplift, compression, and lateral load tests, then the PGHPs were extracted from the sand bed to provide a visual description of the created grout mass along their shafts. The pile load testing results revealed significant improvement in the axial and lateral resistances of PGHP over the conventional helical pile.

The commercial software ABAQUS (SIMULIA, 2013) was then used to simulate the laboratory experiments to further understand the load transfer mechanism and quantify the effects of the novel installation technique on the pile behavior. Following the calibration and validation of the created FE models with the experimental data, the models were used to analyze the PGHP performance under different testing (i.e. relative density and grouting pressure) and loading (uplift, compression, and lateral) conditions. The FE models were extended to simulate the response of full-scale PGHPs and full-scale conventional helical piles under monotonic compression loading considering different shaft and helix diameters. Finally, an approach was developed to allow practitioner engineers from estimating the PGHP capacity analytically.
Co-Authorship statement

This thesis is prepared in accordance with the regulations for a monograph format thesis stipulated by the school of graduate and post-doctoral studies at Western University. All the work presented herein was carried out by the author under the supervision of Prof. M. Hesham El Naggar. Part of the laboratory testing results presented in chapter five are published in two conference papers. Moreover, the following chapters are going to be submitted for possible publication in peer-reviewed journals;

**Chapters 4** and **6**: Performance characteristics of Pressure Grouted Helical Piles under monotonic uplift loading.

**Chapters 5** and **6**: Behavior of Pressure Grouted Helical Piles under monotonic compressive loading.

**Chapters 7**: Lateral monotonic performance of Pressure Grouted Helical Piles.
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All the love and gratitude to my adorable wife Mai. For the affection and care she provided, for believing in me when I had doubts and above all, for the fact that she was always there for me. I would also like to thank my son Malik for being the source of joy for the past year that kept my life in balance.

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List of abbreviations

PGDP  Pressure Grouted Displacement Pile
PGHP  Pressure Grouted Helical Pile
PGHP1 Pressure Grouted Helical Pile installed with the first nozzles configuration
PGHP2 Pressure Grouted Helical Pile installed with the second nozzles configuration
PGHP3 Pressure Grouted Helical Pile installed with the third nozzles configuration
RD    Relative density
OPC 10 Ordinary Portland Cement type 10
LVDT  Linear Variable Displacement Transducer
Pg i.j Grouted pile number (j) installed with nozzles configuration number (i)
Pung i Conventional (un-grouted) helical pile number (i)
FEM   Finite Element Model
3D    Three dimensional
ASB   Actual Soil Boundary
ESB   Extended Soil Boundary
α-bond Grout-ground bond of micropiles
HPM   Helical Pulldown Micropile
RHPM  Steel Reinforced Helical Pulldown Micropile
FRP-RHPM Fiber Reinforced Polymer - Steel Reinforced Helical Pulldown Micropile
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
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<tbody>
<tr>
<td>BWF</td>
<td>Beam on Winkler Foundation</td>
</tr>
<tr>
<td>API</td>
<td>American Petroleum Institute</td>
</tr>
<tr>
<td>APE</td>
<td>American Piledriving Equipment</td>
</tr>
<tr>
<td>USCS</td>
<td>Unified Soil Classification System</td>
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### List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
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<tbody>
<tr>
<td>S</td>
<td>Spacing between two successive helices</td>
</tr>
<tr>
<td>$D_h$</td>
<td>Helix diameter</td>
</tr>
<tr>
<td>$S/D_h$</td>
<td>Inter-helical spacing ratio</td>
</tr>
<tr>
<td>$S_L$</td>
<td>Limiting inter-helical spacing that ensures individual plate bearing response</td>
</tr>
<tr>
<td>N</td>
<td>No. of blows used for the compaction of each sand layer</td>
</tr>
<tr>
<td>RD</td>
<td>Relative density</td>
</tr>
<tr>
<td>SP</td>
<td>Poorly graded sand</td>
</tr>
<tr>
<td>$c_u$</td>
<td>Uniformity coefficient</td>
</tr>
<tr>
<td>$c_c$</td>
<td>Curvature coefficient</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Specific gravity</td>
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<td>$\gamma_{bulk}$</td>
<td>Bulk unit weight</td>
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<tr>
<td>$\varphi$ and $\varphi_p$</td>
<td>Peak angle of internal friction</td>
</tr>
<tr>
<td>$\varphi_{cs}$</td>
<td>Residual friction angle</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Friction angle at pile-soil interface</td>
</tr>
<tr>
<td>E</td>
<td>Elastic (Young’s) modulus of the soil</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Confining pressure</td>
</tr>
<tr>
<td>Pa</td>
<td>Atmospheric pressure</td>
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<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
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<td>--------</td>
<td>-------------------------------------------------</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>Maximum voids ratio</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>Minimum voids ratio</td>
</tr>
<tr>
<td>$P_g$</td>
<td>Grouting pressure</td>
</tr>
<tr>
<td>$T$</td>
<td>Thickness of grout ribs</td>
</tr>
<tr>
<td>$H$</td>
<td>Ribs height</td>
</tr>
<tr>
<td>$\sigma_v'$</td>
<td>Effective overburden pressure</td>
</tr>
<tr>
<td>$H$</td>
<td>Pile depth</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Inclination of plan of failure under uplift loading</td>
</tr>
<tr>
<td>$k_a$</td>
<td>Coefficient of active earth pressure</td>
</tr>
<tr>
<td>$k_o$</td>
<td>Coefficient of at-rest earth pressure</td>
</tr>
<tr>
<td>$k_p$</td>
<td>Coefficient of passive earth pressure</td>
</tr>
<tr>
<td>$k_p'$</td>
<td>Modified coefficient of passive earth pressure</td>
</tr>
<tr>
<td>$P_z$</td>
<td>Axial load at depth (z)</td>
</tr>
<tr>
<td>$E_p$</td>
<td>Elastic modulus of the pile</td>
</tr>
<tr>
<td>$D_{\text{shaft}}$</td>
<td>Shaft diameter</td>
</tr>
<tr>
<td>$D_{\text{toe}}$</td>
<td>Toe diameter</td>
</tr>
<tr>
<td>$D_{\text{eff}}$</td>
<td>Effective pile diameter</td>
</tr>
<tr>
<td>$S_f$</td>
<td>Failure displacement</td>
</tr>
<tr>
<td>$Q_u$</td>
<td>Pullout capacity</td>
</tr>
<tr>
<td>$Q_{\text{gr}}$</td>
<td>Pullout capacity of grouted piles</td>
</tr>
</tbody>
</table>

xxiv
\( Q_{\text{ung}} \) Pullout capacity of un-grouted helical piles

\( Q_s \) Friction resistance along the pile shaft

\( f_s \) Unit shaft resistance

\( A_s \) Surface area of pile shaft

\( A_p \) Cross-section area of pile shaft

\( w \) Pile weight

\( \sigma_r \) and \( \sigma_h \) Radial (lateral) stress around the pile

\( \beta \) Friction capacity factor

\( k_s \) Coefficient of lateral earth pressure around the pile shaft

\( k_{mo} \) Placement method coefficient

\( P_{\text{ult}} \) Ultimate compressive capacity

\( Q_b \) End-bearing resistance at the pile toe

\( f_b \) Unit end-bearing resistance

\( Q_t \) Friction resistance under tensile load

\( Q_c \) Friction resistance under compressive load

\( \delta_e \) Elastic deformation of pile shaft

\( L \) Pile length

\( N_q, N_\gamma \) Bearing capacity factors

\( S_q, S_\gamma \) Shape factors

\( d_q, d_\gamma \) Depth factors
\( N_q' \) General bearing capacity factor = \( N_q S_q d_q \)

\( \gamma' \) Effective unit weight \((\gamma_{\text{bulk}} - \gamma_{\text{water}})\)

\( U_x \) Displacement in x-direction

\( U_y \) Displacement in y-direction

\( U_z \) Displacement in z-direction

\( U_{Rx}, U_{Rz} \) Rotations out of x-z Plane

\( r, \theta, \text{and } z \) Local cylindrical coordinates

\( U_r \) Radial displacement

\( U_{\theta} \) Tangential displacement

\( U_{Rr}, U_{Rz} \) Rotations out of r-z plane

\( \mu \) Coefficient of friction

\( \psi \) Dilation angle

\( c \) Cohesion yield stress

\( \varepsilon_{\text{pl}} \) Absolute plastic strain

\( \Delta r \) Cavity expansion

\( r_c \) Initial cavity radius

\( \rho \) Mass density

\( B \) Distance from pile center to tank boundary

\( r_p \) Helical pile radius

\( a \) Horizontal projection of grout nozzles
\begin{align*}
P_r & \quad \text{Radial stress at cavity wall after expansion} \\
Po & \quad \text{Initial radial stress at cavity wall before expansion} \\
H_{ult} \text{ and } P_{ult} & \quad \text{Lateral pile capacity} \\
H_{ult-gr} & \quad \text{Grouted pile lateral resistance} \\
H_{ult-ung} & \quad \text{Un-grouted pile lateral resistance} \\
S_{L2} & \quad \text{Displacement corresponding to point } L_2 \text{ on the load-displacement curve} \\
BD & \quad \text{Distance from pile surface to rigid tank boundary} \\
H_{BD} & \quad \text{Width/Diameter of steel tank} \\
M_z & \quad \text{Bending moment at depth } (z) \\
H & \quad \text{Horizontal (lateral) load} \\
H_f & \quad \text{Lateral pile resistance in finite boundary} \\
H_{inf} & \quad \text{Lateral pile resistance in infinite boundary} \\
e & \quad \text{Lateral load eccentricity} \\
k_{rs} & \quad \text{Relative stiffness factor} \\
I_p & \quad \text{Moment of inertia of the pile cross-section} \\
E_h & \quad \text{Horizontal soil modulus at pile toe} \\
x & \quad \text{Rotation point depth below soil surface} \\
f & \quad \text{Depth of zero shear} \\
P_u & \quad \text{Lateral soil pressure} \\
S_{bu} & \quad \text{Pile shape factor}
\end{align*}
\( P_{0.6x} \) Lateral soil pressure at depth equal to 0.6x

\( y \) Lateral pile deflection

\( S_{11} \) and \( S_{33} \) Lateral and vertical stresses from FEM, respectively.
Chapter 1

Introduction

1.1 Overview

Pile foundations are usually used to support superstructures and transfer their loads to deeper and stronger soil layers. Such a foundation system is typically considered in the case of shallow soft soil deposits. Conventional pile foundations are generally made of timber, concrete, steel, or composite materials, and they can be installed into the ground by driving, drilling, jetting, or by applying torque to the pile head.

The helical pile is a deep foundation that has been widely used in many engineering applications (e.g. residential buildings, solar panels, telecommunication towers) in the past decades. In general, the pile is composed of a hollow steel shaft fitted with one or more helical blades. The steel shaft can have either a circular shape or a square cross-section with rounded corners. The popularity of this deep foundation option is attributed to its cost-effectiveness, easy and rapid installation in restricted access areas, low levels of noise and vibrations, and the ability to be installed through groundwater without casing. Besides, it can be used in repairing deficient foundations of existing structures. Moreover, helical piles can be installed into the ground at any angle by applying a mechanical torque to its head. By monitoring the installation torque, the pile capacity can be empirically estimated through some capacity-torque correlations (Perko 2009, Sakr 2011, Elsherbiny & El Naggar 2013, and Harnish & El Naggar 2017), which provides a means for quality control during the pile construction.

The installation of helical piles, however, can cause soil disturbance that may affect the pile capacity and performance. Screwing the pile into the ground causes torsional and vertical shearing accompanied by soil displacement and stress changes. Moreover, the helical pile rotation loosens the soil within the zone circumscribed by its helix creating a cylindrical failure surface (Vesic 1971). This effect is more pronounced for multi-helix and large diameter anchors (Bagheri and El Naggar 2013). Other drawbacks of helical piles include
the potential for large displacement to fully mobilize the end-bearing contributions of their plates, low buckling resistance of helical piles with slender hollow shafts, and reduced rigidity at the coupling joints for segmented helical piles (Vickars and Clemence 2000).

Different practical solutions have been developed to address the drawbacks of conventional helical piles; e.g. large diameter helical piles (Elkasabgy and El Naggar 2015), tapered helical piles (Fahmy and El Naggar 2017) and grouted helical piles (Elsharnouby and El Naggar 2018). This study investigates the superiority of grouted helical piles over conventional helical piles, and how it can improve the pile capacity and performance. In grouted helical foundations, grouting is utilized as a means for soil improvement to mitigate the disturbance associated with the installation process, enhance the soil properties within the disturbed zone and increase the size and stiffness of the pile cross-section (Mansour et al., 2016).

Several techniques have been used to install grouted helical foundations (Dyche 1952, Perko 2000, Vicars & Clemence 2000, and Nasr 2008). Recently, a new technology, denoted pressure grouted displacement piles (PGDP), has been developed for pressure grouting hollow shaft helical piles during installation. In this method, a grout pump and a patented rotary driver (Suver 2005) are used for the pile installation. The rotary driver allows the injection of flowable grout into the hollow pipe shaft simultaneously while applying a mechanical torque to the pile head that screws the pile into the ground. The grout is injected into the surrounding soil through two grout ports (nozzles) welded to the pile shaft near the helices. This study refers to grouted helical piles installed with this technique as pressure grouted helical piles (PGHPs).

1.2 Research objectives

Although PGHPs are expected to be successful in many engineering applications with different soil conditions, they are not used in practice due to the lack of knowledge regarding the formed grout column around the pile, which affects the load transfer mechanism and the pile performance under different loading conditions. To better understand the behavior of the new piling system, the following objectives were set for this research:
i. Study the effect of nozzle configurations on the shape of PGHPs installed in sand considering three different configurations.

ii. Assess the axial load carrying capacity and the load transfer mechanism of PGHPs constructed using the three nozzle configurations to determine the best configuration for PGHP construction.

iii. Study the effect of grouting pressure and soil density on the axial and lateral performance of PGHP installed with the chosen nozzle configuration.

iv. Provide design guidelines to estimate the ultimate capacity of PGHPs analytically.

1.3 Statement of novelty

In this research, the monotonic axial and lateral behavior of PGHP in sand is investigated for the first time. The thesis elevates the knowledge about the PGHP capacity, its load transfer mechanism, and the pile failure mode under different loading (i.e. uplift, compression, and lateral loads) and testing (i.e. nozzle configurations, soil densities, and grouting pressures) conditions. The research also supports the development of design guidelines to analytically estimate the monotonic axial and lateral capacities of PGHP in sand.

1.4 Thesis organization

The thesis consists of eight chapters that have been organized according to the guidelines provided by the School of Graduate and Postdoctoral studies at Western University. The descriptions of these chapters are summarized below:

**Chapter 1:** Summarizes the advantages and disadvantages of conventional helical piles, and introduces and discusses the innovative installation technique for grouted helical foundations. It also outlines the research objectives and scope of work for this study.

**Chapter 2:** Provides a survey of the existing literature on conventional and grouted helical piles. The first part of this chapter provides a review of the history of helical piles, the soil disturbance accompanying the pile installation, and the methods developed to determine the pile behavior under monotonic axial and lateral loads. The second part of the chapter introduces grouted helical foundations, describes the different grouting techniques used for
their installation, and discusses the improvement in the pile performance they can offer over the conventional helical piles.

**Chapter 3:** addresses the different configurations of the tested piles, testing equipment, and the instrumentation employed to collect relevant data during pile load testing. It also describes the soil preparation process, provides the representative soil parameters, and includes a detailed description of the model PGHP installation technique. Moreover, it presents the pile load testing setups and testing procedures followed during the monotonic compression, uplift, and lateral pile load testing. Furthermore, it includes a detailed description of the created grout mass along PGHP shaft based on visual observations.

**Chapter 4:** reports on the laboratory pile load testing of the monotonic uplift loading of PGHPs installed in sand and their performance compared to that of un-grouted helical piles. The results are compared to those reported in the literature where applicable.

**Chapter 5:** describes the performance of PGHPs installed in sand under monotonic compressive loading and compares it with the behavior of un-grouted piles under the same testing conditions. The results are compared to those reported in the literature where applicable.

**Chapter 6:** presents the numerical analysis of PGHPs under axial monotonic loading to better understand the pile behavior and the load transfer mechanism. The results are then compared with those available in the literature as well as uplift and compression load testing results presented in Chapters 4 and 5. Finally, a parametric study was conducted to evaluate the performance of prototype PGHPs.

**Chapter 7:** reports on the laboratory pile load testing of the monotonic lateral loading of PGHPs installed in sand and their performance compared to that of un-grouted helical piles under the same testing conditions. This is followed by a numerical investigation of the lateral behaviour of PGHPs using a nonlinear three-dimensional finite element model of the system. The obtained results are compared to those available in the literature where applicable.

**Chapter 8:** summarizes the main conclusions drawn from the previous chapters, and provides some recommendations for future studies.
Chapter 2

Literature review

2.1 Helical piles

2.1.1 Introduction

Helical (screw) piles were first introduced as a deep foundation option by Alexander Mitchell in 1833. The first helical pile was made from wrought iron and was installed into the ground via manual torque provided by human or animal power using a large wood handle wheel called capstan (Perko 2009). The applications of early helical piles were limited by low bearing and uplift capacities. However, owing to the continuous development of powerful installation equipment and practical knowledge in the past decades, the helical pile applications developed substantially, and it was utilized as an appropriate anchorage system for structures with medium to high axial forces and overturning moments (Harnish 2015).

Many engineers and construction industry professionals have recently promoted helical piles in many engineering applications (e.g. power transmission towers, solar panels, bridges, and residential and commercials buildings). This can be attributed to their many advantages including the quick and easy installation with reduced associated disturbance and soil spoils, the ability to verify the load carrying capacity by monitoring the installation torque, the possibility of reusing the piles, and the suitability of installing the piles in remote areas (Perko, 2009). In many of these applications, helical piles are subjected to compressive, uplift, and lateral loading schemes (Elsherbiny and El Naggar 2013).

The installation of helical piles into the ground, however, can cause soil disturbance that may affect the pile capacity and performance. Moreover, helical piles have other drawbacks including the potential for large displacement (i.e. 5.0 to 10.0% of the helix diameter) to fully mobilize the end-bearing contributions of their plates (Terzaghi 1942, and O’Neil & Reese 1999), low buckling resistance of those with slender shafts, and the reduced rigidity at coupling joints for segmented helical piles (Vickars and Clemence 2000).
2.1.2 Load transfer mechanism

Within the bounds of traditional soil mechanics, the axial helical pile capacity can be computed as the sum of the pile shaft resistance and the load carrying capacity of the leading section. In general, the resistance of the leading section can be obtained using two methods as demonstrated in Figure 2.1: the individual plate bearing method and the cylindrical shear method. The usage of either method depends on the ratio of the spacing between two successive helices (S) to the helix diameter (D_h), which is defined herein as the inter-helical spacing ratio (S/D_h).

Figure 2.1: The load transfer mechanism of helical piles under compression loading a) Individual plate bearing, and b) Cylindrical shear

When the helical plates are relatively far apart along the pile shaft, the individual bearing method is employed. In this method, no interaction between the helices occurs, and hence, the axial capacity of the leading section is the summation of the end-bearing components of the helical plates. On the other hand, when the pile helices are relatively close, the helical plates act as a group creating a cylindrical failure surface of the soil enclosed between the
top and bottom helices. Thus, the resistance of the leading section can be calculated as the summation of the developed shear stresses along the surface of the enclosed soil cylinder and the bearing resistance of the bottom helix in compression, or the top helix in uplift.

The limiting inter-helical spacing ($S_L$) that ensures an individual plate bearing response is a function of several factors including the soil type, the average helix diameter, and the direction of loading. For helical piles in sand, a minimum $S_L$ equal to three times the largest helix diameter was observed by Bagheri and El Naggar (2013) to avoid the cylindrical failure mode, which was in good agreement with the recommendations of the Canadian Foundation Engineering Manual (CGS, 2006). The same limiting value was suggested by Zhang (1999) for helical piles in cohesive soils. For those installed in cohesionless soils, Zhang observed an inter-helical spacing ratios ($S/D_h$) of 2 and 3 for helical piles under compression and uplift loading, respectively.

Rao and Prasad (1993) explored the $S_L$ for model helical piles installed in cohesive soils with different ($S/D_h$) ratios. The results revealed that the cylindrical shear failure mechanism should be used for all multi-helix piles with $S/D_h \leq 1.5$ for both compression and uplift loading. However, for $S/D_h > 1.5$, the cylindrical failure surface between the adjacent plates will not be fully mobilized, and a correction factor ($F_c$) has to be applied to the resistance component at the cylindrical failure surface as illustrated by Equations 2-1 through 2-3.

\[ F_c = 1 \text{ for } S/D_h \leq 1.5 \quad \text{eq. 2-1} \]

\[ F_c = 0.863 + 0.069 (3.5 - S/D_h) \text{ for } (1.5 < S/D_h \leq 3.5) \quad \text{eq. 2-2} \]

\[ F_c = 0.7 + 0.148 (4.6 - S/D_h) \text{ for } (3.5 < S/D_h \leq 4.6) \quad \text{eq. 2-3} \]

The correction factor ($F_c$) accounts for the cylindrical shearing surface, which is not going to be fully mobilized, resulting in a condition where there are partial individual bearing and partial cylindrical shear. It should be noted that although the inter-helical spacing between the pile helices exhibited considerable effects when testing small scale (model) helical piles, it may not have the same effect for large helical piles in the field. Thus, the application of these correction factors should be used with caution.
2.1.3 Installation disturbance

According to Mooney et al. (1985) and Ghaly & Clemence (1998), helical piles have the same load transfer mechanism under compression and pullout loading. Thus, their compressive and uplift load capacities are theoretically expected to be equal. However, Trofimenkov and Mariupolski (1965) investigated the performance of helical anchors with single helix under uplift and compression loading and reported that the pile compressive capacity is 1.4 to 1.5 times its pullout capacity in both sand and clay. They attributed the variation in the axial pile resistance to the disturbed soil zone along the installation path. Zhang (1999) came to the same conclusion for identical multi-helix anchors.

2.1.3.1 Causes and solutions

The degree of soil disturbance depends on the soil type, the pile configuration, and the pile geometry. For helical piles installed in sand, the installation process causes torsional and vertical shearing accompanied by sand displacement and stress changes. Moreover, the pile rotation may loosen the sand within the zone circumscribed by its helix creating a cylindrical failure surface around the pile (Vesic 1971). This effect is more pronounced for multi-helix and large diameter anchors (Bagheri and El Naggar 2013). For helical piles in cohesive soils, the installation technique can cause soil remoulding within the affected zone introducing fissures and weakened soil regions within the pile cylindrical failure surface (Mooney et al. 1985, Zhang 1999, Elkasabgy & El Naggar 2015, and Harnish & El Naggar 2017). These fissures reduce the shear strength of the soil and decrease the total pile capacity. Bagheri and El Naggar (2015) studied the installation effects of screw anchors in structured clay. They reported that the helical pile capacity and performance are significantly affected by the degree of soil remoulding caused by the penetration of the pile shaft and helices.

Several practical solutions have been developed to minimize the installation disturbance of helical anchors. First, it is recommended to use the same pitch for all helices. Second, keep the inter-helical spacing between the helices as multiples of their pitch to force all the plates to track a single path during installation (Seider, 2004). Moreover, Ghally and Hanna (1992) reported that helical piles with a larger pitch to helix diameter ratio (P/Dh) cause less soil
disturbance during installation as they can advance greater distance into the ground per revolution. Furthermore, it was found that for a multi-helix pile, the use of tapered helices profile would generally enhance the pile resistance compared to the cylindrical helices profile (Tsuha et al., 2012). Radhakrishna (1976) investigated the effect of helical pile configuration on the pile installation torque. He reported that a tapered multi-helix anchor required about 25% more torque to be installed to a certain depth and showed larger pile capacity than a uniform multi-helix anchor installed to the same depth. Radhakrishna stated that this increase in the capacity of the tapered multi-helix anchor is attributed to the fact that the top two helices, in case of the uniform helix plate arrangement are located entirely in soil that has been disturbed by the leading helix. Finally, Perko (2009) observed that the use of low-speed motors to torque down the piles into the ground would also minimize the resulting installation disturbance.

2.1.3.2 The effect of installation disturbance on soil strength

Soil disturbance occurred during the installation of helical piles can have significant effects on the pile capacity and performance. Thus, considering the appropriate soil strength parameters and failure mechanism is indispensable for calculating the ultimate pile resistance accurately. For helical piles in sand, the large sand displacements with the vertical and torsional shearing observed during the pile installation within the zone circumscribed by its helices, increase the probability of post-peak conditions especially, in dense sand strata that exhibit a considerable strain softening behavior during their shear failure (Bagheri and El Naggar, 2013). Thus, in their study, Bagheri and El Naggar suggested using the residual friction angle for the disturbed soil zone to deliberate the effect of strength loses regarding installation. The residual friction angle can simply be estimated by subtracting the dilatancy angle from the peak friction angle of the sand. This assumption is in good agreement with Ghally et al. (1991) experimental investigation. Ghally et al (1991) revealed that for helical piles in dense sands, the mobilized friction angles were equal to 0.65 of the peak friction angles, while in loose sand the peak friction angles were fully mobilized.

A field study was undertaken by Weech and Howie (2012) to examine the performance of helical piles in slightly over-consolidated clayey silt. The piles were load tested, and the
mobilized shear strength was back calculated. A decrease in the shear strength was observed due to the soil remolding along the disturbed zone. Tappenden (2007) investigated the performance of 29 full–scale helical piles installed in western Canada and came to the same conclusion. An experimental program consisted of two phases were conducted by Elkassabgy and El Naggar (2015) on seven full–scale helical piles installed in cohesive soil. The first pile group were tested two weeks after installation, while the second one was tested nine months later. During phase one, the surrounding soil was still disturbed because of the installation process, and a significant reduction of approximately 66% in the soil strength parameters was observed. This decrease in the strength parameters compared to the intact values was reduced to 31% during phase two. Elkassabgy and El Naggar explained this improvement in the soil resistance to the long time between the two phases that allowed the disturbed soil to regain some of its original strength.

2.1.4 Compressive resistance

The axial helical pile capacity is equal to the resistance of the pile shaft ($Q_s$) plus the load carrying capacity of the leading section ($Q_{LS}$). This section discusses the individual plate bearing method and the cylindrical shear method used to predict the compressive capacity of helical piles in sand.

2.1.4.1 Individual plate bearing method

The individual plate bearing method assumes that each helical plate affixed to the pile shaft contributes individually to the total capacity of the system. Thus, the resistance of the leading section can be obtained by adding the end bearing component of the pile toe to the summation of the bearing resistances of the pile helices. For piles in cohesionless soils, the ultimate end-bearing resistance can be calculated based upon the modified bearing capacity equation proposed by Meyerhof (1951):

$$q_{ult} = q'N_qS_qd_q + 0.5\gamma'DN_\gamma S_\gamma d_\gamma$$  \hspace{1cm} \textbf{eq. 2-4}

where $q'$ is the effective overburden pressure at the bearing depth, $\gamma'$ is the effective soil unit weight, $D$ is the diameter of the bearing element, $N_q$ and $N_\gamma$ are the bearing capacity factors.
depending on the soil friction angle, \( S_q \) and \( S_\gamma \) are the shape factors depending on the shape of the bearing element, and \( d_q \) and \( d_\gamma \) are the depth factors depending on the location of the bearing element below the ground surface.

For deep foundations with depth to diameter ratio (H/D) greater than five, the \( \gamma \)-term is typically less than 10% of the \( q \)-term, thus; it can be safely disregarded (Kulhawy, 1984). From the back analysis of more than 54 full-scale helical pile load tests, Perko (2009) found that combining the bearing capacity factors of Meyerhof (1951) with the shape and depth factors of Hansen (1970) and Vesic (1973) best fit the pile load test data. Therefore, the following equations were recommended by Perko (2009) to determine the bearing capacity, shape, and depth factors of helical piles in sand.

\[
N_q = e^{\pi \tan \phi} \tan^2 \left( 45 + \frac{\phi}{2} \right) \quad \text{eq. 2-5}
\]

\[
S_q = 1 + \frac{B}{L} \tan \phi \quad \text{eq. 2-6}
\]

\[
d_q = 1 + 2k \tan \phi (1 - \sin \phi)^2 \quad \text{eq. 2-7}
\]

\[
K = \tan^{-1} \left( \frac{H}{B} \right) \quad \text{eq. 2-8}
\]

where \( H \) is the depth of the bearing element, \( B \) and \( L \) are the width and the length of the bearing element, respectively. For circular pile foundations, \( B = L = \text{Diameter of the circular bearing element} \).

In general, helical piles are situated substantially deeper than the helix diameter. Thus, the ratio H/D is very large, and the value of \( K \) approaches \( \pi/2 \). Since both \( K \) and \( B/L \) are constants, therefore; the shape and depth factors vary only with the soil’s friction angle. And hence, they can be grouped with the bearing capacity factor according to Perko (2009) and form a general bearing capacity factor \( N_q' = N_q S_q d_q \). The general end-bearing capacity factor can also be calculated using Meyerhof (1976), Janbu (1976), Vesic (1977), and Coyle & Castello (1981) methods. To account for the pile weight or, in case of helical piles, the weight of soil above the helical bearing plates, the overburden pressure (\( q' \)) is typically
subtracted from the $q$-term in the bearing capacity equation. Consequently, Meyerhof’s (1951) bearing capacity equation can be simplified to \textbf{Equation 2-9}.

\begin{equation}
Q_{ult} = q'(N_q'-1)
\end{equation}

The contribution of the shaft resistance ($Q_s$) to the total helical pile capacity can be affected by a number of phenomena (Perko 2009). First, most of the manufactured helical piles are composed of sections connected together using coupling joints with a slightly larger diameter than the pile shaft. These couplings create a loosened soil zone immediately adjacent to the pile shaft as they are turned into the ground. Moreover, wobbling the pile shaft during installation may cause soil separation along the upper portion of the shaft. For helical anchors with square shaft cross-section, the rotation of the pile while being installed into the ground creates a circular hole around the shaft. All these factors significantly reduce the shaft adhesion with the surrounding soil. Therefore, the contribution of $Q_s$ to the total helical pile capacity is typically ignored for slender shafts.

However, in certain circumstances (i.e. larger-diameter and grouted helical piles), the piles may develop a considerable portion of their strength through the friction resistance at the pile-soil interface. In these circumstances, $Q_s$ can be calculated using the $\beta$-method introduced by Burland (1973) as shown in \textbf{Equation 2-10}.

\begin{equation}
Q_s = k_s q' \tan \delta (\pi d H) = \beta q' (\pi d H)
\end{equation}

where $k_s$ is the coefficient of lateral earth pressure, $\delta$ is the interfacial friction angle at the pile-soil interface, $q'$ is the average effective vertical stress along the pile shaft, $\beta$ is the friction capacity factor, $d$ is the pile shaft diameter, and $H$ is the pile shaft length.

The value of $k_s$ is significantly affected by the friction angle ($\varphi$), the pile configuration, the soil compressibility, the initial horizontal stresses in terms of the at rest earth pressure ($k_o$), and the pile installation technique (Meyerhof 1976). According to the Canadian Foundation Engineering Manual (CFEM 2006), $k_s$ is usually assumed as $k_o$ for bored piles, and twice the value of $k_o$ for driven (large displacement) piles. However, for small displacement piles (e.g. helical piles and H-piles), Kulhawy et al. (1983) observed a $k_s$ value between 0.75 and 1.25 of $k_o$. 
Zhang (1999) suggested subtracting a distance equal to one helix diameter from the total shaft length to account for the shadowing effect above the top helix. Therefore, the maximum compressive resistance ($Q_{ult}$) of a helical pile in sand according to the individual plate bearing method is given by Equation 2-11.

$$Q_{ult} = \gamma' H_n N_q' A_t + \sum \gamma' H_i N_q'^i (A_{hi}) + \frac{\pi}{2} d H_{eff}^2 \gamma' k_s \tan \delta$$  \hspace{1cm} \text{eq. 2-11}

where $\gamma'$ is the effective unit weight, $H_n$ is the depth of pile toe, $N_q'$ is the general bearing capacity factor, $A_t$ is the area of pile toe, $H_i$ is the embedment depth of helix (i), $A_{hi}$ is the area of helix (i), $d$ is the diameter of pile shaft, $H_{eff}$ is the effective pile shaft length, and $\delta$ is the angle of friction at the pile-soil interface.

2.1.4.2 Cylindrical shear method

As the inter-helical spacing ($S$) decreases, the influence zones of the pile helices will overlap, and the enclosed soil mass will form a cylindrical failure surface with the same average diameter as the helical plates (Narasimha Rao and Prasad, 1993). Thus, the helical pile capacity under compression loading is equal to the summation of the bearing resistance of the bottom helix, the shear resistance along the cylindrical failure surface, and the friction resistance along the pile shaft (Mooney et al., 1985). Therefore, the maximum compression resistance ($Q_{ult}$) of a helical pile in sand according to the cylindrical shear method can be predicted by Equation 2-12.

$$Q_{ult} = \gamma' H_n N_q' (A_h + A_t) + T(n-1)\pi S D_{avg} + \frac{\pi}{2} d H_{eff}^2 \gamma' k_s \tan \delta$$  \hspace{1cm} \text{eq. 2-12}

where $T$ is the average shear resistance along the cylindrical failure surface, $n$ is number of helices, $S$ is spacing between two successive helices, and $D_{avg}$ is the average helix diameter.

The average shear resistance acting along the cylindrical failure surface ($T$) can be computed using the $\beta$-method for helical piles in cohesionless soils. However, the helical pile installation displaces the sand in direct contact with the helices laterally. This soil displacement increases the lateral stresses immediately adjacent to the pile resulting in higher $k_s$ values. According to Mitsch and Clemence (1985), this increase in the lateral earth
pressure is proportional to the initial relative density of the soil. A best-fit regression analysis was conducted by Perko (2009) to find a relationship between \( k_s \) at the cylindrical failure surface and the angle of internal friction between soil particles \( \varphi \), i.e.

\[
K_s = 0.09e^{0.08\varphi}
\]

where \( \varphi \) is the peak friction angle in degrees.

### 2.1.5 Uplift resistance

For the case of deep installation, the pullout capacity of a helical pile can be calculated similar to its compressive capacity with reduced shear strength parameters to account for the installation disturbance as discussed in Section 2.1.3. To ensure full mobilization for the bearing resistance of the top helix (i.e. deep anchor behavior) a minimum embedment depth \( (H_{cr}) \) is required. Otherwise, shallow failure response may occur (Zhang 1999). For helical anchors installed in sand to a relatively shallow depth, the failure surface is defined by a plane inclined to the vertical at an angle \( (\theta) \) that extends from the outer edge of the top helix to the ground surface (Mitsch and Clemence 1985), with \( \theta = \varphi/4 \) to \( \varphi/2 \) (Meyerhof and Adams 1968).

The minimum embedment depth \( (H_{cr}) \) is a function of the helix diameter \( (D_{helix}) \) and the relative density of the surrounding soil (Ghaly et al., 1991). Greater embedment depths are required to ensure deep anchor condition in denser soil deposits as observed by Ghaly and Hanna (1992). The change in the pullout response from shallow to deep failure condition passes with a transient zone and doesn’t occur suddenly as illustrated in Table 2.1.

Several researchers have investigated the pullout capacity of helical anchors in sand (Turner 1962, Meyerhof & Adams 1968, Clemence 1984, Ghaly et al. 1991, and Nirounmand et al. 2012). According to these studies, the maximum tensile resistance of deep anchors is the summation of the bearing resistance on the top helix, the load carrying capacity of the leading section, and the friction resistance along the anchor shaft. For shallow anchors, the pullout capacity is equal to the summation of forces acting on the failure surface, the weight of soil.
wedge within the failure zone, and the resistance of the leading section. Figure 2.2 presents the shallow and deep failure mechanisms for a helical pile under uplift loading.

Table 2.1: Embedment depths for the different pullout failure modes (After Ghaly and Hanna, 1992)

<table>
<thead>
<tr>
<th>Failure condition</th>
<th>Ratio between embedment depth and diameter of top helix (H/D_helix)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose</td>
</tr>
<tr>
<td>Shallow failure</td>
<td>&lt; 7</td>
</tr>
<tr>
<td>Transient failure</td>
<td>7 to 9</td>
</tr>
<tr>
<td>Deep failure</td>
<td>&gt; 9</td>
</tr>
</tbody>
</table>

Figure 2.2: Failure of helical pile under pullout loading a) shallow anchor behavior, and b) deep anchor behavior

To develop general pullout capacity equations for helical piles experiencing shallow and deep failure conditions, Mitsch and Clemence (1985) analyzed the results of 13 full-scale field tests and 16 model laboratory tests. They observed an increase in the potential for
cylindrical failure condition at the leading section due to the soil disturbance in the zone circumscribed by the helices and the densification of the surrounding soil during the installation process. Thus, they proposed Equations 2-14 and 2-16 to estimate the maximum uplift resistance of shallow and deep screw anchors, respectively.

\[
Q_{ult} = \frac{\pi}{2} D_a \gamma' (H_3^2 - H_1^2) k_u \tan \phi + \pi \gamma' k_u \tan \phi \cos \left( \frac{\phi}{2} \right) (D_a H_1^2 / 2 + H_1^3 \tan \left( \frac{\phi}{2} \right) / 3) + W_s \quad \text{eq. 2-14}
\]

\[
W_s = \frac{\pi}{4} \gamma' \int_0^{H_1} (D_1 + 2\zeta \tan \theta)^2 \, dz \quad \text{eq. 2-15}
\]

\[
Q_{ult} = \frac{\pi}{2} D_a \gamma' (H_3^2 - H_1^2) k_u \tan \phi + \gamma' H_1 N_{qu} ' (A_1 - A_{sh}) + \frac{\pi}{2} d H_1^2 \gamma' k_u \tan \delta \quad \text{eq. 2-16}
\]

where \(Q_{ult}\) is the ultimate anchor uplift capacity, \(\gamma'\) is the effective unit weight, \(k_u\) is the coefficient of lateral earth pressure under uplift loading, \(A_1\) is the area of top helix, \(A_{sh}\) is the area of pile shaft, \(N_{qu}\) is the uplift bearing capacity factor, \(H_1\) is the depth to top helix, \(H_3\) is the depth to bottom helix, \(D_1\) is the diameter of top helix, \(D_a\) is the average helix diameter, \(d\) is the shaft diameter, \(\phi\) is the soil friction angle, \(\delta\) is the friction angle at the pile-soil interface, and \(\theta\) is the inclination of the failure surface above top helix.

The bearing capacity factor under uplift loading (\(N_{qu}\)) can be determined similar to the general bearing capacity factor (\(N_u\)) using the residual friction angle to consider the installation disturbance as recommended by Bagheri and El Naggar (2013). On the other hand, the value of the lateral earth pressure coefficient along the cylindrical failure surface under uplift loading (\(k_u\)) undergoes considerable changes from the initial condition to the failure condition. Screwing the pile into the ground results in an increase in the lateral earth pressure by displacing and churning the sand in the pile vicinity. Besides, as the anchor is pulled out, the lateral stresses of the surrounding soil increase toward the passive state. This increase depends on the degree of soil disturbance, the change in stress level during installation, and the embedment pile depth. Figure 2.3 summarizes the values of \(k_u\) suggested by Meyerhof & Adams (1968) and Mitsch & Clemence (1985) for the design of screw anchors in sand.
2.1.6 Lateral resistance

When a pile foundation is subjected to a lateral load it can behave in one of two ways; either as a rigid (short) pile or as a flexible (long) pile depending on the soil stiffness and the pile geometry. In the rigid pile response, the lateral pile capacity depends entirely on the soil resistance, and it can be observed when the pile length is relatively short with respect to its diameter. While for long piles, the lateral pile resistance is governed by the flexure resistance of the pile section.

2.1.6.1 The effect of the pile helix

The contribution of the helical plate to the lateral pile capacity depends on the helix depth below the ground surface and the expected pile response (short or long). Prasad and Rao (1996) investigated the lateral performance of small-scale helical piles in clay. The piles were installed with a length to helix diameter ratio varied between 12 and 18. They observed an increase of 20 to 50% in the helical pile resistance compared to the straight pile with the same shaft diameter. They related the improvement in the helical pile capacity to the bearing and friction resistances developed at the two sides of the pile helix during its rotation. Similar behavior was observed by Fahmy and El Naggar (2015 and 2017) for rigid full-scale tapered helical piles in sand.
Puri et al. (1984) studied the lateral response of small-scale and full-scale helical piles in both sand and clay. They concluded that the lateral helical pile capacity is independent of the presence of the pile helix and is mainly controlled by the mechanical properties of the pile shaft. Sakr (2009) conducted lateral load tests on full-scale helical piles installed in oil sands. He concluded that the lateral pile behavior was mainly controlled by the cross-section area and the inertia of the pile shaft. Moreover, the increase in the number of helices caused a slight reduction in the pile lateral capacity due to the higher degree of soil disturbance associated with the pile installation.

Elkasabgy and El Naggar (2015) investigated the lateral performance of large-capacity full-scale helical piles. The piles had two helices and were installed in a cohesive soil stratum with two depths (6.0 m and 9.0 m). The piles were subjected to static lateral loads after two weeks and nine months from the installation date. The results showed that the ultimate lateral capacity increased with time. Furthermore, the modulus of subgrade reaction obtained from the analysis of the pile load tests was smaller than what had been calculated for the intact soil, which highlights the installation disturbance effect on the soil resistance.

The discrepancy in the effect of the helical bearing plates reported in the literature can be attributed to the variable soil conditions, the different helix depth relative to the active soil resisting zone, and whether the pile is behaving as short rigid pile where rotation activates the bearing resistance on the top and bottom sides of the helix or as long pile where rotation doesn't take place. Consequently, the effect of the pile helix can be conservatively ignored, and the helical pile lateral resistance can be estimated using the same methods developed for straight piles taking into account the effect of installation disturbance on the surrounding soil (Perko 2009).

2.1.6.2 Analytical estimation of the lateral resistance

Several methods have been developed to categorize the pile response into rigid or flexible and to estimate its ultimate capacity under lateral loading. Kasch et al. (1977) defined the pile rigidity using the embedded length to diameter ratio (L/d). According to their study, flexible pile behavior was observed for L/d > 20, while those with L/d < 6 experienced the
rigid pile response. Poulos and Davis (1980) proposed the stiffness ratio ($k_r$) as a measurement for the pile rigidity, i.e.

$$k_r = \frac{E_p I_p}{E_{soil} L^4} \quad \text{eq. 2-17}$$

where $E_p$ is the elastic modulus of the pile material, $I_p$ is the moment of inertia of the pile cross-section, $E_{soil}$ is the soil modulus of elasticity, and $L$ is the pile length.

If the stiffness ratio is less than $10^{-5}$, flexible pile behavior can be expected, while for $k_r$ greater than 0.01, the pile will behave as rigid pile.

Broms (1964a and 1964b) developed an analytical method to compute the lateral pile capacity in both sands and clays based on the limiting equilibrium analysis. He defined two possible failure mechanisms (rigid and flexible) depending on the ratio between the pile embedment depth ($L$) and the relative stiffness factor ($R$) that defines the relative rigidity of the pile with respect to the surrounding soil:

$$R = \sqrt[4]{EI/K_x} \quad \text{eq. 2-18}$$

where $K_x$ is the modulus of horizontal subgrade reaction, $EI$ is the pile flexural stiffness.

The modulus of horizontal subgrade reaction ($K_x$) introduced in Equation 2-18 can either be constant or increase linearly with depth. For piles with $L/R < 2$, rotation dominates the pile deflection, and rigid pile response is observed. The lateral capacity of a rigid pile is mainly controlled by the soil resistance along the pile shaft. For piles with $L/R > 4$, flexible pile behavior occurs. The flexible pile failure mechanism involves the formation of a plastic hinge within the pile shaft. Thus, the lateral pile capacity is influenced by the flexure capacity of the pile material and the soil resistance along the deflected pile length (Davisson, 1970).

Figure 2.4 presents the rigid and flexible pile responses under lateral loading. It should be mentioned that Brom’s method cannot deal with multi-layered soil profiles and it does not provide any information regarding the pile head deflection.
Winkler (1867) introduced the subgrade reaction approach to analyzing the soil-pile system as a beam on a series of independent linear elastic springs that is also known as Beam on Winkler Foundation (BWF). The BWF model can be represented by the fourth order differential equation, i.e.

$$E_pI_p \frac{d^4 y}{dx^4} + k_x \rho d = 0$$  \hspace{1cm} \text{eq. 2-19}

where $E_pI_p$ is the flexure stiffness of the pile, $k_x$ is the modulus of subgrade reaction, $\rho$ is the pile deflection, and $d$ is the pile diameter.

The solution for this differential equation can be obtained either analytically for constant $k_x$ with depth or numerically for other $k_x$ distributions (Poulos and Davis 1980). The most widely used expression for the distribution of $k_x$ with depth was developed by Palmer and Thompson (1948), i.e.:

$$k_x = k_L \left( \frac{Z}{L} \right)^n$$  \hspace{1cm} \text{eq. 2-20}

where $k_L$ is the modulus of subgrade reaction at the pile tip ($Z = L$), and $n$ is an empirical factor depending on the soil type (i.e. $n = 0$ for clay soils, and 1 for granular soils). For clays under undrained condition, a value of 0.15 was suggested by Davisson and Prakash (1963) to allow for plastic soil behavior at the pile surface.
It should be noted that due to the assumption of linear elastic springs, the BWF model does not take into account the non-linear soil behavior. Thus, it can only be used for small lateral loading conditions that keep the soil strains within the linear elastic range.

To account for the soil non-linearity, Matlock (1970) introduced the p-y curve method. In this method, a series of non-linear springs are used in the BWF model as shown in Figure 2.5. The p-y curve is a load-displacement relationship that relates the soil resistance (p) to the pile deflection (y). Empirical p-y curves obtained from experimental test data (Matlock 1970, Welch & Reese 1972, Reese et al., 1974, and Murchison & O’Neil 1984) were proposed by the American Petroleum Institute (API) and included in geotechnical software packages such as LPILE (Ensoft Inc., 2011) to predict the pile response under lateral loading.

The available p-y curves are back-calculated from a set of full-scale lateral pile load tests. Thus, they are representative of the same pile configuration and soil conditions as the experimental data set (Elsawy et al. 2019). For example, most of the developed p-y curves are established for relatively small diameter cylindrical piles. Therefore, they are not suitable for large diameter piles or those with square shafts. Perko (2009) carried out LPILE analysis, using the p-y curves approach, considering several pile types and found that helical piles offer a lateral capacity of the same order of magnitude as micropiles and small drilled piles with comparable shaft diameters and installed in similar soil conditions. This numerical
analysis supported his conclusion of ignoring the helix effect while estimating the helical pile lateral resistance.

2.2 Grouted helical piles

2.2.1 Introduction

Helical anchors were primarily designed to support structures under uplift and/or overturning loads. However, during the early 1990s, helical piles subjected to compression loading were installed at several sites in western Canada. The soil profile at these sites were composed of soft, unconsolidated alluvial soils with high organic content underlain by competent soil strata. Due to the relatively large slenderness ratio of the steel shaft, there is a potential for buckling of the steel shaft embedded in soft soils, which in return limits the load carrying capacity of helical anchors (Vickars and Clemence 2000). To address the drawbacks of conventional helical piles, grouted helical foundations were developed to enhance the soil properties within the disturbed zone, and increase the size and the stiffness of the shaft cross-section (Mansour et al., 2016). Hence, grouted helical piles are characterized by high buckling resistance in weak soil deposits when they are designed to support compression loads, additional corrosion protection for the pile shaft material, more rigid connections at the coupling joints, and higher shaft resistance contribution to the total pile capacity.

2.2.2 Installation techniques

Several techniques have been developed to install grouted helical foundations. Dyche (1952) developed a method to inject pressurized grout into the surrounding soil through some grout openings at each flight along the leading section. Dyche pile was different from the helical piles used nowadays as it had continuous spiral bearing plates along its leading section as presented in Figure 2.6. Ratliff (1966) patented a grouting method that can be used with a more similar helical pile to what is used nowadays as shown in Figure 2.7. Ratliff placed grout ports at the lead helix and at each coupling along the hollow pipe shaft. The grout is forced through the shaft with an aboveground pump and grout silo.
Perko (2000) introduced a grouting method to control the soil expansion around helical piles. Perko’s technique involved grout ports behind the trailing edge of the piles’ helices to seal their tracks and prevent surface water from reaching the zone of expansive soils as illustrated in Figure 2.8. However, increasing the wetting zone around helices has not proven to affect the performance of helical piles. Thus, Perko’s technique experienced limited use (Perko, 2009).
Vicars and Clemence (2000) developed the pulldown micropile, which implements a procedure for grouting helical piles during installation. The procedure involves soil displacement plates attached to the end of the lead section and each coupling joint along the pile shaft. As the lead section is advanced into the ground, the displacement plates push the soil laterally to create a cylindrical void around the pile shaft, which is immediately filled with flowable cement based grout. The grouted screw pile developed by Vicars and Clemence (2000) is presented in Figure 2.9. The created grout column was found to significantly improve the pile buckling resistance (Vickars and Clemence, 2000), and increase its ultimate capacity (Wesolek et al., 2005).

Nasr (2008) injected grout under high pressure (200 and 300 psi) into the surrounding soil during the installation of helical piles. The grout was injected through side ports and grout conduits extending through the hollow pile shaft as shown in Figure 2.10. He demonstrated that the increase in the grouted helical pile capacity was up to 100% compared to the un-grouted pile.

Recently, a new technology was developed by the American Pile-driving Equipment (APE) for pressure grouting hollow shaft helical piles during installation called pressure grouted displacement pile (PGDP). In this method, a grout pump, and a special rotary driver patented by Paul Suver (2005) for pipe piling are used for the installation process. The rotary driver
allows the injection of flowable grout into the hollow pipe shaft simultaneously while applying a mechanical torque to the pile head that screws the pile into the ground. The grout is injected into the surrounding soil through two grout nozzles (ports) welded to the pile shaft near the helices. This study refers to the helical piles installed with this method as “pressure grouted helical piles (PGHPs).” Figure 2.11 presents the grout ports and the rotary driver used for PGHP installation.

![a- Helical pile with grout ports](image1.png) ![b- Rotary driver](image2.png)

**Figure 2.11: Installation of PGHP a) grout ports near the pile helix, and b) Rotary driver (After APE helical pile test report 2015)**

2.2.3 Grouted helical pile capacity

The grouted helical pile is a relatively new deep foundation system. Minimal effort has been published on the performance of grouted helical foundations under different loading conditions. Vickars and Clemence (2000) studied the behavior of grouted helical piles under axial loading. They found that the maximum pile resistance under compression loading ($Q_u$) is the summation of the skin friction mobilized along the grout shaft ($Q_s$) and the load carrying capacity of the leading section ($Q_{L_S}$). They suggested that the grout column will act as a friction pile, and the load transfer mechanism of the leading section will follow the individual plate bearing method as shown in Figure 2.12. Thus, the ultimate compressive capacity of grouted helical foundations can be obtained using Equations 2-21 and 2-22.
\[ Q_{LS} = \sum A_i (c N_c + q N_q) \]  \hspace{1cm} \text{eq. 2-21}

where \( Q_{LS} \) is the resistance of the leading section, \( A_i \) is the anchor plate area, \( c \) is the soil cohesion, \( q \) is the overburden stress, and \( N_c \) and \( N_q \) are the bearing capacity factors.

\[ Q_s = \sum \pi D f_s \Delta L_g \]  \hspace{1cm} \text{eq. 2-22}

where \( Q_s \) is the friction resistance along the grout shaft, \( D \) is the grout shaft diameter, \( f_s \) is the unit shaft resistance, and \( \Delta L_g \) is the length of the grout shaft segment.

\[ Q_i = Q_s + Q_e \]

\[ Q_1 = Q_2 + Q_3 \]

Figure 2.12: Compression load transfer mechanism along grouted helical pile (After Li 2006)

A series of laboratory tests was conducted by Wilder (1989) on conventional and grouted helical anchors with single and triple helices. The piles were installed in sand to the same depth. An increase of approximately 45% in the ultimate capacity of the grouted helical piles was observed compared to the conventional ones due to the presence of the grout shaft. According to Wilder (1989), the skin friction resistance along the grout shaft can be computed using the \( \beta \)-method, while the capacity of the leading section can be estimated by:

\[ Q_{LS} = \gamma [(N_q - 1) (d_{BH} + d_{MH} + d_{TH} + d_{GS}) + 0.3 N_q (B_{BH} + B_{MH} + B_{TH} + B_{GS})] \]  \hspace{1cm} \text{eq. 2-23}
where $d_{BH}$, $d_{MH}$, and $d_{TH}$, are the depths of the bottom, middle, and top helix, respectively. $B_{BH}$, $B_{MH}$, and $B_{TH}$ are the diameters of the bottom, middle, and top helix, respectively. $d_{GS}$ and $B_{GS}$ are the depth and diameter of grout shaft, respectively. $N_q$ and $N_γ$ are the bearing capacity factors.

Weech (2002) conducted a series of pile load tests on instrumented helical pulldown micropiles in soft marine soils. He showed that a grouted shaft 9.0 m long contributed 8 to 21% to the ultimate pile capacity. Wesolek et al. (2005) conducted two tests on ungrouted helical piles and one test on a grouted helical pile. Wesolek reported that the grouted shaft added 10 to 23% to the ultimate pile capacity and reduced the pile head displacement under the working load from 27.9 mm to 2.5 mm, and under the ultimate load from 38.1 mm to 5.1 mm.

El Sharnouby and El Naggar (2012) studied the axial performance of steel fibre-reinforced helical pulldown micropile (RHPM) and fiber-reinforced Polymer-steel fiber-reinforced helical pulldown micropile (FRP-RHPM) installed in stiff clayey silt till underlain by dense sand. The results showed an increase of 70% in the helical pile capacity and a significant improvement in the pile performance. They predicted the helical pulldown micropile capacity as the summation of the end bearing resistance at the pile helices plus the friction resistance along the grout shaft. The end-bearing capacity factor ($N_q'$) used for predicting the end-bearing resistance of the pile helices was computed according to Meyerhof (1976), and by considering the installation of the helical pulldown micropile is similar to the installation of drilled shafts.

El Sharnouby and El Naggar (2018) investigated the lateral performance of RHPM and FRP-RHPM. They reported that the steel fiber-reinforced grout column has considerably improved the lateral pile capacity. Besides, it exhibited a significant ductile behavior (i.e. no sudden deflection up to 75 mm displacement – 50% of the grout shaft diameter). However, a separation between the steel shaft and grout column was observed during testing.

Seider et al. (2003) predicted the maximum capacity of grouted helical piles by using the torque method to estimate the end bearing capacity, while the friction resistance along the grout shaft was computed using Gouvenot method, modified Reese and O’Neil method, and
FHWA method (FHWA Report No.SA-97-070). He compared the predicted capacities with the measured load capacities and reported that this procedure underestimated the load capacity in most cases.

2.3 Pile load testing and ultimate capacity interpretation

Pile load testing is used to evaluate the pile performance and verify its maximum resistance under the applied load. Static pile load testing involves application of a static load to the pile head. By recording the pile movement and the applied load, the static load-displacement response curve can be developed through which, the load carrying capacity and the global stiffness response of the pile can be determined.

2.3.1 Axial load testing

For piles under axial loading, failure of a single pile is conceptually reached when a rapid increase in the pile displacement occurs at constant or slightly increased loads. This is often referred to as a plunging/slippage failure. However, pile load tests do not always exhibit that behavior. Figure 2.13 presents the different load-displacement responses that can be achieved by a pile foundation under axial loading.

![Figure 2.13: Typical load-displacement curves (after Kulhawy 2004).](image)
Plunging/slippage failure mechanism represented by curves A and B is often realized for friction piles installed in cohesive soils. On the other hand, for bearing piles in cohesionless soils, plunging failure cannot be reached, and a progressive strengthening behavior through soil densification is observed as shown in curve C. Thus, several failure criteria have been developed to interpret the ultimate pile capacity through its load-displacement curve when the plunging/slippage failure load cannot be reached or correlated to a high level of displacement which is not suitable for the design.

The developed criteria are based on either mathematical modelling (Chin 1970, and Decourt 1999), graphical construction (Mansur & Kaufman 1956, Butler & Hoy 1977, and Hirany & Kulhawy 1989), or settlement limitations. Failure criteria incorporating settlement limitations can be grouped under absolute settlement limits (Terzaghi and Peck 1967), settlement limits as a function of the pile diameter (Davisson 1972, O’Neil & Reese 1999, and Livenih & El-Naggar 2008), settlement rates with respect to the applied load (Fuller and Hoy 1970), and limiting settlement ratios (Chellis 1961).

It should be noted that methods depending on mathematical models always tend to overestimate the pile capacity by extrapolating the load-displacement curve to find its asymptote (Hirany and Kulhawy 1989). Mansur and Kaufman (1956) divided the load-displacement curve into three regions: an initial linear segment, a nonlinear segment followed by a final linear zone. They proposed the failure load as the load corresponding to the intersection of tangents to the first and second linear segments. Butler and Hoy (1977) suggested that the ultimate pile capacity is the load that corresponds to the intersection between a tangent with a slope of (0.14 mm/kN) and the extension of the initial straight portion of the curve. Hirany and Kulhawy (1989) defined two points on the settlement curve. Point \( L_1 \) at the end of the initial linear region to designate the elastic limit of the curve, and point \( L_2 \) at the end of the non-linear transitional zone to define the failure threshold. According to this approach, the ultimate pile capacity is the load opposite to point \( L_2 \) beyond which any small increase in the applied load produces a significant increase in the pile head displacement.
Terzaghi and Peck (1967) defined the ultimate pile capacity as the load that causes a pile head displacement equal to 25 mm. This limiting movement was set to be 5% of the pile toe diameter by O’Neil and Reese (1999) and as 10% of the toe diameter by Terzaghi (1942). Davisson (1972) introduced the net deflection concept, which is defined as the total deflection measured at the pile head minus the elastic deformation of the pile shaft (PL/EA). According to Davisson criterion, the ultimate pile capacity is the load corresponds to a pile head displacement that exceeds the elastic deformation of the pile shaft with an offset of 4 mm plus a factor equal to the pile diameter divided by 120. A modified version of Davison criterion with an offset equal to 10% of the helix diameter was developed by practitioner engineers in the helical pile industry (Perko 2009). For helical piles with multi helices, an offset of 8% and 3.5% of the helix diameter was suggested by Livenih & El Naggar (2008) and Elkasabgy & El Naggar (2015), respectively. Fuller and Hoy (1970) defined the failure load as the minimum load that shows a total settlement rate of 0.14 mm/kN. While a ratio of four between the total settlement and elastic settlement was set to be the failure threshold by Chellis (1961). Another failure criteria were defined by Sharma et al. (1984) for piles under uplift loading. According to their study, the ultimate pullout capacity can be defined using one of three criteria: the load corresponding to the point of the sharpest curvature, the load corresponding to a fixed upward displacement of 6.25 mm, or the load corresponding to the point of intersection of the tangents to the load-displacement curve.

### 2.3.2 Lateral load testing

The ultimate lateral resistance of a single pile is computed based on the ultimate limit state; however, the serviceability limit state always controls the pile design. Several interpretation failure criteria have been developed to estimate the lateral failure load. The developed criteria are based on either graphical construction (Prakash and Sharma 1990), pile head rotation (Davidson et al., 1982), or displacement limitations. Failure criteria considering displacement limitations can be divided into absolute displacement limits (MacNulty 1956, and Walker & Cox 1966) and displacement limit as a function of the shaft diameter (Broms 1964a and b, Pyke 1984, and Briaud 1984).
Prakash and Sharma (1990) developed two criteria to define the ultimate lateral pile capacity. The first criterion defines the pile capacity as the load corresponding to the intersection of tangents to the load-displacement curve. While the second one defines the failure load as the load opposite to a specific deflection value (6.25 mm or 12.5 mm). MacNulty (1956) and Walker & Cox (1966) evaluated the maximum pile resistance under lateral loading as the load corresponding to a pile head displacement equal to 6.25 mm and 13.0 mm, respectively. The limiting displacement limit was set to 5%, 10%, and 20% of the pile diameter by Broms (1964), Pyke (1984), and Briaud (1984), respectively. On the other hand, Davidson et al. (1982) computed the maximum lateral capacity at a pile head rotation equal to 2 degrees.
2.4 References


Chapter 3

Experimental program and pile description

3.1 Test piles and instrumentation

Seventeen Pressure Grouted Helical Piles (PGHPs) and five un-grouted helical piles were installed and tested under controlled laboratory environment. The piles were closed-ended hollow shafts fitted with single helices, made of low to medium carbon steel grade with a yield strength of 195 MPa. All piles were 1.0 m long with an outer shaft diameter of 34 mm, an inner shaft diameter of 28 mm, a helix diameter of 150 mm, a helix thickness of 4 mm, and a helix pitch of 76 mm. The selected helix pitch is in good agreement with the acceptance criteria set by ICC-ES (2007) for helical piles.

Two 300 mm long steel arms were welded to the top of the pile shafts for manual torque application. The piles were installed to a depth of 0.8 m into a cylindrical soil bed with a diameter of 1.30 m, and a depth of 1.55 m. The pile shaft diameter (i.e. 34 mm) is a good representative of model (small scale) piles to eliminate the lateral boundary effect of the soil container on the pile behavior (Vesic 1977, and Al-Mahadib 2001). Moreoever, the embedded pile length (i.e. 0.8 m) and the helix diameter (i.e. 0.15 m) were selected to minimize the effect of the tank base on the end-bearing influence zone. For piles under compression loading, the end-bearing influence zone extends to a depth between 3.5 and 5.5 times the helix diameter (Yang 2006).

Furthermore, the piles were fitted with a circular steel plate (pile cap) 120 mm in diameter and 15 mm thick. The cap had four 19 mm diameter openings on the circumference and one with diameter 28 mm at the center for testing and grouting purposes. Each pile was instrumented with six electrical resistance strain gauges (Micro-Measurements general purpose strain gauges CEA-06-250UW-120) at three different levels (1, 39, and 79 cm) from the surface of the soil bed. The strain gauges were connected to a data acquisition system that recorded the readings every 5 seconds. The lead wires connecting the strain gauges with the data acquisition system were passed from inside the pile through small grooves to protect
the gauge from damage during installation due to the high friction at the pile-soil interface. **Figure 3.1** shows the model helical pile used in this study and the locations of strain gauges.

![Model helical pile](image)

**Figure 3.1: Model helical pile**

For PGHPs, two nozzles were welded to the hollow pile shaft with one of the three configurations shown in **Figure 3.2** to enable pressure grouting during pile installation. Accordingly, the PGHPs tested in this study are categorized into three groups as follows: PGHP1 had nozzles above the helix with downward inclination; PGHP2 had nozzles below the helix with downward inclination; and PGHP3 had nozzles below the helix with upward inclination. The grout nozzles were 31 mm long, 9 mm in inner diameter, 3 mm wall thickness, and they were inclined to the pile shaft with an angle of 45°. The vertical spacing between the nozzles’ outlets was 20 mm.

![Nozzle configurations](image)

**Figure 3.2: The three nozzles configurations used for grout injection**
3.2 Geotechnical conditions

As previously mentioned, the test piles were installed in a cylindrical steel tank with a diameter of 1.30 m, and a depth of 1.55 m. The tank was filled with sand in 0.25 m thick layers. A compaction rod, shown in Figure 3.3a, which consisted of a hollow steel rod 34 mm in diameter connected to a rectangular steel plate (300mmx200mmx15mm) and weighing 108 N was used to compact the sand. Each layer was compacted through a number of blows (N = 40, 120, and 200), falling from a height of 0.5 m in a circular manner. The relative densities (RD) achieved employing these compaction efforts were found to be 34%, 53%, and 78%, which represent loose, medium, and dense sand conditions, respectively (Das, 2011). The RD was calculated by measuring the height and weight of each sand layer after compaction using a measuring tape and a calibrated load cell with 100 kN capacity as shown in Figure 3.3b.

![Manual compaction rod](image1)  ![Weighting soil bed](image2)

Figure 3.3: Soil preparation (a) manual compaction, and (b) weighting soil layers

The grain size distribution of the used sand was determined by conducting sieve analysis tests on different soil samples (ASTM C136). The classification curves presented in Figure 3.4 show that the used soil was poorly graded sand (SP) according to the Unified Soil Classification System USCS (ASTM D2487) with an average uniformity coefficient $C_u = 5.6$ and curvature coefficient, $C_c = 0.96$. The soil specific gravity, $G_s = 2.67$ (ASTM D854), its average water content, $w_c = 2.5\%$ (ASTM D2216), and its maximum and minimum voids
ratio were 0.68 and 0.35 (ASTM D4254 and D4253), respectively. The bulk unit weight ($\gamma_{\text{bulk}}$) was measured according to (ASTM D7263) and found to be 17.0, 17.8, and 18.8 kN/m$^3$ for loose, medium, and dense sand beds, respectively.

![Figure 3.4](image)

**Figure 3.4: Grain size distribution of the used sand**

The angles of internal friction ($\phi$) were measured by conducting a series of direct shear box tests following the ASTM (D3080) standard, in which relative density of the tested samples was same as the sand used for the test bed. The samples were sheared at a constant rate of 0.406 mm/min. The measured residual friction angle ($\phi_{r}$) was equal to 33°, while the peak friction angle ($\phi_{p}$) was found to be 34°, 37°, and 41° for the loose, medium and dense sand condition, respectively.

To determine the elastic modulus of the sand bed, Janbu (1963) and Seed & Idriss (1970) provided two power functions given by **Equations 3-1 and 3-2** to define the elastic modulus of the soil as a function of the confining pressure (i.e. soil depth).

$$E_i = m \cdot p_a \left( \frac{\sigma_{i3}}{p_a} \right)^n$$  \hspace{1cm} \textbf{eq. 3-1}

where $E_i$ is the elastic modulus of layer (i), $p_a$ is the atmospheric pressure, $\sigma_{i3}$ is the average confining pressure along layer (i), $m$ is the modulus number, and $n$ is the modulus exponent. It should be noted that both $m$ and $n$ depend on soil porosity.
\[ E_i = 1000 \, k_2 \, \sigma_{i3}^{0.5} \,(1+\nu) \]  \hspace{1cm} \text{eq. 3.2} \\

where \( E_i \) is the elastic modulus of layer (i) in pound per square foot (psf), \( \sigma_{i3} \) is the average confining pressure along layer (i) in psf, \( \nu \) is the Poisson’s ratio, and \( k_2 \) is the modulus number to consider the effect of voids ratio and strain amplitude.

Figure 3.5 shows the variation of the soil elastic modulus estimated by the two functions along the sand bed depth. For small scale models with shallow depths, considering a single \( E \) value for the soil along the pile shaft (upper soil layer) and another value for the soil below the pile toe (lower soil layer) depending on their average confining pressures is a reasonable approximation. Table 3.1 presents the elastic modulus range of each soil layer and summarizes the different soil parameters.

![Figure 3.5: Variation of soil elastic modulus with depth](image)

**Table 3.1: Different soil parameters**

<table>
<thead>
<tr>
<th>Sand Condition</th>
<th>( \gamma_{\text{bulk}} ) (kN/m(^3))</th>
<th>( w_c ) %</th>
<th>( G_i )</th>
<th>( \varepsilon_{\text{max}} )</th>
<th>( \varepsilon_{\text{min}} )</th>
<th>RD %</th>
<th>( \phi_p ) °</th>
<th>( \phi_{cs} ) °</th>
<th>( E ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>17.0</td>
<td>2.5</td>
<td>2.67</td>
<td>0.68</td>
<td>0.35</td>
<td>34</td>
<td>33</td>
<td>33</td>
<td>8.5 – 12.5</td>
</tr>
<tr>
<td>Medium</td>
<td>17.8</td>
<td>2.5</td>
<td>2.67</td>
<td>0.68</td>
<td>0.35</td>
<td>53</td>
<td>37</td>
<td>33</td>
<td>10.0 – 14.0</td>
</tr>
<tr>
<td>Dense</td>
<td>18.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>78</td>
<td>41</td>
<td>33</td>
<td>12.0 – 17.5</td>
</tr>
</tbody>
</table>
3.3 Pile installation

The piles were installed in the prepared sand bed to a depth of 0.8 m by applying manual torque to the pile head using the two steel handles attached to the pile shaft. A torpedo level was frequently used to ensure the vertical alignment of the pile during installation. For PHGPs, a specially fabricated conical steel tank with 9 litres capacity was used to store and inject grout during PGHPs construction. The tank was fitted with a grout inlet, an air inlet, an air valve, a pressure gauge, and a grout valve to control the grouting process. The tank was fixed to the circular pile cap using four high strength bolts with diameter 17 mm at the circumference. Figure 3.6 presents the conical steel tank used for grout injection.

![Conical steel tank and fittings used for grout injection](image)

**Figure 3.6: Conical steel tank and fittings used for grout injection**

The grout was pressurized by applying air pressure inside the conical tank through the air inlet on its base. The air pressure pushed the grout through the hollow pile shaft and the two grout nozzles into the surrounding soil during installation. In this study, two different grouting pressures 70 psi (480 kPa) and 100 psi (690 kPa) were used during PGHPs installation to investigate the effect of grouting pressure on the PGHP performance.

All grout mixtures were prepared by mixing Ordinary Portland Cement type 10 (OPC 10) with water at water/cement ratio of 0.45. Polycarboxylate based high range water reducing agent was added to the grout mixtures with ratio 1% of the cement weight to adjust its
flowability. The grout flowability was measured using flow cone test (ASTM C939) and found to be 17 seconds (reference flow time of water = 8 seconds).

3.4 Test setup and testing procedures

After installation, the grout was allowed to cure for one week. The pile was then subjected to monotonic axial uplift loading up to failure. At the end of the pullout test, the upward pile movement was determined, and a compression load was applied to bring the pile back to its original position. The pile was then tested under compression and lateral loading, respectively. This section describes the pile load test setup and the testing procedures of each loading scheme.

3.4.1 Pullout load test

The uplift test setup comprised of a main steel cross-beam resting on two I-beams with the test pile at the center point of its span. The cross-beam was composed of two C-channels placed back to back at a distance 30.0 mm apart. The upper and lower flanges of the two channels were connected using three 6.4 mm thick plates welded to the flanges. The two middle plates (i.e. at the top and bottom sides of the cross-beam) had a hole with diameter 30.0 mm at the center.

In the uplift load test, the load was applied using a hollow cylinder hydraulic jack located above the cross-beam and pushing against it. The applied load was measured using a calibrated load cell with 25.0 kN capacity. The load cell was connected from one side to the pile using a circular threaded collar bolted to the pile cap. While from the other side, the load cell was attached to the hydraulic jack using a threaded bar with a hexagonal nut at its end. The threaded bar had a diameter of 9.5 mm, a length of 600.0 mm, and it passed the cross-beam through the two middle plates. The pile head displacement was monitored using two independently supported linear variable displacement transducers (LVDTs) at the two steel handles and one at the pile head. The LVDTs, the load cell, and the strain gauges were connected to a data acquisition system that recorded the readings every 5.0 seconds. Figure 3.7 presents the uplift load test setup.
The uplift load test was conducted following the quick maintained load test procedure (ASTM D3689-07). The pile was loaded in increments equal to 5% of the anticipated ultimate capacity. Each load increment was maintained for 5 minutes. Load increments were added until failure occurred. The pile was then unloaded on two equal decrements. Each load decrement was maintained for 5 minutes. After applying the last decrement, the pile was left for 10 minutes to fully capture the rebound response.

### 3.4.2 Compression load test

The compression load was applied to the test pile through a hydraulic jack acting against a reaction frame using a hydraulic hand pump with capacity 10,000 psi (68.5 MPa). The reaction frame composed of three I-beams in the form of a main cross-beam and two columns. The cross-beam was connected to the column’s flange using six high strength bolts. The column on its turn was fixed to the ground using two anchors.

The applied load was measured using a calibrated load cell with capacity 100 kN. The load cell and the hydraulic jack were placed between the pile head and the cross-beam and adjusted to ensure centric loading. The pile settlement was monitored using four linear variable displacement transducers (LVDTs) at the four corners of the loading plate. The LVDTs were independently supported away from the loading system using two reference beams attached to the cylindrical steel tank. The load cell, the LVDTs, and the strain gauges
were connected to the data acquisition system. Figure 3.8 presents the compression load test setup.

![Compression load test setup](image)

**Figure 3.8: Compression load test setup**

The compression load test was carried out following the quick maintained test procedures (ASTM D1143-07). The pile was loaded in increments equal to 5% of the anticipated ultimate capacity. Each load increment was continued for 5 minutes. Load increments were added until the rate of the pile head settlement increased (i.e. failure approached), or the testing equipment reached their limit. The final load increment was kept for 10 minutes. The pile was then unloaded on five equal decrements. Each load decrement was maintained for 5 minutes. After applying the last decrement, the pile was left for 10 minutes to fully capture the rebound response.

3.4.3 Lateral load test

A simple setup was designed to apply lateral loading to the test pile and ensure the free head condition was satisfied. The setup was composed of an L-section steel beam fixed to the cylindrical tank via three steel clamps. The lateral load was applied to the pile shaft through a horizontal hydraulic jack pushing against the reaction beam. The applied load was measured using a calibrated load cell with 10.0 kN capacity. The load cell was attached from one side to the hydraulic jack through a threaded bar, while its other side rested on the reaction beam. The pile head movement was measured using three LVDTs supported on two
reference beams with their measuring tips pushing against the pile shaft. Figure 3.9 presents the lateral load test setup.

Figure 3.9: Lateral load test setup

The monotonic lateral load was applied in equal increments of 0.25 kN. Each load increment was maintained for 5.0 minutes. Load increments were added until a continuous increase in the pile movement was observed with no increase in the lateral load (i.e. failure occurred). The pile was then unloaded on three equal decrements. Each load decrement was maintained for 5.0 minutes. After applying the last decrement the pile was left for 10.0 minutes to capture the rebound response.

3.5 Shape of PGHPs

The lack of knowledge about the shape of PGHP hinders its use in many engineering applications. Thus, one of the main objectives of this study is to investigate the shape of PGHP under various testing conditions to better understand its behavior under different loading schemes (i.e. pullout, compression, and lateral loading). This section discusses the effect of the grout nozzle configuration, the soil density, and the grouting pressure on the PGHP shape, and provides a visual description for the created grout mass in each case.

3.5.1 Effect of nozzle configurations

The effect of grout nozzle configuration is investigated by presenting the results of three PGHP groups (each of three piles) installed in the predefined medium sand stratum (RD =
53.0%) to a depth of 0.8 m. Each group was installed with a grouting pressure of 70 psi (480 kPa) using one of the three nozzle configurations shown in Figure 3.2. After reaching the target depth, the injected grout volume was recorded, and the grouting efficiency, defined as the ratio between the volume of the created grout mass along the pile shaft and the total injected grout volume, was calculated. At the end of this section, the best nozzle configuration for PGHP construction was determined.

PGHP1 had a solid grout column as shown in Figure 3.10 with an average diameter of 97.0 mm. The shape of this grout column is attributed to the presence of the two grout nozzles above the pile helix. The nozzles created a cavity with diameter 78 mm around the pile shaft, which was filled with the pressurized grout during installation. The increase in the grout shaft diameter over that of the cavity created by the grout nozzles is attributed to pressurized grout similar to the installation of hollow bar micropiles (Maclean 2010, Elarabi & Alshareef 2014 and Derbi & El Naggar 2015). Comparing the average diameter of the exhumed piles (97 mm) and the presumed cavity diameter (78 mm), it is deduced that the pressurized grout enlarged the pile diameter by 24%.

![Image](image1.png)

**Figure 3.10: Shape of PGHP installed with the first nozzles configuration (PGHP1)**

To study the structural composition of PGHP1, the shaft was cut using a steel saw and the cross-section is presented in Figure 3.10b. The cross-section shows that the hollow shaft was filled and surrounded with neat cement grout to form a core 68 mm in diameter. This core was covered with a ring of cement-sand mortar, and a thin layer of grouted sand on its
surface. The total grout volume injected during the installation of PGHP1 was 18 litres, and the calculated grouting efficiency was 33%.

**Figure 3.11** shows the grout mass for PGHP2. The pile was comprised of a continuous spiral grout column with an average outer (rib) diameter of 156 mm. The spiral column had a solid grout core around the steel shaft with an average diameter of 109 mm. The solid core was formed due to filling the cavity created by the grout nozzles with the injected grout, and its diameter was enlarged by 40% due to the high grouting pressure. The spiral ribs of the grout column had an average thickness of 15.1 mm, an average height of 23.3 mm, and are attributed to the grout filling the void created by the helical plate as the pile advanced into the ground. Both the core and ribs were covered with a thin layer of grouted sand as shown in **Figure 3.11b**, which presents the cross-section of PGHP2. The total grout volume injected during the installation of PGHP2 varied between 26.0 and 28.0 Litres with an average grouting efficiency of 43%. The increase in the core diameter and the grouting efficiency for this group is attributed to the presence of the helical plate above the nozzles, which decreased the upward movement of the injected grout and forced it to move laterally.

![Figure 3.11](image.png)

**Figure 3.11: Shape of PGHP installed with the second nozzles configuration (PGHP2)**

**Figure 3.12** shows that the third nozzle configuration (PGHP3) formed a solid grouted sand column with an average diameter of 176 mm along the pile shaft. The formation of such large grout column is attributed to the upward inclination of the grout nozzles that loosened the soil beneath the pile helix. Besides, as the helix rotated and advanced into the sand bed, it mixed the grout with the loosened soil and displaced the mixture laterally to create the grout...
column. Figure 3.12b displays the cross-section of the pile shaft, which was composed of the hollow steel pipe filled with neat cement grout at the centre of a large cement-sand mortar column. The entire grout shaft was covered with a thin layer of grouted sand at the pile-soil interface. The total grout volume injected during the installation of PGHP3 was 25 Litres with a grouting efficiency of 78%.

![a- PGHP3 grout column  b- Cross-section through pile shaft](image)

**Figure 3.12: Shape of PGHP installed with the third nozzles configuration (PGHP3)**

Based on the observations discussed above, the third nozzle configuration was considered the best configuration for PGHPs construction under the described testing conditions. PGHP3 had the largest pile diameter, the best grouting efficiency, and the least clogging susceptibility during the pile installation due to the upward inclination of the grout nozzles. Moreover, PGHP3 was found to have the highest pullout and compression resistances as discussed later in Chapters 4 and 5.

3.5.2 Effect of sand relative density and grouting pressure

To elucidate the effect of soil density and grouting pressure on the PGHP shape, eight PGHPs were installed in the loose and dense sand beds with the recommended nozzle configuration (PGHP3) under two different grouting pressures; 70 psi (480 kPa), and 100 psi (690 kPa). It should be noted that from here forward a letter (D or L) and a number (70 or 100 psi) are added to the pile name to refer to the relative density of the sand bed and the grouting pressure used during the pile installation, respectively, i.e. D for dense sand and L for loose sand.

For piles installed in dense sand, a spiral grout column with solid core was observed along the pile shaft as shown in Figure 3.13. The two pile groups had the same outer (rib) diameter
(156 mm), but they varied in the ribs dimensions. For piles installed in dense sand with a grouting pressure of 70 psi (PGHP3-D70), an average ribs thickness of 14.2 mm, an average ribs height of 28.0 mm, and a solid core 100.0 mm in diameter were observed as shown in Figure 3.13a. While those installed with a grouting pressure of 100 psi (PGHP3-D100), had thicker (20.9 mm) and shorter (21.8 mm) grout ribs as presented in Figure 3.13b. On the other hand, PGHP3 installed in loose sand under a grouting pressure of 70 psi (PGHP3-L70) and 100 psi (PGHP3-L100) had solid grout columns with an average diameter of 181 mm and 192 mm, respectively, as presented in Figure 3.14. The shape and dimensions of the created grout mass along the PGHPs under discussion are summarized in Table 3.2.

Figure 3.13: Shaft of PGHP3 shaft installed in dense sand with grouting pressures of a) 70 psi, and b) 100 psi

Figure 3.14: Shaft of PGHP3 shaft installed in loose sand with a grouting pressure of a) 70 psi, and b) 100 psi

In conclusion, when the grout nozzles were placed above the pile helix, a solid grout column was formed along the PGHP shaft. The diameter of such column was the same as the cavity created by the grout nozzles multiplied by an appropriate enlargement factor to take into account the effect of the grouting pressure and the relative density of the surrounding soil.
On the other hand, when the grout nozzles were placed below the pile helix, continuous spiral column with a solid grout core was created along the pile shaft. The diameter of the core and the dimensions of the ribs on its surface (height and thickness) are functions in nozzle configuration, grouting pressure, and soil density. It should be noted that the solid grout column observed along the shaft of PGHP3 installed in medium and loose sand beds can be attributed to the large core diameter (i.e. the core diameter is equal to the total pile diameter).

Table 3.2: Shape of PGHPs shaft

<table>
<thead>
<tr>
<th>Pile</th>
<th>Shaft shape</th>
<th>Pile length (cm)</th>
<th>Shaft diameter (cm)</th>
<th>Ribs thickness (mm)</th>
<th>Ribs height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGHP1</td>
<td>Solid</td>
<td>80.0</td>
<td>10.0</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 1.1</td>
<td>Solid</td>
<td>80.0</td>
<td>9.8</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 1.2</td>
<td>Solid</td>
<td>80.0</td>
<td>9.4</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Average</td>
<td>Solid</td>
<td>80.0</td>
<td>9.7</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PGHP2</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.7</td>
<td>13.2</td>
<td>25.2</td>
</tr>
<tr>
<td>Pg 2.1</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>15.7</td>
<td>22.7</td>
</tr>
<tr>
<td>Pg 2.2</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>16.4</td>
<td>21.9</td>
</tr>
<tr>
<td>Average</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>15.1</td>
<td>23.3</td>
</tr>
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<td>PGHP3</td>
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<td>17.8</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 3.1</td>
<td>Solid</td>
<td>80.0</td>
<td>17.9</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 3.2</td>
<td>Solid</td>
<td>80.0</td>
<td>17.6</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Average</td>
<td>Solid</td>
<td>80.0</td>
<td>17.8</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PGHP3-L70</td>
<td>Solid</td>
<td>80.0</td>
<td>18.1</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 3.4-L70</td>
<td>Solid</td>
<td>80.0</td>
<td>18.2</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Average</td>
<td>Solid</td>
<td>80.0</td>
<td>18.1</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PGHP3-L100</td>
<td>Solid</td>
<td>80.0</td>
<td>19.3</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Pg 3.6-L100</td>
<td>Solid</td>
<td>80.0</td>
<td>19.1</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Average</td>
<td>Solid</td>
<td>80.0</td>
<td>19.2</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>PGHP3-D70</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>14.5</td>
<td>27.0</td>
</tr>
<tr>
<td>Pg 3.8-D70</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>13.8</td>
<td>29.0</td>
</tr>
<tr>
<td>Average</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>14.2</td>
<td>28.0</td>
</tr>
<tr>
<td>PGHP3-D100</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>20.1</td>
<td>22.2</td>
</tr>
<tr>
<td>Pg 3.10-D100</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>21.8</td>
<td>21.3</td>
</tr>
<tr>
<td>Average</td>
<td>Spiral</td>
<td>80.0</td>
<td>15.6</td>
<td>20.9</td>
<td>21.8</td>
</tr>
</tbody>
</table>

NA: Not applicable
By looking at the ribs dimensions of PGHP2, PGHP3-D70, and PGHP3-D100, it can be observed that the thickness of the grout ribs (T) is directly proportional to the grouting pressure (Pg) and inversely proportional to the relative density of the sand bed (R.D). On the contrary, the ribs height (H) was found to be directly proportional to R.D and inversely proportional to Pg. Figure 3.15 presents a relationship to determine the effect of both; Pg and R.D on the ribs dimensions (T and H). In order to eliminate the effect of the grouting pressure unit from the relation, the effect of R.D was considered in terms of the average overburden stress along the pile shaft (σ’). The average overburden stress can be calculated by multiplying the soil unit weight by half the pile length (i.e. 0.4 m). In other words, the higher the relative density, the higher the soil unit weight, and hence; the higher the σ’.

\[ T = 0.2478 \left( \frac{P_g}{\sigma'} \right) - 1.6332 \]  
\[ \text{eq. 3-3} \]

![Figure 3.15: Relation between the ribs dimensions and the grouting pressure/effective stress ratio](image)

The data presented in Figure 3.15 provides good relationships between the ribs dimensions (T and H) and the grouting pressure to the average overburden stress ratio (Pg/σ’). According to these relations, Equations 3-3 and 3-4 can be used to determine the thickness and the height of the spiral ribs, respectively. The diameter of the solid core can then be calculated by subtraction twice the ribs height from the outer shaft diameter of the pile (D_{ shaft}). It should be noted that these equations were developed based on the results of small scale PGHPs, thus; further verification is required before using them in estimating the dimensions of full-scale PGHPs.
\[ H = 35.58 - 0.1558 \left( \frac{P_g}{\sigma_{av\prime}} \right) \]  

where \( T \) and \( H \) are the ribs thickness and height in (mm), respectively. \( P_g \) is the grouting pressure, and \( (\sigma_{av\prime}) \) is the average effective overburden pressure along the pile length.
3.6 References


Chapter 4

Pullout Load Testing

4.1 Introduction

The recent years have witnessed an exponential increase in solar farms construction to meet the continuous growth in the electricity demand. Solar panels are generally subjected to complex loading scheme due to wind pressures. This loading scheme includes lateral loading, bending moment, and uplift forces. Currently, helical piles are the most popular foundation option for such an application. However, the construction industry is pursuing efficient construction techniques, and novel pile configurations to enhance the reliability and economic feasibility, as well as reducing the construction time of the solar panels support systems. In this chapter, the behavior of PGHP under pullout loading is investigated, and its performance is compared with that of conventional helical pile.

Pullout load tests were conducted on 5 un-grouted helical piles and 17 PGHPs constructed under controlled laboratory environment and the results are presented and discussed herein. The piles were installed in loose, medium, and dense sand beds to a depth of 0.8 m by applying manual torque to the pile head through the two steel arms welded to the pile shaft. For PGHPs, two different grouting pressures; 70 psi (480 kPa) and 100 psi (690 kPa) were used during the pile installation. After installation, the piles were subjected to uplift loading following the quick maintained test procedures (ASTM D3689-07). The primary objective of this chapter is to evaluate the performance characteristics of the PGHP under tensile load considering different installation conditions (i.e. grout nozzle configuration, soil relative density, and grouting pressure).

4.2 Effect of nozzle configurations

The effect of the nozzle configurations was investigated by comparing the uplift load test results of two un-grouted helical piles and nine PGHPs installed with the three nozzle configurations shown in Figure 3.2 (i.e. three piles/configuration). All piles were installed
in medium dense sand stratum to a depth of 0.8 m. Moreover, all PGHP groups (i.e. PGHP1, PGHP2, and PGHP3) were installed with the same grouting pressure (i.e. 70 psi = 480 kPa). The shape and dimensions of the different PGHPs were described in Section 3.5.

**Figure 4.1** presents the load-displacement curves for the un-grouted (conventional) helical piles. In general, each load-displacement curve can be divided into three regions: an initial linear segment, a nonlinear transitional zone followed by a plateau. The initial segment extended to a load of 5.5 kN and a corresponding displacement of 4.0 mm. While the nonlinear region continued to a load of 8.0 kN and an average displacement of approximately 10.3 mm, after which, a significant increase in the pile movement was observed with no increase in the load (i.e. failure occurred at 8.0 kN).

![Load-Displacement Curves](image)

**Figure 4.1:** Uplift Load-Displacement curves for the un-grouted helical piles installed in medium dense sand

Due to the small depth to helix diameter ratio (H/D_h = 5.3), the un-grouted pile behavior under uplift loading follows the shallow anchor response (Ghaly & Hanna 1992, and Zhang 1999). For helical anchors installed in sand to a relatively shallow depth, the failure surface is defined by a plane inclined to the vertical at an angle (θ) that extends from the outer edge of the helix to the ground surface (Mitsch and Clemence 1985) as illustrated in **Figure 4.2**, with θ = φ/4 to φ/2 (Meyerhof and Adams 1968).
Ghaly et al. (1991) suggested that the uplift capacity of shallow anchors is given by the summation of forces acting on the failure surface and the weight of the soil wedge within the failure zone. Accordingly the uplift capacity of shallow anchors is given by:

$$ Q_u = \frac{\pi}{2} \gamma H^2 k_{p'} \left[ \frac{D_h + H\tan\theta}{\cos\theta} \right] \tan\delta + \frac{\pi}{3} \gamma H (b^2 + r^2 + br) $$ \hspace{1cm} \text{eq. 4-1}$$

$$ K_{p'} = \frac{1 + \sin\delta}{1 - \sin\delta} $$ \hspace{1cm} \text{eq. 4-2}$$

where $Q_u$ is the ultimate pullout load, $\gamma$ is the soil unit weight, $H$ is the depth of the anchor, $k_{p'}$ is the modified coefficient of passive earth pressure, $D_h$ is the helix diameter, $\theta$ is the surface inclination angle of the inverted failure cone with respect to the vertical, $\delta$ is the average mobilized angle of shearing resistance on the failure plane, $b$ is the helix radius, and $r$ is the radius of the influence failure circle on the sand surface.

Equations 4-1 and 4-2 require determining the mobilized angle of shearing resistance ($\delta$). Ghaly et al. (1991) developed a relationship between the angle of friction between soil particles ($\phi$), the relative depth ratio ($H/D_h$), and $\delta/\phi$ ratio, which is shown in Figure 4.3. For the helical piles under discussion (i.e. $H/D_h = 5.3$, and $\phi = 37^\circ$), the $\delta/\phi$ ratio was found to be 0.63. By substituting in Equations 4-1 and 4-2, the $\theta/\phi$ was calculated as 0.38 from the back analyses of the un-grouted pile resistance (i.e. 8.0 kN).
Figure 4.3: Relationship between $\delta/\varphi$ ratio, relative depth ratio, and angle of friction between soil particles (After Ghaly et al., 1991)

On the other hand, PGHPs were instrumented with three levels of strain gauges (2 gauges/level) located at 1.0, 39.0, and 79.0 cm below the surface of the soil bed. The load transfer mechanism along the pile shaft can be determined by measuring the axial load transferred at each strain gauge elevation ($P_{zi}$) using:

$$P_{zi} = (\varepsilon_i E_p) A_p$$

where $\varepsilon_i$ is the measured strain at level (i), $E_p$ is the elastic modulus of the pile shaft, and $A_p$ is the cross-section area of the pile shaft.

The elastic modulus of the pile shaft ($E_p$) was obtained from Equation 4-3 by equating the load at the first level of strain gauges to the total applied load measured by the load cell and considering the measured strain. The axial force distribution curves along the pile shaft were then established using the measured strain at different elevations. Figure 4.4, Figure 4.5, and Figure 4.6 show the axial load distribution along PGHP1, PGHP2, and PGHP3, respectively. These load distribution curves demonstrate that the uplift resistance of PGHPs was primarily due to the shaft friction along the pile-soil interface. Thus, their pullout
capacities were fully mobilized at low displacement levels. Figure 4.4 also indicates that about 17.2% and 16.7% of the load applied to Pg 1.1 and Pg 1.2 transferred to the pile helix, respectively, because the helix diameter was larger than the grout shaft diameter. Nonetheless, PGHP1 can still be considered as a friction pile.

![Figure 4.4: Axial load distribution along PGHP1 during uplift loading](image)

![Figure 4.5: Axial load distribution along PGHP2 during uplift loading](image)

Figure 4.7 presents the displacement curves of PGHP2 piles under uplift loading. The curves show a linear increase in displacement up to a load of 5.0 kN and a pile movement of 1.0 mm (∼ 0.57% of D_{shaft}). The displacement then increased in nonlinear fashion up to a displacement of 2.5 mm (∼ 1.6% of D_{shaft}) at an applied load of 7.2, 7.5, and 7.8 kN for Pg 2.1, Pg 2.2, and Pg 2.3, respectively. At this point, slippage failure (i.e. pullout capacity) of PGHP2 occurred and a decrease in the applied load was observed. This reduction in the
measured load at the end of the test is attributed to the difficulty of maintaining a constant load during slippage failure while using a hydraulic hand pump.

Figure 4.6: Axial load distribution along PGHP3 during uplift loading

Figure 4.7: Uplift Load-Displacement curves for PGHP2
Similar behavior was observed for PGHP3 with an average pullout capacity of 9.6 kN at a pile head displacement of 4.5 mm (2.6% of $D_{\text{shaft}}$) as illustrated in Figure 4.8.

**Figure 4.8: Uplift Load-Displacement curves for PGHP3**

Figure 4.9 shows the load-displacement curves of PGHP1 piles. It is noted from the figure that the initial linear segment extended to an upward movement of 0.5 mm ($\approx 0.52\%$ of $D_{\text{shaft}}$) and a pullout load of 3.5 kN for Pg 1.1 and Pg 1.2, while for Pg 1.3, the corresponding applied load was 3.0 kN. The three curves then exhibited a nonlinear response until a maximum uplift capacity of 5.8 kN was reached at a pile head displacement equal to 3.5 mm ($\approx 3.5\%$ of $D_{\text{shaft}}$).

**Figure 4.9: Uplift Load-Displacement curves for PGHP1**
The ratio between upward displacement at failure ($S_f$) and the grout shaft diameter ($D_{shaft}$) for PGHP2 and PGHP3 was found to vary between 1.6% and 2.6% with an average of approximately 2.1%. At this level of displacement, the friction resistance would have been mobilized fully along the pile shaft (Randolph 2003; and Fleming et al. 2009). The increase in the $S_f/D_{shaft}$ ratio and the lower load reduction rate during slippage failure observed for PGHP1 is attributed to the notable contribution of the pile helix to the pullout resistance as explained by the load distribution curves shown in Figure 4.4.

Table 4.1 summarizes the uplift capacity ($Q_u$) of the conventional helical piles and PGHPs in the medium dense sand. It also presents the failure displacement ($S_f$) required by each pile to develop its maximum resistance and its ratio to the effective pile diameter ($D_{eff}$). For PGHP, the uplift load was primarily resisted by the shaft friction, thus; $D_{eff}$ is equal to the grout shaft diameter ($D_{shaft}$). Since the uplift capacity of un-grouted helical pile is mainly due to its helix, $D_{eff}$ for un-grouted piles is considered as the helix diameter ($D_h$).

### Table 4.1: Uplift capacity of piles in medium dense sand

<table>
<thead>
<tr>
<th>Pile group</th>
<th>Pile</th>
<th>$Q_u$ (kN)</th>
<th>$S_f$ (mm)</th>
<th>$S_f / D_{eff}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Un-grouted helical piles</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pung 1</td>
<td>8.0</td>
<td>11.0</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>Pung 2</td>
<td>8.0</td>
<td>9.5</td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>8.0</strong></td>
<td><strong>10.3</strong></td>
<td><strong>6.8</strong></td>
<td></td>
</tr>
<tr>
<td><strong>PGHP1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pg 1.1</td>
<td>5.7</td>
<td>3.0</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>Pg 1.2</td>
<td>5.8</td>
<td>4.0</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>Pg 1.3</td>
<td>6.0</td>
<td>4.0</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>5.8</strong></td>
<td><strong>3.7</strong></td>
<td><strong>3.7</strong></td>
<td></td>
</tr>
<tr>
<td><strong>PGHP2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pg 2.1</td>
<td>7.2</td>
<td>2.2</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Pg 2.2</td>
<td>7.5</td>
<td>2.7</td>
<td>1.7</td>
<td></td>
</tr>
<tr>
<td>Pg 2.3</td>
<td>7.8</td>
<td>2.5</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>7.5</strong></td>
<td><strong>2.5</strong></td>
<td><strong>1.6</strong></td>
<td></td>
</tr>
<tr>
<td><strong>PGHP3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pg 3.1</td>
<td>9.7</td>
<td>5.0</td>
<td>2.8</td>
<td></td>
</tr>
<tr>
<td>Pg 3.2</td>
<td>9.7</td>
<td>4.5</td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>Pg 3.3</td>
<td>9.5</td>
<td>4.0</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>9.6</strong></td>
<td><strong>4.5</strong></td>
<td><strong>2.6</strong></td>
<td></td>
</tr>
</tbody>
</table>

Inspecting Table 4.1, a reduction of 27.5% and 6.3% in the uplift capacity of PGHP1 and PGHP2, respectively, was observed compared to the conventional helical piles. While for
PGHP3, the uplift capacity was found to increase by approximately 20%. The higher uplift capacity exhibited by PGHP3 is attributed to its larger shaft diameter as previously discussed in Section 3.5. Although two of the three PGHP groups investigated herein (i.e. PGHP1 and PGHP2) had lower uplift capacity than the conventional helical piles, they developed their maximum capacities at lower displacement levels (i.e. friction piles), which indicates better performance of PGHP compared to the conventional helical pile under design loads.

Poulos and Davis (1980) proposed a formula to estimate the unit shaft resistance ($f_s$) developed at the pile-soil interface during uplift loading of friction piles, i.e.:

$$f_s = (Q_u - w)/A_s$$

where ($Q_u$) is the ultimate pullout capacity, ($w$) is the pile weight, and ($A_s$) is the surface area of the pile shaft.

The weight of each PGHP was measured using a crane equipped with a load cell. The crane used to extract the piles from the sand bed at the end of the experimental program as shown in Figure 4.10.

![Image](image1.png)

a- During extraction  
b- After extraction

Figure 4.10: Extraction of PGHP from the sand bed a) during extraction and b) after extraction
When the pile was removed from the soil, the load cell reading was recorded as the pile weight. The average pile weight was found to be 0.19, 0.34, and 0.42 kN for PGHP1, PGHP2, and PGHP3, respectively. **Equation 4-4** was then used to calculate the average $f_s$, which was found to be 19.7, 18.3, and 21.9 kPa. Results revealed that the grout nozzles configuration has a minor effect on the unit shaft resistance, and considering an average value of 20 kPa ± 10% is a reasonable estimate for PGHPs installed to a depth of 0.8 m into a medium dense sand layer with a grouting pressure of 70 psi (480 kPa).

### 4.3 Effect of relative density and grouting pressure

The PGHP3 was found to have the largest pile diameter and the highest uplift capacity among the three nozzles configuration considered in this study. Thus, PGHP3 was chosen for further parametric studies to evaluate the effect of other installation parameters on the uplift behavior of PGHPs.

Pile load tests were conducted on three ungrouted helical piles and eight PGHP3 piles installed to a depth of 0.8 m in loose and dense sand beds and the results are presented herein. PGHP3 piles were installed with two different grouting pressures 480 kPa (70 psi) and 690 kPa (100 psi) to investigate the grouting pressure effect on the pile performance. It should be mentioned that Letter (D or L) and a number (70 or 100) are added to the pile name to refer to the relative density of the sand bed and the grouting pressure used during installation.

**Figure 4.11** presents the uplift load-displacement curves for the un-grouted helical piles. The curves can be divided into three main regions: an initial linear segment, a nonlinear transitional zone followed by a plateau. For the conventional helical pile installed in loose sand (Pung 3-L), the first linear segment continued to a load of 5.5 kN and a displacement of 7.5 mm, while its non-linear response extended to 6.5 kN and 12.0 mm. Beyond which, the plateau was observed. Thus, the uplift capacity of Pung 3-L was found to be 6.5 kN.

For helical piles in dense sand, the initial linear portion extended to a load of 6.5 kN and a displacement of 3.0 and 4.0 mm for Pung 4-D and Pung 5-D, respectively. This was followed by non-linear zone to a pullout load of 8.5 kN and an upward movement of 7.5 and 9.5 mm,
after which a continuous increase in the pile displacement was observed at the same load (i.e. failure occurred at 8.5 kN).

![Load-displacement curves for conventional helical piles](image)

**Figure 4.11: Uplift load-displacement curves for conventional helical piles installed in dense and loose sand strata**

The interpreted pullout capacities of Pung 3-L, Pung 4-D, and Pung 5-D verifies the shallow anchor response observed for Pung 1 and Pung 2. The failure surface extended to the ground surface at an angle \( \theta = 0.36\phi \) and \( 0.32\phi \) for piles installed with a depth to diameter ratio \( (H/D_h = 5.3) \) into the loose \((\phi = 34^\circ)\) and dense \((\phi = 41^\circ)\) sand beds, respectively. The \( \theta \) values were back calculated from the un-grouted pile resistance using **Equations 4-1 and 4-2** considering a \( \delta/\phi \) ratio = 0.66 and 0.59 as elaborated by **Figure 4.3**.

Ghaly and Hanna (1992) suggested that the critical depth to helix diameter ratio \((H/D_h)_{cr}\) at which shallow anchor behavior is expected increases with increasing the sand relative density as illustrated by **Table 2.1**. Therefore, the shallow anchor behavior for piles installed to the same \((H/D_h)\) is more pronounced in denser soil strata. This explains the decrease in the failure displacement \( (S_f) \) from 12.0 mm for the un-grouted pile in loose sand to an average of 10.3 and 8.5 mm for those installed in medium and dense sand, respectively.

For PGHP3 piles installed in dense sand, the pullout load-displacement curves are presented in **Figure 4.12**. As can be observed from the figure, the displacement curves are characterized by a stiffer initial response up to a load of 6.5 kN and a corresponding displacement of 1.0 mm. The curves then experienced a nonlinear increase to a maximum pullout resistance beyond which slippage failure occurred. For piles installed with a grouting pressure of 70 psi
(PGHP3-D70), failure happened at an upward movement of 3.5 mm (2.2% of D_{shaft}) and an applied tensile load of 9.8 and 9.0 kN for Pg 3.6-D70 and Pg 3.7-D70, respectively. When the grouting pressure was increased to 100 psi (PGHP3-D100), the pile head displacement at failure became 4.5 mm (2.9% of D_{shaft}) at a maximum applied load of 12.5 kN.

![Uplift load-displacement curves for PGHP3 installed in Dense sand with different grouting pressures](image)

**Figure 4.12**: Uplift load-displacement curves for PGHP3 installed in Dense sand with different grouting pressures

To investigate the pullout load transfer mechanism, the strain gauges readings were used to develop the axial load distribution curves along PGHP3-D70 and PGHP3-D100 as presented in Figure 4.13 and Figure 4.14, respectively. The load distribution curves show that the applied tensile load was primarily supported by the friction resistance along the pile shaft, which explains the full mobilization of the uplift capacity at low displacement level as illustrated in Figure 4.12.

![Axial load distribution along PGHP3-D70](image)

**Figure 4.13**: Axial load distribution along PGHP3-D70
Figure 4.14: Axial load distribution along PGHP3-D100

Although both PGHP3-D100 and PGHP3-D70 had the same pile diameter (156 mm) and were installed in dense sand with the same properties, the 33.0% increase in the Pullout resistance of PGHP3-D100 over that of PGHP3-D70 can be attributed to the higher grouting pressure used during the pile construction. During the installation of PGHP3, the grout nozzles created a cavity that was filled with the pressurized grout during installation. The high grouting pressure formed a solid core with a larger diameter than the cavity created by the grout nozzles. Moreover, as the pile advanced into the sand bed, the helix mixed the injected grout with the surrounding sand and displaced it laterally to fill the voids created by the helical blade. The radial soil displacement due to the high grouting pressure and the pile helix is denoted herein as “cavity expansion”. Raising the grouting pressure from 70 psi to 100 psi increased the cavity expansion, which caused an increase in the radial stresses (σh) around the pile, and hence; higher friction resistance at the pile-soil interface was observed.

Figure 4.15 presents the displacement curves of PGHP3 piles installed in loose sand under uplift loading. The curves extended linearly to an upward movement of 0.5 mm and an applied load of 4.5 and 5.0 kN for PGHP3-L70 and PGHP3-L100, respectively. For PGHP3-L70, slippage failure occurred at 2.5 mm (1.4% of Dshaft) and an applied load of 6.7 kN. While for PGHP3-L100, the pullout capacity increased to approximately 8.5 kN at a total pile displacement equal to 3.5 mm (1.8% of Dshaft). An increase of 27% was observed in the pullout capacity of PGHP3-L100 over that of PGHP3-L70. This increase is attributed to two main reasons. First, increasing the grout shaft diameter from 181 mm to 192 mm. Second,
the higher grouting pressure increased the lateral soil displacement (i.e. cavity expansion) during the pile installation resulting in more radial stresses at the pile-soil interface.

![Diagram](image)

**Figure 4.15:** Uplift load-displacement curves for PGHP3 installed in Loose sand with different grouting pressures

The axial load distribution curves presented in **Figure 4.16** and **Figure 4.17** show that the shaft friction represents the entire resistance of PGHP3-L70 and PGHP3-L100 to pullout loading. Thus, the maximum uplift capacity is reached, and slippage failure occurred at small displacements as illustrated in **Figure 4.15**.

![Diagram](image)

**Figure 4.16:** Axial load distribution along PGHP3-L70

The pullout load testing results of all piles installed in dense and loose sand are summarized in **Table 4.2**. The table also presents the ratio between the failure displacement (\(S_f\)) and the
effective pile diameter ($D_{eff}$), as well as the ratio between the uplift resistance of the different PGHPs ($Q_{gr}$) and that of the conventional helical piles installed in the same sand strata ($Q_{ung}$).

![Figure 4.17: Axial load distribution along PGHP3-L100](image)

Table 4.2: Pullout resistance summary for piles installed in dense and loose sand

<table>
<thead>
<tr>
<th>Pile group</th>
<th>Pile name</th>
<th>$Q_u$ (kN)</th>
<th>$S_f/D_{shaft}$ (%)</th>
<th>$Q_{gr}/Q_{ung}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pung-L</td>
<td>Pung 3-L</td>
<td>6.5</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td>Pung-D</td>
<td>Pung 4-D</td>
<td>8.5</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pung 5-D</td>
<td>8.5</td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>8.5</td>
<td>5.7</td>
<td>1.0</td>
</tr>
<tr>
<td>PGHP3-D70</td>
<td>Pg 3.6-D70</td>
<td>9.8</td>
<td>2.2</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>Pg 3.7-D70</td>
<td>9.0</td>
<td>2.2</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>9.4</td>
<td>2.2</td>
<td>1.11</td>
</tr>
<tr>
<td>PGHP3-D100</td>
<td>Pg 3.8-D100</td>
<td>12.5</td>
<td>2.9</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td>Pg 3.9-D100</td>
<td>12.7</td>
<td>3.2</td>
<td>1.49</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>12.6</td>
<td>3.0</td>
<td>1.48</td>
</tr>
<tr>
<td>PGHP3-L70</td>
<td>Pg 3.4-L70</td>
<td>6.9</td>
<td>1.4</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Pg 3.5-L70</td>
<td>6.5</td>
<td>1.7</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>6.7</td>
<td>1.5</td>
<td>1.03</td>
</tr>
<tr>
<td>PGHP3-L100</td>
<td>Pg 3.10-L100</td>
<td>8.2</td>
<td>2.1</td>
<td>1.26</td>
</tr>
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<td>Pg 3.11-L100</td>
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<td>2.1</td>
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<td></td>
<td>Average</td>
<td>8.5</td>
<td>2.1</td>
<td>1.31</td>
</tr>
</tbody>
</table>

The average unit shaft resistance ($f_s$) at the pile-soil interface for each of the four PGHP groups shown in Table 4.2 can be computed using Equation 4-4. The weight of each pile
group was measured while extracted from the sand bed and was found to be 0.28, 0.36, 0.43, and 0.46 kN, which resulted in an $f_s$ equal to 23.3, 31.2, 13.8, and 16.7 kPa for PGHP3-D70, PGHP3-D100, PGHP3-L70, and PGHP3-L100, respectively. By comparing these values with the average $f_s$ (i.e. 20 kPa) observed for PGHP1, PGHP2, and PGHP3, it can be concluded that the unit shaft resistance is directly proportional to the grouting pressure and the relative density (R.D) of the surrounding soil.

The higher $f_s$ observed for PGHPs installed with higher grouting pressure is related to the increase in the radial stresses around the pile due to the larger cavity expansion occurred during installation. While increasing the $f_s$ with the increase in the R.D of the sand bed is attributed to the higher mechanical (i.e. Young’s Modulus) and shear strength (i.e. friction angle) parameters of the denser sand strata.

By inspecting Table 4.2, the pullout resistance of PGHP3 is mobilized fully at an upward movement varies between 1.5% and 3.0% of the average pile diameter, which is in good agreement with the observations of Randolph (2003) and Fleming et al. (2009). Moreover, the $Q_{gr}/Q_{ung}$ ratio was found to increase with increasing the PGHP3 diameter and/or the grouting pressure used during installation. On the other hand, no clear relationship could be obtained between $Q_{gr}/Q_{ung}$ and the R.D of the surrounding sand. This weak relation is ascribed to two contradicted factors. Increasing the R.D of the sand bed increased the $f_s$ at the pile-soil interface, but at the same time, it decreased the PGHP diameter as described in Chapter 3. For PGHPs installed with a grouting pressure of 70 psi, $Q_{gr}/Q_{ung}$ ratio was found to be 1.03, 1.2, and 1.11 for PGHP3-L70, PGHP3, and PGHP3-D70, respectively.

4.4 Analytical estimation of pullout capacity

PGHP involves grout injection under high pressure during the installation of helical piles. Currently, no guidelines are available to evaluate its pullout capacity considering its unique method of installation. Thus, the applicability of existing methods for pile design are discussed, and relevant design guidelines are proposed to account for the effects of PGHP’s distinctive features on the uplift capacity.
The pile load tests discussed herein show that the pullout capacity of PGHP comes from the friction resistance along its grout shaft. The unit shaft resistance \( f_s \) for piles installed in sand can be calculated using the \( \beta \)-method (Burland, 1973), i.e.

\[
f_s = k_s \sigma_v' \tan \delta = \beta \sigma_v'
\]

where \( k \) is the coefficient of lateral earth pressure, \( \delta \) is the interfacial friction angle at the soil-pile interface, \( \sigma_v' \) is the average effective vertical stress along the pile length, and \( \beta \) is the friction capacity factor.

For piles with a rough surface similar to the PGHPs, the angle of pile-soil friction (\( \delta \)) can be taken as the soil friction angle (\( \varphi \)) as recommended by Kulhawy (1984) and Tomlinson & Woodward (2008). Meanwhile, the lateral earth pressure coefficient (\( k_s \)) is affected by the pile configuration, the initial soil condition, and the pile installation technique. Fleming et al. (2009) suggested \( k_s = 0.7, 0.9, \) and 1.2 for bored piles, continuous-flight auger piles (CFA), and driven piles in sandy soils, respectively.

For displacement piles, Tomlinson and Woodward (2008) recorded an increase in \( k_s \) value due to the increase in the radial stresses at the pile-soil interface. For small displacement piles (e.g., H piles and pipe piles), the ratio between \( k_s \) and the coefficient of lateral earth pressure at rest (\( k_o \)) varies between 0.75 and 1.25. While for large displacement piles (e.g., driven closed-end and solid cross-section piles), \( k_s \) can be twice the value of \( k_o \) or even higher. Bowles (1996) found that \( k_s \) is both site and pile type specific, and he summarized some of \( k_s \) values obtained from the analysis of several pile load testing programs as shown in Table 4.3.

Bhushan (1982) and Vesic (1970) developed some other methods to determine the unit shaft resistance of piles in cohesionless soils depending on the soil relative density (R.D) as given by Equations 4-6 and 4-7, respectively

\[
\beta = 0.18 + 0.0065 \text{R.D}
\]

where RD is the relative density of the surrounding sand (%). This equation is suitable for large displacement piles.
\( f_s = x_o (10)^{1.54 \cdot R \cdot D^4} \) \hspace{1cm} \text{eq. 4-7}

where \( x_o \) = 8.0 for large volume displacement piles, and 2.5 for small displacement piles.

### Table 4.3: Summary of lateral earth pressure coefficients \( k_s \) (after Bowles 1996)

<table>
<thead>
<tr>
<th>Source</th>
<th>Pile type</th>
<th>H-pile</th>
<th>Pipe pile</th>
<th>Precast concrete pile</th>
<th>Timber pile</th>
<th>Tapered pile</th>
<th>Tension tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mansur &amp; Hunter (1970)</td>
<td></td>
<td>1.4 to 1.9</td>
<td>1.2 to 1.3</td>
<td>1.45 to 1.6</td>
<td>1.25</td>
<td></td>
<td>0.4 to 0.9</td>
</tr>
<tr>
<td>Tavenas (1971)</td>
<td></td>
<td>0.5</td>
<td></td>
<td>0.7</td>
<td></td>
<td>1.25*</td>
<td></td>
</tr>
<tr>
<td>Ireland (1957)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.11 to 3.64</td>
</tr>
<tr>
<td>API (1984)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

(*) Tapered timber
(+ ) Step-tapered tension: 3.64 is no accepted as the test was made in saturated soil and high value may have resulted from water suction.
(#) Unplugged pipe

For short drilled concrete shafts with a maximum depth of 5.0 to 6.0 m, Bowles (1996) found that \( k_s \) is a function of the pile diameter. For piles with diameter < 300 mm, \( k_s \) is equal to the coefficient of active earth pressure (\( k_a \)). For piles with diameter = 300 – 600 mm, \( k_s \) can be taken as the average between \( k_a \) and \( k_o \). For piles with diameter > 600 mm and those installed with concrete having a slump > 70 mm, \( k_s \) can be estimated using:

\[ k_s = K_a + k_o + k_p \]

where \( k_a, k_o, \) and \( k_p \) are the coefficients of active, at rest, and passive earth pressures, respectively.

The smaller \( k_s \) values for small diameter piles is because the wet concrete does not develop much lateral pressure against the shaft soil, whereas the larger diameter piles and those installed with concrete having slump test > 70 mm allow full lateral pressure from the wet concrete to develop so that a relatively high interface pressure is obtained.

Ismael et al. (1994) studied the uplift behavior of short bored piles in cemented desert sand. They studied two piles with same diameter of 0.3 m and two different length, 3.3 m and 5.3
m. The bottom 2 m of the long pile penetrated an un-cemented sand layer. From the back analysis of the load transferred to this soil layer, $k_s$ was found to be 1.46. The estimated $k_s$ value is in good agreement with that estimated by Equation 4-8, which can be attributed to the small depth to diameter ratio of that part of the pile (i.e. $H/D = 6.7$).

As can be observed from the literature, there is no agreement on the $k_s$ value that should be used to determine the friction resistance at the pile-soil interface due to the little consideration to changes in the soil parameters and/or effective confining pressure with depth as well as obtaining a single $k_s$ value for the full pile length rather than dividing the pile shaft into sections ($\Delta L$).

It is believed that Equation 4-8 provides good estimate for $k_s$ of the test piles due to two reasons. First, the PGHPs were installed by injecting flowable grout, which satisfies the full contact condition with the soil shaft as recommended by Bowles (1996) for short concrete piles with a slump test > 70 mm. Second, the PGHPs has a smaller $H/D$ ratio (4.2 to 5.1) than that investigated by Ismael et al. (1994). Thus, as $k_s = \frac{K_n + k_o + k_p}{3}$ was back calculated from uplift load test of a pile with $H/D = 6.7$, it can be used with piles having smaller $H/D$.

It should be noted that Equation 4-8 was initially developed for short bored piles (i.e. non-displacement piles). Therefore, it underestimates the unit shaft resistance ($f_s$) of the PGHPs discussed herein. The grouting pressure used during PGHP installation is within the range used for the construction of micropiles (Type B and Type E). Thus, the high $f_s$ observed for PGHPs can be attributed to the same cavity expansion theory used to explain the high ($\alpha$-bond) of micropiles. Screwing PGHP into the ground allows the grout nozzles to create a cavity around the steel pile shaft. At the same time, the injection of pressurized grout and the helix rotation displace the soil laterally and increase the lateral earth pressure at the pile-soil interface. The grouting pressures of 480 and 690 kPa are well above the yield cavity expansion pressure that can cause significant effective stresses (Moon and Lee 2016). It can also change the direction of the major principal stress from the vertical to the lateral direction. Therefore, a correction factor should be applied to Equation 4-8 to account for the effect of PGHP installation method on its shaft resistance.
Levacher and Sieffert (1984) studied the effect of different installation techniques (bored piles, driven piles, and vibro-driven piles) on the uplift resistance of model piles in sand. They introduced what is called a placement method coefficient ($k_{mo}$) to take into account the installation effect on the pile shaft resistance. Following the same concept, Equation 4-5 can be rewritten as shown in Equation 4-9 to consider the effect of PGHP installation method on its shaft resistance.

$$f_s = k_{mo} \frac{k_a + k_o + k_p}{3} \sigma_v \tan \varphi$$

Eq. 4-9

where $k_{mo}$ is the placement method coefficient, $\sigma_v$ is the average effective stress along the pile shaft. $k_a$, $k_o$, and $k_p$ are the coefficients of active, at rest, and passive earth pressures, respectively, and $\varphi$ is the friction angle between soil particles.

Table 4.4 compares the $f_s$ values estimated by the $\beta$-method (i.e. $k_s$ form Equation 4-8) and those obtained for PGHPs. Besides, it presents the back-figured $k_{mo}$ values to account for the PGHP installation effect on its shaft resistance.

**Table 4.4: Placement factor coefficient for the different pile groups**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Pile Group</th>
<th>Measured $f_s$ (kPa)</th>
<th>Estimated $f_s$ (kPa)</th>
<th>$k_{mo}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>PGHP3-L70</td>
<td>13.8</td>
<td>6.5</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>PGHP3-L100</td>
<td>16.7</td>
<td>6.5</td>
<td>2.6</td>
</tr>
<tr>
<td>Medium sand</td>
<td>PGHP1</td>
<td>20.0</td>
<td>8.35</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>PGHP2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>PGHP3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense sand</td>
<td>PGHP3-D70</td>
<td>23.3</td>
<td>11.7</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>PGHP3-D100</td>
<td>31.2</td>
<td>11.7</td>
<td>2.7</td>
</tr>
</tbody>
</table>

Results revealed that the placement method coefficient ($k_{mo}$) is slightly affected by the relative density of the sand bed. For PGHPs installed with a grouting pressure of 70 psi (480 kPa), $k_{mo}$ was found to vary between 2.0 and 2.4. On the other hand, $k_{mo}$ was found to increase with the grouting pressure. Increasing the grouting pressure from 70 psi (480 kPa)
to 100 psi (690 kPa) increased the $k_{mo}$ value by 24% and 35% for PGHPs installed in loose and dense sand, respectively.

For design purposes, $k_{mo} = 2.0$ can be used to conservatively estimate the unit shaft resistance of model PGHP (H/D = 4.2 to 5.1) installed with a grouting pressure of 70 psi under uplift loading. For grouting pressure = 100 psi, $k_{mo}$ increased to 2.6 due to the larger cavity expansion, and the higher radial stresses developed during installation. In other words, the mobilized $f_s$ along the shaft of a displacement pile (i.e. PGHP) is 2.0 to 2.6 times that acting at the pile-soil interface of a non-displacement pile (i.e. bored pile) having the same dimensions. This is in good agreement with Vesic (1970) and Meyerhof (1976) who found that $k_s$ for driven pile (i.e. large displacement pile) was approximately 3.2 and 3.6 times that of the bored pile, respectively.

In conclusion, PGHP is an innovative deep foundation option that involves grout injection under high pressure during the installation of a helical pile with a hollow pipe shaft. The grout is injected into the surrounding soil through two grout nozzles welded to the pile shaft. The pullout capacity of PGHP was found to depend mainly on the friction resistance at the pile-soil interface. Thus, it was fully mobilized at lower displacement level compared to the conventional helical pile that experienced shallow anchor response and depended on its helix to resist the applied load.

In this study, the PGHPs were installed with three different nozzles configuration (i.e. PGHP1, PGHP2, and PGHP3) to determine the best configuration for PGHP construction. Results revealed that the nozzles configuration has a minor effect on the unit shaft resistance developed at the pile-soil interface, however; the third configuration (i.e. PGHP3) created the largest pile diameter, and hence; it had the highest pullout capacity.

The innovative installation technique significantly improved the unit shaft resistance at the pile surface. The lateral expansion of the cavity created by the two grout nozzles under the effect of the pressurized grout increased the lateral earth pressure around the pile and changed the direction of the major principal stress from the vertical to the horizontal direction. Thus, to analytically estimate the pullout capacity of PGHPs, a modification has been applied to one of the existing methods to account for the unique features of PGHP installation.
technique. The grout-ground bond of PGHPs can be estimated using the β-method given by Burland (1973) for bored piles multiplied by a correction factor (i.e. placement method coefficient, k\text{mo}) to account for the installation effect on the surrounding soil. This placement method coefficient was found to vary with the grouting pressure. For PGHPs installed with a grouting pressure of 70 psi, k\text{mo} of 2.0 can be used. While for those installed with a grouting pressure of 100 psi, the k\text{mo} value increased to 2.6. Finally, further studies are still required to verify and investigate the pullout behavior of PGHP under different testing conditions.
4.5 References


Chapter 5

Compression Pile Load Testing

5.1 Introduction

PGHP is expected to be an efficient deep foundation option. However, the lack of knowledge regarding its behavior under compression loading hinders its use in many engineering applications. Thus, the primary objective of this chapter is to investigate the performance of PGHP under monotonic compression loading in order to achieve better understanding of its response considering different testing conditions (i.e. grout nozzle configuration, soil relative density, and grouting pressure).

This chapter documents the compression pile load tests of 5 un-grouted helical piles and 17 PGHPs constructed and tested under controlled laboratory environment. The piles were installed in loose, medium, and dense sand strata by applying manual torque to the pile head through two steel arms welded to its shaft. For PGHPs, two different grouting pressures: 480 kPa (70 psi) and 690 kPa (100 psi) were used for the pile installation. The load tests were conducted after 8 days from installation. The compression load tests were carried out following the quick maintained test procedures (ASTM D1143-07). The pile was loaded in increments equal to 5% of the anticipated ultimate capacity. Each load increment was continued for 5 minutes. Load increments were added until the rate of the pile head settlement increased (i.e. failure approached), or the testing equipment reached their limit. The final load increment was kept for 10 minutes.

The load-settlement curves were interpreted to determine the ultimate capacity values. The maximum pile capacity is conceptually reached when a rapid increase in the pile settlement occurs at constant or slightly increased load, i.e. plunging failure (Hirany and Kulhawy 2002). However, compression pile load tests do not always exhibit that behavior due to the practical limitations of the test setup and loading equipment. For the pile load tests presented herein, the load-displacement curves did not exhibit clear plunging failure, rather the
response exhibited three regions: an initial linear-elastic segment; a non-linear transitional zone; and a final linear segment with a lower slope.

5.2 Effect of nozzle configurations

The effect of the nozzle configuration on the pile behavior was investigated by comparing the compression load test results of two un-grouted helical piles and nine PGHPs installed with three different nozzle configurations as shown in Figure 3.2. The three PGHP groups (PGHP1, PGHP2, and PGHP3) were installed to a depth of 0.8 m in the medium dense sand with a grouting pressure of 70 psi (480 kPa). The shape and dimensions of the different PGHP groups were described in Section 3.5.

5.2.1 Load-displacement curves

Figure 5.1 presents the load-displacement curves for the un-grouted piles. By Figure 5.1, it is noted that the initial linear segment of the curve extended to a load = 5.0 kN and a displacement = 1.2 mm, and the final linear portion started at about 18.0 kN, which corresponded to an average pile movement = 11.3 mm. This displacement represents approximately 7.5% of the helix diameter (D_{helix}).

![Figure 5.1: Compression load-displacement curves for un-grouted helical piles](image)

Figure 5.2, Figure 5.3 and Figure 5.4 display the load-displacement curves for PGHP1, PGHP2 and PGHP3 piles, respectively. These figures show that the PGHPs exhibited much
stiffer response at low settlement levels indicating superior performance under design loads. As presented in Figure 5.2, the initial linear portion of PGHP1 piles extended to 18.0 kN at a displacement of 3.0 mm, while the non-linear zone extended to a displacement of approximately 15.0 mm (~ 10% of $D_{\text{helix}}$), and the corresponding load was 32.0 kN, 27.0 kN and 34.0 kN for Pg 1.1, Pg 1.2 and Pg 1.3, respectively.

**Figure 5.2: Compression load-displacement curves for PGHP1.**

Similarly, PGHP2 piles had an initial linear portion that extended to a settlement between 1.5 and 2.0 mm and an applied load of 16.0 kN, 20.0 kN, and 20.0 kN for Pg 2.1, Pg 2.2 and Pg 2.3. The transitional non-linear zone for this pile group continued to 38.0 kN and 12.0
mm for Pg 2.1 and Pg 2.3 as illustrated in Figure 5.3, while for Pg 2.2, it ended at 42.0 kN and 13.5 mm.

For PGHP3, the initial linear segment of Pg 3.1 and Pg 3.3 extended to a load of 23.0 kN at a displacement of 1.6 mm, while for Pg 3.2 it went up to 21.0 kN at a displacement of 1.1 mm. The final linear region of the three piles started at an average load of approximately 55.0 kN and a displacement = 12.0 mm, and it continued to a maximum applied load equal to 72.0 kN as shown in Figure 5.4.

![Figure 5.4: Compression load-displacement curves for PGHP3](image)

5.2.2 Interpreted ultimate capacity


The graphical interpreted failure criteria depend mainly on the actual pile performance under the applied load and are not affected by the pile geometry (Fahmy, 2015). Given the varying shapes and diameters of the tested PGHPs, the graphical failure criteria were found to be
more suitable in estimating and interpreting the failure loads to facilitate the comparison between the different PGHP groups.

Mansur and Kaufman (1956) defined the failure load as the load corresponding to the intersection of tangents to the first and the final linear portions of the settlement curve. One of the major drawbacks of this method is its susceptibility to personal judgement and the drawing scale of the displacement curve (Hirany and Kulhawy 1988). Besides, it may significantly underestimate the pile capacity (Elkasabgy and El Naggar 2015). Butler and Hoy (1977) considered the ultimate pile capacity as the load at the intersection between a tangent sloping with 0.14 mm/kN and the extension of the initial straight portion of the settlement curve. It should be noted that the tangent sloping at 0.14 mm/kN was found to be very steep for the load-displacement curves of the tested piles. Thus, this criterion was excluded from the analysis.

Hirany and Kulhawy (1988) defined two points on the displacement curve: Point \((L_1)\) at the end of the initial linear region to designate the elastic limit of the curve; and point \((L_2)\) at the end of the non-linear transitional zone. Due to the difficulty of maintaining constant loads within the final linear segment (i.e. high load levels), the measured pile displacement may not be representative of the actual pile behavior where failure could happen suddenly. Thus, it is plausible to consider the end of the nonlinear segment as the on-set of failure (i.e. \(L_1-L_2\) failure criterion).

According to Hirany and Kulhawy (1988) approach and considering the fact that the other graphical failure criteria are not suitable for our load testing results, the ultimate pile capacity is taken as the load corresponding to point \(L_2\).

Table 5.1 presents the interpreted ultimate capacities \((P_{ult})\) of the different pile groups as well as the corresponding settlement \((S_t)\) and its percentage to the pile toe diameter \((D_{toe})\). By inspecting Table 5.1, a significant increase in the interpreted compression capacity can be observed for PGHPs over that of the un-grouted helical piles. This improvement in the pile capacity is mainly attributed to the additional friction resistance along their grout shafts. For PGHP1, the average compression capacity was found to be 1.72 times that of the conventional helical piles. While for PGHP2 and PGHP3, the improvement was more
pronounced (i.e. 2.2 and 3 times the un-grouted pile capacity, respectively) due to their larger shaft diameters.

Table 5.1: Ultimate capacities of piles in medium dense sand, and the corresponding failure displacement.

<table>
<thead>
<tr>
<th></th>
<th>Pult (kN)</th>
<th>Sf (mm)</th>
<th>Sf/Dtoe (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pung</td>
<td>18.0</td>
<td>11.0</td>
<td>7.3</td>
</tr>
<tr>
<td>Pung 2</td>
<td>18.0</td>
<td>11.5</td>
<td>7.7</td>
</tr>
<tr>
<td>PGHP1</td>
<td>Pg 1.1</td>
<td>32.0</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td>Pg 1.2</td>
<td>27.0</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td>Pg 1.3</td>
<td>34.0</td>
<td>15.0</td>
</tr>
<tr>
<td>PGHP2</td>
<td>Pg 2.1</td>
<td>38.0</td>
<td>12.0</td>
</tr>
<tr>
<td></td>
<td>Pg 2.2</td>
<td>42.0</td>
<td>13.5</td>
</tr>
<tr>
<td></td>
<td>Pg 2.3</td>
<td>38.0</td>
<td>12.0</td>
</tr>
<tr>
<td>PGHP3</td>
<td>Pg 3.1</td>
<td>54.0</td>
<td>11.0</td>
</tr>
<tr>
<td></td>
<td>Pg 3.2</td>
<td>57.0</td>
<td>12.0</td>
</tr>
<tr>
<td></td>
<td>Pg 3.3</td>
<td>54.0</td>
<td>12.5</td>
</tr>
</tbody>
</table>

It can also be observed that the failure threshold for conventional helical piles (i.e. point L2) existed at an average displacement between 7.3 and 8.0% of the helix diameter. Similarly, the three PGHP groups (i.e. PGHP1, PGHP2, and PGHP3) reached their ultimate capacities at a settlement level varying between 6.3% and 10% of the grout shaft diameter. These observations are in good agreement with Liveneh and El Naggar (2008) failure criterion that defined the ultimate helical pile capacity as the load corresponding to a pile head movement equal to the elastic deformation of the pile shaft plus 8.0% of the helix diameter.

5.2.3 Load transfer mechanism

The ultimate pile capacity is composed of two components: friction resistance along the pile shaft \( (Q_s) \), and end-bearing resistance at the pile toe \( (Q_b) \). The contribution of each component can be established by developing the axial load distribution along the pile shaft. Each pile was instrumented with three levels of strain gauges (2 gauges/level) located at 1.0, 39.0, and 79.0 cm below the surface of the soil bed. Thus, the load transferred at each strain gauge elevation \( (P_{zi}) \) can be given by:
where \( \varepsilon_i \) is the measured strain at level (i), \( E_p \) is the elastic modulus of the pile shaft, and \( A_p \) is the cross-section area of the pile shaft.

The elastic modulus of the pile shaft (\( E_p \)) was obtained from Equation 5-1 by equating the load at the first level of strain gauges considering the measured strain to the total applied load measured by the load cell. The load transfer curves (i.e. the axial force distribution along the pile shaft) is then established using the measured strain at different elevations. The total shaft resistance, \( Q_s \), is then calculated as the difference between the measured axial forces at the top and bottom levels of strain gauges. Consequently, the mobilized unit skin friction at the pile-soil interface (\( f_s \)) can be estimated by dividing \( Q_s \) over the total surface area between the two levels. On the other hand, the toe resistance contribution (\( Q_b \)) is calculated as the axial force transferred to the last level of strain gauges, and the unit end-bearing resistance (\( f_b \)) is obtained by dividing \( Q_b \) by the cross-section area of the pile toe (Bowles 1996).

For the un-grouted helical piles, the strain gauges measurements showed that \( Q_s \) along the smooth, slender steel shaft was very small, and its contribution to the total pile capacity was negligible. This resulted in the small initial stiffness observed in the displacement curves in Figure 5.1. The unit end-bearing resistance for the un-grouted pile can then be calculated as the pile capacity divided by the helix area, i.e., average \( f_b = 1018 \text{ kPa} \).

On the other hand, PGHPs exhibited a much stiffer response at small pile movement due to the significant frictional resistance along their grout shafts. The relations between the total applied load measured by the load cell and the load transferred to the different levels of strain gauges are shown in Figure 5.5, Figure 5.6, and Figure 5.7 for PGHP1, PGHP2, and PGHP3, respectively. It is useful to refer to Table 3.2, which summarizes the visual description of the different PGHP groups when examining the contribution of each resistance component (i.e. \( Q_s \) and \( Q_b \)) in the PGHP capacity.
Figure 5.5: Axial load distribution along PGHP1 a) Pg 1.1 and b) Pg 1.2

Figure 5.6: Axial load distribution along PGHP2 a) Pg 2.1 and b) Pg 2.2
The variation of $Q_s$ and $Q_b$ with the pile settlement for PGHP1 are presented in Figure 5.8. As can be observed from Figure 5.8, $Q_s$ increased linearly with displacement up to a load of 10.0 kN and a displacement of 2.0 mm. This displacement represents approximately 2.0% of the pile shaft diameter ($D_{shaft}$). Beyond this point, the friction component increased non-linearly to a maximum value of 11.0 kN extending from 9.0 to 16.0 mm as illustrated in Figure 5.8a. When the pile displacement exceeded 16 mm ($\approx 16\%$ of $D_{shaft}$), a slight decrease in the friction resistance was observed. Thus, $Q_s$ for PGHP1 is presumed to be 11.0 kN and $f_s = 44.5$ kPa.

Furthermore, the relation between $Q_b$ and the pile movement can be represented by a bi-linear function. The first linear segment of the curve extended to a toe resistance of 9.5 kN and displacement equal to 4.0 mm ($\approx 2.6\%$ of $D_{helix}$). While the second linear portion
continued with a lower slope until the test termination as shown in Figure 5.8b. Thus, the contribution of $Q_b$ in the ultimate capacity of PGHP1 was 20.5 and 16.5 kN for $P_g$ 1.1 and $P_g$ 1.2, respectively.

Figure 5.9 presents the variation of $Q_s$ and $Q_b$ with the pile settlement for PGHP2. As can be noted from Figure 5.9a, the shaft resistance increased linearly with displacement up to a load of approximately 11.5 kN and displacement of approximately 1.2 mm ($\approx 0.8\%$ of $D_{shaft}$). This was followed by a non-linear increase of load up to 12.5 kN which occurred at a pile displacement of 3.0 mm ($\approx 2.0\%$ of $D_{shaft}$). Beyond which, a slight decrease in $Q_s$ was observed as the sand near the pile-soil interface approached the critical state at a displacement equal to 5.0 mm ($3.2\%$ of $D_{shaft}$), which is in good agreement with the findings of (Han et al., 2016). As the pile displacement continued to increase, $Q_s$ started to increase again as shown in Figure 5.9a.

Figure 5.9: Resistance components of PGHP2

The second increase in $Q_s$ with the pile displacement can be attributed to the mobilization of the end-bearing failure envelope above the pile toe elevation that caused a decrease in the measured strain at the bottom level of strain gauges (Yang 2006). Accordingly, $Q_s$ for PGHP2 was found to be 15.5 kN. To estimate $f_s$ at the ultimate pile capacity, $Q_s$ was divided by the surface area of the pile shaft considering an average pile diameter ($D_{avg} = 156$ mm). For PGHP2, $f_s$ was found to be 39.5 kPa.

Figure 5.9b also shows that there is a bi-linear relationship between $Q_b$ and the displacement of PGHP2. The first linear portion extended to a bearing resistance of 12.0 kN and a
corresponding displacement of 4.0 mm (≈ 2.6% D\text{toe}). The curve continued with a much lower slope up to a maximum settlement of 26.0 mm. At the interpreted ultimate resistance, Q\text{b} was 22.5 kN and 26.5 kN for Pg 2.1 and Pg 2.2. This represents 23.3% increase in the average f\text{b} over the un-grouted helical piles.

**Figure 5.10** presents the variation of Q\text{s} and Q\text{b} with the pile head displacement of PGHP3. As can be observed from **Figure 5.10a**, the Q\text{s} response curves for Pg 3.1 and Pg 3.3 display two linear segments. The first one extended to a load = 18.5 kN at a displacement of 2.1 mm (≈ 1.2% of D\text{shaft}), while the second one extended to the end of the test with a much lower slope. For Pg 3.2, the initial linear segment extended to 15.0 kN at 1.0 mm settlement, after which, the increase in the shaft resistance became non-linear up to a maximum load of 27.0 kN. Due to the larger pile diameter of PGHP3, the end-bearing failure envelope extended to a further distance above the toe level, and hence; no decrease in the Q\text{s} was recorded as the surrounding soil approached the critical state. Accordingly, Q\text{s} would be approximately 23.2 kN, which yields an average f\text{s} = 52.4 kPa.

![Figure 5.10: Resistance components of PGHP3](image)

Unlike the other PGHP groups, the toe resistance of PGHP3 exhibited a non-linear behavior with the pile settlement as shown in **Figure 5.10b**. The toe capacity, Q\text{b}, was 31.5, 33.0, and 31.0 kN for Pg 3.1, Pg 3.2, and Pg 3.3, respectively, which yields, an average f\text{b} = 1308.5 kPa. The shaft (Q\text{s}) and end-bearing (Q\text{b}) resistances of the un-grouted helical piles and PGHPs installed in medium dense sand are summarized in **Table 5.2**.
Table 5.2: Resistance components of piles in medium dense sand.

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<th>Ultimate capacity (kN)</th>
<th>Shaft resistance (kN)</th>
<th>Unit shaft resistance (kPa)</th>
<th>End-bearing resistance (kN)</th>
<th>Unit end-bearing resistance (kPa)</th>
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</table>

NA: Not applicable as the strain gauges mounted to the pile shaft got damaged during installation.

These results clearly indicate that the pressure grouted helical pile (PGHP) offers a significant increase in the pile capacity, which amounted to 72%, 119%, and 206% for PGHP1, PGHP2, and PGHP3, respectively, over the un-grouted helical piles. This increase is related to the significant improvement in the pile shaft resistance ($Q_s$) and the end-bearing capacity of the supporting soil ($f_b$).

The observed increase in $Q_s$ is attributed to the formation of a larger diameter grout column, the higher friction angle ($\delta$) at the pile-soil interface, and the higher lateral earth pressure (radial stress) due to the cavity expansion associated with the pile installation. The results demonstrated that the unit shaft resistance, $f_s$, of the three PGHP configurations was found to vary between 39.6 and 52.4 kPa. An average $f_s$ value of 45.5 kPa $\pm$ 15% is a reasonable estimate for PGHPs installed in medium dense sand with a grouting pressure of 70 psi (480 kPa).

The estimated $f_s$ for PGHP1, PGHP2, and PGHP3 under compression loading is approximately twice the measured value for the same pile groups under uplift loading. The difference in the shaft friction between piles loaded in compression and uplift can be attributed to two main reasons. First, the contraction and expansion of the pile shaft due to
Poisson’s ratio, which is not significant for short piles under low load levels. Second, the difference in the effective stress changes in the soil as the pile is loaded in either direction (i.e. uplift or compression), which control the behavior of short piles (Nicola and Randolph 1993).

Based on the results of an extensive parametric study, Nicola and Randolph proposed **Equations 5-2 and 5-3** to determine the ratio of the shaft friction in tension to that in compression.

\[
\frac{Q_t}{Q_c} = \left\{1 - 0.2 \log_{10}\left(\frac{100}{L/D}\right)\right\} \left(1 - 8\hat{\eta} + 25\hat{\eta}^2\right)
\]

**eq. 5-2**

\[
\hat{\eta} = \upsilon_p \tan \delta \left(\frac{L}{D}\right) \left(\frac{G_{avg}}{E_p}\right)
\]

**eq. 5-3**

where \(Q_t\) and \(Q_c\) are the friction resistances under tensile and compressive loads, respectively, \(\upsilon_p\) is the Poisson’s ratio of the pile, \(\delta\) is the friction resistance at the pile-soil interface, \(G_{avg}\) is the average shear modulus of the soil, and \(E_p\) is the elastic modulus of the pile shaft.

The substitution in the above equations by our piles and soil parameters resulted in a \(Q_t/Q_c\) ratio of 0.7. It should be noted that Nicola and Randolph conducted their parametric study on piles with length to diameter ratios (L/D) between 10 and 80. Thus, lower ratios can be expected for piles with smaller L/D like the PGHPs considered herein.

Likewise, the higher \(f_b\) observed for PGHPs is attributed to two reasons: the weight of the grout tank caused some densification at the soil directly below the pile helix, and the injected grout permeated through the soil voids and increased its strength. For PGHP1, no grout was observed at the bottom of the pile helix. Thus, the increase in \(f_b\) (11.1%) over what was observed for conventional helical piles may be attributed to soil densification under the grout tank weight. While for PGHP2 and PGHP3, the location of the grout nozzles below the pile helix allowed some of the injected grout to permeate through the supporting soil voids, which cemented the soil particles together and enhanced its shear strength. For PGHP2 and PGHP3, the increase in the unit end-bearing resistance was found to be 23.3% and 28.5%, respectively.
The observations from Chapters 3 and 4 demonstrate that PGHP3 had the largest pile diameter, the best grouting efficiency, and the highest pullout capacity. Moreover, the results from Table 5.2 shows that it also had the maximum compression resistance. Thus, additional laboratory experiments were conducted to investigate the behavior of PGHP3 under compression loading considering different testing conditions (i.e. sand relative density and grouting pressure).

5.3 Effect of sand relative density and grouting pressure

5.3.1 Ultimate capacity interpretation

The effects of the grouting pressure and the relative density of sand are examined by inspecting the load testing results of three un-grouted helical piles and eight PGHP3 piles installed in loose and dense sand strata with two different grouting pressures: 480 kPa (70 psi) and 690 kPa (100 psi). The properties of the two sand strata were summarized in Table 3.1, in which a letter (D or L) and a number (70 or 100) are added to the pile name to refer to the relative density of the sand bed and the grouting pressure used during installation. The Load-displacement curves show that all the piles installed in loose sand displayed the typical settlement curve with its three distinct regions; an initial linear region, followed by a nonlinear transitional zone, and a final linear segment that extended until the test termination. While for those installed in dense sand, the final linear zone could not be detected due to the continuous soil hardening and the limitations of the testing equipment.

Figure 5.11 shows that for the un-grouted helical pile in loose sand (Pung 3-L), the initial linear portion of the displacement curve is very small due to the negligible contribution of the friction resistance along the steel pipe shaft. At the same time, the starting point of the second linear segment (L2) that defines the failure threshold according to the L1-L2 failure criterion can be observed at a load of 12.0 kN and a corresponding displacement equal to 12.0 mm (≈ 8.0% of D_helix).

Figure 5.12 displays the results for PGHP3 installed in loose sand with a grouting pressure of 70 psi (PGHP3-L70) and 100 psi (PGHP3-L100). The piles exhibited a stiffer initial response compared to the ungrouted piles due to the considerable shaft resistance. For these
two pile groups, the failure threshold (i.e. point $L_2$) was observed at an average applied load of 43.0 kN and 51.0 kN, and a corresponding pile movement of 14.0 mm ($\approx 7.7\%$ of $D_{shaft}$) and 13.5 mm ($\approx 7.0\%$ of $D_{shaft}$) for PGHP3-L70 and PGHP3-L100, respectively.

![Graph showing load-settlement curves for un-grouted helical piles in loose and dense sand.](image)

**Figure 5.11:** Load-settlement curves for un-grouted helical piles in loose and dense sand.

![Graph showing load-settlement curves for PGHP3 installed in loose sand with grouting pressures of 70 and 100 psi.](image)

**Figure 5.12:** Load-settlement curves for PGHP3 installed in loose sand with grouting pressures of 70 and 100 psi.

**Figure 5.13** presents the results for grouted piles installed in dense sand. It is difficult to identify the start of the second linear portion of the settlement curves, which affected the accuracy of detecting the exact location of point $L_2$. Thus, using the $L_1$-$L_2$ failure criterion to interpret the ultimate pile capacity is not recommended.
By inspecting Table 5.1 and the load-settlement curves of piles installed in loose sand, it can be observed that the failure threshold (i.e. point \( L_2 \)) for conventional helical piles existed at an average displacement between 7.3 and 8.0% of the helix diameter. Similarly, PGHP1, PGHP2, PGHP3, PGHP3-L70, and PGHP3-L100 reached their ultimate capacities at a settlement level varying between 6.3% and 10% of the grout shaft diameter. These observations are in good agreement with Liveneh and El Naggar (2008) failure criterion that defined the ultimate helical pile capacity as the load corresponding to a pile head movement equal to the elastic deformation of the pile shaft plus 8.0% of the helix diameter. Therefore, Liveneh and El Naggar failure criterion was used to interpret the ultimate capacities for piles installed in dense sand. The elastic deformation of the pile shaft (\( \delta_e \)) is given by:

\[
\delta_e = \frac{PL}{EA}
\]

where, \( \delta_e \) is the elastic deformation of the pile shaft, \( P \) is the applied load, \( L \) is the pile length, \( E \) is the Young’s modulus of the pile shaft, and \( A \) is the cross-section area of the pile.

For the piles tested in the current study, \( \delta_e \) is small (< 1.0 mm) and its contribution to the total pile settlement is negligible due to the short pile length and the low loading levels. Therefore, for piles installed in dense sand, the ultimate capacity can be considered as the
load corresponding to a pile displacement equal to 8.0% of the helix diameter. The helix diameter is replaced by the grout shaft diameter in the failure criterion for PGHPs.

5.3.2 Load transfer mechanism

The shaft and bearing resistance components (i.e. $Q_s$ and $Q_b$) are obtained by developing the axial load distribution diagram along the pile shaft. The un-grouted helical piles depends mainly on the helix resistance to support the applied compression load. While for PGHPs, the strain gauges measurements show that both $Q_s$ and $Q_b$ contributed to the compression resistance of the tested piles.

**Figure 5.14** presents the axial load distribution along PGHP3-L70 piles. $Q_b$ is calculated as the axial force measured at the last level of strain gauges. While $Q_s$ is considered as the difference between the load cell reading and the estimated $Q_b$ value. For PGHP3-L70, two different $Q_s$-displacement relationships were observed as shown in **Figure 5.15a**. Pile (Pg 3.4-L70), followed a bi-linear behavior. The first linear segment extended to 8.5 kN and 1.5 mm ($\approx 0.8\%$ of $D_{shaft}$), while the other straight line continued with a lower slope until the end of the test.

![Figure 5.14: Axial load distribution along PGHP3-L70](image)

For Pg 3.5-L70, the $Q_s$-displacement curve was linear up to an applied load of 12.0 kN, which corresponded to 3.6 mm ($\approx 2.0\%$ of $D_{shaft}$). A non-linear increase was then observed up to a load of 17.0 kN and a corresponding pile movement equal to 10.0 mm ($\approx 5.5\%$ of $D_{shaft}$). Beyond this point, a plateau was observed up to a final displacement = 24.0 mm as illustrated...
in Figure 5.15a. On the other hand, the end-bearing resistance varied in a non-linear fashion as the pile settlement increased as shown in Figure 5.15b. The average \( Q_s \) and \( Q_b \) values for PGHP3-L70 at the interpreted ultimate capacity (i.e. \( P_{ult} = 43.0 \text{ kN} \)) were 15.7 kN and 27.3 kN, respectively.

![Figure 5.15: Resistance components of PGHP3-L70](image)

Similarly, the axial load distribution curves along the shafts of PGHP3-D70, PGHP3-L100, and PGHP3-D100 are presented in Figure 5.16, Figure 5.17, and Figure 5.18, respectively. While the variation of their resistance components with the pile movement are given by Figure 5.19 through Figure 5.21.

![Figure 5.16: Axial load distribution along PGHP3-D70](image)
Figure 5.17: Axial load distribution along PGHP3-L100

Figure 5.18: Axial load distribution along PGHP3-D100

Figure 5.19: Resistance components of PGHP3-D70
Table 5.3 summarizes the ultimate capacities and the shaft and end-bearing resistances for piles installed in loose and dense sand. By inspecting the results in Table 5.3, the following observations can be made relating to the effect of grouting pressure and sand relative density on the compression performance of PGHP3. First, the improvement in the ultimate pile capacity due to grouting was found to decrease as the relative density of the sand increased. For PGHP3 installed in loose, medium, and dense sand beds with a grouting pressure of 70 psi, the increase in the pile capacity was 258%, 206%, and 111%, respectively, over the un-grouted piles. Besides, increasing the grouting pressure from 70 psi to 100 psi increased the compression capacity of PGHP3 by 18.6% and 12.2% for piles installed in loose and dense sand, respectively.
Table 5.3: Resistance components for piles installed in loose and dense sand

<table>
<thead>
<tr>
<th></th>
<th>Ultimate capacity (kN)</th>
<th>Qs (kN)</th>
<th>fs (kPa)</th>
<th>Qb (kN)</th>
<th>fb (kPa)</th>
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Second, the unit shaft resistance at the pile-soil interface is directly proportional to the sand relative density and the grouting pressure. For PGHP3-L70, PGHP3, and PGHP3-D70, $f_s$ was 34.7, 52.4, and 62.5 kPa. The effect of the grouting pressure on $f_s$ was less pronounced. An increase of 16% and 12.3% was observed for PGHP3 installed in loose and dense sand, respectively, when the grouting pressure was increased from 70 psi to 100 psi. It is worth noting that for pressure grouted micropiles (Type B) installed with a similar grouting pressure range (500 to 1000 kPa), a grout-ground bond ($\alpha$-bond) varies between 70 and 190 kPa was suggested by the FHWA (2005) for piles installed in loose to medium dense sand. While those installed in medium to very dense sand, the recommended $\alpha$-bond was 120 – 360 kPa.

Moreover, the improvement in $f_b$ associated with the installation of PGHP3 is a function of the relative density (i.e. voids size) of the sand bed. Looser sand beds with higher voids ratio can be easily densified under the grout tank weight. Furthermore, larger voids allowed more grout permeation, and hence; more improvement in the $f_b$ is expected. For PGHP3-L70, PGHP3, and PGHP3-D70, the increase in $f_b$ over those observed for un-grouted helical piles...
was 56%, 28.5%, and 26.0%, respectively. Higher grouting pressure was found to promote the penetration of the grout into the soil voids. Therefore, the improvement in $f_b$ jumped to 60% and 36% for PGHP3-L100 and PGHP3-D100, respectively.

5.4 Analytical estimation of PGHP capacity

PGHP construction involves grout injection under high pressure. Currently, no guidelines are available to evaluate its ultimate capacity considering its unique method of installation. Thus, the applicability of existing methods for pile design are discussed, and relevant design guidelines are proposed to account for the effects of PGHP’s distinctive features on the ultimate capacity. The estimation of the friction resistance along the grout shaft ($Q_s$) and end-bearing resistance at the pile toe ($Q_b$) is discussed herein.

5.4.1 Shaft friction resistance

The unit shaft resistance ($f_s$) for piles installed in sand can be calculated using the $\beta$-method developed by Burland (1973) and given by Equation 5-5.

$$f_s = k_s \sigma'_v \tan \delta = \beta \sigma'_v$$

where $k_s$ is the coefficient of lateral earth pressure, $\delta$ is the interfacial friction angle at the soil-pile interface, $\sigma'_v$ is the average effective vertical stress along the pile length, and $\beta$ is the friction capacity factor.

The main challenge in this method is determining the lateral earth pressure coefficient ($k_s$), which is significantly affected by the pile configuration, the initial soil conditions, and the pile installation technique (Tomlinson and Woodward 2008). Several methods have been developed to determine ($k_s$) for piles installed in sand with different construction methods (drilling, driving, and screwing). However, as discussed in Section 4.4, the one developed by Bowles (1996) for short piles was found to be more suitable for the PGHPs considered in this study. According to Bowles, $k_s$ can be estimated using Equation 5-6.

$$k_s = \frac{K_a + k_o + k_p}{3}$$
where $k_a$, $k_o$, and $k_p$ are the coefficients of active, at rest, and passive earth pressures, respectively.

Moreover, torquing PGHP into the ground causes the grout nozzles to create a cavity around the steel pile shaft. At the same time, the injection of pressurized grout and the rotation of the helix displace the soil laterally and increase the lateral earth pressure at the pile-soil interface. The unit shaft friction for PGHPs is expected to be affected by the shaft expansion and the high grouting pressure, similar to the high shaft resistance ($\alpha$-bond) of micropiles (Type B and Type E) that are constructed using high grouting pressure. Thus, a placement method coefficient ($k_{mo}$) needs to be considered to take into account the effect of the novel installation technique on the pile shaft resistance (Levacher and Sieffert, 1984), i.e.:

$$f_s = K_{mo} k_s \sigma_v ' \tan \delta$$  \hspace{1cm} eq. 5-7

where $k_{mo}$ is the placement method coefficient, and $k_s$ is the coefficient of lateral earth pressure from Equation 5-6.

For the PGHPs under discussion, the piles reached about 69% to 91% of their $Q_s$ at a displacement = 2.0% of the grout shaft diameter. At this level of displacement, the friction resistance would be mobilized fully along the pile shaft (Randolph 2003 and Fleming et al. 2009). Moreover, the piles experienced slippage failure under pullout loading at the same level of displacement. Therefore, the friction resistance can be evaluated conservatively at a displacement = 2.0% of the pile diameter in developing the placement factor coefficients. Table 5.4 compares $f_s$ values estimated by the $\beta$-method and those obtained at displacement = 0.02D and presents the back-figured $k_{mo}$ values to account for the PGHP installation effect on its shaft resistance.

For PGHPs installed with a grouting pressure of 70 psi (480 kPa), $k_{mo}$ was found to vary between 3.72 and 4.52. Increasing the grouting pressure from 70 psi (480 kPa) to 100 psi (690 kPa) increased the $k_{mo}$ value by 20% and 9% for PGHPs installed in loose and dense sand, respectively. For design purposes, $k_{mo} = 3.72$ can be used to conservatively estimate the unit shaft resistance of model PGHP (H/D = 4.2 to 5.1) installed with a grouting pressure of 70 psi under compression loading. For grouting pressure = 100 psi (690 kPa), $k_{mo}$
increased to 4.14 due to the larger cavity expansion, and the higher radial stresses developed during installation.

Table 5.4: Placement method coefficient

<table>
<thead>
<tr>
<th>Soil type</th>
<th>PGHP group</th>
<th>$Q_s$ (kN)</th>
<th>$Q_s$ at 0.02D (kN)</th>
<th>$Q_s$ at 0.02D/ $Q_s$</th>
<th>$f_s$ at 0.02D (kPa)</th>
<th>$f_s$ with $\beta$-method (kPa)</th>
<th>$(K_{mo})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>PGHP3-L70</td>
<td>15.8</td>
<td>11.0</td>
<td>0.70</td>
<td>24.2</td>
<td>6.5</td>
<td>3.72</td>
</tr>
<tr>
<td></td>
<td>PGHP3-L100</td>
<td>19.5</td>
<td>14.0</td>
<td>0.72</td>
<td>29.0</td>
<td>6.5</td>
<td>4.46</td>
</tr>
<tr>
<td>Medium sand</td>
<td>PGHP1</td>
<td>11.0</td>
<td>10.0</td>
<td>0.91</td>
<td>38.0</td>
<td>8.4</td>
<td>4.52</td>
</tr>
<tr>
<td></td>
<td>PGHP2</td>
<td>15.5</td>
<td>11.5</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>PGHP3</td>
<td>23.2</td>
<td>19.8</td>
<td>0.85</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense sand</td>
<td>PGHP3-D70</td>
<td>24.5</td>
<td>17.5</td>
<td>0.71</td>
<td>44.6</td>
<td>11.7</td>
<td>3.81</td>
</tr>
<tr>
<td></td>
<td>PGHP3-D100</td>
<td>27.5</td>
<td>19.0</td>
<td>0.69</td>
<td>48.5</td>
<td>11.7</td>
<td>4.14</td>
</tr>
</tbody>
</table>

5.4.2 End bearing resistance

The unit end-bearing resistance can be predicted using the general bearing capacity formula (Meyerhof, 1951):

$$f_b = q'N_qS_qd_q + 0.5B\gamma N_\gamma S_\gamma d_\gamma \quad \text{eq. 5-8}$$

where $q'$ is the effective vertical stress at the pile toe, $B$ is the pile diameter, $\gamma$ is the effective unit weight, $N_q$ and $N_\gamma$ are the bearing capacity factors, $S_q$ and $S_\gamma$ are the shape factors of the pile toe (i.e. pile helix in case of helical piles), and $d_q$ and $d_\gamma$ are the depth factors.

The bearing capacity factors ($N_q$ and $N_\gamma$) derived by Meyerhof, i.e.

$$N_q = e^{\pi \tan \theta} \tan^2 (45 + \phi/2) \quad \text{eq. 5-9}$$

$$N_\gamma = (N_q - 1) \tan 1.4\phi \quad \text{eq. 5-10}$$

For helical piles, Perko (2009) recommended using the shape and depth factors proposed by Vesic (1973), i.e.

$$S_q = 1 + \frac{B}{L} \tan \phi \quad \text{eq. 5-11}$$

$$S_\gamma = 1 - 0.4 \frac{B}{L} \quad \text{eq. 5-12}$$
\[ d_q = 1 + 2ktan\varphi (1-sin\varphi) \]
\[ k = \tan^{-1}\left(\frac{H}{B}\right) \]
\[ d_r = 1 \]

where \( \varphi \) is the angle of internal friction, \( H \) is the pile length. For circular piles, \( L = B = D \).

For deep foundations with length to diameter ratio \((H/D) > 5\), the \( \gamma \)-term is relatively small and can be neglected (Perko 2009 and Bowles 1996). For the test piles, the \( \gamma \)-term did not exceed 4.0\% of the \( q \)-term and can be ignored. Furthermore, for circular piles, \( k \) approaches a value of \( \pi/2 \), and \( B/L = 1 \). Consequently, the shape, depth, and bearing capacity factors vary only with the soil friction angle \((\varphi)\). Hence, they can be grouped according to Perko (2009) and the unit bearing resistance can be given in terms of a general capacity factor \( N_q' \):

\[ f_b = qN_q' \]

where \( N_q' \) is the general bearing capacity factor (i.e. \( N_q' = N_qS_qd_q \)).

The \( N_q' \) values of the un-grouted helical piles were back-calculated from the pile load tests and found to be 49.9, 71.5, and 124.2 for piles installed in loose, medium, and dense sand, respectively. The experimental data was then compared with other \( N_q' \) values from the literature including those recommended by Coyle & Castello (1981), Janbu (1976), Meyerhof (1975 and 1951), Vesic (1973), Berezantzev et al. (1961), and Terzaghi (1943) as illustrated in Figure 5.22. The comparison indicates that the measured \( N_q' \) values lie within the range found in the literature for the same friction angles.

Coyle and Castello (1981) was found to underestimate the \( N_q' \) values by 26.0, 19.2, and 16.7\% for un-grouted piles in loose, medium, and dense sand, respectively. On the other hand, Hansen (1970) method was found to overestimate the \( N_q' \) values by approximately 26.5\%. Alternatively, considering a shape factor of 1.3 with Hansen (1970) bearing capacity theory to allow for the circular cross-section of the pile as recommended by Tomlinson and Woodward (2008) was found to estimate the measured end-bearing capacity factor \((N_q')\) with an accuracy of 96\% as shown in the figure.
For PGHPs, some improvement in the unit end-bearing resistance \( (f_b) \) was observed over the conventional helical piles due to two reasons. First, the weight of the grout tank caused some densification at the soil directly below the pile helix resulting in an increase of approximately 11.0% for PGHPs installed to a depth of 0.8 m. For full-scale PGHPs installed to a deeper depth, the densification effect will be minor and can be neglected.

Second, some of the injected grout permeated through the soil voids and increased its strength. According to Warner (2004), this increase in the soil resistance due to grout permeation depends on the amount of grout and how it penetrates the soil voids (i.e. the degree of saturation), which cannot be predicted during the pile construction. Thus, for conservative design purposes, it is not safe to rely on these two factors and Equation 5-16 can be used to determine the unit end-bearing resistance of PGHPs. The \( N_q' \) values of the PGHPs were back-calculated from the pile load tests and found to agree well with Hansen (1970) method as presented in Figure 5.22.

In conclusion, Equations 5-7 and 5-16 are guidelines to estimate the unit shaft and end-bearing resistances for PGHPs installed in sand with grouting pressures of 70 psi (480 kPa) and 100 psi (690 kPa). However, further studies are still required to verify these two equations considering different soil and/or grouting properties.
5.5 References


6.1 Introduction

Finite element (FE) analysis is a powerful tool in the geotechnical engineering field. It can simulate engineering problems that are too complex for conventional analytical solutions. In this chapter, a three-dimensional (3D) finite element model is developed using ABAQUS software package (SIMULIA 2013) to simulate the performance of PGHP3 under monotonic axial loading. The developed model is calibrated using some of the pullout pile load testing results and is then verified using the compression load test results. The verified model is then used to further investigate the load transfer mechanism, the effect of soil relative density and grouting pressure on the installation and performance of the model test piles. Moreover, the influence of the rigid boundaries of the laboratory setup on the test piles performance is investigated.

The same modelling methodology is adopted to simulate full-scale PGHPs in order to perform a parametric study to investigate the effect of shaft and helix diameters on the ultimate pile capacity. This chapter presents a detailed discussion about the different aspects, techniques, and properties of the developed FE model including its geometry, the selection of the element type, the constitutive material models, and the boundary conditions.

6.2 General Model Features

6.2.1 Geometry

Two three-dimensional finite element models were developed to simulate the performance of PGHPs (FEM-1) and conventional helical piles (FEM-2) under axial loading. The models utilized symmetry in the y-direction (x-z plane) to reduce the model size and the computational time. The soil continuum was modelled in two parts (Upper soil layer and Lower soil layer) considering a cylindrical configuration. To model the effects of pile installation method, a hole with radius slightly smaller than the radius of the modelled pile
was created along the centerline of the upper soil layer. **Figure 6.1** and **Figure 6.2** present the geometries of upper and lower soil layers for FEM-1 and FEM-2, respectively.

![Figure 6.1: Soil Continuum for FEM-1 (PGHP): a) lower soil and b) upper soil](image)

![Figure 6.2: Soil Continuum for FEM-2 (conventional helical pile): a) lower soil and b) upper soil](image)

The piles were modelled considering the y-symmetry and was placed along the axial direction (z-axis) of the model. The pile helix was simplified as a planar disk as shown in **Figure 6.3**. The boundaries were placed at a distance similar to the dimensions of the steel sand container employed in the test setup. The radius of the soil medium was 650.0 mm, and
the heights for the top and the bottom soil layers were 800 mm and 700 mm, respectively. The top surface of the soil model was considered as a stress-free boundary.

Figure 6.3: The Modelled piles a) PGHP and b) conventional helical piles

6.2.2 Finite elements

The 3D pile and soil parts were discretized into 8-noded, first order, and reduced integration continuum solid elements (C3D8R), shown in Figure 6.4. The C3D8R element has three translational degrees of freedom at each node and consists of one integration point at the centroid. The displacement field is linearly interpolated throughout the element. This type of elements is recommended for large-strain analyses where large mesh deformation is expected (Elsherbiny and El Naggar 2013).

Figure 6.4: Continuum solid element (C3D8R) used for soil and pile discretization (After SIMULIA, 2013)
6.2.3 Boundary conditions and constraints

The boundary conditions were chosen to simulate the laboratory testing conditions. For the bottom soil layer (part), the base and the side of the soil cylinder were prevented from moving in any direction such that $U_x = U_y = U_z = 0$ (i.e. pinned support). While the displacement and rotations out of the x-z plane were not allowed so that $U_y = UR_x = UR_z = 0$ to account for the model symmetry (i.e. y-symmetry). Local cylindrical coordinates were defined for the upper soil layer ($r$, $\theta$, and $z$) to allow for radial cavity expansion and enable simulating effects of pile installation. Pin support was set for the side of the top cylinder ($U_r = U_\theta = U_z = 0$), and a symmetric boundary condition ($\theta$-symmetry) was used at the r-z plane to constrain the out of plane movement and rotations ($U_\theta = UR_r = UR_z = 0$). The boundary condition at the top soil cavity varied during the different analytical steps as discussed later. Figure 6.5 presents the boundary conditions used for the upper and lower soil layers.

For FEM-2, additional boundary conditions were used. Two pinned supports were defined at the initial analysis step and continued to restrict the horizontal and vertical movements ($U_x = U_y = U_z = 0$) of the cavity created for the leading pile section (i.e. lower pile shaft and helix) in the lower soil continuum up to the pile activation step. Moreover, roller support was used to prevent the vertical movement ($U_z = 0$) of the upper soil layer at the area circumscribed by the pile helix until the pile was reactivated in the pile activation step.
Figure 6.6 presents the additional boundary conditions for FEM-2. A y-symmetry boundary condition (i.e. $U_y = URx = URz = 0$) was applied to the half-cylinder pile model during the pile activation and loading stages.

Figure 6.6: Additional boundary conditions for FEM-2

6.2.4 Interface model

The PGHP transfers the load to the surrounding soil through two mechanisms: friction resistance along the pile shaft and end-bearing resistance at the pile toe. The interaction between the PGHP and soil along the shaft and at the toe were modelled using the surface to surface contact algorithm available in ABAQUS (SIMULIA 2013). In this algorithm, the interaction between the contacting surfaces consists of two components: normal component and tangential component. The normal component between the surfaces was defined using the “hard” contact behavior model. The “hard” contact model minimizes the penetration of the slave surface (i.e. soil) into the master surface (i.e. pile). Figure 6.7 presents the defaults contact pressure – overclosure relationship.

On the other hand, the tangential component between the surfaces was described by the penalty type – Coulomb’s frictional model. In this model, the contacting surfaces can transfer shear stresses up to a maximum value ($\tau_{\text{max}}$) beyond which, the surfaces start sliding relative to each other (i.e. no more shear stresses transferred to this part of the soil) as shown in Figure 6.8.
The maximum shear stress at the pile-soil interface depends on the contact pressure at the interface ($p$) and the surface roughness (i.e. coefficient of friction, $\mu$) of the pile shaft. The coefficient of friction ($\mu$) for cohesionless soil is commonly given by:

$$\mu = \tan \delta$$  \hspace{1cm} \text{eq. 6-1}

where $\delta$ is the friction angle at the pile-soil interface.

For conventional helical piles with smooth steel surface, $\delta$ was considered as 2/3 of the soil friction angle ($\varphi$), while for PGHP with rough grout shaft surface, $\delta$ was equal to $\varphi$ as recommended by Kulhawy (1984) and Tomlinson & Woodward (2008) for cast in place concrete piles. The same interaction models (i.e. hard contact and penalty type – Coulomb’s
frictional model) were used to define the interface between the upper and lower soil layers. For PGHPs, no interaction was assigned between the created grout column and the steel pile (i.e. assumed to be bonded and modelled as one part).

6.2.5 Material model

The soil was modelled as an isotropic elastic-perfectly plastic continuum with failure described by the Mohr-Coulomb criterion. For such constitutive model, the elastic behavior is defined by Young’s Modulus (E) and Poisson’s ratio (ν), while the plastic behavior is defined by the critical angle of internal friction (φ<sub>cr</sub>) and the dilation angle (ψ). The hardening property of the soil material can be described by the cohesion yield stress (c) and the absolute plastic strain (ε<sub>pl</sub>). In the Mohr-Coulomb criterion, failure occurs when the shear stress at any plane in the material reaches the failure envelop shown in Figure 6.9.

![Mohr-Coulomb failure envelope](image)

**Figure 6.9: Mohr-Coulomb failure envelope (After SIMULIA, 2013)**

The Mohr-Coulomb failure envelope can be constructed by plotting Mohr’s circles at different stress states and draw the best tangent to these circles. Moreover, the Mohr-Coulomb plasticity criterion is characterized by a smooth flow potential that has a hyperbolic shape in the meridional plane and an elliptic shape in the deviatoric stress plane (SIMULIA 2013). For PGHPs, the pile was modelled as an elastic material that depends on the Poisson’s ratio (ν), and Young’s modulus (E) to describe its behavior under the small load levels considered in our study.
6.2.6 Meshing properties

For FEM-1, the PGHP consisted of 13064 C3D8R elements with an average element size of 12.0 mm and an average aspect ratio of 2.95. While, the soil continuum was modelled using 51446 C3D8R elements with an average aspect ratio of 3.2, including about 0.8% of the elements having an aspect ratio greater than 7.0. The mesh was refined in the pile vicinity (i.e. near and below the pile) with an average element size = 10.0 mm and was coarser in a single bias arrangement to the outside with a maximum element size = 60.0 mm at the side boundary as shown in Figure 6.10.

![Figure 6.10: FEM-1 meshing a) soil continuum and b) PGHP3](image)

The same meshing procedure was used for FEM-2. The un-grouted helical pile consisted of approximately 4000 C3D8R elements, while the soil continuum was modelled using 42928 elements as illustrated in Figure 6.11. Changing the soil elements’ size close to the pile from 10.0 mm to 7.0 mm and those at the side boundaries from 60.0 mm to 42.0 mm had a negligible effect on the pile response (i.e. less than 3% of the maximum pile resistance). However, it increased the computational time to the double.

6.2.7 Analysis steps

All the finite element simulations performed in this study involved five analytical steps. First, the initial step is created at the beginning of the model step sequence to define the boundary
conditions, and the interactions that are applicable at the beginning of the analysis (SIMULIA 2013). The initial step was characterized by the following features;

1- The pile was deactivated, and only the two soil blocks existed.

2- An additional boundary condition to restrict the radial movement of the soil cavity (Ur = 0) were applied beside those presented in Figure 6.5 and Figure 6.6.

Second, establishing a geostatic equilibrium plays an important role in any geotechnical problem. Thus, the initial step was followed by the geostatic step to simulate the in-situ stress condition. In this step, the soil own weight (i.e. gravity load) was defined and a pressure load was applied to the bottom soil layer (i.e. at the location of the pile toe) to account for the weight of the missing soil column along the cavity. Moreover, the geostatic step witnessed the definition of the interaction between the top and bottom soil layers.

To achieve geostatic equilibrium under the prescribed loads and boundary conditions, ABAQUS allows its users to define an elevation-dependent stress state (i.e. predefined geostatic stress field) during the initial analytical step from which, ABAQUS starts the iteration process to achieve equilibrium during the geostatic step. The initial geostatic stress field is defined by two pairs of stress and elevation values. For material elements between
the two given elevations, ABAQUS uses linear interpolation to determine the initial stress state. It is worth mentioning that the predefined initial stresses should be close to the equilibrium state; otherwise, the iteration process may fail or the displacement corresponding to equilibrium might be large indicating inappropriate modelling for the in-situ stress condition.

The installation of PGHP incorporates some lateral soil displacement (i.e. cavity expansion) due to the grouting pressure and the rotation of pile helix during installation. To account for this radial soil displacement, a cavity expansion step was added after the geostatic step. The soil cavity was created with a radius ($r_c$) less than that of the PGHP ($r_p$) by the required cavity expansion ($\Delta r$). In the cavity expansion step, the radial boundary condition at the soil cavity was modified to allow for the expansion of the cavity diameter ($U_r = \Delta r$).

In step number four (pile activation step), the pile was reactivated, and its own weight was defined, the pile shaft symmetry was specified, and the soil-pile interfaces were activated. Besides, the radial boundary condition at the soil cavity and the equivalent soil pressure at the pile toe were deactivated. In the final step (loading step), the required load was applied to the pile.

### 6.2.8 Model calibration and validation

FEM-2 was verified (i.e. soil parameters, boundary conditions, and interface properties) by comparing its prediction of the compression load-displacement curves with the observed response for the conventional helical piles installed in loose (Pung 3-L) and dense (Pung 4-D and Pung 5-D) sand strata. The material properties of conventional helical piles used in the analysis were: mass density ($\rho$) = 7850 kg/m$^3$, elastic modulus ($E_p$) = 210 GPa, and Poisson’s ratio ($\nu$) = 0.3. A cavity expansion equal to zero ($\Delta r = 0$) was considered during the cavity expansion step. The soil properties were adjusted until a satisfactory match was observed between the calculated and measured responses. Figure 6.12 shows the agreement between the calculated and measured load-displacement curves and Table 6.1 provides the initial soil properties as well as those provided best match.
Figure 6.12: Comparison between the observed load-displacement curves and those estimated by the finite element model

The dilation angle (i.e. $\psi = \phi_p - \phi_{cr}$) and the modulus of elasticity (E) were adjusted to achieve good match as shown in Table 6.1, and a small cohesion value of 3.0 kPa was necessary to overcome convergence problems. It should be noted that $\psi$ and E values assigned to the medium dense sand were taken as the average of the values that provided best match for the loose and the dense sand beds.

Table 6.1: Initial vs Calibrated soil parameters

<table>
<thead>
<tr>
<th>Sand Condition</th>
<th>Soil layer</th>
<th>Depth (m)</th>
<th>$\gamma_{bulk}$ (kN/m$^3$)</th>
<th>$\phi_{cr}$</th>
<th>$\psi$</th>
<th>E (MPa)</th>
<th>Cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial soil parameters</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>Top layer</td>
<td>0.0 – 0.8</td>
<td>17.0</td>
<td>33.0</td>
<td>1.0</td>
<td>8.5 – 12.5</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Bottom layer</td>
<td>0.8 – 1.5</td>
<td>17.0</td>
<td>33.0</td>
<td>1.0</td>
<td>14.5 – 19.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Medium</td>
<td>Top layer</td>
<td>0.0 – 0.8</td>
<td>17.8</td>
<td>33.0</td>
<td>4.0</td>
<td>10.0 – 14.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Bottom layer</td>
<td>0.8 – 1.5</td>
<td>17.8</td>
<td>33.0</td>
<td>4.0</td>
<td>17.0 – 23.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Dense</td>
<td>Top layer</td>
<td>0.0 – 0.8</td>
<td>18.8</td>
<td>33.0</td>
<td>8.0</td>
<td>12.0 - 17.5</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Bottom layer</td>
<td>0.8 – 1.5</td>
<td>18.8</td>
<td>33.0</td>
<td>8.0</td>
<td>19.5 – 30.0</td>
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<tr>
<td>Calibrated soil parameters</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>Loose</td>
<td>Top layer</td>
<td>0.0 – 0.8</td>
<td>17.0</td>
<td>33.0</td>
<td>1.0</td>
<td>8.0</td>
<td>3.0</td>
</tr>
<tr>
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<td>0.8 – 1.5</td>
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<td>2.0</td>
<td>17.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Medium</td>
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<td>17.8</td>
<td>33.0</td>
<td>6.0</td>
<td>11.0</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>Bottom layer</td>
<td>0.8 – 1.5</td>
<td>17.8</td>
<td>33.0</td>
<td>7.0</td>
<td>23.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Dense</td>
<td>Top layer</td>
<td>0.0 – 0.8</td>
<td>18.8</td>
<td>33.0</td>
<td>9.0</td>
<td>16.5</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>Bottom layer</td>
<td>0.8 – 1.5</td>
<td>18.8</td>
<td>33.0</td>
<td>11.0</td>
<td>34.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>
FEM-2 was then verified by comparing the load-displacement curve predicted for un-grouted helical piles installed in medium dense sand considering the soil properties summarized in Table 6.1 with those observed for Pung 1 and Pung 2 during pile load tests as presented in Figure 6.13. The excellent agreement between the calculated and measured pile responses shown in Figure 6.13 indicates that FEM-2 can be used to accurately predict the conventional helical pile performance.

Figure 6.13: Comparison between the estimated and observed load-displacement curves for conventional helical pile in medium dense sand

The calibration of FEM-1 aims to assess the model ability to simulate the cavity expansion occurred during PGHP installation and its ability to capture the pile axial response. To achieve this goal, FEM-1 was used to simulate the pullout response of PGHP3 considering the same soil properties summarized in Table 6.1. The PGHP3 pile was composed of a hollow steel shaft at the center filled with neat cement grout and surrounded by a grouted sand column. It was modelled as an elastic material with specified Poisson’s ratio (ν), and Young’s modulus (E) to describe its behavior. The equivalent elastic modulus of the pile material (E_p) was obtained by equating applied load measured by the load cell to the load calculated at the first level of strain gauges considering the measured strains. Thus, the following material properties were assigned to the PGHP3 in the FEM; mass density (ρ) = 2243 kg/m³, (E_p) = 9.0 GPa, and Poisson’s ratio (ν) = 0.3.

For cohesionless soils, the increase in the radius of a cavity from an initial value (r_c) to a final value (r_c + Δr) causes an increase in the radial soil stresses around the cavity (Yu and
This increase in the lateral stresses is a function of the amount of expansion ($\Delta r$) and the properties of the surrounding sand (i.e. $\varphi$, $\psi$, $E$, and $\nu$). The developed FEM was used to establish the radial stress profile along the pile shaft, and the results are compared with the theoretical lateral earth pressure. The lateral stress value increased gradually in several iterations of the analysis by increasing the cavity expansion ($\Delta r$) until a good match was achieved between the calculated, and measured pullout load-displacement curves as shown in **Figure 6.14a**.

For PGHP installed in medium dense sand with a grouting pressure of 70 psi (i.e. PGHP3), a cavity expansion of 1.6 mm was observed, which represents approximately 4.1% of the cavity radius created by the grout nozzles ($r_c$). The load transfer mechanism along the pile shaft was determined by measuring the axial force distribution along the pile centerline as illustrated in **Figure 6.14b**.

![Figure 6.14: Comparison between FEM and pullout load test results of PGHP3](image)
It can be inferred from Figure 6.14b that the pullout resistance of PGHP3 is derived entirely from the friction resistance along its shaft. This is in agreement with the experimental observations. The unit shaft resistance distribution along the pile shaft can be evaluated by:

$$\Delta f_s = (\Delta Q_u - \Delta w) / \Delta A_s$$  \hspace{1cm} \text{eq. 6-2}

where $\Delta f_s$ is the unit shaft resistance along an infinitesimal pile segment ($\Delta L$), $\Delta Q_u$ is the decrease in the axial force along the pile segment ($\Delta L$), $\Delta w$ is the weight of this pile segment, and $\Delta A_s$ is the surface area of this pile segment.

Figure 6.14c presents the calculated unit shaft resistance profile and shows that $f_s$ increased with depth with an average value of 21.9 kPa, which agrees well with the laboratory test data (i.e. $f_s = 21.9$ kPa).

To verify FEM-1, it was used to simulate the behavior of PGHP3 under compression loading considering the same cavity expansion ($\Delta r = 1.6$ mm). Figure 6.15 compares the measured and predicted pile responses. The calculated response agrees well with the measured response, which verifies the ability of FEM-1 to predict the axial performance of PGHP3.

As discussed in Chapter 5, the pile compression resistance is composed of two components: friction resistance along the pile shaft ($Q_s$) and end-bearing resistance at the pile toe ($Q_b$). $Q_b$ was taken as the load measured at the pile toe, while $Q_s$ was calculated as the load measured at the pile head minus $Q_b$. The axial load distribution along PGHP3 is shown in Figure 6.15b, from which $Q_b$ and $Q_s$ can be determined.
The variations of $Q_s$ and $Q_b$ with the pile head displacement are shown in Figure 6.15c and Figure 6.15d, respectively. At the ultimate capacity (i.e. $P_{\text{ult}} = 55.0 \text{ kN}$), $Q_s$ and $Q_b$ values obtained from FEM-2 were 26.7 and 28.3 kN, respectively. These values agree well with the measured values (i.e. $Q_s = 23.2 \text{ kN}$ and $Q_b = 31.8 \text{ kN}$). The agreement between the calculated and measured responses confirm the ability of the developed finite element models to predict the axial behavior of PGHP3 and conventional helical piles.

### 6.3 Axial loading and cavity expansion

#### 6.3.1 Uplift loading

Following the same methodology, the lateral soil displacement ($\Delta r$) was determined for different PGHP groups. For PGHP3 installed in dense sand with the same grouting pressure (i.e. 70 psi), a smaller cavity expansion ($\Delta r = 1.0 \text{ mm}$) was predicted by FEM-1 to get a match with the experimental data set as shown in Figure 6.16a. The smaller cavity expansion observed for PGHP3-D70 is attributed to the higher resistance of dense sand to the cavity expansion.

Figure 6.16b and Figure 6.16c showed that the radial (lateral) stresses after the PGHP construction and before loading had a maximum value of 78 kPa near the pile toe and it decreased toward the surface due to the lower soil resistance to the cavity expansion (El Naggar and El Naggar 2012). The average radial stress along the pile shaft after construction was found to be 51.3 kPa, which is approximately 4.0 times the theoretical value (i.e. 13.46
kPa) given by \( \sigma_h = k_s \sigma_v \) (where \( \sigma_h \) is the lateral earth pressure, \( k_s \) is the lateral earth pressure coefficient for conventional piles, and \( \sigma_v \) is the overburden pressure). This considerable increase in the radial stresses at the pile-soil interface shows the importance of using the placement method coefficient \( (k_{mo}) \) as a correction factor to account for the influence of the pile installation technique on the surrounding soil and consequently, the pile shaft resistance.

By multiplying the radial stresses (i.e. normal to the pile shaft) by the friction coefficient at the pile-soil interface \( (\mu = \tan \delta) \), the unit shaft resistance \( (f_s) \) profile along the pile length can be obtained as presented in Figure 6.16d. For PGHP3-D70, the average \( f_s \) predicted by the FEM-1 was equal to 24.1 kPa, which is very close to the 23.3 kPa observed for this pile group during the laboratory experiments.

Figure 6.16: Uplift behavior of PGHP3-D70 from FEM-1 a) pullout load-displacement curves, b) radial stress profile across soil continuum, c) radial stresses at the pile-soil interface, and d) unit shaft resistance along the pile length
A decrease in the radial stresses was observed upon applying uplift loading to the pile. This decrease is attributed to two reasons. First, the contraction of the pile shaft under pullout loading due to Poisson’s ratio; however; this effect is minor at low load levels (Nicola and Randolph, 1993). Second, the upward movement of the pile and the soil in direct contact with its shaft during uplift loading causes some stress relief. This effect is more pronounced closer to the soil surface because the upward soil displacement is maximum at the sand surface and decreases with depth. On the other hand, as the pile moves upward, a gab is created below the pile toe resulting in significant reduction in the radial stress near the pile base as illustrated in Figure 6.16c.

6.3.1.1 Evaluation of tank boundary effect

The test piles were installed in a sand bed enclosed in a steel tank with limited dimensions. It is essential to evaluate the effect of the boundary conditions on the experimental results to enable proper evaluation of the influence of the PGHP installation technique on the surrounding soil, and hence understand the improvement in the PGHP response under axial loading.

For small scale piles tested in a steel tank, the rigid tank boundary may influence the pile load test results. Thus, a sufficient distance (B) should be maintained between the pile center and the tank sides (Alharthi 2018). The values recommended for distance B vary between 6.0 times the pile diameter (Chari & Meyerhof 1983, and Shalabi & Bader 2014) up to 10.0 times the pile diameter (Al-Mhaidib 2012). For PGHP3-D70 in the current experimental setup, the distance between the pile center and the rigid boundary of the sand bed was 0.65 m, which is approximately 4.2 times the shaft diameter. Thus, the tank boundary may influence the measured pullout capacity and its effect should be assessed for realistic evaluation of the axial performance of PGHPs established from the current study.

To investigate the boundary effect on the PGHP3-D70 performance under uplift loading, the radius of the soil continuum in FEM-1 was increased to 10 times the pile diameter (i.e. 1.56 m). The comparison between the results considering the actual soil boundary (ASB) and those for extended soil boundary (ESB) revealed a 10% reduction in the pullout resistance of PGHP3-D70 as illustrated in Figure 6.17a. This decrease in the pile resistance is attributed
to the lower radial stress and consequently, the unit shaft resistance at the pile-soil interface observed for the ESB model as shown in Figure 6.17b.

**Figure 6.17: Effect of rigid boundary on a) pullout load-displacement curve and b) radial stresses at the pile-soil interface**

For ESB model, Figure 6.18 shows a rapid decrease in the radial stress within 0.2 m from the pile shaft ($\approx 1.28 \text{ of } D_{shaft}$). Beyond this distance, the attenuation in the radial stress continued at a decreasing rate until an asymptote is reached at a distance of 0.8 m ($\approx 5.13 \text{ of } D_{shaft}$). Beyond this distance, the measured radial stress was equal to the at rest earth pressure. On the other hand, for the ASB model, the rigid tank boundary affected
the lateral stress field at a distance ≈ 0.57 m resulting in some increase in the confinement pressure around the pile shaft, and hence, a higher pullout resistance was exhibited.

The same pullout performance was observed for the other PGHP3 groups. For PGHP3-D100, $\Delta r = 1.5$ mm was required to achieve a good match between the measured and calculated responses as shown in Figure 6.19. This magnitude of $\Delta r$ represents approximately 3.8% of the initial cavity radius ($r_c$) created by the grout nozzles. The increase in $\Delta r$ observed for this pile group is attributed to the higher grouting pressure used during the pile installation.

![Figure 6.19: Uplift behavior of PGHP3-D100 from FEM-1 a) pullout load-displacement curves, b) radial stress profile across soil continuum, c) radial stresses at the pile-soil interface, and d) unit shaft resistance along the pile length](image)

Considerably larger ($\Delta r$) were observed for PGHPs installed in loose sand with the same grouting pressures, i.e. 2.5 mm ($\approx 6.4\%$ of $r_c$) and 2.9 mm ($\approx 7.5\%$ of $r_c$) for PGHP3-L70 and PGHP3-L100, respectively. This increase in lateral soil displacements is attributed to the
lower resistance of loose sand to the cavity expansion (El Naggar and El Naggar 2012). **Figure 6.20** and **Figure 6.21** present the pullout responses of PGHP3-L70 and PGHP3-L100 estimated by FEM-1, respectively.

**Figure 6.20**: Uplift behavior of PGHP3-L70 from FEM-1 a) pullout load-displacement curves, b) radial stress profile across soil continuum, c) radial stresses at the pile-soil interface, and d) unit shaft resistance along the pile length.
Figure 6.21: Uplift behavior of PGHP3-L100 from FEM-1 a) pullout load-displacement curves, b) radial stress profile across soil continuum, c) radial stresses at the pile-soil interface, and d) unit shaft resistance along the pile length

A summary of the cavity expansions estimated by FEM-1 for the different PGHP groups is presented in Table 6.2.

Table 6.2: Estimated cavity expansion of the different PGHP groups

<table>
<thead>
<tr>
<th>Pile group</th>
<th>PGHP3</th>
<th>PGHP3-D70</th>
<th>PGHP3-D100</th>
<th>PGHP3-L70</th>
<th>PGHP3-L100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δr (mm)</td>
<td>1.6</td>
<td>1.0</td>
<td>1.5</td>
<td>2.5</td>
<td>2.9</td>
</tr>
<tr>
<td>Δr/rc (%)</td>
<td>4.11</td>
<td>2.57</td>
<td>3.85</td>
<td>6.42</td>
<td>7.45</td>
</tr>
</tbody>
</table>

Δr: Cavity expansion  
rc: Initial cavity radius

6.3.2 Compression loading

The piles response under compressive loading were simulated considering the same (Δr) values in Table 6.2. Figure 6.22a shows the excellent agreement between the predicted and the measured load-displacement curves achieved for PGHP3-L70. For this pile group, the interpreted ultimate capacity was 43.0 kN at a corresponding pile movement of 14.5 mm. A slight increase in the pile settlement (i.e. 17.0 mm) was estimated by FEM-1 at the same load. To determine the load transfer mechanism, a section was taken along the pile centerline to generate the axial load distribution curves shown in Figure 6.22b, which can be used to determine the shaft resistance, (Qs) and the end-bearing resistance, (Qb).
Figure 6.22: Compression behavior of PGHP3-L70 from FEM-1 a) compression load-displacement curves, b) axial load distribution along the pile shaft, c) friction resistance at the pile-soil interface, and d) end-bearing resistance at the pile toe

The variations of $Q_s$ and $Q_b$ with pile head settlement are presented in Figure 6.22c and Figure 6.22d, respectively. At the interpreted ultimate capacity, the estimated $Q_s$ and $Q_b$ values by FEM-1 were 16.0 and 27.0 kN, respectively. These values are in agreement with the measured values (i.e. $Q_s = 15.7$ kN and $Q_b = 27.3$ kN).

It should be noted that the compression and uplift responses were investigated considering the same values of $(\Delta r)$. Thus, the developed radial stresses at the pile-soil interface before loading were similar for both cases. However, unlike the uplift loading case, no decrease in the radial stresses was observed around PGHP3-L70 during compression loading as shown in Figure 6.23.
Figure 6.23: radial stresses along PGHP3-L70 during compression loading

This can be attributed to the high resistance of the lower soil layers to the downward movement of the soil encasing the pile (Nicola and Randolph, 1993). Hence, no radial stress relief is observed. Similarly, Figure 6.24 presents the response of PGHP3-L100 under compression loading obtained by FEM-1. At an interpreted ultimate capacity of 51.0 kN, \( Q_s \) and \( Q_b \) are 19.0 and 32.0 kN, respectively. The results agree well with \( Q_s \) and \( Q_b \) values measured during the experimental program (i.e. \( Q_s = 19.5 \) kN and \( Q_b = 31.5 \) kN).

Figure 6.25 displays the excellent match achieved between the calculated and measured responses for PGHP3 installed in dense sand with a grouting pressure of 70 psi (PGHP3-D70. As can be observed from Figure 6.25b, the friction resistance-displacement curve exhibits an initial linear segment followed by a nonlinear segment then a final linear segment. However, in contrast with the \( Q_s \)-displacement curves for PGHP3-L70 and PGHP3-L100, the final linear segment for PGHP3-D70 continues to increase up to the test termination.
Figure 6.24: Compression behavior of PGHP3-L100 from FEM-1 a) compression load-displacement curves, b) axial load distribution along the pile shaft, c) friction resistance at the pile-soil interface, and d) end-bearing resistance at the pile toe.

Figure 6.25: Compression behavior of PGHP3-D70 from FEM-1 a) compression load-displacement curves, b) friction resistance at the pile-soil interface, and c) end-bearing resistance at the pile toe.
This continuous increase in the friction resistance for this pile group is attributed to the higher dilation angle of the dense sand stratum (i.e. \( \psi = 1.0 \) degree for loose sand and 9.0 degrees for dense sand). In other words, the downward pile movement during compression loading causes shear stresses to the surrounding sand. In case of dense sand, shearing is generally accompanied by some increase in the sand volume due to its high dilation angle. This increase in the soil volume is partially restricted by the rigid tank boundary. Thus, a continuous increase in the radial stresses, and consequently the shear resistance at the pile-soil interface is expected as illustrated in Figure 6.26a. For PGHP3-D70, the calculated \( Q_s \) and \( Q_b \) at the interpreted ultimate capacity (i.e. 69.5 kN) are 28.5 and 41.0 kN, respectively. PGHP3-D100 displayed similar behavior during compression loading as shown in Figure 6.26b. The calculated compression response of PGHP3-D100 are presented in Figure 6.27.

![Figure 6.26: Increasing the radial stress around the pile in dense sand during compression loading a) PGHP3-D70, and b) PGHP3-D100](image-url)
According to Figure 6.27b and c, the $Q_s$ and $Q_b$ contributions to the compression resistance of PGHP3-D100 are equal to 29.2 kN and 47.8 kN, respectively, which are in good agreement with the experimental results summarized in Table 5-3.

6.3.2.1 Evaluation of tank boundary effect

To investigate the rigid boundary effect on the PGHP performance under compression loading, PGHP3-L70 was re-simulated considering an extended soil continuum. The radius of the soil continuum and its depth below the pile toe was taken as 10 times the pile diameter (i.e. 1.81 m). It is worth noting that the interpreted ultimate capacity of PGHP3-L70 = 43.0 kN, and the corresponding settlement = 17.0 mm. A reduction of approximately 19.5% in the total pile capacity was obtained from the extended soil boundary (ESB) model as presented in Figure 6.28a.

The axial load distribution curve along the pile shaft was developed considering the actual soil boundary (ASB) and the extended soil boundary (ESB) conditions as shown in Figure 6.28b. The results revealed a reduction of 25% in the friction resistance (i.e. decreased from 16.0 kN to 11.94 kN) due to the decrease of the radial stresses around the pile shaft. The higher lateral stresses observed at the pile-soil interface in case of the ASB condition is attributed to the interference of the side tank boundaries with the radial stress field as illustrated in Figure 6.18. The situation was not better for the end bearing resistance due to
the presence of the steel tank base within the end-bearing influence zone that extends beneath the pile tip to a distance between 3.5 and 5.5 times the pile diameter (yang 2006). According to Figure 6.28b, \( Q_b \) witnessed a reduction of approximately 20% (i.e. dropped from 27.0 kN to 21.4 kN) in the ESB model.

Figure 6.28: Effect of rigid boundary on a) pullout load-displacement curve and b) radial stresses at the pile-soil interface

6.3.3 Cavity expansion equation

The developed FEM-1 was used to simulate the uplift and compression responses of the five PGHP groups (i.e. PGHP3, PGHP3-L70, PGHP3-L100, PGHP3-D70, and PGHP3-D100). The results revealed that determining the cavity expansion (\( \Delta r \)) due to the PGHP installation is important for understanding the influence of the installation technique on the surrounding soil and the pile axial capacity.

The developed FEM-1 was used to back-calculate the cavity expansion (\( \Delta r \)) for the different PGHP groups. The results revealed a strong relationship between (\( \Delta r \)), the grouting pressure (\( P_g \)), and the sand relative density (R.D). It was found that \( \Delta r \) is directly proportional to \( P_g \) and inversely proportional to R.D. Thus, \( \Delta r \) can be represented by:

\[
\Delta r = b \frac{P_g}{R_D} + c \quad \text{eq. 6-3}
\]
where $\Delta r$ is the cavity expansion occurred during PGHP installation, $b$ and $c$ are the equation constants, $P_g$ is the grouting pressure used for pile construction, and $R.D$ is the sand relative density.

**Equation 6-3** can be formulated in a non dimensional form by normalizing the different parameters as follows: $P_g$ is normalized by the average overburden stress along the pile shaft ($\sigma_v$); $\Delta r$ is normalized by the initial cavity radius ($r_c$). **Figure 6.29** presents the relationship between the normalized cavity expansion, $\frac{\Delta r}{r_c}$, the normalized grouting pressure, $\frac{P_g}{\sigma_v}$, and the relative density ($R.D$).

![Figure 6.29: Relationship between cavity expansion, grouting pressure, and relative density](image)

By curve fitting the data in **Figure 6.29**, the following relationship is obtained:

\[
\frac{\Delta r}{r_c} (\%) = 0.0227 \left( \frac{P_g}{R.D \sigma_v} \right) + 1.118
\]

**eq. 6-4**

where $\Delta r$ is the cavity expansion occurred during PGHP installation, $r_c$ is the initial cavity radius, $P_g$ is the grouting pressure used for pile construction, $R.D$ is the sand relative density, and $\sigma_v$ is the average overburden stress along the pile shaft.
6.3.4 Full scale model

Helical piles are generally composed of steel shafts fitted with one or more helical plates and are installed into the ground by applying mechanical torque to the pile head. Conventional helical piles can be used to resist uplift and compressive loads, however; their compressive load applications are subjected to some limitations due to the large settlement required to fully mobilize the end-bearing contribution of the pile helix that represents most of its capacity. Moreover, the low buckling resistance of helical piles with slender hollow shafts reduces their load carrying capacity (Vickars and Clemence 2000). In this study, the PGHP is presented as an alternative that can overcome the drawbacks of conventional helical piles.

The compression performance of short PGHPs (i.e. length = 0.8 m) and the effect of the novel installation technique on the surrounding soil have been investigated in Chapters 5 and 6. The results revealed a substantial increase in the radial stresses, and consequently, the friction resistance at the pile-soil interface due to the cavity expansion occurred during the pile installation. A FEM was then developed to simulate the PGHP performance under different conditions (i.e. grouting pressures and relative densities) and predict the expansion of the soil cavity accompanied the pile construction.

The numerical analysis resulted in the relationship given by Equation 6-4 that can determine the cavity expansion as a function of the grouting pressure (P_g), the relative density of the surrounding sand (R.D), and the average overburden stress along the pile shaft (\sigma_v). In this section, the FEM was extended to simulate the compression performance of PGHPs and conventional helical piles under full-scale condition. The piles have a length of 9.5 m and were installed in medium dense to dense sand strata. Table 6.3 summarizes the soil properties used in the FEM.

The elastic modulus of each soil layer was determined using Janbu’s (1963) power function given by Equation 6-5 to define the soil elastic modulus as a function of the confining pressure.

\[ E_i = m_p \left( \frac{\sigma_{i3}}{P_a} \right)^n \]  \hspace{1cm} \text{eq. 6-5}
where $E_i$ is the elastic modulus of layer (i), $p_a$ is the atmospheric pressure, $\sigma_{13}$ is the average confining pressure of layer (i), $m$ is the modulus number, and $n$ is the modulus exponent. Both $m$ and $n$ depend on the soil porosity (i.e. relative density). For the considered sand strata, the relative density was assumed to be 75%, thus; $m$ and $n$ were taken as 500 and 0.48, respectively (Janbu 1963).

**Table 6.3: Assumed full-scale soil conditions**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Unit weight (kN/m$^3$)</th>
<th>Elastic modulus (MPa)</th>
<th>Poisson’s ratio</th>
<th>Critical state friction angle ($\varphi_{cr}$)</th>
<th>Dilation angle ($\psi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.0</td>
<td>18.0</td>
<td>24.5</td>
<td>0.3</td>
<td>30.0</td>
<td>5.0</td>
</tr>
<tr>
<td>2.0 – 4.5</td>
<td>41.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5 – 7.0</td>
<td>53.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0 – 9.5</td>
<td>61.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5 – 12.1</td>
<td>69.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.1 – 14.6</td>
<td>77.7</td>
<td></td>
<td>0.3</td>
<td>30.0</td>
<td>10.0</td>
</tr>
<tr>
<td>14.6 – 17.1</td>
<td>84.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

This section presents the compression load test results of five ungrouted helical piles and five PGHPs estimated by the developed FEM. The piles were simulated with three shaft and three helix diameters as summarized in **Table 6.4.** All piles have a wall thickness of 9.5 mm and a helix thickness of 25.4 mm.

**Table 6.4: Summary of the different pile configurations**

<table>
<thead>
<tr>
<th></th>
<th><strong>Un-grouted helical piles</strong></th>
<th></th>
<th><strong>Pressure grouted helical piles</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile type</td>
<td>Pung (24/12.75)</td>
<td>Pung (30/12.75)</td>
<td>Pung (40/12.75)</td>
</tr>
<tr>
<td></td>
<td>Pung (30/10)</td>
<td>Pung (30/16)</td>
<td>PGHP (24/12.75)</td>
</tr>
<tr>
<td></td>
<td>Helix diameter (inches/mm)</td>
<td>24 (609.6)</td>
<td>30 (762.0)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40 (1016.0)</td>
<td>30 (762.0)</td>
</tr>
<tr>
<td></td>
<td>Shaft diameter (inches/mm)</td>
<td>12.75 (323.85)</td>
<td>12.75 (323.85)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12.75 (323.85)</td>
<td>10 (254)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12.75 (323.85)</td>
<td>16 (406.4)</td>
</tr>
<tr>
<td></td>
<td>Grout shaft diameter (mm)</td>
<td>380.83</td>
<td>380.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>380.83</td>
<td>380.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>309.8</td>
<td>464.74</td>
</tr>
</tbody>
</table>
For PGHPs, the piles were installed with a grouting pressure of 1.5 MPa. The grout nozzles were assumed to have the third configuration shown in Figure 3.2c (i.e. PGHP3). The nozzles had a length of 36.0 mm and were inclined to the pile shaft at an angle of 45°. The expected cavity expansions (\(\Delta r\)) according to Equation 6-4 are 2.5, 3.09, and 3.77 mm for PGHPs with shaft diameters = 254, 323.85, and 406.4 mm, respectively.

As discussed in chapter 3, PGHPs installed in dense sand with relative density = 75% had a spiral grout column with solid core along the entire shaft. The solid core was formed by filling the cavity created by the grout nozzles with the pressurized grout, while the spiral ribs were attributed to the grout filling the void created by the helical plate as the pile advanced into the ground. It is worth noting that these spiral ribs are made of grouted sand, thus; it can be easily broken under high load levels. Therefore, the author believes it is more conservative to consider the grout shaft radius in our analysis as the initial cavity radius (\(r_c\)) given by Equation 6-6 plus the estimated cavity expansion (\(\Delta r\)). The grout shaft diameters of the different full-scale PGHPs are presented in Table 6.4.

\[
r_c = r_p + a
\]  
\text{eq. 6-6}

where \(r_c\) is the initial cavity radius, \(r_p\) is the pipe shaft radius, and \(a\) is the horizontal projection of the grout nozzles.

Figure 6.30 presents the full scale conventional helical pile and PGHP. The green colour is an indication for the steel material (\(\rho = 7850 \text{ kg/m}^3\), \(E = 210 \text{ GPa}\), and \(\nu = 0.3\)). While the grey colour represents the grout used during the PGHP installation. In the FEM, the grout was assumed to be a linear elastic material with a unit weight of 21.0 kN/m\(^3\), Young’s modulus of 15.0 GPa, and a Poisson’s ratio of 0.28.

As can be observed from Figure 6.30, the helix in the case of full-scale PGHPs has a larger diameter than the pile shaft, thus; FEM-1 cannot be used to simulate their response as it was initially developed for short PGHPs with constant pile shaft. Thus, for the full-scale PGHPs, FEM-2 will be used to simulate their performance under compression loading. FEM-2 was previously used to simulate the performance of conventional helical piles (i.e. \(\Delta r = 0\)).
Therefore, an additional validation step is required to ensure that it can be used to accurately simulate the PGHP behavior (i.e. $\Delta r \neq 0$).

Figure 6.30: Full scale piles a) conventional helical piles and b) PGHP

According to Figure 6.30b, full-scale PGHPs is expected to have the same shape as PGHP1 presented in Figure 3.10. Thus, for verification purposes, FEM-2 was used to simulate the compression response of PGHP1 and compare it with the experimental test results. PGHP1 piles had an average grout shaft diameter = 97.0 mm and were installed in medium dense sand to a depth of 0.8 m with a grouting pressure of 70 psi (480 kPa). Thus, the pile was simulated considering a cavity expansion of 1.6 mm. Figure 6.31 presents a comparison between the estimated and the measured compression responses of PGHP1 piles.
Figure 6.31: Compression behavior of PGHP1 from the modified FEM a) compression load-displacement curves, b) axial load distribution along the pile shaft, c) friction resistance at the pile-soil interface, and d) end-bearing resistance at the pile toe.

The axial load distribution along the pile shaft shown in Figure 6.31b shows that the compression pile resistance is composed of two components; friction resistance along the pile shaft ($Q_s$) and end-bearing resistance at the pile helix ($Q_b$). As previously discussed, the $Q_b$ contribution was considered as the load transferred to the pile toe, while the friction resistance at the pile-soil interface was calculated as the load measured at the pile head minus the estimated $Q_b$. Figure 6.31c and d present the developed friction and end-bearing resistances with the pile settlement, respectively.

According to Table 5.2, the interpreted ultimate capacity of PGHP1 was approximately 31.0 kN. At this ultimate resistance, the estimated $Q_s$ and $Q_b$ values by FEM-2 was 9.4 and 21.6 kN, respectively, which are in a good agreement with the resistance components summarized in Table 5.2 (i.e. $Q_s = 11.0$ kN and $Q_b = 20.0$ kN). Therefore, we can conclude that FEM-2 has the ability to simulate the performance of full-scale PGHPs under compression loading.

The location of the boundaries was optimized to minimize the effects of the boundary conditions on the results while reducing the computational effort. The radius of the soil cylinder extended to a distance = 4.86 m (i.e. 12 times the greatest shaft diameter). While the bottom boundary was placed at 7.6 m below the pile helix, which is equivalent to 7.5 times the largest helix diameter. Moreover, a stress-free boundary was considered for the soil surface. Figure 6.32 presents the assembly for the full scale PGHP. The continuum was
meshed using approximately 50000 C3D8R elements in a single bias arrangement. The soil element had a size of 50.0 mm at the helix location and it increased to 150 and 350 mm at the vertical and side boundaries, respectively.

![Figure 6.32: FEM-2 assembly](image)

6.3.4.1 Effect of shaft diameter

A comparison between the compressive performance of three PGHPs (i.e. PGHP 30/10, PGHP 30/12.75, and PGHP 30/16) and three un-grouted helical piles (i.e. Pung 30/10, Pung 30/12.75, and Pung 30/16) was conducted herein to investigate the effect of the pile shaft diameter on the PGHP capacity and improvement over the corresponding helical pile.

The load-settlement curves shown in Figure 6.33 shows that all the six piles had the same bearing resistance, which can be observed from the parallel linear segments at the end of their displacement curves. This can be attributed to the same helix diameter (i.e. 762 mm) used for the six piles and the same soil properties. The figure also indicates a much stiffer...
initial response for PGHPs due to the considerable increase in their shaft resistance as a result of the novel PGHP installation technique (i.e. cavity expansion).

![Graph](image)

**Figure 6.33: Load-settlement curves of PGHP and un-grouted piles with different shaft diameters**

For conventional helical piles with closed pile tip, the installation is generally accompanied by some lateral soil displacement causing increased lateral stresses along the pile shaft. According to Tomlinson and Woodward (2008), the ratio between the lateral earth pressure coefficient ($k_s$) and the at rest earth pressure coefficient ($k_o$) after the installation of a large displacement pile varies between 1.0 and 2.0. Thus, a $k_s/k_o$ ratio of 1.75 (i.e. $k_s = 0.75$) was considered for the un-grouted helical piles.

On the other hand, for the PGHPs under discussion, the installation is accompanied by cavity expansions = 2.5, 3.09, and 3.77 mm for PGHP 30/10, PGHP 30/12.75, and PGHP 30/16, respectively. Hughes et al. (1977) derived an elastic-plastic closed form solution assuming Mohr-Coulomb failure criterion. According to their equation, the radial stress increase at a cavity wall due to an expansion ($\Delta r$) can be given by:

\[
p_r = p_o \left( 1 + \sin \phi \right) \left\{ \frac{E \Delta r}{1 + v} \frac{1}{r n p_o \sin \phi} \right\}^{1-N/(1+n)} \tag{6-7}
\]

\[
N = \frac{1 - \sin \psi}{1 + \sin \psi} \tag{6-8}
\]

\[
n = \frac{1 - \sin \psi}{1 + \sin \psi} \tag{6-9}
\]
where \( p_r \) is the radial stress at the cavity wall after expansion, \( p_o \) is the initial radial stress at the cavity wall before expansion, \( \phi \) is the soil friction angle, \( \psi \) is the dilation angle, \( E \) is the initial Young’s modulus, \( \nu \) is the Poisson’s ratio, \( \Delta r \) is the cavity expansion, and \( r \) is the cavity radius.

For the full-scale PGHPs, the lateral stresses around the pile shaft after installation (i.e. \( p_r \)) was found to be 6.1 times the at rest earth pressures (\( p_o \)). The result agrees well with the radial stress distribution curves shown in Figure 6.34. According to the figure, the average radial stresses along the shafts of PGHP 30/10, PGHP 30/12.75, and PGHP 30/16 were 225 kPa, 217 kPa, and 240 kPa, respectively, while the average at rest earth pressure was found to be 36.4 kPa. The results show the importance of using a placement method coefficient (\( k_{mo} \)) to take into account the effect of the pile installation technique on the surrounding soil and the pile shaft resistance. For PGHPs, \( k_{mo} \) (≈ 6.1) can be easily determined by estimating the cavity expansion (\( \Delta r \)) from Equations 6-4, then use Equation 6-7 to calculate the increase in the radial stresses around the pile shaft.

![Figure 6.34: Radial stress distribution curve along the pile shaft](image)

By multiplying the radial stresses by the friction coefficient at the pile-soil interface (\( \mu = \tan \delta \)), the unit shaft resistance (\( f_s \)) can be obtained as presented in Figure 6.35. For conventional helical piles, an average \( f_s \) of 27.5 kPa was observed, while for PGHPs, an average \( f_s \) of 154.6, 151, and 163 kPa were obtained along the shafts of PGHP 30/10, PGHP 30/12.75, and PGHP 30/16, respectively. The unit shaft resistances observed for PGHPs lie
within the range of the grout-ground bond ($\alpha$-bond) recommended for Type B and Type E micropiles installed with the same grouting pressure (FHWA 2005 and Derbi 2013).

![Unit shaft resistance distribution at the maximum pile resistance](image)

**Figure 6.35:** Unit shaft resistance distribution at the maximum pile resistance

### 6.3.4.2 Effect of helix diameter

This section presents the results of three un-grouted helical piles (i.e. Pung 24/12.75, Pung 30/12.75, and Pung 40/12.75) and three PGHPs (i.e. PGHP 24/12.75, PGHP 30/12.75, and PGHP 40/12.75) installed in the predefined sand stratum with three different helix diameters (i.e. 609.6 mm, 762 mm, and 1016 mm). All piles have a shaft diameter of 12.75 inches (323.85 mm) and were tested under compression loading. PGHPs were simulated considering a cavity expansion of 3.09 mm. **Figure 6.36** presents the load-displacement curves of the six piles.

According to **Figure 6.36**, the compression load-displacement curves can be divided into three main regions; an initial linear segment; a non-linear transitional zone; and a second linear segment with a lower slope. For PGHPs, the initial linear portion of the settlement curves coincides with each other indicating equal friction resistance along their shafts. This can be attributed to the same shaft diameter and cavity expansion used for the piles’ simulation. The initial linear segment of the curve extended to a load of 1650 kN and a corresponding pile settlement of 7.5 mm ($\approx 2.0\%$ of $D_{\text{shaft}}$). Similar behavior was observed for the conventional helical piles but with a significant decrease in the shaft resistance.
The effect of the helix diameter on the compression pile response appears during the non-linear transitional zone and becomes more pronounced at the second linear portion of the displacement curve (i.e. high displacement level). In other words, the larger the helix diameter, the higher the end-bearing resistance, and consequently, the steeper the slope of the final linear segment.

Failure of a single pile under compression loading is conceptually reached when a rapid increase in the pile settlement occurs at constant or slightly increased load, i.e. plunging failure (Hirany and Kulhawy 2002). However, the PGHPs and conventional helical piles under discussion did not exhibit that behavior and continued to resist higher loads until the end of the FEM simulation. This continuous increase in the pile resistance is attributed to the high end-bearing resistance of the pile helix that requires high displacement to be fully mobilized. Such high settlement level is not acceptable by many design guidelines. Thus; several failure criteria have been developed to determine the interpreted pile capacity based on settlement limitations. Terzaghi and Peck (1967) defined the failure load as the load corresponding to an absolute settlement limit of 25.0 mm. The settlement limit was set as a function of the pile diameter by O’Neil and Reese (1999) who defined the ultimate pile resistance as the load that causes a pile settlement = 5.0% of the toe diameter. For helical piles with large helix diameter, Elkasabgy and El Naggar (2015) considered the maximum
pile resistance as the load corresponding to a total settlement equal to the elastic deformation of the pile shaft plus 3.5% of the helix diameter.

Given the varying shaft and helix diameters of the PGHPs and conventional helical piles under discussion, the 25.0 mm absolute settlement criterion (i.e. Terzaghi and Peck 1967) was found to be more suitable in estimating and interpreting the pile capacity to facilitate the comparison between the different pile groups. Table 6.5 provides a comparison between the different full-scale PGHPs and un-grouted helical piles.

**Table 6.5: Compression capacities of full-scale piles**

<table>
<thead>
<tr>
<th></th>
<th>Un-grouted helical piles (Pung)</th>
<th>Pressure grouted helical piles (PGHPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pung (24/12.75)</td>
<td>Pung (30/12.75)</td>
</tr>
<tr>
<td>Pile type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile capacity (kN)</td>
<td>990</td>
<td>1153</td>
</tr>
<tr>
<td>Friction resistance (kN)</td>
<td>268.7</td>
<td>268.7</td>
</tr>
<tr>
<td>End-bearing resistance (kN)</td>
<td>721.3</td>
<td>884.3</td>
</tr>
<tr>
<td>PGHP/Pung capacity ratio</td>
<td>2.41</td>
<td>2.23</td>
</tr>
</tbody>
</table>

As can be observed from Table 6.5, the end-bearing resistance of any PGHP is approximately equal to that of the corresponding un-grouted helical pile with the same helix diameter. The results also revealed an increase between 106% and 141% in the PGHP capacity over the conventional helical pile due to the formation of a grout column with bigger diameter and the significant increase in the radial stresses and the unit shaft resistance at the pile-soil interface after installation. Moreover, the improvement in the PGHP capacity was found to be directly proportional with the shaft diameter to helix diameter ratio, which can be attributed to the major contribution of the grout shaft resistance to the total pile capacity.
6.4 References


SIMULIA. 2013. 6.13 User Documentation. Abaqus user’s guide.


Chapter 7

Lateral Load Testing

7.1 Introduction

Pile foundations are subjected to lateral loads and bending moments due to environmental and manmade causes (Fleming et al., 2009). The effects of these loads may govern the foundation design of many structures such as: high rise buildings, offshore and harbour structures, transmission towers, and bridge abutments. In these situations, the serviceability limit state governs the design unless the piles are relatively short or large deflections can be tolerated (Poulos 2001).

Helical piles have recently gained wide popularity for such applications due to the vast advances in the equipment used in helical pile installation. However, the installation of helical piles can cause disturbance of the adjacent soil within the zone circumscribed by the pile helix reducing its shear strength. Consequently, the lateral pile capacity is significantly affected (Puri et al. 1984, and Bagheri & El Naggar, 2013, 2016). Vickars and Clemence (2000) introduced the helical pulldown micropile (HPM) that consists of a helical pile with a grout column surrounding its steel shaft. The grout column was found to improve the helical pile resistance to lateral loading (El Sharnouby and El Naggar 2018). This chapter evaluates the behavior of PGHP under lateral loading in comparison with the conventional helical pile response.

7.2 Pile load testing results

Lateral load tests were conducted on three model un-grouted helical piles and six PGHP3 piles installed in loose and dense sand to a depth of 0.8 m. The PGHP3 piles were constructed with two different grouting pressures 480 kPa (70 psi) and 690 kPa (100 psi) to investigate the effect of grouting pressure on the lateral pile response. The piles are denoted by a letter (D or L) and a number (70 or 100) to indicate the relative density of the sand bed and the grouting pressure used for the pile construction.
The piles were loaded monotonically in equal increments of 0.25 kN. Each increment was maintained for 5.0 minutes. Load increments were added until a continuous increase in the pile movement was observed at constant or slightly increased load (i.e. failure occurred). The piles were then unloaded in three equal decrements. Each load decrement was maintained for 5.0 minutes. After applying the last decrement, the pile movement was monitored for 10.0 minutes to measure the rebound response.

The pile load test results are presented in terms of load-deflection curves measured at the loading point (i.e. 15.0 cm above the surface of the soil bed). The lateral capacities of the different PGHPs and un-grouted helical piles were evaluated from the load-deflection curves. Furthermore, the bending moment was measured along the pile shaft.

7.2.1 Load-deflection curves and interpretation of lateral capacity

The lateral load-deflection curves for the un-grouted helical piles are presented in Figure 7.1. The lateral load-displacement curves can be divided into three regions: an initial linear segment, a non-linear zone and a second linear segment with a lower slope. It is also noted from Figure 7.1 that, as expected, the helical piles installed in dense sand exhibited a stiffer response (i.e. slope of the initial linear segment) than those installed in loose sand.

![Figure 7.1: Lateral load-displacement curves for the conventional helical piles](image)

For helical piles in dense sand, the initial linear segment extended to a lateral load of 1.2 kN and a pile displacement of 6.0 mm (≈ 17.6% of D_shaft), while the helical pile installed in loose
sand (Pung 3-L) displayed linear behavior to only 0.4 kN and a pile head displacement of 4.0 mm. The non-linear zone continued to a horizontal pile movement of 17.0, 15.0, and 14.0 mm at corresponding loads of 1.45, 1.75, and 1.75 kN for Pung 3-L, Pung 4-D, and Pung 5-D, respectively. Pung 4-D and Pung 5-D exhibited stiffer response, which is attributed to the higher soil stiffness and strength of the dense sand (E = 8.0 MPa for loose sand and 16.5 MPa for dense sand, and peak frictional angle, $\varphi = 34^\circ$ for loose sand and $41^\circ$ for dense sand).

**Figure 7.2** compares the lateral load-displacement curves for PGHP3 with un-grouted piles in dense sand. The response curves of PGHPs have the same trend as the un-grouted helical piles but with a considerable increase in the lateral resistance of Pg 3.6-D70 and Pg 3.8-D100 over that of the un-grouted piles. As can be seen in **Figure 7.2**, the initial linear segment of the response curve extended to a load of 1.8 kN and a pile head movement of 3.5 mm ($\approx 2.2\%$ of $D_{\text{shaft}}$) while the final linear segment (i.e. initiation of failure) started at 3.75 kN and a corresponding horizontal displacement of 14 mm ($\approx 9.0\%$ of $D_{\text{ shaft}}$) and 15.5 mm ($\approx 10.0\%$ of $D_{\text{ shaft}}$) for Pg 3.6-D70 and Pg 3.8-D100, respectively.

![Figure 7.2: Lateral load-deflection curves for piles installed in dense sand](image)

This increase in the lateral resistance of Pg 3.6-D70 and Pg 3.8-D100 over that of Pung 4-D and Pung 5-D is mainly attributed to the larger shaft diameter. Besides, the rigid tank boundary may have some effect on the observed response (i.e. distorted the stress and strain fields), which resulted in increased lateral resistance of piles. This effect would be more pronounced for piles with larger shaft diameters (Dong et al. 2018).
Figure 7.2 also shows that the response curves of the two PGHPs are similar even though different grouting pressures (i.e. 70 psi and 100 psi) were used in their construction. This is because the lateral response of piles in the lateral direction is primarily controlled by pile diameter and the soil properties within a wedge that extends to a distance of 6.1Dshaft from the pile surface (Lin et al., 2015), which were the same for both piles. The confining pressure in the annular zone around the pile had a minor effect on its lateral resistance.

Figure 7.3 presents the lateral response curves for piles installed in loose sand. Piles PGHP3-L70 and PGHP3-L100 displayed a much higher lateral resistance compared to the un-grouted helical pile (Pung 3-L). PGHP3-L70 displayed linear response up to a load of 1.5 kN and a displacement of 5.0 mm and 4.0 mm for Pg 3.4-L70 and Pg 3.5-L70, respectively. The final linear segment of the response curve initiated at an average load of 2.6 kN and a corresponding movement of approximately 14.5 mm (≈ 8.0% of Dshaft).

Figure 7.3: Lateral load-deflection curves for piles installed in loose sand

For PGHP3-L100 piles, the initial linear segment extended to 2.0 kN and 6.0 mm due to the larger pile diameter. It can also be observed from Figure 7.3 that the initial linear segment for the two pile groups has the same slope, which can be attributed to the similarity in the soil parameters and the weak effect of the radial confining pressure in the immediate vicinity of the pile on its lateral response. The final linear segment of PGHP3-L100 piles (i.e. on-set of failure) initiated at a load of 3.0 kN and a lateral pile movement of 15.0 mm (≈ 7.8% of Dshaft).
The ultimate lateral pile capacity is often calculated by limit equilibrium analysis. However, the horizontal movement tolerance of the piles-supported structure (i.e. serviceability limit state) always controls the pile design. Thus, several criteria have been developed to estimate the lateral pile capacity based on its load-deflection curves. These criteria are based on either graphical construction (Prakash and Sharma 1990), absolute displacement limits (MacNulty 1956, and Walker & Cox 1966), and displacement limit as a function of the shaft diameter (Broms 1964, and Pyke 1984).

Prakash and Sharma (1990) proposed two criteria to evaluate the lateral pile capacity. The first criterion defines the pile capacity as the load that corresponds to the intersection of tangents to the load-displacement curve. The second criterion defines the failure load as the load corresponding to a specific deflection value (6.25 mm or 12.5 mm). Macnulty (1956) and Walker & Cox (1966) evaluated the lateral pile capacity as the load corresponding to a pile head displacement equal to 6.25 mm and 13.0 mm, respectively. The limiting displacement was set to 5%, and 10% of the pile diameter by Broms (1964), and Pyke (1984), respectively.

Failure loads estimated by the above criteria either lie within the non-linear transitional region or the final linear segment. Due to the difficulty of maintaining constant loads while using a hydraulic jack and a hand pump at high load levels within the final linear segment, the measured pile displacement along the final linear segment may not be representative of the actual pile behavior where failure could happen suddenly. Thus, it is plausible to consider the end of nonlinear segment as the on-set of failure. Hence, the selected interpreted failure criteria should give failure loads close to the upper limit of the non-linear transitional zone (Kulhway, 1988). For the test piles, the ratio between the loads at point L2 (end of non-linear zone) and point L1 (end of first linear segment) varied between 1.5 and 2.1. Therefore, using a factor of safety of two or more will keep the pile performance under design loads within the linear range, which is required by many design guidelines.

The failure criteria proposed by Prakash (1990), Broms (1964), and the 12.5 mm deflection limit criteria were found to give pile capacity within the nonlinear segment. Table 7.1 summarizes the lateral capacities ($H_{ult}$) of the tested piles estimated by the three criteria, the
displacement corresponding to each interpreted failure load (i.e. failure displacement $S_f$), and the ratio between $S_r$ and the displacement at the end of nonlinear segment ($S_{L2}$). Kulhawy (1988) and Prasad & Chari (1999) defined point $L_2$ as the failure threshold of the displacement curve beyond which, any small increase in the applied lateral load results in a substantial increase in the pile displacement (i.e. the surrounding soil within the failure wedge is yielded).

**Table 7.1: Interpreted lateral capacity and the corresponding failure displacement**

<table>
<thead>
<tr>
<th>Pile group</th>
<th>Pile</th>
<th>Tangents intersection</th>
<th>5.0% of the pile diameter</th>
<th>12.5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H_{alt}$ (kN)</td>
<td>$S_f$ (mm)</td>
<td>$S_f/S_{L2}$</td>
</tr>
<tr>
<td>Un-grouted helical piles</td>
<td>Pung 3-L</td>
<td>1.00</td>
<td>10.0</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>Pung 4-D</td>
<td>1.50</td>
<td>12.0</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>Pung 5-D</td>
<td>1.60</td>
<td>12.5</td>
<td>0.89</td>
</tr>
<tr>
<td>PGHP3-D</td>
<td>Pg 3.6-D70</td>
<td>3.30</td>
<td>12.0</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Pg 3.8-D100</td>
<td>3.00</td>
<td>10.0</td>
<td>0.65</td>
</tr>
<tr>
<td>PGHP3-L70</td>
<td>Pg 3.4-L70</td>
<td>2.00</td>
<td>10.0</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>Pg 3.5-L70</td>
<td>2.25</td>
<td>10.0</td>
<td>0.67</td>
</tr>
<tr>
<td>PGHP3-L100</td>
<td>Pg 3.10-L100</td>
<td>2.70</td>
<td>12.0</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>Pg 3.11-L100</td>
<td>2.40</td>
<td>8.0</td>
<td>0.53</td>
</tr>
</tbody>
</table>

N.A: Not applicable
PGHP3-D: PGHP3 installed in dense sand with any grouting pressure

The criterion based on pile head movement = 5.0% $D_{shaft}$ was not suitable for un-grouted helical pile ($D_{shaft} = 34.0$ mm) as the interpreted failure load was within the initial linear segment, and it underestimated the PGHP capacity (i.e. low $S_f/S_{L2}$ ratio). Thus, it was excluded from the analysis. The tangents intersection criterion gave failure loads similar to those obtained by the 12.5 mm criterion; however, it is susceptible to individual judgement. The criterion based on the 12.5 mm pile head movement provided failure load close to the failure threshold (i.e. high $S_f/S_{L2}$ ratio). Thus, it was used to evaluate the pile lateral capacity in this study.

The lateral capacity for Pg 3.6-D70 and Pg 3.8-D100 was found to be almost the same (3.5 kN) because the grouting pressure didn’t have any noticeable effect on the pile capacity.
Similarly, for PGHP3 piles installed in loose sand, the effect of the grouting pressure on the surrounding soil can be neglected, and the observed increase in the lateral capacity of PGHP3-L100 over that of PGHP3-L70 (12.6\%) is attributed to the larger shaft diameter.

The results also revealed a significant increase in the lateral resistance of PGHPs (H_{ult-gr}) over that of the un-grouted helical piles (H_{ult-ung}). For piles installed in dense sand, the H_{ult-gr}/H_{ult-ung} ratio was found to be 2.26 (i.e. 126\% increase in the lateral resistance), while for piles in loose sand, the H_{ult-gr}/H_{ult-ung} ratio was 1.98 and 2.23 for PGHP3-L70 and PGHP3-L100, respectively. As discussed earlier, this considerable improvement in the lateral capacity of PGHPs is mainly attributed to the larger shaft diameter and the existence of a disturbed soil zone around the un-grouted pile shaft. It may have also been affected by the presence of the tank boundary at a relatively small distance.

Several studies have investigated the influence of the boundary condition of the model test setup on the lateral pile response. Most of which recommended a distance to the rigid boundary (i.e. tank, B_D) between 8 and 20 times the pile diameter (D_{shaft}) to eliminate the boundary effect depending on the soil density and the pile installation method (Fahmy 2015, Chandrasekaran et al. 2010, Bolton et al., 1999, Turner & Kulhawy 1987, and Robinsky & Morrison 1964). However, due to the limitations of test facilities, containers with boundaries varying between 4.0D_{shaft} and 15.0D_{shaft} have been widely used in the 1g model tests and centrifuge tests (Liu et al. 2010, and Tamura et al. 2009).

Lin et al. (2015) studied the soil-pile interaction pressure at the circumference of laterally loaded piles in sand with a relative density equal to 32\% using thin tactile pressure sensor and in-soil null pressure sensor. They found that for short stiff piles under lateral loading, a pressure wedge with a bulb shape was formed around the pile. At the ultimate load, the pressure bulb extended to a distance of 6.1D_{shaft} measured from the pile surface, which represented “the influence zone”. In other words, if the rigid boundary exists at a distance greater than the influence zone, the effect of the rigid boundary on the lateral pile behavior is negligible.

For the conventional helical piles, B_D from the outer surface of the steel shaft was equal to 18.6D_{shaft}. Hence, it is concluded that the tank boundary did not affect their interpreted lateral
capacity. On the other hand, for the PGHPs, $B_D$ varied between $2.9D_{\text{shaft}}$ for PGHP3-L100 and $3.7D_{\text{shaft}}$ for PGHP3-D. Therefore, the rigid boundaries influenced the field of the pressure bulb and is expected to have caused some increase in the lateral pile capacity.

Dong et al. (2018) conducted an extensive parametric study to investigate the boundary effect on the lateral response of short-rigid piles considering a wide range of container width ($H_{BD}$) to the pile diameter ($D_{\text{shaft}}$) ratios. The boundary effect was defined as the ratio between the lateral pile resistance in finite boundaries ($H_f$) to that in infinite boundaries ($H_{inf}$). For $H_{BD}/D_{\text{shaft}} = 6.7$ (i.e. PGHP3-L100) and 8.3 (i.e. PGHP3-D), the $H_f/H_{inf}$ ratio was found to be 1.13 and 1.09, respectively. However, Dong et al. (2018) found that the effect of the rigid boundaries (i.e. $H_{BD}/D_{\text{shaft}} > 6.7$) is reduced to less than 5.0% at a lateral displacement $= 2.5\%$ of the pile diameter. Further discussion about the effect of rigid boundary on the pile load test results is provided later in this chapter using the developed FEM-1.

7.2.2 Bending moment along the pile shaft

Strain gauges were used in the current study to measure the strain at discrete points along the pile shaft, which can then be converted into bending moments, i.e.:

$$M_z = \frac{E_p I_p (\varepsilon_1 - \varepsilon_2)}{D}$$

eq. 7-1

where $M_z$ is the bending moment at any depth ($z$), $E_p$ is the modulus of elasticity of the pile shaft, $I_p$ is the moment of inertia of the pile shaft, $\varepsilon_1$ and $\varepsilon_2$ are the strain gauges readings at the opposite side of the pile cross-section, and $D$ is the distance between the two strain gauges (i.e. outer pile diameter).

Unfortunately, most of the strain gauges used in this study were damaged by the end of the compression and pullout load tests. Only a few of them survived and their results were used to approximate the bending moment along Pg 3.4-L70, Pg 3.10-L100, and Pg 3.6-D70. These piles represent the three PGHP groups presented in Table 7.1 (i.e. PGHP3-D, PGHP3-L70, and PGHP3-L100). The distribution of bending moments along the three piles at different load increments up to the interpreted failure loads are shown in Figure 7.4.
Figure 7.4: Bending moment distribution along the different PGHP groups

It can be observed from Figure 7.4 that the bending moment increased from a value equal to lateral load (H) multiplied by the load eccentricity (e = 0.15 m) at the surface of the sand bed to a maximum value at the second level of strain gauges (depth = 0.39 m). The bending moment then decreased to approximately zero at the pile toe. For Pg 3.6-D70, Pg 3.4-L70, and Pg 3.10-L100, a maximum bending moment of approximately 1.11, 0.69, and 0.73 kN.m was recorded at an ultimate lateral resistance (H_{ult}) of 3.5, 2.25, and 2.75 kN, respectively.

The higher bending moment observed for PGHP installed in dense sand is attributed to the higher shear strength and modulus of elasticity of the surrounding soil, which increased its ability to sustain more loads. In addition, the larger pile diameter of PGHP3-L100 allowed it to carry higher lateral load and experience higher bending moment along its shaft compared to PGHP3-L70.
7.2.3 Analytical estimation of lateral pile capacity

The lateral resistance of PGHPs can be estimated by adopting the same techniques used for conventional piles. However, the soil parameters should be suitably selected to account for the effect of the grouting pressure employed in pile installation.

The pile response to lateral loading depends on its relative rigidity: short (rigid) or long (flexible) pile behavior. The rigid pile behavior is observed when the ratio of pile length (L) to its diameter (D_{shaft}) is less than 10. The response of rigid piles to lateral loads depends entirely on the soil resistance, i.e., it is assumed that the pile does not deflect. All PGHPs tested in the current study had an L/D_{shaft} ratio between 4.2 and 5.1. Furthermore, Poulos and Davis (1980) proposed a relative stiffness factor (k_{rs}) to differentiate between the rigid and flexible pile behaviors, i.e.

\[ k_{rs} = \frac{E_p I_p}{E_h L^4} \]  \hspace{1cm} \text{eq. 7-2}

where \( E_p \) is the modulus of elasticity of the pile material, \( I_p \) is the moment of inertia of the pile cross-section, \( E_h \) is the horizontal soil modulus at the pile toe (i.e. \( E_h = 8.0 \) MPa and 16.5 MPa for loose and dense sand strata, respectively), and \( L \) is the embedded depth of the pile.

Meyerhof (1995) suggested that laterally loaded piles with \( k_{rs} \) less than 0.01 are considered flexible piles. The calculated \( k_{rs} \) values using Equation 7-2 were higher than 0.039 for all PGHPs tested in this study, which represents short pile behavior.

Several methods have been developed to determine the capacity of short-rigid piles in sand with a free head condition under lateral loading (Broms 1964, Petrasovits & Award 1972, and Meyerhof et al. 1981). These methods assume the pile will rotate as a rigid body around a certain point at a depth (x) from the soil surface. They consider the lateral soil pressure (\( P_u \)) to be uniform across the pile width (i.e. diameter) but make different assumption for its distribution along the pile length as presented in Figure 7.5. In Figure 7.5, \( k_p \) and \( k_a \) are the passive and active coefficients of lateral earth pressure, \( \sigma_v' \) is the effective vertical stress, \( \gamma \) is the soil unit weight, \( \varphi \) is the soil friction angle, and \( S_{bu} \) is a shape factor.
Prasad and Chari (1999) conducted laboratory load tests on fully instrumented rigid circular piles in sand. They found that for circular pile cross-section, the \( P_u \) distribution is not uniform across the shaft diameter, but rather, it has a maximum value at the center and approaches zero at the sides. Thus, they recommended applying a correction factor of 0.8 to the assumed \( P_u \) value at any given depth. Prasad and Chari (1999) also reported that \( P_u \) increased linearly with depth up to a distance of 0.6x (\( P_u \_0.6x \)), where x is the depth of point of rotation. Beyond this point, \( P_u \) decreased linearly to zero at depth x, then increased in the opposite direction as shown in Figure 7.6 until it reached a maximum value of 1.7 \( P_u \_0.6x \) at the pile toe. They suggested that \( P_u \) at 0.6x can be estimated by:

\[
\begin{align*}
\text{eq. 7-3} \\
\quad P_u (0.6x) &= 10^{(1.3\tan \varphi + 0.3)} \gamma 0.6x
\end{align*}
\]

It should be noted that all the above methods were derived from the theory of lateral earth pressure on rigid walls. However, the ultimate lateral resistance per unit width of a rigid pile is higher than that acting on a corresponding wall due to the shearing resistance on the vertical sides of the soil failure wedge (Terzaghi 1943). Meyerhof et al. (1981) proposed multiplying the net lateral earth pressure acting on a wall by a shape factor (\( S_{bu} \)) to account for this 3D effect as shown in Figure 7.5c. Broms (1964) assumed \( S_{bu} = 3.0 \). Petrasovits and
Award (1972) assumed $S_{bu} = 3.7$. On the other hand, Prasad and Chari (1999) found that $S_{bu}$ is a function of the soil friction angle ($\varphi$) and combined it with the passive earth pressure coefficient in the first term of their equation (i.e. $10^{(1.3\tan \varphi + 0.3)})$.

![Figure 7.6: Prasad and Chari (1999) Lateral earth pressure distribution](image)

The ultimate lateral resistance of short-rigid piles in sand can then be estimated using the following steps:

1. Assume a rotation depth ($x$) and establish the lateral soil pressure ($P_u$) profile along the pile shaft using any of the given distributions in Figure 7.5 and Figure 7.6.

2. Calculate the forces acting at each segment of the pile using Equation 7-4. A correction factor of 0.8 should be applied to Prasad and Chari (1999) lateral earth pressure to account for the non-uniform distribution of $P_u$ across the pile shaft. For all other methods, this correction factor is included either in the equation constant or in the shape factor.

$$H_i = \int_{z_1}^{z_2} P_{uz} D \, dz$$

where $H_i$ is the horizontal force acting at segment (i) of the pile, $Z_1$ and $Z_2$ are the depth below the soil surface at the beginning and the end of the segment, $P_{uz}$ is the lateral soil pressure at any depth ($z$), and $D$ is the pile diameter.
3. Calculate the moment at the point of load application (i.e. point a)

4. Repeat steps 1 to 3 until the summation of moments at point (a) is equal to zero.

5. The lateral pile capacity \( H_{ult} \) can then be calculated as the summation of forces in the horizontal direction.

The depth of rotation for the test piles was found to be 0.56 m, 0.61 and 0.62 using methods proposed by Meyerhof et al. (1981), Prasad & Chari (1999) and Petrasovits & Award (1972), respectively (about 74% of embedded pile length, L).

Broms (1964) assumes the rotation occurs at a point very close to the pile toe, and the high pressure acting below this point is replaced by a single concentrated force \( F \) at the pile toe as shown in Figure 7.5a (Poulos and Davis 1980). Taking the moments summation about the pile toe, the \( H_{ult} \) can then be calculated by:

\[
H_{ult} = \frac{0.5\gamma DL^3 K_P}{e+L} \quad \text{eq. 7-5}
\]

where \( e \) is the load eccentricity (i.e. height of the load above the soil surface), and \( L \) is the embedded pile length. In our study, \( e \) and \( L \) were equal to 0.15 m and 0.8 m, respectively.

Table 7.2 summarizes the PGHPs lateral capacities estimated using the analytical methods discussed above and compares them with the measured lateral resistance.

**Table 7.2: Estimated and measured lateral capacity of PGHPs**

<table>
<thead>
<tr>
<th>Pile group</th>
<th>Pile</th>
<th>Measured lateral resistance (kN)</th>
<th>Estimated lateral resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGHP3-D</td>
<td>Pg 3.6-D70</td>
<td>3.50</td>
<td>3.81</td>
</tr>
<tr>
<td></td>
<td>Pg 3.8-D100</td>
<td>3.50</td>
<td>3.81</td>
</tr>
<tr>
<td>PGHP3-L70</td>
<td>Pg 3.4-L70</td>
<td>2.25</td>
<td>2.81</td>
</tr>
<tr>
<td></td>
<td>Pg 3.5-L70</td>
<td>2.50</td>
<td>2.81</td>
</tr>
<tr>
<td>PGHP3-L100</td>
<td>Pg 3.10-L100</td>
<td>2.60</td>
<td>2.98</td>
</tr>
<tr>
<td></td>
<td>Pg 3.11-L100</td>
<td>2.75</td>
<td>2.98</td>
</tr>
</tbody>
</table>
The lateral capacity of PGHPs in dense sand predicted using, Petrasovits & Award (1972) and Prasad & Chari (1999) methods agree well with the experimental data, with a difference less than 7%. The performance of Petrasovits and Award method for PGHPs in loose sand was also excellent. On the other hand, Prasad and Chari method underestimated the lateral capacity of PGHP3-L70 and PGHP3-L100 by approximately 25% and 30%, respectively.

The method proposed by Meyerhof et al. (1981) generally underestimated the PGHP capacity by approximately 36% while the Broms (1964) method overestimated the lateral pile capacity by approximately 8–25%.

These results demonstrate that Petrasovits and Award (1972) method predicted the lateral capacity of PGHPs well despite the rigid tank boundaries. This is consistent with the finding of Dong et al. (2018) that the effect of the rigid boundaries at $H_{BD}/D_{shaft} > 6.7$ is less than 5.0% at a lateral displacement = 2.5% of the pile diameter.

After interpreting the ultimate lateral resistances ($H_{ult}$) of the different PGHP groups, the maximum bending moments acting along their shafts can be calculated. The depth ($f$) of the maximum bending moment (i.e. depth of zero shear) is evaluated first, then the maximum bending moment is given by moment summation at that point due to the lateral load ($H$) and soil resistance. Using Petrasovits and Award (1972), the depth of zero shear ($f$) was equal to 0.37 m for the different PGHP groups, which is close to the depth of the second strain gauges level (i.e. 0.39 m). Thus, it is reasonable to compare the calculated moment at this depth with the measured values.

The estimated bending moment was found to be 1.37, 1.00, and 1.06 kN.m for PGHP3-D, PGHP3-L70, and PGHP3-L100, respectively. For the same pile groups, the measured bending moment at the second level of strain gauges was equal to 1.11, 0.69, and 0.73 kN.m. These results demonstrate that Petrasovits and Award (1972) method overestimates the bending moment along the pile shaft by approximately 40%. A better agreement was obtained by Prasad and Chari (1999) method that can estimate the maximum bending moment along PGHP shaft with an accuracy greater than 87%. For the same pile groups, the estimated bending moment was found to be 1.11, 0.60, and 0.64 kN.m, respectively.
In conclusion, good predictions of the lateral resistance and the maximum bending moment acting along the pile shaft can be obtained by Petrasovits & Award (1972) and Prasad & Chari (1999) methods, respectively. Therefore, they are recommended to be used with short-rigid PGHPs tested under similar conditions. However, further investigation is still required to determine their validity under different testing environments.

7.2.4 Numerical analysis of lateral load testing

The developed FEM-1 model was used to simulate the lateral performance of PGHP3-L70, PGHP3-L100, and PGHP3-D considering the same Δr values summarized in Table 6.2. The material properties of both pile and soil were similar to those presented in Chapter 6. However, the top soil layer was divided into four sublayers each of 0.2 m thickness. The modulus of elasticity for each sub-layer was determined according to Seed and Idriss (1970) power function given by Equation 7-6.

\[ E_i = 1000 k_2 \sigma_{i3}^{0.5} (1+\nu) \]  
\[ \text{eq. 7-6} \]

where \( E_i \) is the elastic modulus of layer (i) in pound per square foot (psf), \( \sigma_{i3} \) is the average confining pressure along layer (i) in pound per square foot, \( \nu \) is the Poisson’s ratio, and \( k_2 \) is the modulus number to consider the effect of voids ratio and strain amplitude. For loose sand, \( E \) was found to be 4.0, 7.0, 9.1, and 10.8 MPa, respectively. While for dense sand \( E \) was taken as 5.3, 9.1, 11.8, and 13.9 MPa, respectively.

FEM-1 was found to overestimate the lateral pile resistance. However, it should be noted that the piles were subjected to compression and uplift loading before they were tested under lateral loads. Thus, the downward and upward movements of the tested pile may have reduced the shearing resistance at the pile-soil interface along the side of the piles, and the additional stresses due to the cavity expansion may have been relieved.

For PGHP3-L70, it was found that \( \Delta r = 0 \) provided best match with the experimental data as shown in Figure 7.7a. A section was then taken at the pile centerline to track the lateral displacement \( y \) along the pile shaft at different loading steps and the results are illustrated in Figure 7.7b. It is noted from Figure 7.7b that PGHP3-L70 rotated as a rigid pile around
a point at depth \((x = 0.57 \text{ m})\), which is approximately equal to 71\% of the embedded pile length \((L)\). The rotation depth is in excellent agreement with the findings of Meyerhof et al. (1981).

The soil resistance \((P_u)\) profile can be developed by measuring the lateral stresses \((S_{11})\) acting at the inner soil sides as presented in Figure 7.8.

The shape of the \(P_u\) distribution curve obtained from the FEM matches the profile suggested by Prasad and Chari (1999). However, Prasad and Chari suggested that \(P_u\) increases linearly to a depth \(= 0.6x\) (i.e. \(x\) is the rotation depth), then starts to increase in the opposite direction until it reaches the pile toe with a \(P_u\) value equal to 1.7 times the one recorded at 0.6x. The results obtained from the FEM exhibited a linear increase in \(P_u\) profile up to a depth \(= 0.22\)
m (≈ 39% of the rotation depth), and $P_u$ at the pile tip is approximately equal to the value at the curve threshold.

Two other sections were defined at the opposite sides of the pile cross-section to measure the vertical stresses ($S_{33}$) acting on the pile shaft under lateral loading as illustrated in Figure 7.9a. The measured $S_{33}$ values were then used to determine the bending moment profile along the pile depth

$$M_z = \frac{I_p(S_{331} - S_{332})}{D} \quad \text{eq. 7-7}$$

where $M_z$ is the bending moment at any depth ($z$), $I_p$ is the pile moment of inertia, $S_{331}$ and $S_{332}$ are the vertical stresses at the opposite sides of the pile cross-section, and $D$ is the pile diameter.

Figure 7.9b presents the obtained bending moment profile as well as those measured by the strain gauges during different loading stages (i.e. $P_{ult}/4$, $P_{ult}/2$, and $P_{ult}$, where $P_{ult}$ is the maximum lateral resistance).

**Figure 7.9:** Estimated bending moment along the pile shaft a) vertical stresses along the pile shaft, and b) bending moment profile

Figure 7.9b shows a good agreement between the calculated and measured bending moment profiles at the first and third levels of strain gauges (i.e. depth = 0.01 m and 0.79 m). The calculated bending moment at the second strain gauges level (i.e. depth = 0.39 m) was 35.0%
less than the measured bending moment at $P_{ult}/4$ loading step, and 20.0% less than the measured values at $P_{ult}/2$ and $P_{ult}$.

7.2.5 P-y curve

Matlock (1970) introduced the p-y curve method for the analysis of lateral pile response accounting approximately for the nonlinear behavior of soil. In this method, a series of non-linear springs are used in a beam on Winkler foundation (BWF) model as shown in Figure 7.10. The p-y curve represents a force-deformation relationship that relates the soil resistance ($p$) to the pile deflection ($y$).

![Diagram showing non-linear springs and p-y curves](image)

**Figure 7.10: p-y curve approach**

The distribution of the soil resistance ($p$) along the pile shaft can be obtained by double differentiating the bending moment function as illustrated by **Equation 7-8**.

$$P_z = \frac{d^2M}{dz^2} \quad \text{eq. 7-8}$$

The pile deflection ($y$) can be determined by double integration of the moment function given by **Equation 7-9**.

$$y_z = \frac{1}{E_pI_p} \int \int M_z \ dz \ dz \quad \text{eq. 7-9}$$
Unfortunately, the bending moment along the pile shaft was established using only three discrete points (i.e. second-degree polynomial). Thus, the double differentiation of the bending moment function will result in a constant soil resistance ($p$) along the pile shaft, which is not representative for piles in cohesionless soils (Matlock & Ripperger 1956, and Broms 1964). Therefore, after verifying the FEM, it was used to establish the $p$-$y$ curves at any soil depth by combining pile deflection shown in Figure 7.7b and the soil resistance curves presented in Figure 7.8b multiplied by the pile diameter (i.e. $D_{\text{shaft}} = 0.181$ m). Figure 7.11 illustrates the derived $p$-$y$ curves along the pile shaft.

![Figure 7.11: p-y curves along PGHP3-L70 developed by the FEM](image)

The deflection curves at 0.5 m and 0.6 m were not derived because the lateral pile deflection was negligible as they were close to the rotation depth (i.e. 0.57m). As can be noted from Figure 7.11, the $p$-$y$ curves up to a depth = 0.4 m had an initial linear segment, whose slope was found to be 2350 kN/m/m. This gives a modulus of subgrade reaction $k_h = \text{slope} / D_{\text{shaft}} = 12983$ kN/m$^3$ up to a deflection of approximately 2.5 mm ($\approx 1.4\%$ of $D_{\text{shaft}}$). This was followed by a nonlinear response due to soil yielding. The non-linear soil response is more pronounced closer to the surface due to the small soil resistance as a result of the lower overburden stress. A much stiffer initial response was exhibited for the $p$-$y$ curves at 0.7 and 0.8 m depths. This increase can be attributed to the increase in confining pressure of the surrounding soil (Wei 1998 and Fatahi et al. 2014).
7.2.5.1 Evaluation of tank boundary effect

The rigid boundary of the experimental tank may have influenced the lateral pile capacity and is expected to have increased the soil resistance. To investigate this effect, the lateral boundaries of FEM-1 was placed such that \( B_D = 10 \) times the shaft diameter (i.e. \( B_D \) is the distance from the pile surface to the side boundaries). The boundary effect is represented herein as the ratio between the interpreted lateral resistance (i.e. at 12.5 mm) within the actual soil boundary (ASB) and that estimated considering the extended soil boundary (ESB). Figure 7.12 presents the lateral load-deflection curves of the different PGHP groups (i.e. PGHD-D, PGHP-L70, and PGHP-L100).

![Figure 7.12: Effect of rigid boundary on the lateral resistance of PGHPs](image)

The results revealed a 7.6% increase in the lateral resistance of PGHP3-L70 due to the close tank boundary. The effect was more pronounced (i.e. 10.6%) for PGHP3-L100 due to the larger pile diameter. Moreover, despite the smaller shaft diameter of PGHP3 installed in
dense sand (PGHP3-D), the influence of the rigid tank boundary was found to be 8.5%. Thus, it is concluded that the effect of rigid tank boundary is more critical for large diameter piles and those installed in dense soil strata. For the PGHPs under discussion, the boundary effect was less than 11.0%. Hence, the measured data can be considered a good representative of the actual pile response. The lateral load test results estimated by FEM-1 for PGHP3-L100 and PGHP3-D are presented in Figure 7.13 and Figure 7.14, respectively.

Figure 7.13: Lateral load test results of PGHP3-L100 estimated by FEM-1
Figure 7.14: Lateral load test results of PGHP3-D estimated by FEM-1
7.3 References


Brinch-Hansen, J. 1961. The ultimate resistance of rigid piles against transversal forces. Bulletin Representative No. 12, Danish Geotechnical Institute, Copenhagen, Denmark, pp. 5–9.


Chapter 8

Conclusions and Recommendations

8.1 Summary

Pressure grouted helical pile (PGHP) is an innovative deep foundation system that involves grout injection under high pressure during the installation of a closed ended helical pile with a hollow pipe shaft. The grout is injected into the surrounding soil through two grout nozzles welded to the pile shaft. Although PGHPs are expected to be successful in many engineering applications with different soil conditions, they are not used in practice due to the lack of knowledge regarding the shape of the created grout column around the pile and its performance under different loading conditions.

A comprehensive investigation program was designed and implemented that included laboratory experiments and three-dimensional finite element modelling. The laboratory experiments comprised the installation of 5 small helical piles and 17 model PGHPs into cylindrical sand beds with different relative densities to represent loose, medium, and dense soil conditions. The PGHPs were installed with two different grouting pressures; 70 psi (480 kPa) and 100 psi (690 kPa). The piles were subjected to monotonic uplift, compression, and lateral load tests, then the PGHPs were extracted from the sand bed to provide a visual description of the created grout mass along their shafts.

The commercial software ABAQUS (SIMULIA, 2013) was then used to simulate the laboratory experiments to further understand the load transfer mechanism during loading and also to quantify the effects of the novel installation technique on the pile capacity and behavior. Following the calibration and the validation of the created models with the experimental data, the FE models were used to analyze the PGHP performance under different testing (i.e. relative density and grouting pressure) and loading (uplift, compression, and lateral) conditions. Finally, the FE models were extended to simulate the response of full-scale PGHPs and full-scale conventional helical pile under monotonic compression loading considering different shaft and helix diameters.
8.2 Conclusions

Based on the results of our investigation program, the following conclusions can be drawn:

8.2.1 Visual description of the created grout mass

1. PGHPs were installed with three different nozzle configurations (i.e. PGHP1, PGHP2, and PGHP3) to determine the best configuration for PGHP construction. Among the three nozzle configurations, PGHP3 was found to have the largest pile diameter, the highest grouting efficiency, and the lowest clogging susceptibility. Thus, it is recommended for PGHP construction.

2. When the grout nozzles were placed above the pile helix (i.e. PGHP1), a solid grout column was formed along the PGHP shaft. The diameter of such column was the same as the cavity created by the grout nozzles multiplied by an appropriate enlargement factor to take into account the effect of the grouting pressure and the relative density of the surrounding soil.

3. When the grout nozzles were placed below the pile helix (i.e. PGHP2 and PGHP3), a continuous spiral column with a solid grout core was created along the pile shaft. The diameter of the core and the dimensions of the ribs (i.e. thickness and height) are functions in nozzles configuration, grouting pressure, and soil density.

8.2.2 Monotonic uplift performance

1. The uplift resistance of PGHPs was primarily due to the shaft friction along the pile-soil interface, and it can be fully mobilized at low displacement levels ($\approx 2.1$ of $D_{\text{shaft}}$).

2. A reduction of 27.5% and 6.3% in the uplift capacity of PGHP1 and PGHP2, respectively, was observed compared to the conventional helical piles. While for the other PGHP groups, the uplift capacity was found to increase.

3. Although PGHP1 and PGHP2 had lower uplift capacities than the conventional helical piles, they developed their maximum resistances at lower displacement levels (i.e. friction piles), which indicates better performance under design loads.
4. The configuration of the grout nozzles was found to have a minor effect on the unit shaft resistance, $f_s$, at the pile-soil interface. On the contrary, $f_s$ was found to increase with increasing the grouting pressure and the sand relative density.

5. Screwing PGHP into the ground allows the grout nozzles to create a cavity around the steel pile shaft. At the same time, the injection of pressurized grout and the helix rotation displace the soil laterally (i.e. cavity expansion) and increase the lateral earth pressure and the unit shaft resistance at the pile-soil interface.

6. The unit shaft resistance of PGHPs under pullout loading can be analytically estimated using the $\beta$-method given by Burland (1973) multiplied by a placement method coefficient, $k_{mo}$, to account for the installation effect on the surrounding soil.

7. The placement method coefficient was found to vary with the grouting pressure. For PGHPs installed with a grouting pressure of 70 psi, $k_{mo}$ of 2.0 can be used. While for those installed with a grouting pressure of 100 psi, the $k_{mo}$ value increased to 2.6.

8.2.3 Monotonic compression performance

1. PGHP offers a significant increase in the pile capacity, over the un-grouted helical piles. This increase was more pronounced for piles installed in looser sand beds and those constructed with higher grouting pressure.

2. The increase in the pile capacity is related to the significant improvement in the pile shaft resistance ($Q_s$) and the end-bearing capacity of the supporting soil ($f_b$).

3. The observed increase in $Q_s$ is attributed to the formation of a larger diameter grout column, the higher friction angle ($\delta$) at the pile-soil interface, and the higher lateral earth pressure around the pile shaft.

4. The configuration of the grout nozzles was found to have a minor effect on the unit shaft resistance. On the contrary, $f_s$ was found to increase with increasing the grouting pressure and the sand relative density.

5. The unit shaft resistance of PGHPs under compression loading can be analytically estimated using the $\beta$-method given by Burland (1973) multiplied by a placement method coefficient ($k_{mo}$) equal to 3.72 for PGHPs installed with a grouting pressure of 70 psi and 4.14 if we increased the grouting pressure of 100 psi.
6. The higher $f_b$ observed for PGHPs is attributed to the weight of the grout tank that caused some densification at the soil directly below the pile helix, and/or the injected grout permeated through the soil voids and increased its strength.

7. The improvement in $f_b$ associated with the installation of PGHP is a function of the relative density (i.e. voids size) of the sand bed. Looser sand beds with higher voids ratio can be easily densified under the grout tank weight. Besides, larger voids allowed more grout permeation, and hence; more increase in the $f_b$.

8. For full-scale PGHPs installed to a deeper depth, the densification effect will be minor, and the permeation of grout into the soil voids cannot be controlled. Thus, it is not safe to rely on this increase for conservative design purposes.

8.2.4 Numerical analysis of axial load testing

1. Screwing PGHP into the ground allows the grout nozzles to create a cavity around the steel pile shaft. The cavity then got expanded under the effect of pressurized grout and helix rotation.

2. A strong relationship was observed between the cavity expansion ($\Delta r$), the grouting pressure ($P_g$), and the sand relative density (R.D). It was found that $\Delta r$ is directly proportional to $P_g$ and inversely proportional to R.D.

3. The results revealed a substantial increase in the radial stresses, and consequently, the friction resistance at the pile-soil interface due to the cavity expansion. Thus, a placement method coefficient ($k_{mo}$) has to be considered to account for the effect of the PGHP installation method on the surrounding soil.

4. The placement method coefficient ($k_{mo}$) can be determined by estimating the cavity expansion ($\Delta r$) form Equations 6-4, then use Equation 6-7 to calculate the increase in the radial stresses around the pile shaft.

5. The comparison between the results considering the actual soil boundary (ASB) and those for extended soil boundary (ESB) revealed 10% and 19.5% reductions in the pullout and compressive resistances of PGHP due to the influence of rigid tank boundary on the pile load test results.

6. For full-scale piles, an increase between 106% and 141% in the PGHP capacity over the conventional helical pile was observed due to the formation of a grout column
with slightly bigger diameter and the significant increase in the radial stresses at the pile-soil interface after installation.

7. The improvement in the PGHP capacity was found to be directly proportional with the shaft diameter to helix diameter ratio.

### 8.2.5 Monotonic lateral performance

1. PGHP has a significantly higher lateral capacity over the un-grouted helical piles. This considerable increase in the pile capacity is attributed to the larger shaft diameter, the existence of a disturbed soil zone around the un-grouted pile shaft, and the presence of the tank boundary at a relatively small distance.

2. The improvement is more pronounced for PGHPs installed in dense sand and those with larger shaft diameters.

3. The confining pressure in the annular zone around the pile (i.e. grouting pressure) did not have noticeable effect on the lateral pile capacity because the lateral pile response is primarily controlled by the soil properties within a wedge that extends to a distance \( \approx 6.1D_{\text{shaft}} \) from the pile surface.

4. Under lateral loading, PGHPs rotated as a rigid body around a point at depth \((x = 0.57 \text{ m})\), which is approximately equal to 71\% of the embedded pile length \((L)\).

5. For PGHPs, the soil resistance \((p_u)\) increases linearly to a depth \(= 0.22 \text{ m} \approx 0.39 \text{ the rotation depth, } x\), then it starts to increase in the opposite disrection until it reaches the pile toe with a \(p_u\) value equal to the one recorded at the curve threshold (i.e. 0.39x).

6. Good predictions of the lateral resistance and the maximum bending moment acting along the pile shaft can be obtained by Petrasovits & Award (1972) and Prasad & Chari (1999) methods, respectively.

7. The effect of rigid tank boundary is more critical for large diameter piles and those installed in dense soil strata. However, for the PGHPs under discussion, the boundary effect was less than 11.0\%. Hence, the measured data can be considered a good representative of the actual pile response.
8.3 Recommendations for future research

The results of the present study revealed the improved performance of model PGHPs in sand over conventional helical piles under various loading conditions (i.e. monotonic uplift, compression, and lateral loading). To further evaluate the system’s efficiency and the validity of the proposed design guidelines, the following are recommended for future research:

1. Monotonic axial and lateral full-scale field testing of the proposed pile in sand.
2. Monotonic axial and lateral full-scale field testing of the proposed pile in clay.
3. Perform full-scale field testing on pile groups to examine the group effect on the piles' performance.
4. Evaluate the PGHP’s performance under cyclic and dynamic loads.
Curriculum Vitae

Name: Mohamed Mansour

Place of Birth: Egypt

Post Secondary Education and Degrees:
- Master of Science in Public Works Engineering
  Cairo University, Giza, Egypt
  2011 - 2014
- Bachelor of Science in Civil Engineering
  Cairo University, Giza, Egypt
  2006 - 2011

Academic Appointments
- Research and Teaching Assistant
  Civil and Environmental Engineering
  Western University, London, Ontario, Canada
  2014 - 2019
- Research and Teaching Assistant
  Civil and Environmental Engineering
  Cairo University, Giza, Egypt
  2011 - 2014

Professional Appointments
- Full-time Geotechnical Engineer
  Roterra Piling Ltd.
  Acheson, Alberta, Canada
  2019 -