Investigation of Hollow Bar Micropiles in Cohesive Soil

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Graduate Program in Civil and Environmental Engineering

A thesis submitted in partial fulfillment of the requirements for the degree in Master of Engineering Science

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Investigation of Hollow Bar Micropiles in Cohesive Soil

(Thesis format: Integrated Article)

by

Osama F. El Hadi Drbe

Department of Civil and Environmental Engineering

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Engineering Science

The School of Graduate and Postdoctoral Studies
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London, Ontario, Canada

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Abstract

Micropiles are used in various applications, including low capacity micropile networks, underpinning and seismic retrofitting of existing foundations and high capacity foundations for new structures. They facilitate fast installation with a high degree of ground improvement. The current Federal Highway Administration (FHWA) design guidelines designate hollow bar micropiles as Type B micropiles, even though their construction technique is different than typical Type B, which results in overly conservative design. In addition, the current practice for construction of hollow bar micropiles is limited to drilling bit/hollow bar diameter ratio of 2.5 or less. In this thesis, full-scale load tests were conducted in order to: evaluate the suitability of FHWA design guidelines to hollow bar micropiles installed in cohesive soil; and to evaluate the performance of hollow bar micropiles constructed with drilling bit/hollow bar diameter ratio of 3. The load tests included axial monotonic and cyclic axial loading and monotonic lateral loading. Eight micropiles were constructed using 76 mm (3 in) hollow bars (76 mm OD and 48 mm ID) with air/water flushing technique and advanced to a depth of 5.75 m: six micropiles were installed using 228 mm (9 in) drill bit and two micropiles were installed using 178 mm (7 in) drill bit. All micropiles were instrumented with vibrating wire strain gauges to measure the axial strain at three stations along the micropile shaft. The axial load test results are presented and discussed in terms of load-displacement curves and load transfer mechanism. The load tests results showed that the grout/ground bond strength values proposed by FHWA (2005) for Type B micropiles grossly underestimate the bond strength for calculating the ultimate capacity. In addition, the toe resistance can be significant for micropiles resting on sand due to the increased toe diameter. No stiffness degradation was observed in the micropile capacity after applying 15 load cycles. Finally, lateral capacity of micropiles is moderate due to their small diameter. However, the larger drilling bit resulted in enhanced lateral performance and increased capacity due to the larger diameter. In addition, using fibre reinforced grout can increase the micropile lateral capacity and enhance its ductility.

Keywords: Micropiles, hollow bar micropiles, monotonic and cyclic loads, shaft and toe resistance, toe displacement, lateral load, LPILE, cohesive soils, lean clay.
Co-Authorship Statement

This thesis is prepared in accordance with the regulation for Integrated-Article format thesis stipulated by the school of graduate and post graduate studies at Western University, London, Ontario, Canada. All the field testing, numerical modeling, interpretation of results and writing of the draft and the final thesis were carried out by the candidate himself, under the supervision of Dr. M. Hesham El Naggar. The supervisor’s contribution consisted of providing advice throughout the research program, and reviewing the draft and the final thesis and publications results from this research. The results of the field tests, and numerical modeling presented will be used in journals and conferences publications, which will be co-authored with Dr. El Naggar.

A Part of Chapter 4: Axial Monotonic and Cyclic Compression Behaviour of Hollow Bar Micropiles

A part of this chapter is co-authored by Osama Drbe, and Hesham El Naggar and submitted in the “DFI-EFFC International Conference on Piling and Deep Foundations”.

Chapter 4: Axial Monotonic and Cyclic Compression Behaviour of Hollow Bar Micropiles

A version of chapter 4 will be submitted to the Canadian Geotechnical Journal

Chapter 5: Axial Uplift Behaviour of Hollow Bar Micropiles

A version of chapter 5 will be submitted to ASCE Journal of Geotechnical and Geoenvironmental Engineering.

Chapter 6: Monotonic Lateral Behaviour of Hollow Bar Micropiles

A version of chapter 6 will be submitted to DFI Journal.
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<tr>
<td>A</td>
<td>Pile cross-sectional area</td>
</tr>
<tr>
<td>(A_g)</td>
<td>Area of the grout</td>
</tr>
<tr>
<td>(A_s)</td>
<td>Area of the steel pile</td>
</tr>
<tr>
<td>(A_t)</td>
<td>Pile tip area</td>
</tr>
<tr>
<td>BH</td>
<td>Borehole</td>
</tr>
<tr>
<td>(C_d)</td>
<td>Correction for the borehole diameter</td>
</tr>
<tr>
<td>(C_h)</td>
<td>Rod energy ratio normalized to 60%</td>
</tr>
<tr>
<td>(C_N)</td>
<td>Overburden correction factor and calculated from</td>
</tr>
<tr>
<td>(C_r)</td>
<td>Correction for the rod length</td>
</tr>
<tr>
<td>(C_s)</td>
<td>Sampler correlation</td>
</tr>
<tr>
<td>(C_u)</td>
<td>Untrained shear strength of the soil adjacent to the foundation</td>
</tr>
<tr>
<td>(d_{\text{bar}})</td>
<td>Hollow bar diameter</td>
</tr>
<tr>
<td>(d_{\text{bit}})</td>
<td>Drill bit diameter</td>
</tr>
<tr>
<td>D</td>
<td>Diameter of pile</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>(E_g)</td>
<td>Modulus of elasticity of grout</td>
</tr>
<tr>
<td>(E_p)</td>
<td>Modulus of elasticity of pile</td>
</tr>
<tr>
<td>(E_s)</td>
<td>Modulus of elasticity of steel</td>
</tr>
<tr>
<td>(E_{\text{soil}})</td>
<td>Modulus of elasticity of soil</td>
</tr>
</tbody>
</table>
\(f_a\)  
Axial applied stress

\(F_a\)  
Allowable axial stress that would be permitted if only the axial force existed = 0.47 \(f_y\)

\(f_b\)  
Bending stress

\(F_b\)  
Allowable bending stress that would be permitted if only the axial force existed = 0.55 \(f_y\)

\(f'_c\)  
Compression strength of the grout

\(f'_e\)  
Euler buckling stress

\(FS\)  
Factor of safety

\(f_s\)  
Unit skin friction resistance

\(f_y\)  
Specific yield point of steel

\(I\)  
Moment of inertia of pile cross-section

\(I_{casing}\)  
Moment of inertia of the pile casing

\(ID\)  
Inner diameter

\(K\)  
Pile head stiffness

\(k\)  
Soil modulus parameter

\(k_{eff}\)  
Effective length factor

\(K_i\)  
Initial pile head stiffness

\(k_r\)  
Stiffness ratio

\(L\)  
Pile length

\(LL\)  
Liquid limit
\( l_e \)  
Effective pile length

\( L_{\text{unsupported}} \)  
Unsupported length of the micropile

\( M_{\text{allowable}} \)  
Allowable bending moment

\( M_{\text{max}} \)  
Maximum bending moment

\( \text{MP} \)  
Micropile

\( N \)  
Measured SPT number

\( N'_{c} \)  
Bearing capacity factor

\( N_{1,60} \)  
Standard penetration number, corrected for field conditions to an average energy ratio of 60\%, and overburden pressure

\( N_{60} \)  
Standard penetration number, corrected for field conditions to an average energy ratio of 60\%

\( \text{OCR} \)  
Overconsolidation ratio

\( \text{OD} \)  
Outer diameter of the pile

\( P_b \)  
Tip resistance

\( P_{C-\text{Allowable}} \)  
Allowable structural axial compression load

\( \text{PL} \)  
Plastic limit

\( \text{PI} \)  
Plasticity index

\( P_{\text{max}} \)  
Maximum axial compression load

\( P_{\text{max}} \)  
Maximum applied load at each cycle

\( P_{\text{min}} \)  
Minimum applied load at each cycle

\( P_p \)  
Applied load at the pile head
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{T-allowable}$</td>
<td>Allowable structural axial tension load</td>
</tr>
<tr>
<td>$P_{ult}$</td>
<td>Ultimate capacity of micropiles in tension or compression</td>
</tr>
<tr>
<td>$P_z$</td>
<td>Axial forces at different depths</td>
</tr>
<tr>
<td>$Q$</td>
<td>Applied Load</td>
</tr>
<tr>
<td>$Q_t$</td>
<td>Toe resistance</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Resistance of the shaft based on the diameter of the drill bit</td>
</tr>
<tr>
<td>$Q_{shaft}$</td>
<td>Shaft resistance obtained from the enlargement geometry</td>
</tr>
<tr>
<td>$r$</td>
<td>Radius of gyration of the steel casing</td>
</tr>
<tr>
<td>$S$</td>
<td>Elastic section modulus of the steel casing</td>
</tr>
<tr>
<td>$T_1$</td>
<td>Current temperature reading, in °C</td>
</tr>
<tr>
<td>$T_0$</td>
<td>Current temperature reading, in °C</td>
</tr>
<tr>
<td>$V_{grout}$</td>
<td>Volume of grout used in the micropile construction</td>
</tr>
<tr>
<td>$V_{hole}$</td>
<td>Volume of the hole based on the diameter of the drill bit</td>
</tr>
<tr>
<td>$V_{inc}$</td>
<td>Increase in pile volume</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Adhesion factor</td>
</tr>
<tr>
<td>$\alpha_{bond}$</td>
<td>Unit value for the grout/ground bond</td>
</tr>
<tr>
<td>$\alpha_c$</td>
<td>Linear expansion factor of EM-5 gauge wire = 11.5 (\mu)m/m°C</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Concrete expansion factor, in (\mu)m/m°C</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>Elastic displacement</td>
</tr>
<tr>
<td>$\Delta_{max}$</td>
<td>Maximum displacement at each cycle</td>
</tr>
</tbody>
</table>
\( \Delta_{\text{min}} \)  Minimum displacement at each cycle

\( \Delta_{sp} \)  Compression of the micropile shaft

\( \varepsilon \)  Total strain measurement, in \( \mu \)strains

\( \varepsilon_{50} \)  Strain corresponding to a shear stress equal to one-half of the shear strength of the material

\( \varepsilon_o \)  Initial reading, in \( \mu \)strains

\( \varepsilon_1 \)  Current reading, in \( \mu \)strains

\( \varepsilon_c \)  Creep strain, in \( \mu \)strains

\( \varepsilon_{e} \)  Total strain due to the applied effective stress, in \( \mu \)strains

\( \varepsilon_{h} \)  Strain due to the hydric and moisture effect, in \( \mu \)strains

\( \varepsilon_{\text{nsq}} \)  Total strain in the no-stress gauges, in \( \mu \)strains

\( \varepsilon_r \)  Real strain, in \( \mu \)strains

\( \varepsilon_s \)  Strain caused by other factors such as local strain discontinuities, in \( \mu \)strains

\( \phi' \)  Effective friction angle

\( \mu \)  Freedom factor of the concrete structure in the surrounding material \((0\leq\mu\leq1)\)

\( \mu\beta \)  Expansion factor

\( \sigma'_{o} \)  Effective vertical stress
Chapter 1

Introduction

1.1 Overview

Micropiles are a small-diameter (typically less than 300 mm) drilled and grouted-in-place deep foundation. They are classified as drilled and grouted non-displacement piles that are typically reinforced. The construction of micropiles involves three steps: drilling, placing reinforcement, and placing or pressurizing the grout. Due to innovative drilling and grouting techniques, high strength grout-ground bond can be achieved. Micropiles can carry substantial axial loads and moderate lateral loads, which makes them viable alternative for conventional driven piles or drilled shafts. Micropile applications include underpinning for existing foundations, in situ soil reinforcement, seismic retrofitting and as foundations for new construction.

Similar to conventional piles, high structural capacity of micropiles can be attained through the steel reinforcement and the grout. Geotechnically, however, micropiles are typically designed to transfer external loads to soil through skin friction rather than end bearing due to the large shaft area of the micropile compared to its toe area.

Numerous factors influence the selection of micropiles for applications in foundation and slope stabilization. Micropiles are appropriate for any type of ground conditions and can penetrate most obstacles. Because micropile rigs are physically small compared with other deep foundation equipment, they can be mobilized to remote and limited access sites. Additionally, the availability of micropile reinforcement as threaded sections of any length facilitates installation in low headroom conditions. Furthermore, micropile installation causes minimal vibration, noise, and spoil, making them a favourable option for urban construction. In contaminated soil sites, the use of micropiles offers an added advantage as it reduces the potential for surface contamination due to reduced soil spoils at the surface, and minimized corrosion deterioration as a result of the special admixtures used in the grouting. When added to an existing foundation, micropiles can provide
additional compression, tension, and moment resistance without the necessity of increasing the dimensions of the foundation. However, micropiles have some limitations related to their lateral capacity and cost effectiveness compared to conventional drilled shafts.

1.2 History of Micropiles

Micropiles have been in use since 1952, when Dr. Fernando Lizzi introduced them in Italy for underpinning historic buildings damaged during World War II (Bruce, 1988; and Cadden et al., 2004). They were originally employed in groups with small diameters of about 100 mm. Since then, their use has expanded rapidly to include both single elements and groups, with increased diameters of up to 300 mm.

The second generation of micropiles was developed in the 1970s, which were constructed using either an open or cased hole drilling method with a central threaded bar filled with grout. This type of micropile was indicated by a variety of names: minibales, pin piles, needle piles, and in North America, the “GEWI-Pile”. In 1993, the Federal Highway Administration (FHWA) and the International Society of Micropiles (ISM) standardized the name for the new piles as micropiles.

Ischebeck introduced a new generation of micropile in 1983, known as a Titan Injection Bore (IBO) micropile or a self-drilling micropile (CON-TECH SYSTEM, 2011; and Abd Elaziz 2012), with a central hollow bar as the reinforcing element, and at the same time is used as conduit for grouting. The use of hollow bar micropiles has increased markedly over the last 10 years. Hollow bar micropiles are gaining popularity because they provide fast and easy installation with a high degree of ground improvement. They can be placed in a one-step operation in which the hole is drilled and grouted simultaneously.
1.3 Research Objectives and Methodology

Typical hollow bar micropiles involves drilling bit diameter ($d_{bit}$) to hollow bar diameter ($d_{bar}$) ratio, $d_{bit}/d_{bar} = 2.5$ or less. As the micropile diameter is related to the drilling bit diameter, it is advantageous to increase $d_{bit}/d_{bar}$ ratio in order to increase the micropile diameter and hence increase the micropile axial and lateral capacity. The main objective of the research presented in this thesis was to investigate the effects of $d_{bit}/d_{bar}$ on the capacity and performance of hollow bar micropiles in cohesive soils. Therefore, the micropiles tested in this study were constructed employing two different sizes of drilling bits: six micropiles were constructed with a 228 mm drill bit diameter ($d_{bit}/d_{bar} = 3$) and two with a 178 mm drill bit diameter ($d_{bit}/d_{bar} = 2.35$). The hollow bar micropiles constructed with $d_{bit}/d_{bar} = 3$ were constructed using drilling bits designed and manufactured specially for this research, representing the first time ever construction of a hollow bar micropile with $d_{bit}/d_{bar} = 3$. The specific objectives of the study were to:

- Investigate the monotonic axial compression behaviour of hollow bar micropiles with two different drill bits and evaluate the optimal failure criteria for defining the axial ultimate capacity.
- Study the load transfer mechanism of hollow bar micropiles using vibrating wire strain gauges under axial monotonic compression loads.
- Examine the axial cyclic compression behaviour of hollow bar micropiles with two different drill bits.
- Investigate the load transfer mechanism of hollow core micropiles under axial cyclic compression loads, and assess the stiffness degradation of hollow bar micropile due to axial cyclic loads.
- Examine the axial monotonic uplift behaviour of hollow bar micropiles with two different drill bits, and ascertain the optimal method of determining failure criteria for defining the uplift ultimate capacity.
- Investigate the load transfer mechanism of hollow bar micropiles under axial monotonic uplift loads.
Study the lateral behaviour of hollow bar micropiles under lateral monotonic loads, and develop a numerical model for studying the effect of using different reinforced grout with different micropile head fixity conditions for improving the lateral capacity.

Study the effect of using two different drill bits on the behaviour of hollow bar micropiles, based on consideration of different grouting pressures created by differences in drill bit specifications.

To achieve the stated objectives, full-scale load tests were conducted on eight hollow bar micropiles: six constructed with a 228 mm drill bit diameter and two constructed with a 178 mm drill bit diameter. The load-testing program involved four different and consecutive phases:

1) Four axial monotonic tests were conducted to determine the axial ultimate capacity of the micropiles, three micropiles were constructed with the 228 mm drill bit and one was constructed with the 178 mm drill bit.

2) Four axial cyclic load tests were conducted to evaluate the cyclic performance of hollow bar micropiles, three micropiles were constructed with the 228 mm drill bit and one was constructed with the 178 mm drill bit.

3) Four axial uplift load tests were conducted to determine the micropiles uplift ultimate capacity, three micropiles were constructed with the 228 mm drill bit and one was constructed with the 178 mm drill bit.

4) Eight lateral load tests were conducted on all micropiles constructed in this study.

The original contributions of this study are:

- Evaluate the available design guidelines for hollow bar micropiles in compression with the available recommendations for the design of Type B micropile.
- Using a drill bit diameter (d_{bit}) to hollow bar diameter (d_{bar}) ratio, d_{bit}/d_{bar} > 2.5.
• Exploring the advantage of using fibre reinforced grout in construction of hollow bar micropiles in order to improve its lateral capacity and performance.

1.4 Thesis Organization

This thesis has been produced in accordance with the guidelines of the School of Graduate and Postdoctoral Studies. Parts of this thesis have been submitted and others will be submitted at international conferences and for publication in journals.

This thesis consists of seven chapters. Chapter 1 includes an introduction that highlights the advantages of using micropiles, provides a historical background, and outlines the objectives and methodology of this study.

Chapter 2 provides a review of the state of the art and practice with respect to micropiles, including the classification system and design philosophy. A brief description of the proposed system is also presented along with a review of previous studies of micropiles.

Chapter 3 describes the soil investigation program for the test site, including the field and laboratory tests. It also presents the materials used in this study, including different parts of hollow bar micropiles with their specifications as well as the installation process. The interpretation technique used with the vibrating wire strain gauges is also explained.

Chapter 4 provides a short introduction to the axial behaviour of micropiles and a description of the site conditions along with the installation technique used with the hollow bar micropiles. The axial monotonic and cyclic loading test procedures under compression loads are also described. The test results are presented and discussed in terms of load-displacement curves, skin friction, and toe resistance. The chapter concludes with a summary of the results.

Chapter 5 presents a brief introduction to the axial uplift behaviour of micropiles, along with a description of the site conditions and the installation technique. The axial monotonic loading test procedure under uplift loads is explained. The test results are
presented and discussed in terms of load-displacement curves, skin friction, and toe resistance. A summary of the results concludes the chapter.

Chapter 6 explains the lateral behaviour of micropiles, and provides a brief description of the site conditions and installation technique. The test procedure is also described, and the results are presented and discussed in terms of load-displacement curves. The chapter also describes the development of a numerical model and explains a parametric study that was conducted using a variety of fibres in order to study their effect on grout behaviour under different micropile head fixity conditions, and hence on lateral resistance.

Chapter 7 provides a summary of the research work completed, the conclusions drawn, and recommendations for future research.
1.5 References


Chapter 2

Literature Review

This chapter introduces the current state of practice and art with respect to micropiles, followed by a discussion of their classification and associated design philosophy. A review of previously completed studies of micropiles is then presented, concentrating only on studies that focus on the behaviour of micropiles under axial static, axial cyclic and lateral static forces.

2.1 Introduction

Piles are structural members used as a means of transferring structural loads to deeper and stronger soil layers where adequate support is available. Loads are transferred either through the distribution of the load along the pile shaft, which is denoted frictional pile, or through the use of the pile toe to transfer the load to a strong layer with good bearing capacity. In many cases, a combination of skin friction and end bearing transfers the load. Piles are generally classified according to the pile material, amount of ground disturbance during installation, and load transfer mechanism.

The term micropile refers to a specific type of pile that has been defined in a variety of ways (Federal Highway Administration (FHWA), 2005; Bruce et al., 1999; and Scherer et al., 1996). Most authors, however, agree that micropiles can be classified as small diameter (less than 300mm), bored, and grouted in place piles. These small diameter elements can sustain axial (compression and tension) and/or lateral loads. First introduced by Dr. Fernando Lizzi (Cadden et al., 2004), micropiles have been in use since 1952 when they were employed in Italy for underpinning historic buildings damaged during World War II (Bruce, 1988). Since then, micropile technology has evolved to cover a variety of applications: underpinning for existing foundations, in situ soil reinforcement, and seismic retrofitting. The last 20 years, however, have seen a significant expansion from use in low-capacity micropile networks to employment as single high-capacity foundations. One of the popular micropiles used in the foundation industry nowadays is
the hollow bar micropile, which provides fast installation with a high degree of ground improvement.

### 2.2 Classification of Micropiles

The piles can be classified based on material including: timber, concrete (cast-in-place or precast), steel (pipe pile or H-section), and composite piles. The classification related to ground disturbance during installation is categorized as large displacement, small displacement, or non-displacement. Large-displacement piles are driven or jacked timber, precast concrete, close-ended steel pipe, and fluted and tapered steel tube piles. The small-displacement category encompasses open-ended pipe piles, H-sections, steel box sections, and screw piles. No-displacement piles include bored piles and cast-in-place concrete piles. The final classification, associated with load transfer from the pile to the surrounding soil, denotes end-bearing, friction (floating), combined end-bearing and friction, or laterally loaded, piles (Poulos & Davis, 1980).

In the case of micropiles, the Federal Highway Administration (FHWA) introduced two classification criteria in addition to the designation based on diameter, constructional process, or nature of the reinforcement. As explained in the following subsection, these additional classifications are based on the philosophy of behaviour (design classification) and the method of grouting (construction classification) (Bruce et al., 1999).

#### 2.2.1 Design classification

Design classification can be further subdivided into two cases that differ based on the way the micropiles carry and transfer external loads.

CASE 1 micropile elements are loaded directly, with the micropiles resisting the majority of the applied loads both geotechnically through the grout/ground interaction and the end bearing resistance, and structurally through the pile reinforcement. This type of micropile is usually designed to transfer structural loads to stable soil or a bearing stratum, as shown in Figure 2.1. It is used as a substitute for conventional piles to carry axial or
lateral loads and is installed as either a single micropile or as a group of micropiles (FHWA, 2005).

On the other hand, CASE 2 micropile elements confine and reinforce the soil to create a composite mass system of reinforced soil that resists the applied load, as shown in Figure 2.2. The external loads are applied to the entire soil mass system, as opposed to the individual piles. CASE 2 is referred to as a reticulated network (FHWA, 2005).

Figure 2.1 CASE 1 micropiles (After FHWA, 2005)
In some situations, another subclass of micropile may be employed: a combination of CASE 1 and CASE 2 (Pearlman et al. 1992). In this case, the interaction of the pile and the ground occurs near the slide plane and that the pile group resists the external loads, in which case, the piles can be viewed as CASE 1 elements. The pile also adds a degree of stability to the reinforced composite soil structure above the plane of failure. From this perspective, the pile group acts as CASE 2 elements.

Figure 2.2 CASE 2 micropiles (After FHWA, 2005)
2.2.2 Construction classification

Drilling, placing reinforcement, and placing or pressurizing the grout are the three main steps in constructing micropiles, which affect the capacity of piles in different ways. The drilling method, for example, influences the degree of bonding between the grout and the ground, and both the placement of the reinforcement and the choice of grouting method have an impact on the development of the bond. In practice, in both micropiles and ground anchors, it is the grouting method that most affects the development of the grout/ground bond (Bruce et al., 1997). Neat cement grouts and sand-cement “mortars” are used as grout material. Based on the type of pressure applied during the grouting process, micropiles are classified as follows:

- **Type A**: The grout is placed under gravity pressure only using either sand-cement mortars or neat cement grout.

- **Type B**: Pressures typically in the range of 0.5 MPa to 1 MPa are applied in order to inject the neat cement grout into the drilled hole while the temporary drill casing is withdrawn.

- **Type C**: In this two-step process, neat cement grout is first placed in the hole under gravity pressure head only, as in type A. Prior to the hardening of this primary grout, a similar grout is then injected via a preplaced sleeved grout pipe at a pressure of at least 1MPa.

- **Type D**: Like type C, this type is a two-step process: first, neat cement grout is placed in the hole under gravity pressure head only, as in type A. However, in some cases, pressure could be applied as in type B. After several hours, once the primary grout has hardened, additional grout is injected via a sleeved grout pipe at a pressure of 2 MPa to 8 MPa. In some cases, a packer is used inside the sleeved pipe, enabling specific horizons to be treated. Figure 2.3 shows the four original types.

- **Type E**: This type is supplementary to the original four types (A to D) indicated by the FHWA. A threaded hollow bar connected to a drilling bit is advanced into the soil using air, water, or grout. The grout is then injected (typically up to 200 psi (1.38 MPa)) through the centre of the hollow pile, passing the nozzles in the drill bit to the
hole while the system is rotated. This type is also called a hollow bar (or core) micropile, which is described in detail in the next section (Timothy & Bean, 2012). Figure 2.4 shows Type E.

Based on the philosophy of behaviour and the method of grouting, a combination classification can also be used, which consists of a pattern of letters and numbers, e.g., A1, B2, C1, and D3. The number denotes the philosophy of behaviour (1 = CASE 1, 2 = CASE 2, and 3 = a combination of the two cases), and the letters relate to the grouting method (FHWA, 2005).

Figure 2.3 The four original micropile grouting classifications (After FHWA, 2005)
2.3 Hollow Bar Micropiles

As mentioned, type E micropiles, also known as hollow bar micropiles or self-drilling micropiles, are supplementary to the original four types (A to D). This type of micropile is employed in order to save time because it provides easy fast installation with a high degree of ground improvement. The process for placing types A to D requires multiple steps: drilling the hole, installing casing, installing the steel reinforcement, and then placing the grout. Hollow bar micropiles, in contrast, can be placed in a one-step operation in which the hole is drilled and reinforced simultaneously. However, in some of the published literature, hollow bar micropiles are categorized as type B.

This system has three main components as shown in Figure 2.4: a sacrificial drill bit with two or more nozzles, threaded hollow steel bars, and couplers for attaching the hollow bars. The installation process entails drilling the hole using the drill bit with an air and/or water and/or grout flushing technique. A competent grout is flushed through the hollow bars after the desired depth is reached so that visible evidence of the flushed disposal soil and grout is obtained. The competent grout is typically injected at a pressure up to 1.38 MPa (200 psi). The flushing technique necessitates a borehole diameter slightly larger than the diameter of the drill bit.
Figure 2.4 Hollow bar micropiles
2.4 Micropile Design Philosophy

Micropiles should be designed to be capable of sustaining the anticipated loading conditions at safe stress levels and within acceptable displacement limits. This section focuses on methods of estimating the structural axial and geotechnical capacities.

2.4.1 Structural axial capacity

Micropiles transfer external loads to a deeper, more competent or stable stratum. The structural load is resisted primarily by the steel reinforcement and the geotechnical capacity of the grout/ground bond zone of the individual piles. The grout also increases both the axial and lateral structural load capacity, and provides some protection from corrosion. As well, the geotechnical uplift capacity is increased slightly because of the additional load of the grout.

The American Association of State Highway and Transportation Officials (ASSHTO) (1996), FHWA (2005), and the International Building Code (IBC) (2006) provide guidelines for consideration of the compressive and tensile capacity of micropiles:

- For compression, per FHWA:
  \[ P_{C-Allowable} = 0.4f'_c A_g + 0.47f_y A_s \]  \hspace{1cm} (2.1)

- For compression, per IBC:
  \[ P_{C-Allowable} = 0.33f'_c A_g + 0.40f_y A_s \]  \hspace{1cm} (2.2)

- For tension, per FHWA:
  \[ P_{T-Allowable} = 0.55f_y A_s \]  \hspace{1cm} (2.3)

- For tension, per IBC:
  \[ P_{T-Allowable} = 0.60f_y A_s \]  \hspace{1cm} (2.4)
where

\( A_g \) = the area of the grout

\( A_s \) = the area of the steel bar

\( P_{C-allowable} \) = the allowable structural axial compression load

\( P_{T-allowable} \) = the allowable structural axial tension load

\( f'_c \) = the compression strength of the grout

\( f_y \) = the specific yield point of steel

Calculating the compressive capacity of micropiles should include consideration of limiting the allowable compressive strength (strain compatibility) of the micropile components. The maximum yield stress of steel should therefore be included in Equations 2.1 and 2.2 as the lesser of a) the yield stress of steel and b) the maximum stress of the grout based on the compression strain of the grout (0.003). That is to say, the grout and steel are limited to a value of 0.003 of the compression strain of the grout, and the stress in the steel at 0.003 of the strain is equal to Young’s modulus (E) multiplied by the strain (0.003) (FHWA, 2005). Therefore, the allowable yield stress of steel is 200,000 MPa × 0.003 = 600 MPa. The lesser of the yield stress of the steel as provided by the manufacturer and 600 MPa is used in Equations 2.1 and 2.2.

Owing to their small diameter, micropiles are considered to have a small lateral capacity. Richards and Rothauaer (2004) studied the performance of micropiles under lateral loads and suggested a combined stress check equation for micropiles that are subjected to lateral loads or overturning moment. This method accounts for the contribution of the grout inside the casing to the compression capacity, and neglects the potential of the buckling. A combination of the axial compression and bending that result from a lateral load is included in the following equation:
\[
\frac{P_c}{P_{c-allowable}} + \frac{M_{\text{max}}}{M_{\text{allowable}}} \leq 1
\]  
(2.5)

where

\[
M_{\text{allowable}} = 0.55f_y \times \left(\frac{2I_{\text{casing}}}{OD}\right)
\]  
(2.6)

\(P_{c-allowable}\) = the allowable structural axial compression load

\(P_{\text{max}}\) = the maximum axial compression load

\(M_{\text{max}}\) = the maximum bending moment

\(M_{\text{allowable}}\) = the allowable bending moment

\(I_{\text{casing}}\) = the moment of inertia of the pile casing

\(OD\) = the outer diameter of the pile

In the structural steel sections of its guide, AASHTO (2002) offers a conservative method of accounting for the combined stress. It is worth mentioning that this method ignores the contribution of the grout and assumes that the steel casing carries only the applied compression force. The design check for combined stresses is given by

\[
\frac{f_a}{F_a} + \frac{f_b}{\left(1 - \frac{f_a}{F_e}\right) f_b} \leq 1
\]  
(2.7)

where

\(f_a\) = the axial applied stress

\(f_b\) = the bending stress = \(M_{\text{max}}/S\)

\(S\) = the elastic section modulus of the steel casing
fa = the allowable axial stress that would be permitted if only the axial force existed
= 0.47 fy

fb = the allowable bending stress that would be permitted if only the axial force
existed = 0.55 fy

Fe = the Euler buckling stress

\[ Fe = \frac{\pi^2 E}{FS \left( \frac{Kl_{unsupported}}{r} \right)^2} \]  

where

E = Young’s modulus

FS = a factor of safety equal to 2.12

keff = an effective length factor equal to 1

L_{unsupported} = the unsupported length of the micropile

r = the radius of gyration of the steel casing
2.4.2 Geotechnical capacity

According to FHWA (2005), micropiles attain load carrying capacity mainly through skin friction rather than end bearing. Multiple factors are involved in the load transfer mechanism described. First, the area of contact with the ground is greater along the length of the pile than the area of contact at the pile toe. A smaller movement is sufficient to mobilize the skin friction than is required to produce an effect at the end bearing. An additional factor is the innovative grout pressurizing technique in micropile installation that enables the attainment of the grout-to-ground bond capacity. Because the design of the micropile depends on skin friction, the geotechnical capacity in compression is equal to the geotechnical capacity in tension.

FHWA (2005) provides guidance for estimating the geotechnical capacity of micropiles. The ultimate capacity of micropiles in tension and compression is given by

$$P_{ult} = \alpha_{bond} \times \pi \times D_{bond} \times (bond \ length)$$  \hspace{1cm} (2.9)

where

$\alpha_{bond} =$ the unit value for the grout/ground bond, as shown Table 2.1

Table 2.1 provides ranges for the nominal strength of the grout-to-ground bond according to the four methods of pile grouting. These values are based on the experience of geotechnical engineers and are used only for preliminary micropile design. To ensure that no geotechnical failure occurs, FHWA suggests that an overall factor of safety of 2.5 be used when the piles are tested at double the design load. Maclean (2010) recommended using an enlargement factor that covers the increase in the diameter of the micropiles during installation and grouting. He suggested a value of 1.3 for micropiles in sand, 1.2 for micropiles in clay, and 1 for micropiles in rock.
Table 2.1 Typical Ranges of the Ultimate Strengths of the Grout/Ground Bond (FHWA, 2005)

<table>
<thead>
<tr>
<th>Soil/Rock Description</th>
<th>Typical Range of Grout/Ground Bond Ultimate Strengths (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type A</td>
</tr>
<tr>
<td>Silt &amp; Clay (Some Sand) (Soft, Medium Plastic)</td>
<td>35-70</td>
</tr>
<tr>
<td>Silt &amp; Clay (Some Sand) (Stiff, Dense to Very Dense)</td>
<td>50-120</td>
</tr>
<tr>
<td>Sand (Some Silt) (Fine, Loose to Medium Dense)</td>
<td>70-145</td>
</tr>
<tr>
<td>Sand (Some Silt, Gravel) (Fine-Coarse, Medium to Very Dense)</td>
<td>95-215</td>
</tr>
<tr>
<td>Gravel (Some Sand) (Medium to Very Dense)</td>
<td>95-265</td>
</tr>
<tr>
<td>Glacial Till (Silt, Sand, Gravel) (Medium to Very Dense, Cemented)</td>
<td>95-190</td>
</tr>
<tr>
<td>Soft Shales (Fresh to Moderate Fracturing, Little to no Weathering)</td>
<td>205-550</td>
</tr>
<tr>
<td>Slates and Hard Shales (Fresh to Moderate Fracturing, Little to no)</td>
<td>515-1380</td>
</tr>
<tr>
<td>Limestone (Fresh to Moderate Fracturing, Little to no Weathering)</td>
<td>1035-2070</td>
</tr>
<tr>
<td>Sandstone (Fresh to Moderate Fracturing, Little to no Weathering)</td>
<td>520-1725</td>
</tr>
<tr>
<td>Granite and Basalt (Fresh to Moderate Fracturing, Little to no Weathering)</td>
<td>1380-4200</td>
</tr>
</tbody>
</table>
$\alpha$-Method can be used for evaluating capacity of piles installed in clay. The undrained load capacity is generally taken to be the critical value unless the clay is highly over-consolidated. In this method, the unit friction resistance is given by

$$f_s = \alpha C_u$$  \hspace{1cm} (2.10)

where

$f_s =$ the unit-side friction resistance

$\alpha =$ the adhesion factor

$C_u =$ the untrained shear strength of the soil adjacent to the foundation

The ultimate capacity of the soil grout interface is calculated using

$$Q_s = \pi d \int_0^{l_e} \alpha C_u dz$$  \hspace{1cm} (2.11)

where

$l_e =$ the effective pile length

Bruce (1994) suggested 0.6-0.8 as the adhesion factor for type A and B micropiles. Elkasabgy and El Naggar (2007) found that the range of the adhesion factor for hollow bar micropiles is 0.8 to 1, with a best estimate of 0.9, which is higher than that proposed by Bruce (1994).

FHWA ignores the end bearing capacity in clay; however, the concept of end bearing resistance for drilled shafts or driven piles can be used to estimate the toe resistance. An untrained condition is assumed in clays beneath the toe of a deep foundation due to their low hydraulic conductivity (Coduto, 2001). Therefore, the ultimate toe resistance can be calculated from the following:

$$Q_t = N_t C_u A_t$$  \hspace{1cm} (2.12)
where

\[ N'_c = \text{the bearing capacity factor (O’Neill & Reese, 1999)} \]

\[ = 6.5 \text{ at } S_u = 25 \text{ kPa} \]

\[ = 8.0 \text{ at } S_u = 50 \text{ kPa} \]

\[ = 9.0 \text{ at } S_u \geq 100 \text{ kPa} \]

\[ C_u = \text{the undrained shear strength of the soil under the toe} \]

\[ A_t = \text{the pile toe area} \]

Abdelaziz (2012) correlated the increase in the resistance of the hollow bar micropile shaft under axial compression and uplift loads to the increase in its diameter during installation and grouting. He established the correlation through a parametric study of the following parameters: undrained shear strength varying from 90 kPa to 175 kPa; adhesion factor varying between 0.9 and 1; slenderness ratio varying between 30 and 50; and increase in micropile diameter varying between 25% to 100% of the diameter of the drill bit. He found that the increase in the compressive shaft resistance can be given by

\[ Q_{shaft} = (1 + 0.35V_{inc})Q_s \]  \hspace{1cm} (2.13)

where

\[ Q_{shaft} = \text{the shaft resistance obtained from the enlargement geometry} \]

\[ V_{inc} = \text{the percentage increase in pile volume} \]

\[ V_{inc} = \frac{(V_{grout} - V_{hole})}{V_{hole}} \]  \hspace{1cm} (2.14)
$V_{\text{grout}}$ = the volume of grout used in the micropile construction

$V_{\text{hole}}$ = the volume of the hole based on the diameter of the drill bit

$Q_s$ = the resistance of the shaft based on the diameter of the drill bit

$$Q_s = \pi d \int_0^{lev} \propto S_u dz$$  \hspace{1cm} (2.15)

The increase in the uplift shaft resistance is given by

$$Q_{\text{shaft}} = (1 + 0.275V_{inc})Q_s$$  \hspace{1cm} (2.16)

In piles, the load is transferred to soil through both the shaft and toe resistances. Abdelaziz (2012) found that the percentage of toe resistance depends on the amount of increase in the micropile diameter during installation. The toe resistance for hollow bar micropiles under compression is given by

$$Q_t = 9C_u A_{\text{hole}}$$  \hspace{1cm} (2.17)

The toe resistance for hollow bar micropiles under tension is given by

$$Q_t = 9C_u 2.5A_{\text{inc}}$$  \hspace{1cm} (2.18)

where

$C_u$ = the undrained shear strength of the soil under the toe

$A_{\text{hole}} = V_{\text{grout}}/L$

$A_{\text{inc}} = V_{\text{inc}} \cdot V_{\text{hole}}/L$
2.5 Previous Studies of Micropiles

Many researchers have investigated the performance of micropiles under axial and lateral monotonic and cyclic loads using field and laboratory experiments as well as numerical modeling. The load transfer mechanism theories proposed by different researchers are reviewed first. The axial and lateral monotonic and cyclic behaviour of micropiles are then discussed. Finally, the studies involving hollow bar micropiles are summarized.

2.5.1 Load transfer mechanism theories

Bruce and Yeung (1984) suggested that the design limitation of micropiles is related to its small cross-sectional area. In addition, the geotechnical capacity is dictated by skin friction, as opposed to end bearing. Their explanation included an example of a pile with a diameter of 200 mm and a length of 5 m, which has a surface area more than 100 times greater than the cross-sectional area. They found that the pile requires a settlement of 10\% to 20\% to mobilize the bearing capacity, compared with only 0.5\% to 1\% to mobilize the maximum skin resistance. Juran et al. (1999) identified the skin friction of the micropiles as the main contributor to the load transfer so that a micropile is designed to transfer the load only through the resistance of the shaft. They used the sample pile from the Bruce and Yeung (1984) study in order to explain the load transfer mechanism. They found a pile with 5 m long and 200 mm in diameter requires 20 to 40 times less movement to mobilize the skin friction than to mobilize the end bearing.

FHWA (1997 and 2005) recommends that the end bearing resistance can be neglected due to the slenderness of the micropile and its small toe area and that the shaft resistance carries the external applied load. It is stated state that the end bearing resistance should be taken into account only in the case of piles on rock and, in this case, the end bearing resistance should be evaluated same as in both a drilled shaft and a driven pile.

Cadden et al. (2004) indicated that micropiles carry the load through the friction resistance along the length of the pile between the grout and the surrounding soil. This assumption is based on the small cross-sectional area of the pile compared to the surface
area plus the softening of the soil at the toe due to the installation techniques. They mentioned that the end bearing contribution could be considered in the case of short piles installed in hard rock

2.5.2 Post-grouted micropiles

Jones and Turner (1980) demonstrated significant increase in the capacity of the repeated post-grouted micropiles (types C and D) installed in stiff clay soil. However, they did not observe any evidence of enhanced behaviour for these micropiles in very soft un-consolidated or soft clay. Mascardi (1982) indicated that a significant increase is anticipated in the diameter of post-grouted micropiles, which effectively increases their capacity in cohesionless soils, shales, residual soils, and weaker sedimentary and low-grade metamorphic formations. FHWA (1997) reported that the skin friction of post-grouted micropiles has been established for limited types of soils, such as clay with medium to high plasticity. Additional research should be conducted with the goal of examining the effect of post-grouting on skin friction for different types of cohesive soils.

2.5.3 Axial behaviour of micropiles under monotonic and cyclic loads

Bruce et al. (1993) conducted extensive laboratory and full-scale field tests with different pile configurations. The laboratory work included three phases: first, simulating the upper section of a typical high-capacity micropile using single grout-filled steel casings; second, testing the same configuration but with threaded ends connected to the sections; and third, installing internal reinforcement bars to simulate the lower section of the micropiles. The full-scale load tests were conducted on two underpinning projects and enabled the investigation of both the elastic performance of the micropiles and the debonding phenomenon. Based on the results of these investigations, they developed the elastic ratio (ER) concept.

Jeon and Kulhawy (2001) presented the results for 21 full-scale tests in both cohesive and cohesionless soil. The findings were used to establish a design procedure that incorporates both estimated axial displacements and soil properties. They tested
micropile types B, C, and D, and investigated the load displacement behaviour in order to estimate the axial compression capacity since most of the tests were terminated before geotechnical failure. The results revealed that the load-carrying capacity of a drilled shaft differs from that of pressure-grouted micropiles. At shallower depths, i.e., depth to diameter (D/B) ratio < 100, a significant increase in capacity relative to larger diameter drilled shafts was observed. In cohesive soil, the increase can be from 1.5 to 2.5 times the capacity of the drilled shaft. In cohesionless soils, the increase can be in the range of 1.5 to 2.5 times with a maximum of 6 times. For D/B > 100, the behaviour and capacity of the micropiles and the drilled shaft appear to be identical, with the exception of the expected increase in the effective diameter of the micropiles.

Russo (2004) presented the load test results for two full-scale piles installed in soft rock of volcanic origin, overlain by layered pyroclastic soils (volcanic sands, pozzolana, and pumices). The micropiles were installed by drilling 200 mm diameter holes and inserting steel pipes equipped with injection valves. Both micropiles were instrumented with vibrating wire strain gauges but the grouting procedure was different for each micropile. The load tests were terminated before the ultimate bearing capacity was reached; however, the readings from the vibrating wire strain gauges along with finite element modeling enabled an estimation of the ultimate skin friction based on the predication of the ultimate load using method. The results showed that the micropile performance is influenced by grouting and installation methods.

Han and Ye (2006) reported a full-scale load test of four piles installed in soft clay. Two piles were subjected to compression loading and two piles to tension loading. Rebar strain gauges were inserted into the micropiles to monitor the amount of strain during the tests. The researchers presented the data collected in terms of load-displacement responses, elastic moduli, axial forces, tip resistance, and skin friction. The compression results showed that the skin friction values measured were 19 % to 59 % higher than the values suggested for bored concrete piles, and the tension results showed that the skin friction values measured were 4 % to 10 % higher than those suggested for bored concrete piles. In addition, the ultimate skin friction for the micropiles was 0.9 to 1.2
times the undrained shear strength with respect to compression and 0.68 to 0.73 times the undrained shear strength with respect to tension.

Holman and Tuozzolo (2007) examined three piles from two different case histories, all of which were instrumented with 34 vibrating wire strain gauges. The vibrating wire strain gauges were welded to centralized reinforcing bars and inserted into neat cement grout. All piles were type B pressure grouted (345 kpa). The first two piles were installed in cohesionless soil. One of the piles was tested to plunging failure and one to impending failure. The third pile was tested to plunging failure in a layer of silt, sand, and clay overlying glauconitic fine to medium sand. The results showed that, with increased strain levels within the pile, the modulus of elasticity decreases and also that the mobilized unit bond stress is inconsistent with the non-uniform behaviour of the grout. The data were used to demonstrate a relationship between the secant modulus and the measured strain. Linearized degradation relationships were produced for the load test data sets and were found to be reasonable when compared to the field data. The ultimate bond stress developed at 6 mm to 8 mm of displacement. For the two piles that plunged, the maximum tip resistance developed at displacement of 8 % to 10 % of the diameter of the pile tip.

Thomson at al. (2007) reported the results of axial compression, axial tension, and lateral load tests on pre-production micropiles prior to use in upgrading the existing pier foundations of the Nipigon River Bridge in Ontario, Canada. They found that the mobilized grout-to-ground strengths for two micropiles under an uplift load were approximately 150 kPa and 190 kPa based on the outside drill casing diameter. They also determined that the volume of grout used in the hole was greater than the theoretical volume of the hole, an indication that the diameter of the uncased portion of the micropiles was increased.

Cavey et al. (2000) studied the axial cyclic performance of pressure-grouted micropiles installed in loose to medium dense sand and silt. They observed a 60 % reduction in ultimate capacity after two cycles of loading and found that micropiles installed in
cohesionless soils behave like drilled shafts, with a critical level of repeated loading and a low ultimate capacity value compared with static conditions.

Gómez et al. (2003) investigated the performance of a micropile installed in rock under axial cyclic loading. The load transfer mechanism and the bond strength during the cyclic loading were evaluated using the readings from four vibrating wire strain gauges installed within the grout. They did not detect any physical debonding of the grout-ground interface although a post-peak reduction in bond strength was observed. Progressive increases in both the elastic length and elastic ratio of the micropiles were observed as the applied load was increased. They concluded that the elastic length of a micropile can be useful for assessing its response, although locked-in bond stresses along the micropile may lead to unconservative estimates of bond strength in the case of significant residual elastic compression upon unloading.

2.5.4 Lateral behaviour of micropiles

FHWA (1997) indicated that small-diameter piles need a high degree of relative soil-pile movement in order to mobilize the ultimate lateral earth pressure, and hence, the pile bending resistance. Due to the slenderness of micropiles, their lateral loading capacity is smaller than their axial capacity and is governed by their yield moment. FHWA (1997) recommends the use of reinforcement within the upper portion of micropiles.

Richards and Rothbauer (2004) examined the lateral load performance and design of micropiles through load testing 20 encased micropiles that were installed at eight different sites. Eight micropiles were installed in cohesionless soil and 12 in cohesive soil. The lateral load results were compared with responses calculated using the program LPILE (Ensoft, 2000), Foundations and Earth Structures, Design Manual (NAVFAC, 1986), and the characteristic load method developed by Duncan et al. (1994). The comparison of measured and calculated results indicated that analyses overestimated the response. They explained these findings as resulting from the conservatism usually applied in assigning soil parameters or in neglecting the passive surcharge due to the top
of the pile being below the ground surface. They also found that the lateral response of the micropiles to be very sensitive to the soil type along the upper 2 m to 5 m of the pile.

Long et al. (2004) conducted 10 lateral load tests on micropiles that were installed in a clay layer 5 m thick overlying a thick layer of sand. The micropiles were 15.2 m long, and consisted of a steel casing 244 mm in diameter 13.8 mm wall thickness filled with grout and reinforced with a central high-strength threaded bar along the entire length. The results of the lateral load tests were compared with the behaviour predicted using the p-y method, and good agreement was revealed between the predicted and measured displacement with 10 percent margin of error.

Thomson at al. (2007) tested four micropiles that had an external steel casing and instrumented with dial gauges at the head and inclinometers along their shafts. They evaluated the lateral deflection at the heads using the dial gauges, and the load deflection curve along the shafts using inclinometers readings. They observed different responses of the four micropiles due to the variation in the installation conditions, although identical procedures were followed for all micropiles. The discrepancies were attributed to differences in the amount of grout used; in some cases, the amount of grout used was less than the theoretical volume of the hole, and in other cases, it was 1.8 times the volume of the hole. Micropiles in which large amounts of grout were used generally exhibited stiffer behaviour than others in which smaller amounts were employed.

### 2.5.5 Hollow core micropiles

Bishop et al. (2006) studied the performance of hollow bar micropiles under service loads in five different projects. The hollow bar micropiles were installed in different soil types using the single-stage construction technique. They found the capacity of the hollow-bar micropiles to be greater than their crushing strength under compression loads. A high level of skin friction was created, which allows carrying moderate to moderately high loads. A low level of deflection under the design load was observed, which was attributed to the combined stiffness of the soil and the hollow bar system. They also determined that
the capacity of a pile group is equal to the sum of the individual micropiles and is greater in cases in which the piles are placed at a distance of 0.5 m apart.

Gómez et al. (2007) studied the performance of 260 hollow bar micropiles under monotonic axial loading; 180 were installed in submerged sand and 80 were installed in stiff, silty clay. The micropiles consisted of Titan IBO 52/26 (52 mm O.D./26 mm I.D.) hollow bars with a casing over the upper part. The micropiles were drilled using a grout flushing technique. All production micropiles were proof tested up to 150% of the design load (80 kip), and verification tests were performed on four micropiles to 250% of the design load. They reported that hollow bar micropiles provided faster installation than other micropile types allowed more reliable quality control procedures. They found that the grout-ground bond was greater than that calculated for pressure-grouted (type B) micropiles in granular soils.

Telford et al. (2009) conducted verification tests on 25 micropiles consisting of threaded Titan 73/45 hollow bars installed to a depth of 9.8 m in cohesionless soil using a 115 mm cross drill bit. The verification tests involved compression and uplift loading to confirm the ability of the micropiles to carry high compression and tensile loads with only small movements of the pile head. The maximum compression load reported was 1331 kN, with elastic and residual movements measured at the pile head of 8.9 mm and 6.3 mm, respectively. The maximum tension load reported was 953 kN, with elastic and residual movements measured at the pile head of 13 mm and 7.4 mm, respectively. The results showed that micropiles were capable of carrying high loads in both compression and tension with a small amount of deformation. Considering that the pile diameter was enlarged by 1.5 times over the nominal size (115mm), the grout-to-soil bond strength test values were close proposed by FHWA (2005).

Bennett and Hothem (2010) conducted load tests on four pairs of micropiles installed in a layer of soft clay/sandy clay/very loose to loose clayey sands extending from the ground surface to approximately 6 m. Below that, a layer of medium dense to very dense silty sand/sand (extending to a depth varying from 7.6 m to 9 m) and a layer of medium to very stiff sandy clay/dense to very dense clayey sand/ sand extending to depths greater
than 15 m. The four pairs were constructed to depths of 7 m, 8.5 m, 10 m, and 11.5 m. Two different drill bits were used for each pair: a 150 mm clay bit and a 115 mm cross bit. The shortest pile carried 480 kN and 460 kN for the 150 m clay bit and the 115 mm cross bit, respectively. The 150 mm clay bit performed marginally better than the 115 mm cross bit.

Abd Elaziz and El Naggar (2010, 2011, and 2012) examined five micropiles installed in a thick layer of overconsolidated clayey silt to silty clay till overlying a layer of compact to dense sand extending to a depth of 9 m. The groundwater table was at a depth of 3.7 m to 4.0 m below the ground surface. Three micropiles were subjected to monotonic compression loads, and two to monotonic tension loads. All piles were consisted of hollow bars (76 mm OD and 48 mm ID) connected to a 178 mm carbide drill bit. The results revealed that considering hollow bar micropiles as type B underestimates the grout-ground bond strength and that the micropiles performance can be explained based on the elastic length approach. Abd Elaziz and El Naggar (2012) investigated the behaviour of micropiles under axial cyclic loads. Four compression and one tension cyclic load tests were conducted on four micropiles. Each micropile was subjected to 15 cyclic loads. They reported that the cyclic loading resulted in small total increases in the pile head displacement (6 % to 18 % of the monotonic loading displacement). They observed no change to a slight increase in the pile head stiffness after 15 load cycles, and no debonding between the grout and the ground was observed during the tests.

This review shows that the available data with respect to hollow bar micropiles behaviour is limited, and is mostly for medium to high plasticity clay and sand. In addition, most of the available research has focused on the behaviour of micropiles under axial monotonic loads. Finally, all hollow bar micropiles construction reported in the literature involves piles constructed with \( \frac{d_{bit}}{d_{bar}} \) ratio < 2.5. Further investigation is thus required with respect to behaviour of hollow bar micropiles installed in low plasticity clay under different types of loading: axial static, axial cyclic and lateral static. Also, the construction of hollow bar micropiles with higher \( \frac{d_{bit}}{d_{bar}} \) ratios should be explored to enhance the capacity of hollow bar micropiles under axial and lateral loads.
2.6 References


Chapter 3

Site Conditions and Micropile Material Specifications and Installation

3.1 Introduction

This chapter provides a description of the site conditions and soil investigation program as well as a description of the hollow bar micropiles components along with their specifications. In addition, the mechanical properties of the grout determined from cylinder tests are presented. It also explains the micropile installation procedure as well as the installation of the strain gauges employed to measure the strain along the micropile shaft.

3.2 Site Location and Description

The micropiles were installed at the Western Environmental Site, located 8 km north of London, Ontario. Nine initial boreholes (see Figure 3.1) were advanced to depths of 2.0 m to 4.6 m. Boreholes 1 and 2 showed about 230 mm of granular fill at the surface overlying a layer of firm to stiff clayey silt with seems of sand and gravel underlies the granular fill and extends to a depth of 2 m, the termination point of the boreholes. The very stiff to hard clayey silt till was encountered in boreholes 3 and 4 from 0.2m below ground surface extending to depths of at least 4.6 m and 3.4 m, respectively. Borehole 6 and 7 displayed the very stiff clayey silt/silty clay till to termination depth of 4.6 m. Under the top soil in boreholes 5, 8, and 9, a 0.6 m to 1.4 m a layer of silt to sandy silt was found overlying the very stiff to hard clayey silt/silty clay till, and a layer of very dense silt appeared within the till in borehole 5 from 2.5 m to 3.5 m. Borehole 5 was terminated within the till at a depth of 4.6 m while boreholes 8 and 9 were terminated at a depth of 4.0 m. The groundwater level was measured in borehole 5 at a depth of 4.0 m.
3.3 Site Investigation Program

The initial boreholes confirmed that the site is comprised mainly of cohesive soil. Two additional boreholes (denoted BH-I and BH-II) were drilled close to the micropiles test area and were located 8.4 m apart, as shown in Figure 3.2. This task was accomplished by Aardvark Drilling Inc. using a CME 55 truck mounted drill rig. Samples were extracted from each borehole using hollow stem augur followed by a standard penetration test (SPT) and split spoon sampling. The stiff clay encountered at the site made taking undisturbed Shelby tube samples difficult. A monitoring well was installed in order to measure the groundwater level. Laboratory testing was performed on samples collected during the field work as described in subsection 3.3.2.
3.3.1 Fieldwork

The hollow stem auger used to advance Boreholes I and II had an internal diameter of 203.2 mm. After the desired depth was reached, the SPT was performed employing a 50.8 mm split spoon sampler and a 63.5 kg hammer connected via steel rods. The hammer was allowed to fall freely from a height of 760 mm, and the process was repeated until the sample penetrated a distance of 450 mm. The number of blows was recorded for each 150 mm interval. Samples were collected using the split spoon sampler.
After completion of the borehole, a monitoring well was installed to monitor changes in the groundwater table.

The borehole logs for BH-I and BH-II are presented in Figures 3.3 and 3.4. They show a top soil layer 200-300 mm thick underlain by a layer of firm to stiff clay with some sand and gravel that extends to a depth of 5 m. The colour of the soil changed from brown to light grey at a depth 3.8 m as the drilling approached the water table. In both boreholes, a soft to firm dark grey clay layer appeared between 5 m and 6.5 m. This layer is underlain by very stiff dark grey lean clay with seams of sand that extends to a depth of 9m, as indicated in Tables 3.1 and 3.2. In borehole II, the groundwater level was measured at a depth of 6.41 m.

3.3.2 Laboratory work

Laboratory testing was carried out on the split spoon samples from boreholes I and II. For each borehole, 11 samples were retrieved from different depths for the purpose of laboratory testing. The tests performed included: determination of natural water content (ASTM D2216), dry unit weight (ASTM D7263), Atterberg limits (ASTM D4138), and grain size distribution. Figure 3.5 summarizes the results of the Atterberg limit tests. Sieve analysis and laser diffraction methods were used to determine the particle size distribution for 22 samples representing the soil profile. The sieve analysis was conducted on coarse-grained soil while the laser diffraction was conducted on fine-grained soil. The particle size distributions from the laser diffraction method at different depths are shown in Figure 3.6.

Figure 3.5 shows that the soil has plasticity index between 3 % and 15 %, i.e., the soil is classified as slightly plastic. This indicates that the soil has greater silt content than does clay as confirmed by the grain size distributions presented in Figure 3.6. The percentage of clay content, however, increased as the depth increased in both boreholes. According to the plasticity chart shown in Figure 3.5, the tested soil is defined as lean clay (Unified Soil Classification System). The measured bulk unit weight (γ) values are listed in Tables 3.1 and 3.2.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Thick. (m)</th>
<th>Description</th>
<th>Legend</th>
<th>SPT Counts</th>
<th>N Value</th>
<th>w %</th>
<th>L.I. %</th>
<th>P.L. %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.25</td>
<td>Hard, gray CLAY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>0.25</td>
<td>Very Stiff to hard, dark gray, lean clay with seams fine sand.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>0.50</td>
<td>Firm, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>0.50</td>
<td>Very Stiff, dark gray, lean clay with seams fine sand.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>4.00</td>
<td>Firm to Stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td></td>
<td>Soft, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td></td>
<td>Firm to Stiff, brown becoming gray at a depth of 3.8 m, lean CLAY with coarse sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>0.00</td>
<td>200 mm to 300 mm Top Soil</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Project: Borholes Samples
Owner: Western University
Location: Western Environmental Site
Rig Type: CME 55 Mount Drill
Casing Depth (m): 9.00 m
End Date: 17/07/2012
Location: Western Environmental Site
Casing Depth: 9.00 m
Rig Type: CME 55 Mount Drill
End Date: 17/07/2012

Figure 3.3 Borehole I soil log
### Figure 3.4 Borehole II soil log

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Thickness (m)</th>
<th>Description</th>
<th>Legend</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>0.50</td>
<td>200 mm to 300 mm Top Soil</td>
<td>1 6 10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Firm to stiff, brown becoming gray at a depth of 3.8 m, lean CLAY/SILT with coarse sand and some gravel.</td>
<td>3 4 5</td>
</tr>
<tr>
<td>2.00</td>
<td>0.50</td>
<td>Firm to stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY/SILT with fine sand and some gravel.</td>
<td>7 8 10</td>
</tr>
<tr>
<td>3.00</td>
<td></td>
<td>Hard, dark gray, lean CLAY/SILT with seams fine sand.</td>
<td>2 7 5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Medium Dense to Dense, SAND, trace to some silty clay.</td>
<td>3 7 10</td>
</tr>
<tr>
<td>4.00</td>
<td></td>
<td>Dense, SAND, trace to some silty clay.</td>
<td>4 6 7</td>
</tr>
<tr>
<td>5.00</td>
<td>0.50</td>
<td>Dense, SAND, trace to some silty clay.</td>
<td>3 3 6</td>
</tr>
<tr>
<td>6.00</td>
<td>0.25</td>
<td>Firm to stiff, brown becoming gray at a depth of 3.8 m, lean CLAY/SILT with coarse sand and some gravel.</td>
<td>1 2 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hard, dark gray, lean CLAY/SILT with seams fine sand.</td>
<td>1 2 2</td>
</tr>
<tr>
<td>7.00</td>
<td>0.50</td>
<td>Medium Dense to Dense, SAND, trace to some silty clay.</td>
<td>7 10 17</td>
</tr>
<tr>
<td>8.00</td>
<td>0.25</td>
<td>Hard, gray CLAY</td>
<td>15 27 23</td>
</tr>
<tr>
<td>9.00</td>
<td></td>
<td>END OF BORING</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SPT Counts</th>
<th>N Value</th>
<th>w</th>
<th>LL</th>
<th>PL</th>
</tr>
</thead>
<tbody>
<tr>
<td>15 cm</td>
<td>15 cm</td>
<td>10 cm</td>
<td>0 cm</td>
<td>10 cm</td>
</tr>
<tr>
<td>0</td>
<td>40</td>
<td>50</td>
<td>30</td>
<td>20</td>
</tr>
</tbody>
</table>

### Details

- **Project:** Borholes Samples
- **B.H. No.:** II
- **Sheet:** 1/1
- **Start Date:** 17/07/2012
- **End Date:** 17/07/2012
- **Location:** Western Environmental Site
- **Casing Depth:** 9.00 m
- **Rig Type:** CME 55 Mount Drill
- **G.W. Depth (m):** 6.41 m
- **Weather:** Sunny
- **B.H. Elev.:** ---

**Description:**
- 200 mm to 300 mm Top Soil: Firm to Stiff, brown becoming gray at a depth of 3.8 m, lean CLAY/SILT with coarse sand and some gravel.
- 2.00 m: Firm to stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY/SILT with fine sand and some gravel.
- 3.00 m: Hard, dark gray, lean CLAY/SILT with seams fine sand.
- 4.00 m: Firm to stiff, brown becoming gray at a depth of 3.8 m, lean CLAY/SILT with coarse sand and some gravel.
- 5.00 m: Medium Dense to Dense, SAND, trace to some silty clay.
- 6.00 m: Hard, dark gray, lean CLAY/SILT with seams fine sand.
- 7.00 m: Medium Dense to Dense, SAND, trace to some silty clay.
- 8.00 m: Dense, SAND, trace to some silty clay.
- 9.00 m: Hard, gray CLAY
Figure 3.5 Chart of Atterberg limits relative to plasticity

Figure 3.6 Grain size distribution

a) At a depth of 1.5 m
b) At a depth of 2.5 m
c) At a depth of 5.5 m
d) At a depth of 6.5 m
3.3.3 Shear strength

The soil strength during pile load testing is given by its undrained shear strength \( C_u \) because the soil consists primarily of cohesive material, and the piles were loaded following the quick maintained load test procedure. This precluded sufficient time for the induced pore water pressure to dissipate and for consolidation settlement to occur during the loading. Values for the undrained shear strength were obtained through correlations with the measured SPT values.

Many correlations were suggested to evaluate the undrained shear strength from corrected and uncorrected SPT N-values. Terzaghi and Peck (1967) proposed a correlation between the \( C_u \) and \( N_{60} \) for fine-grained soil, i.e.:

\[
C_u = 6.5 \, N_{60} \, (\text{kPa})
\]  
(3.1)

For insensitive clay, Stroud (1974) suggested the following correlation:

\[
C_u = (3.5 - 6.5) \, N_{60} \, (\text{kPa})
\]  
(3.2)

And for soil with a plasticity index \( (\text{PI}) < 20 \), he suggested:

\[
C_u = (6 - 7) \, N \, (\text{kPa})
\]  
(3.3)

where

\( N_{60} = \) the standard penetration number, corrected for field conditions to an average energy ratio of 60 %

\[
N_{60} = C_h C_r C_s C_d N
\]  
(3.4)

\( N = \) the measured SPT number

\( C_h = \) the rod energy ratio normalized to 60 %

\( C_r = \) the correction for the rod length
Cs = the sampler correlation

Cd = the correction for the borehole diameter

Seed et al. (1984) and Skempton (1986) recommended a generalized energy ratio \( (C_h) \). Skempton (1986) suggested a correction for the rod length \( (C_r) \), the sampler \( (C_s) \), and the borehole diameter \( (C_d) \). The \( C_r \) value was taken to be 0.75 for a rod length between 3 m and 4 m, 0.85 for a rod length between 4 m and 6 m, and 0.95 for a rod length between 6 m and 10 m. The \( C_s \) value was selected as 1.2 for a sampler without liners, and the \( C_d \) value used was 1.15 for a borehole diameter of 200 mm. Figure 3.7 shows the undrained shear strength profile.

Peck et al. (1974) and Terzaghi et al. (1996) provided an empirical correlation between \( N_{1,60} \), and the effective friction angle for both, fine and coarse grained sands in a graphic form. Anderson et al. (2003) approximated the relation into a logarithmic equation to be

\[
\phi' = 53.881 - 27.6034e^{-0.0147N_{1,60}}
\]

where

\[
N_{1,60} = N_{60} C_N
\]

\( C_N \) = the overburden correction factor and calculated from,

\[
C_N = 0.77 \log \frac{1920}{\sigma'_o}
\]

where

\( \sigma'_o \) = the effective vertical stress in kN/m\(^2\)
Figure 3.7 $C_u$ versus depth
### Table 3.1 Summary of soil properties for BH I

<table>
<thead>
<tr>
<th>Layer</th>
<th>H (m)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$C_u$ (kPa)</th>
<th>$\phi^o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top soil</td>
<td>0.2-0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Firm to Stiff Lean Clay (6% gravel, 16% sand, 78% silt/clay)</td>
<td>4.0</td>
<td>12.28</td>
<td>24.92/</td>
<td>15.68/</td>
<td>22.3</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>Firm to Stiff Lean Clay (11% gravel, 32% sand, 60% silt/clay)</td>
<td>1.0</td>
<td>13.00</td>
<td>17.53/</td>
<td>12.76/</td>
<td>22.3</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>Soft Lean Clay (0% gravel, 2% sand, and 98% silt and clay)</td>
<td>1.0</td>
<td>23.78</td>
<td>37.46</td>
<td>21.09</td>
<td>22.3</td>
<td>65</td>
<td>-</td>
</tr>
<tr>
<td>Firm Lean Clay (14% gravel, 17% sand, and 69% silt and clay)</td>
<td>0.5</td>
<td>14.79</td>
<td>29.65</td>
<td>15.91</td>
<td>19.9</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>Stiff to Hard Lean Clay (1% gravel, 9% sand, 90% silt/clay)</td>
<td>2.5</td>
<td>11.9/</td>
<td>18.62/</td>
<td>12.37/</td>
<td>19.9</td>
<td>100/</td>
<td>-</td>
</tr>
</tbody>
</table>

*According to Terzaghi and Peck (1967) and Stroud (1974)*

### Table 3.2 Summary of soil properties for BH II

<table>
<thead>
<tr>
<th>Layer</th>
<th>H (m)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$C_u$ (kPa)</th>
<th>$\phi^o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top soil</td>
<td>0.2-0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Firm to Stiff Lean Clay (8% gravel, 18% sand, 74% silt/clay)</td>
<td>4.0</td>
<td>11.70</td>
<td>23.70/</td>
<td>16.00/</td>
<td>22.3</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>Firm to Stiff Lean Clay (6% gravel, 24% sand, 70% silt/clay)</td>
<td>2.75</td>
<td>10.50</td>
<td>20.40/</td>
<td>13.30/</td>
<td>22.3</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>Medium Dense/Dense Sand (1% gravel, 53% sand, 46% silt clay)</td>
<td>0.5</td>
<td>17.30</td>
<td>24.30</td>
<td>12.60</td>
<td>22.3</td>
<td>-</td>
<td>35</td>
</tr>
<tr>
<td>Hard Lean Clay (0% gravel, 3% sand, and 97% silt and clay)</td>
<td>1.0</td>
<td>18.10</td>
<td>20.20</td>
<td>12.30</td>
<td>19.8</td>
<td>325</td>
<td>-</td>
</tr>
<tr>
<td>Medium Dense to Dense Sand (1% gravel, 72% sand, and 27% silt and clay)</td>
<td>0.5</td>
<td>11.70</td>
<td>21.20</td>
<td>14.40</td>
<td>19.8</td>
<td>-</td>
<td>39</td>
</tr>
<tr>
<td>Hard Clay</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*According to Terzaghi and Peck (1967) and Stroud (1974)*
3.4 Hollow Bar Micropile Parts

Three types of hollow bar micropiles are employed in North America: the CTS/TITAN IBO manufactured by Ischebeck Titan; the DYWI® drill hollow bars manufactured by DYWIDAG-Systems International; and the Geo-Drill Injection Anchor manufactured by Williams Form Engineering Corp. The type used in this study was the Williams Geo-Drill Injection Anchor.

Hollow bar micropile systems are generally comprised of six parts: threaded bars, couplings, drill bits, hex nuts, centralizers, and bearing plates. The threaded bar provided by Williams is high-strength, impact-resistant, heavy wall steel tubing that conforms to ASTM A519 or A513 and is threaded continuously over its entire length with a heavy-duty left-hand thread/deformation pattern. The properties of the Geo-Drill bar used in this study are listed in Table 3.3. Figure 3.8 shows the threaded bars used in this study. The bars have 76 mm OD and 48 mm ID.

Table 3.3 Geo-Drill Bar Specifications (Williams Form Engineering Corp, 2011)

<table>
<thead>
<tr>
<th>Part Number</th>
<th>Bar Diameter (mm)</th>
<th>Average Inner Diameter (mm)</th>
<th>Minimum Net Area (mm²)</th>
<th>Nominal Weight (kg/m)</th>
<th>Minimum Yield Strength (kN)</th>
<th>Minimum Ultimate Strength (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B7X 1-76</td>
<td>76</td>
<td>48</td>
<td>2503</td>
<td>20.5</td>
<td>1466</td>
<td>1811</td>
</tr>
</tbody>
</table>

Figure 3.8 B7X 1-76 threaded bars (Geo-Drill bar)
The couplings used have a tapered centre stop (Figure 3.9) to prevent grout leakage during grouting, and to ensure full positive thread connection in both injection bar ends. The couplings are machined from grade C1045 high-strength steel according to ASTM A29. Table 3.4 lists the specifications of the coupling used in this study. The hex nuts are machined from high-strength steel and comply with ASTM A108. Figure 3.10 shows the hex nuts used, and their specifications are listed in Table 3.5.

![Coupler Image](image)

**Figure 3.9** B7X2-76 coupler

**Table 3.4 Coupler Specifications (Williams Form Engineering Corp, 2011)**

<table>
<thead>
<tr>
<th>Part Number</th>
<th>Nominal Bar Diameter</th>
<th>Outside Diameter</th>
<th>Overall Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>B7X 2-76</td>
<td>76</td>
<td>98.4</td>
<td>251</td>
</tr>
</tbody>
</table>
Table 3.5 Hex nut Specifications (Williams Form Engineering Corp, 2011).

<table>
<thead>
<tr>
<th>Part Number</th>
<th>Nominal Bar Diameter</th>
<th>Across Flats</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>B7X3-76</td>
<td>76</td>
<td>102</td>
<td>108</td>
</tr>
</tbody>
</table>

To centre the bar in the hole, steel centralizers are attached in front of the coupling during the drilling operation. Bearing plates are used to connect the pile cap to the hollow bar micropile to ensure that the structural load is correctly transferred to the micropiles.

The drill bit is sacrificial, allowing the hollow bars to serve as both the drill string and the grouting conduit so that the installation is performed in a single operation. The sacrificial drill bit is threaded onto the end of the Geo-Drill bar and has two to four nozzles to allow the drilling fluid to pass outside the system, which facilitates the drilling and grouting processes. Williams developed a drill bit 228 mm (9 in) in diameter specifically for this study to achieve $d_{bb}/d_{bar}$ ratio of 3, in order to increase the capacity of the micropiles. The newly developed drill bit has carbide buttons to break up the rock formation. For the purposes of this study, the performance of the hollow bar micropile in a firm to stiff clay using the new 228 mm (9 in) drill bit was compared with that of the 178 mm (7 in) drill bit used by Abd Elaziz and El Naggar (2010, 2011, and 2012) and Abd Elaziz (2012) in a previous study. Figure 3.11 shows the two types of drill bit used in the study.
Figure 3.11 Carbide button cross cut drill bit (d = 228 mm (9 in)), and double cross cut bit (d = 178 mm (7 in))
3.5 Hollow Bar Micropile Installation

Eight hollow bar micropiles were installed: six with a 228 mm (9 in) drill bit and two with a 178 mm (7 in) drill bit. The micropiles were placed in two rows 3 m apart; each row had four micropiles placed 4 m apart (Figure 3.2). All micropiles had a bond length of 5.75 m and were tested after 6 weeks of curing time.

The installation involved the following steps:

- After the hollow bar was attached to the drill bit at one end and to the adapter at the other end, as shown in Figure 3.12a, pressurized water was fed into the system to ensure that the system contained no leaks (Figure 3.12b).

- The hollow bar was set into position for installation (Figure 3.12a).

- The rotary percussive drilling started with air and water flushing (Figure 3.12c). The cohesive nature of soil at the site facilitated the use of this technique (i.e. grout flushing was not necessary). The drilling rotation speed was about 90 rpm.

- After the hollow bar was installed to 3 m, drilling was stopped in order to add the second section using a B7X2-76 coupler (Figure 3.12d).

- At this point, the hollow bars were raised from the hole to visually ensure that the flush is returning from the mouth of the borehole, and the drilling then began again.

- After the desired depth (5.75 m) was reached, grout with water-to-cement ratio = 0.45 was introduced through the hollow bar system under approximately 1.1 MPa (157 psi) of pressure (Figure 3.13). The grouting pressure enabled all the drilling water and debris soil to be flushed from the bar. In some cases, water was used to clean debris from the hollow bar before flushing with the competent grout. The properties of the grout are discussed in Section 3.6.
Figure 3.14 shows the micropile arrangement within the test area. It is worth mentioning that during the drilling MP3 and MP4, a shallow layer of new fill was found, extending approximately 2 m to 3 m. In addition, a chunk of concrete was found at the location of MP3. The ground surface at MP2, MP4, MP6, MP8 was lower than the ground surface at BH I, and BH II by 300 mm to 400 mm.

The average quantity of cement bags per hole was 7-8 bags (280 kg to 320 kg) for the 178 mm drill bits and 9-10 bags (360 kg to 400 kg) for the 228 mm drill bits. These quantities correspond to approximately 0.215 m$^3$ and 0.276 m$^3$ of grout for drill bit diameters of 178 mm and 228 mm, respectively. Based on the installation records and grout quantities used, the size of the holes appeared to be $1.1 \, d_{\text{bit}}$ for the 228 mm drill bit and $1.2 \, d_{\text{bit}}$ for the 178 mm drill bit, which agrees with Maclean’s (2010) observations in which the diameters of the micropiles enlarged. The difference in the enlargement factor is attributed to the differences in the grouting pressure and the size of the nozzles. The nozzles in the 178 mm drill bit were smaller than those in the 228 mm drill bit.
a) The hollow bar set into position for installation

b) Feeding pressurized water into the system

c) Drilling with an air/water technique

d) Adding a second section

Figure 3.12 Steps of micropile installation
Figure 3.13 Flushing all drilling water and debris with the competent grout
Figure 3.14 Flushing all drilling water and debris with the competent grout
3.6 Grout Properties

The load transfer mechanism and the quality of the micropiles produced are governed by the quality of both the hollow steel bars and the grout. The quality of the grout should thus be carefully monitored. In hollow bar micropiles, neat cement grout is used as the grout body. Since water and cement are the only components of neat cement, an assurance program has been developed to ensure the quality of neat cement.

The specific gravity and compressive strength of the grout are examined as part of a quality assurance program for the production of micropiles. A mud balance test is used to measure the specific gravity of the neat grout, which, should be between 1.8 and 1.9 for micropile applications. Evaluating the compressive strength of the neat cement takes at least a week; however, the water-to-cement ratio can be used as a means of predicting the grout compressive strength. A correlation between the water-to-cement ratio, the specific gravity, and the compressive strength was defined by the Post Tensioning Institute (PTI) (2004). The specific gravity measurement is therefore suitable for monitoring the quality of the fresh grout and provides a fast indication of whether the grout is adequate.

In this study, neat grout with used employing Type 10 Portland cement with water-to-cement ratio $= 0.45$. A colloidal mixer was employed so that the fine grout particles would be dispersed into the small voids of the surrounding soil. The mud test results indicated a specific gravity of about 1.85 to 2.0.

To evaluate the strength of the grout, 12 samples were tested for both compression and tension. The mixtures were placed in cylinder moulds 75 mm in diameter and 150 mm high. After one day, all specimens were demoulded and placed inside a curing room with a constant temperature of 23 ± 2°C and a relative humidity of 100 %. Six samples were tested for compression according to ASTM C39: three samples after 7 days, and three samples after 28 days. Six samples were tested for tensile strength (split tension test) according to ASTM C496: three samples after 7 days, and three samples after 28 days. In addition, the modulus of elasticity of four samples after 28 days was also determined.
according to ASTM C469, whereby the specimens are loaded to approximately 40 % of the compressive strength. The test setup used is shown in Figure 3.15.

Flexural tensile strength was determined according to ASTM C78: six grout samples were prepared and tested after 14 and 28 days. The grout was placed in 100 mm by 100 mm by 350 mm steel moulds. All specimens were demoulded after one day and were placed inside a curing room with a constant temperature of 23 ± 2°C and a relative humidity of 100 %.

![Figure 3.15 Setup for testing the modulus of elasticity](image)

The compressive strength after 28 days was 41.4 MPa with an average specific gravity of 1.9, which is higher than the minimum requirement (27.6 MPa) indicated by Gómez et al. (2007). The compressive strength meets the limit specified by FHWA (2005). The average split tensile strength and the flexural tensile strength after 28 days were 4.2 MPa and 5.2 MPa, respectively. Figure 3.16 presents the variation of the flexural stress and the machine vertical movement. The average modulus of elasticity of the four samples was 15,052 MPa. Figure 3.17 shows the tests results of the modulus of elasticity and Table 3.6 provides a summary of the grout sample test results.
Table 3.6 Mechanical proprieties of grout

<table>
<thead>
<tr>
<th>Curing Time</th>
<th>Compressive Strength MPa</th>
<th>Split Tension Test MPa</th>
<th>Flexural Strength MPa</th>
<th>Modulus of Elasticity MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 days</td>
<td>25.4</td>
<td>2.7</td>
<td>*4.38</td>
<td>---</td>
</tr>
<tr>
<td>28 days</td>
<td>41.4</td>
<td>4.2</td>
<td>5.20</td>
<td>15,052</td>
</tr>
</tbody>
</table>

* 14 days rather than 7 days.

Figure 3.16 Flexural stress versus vertical movement
Figure 3.17 Stress versus strain curve for grout cylinders

3.7 Axial Capacity of Hollow Bar Micropiles

In micropiles, the structural load is carried primarily by the steel reinforcement. The concrete also increases both the axial and lateral structural load capacity.

Based on the nominal diameters of micropiles, grout and steel properties, the allowable structural capacity will be 1420 kN, and 1240 kN for micropiles with 228 mm drill bit and 215 for the 178 mm drill bit, respectively while the allowable tensile capacity will be 800 kN. The micropiles should not be loaded more than the obtained values for the structural capacity.

3.8 Embedded Strain Gauges (EM-5)

After the grout was placed into the hollow bars, strain gauges were embedded in order to monitor the strains during the load testing of the micropiles. This section describes the installation of the gauges and explains the interpretation methodology of the readings.
3.8.1 Installation

Model EM-5 vibrating wire strain gauges were used in this study because of their long-term reliability and high resolution. The gauge consists of two end flanges separated by a stainless steel tube with a tensioned high-strength steel wire clamped into the end flanges and running axially through the centre of the tube. Once the gauge is embedded in grout, the strains in the grout are conveyed to the gauge through the end flanges and measured by the readout unit as changes in the vibration period of the wire (Roctest Group, 2006).

Twenty-four vibrating wire strain gauges were used, three in each micropile. The locations of the strain gauges are shown in Figure 3.18a, which also illustrates the attachment of each strain gauge to a steel cage that are connected via 12.5 mm steel bars. The gauges were installed immediately after grouting when the grout was still liquid and were pushed into the grout-filled hollow bar. The wires from the gauges were fed through three holes at a distance of 150 mm from the top of the hollow bar 7 days after installing the micropiles.

A reading was taken for each gauge before installation to ensure it was close to 2500 linear units, which corresponds to approximately 1275±100 microsecond. At this setting, the measurement is about 1500 microstrains, in tension or compression. This step is to ensure the vibrating wire strain gauges are working properly.

3.8.2 Interpreting the readings

The measured strains are comprised of strains due to stresses and those due to other causes: variations in temperature; variations in humidity (moisture) that create hydric effects; the setting of the concrete itself, called autogenous volume changes in the concrete; and those caused by the presence of the gauge itself. Interpretation is clearer when readings are taken at the same time the loads are applied (Roctest Group, 2006).

The total strain readings from the EM-5 gauges include the strain from several sources in addition to the effective stress from the applied load:
\( \varepsilon = \varepsilon_e + \varepsilon_c + \varepsilon_h + \varepsilon_s \)  \hspace{1cm} (3.8)

where

\( \varepsilon = \) the total strain measurement, in \( \mu \)strains

\( \varepsilon_e = \) the total strain due to the applied effective stress, in \( \mu \)strains

\( \varepsilon_c = \) the creep strain, in \( \mu \)strains, which can be considered hidden in the \( \varepsilon_e \) value

\( \varepsilon_h = \) the strain due to the hydric and moisture effect, in \( \mu \)strains

\( \varepsilon_s = \) the strain caused by other factors such as local strain discontinuities, which can be considered hidden in the \( \varepsilon_e \) value

The final equation then becomes

\( \varepsilon = \varepsilon_e + \varepsilon_h \) \hspace{1cm} (3.9)

The two methods for interpreting the readings are explained below.

**Method 1 - Direct Correction from a No-Stress Gauge:**

In this method, a no-stress gauge can be used to monitor environmental strain conditions identical to those subjected to the applied stress. Since \( \varepsilon_h \) is equal in both gauges, the total strain read by the no-stress gauge can reasonably be subtracted directly from the total strain read by the EM-5 gauge subjected to the applied stress. The strain due to the applied stress is then equal to

\( \varepsilon_e = \varepsilon - \varepsilon_{nsg} \) \hspace{1cm} (3.10)

where

\( \varepsilon_{nsg} = \) the total strain in the no-stress gauges, in \( \mu \)strains
Method 2 - Interpreting the Readings with Theoretical Corrections:

In the second method, the use of a no-stress gauge can be omitted if all behaviour parameters are known. From Equation 3.8, \( \varepsilon_s \) and \( \varepsilon_c \) are considered to be hidden in \( \varepsilon_e \). The real strain \( \varepsilon_r \) is then the total strain plus the value of the thermal expansion of the wire if the EM-5 strain meter was not confined (Roctest Group, 2006):
\[ \varepsilon_r = \varepsilon + (\alpha_c - \mu \beta) \times (T_1 - T_0) \]  

(3.11)

And

\[ \varepsilon_e = \varepsilon_r - \varepsilon_c - \varepsilon_h \]  

(3.12)

where

\( \varepsilon \) = the total strain measurement, in \( \mu \)strains

\( \varepsilon_r \) = the real strain, in \( \mu \)strains

\( \alpha_c \) = the linear expansion factor of EM-5 gauge wire = 11.5 \( \mu \)m/m\(^\circ\)C

\( T_1 \) = the current temperature reading, in \( ^\circ\)C

\( T_o \) = the initial temperature reading, in \( ^\circ\)C

\( \mu \) = the freedom factor of the concrete structure in the surrounding material (0 \( \leq \mu \leq 1 \))

\( \beta \) = the concrete expansion factor, in \( \mu \)m/m\(^\circ\)C

\( \mu \beta \) is called the expansion factor and can be determined from laboratory tests or can be estimated from each EM-5 reading, using a linear regression of \( \varepsilon \) versus \( T^\circ \). The freedom factor \( \mu \) is equal to 1 since the surrounding material is confining the gauge with no movement allowed. \( \beta \) can be estimated from the slope of the graph when \( \mu = 1 \).

The effective strain can then be written as

\[ \varepsilon_e = (\varepsilon_1 - \varepsilon_o) + (\alpha_c - \mu \beta) \times (T_1 - T_o) - \varepsilon_h \]  

(3.13)

where

\( \varepsilon_o \) and \( \varepsilon_1 \) = the initial, and current readings

In this study, Method 1 was used for interpreting the readings.
3.9 References


Chapter 4
Axial Monotonic and Cyclic Compression Behaviour of Hollow Bar Micropiles

4.1 Introduction

Micropiles can be classified as small diameter (less than 300mm), bored, and grouted in place piles. These small elements can sustain axial (compression and tension) and/or lateral loads. First introduced by Dr. Fernando Lizzi (Cadden et al., 2004), micropiles have been in use since 1952 when they were employed in Italy for underpinning historic buildings damaged during World War II (Bruce, 1988). Micropile technology has evolved to include a variety of applications: underpinning for existing foundations, in situ reinforcement, and seismic retrofitting, and witnessed a significant expansion from use in low-capacity micropile networks to single high-capacity foundations.

The construction of micropiles involves three main steps: Drilling, placing reinforcement, and placing or pressurizing the grout. These activities affect the capacity of micropiles in different ways. The drilling method, for example, influences the degree of bonding between the grout and the ground, and both the reinforcement placement and grouting method have an impact on the bond strength. In practice, it is the grouting method that most affects the development of the grout/ground bond (Bruce et al., 1997). Based on the type of pressure applied during the grouting process, micropiles are classified as follows (FHWA, 2005): Type A, the grout is placed under gravity, using either sand-cement mortars or neat cement grout. Type B: Pressures, typically in the range of 0.5 MPa to 1 MPa, are applied in order to inject the neat cement grout into the drilled hole while the temporary drill casing is withdrawn. Type C involves a two-step process: neat cement grout is first placed in the hole under gravity pressure head only, as in type A, and prior to the hardening of this primary grout, a similar grout is then injected via a preplaced sleeved grout pipe at a pressure of at least 1MPa. Like type C, type D also entails a two-step process: first, neat cement grout is placed in the hole under gravity pressure head only, as in type A. However, sometimes pressure can be applied as in type B. After
several hours, once the primary grout has hardened, additional grout is injected via a sleeved grout pipe at a pressure of 2 MPa to 8 MPa. In some cases, a packer is used inside the sleeved pipe, enabling specific horizons to be treated.

The new generation of micropiles was first devised by Ernst Ischebeck in 1983, and named The Titan Injection Bore (IBO) micropile (CON-TECH SYSTEM, 2011; and Abd Elaziz 2012). Type E involves a threaded hollow bar connected to a drill bit advanced into the soil using air, water, or grout. The grout is then injected, typically at up to 1.38 MPa (200 psi) through the centre of the hollow bar, passing through the holes in the drill bit to the main hole while the system is rotated. Timothy and Bean (2012) denoted this type as Type E micropile (i.e. supplementary to the original four types A to D indicated by the FHWA). In some published literature, however, hollow bar micropiles are categorized as Type B.

Hollow bar micropiles are gaining popularity because they provide fast installation with a high degree of ground improvement. They can be placed in a one-step operation in which the hole is drilled, reinforced and grouted simultaneously.

Bruce and Yeung (1984) stated that the design of micropiles is limited by its small cross-section area. In addition, its geotechnical capacity is dictated by skin friction, as opposed to end bearing. They found that the pile requires a settlement of 10% to 20% in order to mobilize the bearing capacity, compared with only 0.5% to 1% to mobilize the maximum skin resistance. Juran et al. (1999) identified the skin friction of the micropiles as the primary contributor to the load transfer, so they concluded that a micropile is designed to transfer the load only through the shaft resistance. They found that a pile 5 m long and 200 mm in diameter requires 20 to 40 times less movement to mobilize the skin friction than to mobilize the end bearing. FHWA (2005) stated that the end bearing resistance of a micropile can be neglected due to its small toe area and that the shaft resistance carries the external applied load. The contribution of the end bearing to the load capacity can be taken into account only in the case of piles founded on rock. Cadden et al. (2004) indicated that micropiles carry the load through the shaft friction resistance along the grout/ground interface only due to the small cross-sectional area of the pile.
compared to the shaft surface area. They mentioned that a contribution could be made by the end bearing in the case of short piles installed in hard rock.

Han and Ye (2006) reported a full-scale load test of four piles installed in soft clay. Two piles were subjected to compression loading and two piles to tension loading. Rebar strain gauges were inserted into the micropiles to monitor strain during the load tests. They evaluated load-displacement responses, elastic moduli, axial forces, toe resistance, and skin friction. The results showed that the skin friction values under compressive loading were 19% to 59% higher than the values suggested for bored concrete piles. In addition, the ultimate compressive skin friction for the micropiles was 0.9 to 1.2 times the undrained shear strength (i.e. adhesion ≥ cohesion). Holman and Tuozzolo (2007) examined the load test data of three micropiles and reported decrease in the pile elastic modulus with the increase in strain levels. They also reported that the mobilized unit bond stress is inconsistent with the non-uniform behaviour of the grout. Two micropiles experienced plunging, the maximum toe resistance developed at displacements of 8% to 10% of the diameter of the pile tip. Gómez et al. (2007a) and Gómez et al. (2007b) investigated the performance of 260 hollow bar micropiles: 180 installed in submerged sand and 80 installed in stiff, silty clay. They found that the grout-ground bond strength was greater than that calculated for pressure-grouted (Type B) micropiles in granular soils.

Telford et al. (2009) conducted verification testing on threaded hollow stem Titan 73/45 (73 mm O.D./45 mm I.D.) bars installed to a depth of 9.8 m in cohesionless soil using a 115 mm cross drill bit. The piles were loaded close to capacity in both compression and tension with only a small amount of deformation. Considering that the pile diameter was enlarged by 1.5 times, the grout-to-soil bond strength values were close to those proposed by FHWA (2005) for Type B micropiles. Bennett and Hothem (2010) conducted load tests on four pairs of micropiles installed in soft clay/sandy clay/very loose to loose clayey sands extending from ground surface to approximately 4.5 m to 6 m. Two different drill bits were used for each pair: a 150 mm clay bit and a 115 mm cross bit. The shortest pile carried 480 kN and 460 kN for the 150 mm clay bit and the 115 mm cross
bit, respectively. The 150 mm clay bit performed marginally better than the 115 mm cross bit. Abd Elaziz and El Naggar (2010, 2011, and 2012) examined five hollow bar micropiles installed in overconsolidated clayey silt/silty clay till overlying compact to dense sand. All piles consisted of 76 mm OD and 48 mm ID hollow bars connected to 178 mm carbide drill bits. The results revealed that considering hollow bar micropiles as Type B underestimates the grout-ground bond strength.

Cavey et al. (2000) observed 60% reduction in ultimate capacity for pressure-grouted micropiles installed in loose to medium dense sand after being subjected to cyclic loading. Gómez et al. (2003) did not detect any physical debonding of the grout-ground interface although a post-peak reduction in bond strength was observed under cyclic loading for micropiles installed in rock. Abd Elaziz and El Naggar (2012) observed no change to a slight increase in the micropile head stiffness after 15 load cycles, and no debonding occurred between the grout and the ground during the tests.

This review indicates that even though there are many studies that were focused on the behaviour of micropiles, only very few studies were focussed the performance of hollow bar micropiles. Several factors should be better evaluated in order to enhance the design practices for hollow bar micropiles. For example, the effects of type of grout, grouting pressure, and the \( d_{\text{bi}}/d_{\text{bar}} \) ratio on the micropile capacity should be better evaluated. The data available with respect to hollow bar micropiles are limited to primarily cohesionless soils or medium to high plasticity clay. For this reason, the research presented in this thesis involved a field study of the performance of hollow bar micropiles in firm to stiff clay. The study investigated the axial behaviour of hollow bar micropiles constructed using two different sizes of drill bit. The results of the full scale load tests are presented and discussed with respect to load-displacement curves, skin friction, and toe resistance.

4.2 Site Conditions

The micropiles were installed at the Western University Environmental Site, located 8 km north of London, Ontario. Two boreholes (denoted BH-I and BH-II) were drilled within the test area and were 8.4 m apart as shown in Figure 4.1. Samples were extracted
from each borehole by means of hollow stem auguring followed by a standard penetration test (SPT) and split spoon sampling. A monitoring well was installed in order to measure the groundwater level. Laboratory testing was performed on the samples collected.

The boreholes BH-I and BH-II shown in Figures 4.2 and 4.3 indicate a layer of top soil 200-300 mm thick overlying a layer of firm to stiff clay with some sand and gravel that extends to a depth of 5 m. The colour of the soil changed from brown to light grey at a depth of 3.8 m as the drilling approached the water table. In both boreholes, a soft to firm dark grey clay layer appeared between 5 m and 6.5 m. This layer is underlain by very stiff dark grey lean clay with seams of sand that extend to a depth of 9 m. In borehole II, the groundwater level was measured at a depth of 6.41 m. Figures 4.2 and 4.3 also summarize the results of both the natural water content evaluation and the Atterberg limit tests conducted on soil specimens retrieved as part of the field work. It is noted from the figures that the soil PI varied between 3 % and 15 %, based on which the soil is classified as slightly plastic.

According to the plasticity chart shown in Figure 4.4, the tested soil is classified as lean clay based on the Unified Soil Classification System (most samples fall above the A-Line and under the U-Line with liquid limit values less than 50).
a) Building distribution

b) Borehole and pile distribution in the study area

Figure 4.1 Locations of borehole tests conducted by Aardvark Drilling Inc. in 2012
### Figure 4.2 Borehole I soil log

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Thickness (m)</th>
<th>Description</th>
<th>Legend</th>
<th>SPT Counts</th>
<th>N Value</th>
<th>w (%)</th>
<th>LI (%)</th>
<th>PL (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.25</td>
<td>Hard, gray CLAY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>0.25</td>
<td>Very Stiff to hard, dark gray, lean clay with seams fine sand.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.50</td>
<td>Firm, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.00</td>
<td>0.75</td>
<td>Soft, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.80</td>
<td>Very Stiff, dark gray, lean clay with seams fine sand.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50</td>
<td>0.50</td>
<td>Firm to Stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.00</td>
<td>1.00</td>
<td>Soft, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00</td>
<td>1.00</td>
<td>Firm to Stiff, brown becoming gray at a depth of 3.8 m, lean CLAY with coarse sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>4.00</td>
<td>Firm to Stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY with fine sand and some gravel.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td></td>
<td>Description</td>
<td>Legend</td>
<td>SPT Counts</td>
<td>N Value</td>
<td>w (%)</td>
<td>LI (%)</td>
<td>PL (%)</td>
</tr>
<tr>
<td>4.00</td>
<td></td>
<td>200 mm to 300 mm Top Soil</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
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<td></td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**Project:** Borholes Samples  
**Owner:** Western University  
**Location:** Western Environmental Site  
**Rig Type:** CME 55 Mount Drill  
**Drilling Method:** Rotary Drilling  
**Casing Depth:** 9.00 m  
**G.W. Depth (m):** 6.41 m  
**Weather:** Sunny  
**B.H. Elev:** ---
### Figure 4.3 Borehole II soil log

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Thick. (m)</th>
<th>Description</th>
<th>Legend</th>
<th>SPT Counts</th>
<th>N Value</th>
<th>w</th>
<th>LL</th>
<th>PL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td></td>
<td>Medium Dense to Dense, SAND, trace to some silty clay.</td>
<td></td>
<td>0.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td></td>
<td>200 mm to 300 mm Top Soil, Firm to Stiff, brown becoming gray at a depth of</td>
<td></td>
<td>1</td>
<td>6</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.8 m, lean CLAY/SILT with coarse sand and some gravel.</td>
<td></td>
<td>3</td>
<td>4</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>4.00</td>
<td>Firm to stiff, light gray becoming dark gray at a depth of 6.5m, lean CLAY/SILT</td>
<td></td>
<td>2</td>
<td>7</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>with fine sand and some gravel.</td>
<td></td>
<td>3</td>
<td>7</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00</td>
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<td></td>
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<tr>
<td>5.00</td>
<td></td>
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<tr>
<td>6.00</td>
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<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.00</td>
<td>0.50</td>
<td>Hard, dark gray, lean CLAY/SILT with seams fine sand.</td>
<td></td>
<td>7</td>
<td>10</td>
<td>17</td>
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</tr>
<tr>
<td>8.00</td>
<td>1.00</td>
<td>Hard, dark gray, lean CLAY/SILT with seams fine sand.</td>
<td></td>
<td>15</td>
<td>27</td>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.00</td>
<td>0.25</td>
<td>Dense, SAND, trace to some silty clay.</td>
<td></td>
<td>15</td>
<td>27</td>
<td>22</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: N Value: Number of SPT counts, w: Water content, LL: Liquid limit, PL: Plastic limit.
The soil strength is represented herein by its undrained shear strength ($C_u$) because the soil consisted primarily of cohesive material, and the micropiles were loaded rapidly following the quick maintained load test procedure (ASTM D1143, 2007). This precluded sufficient time for the induced pore water pressure to dissipate and for consolidation settlement to occur during the loading. Values for the undrained shear strength were obtained from the measured SPT values along with SPT-undrained shear strength relationships. Terzaghi and Peck (1967) correlated $C_u$ and $N_{60}$ for fine-grained soil as follows:

$$C_u = 6.5 \, N_{60} \, (\text{kPa})$$  \hspace{1cm} (4.1)

For insensitive clay, Stroud (1974) suggested the following correlation:

$$C_u = (3.5 - 6.5) \, N_{60} \, (\text{kPa})$$  \hspace{1cm} (4.2)

For soil with a plasticity index (PI) of less than 20, the correlation is

$$C_u = (6 - 7) \, N \, (\text{kPa})$$  \hspace{1cm} (4.3)
where \( N_{60} \) is the standard penetration number, corrected for field conditions to an average energy ratio of 60%:

\[
N_{60} = C_h C_r C_s C_d N
\]  

(4.4)

where

\( N \) = the measured SPT number

\( C_h \) = the rod energy ratio normalized to 60% 

\( C_r \) = the correction for the rod length 

\( C_s \) = the sampler correlation 

\( C_d \) = the correction for the borehole diameter

Skempton (1986) suggested a correction for the rod length (\( C_r \)), the sampler (\( C_s \)), and the borehole diameter (\( C_d \)). The \( C_r \) value was taken to be 0.75 for a rod length between 3 m and 4 m, 0.85 for a rod length between 4 m and 6 m, and 0.95 for a rod length between 6 m and 10 m. The \( C_s \) value was selected as 1.2 for a sampler without liners, and the \( C_d \) value used was 1.15 for a borehole diameter of 200 mm. Figure 4.5 shows the variation of Cu with depth for BH-I and BH-II.
Eight hollow bar micropiles were installed: six with a 228 mm drill bit diameter (MP1 to MP6) and two with a 178 mm drill bit diameter (MP7 and MP8). The micropiles were placed in two rows with 4 micropiles, spaced at 4m centre-to-centre, in each row. The distance between the rows was 3m. The drill bits used are shown in Figure 4.6. All micropiles had a bond length of 5.75 meters and were tested after 6 weeks of curing time.

The installation proceeded as follows. Rotary percussive drilling with air and water flushing was executed. After the hollow bar (Geo-Drill BX 76/48) was installed to 3 m, drilling was stopped in order to add the second section using a B7X2-76 coupler. After the desired depth (5.75 m) was reached, grout with water-to-cement ratio = 0.45 was introduced through the hollow bar system under pressure of 1.1 MPa (157 psi). The pressurized grout flushed all the drilling water and debris soil from the bar and the hole. A shallow layer of new fill was encountered during the drilling of MP3 and MP4 extending approximately 2 m to 3 m. The ground surface at MP2, MP4, MP6, MP8 was lower than the ground surface at BH I, and BH II by 300 mm to 400 mm.
The average quantity of cement bags per hole was 7-8 bags (280 kg to 320 kg) for the 178 mm drill bits and 9-10 bags (360 kg to 400 kg) for the 228 mm drill bits. These quantities correspond to approximately 0.215 m$^3$ and 0.276 m$^3$ of grout for drill bit diameters of 178 mm and 228 mm, respectively. Based on the installation records and grout quantities used, the size of the holes appeared to be 1.1 $d_{bit}$ for the 228 mm drill bit and 1.2 $d_{bit}$ for the 178 mm drill bit, which agrees with Maclean’s (2010) observations in which the diameters of the micropiles enlarged. The difference in the enlargement factor is attributed to the differences in the grouting pressure and the size of the nozzles. The nozzles in the 178 mm drill bit were smaller than those in the 228 mm drill bit.

The specific gravity and compressive strength of the grout are examined as part of a quality assurance program for micropiles production. A mud balance test is used to measure the specific gravity of the neat grout, which should be between 1.8 and 1.9 for micropile applications (Gómez et al. 2007). The water-to-cement ratio can be used as a means of predicting the grout compressive strength. The Post Tensioning Institute (PTI) (2004) proposed a correlation between water-to-cement ratio, specific gravity, and the compressive strength. The specific gravity measurement is therefore suitable for monitoring the quality of the fresh grout.
In this study, Type 10 Portland cement was mixed at 0.45 water-to-cement ratio using a colloidal mixer. The mud test results indicated a specific gravity of about 1.85 to 2.0. The average compressive strength after 28 days was determined from test cylinders and was found to be 41.4 MPa, which is higher than the minimum requirement (27.6 MPa) meets the limit specified by FHWA (2005). The average split tensile strength and the flexural tensile strength after 28 days were 4.2 MPa and 5.2 MPa, respectively. The average modulus of elasticity of four cylindrical samples tested was 15.05 GPa. These results are in agreement with the U.S. Army Corps of Engineers (1984) stipulations that flexural strength is 10-15% of the compressive strength, and the elastic modulus of grout is approximately half of the elastic modulus of concrete at the same strength.

The above values were obtained from testing cylindrical and beam samples that were placed for 28 days inside a curing room with a constant temperature of 23 ± 2°C and a relative humidity of 100%.

### 4.4 Test setup, Instrumentation, and Test Method

Eight monotonic compression tests were conducted on eight micropiles: six micropiles with a 228 mm (9 in) nominal diameter drill bit, and two micropiles with a 178 mm (7 in) nominal diameter drill bit. Four micropiles (MP2, MP4, MP6, and MP8) were loaded to the point of failure. The other four micropiles (MP1, MP3, MP5, and MP7) were loaded up to 133% of the design load.

Quasi-static cyclic tests were conducted on four micropiles (MP1, MP3, MP5, and MP7): three micropiles with a 228 mm (9 in) nominal diameter drill bit and one micropile with a 178 mm (7 in) nominal diameter drill bit. The axial cyclic tests involved 15 load cycles to a maximum load of 400 kN and minimum load of 200 kN in each cycle.

#### 4.4.1 Testing equipment

A reaction frame consisting of a main beam and two secondary beams, and four helical piles was used for the compression test. The main beam rested on the two secondary beams, which were connected to four square shaft Chance SS200 helical piles. The
helical piles consisted of a 50.8 mm square lead section welded to three helical plates whose sizes increased with distance from the bottom. The sizes of the helices were 200 mm, 250 mm, and 300 mm, with a 9.5 mm thickness. The lead section was connected with plain extensions of different lengths: 1.5 m, 2.0 m, and 3.1 m. The reaction piles were advanced to a depth of 9 m below the ground surface and were located at 2.5 m from the test piles (approximately 10 times the micropile diameter). Figure 4.7 shows the compression test setup.

![Figure 4.7 Compression test setup](image_url)

4.4.2 Pile instrumentation

A load cell was used to measure the load applied at the micropile head, linear potentiometers were used to measure the deflection at the micropile head and vibrating wire strain gauges were used to measure strain distribution along the micropile shaft during loading. Figure 4.8 shows the load cell and the linear potentiometers arrangement.
during the load test. The load was applied through a hollow cylinder hydraulic jack connected to a hydraulic pump. The jack had an advance capacity of 1100 kN and a maximum stroke of 150 mm. The load was recorded through a load cell with a capacity of 900 kN. The load cell was attached to a loading plate connected to the 76 mm hollow core bar. A thread bar socket was welded to the top face of the bearing plate, and the bottom face was welded to a 76 mm circular threaded collar. The bearing plate was 300 x 300 x 38 mm. The hydraulic jack was located above the load cell, pushing against the reaction frame. Four HLP 190 linear potentiometers were attached to the bearing plate to measure the vertical displacement. The linear potentiometers had a 100 mm stroke with an accuracy of 0.01 mm.

![Diagram of instrumentation](image)

**Figure 4.8 Head instrumentation**

Each micropile was instrumented with three gauges whose locations are shown in Figure 4.9. Each strain gauge was attached to a steel cage, and the cages were connected via 12.5 mm steel bars as shown in Figure 4.9. The gauges were installed immediately after grouting while the grout was still fresh; they were pushed into the grout-filled hollow bar. The lead wires of the gauges were fed through three holes at the top of the micropile head.
7 days after the installation. Prior to installation, a reading was taken for each gauge to ensure that the reading was close to 2500 linear units, which corresponds to approximately 1275±100 μsecond. At this setting, that measurement is equivalent to about 1500 μstrain, in tension or compression. This step ensured that the vibrating wire strain gauges were working properly.

![Diagram of instrumented micropile](image)

Figure 4.9 Schematic of an instrumented micropile

4.4.3 Test procedures

The monotonic compression tests included two sets. Four micropiles (MP2, MP4, MP6, and MP8) were loaded to the point of failure, while the other four micropiles (MP1, MP3, MP5, and MP7) were loaded up to 133% of the design load.
A quick maintained loading procedure was implemented for the eight micropiles. The load was applied in increments of 5 % of the anticipated failure load. For each load increment, the load was maintained at an almost constant level for 4 to 5 min, as set out in ASTM D1143 (2007). In the first set of load tests, after the failure load was reached, the load was removed over five approximately equal intervals, each maintained for 4 to 5 min. In the second set of tests, the test was stopped after 133 % of the design load was reached. Stage 2 represented part of the loading procedure for investigating the cyclic behaviour of hollow bar micropiles.

Four micropiles were subjected to cyclic compression load tests. The micropile was loaded to its design capacity, 300 kN (calculated as half of the observed ultimate compression capacity), and was then subjected to 15 load cycles to a maximum of 400 kN (i.e. 133 % of the design capacity) and a minimum of 200 kN (i.e. 67 % of the design capacity). During the initial loading (i.e. from 0 to 400 kN) the load was increased in 40 kN increments and was maintained for 4 to 5 min after each increment. After the maximum load was reached, the cycling loading started varying between 400 kN and 200 kN in each load cycle and was maintained for 2 min at the end of each cycle.

4.5 Monotonic Test Results and Analysis

4.5.1 Load-displacement curves

Micropiles MP2, MP4, MP6, and MP8 were loaded to failure in order to evaluate their ultimate capacity and the results are shown in Figure 4.10. The load-settlement curve for MP2 shows a plunging failure at approximately 658 kN as shown in Figure 4.10 a. The vibrating wire strain gauges for this micropile did not operate properly because the data acquisition system was not grounded appropriately. However, the plunging failure achieved indicates that most of the applied load was transferred through the micropile shaft.

MP4 experienced plunging failure at approximately 600 kN as shown in Figure 4.10 b. A softening behaviour was observed during the initial stages of loading and also during the
late stages of loading, when the resistance decreased with increasing vertical movement. The softening behaviour can be attributed to the presence of the shallow layer of new fill that extended approximately 2 m to 3 m along the micropile shaft.

The load-settlement curve for MP6 shows that the pile failed at 721 kN as shown in Figure 4.10c, while plunging failure was observed in MP8 at a load of 640 kN as shown in Figure 4.10d.

![Load-displacement curves for MP2, MP4, MP6, and MP8](image)

**Figure 4.10 Load-displacement curves for MP2, MP4, MP6, and MP8**

### 4.5.2 Interpreted failure criteria

If plunging failure does not occur, interpreted failure criteria can be applied as a means of determining the ultimate capacity of different types of piles. This subsection describes the interpreted failure criteria that are typically used for micropiles and explains the
identification of a criterion appropriate for use with hollow bar micropiles. Fuller and Hoy (1970) defined the failure load using a tangent to the load-settlement curve sloping at 0.15 mm/kN, as shown in Figure 4.10. This method is suitable for short piles tested under quick maintained tests. This criterion is recommended by FHWA (2005) for micropiles and can be used for verification tests.

Davisson (1972) defined the failure load as the load corresponding to settlement that exceeds the pile elastic shortening by 4 mm + D/120 (D = diameter of the pile in mm).

\[
\Delta = \frac{QL}{A_p E_p} + 4 \text{ (mm)} + \frac{D}{120}
\]  

(4.5)

where \( \Delta \) = final settlement, \( Q \) = applied load, \( L \) and \( A_p \) are micropile length and cross-sectional area, and \( E_p \) is its elastic modulus. For compression, \( A_p E_p \) is calculated as follows:

\[
A_p E_p = A_s E_s + A_g E_g
\]  

(4.6)

where \( A_s \) is the steel cross-sectional area, \( E_s \) the elastic modulus of steel, \( A_g \) is the grout cross-sectional area, and \( E_g \) is the elastic modulus of grout.

The elastic shortening line (linear portion of the load settlement curve) is represented by \( A_p E_p / L \) and can be drawn before the test is begun. In the absence of material properties, the elastic shortening line can be evaluated based on the linear portion of the load settlement curve. This method is appropriate for a quick maintained test. Butler and Hoy (1977) defined failure as the intersection of the 0.15 mm/kN slope line with the initial straight portion of the load settlement curve as shown in Figure 4.10.

Table 4.1 summarizes the results of failure load obtained using the different methods. It is noted that the failure load derived from Fuller and Hoy's method is closest to the observed plunging failure load. Both Davisson's and Butler and Hoy's methods underestimate the failure load because they assume that the axial stiffness over the length
is constant, which is not the case for micropiles. For that reason, FHWA (2005) recommends using Fuller and Hoy’s method in the absence of plunging failure.

Table 4.1 Ultimate capacity values using common failure criteria for micropiles

<table>
<thead>
<tr>
<th>Micropile Method</th>
<th>MP2</th>
<th>MP4</th>
<th>MP6</th>
<th>MP8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plunging Point</td>
<td>658 kN</td>
<td>600 kN</td>
<td>721 kN</td>
<td>640 kN</td>
</tr>
<tr>
<td>Fuller and Hoy's Method</td>
<td>640 kN</td>
<td>580 kN</td>
<td>720 kN</td>
<td>600 kN</td>
</tr>
<tr>
<td>Davisson Method</td>
<td>600 kN</td>
<td>390 kN</td>
<td>480 kN</td>
<td>520 kN</td>
</tr>
<tr>
<td>Butler and Hoy's Method</td>
<td>580 kN</td>
<td>415 kN</td>
<td>545 kN</td>
<td>540 kN</td>
</tr>
</tbody>
</table>

The average ultimate capacity of the micropiles constructed with the 228 mm drill bits was 690 kN (based on MP2, and MP6) while the ultimate capacity of the micropiles constructed with the 178 mm drill bits was 640 kN. On the other hand, FHWA (2005) classifies hollow bar micropiles as Type B micropiles with a grout-to-ground nominal strength between 70 kPa and 190 kPa for micropiles constructed in stiff silt and clay. The average undrained shear strength along the micropile length was about 100 kPa; therefore, the ultimate capacity for micropiles with 228 mm and 178 mm drill bits should be 415 kN and 325 kN, respectively, considering the nominal diameter and considering the firm to stiff soil at the test site, an average value of the grout-to-ground bond strength is expected to be 100 kPa. It is obvious from the load displacement curve that the ultimate resistance is higher than the values suggested by FHWA (2005) when the nominal diameter of the drill bit is considered in calculating the ultimate capacity. Using the approach proposed by Abdelaziz and El Naggar (2013) considering the volume of grout used (see Eqs. 2.13, and 2.17), the ultimate capacity for micropiles with 228 mm and 178 mm drill bits should be 480 kN and 410 kN. Even though these values are higher than the values calculated according to FHWA (2005), they are lower than the observed failure loads. This is attributed to the fact this approach is suitable for micropiles founded on sand.
The ultimate capacity of the micropiles that were constructed with a 228 mm drill bit was only slightly higher than those constructed with a 178 mm drill bit. This observation can be attributed to two factors. Firstly, the grout volume/hole volume ratio is equal to 1.2 and 1.44 for the 228 mm and 178 mm drill bits, respectively. This indicates that more grout penetrated the surrounding soil with the 178 mm drill bit than with the 228 mm drill bit, hence increasing the grout-to-ground bond. Secondly, the back pressure measured in the case of the 178 mm drill bit was 30% greater than the back pressure with the 228 mm drill bit due to the smaller size of the 178 mm drill bit nozzles (6.35 mm) compared to the size of the 228 mm drill bit nozzles (12.7 mm).

Using the FHWA ultimate capacity equation and considering the nominal drill bit diameter, the average bond strength was back-calculated to be 200 kPa and 168 kPa for micropiles with drill bits of 178 mm and 228 mm, respectively. These values are high for the type of soil at this site. Thus, the micropile capacity should be calculated considering the increased diameter of the hole during drilling and grouting. Considering the volume of grout used, the actual micropile diameter was probably close to 215 mm and 245 mm for piles constructed with 178 mm and 228 mm drill bits. The average bond strength would then be approximately 165 kPa and 156 kPa, respectively. These bond strength values were established based on the FHWA standards, which assume zero toe resistance. This will be discussed further later.

### 4.5.3 Load transfer mechanism

The readings of the vibrating wire strain gauges were used to evaluate the load transfer mechanism. The axial force at different depths was calculated based on the strains measured, as follows:

\[
P_z = \varepsilon A_p E_p
\]  
(4.7)

where \(\varepsilon\) is the measured strain, \(A_p\) is the cross-sectional area of the micropile, and \(E_p\) is the elastic modulus of the micropile material. Because the hollow bar is fully bonded
with the grout, the strains in the grout and hollow bar are equal. The elastic modulus of the micropiles under compression can be calculated using

\[ A_p E_p = A_s E_s + A_g E_g \]  \hspace{1cm} (4.8)

where \( A_s \) is the steel cross-sectional area, \( E_s \) the elastic modulus of steel (200 GPa), \( A_g \) is the grout cross-sectional area, and \( E_g \) is elastic modulus of the grout used (15.05 GPa).

### 4.5.4 Load distribution

Due to challenges related to the grounding of the data acquisition system, the strain gauges of MP2 did not read properly. However, the load settlement curve indicated that the skin friction was fully mobilized, with an insignificant contribution from the tip resistance (the curve demonstrates slight nonlinearity followed by plunging failure). Based on the measured strains and the computed equivalent micropile modulus, the axial forces at different strain gauge levels were calculated using Equation No. 4.7. The load transfer curves (i.e., distribution of axial force along the shaft) are shown in Figure 4.11 for MP4, MP6, and MP8. It is noted from Figure 4.11 that the skin friction was fully mobilized as manifested by an increase at the toe load equal to the load increment at the pile head. It is also noted that MP4 and MP8 displayed small toe resistance (12% and 6%, of the applied load). It is clear that in MP4 about 88% of the applied load was transferred to the soil through the shaft resistance while 94% of the applied load was transferred to the soil through the shaft resistance in MP8. This result can be explained by the fact that MP4 was installed in a shallow layer of fill and MP8 has a stronger bond between the soil and the grout body due to the greater pumping power during the grouting. MP6 had significant toe resistance (34% of the applied load). MP6 was overlying a sand layer (shown in BH-II). Abd Elaziz (2012) reported similar results for a hollow bar micropile that was installed in the same site.
Figure 4.11 Load distribution for each applied load for MP4, MP6, and MP8

The load transfer curves for the micropiles that were loaded to only 400 kN (no failure) reveal that the applied load was primarily resisted by skin friction (Figure 4.12). It is noted from Figure 4.12 that the shaft resistance accounted for 90, 84, 90 and 97% of the applied load for MP1, MP3, MP5 and MP7. The shaft resistance of MP3 was installed in a shallow layer of new fill and its shaft resistance may have been affected accordingly. These observations indicate that the skin friction dominates the load transfer mechanism for hollow bar micropiles except for situations where the micropile overlies a sand layer, in which case, the toe resistance may provide a significant contribution.
4.5.5 Toe resistance

The toe resistance was calculated using the strain reading at the pile toe. The toe resistance for MP4 and MP8 accounted for only 12% and 6% of the applied load as shown in Figures 4.14a and 4.14c. MP4 has a larger toe diameter ($d_{bit} = 228$ mm) than MP8 ($d_{bit} = 178$ mm). The much larger increase in toe resistance for MP6 is attributed to enlarged toe diameter due to the penetration of the grout through the cohesionless soil.
The toe displacement was obtained by subtracting the compression of the micropile shaft from the total measured displacement. The compression of the micropile shaft was approximated by the following (Han & Ye, 2006):

\[
\Delta_{sp} = \frac{(P_p + P_b)L}{2A_pE_p}
\]

(4.9)

where \(\Delta_{sp}\) is the compression of the micropile shaft, \(P_p\) is the applied load at the micropile head (measured by the load cell) and \(P_b\) is the toe resistance evaluated from the strain reading at the pile toe.

Figure 4.13 indicate the variation of the head and toe movements with the applied load, and Figure 4.14 shows the toe load-displacement curves for the micropiles tested in this study. Figure 4.13 shows that the head and toe resistance-displacement curves displayed very much the same behaviour, indicating that the performance is dominated by the shaft resistance. This confirms the observations made based on the load transfer curves. Figure 4.14 shows that the toe resistance displayed strong nonlinearity and it has reached plateau (i.e. yield was attained) at a toe displacement = 10 mm (i.e. 4 % to 5 % of the average micropile diameter). This yield point corresponded to 55 kN and 35 kN for micropiles MP2 and MP8, respectively. Based on these values and the nominal micropile diameter, the ultimate toe bearing capacity, \(q_t\) of MP2 and MP8 can be calculated to be 1167 kPa and 965 kPa, respectively. These values are higher than toe bearing capacity following the Canadian Foundation Engineering Manual (CFEM, 2006), i.e., \(q_t = 9 \ C_u = 810 \) kPa. This discrepancy can be attributed to two factors: increase in the micropile toe diameter; and improved strength of soil below toe due to the penetration of the pressurized grout (Abdelaziz & El Naggar, 2012). For micropiles installed in soft clay, Han and Ye (2006) found that the actual diameter at the bottom of the micropile can be increased by up to 1.5 times the nominal micropile diameter. Considering the toe resistance and increased diameter (as evaluated previously) of MP2 and MP 8, the undrained strength of the soil below their toes was expected to have increased by 20-44%.
The toe of MP6 was resting on the medium dense to dense sand layer, and hence its toe resistance was significant. The toe resistance of piles resting on sand can be given by, 
\[ P_b = \sigma' N_q A, \]
where \( N_q \) is the net bearing capacity factor as per CFEM (2006). For medium dense/dense sand with \( \phi = 35^\circ \), \( N_q = 26 \), which leads to \( P_b = 155 \text{ kN} \) for MP6. Considering the measured toe resistance of 250 kN, either the toe diameter was enlarged to 310 mm or the strength of the soil has increased by 50-60%.

The ultimate toe resistance for MP6 was reached at toe displacement = 20-25 mm (10% of the micropile toe diameter). This is consistent with observations for small diameter drilled shafts resting on cohesionless soil (O'Neil & Reese, 1999) and the conclusions reached by Bruce and Yeung (1984) for micropiles in cohesionless soil.
Figure 4.13 Load-displacement diagram for micropile head and toe

Figure 4.14 Toe resistance versus toe displacement
4.5.6 Unit skin friction

The unit skin friction can be calculated by the change in axial force between two strain gauge levels divided by the micropile surface area between the two strain gauge levels. In general, the unit skin friction increases as the applied load increases up to the ultimate unit skin friction, which characterizes the grout-ground bond strength. Figure 4.15 presents the calculated unit skin friction for MP4, MP6 and MP8. As can be noted from Figure 4.15, MP6 and MP8 demonstrated high ultimate unit skin friction, 137 KPa and 171 KPa, along the top 3 m of soil (stiff clay). However, MP4 exhibited ultimate unit skin friction along the top 3 m of soil less than 100 kPa. This is attributed to the presence of the weak new fill layer.

The ultimate unit skin friction for the bottom layer varied between the three micropiles for different reasons. MP4 exhibited an ultimate unit skin friction of 175 kPa, which is consistent with the value offered by the stiff clay layer (as observed for the top 3 m along MP6 and MP8). For MP8, the ultimate unit shaft friction along the bottom layer was 152 kPa, which is also consistent with the ultimate unit shaft friction for the stiff clay. For MP6, the shaft friction along the bottom part seems to be affected by the soft to firm clay lay that appears in BH-II. In addition, the shaft friction could not be mobilized along a length above the pile toe at least equal to the expanded diameter of the pile toe. Similar results were reported by Narasimha Rao et al. (1991) and Zhang (1999) based on load tests of helical piles. They suggested that the shaft adhesion could not be mobilized along a length of one helix diameter, \( D \), above the helix because of the “shadowing effect”. This shadowing effect resulted in reduced average ultimate unit shaft friction over the bottom layer, which was about 66 kPa. It is also noted that the average skin friction value of MP8 (178 mm drill bit) is higher than that of MP4 or MP 6 (228 mm drill bit). This may have been affected by the different soil conditions for MP4 and MP6 as discussed. Nonetheless, the average ultimate unit skin friction of MP4 and M6 is about 1 to 1.25 times the undrained shear strength of the adjacent soil; while for MP8, the average ultimate unit skin friction is about 1.6 times the undrained shear strength of the adjacent soil. This can be attributed to that MP8 has a stronger bond between the soil and the
grout body due to the greater pumping power during the grouting that influenced by smaller size of the 178 mm drill bit nozzles (6.35 mm) compared to the size of the 228 mm drill bit nozzles (12.7 mm). Table 4.2 compares the ultimate unit skin friction values calculated from Figure 4.15 and the bond strength based on FHWA (2005) accounting for the toe resistance.

**Table 4.2 Summary of the ultimate skin friction values**

<table>
<thead>
<tr>
<th>Skin friction (kPa)</th>
<th>MP2</th>
<th>MP4</th>
<th>MP6</th>
<th>MP8</th>
</tr>
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<tbody>
<tr>
<td>FHWA equation</td>
<td>---</td>
<td>120</td>
<td>102</td>
<td>156</td>
</tr>
<tr>
<td>Upper part: 0 m - 3.25 m</td>
<td>89.9</td>
<td>137</td>
<td>171</td>
<td></td>
</tr>
<tr>
<td>Lower part: 3.25 m - 5.5 m</td>
<td>175</td>
<td>66</td>
<td>153</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>---</td>
<td>125</td>
<td>108</td>
<td>163</td>
</tr>
</tbody>
</table>
4.6 Cyclic Test Results and Analysis

4.6.1 Load-displacement curves

The behaviour of piles under a cyclic load is complex (El Naggar & Wei, 2000). The results of the cyclic load tests are presented and discussed in order to shed some light on the effects of cyclic loading on the micropile stiffness and load carrying capacity. The results are discussed in terms of the load displacement curve, the micropile head displacement with each cycle, the change in stiffness with each cycle, and the change in transferred loads with each cycle.
Figure 4.16 shows that all micropiles exhibited an increase in head displacement with the number of load cycles. In addition, micropiles constructed with 228 mm drill bits displayed less head displacement compared with the micropile constructed with a 178 mm drill bit, except for MP3, which was constructed in the new fill layer. After 15 load cycles, the head displacements were 2.1 mm, 1.9 mm, 2.9 mm and 4.7 mm for MP1, MP5, MP7 and MP3, respectively.

Figure 4.16 Load displacement curves for MP1, MP3, MP5, and MP7
The cumulative head displacement values for each cycle are shown in Figure 4.17. The vertical displacement of MP1 increased from 3.7 mm at the end of first cycle to 5.8 mm after 15 cycles; while for the head displacement for MP5 increased from 4.0 mm to 5.9 mm after the first and last cycles and for MP7 increased from 4.9 mm to 7.7 mm. MP3 displayed the largest head displacement after 15 cycles as it increased from 6.2 mm to 10.9 mm. The small change in displacement with each cycle can be attributed to small breakdown of the bond between the clay particles, resulting a small plastic deformation in those particles at the interface between the micropile and the soil.

![Figure 4.17 Vertical displacement versus number of cycles for MP1, MP3, MP5, and MP7](image-url)
4.6.2 Stiffness

The effect of cyclic loading on the micropile stiffness is evaluated by calculating the stiffness in each load cycle. The micropile stiffness is calculated as the slope of the load-displacement curve for each loading cycle, i.e.:

\[ K = \frac{P_{\text{max}} - P_{\text{min}}}{\Delta_{\text{max}} - \Delta_{\text{min}}} \]  

where

\[ P_{\text{max}} \text{ and } P_{\text{min}} \text{ = the maximum and minimum applied load at each cycle, respectively} \]

\[ \Delta_{\text{max}} \text{ and } \Delta_{\text{min}} \text{ = the maximum and minimum displacement at each cycle, respectively} \]

The change in stiffness at each cycle is represented by the stiffness ratio, \( K/K_i \), where \( K_i \) is the initial stiffness (first cycle).

Figure 4.18 shows the change in head stiffness for the micropiles with each cycle. It is noted from Figure 4.18 shows that the stiffness ratio varied between 0.94 and 1.24, but was mostly around 1. These values confirm that the cyclic loading had an insignificant effect on the axial performance of micropiles in lean clay. The results observed agree with the findings of Abd Elaziz and El Naggar (2012). Finally, the results indicate that no debonding occurred and no stiffness degradation.
4.6.3 Effect of cyclic loading on load distribution

The load transfer along the micropiles during the cyclic loading was evaluated from the vibrating wire strain gauge readings for MP1, MP3, and MP7 and the results are shown in Figures 4.19. In addition, there was 17% decrease in the load transferred to the lower part of the micropile after 15 load cycles, while the load transfer through the top increased by 24.5% in case of MP1. This means the cyclic loading redistributed the load transfer...
along the micropile shaft. This was accompanied by a small increase in the toe resistance. The same behaviour was observed in MP3 and MP7 but with different percentages.

The results shown in Figure 4.19 demonstrate that the unit shaft friction of the top part of the micropiles was mobilized fully due to the cyclic loading while the shaft friction over the lower part of micropile had decreased due to the load distribution. However, the average shaft friction along the micropile remained almost the same after the cyclic loading.

![Figure 4.19 Measured load versus applied load for MP1, MP3, and MP7](image-url)
4.7 Summary

Full-scale compression pile load tests were conducted on four micropiles in a firm to stiff clay. The micropiles consisted of Type BX76 geo-drilled anchors with 76 mm OD and 48 mm ID and either a 178 mm or a 228 mm carbide bit threaded onto the bar to advance the micropile down the hole using an air/water flushing technique. The study indicated that, in the absence of plunging failure, Fuller and Hoy’s method provides a good estimation of the ultimate capacity. The values proposed by FHWA (2005) for type B micropiles underestimate bond strength for calculations the ultimate capacity. The increase in the micropiles diameter ranged from 10 % to 20 % with drill bit diameters of 228 mm and 178 mm, respectively. The enlargement of the toe diameter of the micropile resting on sand was about 35 % of the drill bit diameter.

The ultimate capacity of micropiles installed in stiff clay was mobilized at a head displacement of 5 % of the micropile diameter. For micropiles resting on medium dense/dense sand, the ultimate capacity was fully mobilized at head displacement equal to 10% of the micropile diameter.

The average ultimate skin friction was about 1 to 1.25 times the undrained shear strength for micropiles with a 228 mm drill bit and 1.6 times the undrained shear strength for micropiles with a 178 mm drill bit due to the difference in nozzle size, which resulted in higher back pressure for the case of the 178 mm drill bit. It is recommended that the design of the 228 mm drill bit to be revised to reduce the nozzle size to the same size used in the 178 mm drill bit. These observations demonstrate that the performance of hollow bar micropiles is sensitive to the construction technique and the drill bit specifications.

The micropiles generally exhibited excellent performance under axial cyclic loading. No degradation in stiffness or debonding was observed after 15 load cycles.
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Chapter 5

Axial Uplift Behaviour of Hollow Bar Micropiles

5.1 Introduction

Micropiles are employed in applications where it can sustain axial compression and/or tension. Generally, three main steps are involved in constructing micropiles: drilling, placing reinforcement, and grouting, all of which can affect the capacity of the micropiles. For example, the drilling method and quality of grout affect the degree of bonding between the grout and the ground. In the last ten years, the use of hollow bar micropiles has become more popular than other micropile systems because they provide fast and efficient installation with a high degree of ground improvement. In contrast with other micropile systems, hollow bar micropiles need only a one-step operation for installation: the hole is drilled, reinforced and grouted simultaneously.

The geotechnical uplift capacity of a micropile is a function of its skin friction, which is the primary contributor to the load transfer mechanism (Bruce and Yeung, 1984; Juran et al. 1999; and Cadden et al. 2004). According to FHWA (2005), the geotechnical capacity in tension is equal to the geotechnical capacity in compression because the design of the micropile depends on skin friction.

Han and Ye (2006) investigated a full-scale load test of four piles installed in soft clay. Two piles were subjected to compression loading and two to tension loading. The ultimate tension skin friction was 0.68 to 0.73 times the undrained shear strength, but the skin friction values were 4 % to 10 % higher than those suggested for bored concrete piles. Thomson at al. (2007) reported the results of axial compression, axial tension, and lateral load tests on encased micropiles. They found that the mobilized grout-to-ground bond strength values for two micropiles under an uplift loading were approximately 150 kPa and 190 kPa based on the outside drill casing diameter. They also determined that the volume of grout used in the hole was greater than the theoretical volume of the hole, an indication that the diameter of the uncased portion of the micropiles was increased.
Telford et al. (2009) conducted verification tests for threaded hollow stem Titan 73/45 (73 mm O.D./45 mm I.D.) bars installed to a depth of 9.8 m in cohesionless soil using a 115 mm cross drill bit. They reported that the micropiles were able to sustain high loads in both compression and tension, with only a small amount of deformation. In addition, the grout-to-ground bond strength varied between 265 kPa to 400 kPa and the pile diameter was enlarged by 1.5 times the nominal drill bit. The grout-to-soil bond strength test values were very close to the FHWA (2005). Bennett and Hothem (2010) conducted load tests on four pairs of micropiles installed in a layer of very soft to soft clays and sandy clays or very loose to loose clayey sands extending from near the ground surface to typical depths of approximately 4.5 m to 6 m. Two different drill bits were used for each pair: a 150 mm clay bit and a 115 mm cross bit. The shortest pile carried 480 kN and 460 kN for the 150 mm clay bit and the 115 mm cross bit, respectively. The 150 mm clay bit performed marginally better than the 115 mm cross bit.

Abd Elaziz and El Naggar (2010, 2011, and 2012) examined five micropiles installed in a thick layer of overconsolidated clayey silt to silty clay till overlying a layer of compact to dense sand extending to a depth of 9 m. The results revealed that considering hollow core micropiles as type B underestimates the grout-ground bond strength.

The literature related to micropiles behaviour is limited. In addition, there is insufficient data related to hollow bar micropiles, especially for hollow bar micropiles in clay. This variation arises from neglecting observations at the site during drilling and grouting, such as the type of grout, the amount of pressure during grouting, and the extent of increase in the diameter during grouting. Hence, the research presented in this chapter involved a field study of the performance of hollow bar micropiles in cohesive soils under tension. The study investigated the axial behaviour of hollow bar micropiles installed using two sizes of drill bit. The results of the full-scale load tests are presented and discussed with respect to load-displacement curves, and skin friction.
5.2 Site Conditions

The micropiles were installed at the Western University Environmental Site, located 8 km north of London, Ontario, which is comprised primarily of cohesive soil. Two boreholes (denoted BH-I and BH-II) were drilled close the location where the micropiles were to be installed and 8.4 m apart, as shown in Figure 5.1. Samples were extracted from each borehole by means of hollow stem auguring followed by a standard penetration test (SPT) and split spoon sampling. A monitoring well was installed in order to measure the groundwater level. Laboratory testing was performed on the samples collected.

BH-I and BH-II (summarized in Tables 5.1 and 5.2) show that the soil profile at the test site is composed of: 200-300 mm top soil underlain by a layer of firm to stiff clay that extends to a depth of 5 m. A soft to firm dark grey clay layer appeared between 5 m and 6.5 m. This layer is underlain by very stiff dark grey lean clay with seams of sand that extend to a depth of 9m. In BH-II, the groundwater level was measured at a depth of 6.41 m.

![Figure 5.1 Borehole and pile distribution in the study area (all dimensions in mm)](image)
Due to the cohesive nature of the soil, and the rapid loading during the load tests, the strength of the soil is represented by its undrained shear strength ($C_u$). Values for the undrained shear strength were obtained from measured SPT values using correlations between undrained shear strength and SPT. Terzaghi and Peck (1967) suggested the correlation between the $C_u$ and $N_{60}$ for fine-grained soil, expressed as follows:

$$C_u = 6.5 \ N_{60} \ (\text{kPa}) \quad (5.1)$$

For insensitive clay, Stroud (1974) suggested the following correlation:

$$C_u = (3.5 - 6.5) \ N_{60} \ (\text{kPa}) \quad (5.2)$$

where $N_{60}$ = the standard penetration number, corrected for field conditions to an average energy ratio of 60%:

$$N_{60} = C_h C_r C_s C_d N \quad (5.3)$$

where

- $N$ = the measured SPT number
- $C_h$ = the rod energy ratio normalized to 60% 
- $C_r$ = the correction for the rod length 
- $C_s$ = the sampler correlation 
- $C_d$ = the correction for the borehole diameter

Skempton (1986) suggested a correction for the rod length ($C_r$), the sampler ($C_s$), and the borehole diameter ($C_d$). The $C_r$ value was taken to be 0.75 for a rod length between 3 m and 4 m, 0.85 for a rod length between 4 m and 6 m, and 0.95 for a rod length between 6 m and 10 m. The $C_s$ value was selected as 1.2 for a sampler without liners, and the $C_d$ value used was 1.15 for a borehole diameter of 200 mm. The undrained shear strength values calculated using these methods are listed in Tables 6.1 and 6.2.
For the sand layer, Peck et al. (1974) and Terzaghi et al. (1996) provided an empirical correlation between N_{1.60}, and the effective friction angle for both fine and coarse grained sands in a graphic form. Anderson et al. (2003) approximated the relation into a logarithmic equation:

\[ \varphi' = 53.881 - 27.6034e^{-0.0147N_{1.60}} \]  \hspace{1cm} (5.4)

where

\[ N_{1.60} = N_{60} C_N \]  \hspace{1cm} (5.5)

where C_N = the overburden correction factor and is calculated from

\[ C_N = 0.77 \log \frac{1920}{\sigma'_o} \]  \hspace{1cm} (5.6)

where \( \sigma'_o \) = the effective vertical stress in kN/m\(^2\)
### Table 5.1 Summary of soil properties for BH I

<table>
<thead>
<tr>
<th>Layer</th>
<th>H (m)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>γ kN/m³</th>
<th>*C_u kPa</th>
<th>φ°</th>
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<td><strong>Top soil</strong></td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>0.2-0.3</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Firm to Stiff Lean Clay</strong></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(6% gravel, 16% sand, 78% silt/clay)</td>
<td>4.0</td>
<td>12.28</td>
<td>24.92/</td>
<td>15.68/</td>
<td>22.3</td>
<td>100</td>
<td>-</td>
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<tr>
<td><strong>Firm to Stiff Lean Clay</strong></td>
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<tr>
<td>(11% gravel, 32% sand, 60% silt/clay)</td>
<td>1.0</td>
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<td>17.53/</td>
<td>12.76/</td>
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<td>(14% gravel, 17% sand, and 69% silt and clay)</td>
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<td>19.9</td>
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<tr>
<td>(1% gravel, 9% sand, 90% silt/clay)</td>
<td>2.5</td>
<td>11.9/</td>
<td>18.62/</td>
<td>12.37/</td>
<td>19.9</td>
<td>100/ 300</td>
<td>-</td>
</tr>
<tr>
<td>*According to Terzaghi and Peck (1967) and Stroud (1974)</td>
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<td></td>
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### Table 5.2 Summary of soil properties for BH II

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<th>Layer</th>
<th>H (m)</th>
<th>w (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>γ kN/m³</th>
<th>*C_u kPa</th>
<th>φ°</th>
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<td><strong>Firm to Stiff Lean Clay</strong></td>
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<td></td>
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<td>16.00/</td>
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<tr>
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<td>(6% gravel, 24% sand, 70% silt/clay)</td>
<td>2.75</td>
<td>10.50</td>
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<td>13.30/</td>
<td>22.3</td>
<td>75</td>
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<td><strong>Medium Dense/Dense Sand</strong></td>
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</tr>
<tr>
<td>(1% gravel, 53% sand, 46% silt clay)</td>
<td>0.5</td>
<td>17.30</td>
<td>24.30</td>
<td>12.60</td>
<td>22.3</td>
<td>-</td>
<td>35</td>
</tr>
<tr>
<td><strong>Hard Lean Clay</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(0% gravel, 3% sand, and 97% silt and clay)</td>
<td>1.0</td>
<td>18.10</td>
<td>21.20</td>
<td>14.30</td>
<td>19.8</td>
<td>325</td>
<td>-</td>
</tr>
<tr>
<td><strong>Medium Dense to Dense Sand</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1% gravel, 72% sand, and 27% silt and clay)</td>
<td>0.5</td>
<td>11.70</td>
<td>21.20</td>
<td>14.40</td>
<td>19.8</td>
<td>-</td>
<td>39</td>
</tr>
<tr>
<td><strong>Hard Clay</strong></td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>*According to Terzaghi and Peck (1967) and Stroud (1974)</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
5.3 Micropile Installation

Eight hollow bar micropiles were installed: six with 228 mm (9 in) diameter and two with 228 mm (7 in) diameter drill bits. They were placed in two rows: four micropiles placed 4 m apart in each row, with the rows being 3 m apart (Figure 5.1). The drill bits are shown in Figure 5.2. The installation involved the following steps: rotary percussive drilling with air and water flushing; installation of first 3m hollow bar (Geo-Drill BX 76/48 with 76 mm OD and 48 mm ID); adding second hollow bar section using a B7X2-76 coupler; after the desired depth (5.75 m) was reached, grout (Type 10 Portland cement mixed at water-to-cement ratio = 0.45) was introduced under 1.1 MPa (157 psi) pressure. Only 4 micropiles (i.e., MP1, MP3, MP5, and MP7) were tested in tension after 6 weeks of curing time, and have never been tested for compression to the point of failure.

The installation records and grout quantities used suggested that the size of the holes increased by a factor of 1.1 of the drill bit diameter for the 228 mm drill bit and by a factor of 1.2 for the 178 mm drill bit. Maclean’s (2010) made similar observations. The difference in enlargement ratio is attributed to the differences in the grouting pressure and the nozzles size (the nozzles in the 178 mm drill bit are smaller than those in the 228 mm drill bit). The average quantity of cement bags used per hole was 7-8 bags (280 kg to 320 kg) and 9-10 bags (360 kg to 400 kg) when drill bits with diameters of 178 mm and 228 mm, respectively, were used. These quantities correspond to approximately 0.215 m$^3$ and 0.276 m$^3$ of grout for drill bit diameters of 178 mm and 228 mm, respectively.

The 28-day grout compressive strength was 41.4 MPa, which meets the limit specified by FHWA (2005). The average 28-day split tensile strength and flexural tensile strength were 4.2 MPa and 5.2 MPa, respectively. The average elastic modulus was 15.05 GPa.
Figure 5.2 228 mm (9 in) and 178 mm (7 in) drill bits

a) 228 mm (9 in) diameter drill bit: top view
b) 228 mm (9 in) diameter drill bit: side view
c) 178 mm (7 in) diameter drill bit: top view
d) 178 mm (7 in) diameter drill bit: side view
5.4 Test setup, Method, and Instrumentation

Four tension tests were conducted on four micropiles: three micropiles with 228 mm (9 in) nominal diameter drill bits, and one micropile with a 178 mm (7 in) nominal diameter drill bit. All micropiles (MP1, MP3, MP5, and MP7) were loaded to the point of failure.

5.4.1 Testing equipment and micropile instrumentation

In the uplift load tests, the reaction frame included a main beam resting on wooden beams as shown in Figure 5.3. A hollow cylinder hydraulic jack was located above the loading beam, pushing against the loading beam, and a hex nut was positioned above the hydraulic jack. The load was applied through the hydraulic jack, which was connected to a hydraulic pump, and was recorded through a load cell. The hydraulic jack was connected to the load cell via a 76 mm threaded bar. The load cell was attached to a loading plate connected to the 76 mm hollow core bar. A thread bar socket was welded to the top face of the bearing plate, and the bottom face was welded to a 76 mm circular threaded collar. The bearing plate was 300 x 300 and 38 mm thick. The hydraulic jack had an advance capacity of 1100 kN and a maximum stroke of 150 mm. The capacity of the load cell was 900 kN. Four HLP 190 linear potentiometers were attached to the bearing plate to measure the vertical displacement. The linear potentiometers had a 100 mm stroke with an accuracy of 0.01 mm. Figure 5.4 shows the pile head instrumentation.

Vibrating wire strain gauges were used for measuring the micropiles internal strains. Each micropile was instrumented with three gauges whose locations are shown in Figure 5.5a. Each strain gauge was attached to a steel cage, and the cages were connected via 12.5 mm steel bars as shown in Figure 5.5b. The gauges were installed immediately after grouting while the grout was still fluid. They were pushed into the grout-filled hollow bar. The lead wires of the gauges were fed through three holes at the top of the micropile head 7 days after the installation was completed. The strain gauges read 1500 microstrains, in either tension or compression.
5.4.2 Test procedures

A quick maintained load test procedure was employed, in which the load was applied in increments of 5% of the anticipated failure load. After the point of failure was reached, the load was removed in five approximately equal increments. For each increment, the load was maintained for 4 min to 5 min as set out in ASTM D3689, (2007).
Figure 5.4 Head instrumentation
5.5 Monotonic Test Results

5.5.1 Load-displacement curves

All micropiles were loaded until plunging failure has occurred, i.e. the peak load could not be maintained accompanied by a large increase in displacement rate. Figure 5.6 shows the load-displacement curves for the micropiles tested in tension. It is noted from Figure 5.6 that all micropiles exhibited almost the same behaviour. Only MP3 demonstrated some softening behaviour in the early stage of loading that started at a load of 120 kN. However, it continued to sustain load until it plunged to failure at approximately 575 kN. The softening behaviour confirms the observation made during the drilling that a shallow layer of new fill, extending approximately 2 m to 3 m along the micropile shaft. The other 3 micropiles, MP1, MP5 and MP7 exhibited plunging failure at about 600 kN.
5.5.2 Interpreted failure criteria

Several interpreted failure criteria can be applied for different types of piles in order to determine the ultimate uplift capacity, but not all of them necessarily apply to micropiles. This section describes the investigation of a number of failure criteria and compares them with the ultimate uplift capacity that was found based on the plunging point.

FHWA (2005) recommends using the criterion suggested by Fuller and Hoy’s (1970), which defines the failure load by the tangent of the load-displacement curve is the movement curve that slopes at 0.15 mm/kN. Davisson (1972) defined the failure load as
the load corresponding to the amount of displacement that exceeds the elastic displacement by 4 mm + D/120 (D = diameter of the pile in mm):

\[
\Delta = \frac{Q L}{A_p E_p} + 4 \text{ (mm)} + \frac{D}{120}
\] (5.7)

where \(\Delta\) is the final displacement, \(Q\) is the applied load, \(L\) is the pile length, \(A_p\) is the cross-sectional area of the pile, and \(E_p\) is the modulus of elasticity of the pile material. For tension, \(A_p E_p\) is calculated as follows:

\[
A_p E_p = A_s E_s
\] (5.8)

where \(A_s\) and \(E_s\) are steel cross-sectional area, and its elastic modulus.

Butler and Hoy (1977) defined the failure load at the intersection of the 0.15 mm/kN slope line with the initial straight portion of the load displacement curve.

The failure load interpreted from the load-displacement curves employing the three methods are summarized in Table 5.3, in addition to the plunging point observed in the tests. Fuller and Hoy’s method clearly provides the estimate of the failure point that is closest to the observed plunging point. The failure loads derived from both Davisson’s and Butler and Hoy’s methods are the same for all of the test micropiles, with the exception of MP3, which was constructed on new fill; however, both of these methods underestimate the failure load.

<table>
<thead>
<tr>
<th>Method</th>
<th>MP1</th>
<th>MP3</th>
<th>MP5</th>
<th>MP7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plunging Point</td>
<td>656 kN</td>
<td>575 kN</td>
<td>600 kN</td>
<td>600 kN</td>
</tr>
<tr>
<td>Fuller and Hoy’s Method</td>
<td>600 kN</td>
<td>500 kN</td>
<td>560 kN</td>
<td>560 kN</td>
</tr>
<tr>
<td>Davisson Method</td>
<td>500 kN</td>
<td>240 kN</td>
<td>440 kN</td>
<td>440 kN</td>
</tr>
<tr>
<td>Butler and Hoy’s Method</td>
<td>500 kN</td>
<td>360 kN</td>
<td>430 kN</td>
<td>440 kN</td>
</tr>
</tbody>
</table>
FHWA (2005) categorizes hollow bar micropiles as type B micropiles with average values of grout-to-ground nominal strength ranging from 70 kPa to 190 kPa for micropiles constructed on silt and clay. The grout-to-ground strength is expected to be 100 kPa for the site soils (firm to stiff clay with $C_u = 75-100$ kPa). Consequently, the uplift capacity calculated considering the nominal pile diameter (i.e., drill bit diameter) would be 415 kN for a micropile with 228 mm drill bit, and 325 kN for 178 mm drill bit.

Based on the load displacement curves, the failure load for the micropile with a 178 mm drill bit (MP7) is 600 kN, and for micropiles with 228 mm drill bit is 628 kN (average of MP1 and MP5 failure loads). It is obvious that the difference between the micropile with a 178 mm drill bit and those with a 228 mm drill bit is insignificant. This observation is attributed to the greater grouting back pressure of MP7 than the case for MP1 and MP5. The higher back pressure was due to the smaller nozzles size of the 178 mm drill bit (6.35 mm) compared to the nozzles size of the 228 mm drill bit (12.7 mm). The different grouting back pressures caused correspondingly different increases in the hole volume: 1.2 and 1.44 of its nominal volume for the 228 mm and 178 mm drill bits, respectively. Also, the higher back pressure resulted in increased grout-ground bond strength.

The bond strength can be back-calculated from FHWA’s ultimate capacity equation, taking into consideration the enlargement in micropile diameter. The actual micropile diameters based on the site observations during the drilling and grouting were probably closer to 215 mm and 245 mm for micropiles constructed with drill bits with $d_{bit} = 178$ mm and 228 mm, respectively. Therefore, the average bond strengths are 155 kPa and 142 kPa for micropiles with 178 mm and 228 mm drill bits, respectively.
5.5.3 Load transfer mechanism

The axial forces at any depth were calculated based on the strain readings from the vibrating wire strain gauges as follows:

\[ P_z = \varepsilon A_p E_p \]  (5.9)

where \( \varepsilon \) is the strain in the vibrating wire strain gauges, \( A_p \) is the cross-sectional area of the pile, and \( E_p \) is the modulus of elasticity of the pile material.

Because the tensile strength of grout is approximately 10% of its compressive strength, the grout contribution to the micropile tensile stiffness is insignificant. The \( A_p E_p \) of micropiles under tension can then be calculated as:

\[ A_p E_p = A_s E_s \]  (5.10)

where \( A_s \) and \( E_s \) are steel cross-sectional area, and its elastic modulus (200 GPa).

5.5.4 Load transfer

Figure 5.7 shows the load distribution along the micropile shaft for different levels of load applied load at its head. As expected, the load transfer was through the shaft during the uplift loading. This was clearly demonstrated for MP1, MP5 and MP7. However, MP3 shows a small seemingly contribution of toe resistance (less than 6% of the applied load). This is most likely the contribution of the shaft below the bottom strain gauge (0.25 m). As shown in Figure 5.5a, strain gauge No. 1 is placed at 0.25 m above the pile toe. The percentage contribution of this part is noticeable in MP3 because it was installed in a shallow layer of new fill, where the shaft resistance from the upper part was less than other piles. Figure 5.6 shows that all piles performed basically the same, without any discernible effect of the drill bit size due the higher grout-to-ground bond strength for the micropile constructed using the smaller drill bit, which compensated for the smaller diameter.
Figure 5.7 Load distribution for each applied load for MP1, MP3, MP5, and MP7
5.5.5 Unit skin friction

The unit skin friction values are calculated as the difference of axial forces at two consecutive strain gauge divided by the surface area between the two strain gauges. The unit skin friction values were evaluated at different levels of applied load at the micropile head for the four micropiles tested in tension, i.e. MP1, MP3, MP5 and MP7. The results obtained are presented in Figure 5.8.

Figure 5.8 Shows that unit skin friction along micropile shaft. MP1 showed that the ultimate skin friction over the top part was 134 kPa, while the ultimate skin friction along the bottom part was about 178 kPa, with an average ultimate unit skin friction for MP1 equal to 152 kPa. MP3 and MP5 exhibited similar behaviour, but with average skin friction values of 126 kPa and 140 kPa for MP3 and MP5, respectively. For MP3, the ultimate skin friction over the top part was 113 kPa, while the ultimate skin friction along the bottom part was about 144 kPa. For MP5, the ultimate skin friction over the top part was 127 kPa, while the ultimate skin friction along the bottom part was about 157 kPa. The lower ultimate unit skin friction observed in the case of MP3 is attributed to the presence of the new fill layer. MP7 showed that the ultimate skin friction over the top part was 215 kPa, while the ultimate skin friction along the bottom part was about 84 kPa. MP7 exhibited an average ultimate average skin friction value of 162 kPa due to the greater back pressure during the grouting process as discussed previously. The average ultimate skin friction value for MP1 and M5 was about 1.5 times the undrained shear strength; however, it was 1.6 times the undrained shear strength for MP7. Table 5.4 provides a comparison of the ultimate skin friction values calculated from Figure 5.8 and the bond strength based on FHWA standards.
Table 5.4 Summary of Unit skin friction values

<table>
<thead>
<tr>
<th>Unit Skin friction (kPa)</th>
<th>MP1</th>
<th>MP3</th>
<th>MP5</th>
<th>MP7</th>
</tr>
</thead>
<tbody>
<tr>
<td>FHWA equation</td>
<td>146</td>
<td>121</td>
<td>133</td>
<td>156</td>
</tr>
<tr>
<td>Upper part: 0 m - 3.25 m</td>
<td>134</td>
<td>113</td>
<td>127</td>
<td>215</td>
</tr>
<tr>
<td>Lower part: 3.25 m - 5.5 m</td>
<td>178</td>
<td>144</td>
<td>156</td>
<td>84</td>
</tr>
<tr>
<td>Average</td>
<td>152</td>
<td>126</td>
<td>140</td>
<td>162</td>
</tr>
</tbody>
</table>

Figure 5.8 Skin friction distribution
5.6 Summary

Full-scale tension load tests were conducted on four micropiles in a firm to stiff clay. The micropiles consisted of Type BX76 geo-drilled anchors with 76 mm OD and 48 mm ID and with either a 178 mm or a 228 mm carbide bit threaded onto the bar to advance it down the hole using an air/water flushing technique.

The study indicated that, in the absence of plunging failure, Fuller and Hoy’s method provides a good estimation of the ultimate uplift capacity. The values proposed FHWA (2005) for type B micropiles underestimate bond strength for calculating the ultimate uplift capacity.

The increases in the micropile diameters were 10 % and 20 % for drill bit diameters of 228 mm and 178 mm, respectively. The average ultimate skin friction value was about 1.5 times the undrained shear strength for a micropile with a 228 mm drill bit and 1.6 times the undrained shear strength for a micropile with a 178 mm drill bit. The 228 mm drill bit resulted in marginally higher capacity micropile than the 178 mm drill bit, although a micropile with a 178 mm drill bit has greater bond strength. A final observation is that hollow bar micropiles are sensitive to the construction technique and the drill bit specifications.
5.7 References


Chapter 6

Monotonic Lateral Behaviour of Hollow Bar Micropiles

6.1 Introduction

Micropiles are employed in a variety of applications: underpinning for existing foundations, in situ reinforcement, and seismic retrofitting. Micropiles can sustain axial (compression and tension) and/or lateral loads. A type of micropile that has become more popular over the last ten years is the hollow bar micropile because it provides quick and efficient installation and results in significantly improved ground in the vicinity of the micropile. They can be placed in a one-step operation in which the hole is drilled and reinforced simultaneously. In contrast, the process for placing other types of micropiles requires multiple steps: drilling the hole, installing the casing, installing the steel reinforcement, and then placing the grout.

The lateral capacity of hollow bar micropiles is considered to be small owing to their small cross-sectional area. Due to the small diameter of hollow bar micropiles, their bending resistance must be high to accommodate the mobilization of a significant degree of relative soil-micropile movement. The lateral capacity is also small compared to the axial capacity; FHWA (1997 and 2005) therefore recommends the use of reinforcement at the upper portion of micropiles.

Richards and Rothbauer (2004) tested 20 micropiles: eight micropiles were installed in sand soil and 12 were installed in clay. All micropiles studied had steel casing filled with grout. The lateral load results were compared with responses calculated using the program LPILE (Ensoft, 2000), Foundations and Earth Structures, Design Manual (NAVFAC, 1986), and the characteristic load method developed by Duncan et al. (1994). The comparison revealed that the deflections measured in the field tests were less than the calculated values. The differences were attributed to the conservatively assigned soil parameters and the fact that the passive surcharge due to the top of the pile
being below the ground surface was neglected. In addition, they found that the micropile response is sensitive to the upper layers of soil.

Long et al. (2004) compared the results of 10 lateral load tests conducted on micropiles in clay with those calculated using the p-y method. The micropiles consisted of a steel casing filled with grout and reinforced with a central high-strength threaded bar along the entire length. The calculated responses were in good agreement with the field results, with a 10 % error margin. Thomson et al. (2007) conducted lateral load tests on four micropiles that had an external steel casing. They measured the lateral load deflection of the micropile head using a dial gauge, and the load deflection curve at different depths using inclinometers. The four micropiles displayed varying responses due to differences in the installation conditions, even though identical procedures were followed for all micropiles. The discrepancies were attributed to differences in the amount of grout used: in some cases, less than the theoretical volume of the hole, and in other cases, 1.8 times the volume of the hole. Micropiles in which large amounts of grout were used, generally exhibited stiffer behaviour than others in which smaller amounts were employed.

Abdelaziz (2012) conducted lateral load tests on hollow bar micropiles installed in cohesive soil: two micropiles were subjected to monotonic loading, while six micropiles were subjected to cyclic loading micropiles. He noted that the load-displacement curve was nonlinear from the start of loading, indicating that the piles were flexible. He also noted that the properties of soil along a depth equal to 10 times the pile diameter had a significant effect on the performance of hollow bar micropiles. He also conducted a parametric study to explore the effect of a steel casing on the lateral performance of the hollow bar micropile revealed that with proper reinforcement, micropiles can carry moderate lateral loads. The cyclic test results indicated a shakedown phenomenon, in which the micropile stiffness initially degraded with an increasing number of cycles and then became constant after reaching a number of cycles.

The above studies demonstrated clearly insufficient data related to hollow bar micropiles constructed in clay. Numerous factors that have a significant impact on micropile performance should be monitored during installation: type of grout, pressure during
grouting, and any increase in diameter during grouting. For these reasons, the research for this thesis included a field study of the performance of hollow core micropiles under monotonic lateral loads. The study was conducted on eight hollow core micropiles, each fitted with a 228 mm drill bit, and two with a 178 mm drill bit. This chapter presents the results of the full-scale load tests along with a discussion of the findings with respect to load-displacement curves, load-rotation curve, and crack behaviour. Using LPILE software, a numerical model was developed in order to study the effect of using different fibres with varying micropile head fixity conditions.

6.2 Site Conditions

The micropiles were installed at the Western University Environmental Site, located 8 km north of London, Ontario. Two boreholes, BH-I and BH-II located 8.4 m, were drilled within the micropiles test area as shown in Figure 6.1. Samples were extracted from each borehole by means of hollow stem auguring followed by a standard penetration test (SPT) and split spoon sampling. A monitoring well was installed in order to measure the groundwater level. After the fieldwork, laboratory testing was performed on the samples collected.

The soil profile as revealed by BH-I and BH-II is comprised of 200-300 mm top soil layer underlain by a thick layer of firm to stiff clay that extends to a depth of 5 m, followed by a soft to firm dark grey clay layer that appeared between 5 m and 6.5 m. At the bottom of the boreholes, there is very stiff dark grey lean clay with seams of sand that extends to a depth of 9 m. The ground water table appeared at level of 6.41 m. The undrained shear strength, $C_u$, of the soil were obtained from measured SPT values and SPT-undrained shear strength correlations proposed by Terzaghi and Peck (1967) for fine-grained soil and those by Stroud (1974) for insensitive clay. The undrained shear strength values of the soil profile are shown in Figure 6.2.
Figure 6.1 Borehole and pile distribution in the study area

a) $C_u$ versus depth for borehole I

b) $C_u$ versus depth for borehole II

Figure 6.2 The undrained shear strength, $C_u$, profile
6.3 Micropile Installation

Eight hollow bar micropiles were installed: six with a 228 mm drill bit diameter (MP1 to MP6) and two with a 178 mm drill bit diameter (MP7 and MP8). They were placed in two rows: four micropiles in each row spaced at 4 m centre-to-centre, with the rows being 3 m apart (Figure 6.1). The drill bits configurations are shown in Figure 6.3. All micropiles were installed using Type BX76 geo-drilled anchors (76 mm OD and 48 mm ID) and had a bond length of 5.75 m. The installation procedure involved rotary percussive drilling with air and water flushing followed by installation of first segment hollow bar segment (3 m long) then a second segment using a B7X2-76 coupler. After the desired depth (5.75 m) was reached, grout (Type 10 Portland cement mixed at water-to-cement ratio = 0.45) was introduced through the hollow bar. Grout was used to flush debris from the bar and the hole. In some cases, water was used to clean debris from the hollow bar before flushing with the grout. A shallow layer 2 m to 3 m thick of backfill was encountered during drilling of MP3 and MP4.

![228 mm (9 in) diameter drill bit](image1)
![178 mm (7 in) diameter drill bit](image2)

a) 228 mm (9 in) diameter drill bit  b) 178 mm (7 in) diameter drill bit

Figure 6.3 228 mm (9 in) and 178 mm (7 in) drill bits
Based on the installation records and grout quantities used, the size of the holes increased by a factor of 1.1 times the drill bit diameter for the 228 mm drill bits and by a factor of 1.2 for the 178 mm drill bits, which agrees with Maclean’s (2010) observations in which the diameters of the micropiles enlarged. The difference in the enlargement ratio is attributed to the differences in the back-pressure used during grouting because of the difference in nozzles size (the nozzles in the 178 mm drill bit were smaller than the nozzles in the 228 mm drill bit). The average quantity of cement bags per hole was 280-320 kg for the 178 mm drill bits and 360-400 kg for the 228 mm drill bits. These quantities correspond to 0.215 m$^3$ and 0.276 m$^3$ of grout for the 178 mm and 228 mm drill bits, respectively.

The 28-day grout compressive strength was 41.4 MPa and conforms to the limit specified by FHWA (2005). Its average 28-day split tensile strength and flexural tensile strength were 4.2 MPa and 5.2 MPa, respectively. The average modulus of elasticity of four grout samples tested was 15.05 GPa. These values are within the range of values suggested by U.S. Army Corps of Engineers (1984) for grout flexural strength (10-15 % of compressive strength) and elastic modulus (approximately 50% of concrete elastic modulus at the same strength level).

6.4 Test Setup, Method and Instrumentation

Eight micropiles were tested after 8 weeks after installation under lateral monotonic loads. Three micropiles (MP2, MP4, MP6) were tested laterally after at least 10 days of axial testing, two (MP1 and MP8) were tested laterally after one week of axial testing, and three (MP3, MP5 and MP7) were tested laterally after at least 3 days of axial testing. All micropiles were loaded until lateral load could not be maintained. According to ASTM Subcommittee (1970), the waiting time for testing piles laterally should be at least 3 days after testing them both vertically and horizontally.
6.4.1 Testing equipment and pile instrumentation

The loading system consisted of a loading plate that was pinned to a steel rod at one end and threaded to load cell at the other end, as shown in Figure 6.4. The load was applied through a hollow cylinder hydraulic jack connected to a hydraulic pump and clamped between two plates: one connected to the load cell through a threaded bar, and the other resting on a reaction beam. The hydraulic jack was pushing against the micropile and the reaction beam that was supported by the excavator. The dimensions of the loading plate were 300 mm X 300 mm X 38 mm. The hydraulic jack had 1100 kN advance capacity and maximum stroke of 150 mm. The load was recorded through a load cell whose capacity was 900 kN. Two HLP 190 linear potentiometers were attached to the loading plate to measure the vertical displacement. The linear potentiometers had 100 mm stroke with accuracy of 0.01 mm. Attached to the loading plate to measure rotation was an angle finder with an accuracy of 0.3°. The linear potentiometers and the angle finder were centred to be in a plane parallel to the plane of the applied load. Figure 6.5 shows the pile head instrumentation; as indicated, the micropile head is pinned (i.e. free to rotate).

![Figure 6.4 Lateral test setup](image-url)
6.4.2 Test procedures

ASTM D3966, (2007) standard loading test produces specify an increment in the applied load of 25% of the design load, with varied time intervals starting with 10 min and ending with 60 min at 200% of the design load. However, the specifications allow the engineer to modify the test. A quick maintained load test, which was implemented on the eight micropiles to which the load was applied in increments of 5 kN, with each increment being maintained for 3 min to 4 min. The load was increased until it could no longer be maintained, and a substantial increase in the rate of displacement occurred.

6.5 Load-Displacement Curves

The micropiles tested were divided into two groups. The first group included micropiles that were tested after at least 7 days of axial loading (MP2, MP4, MP6 and MP1 and MP8), while the second group included the micropiles tested laterally after 3 days of axial testing (MP3, MP5 and MP7). The load-displacement curves for the two groups are shown in Figures 6.6 and 6.7. The figures indicate that both groups display generally
three regions of the load-displacement curves: an initial linear response region from start of loading until displacement of 6-7 mm was reached; a nonlinear response from end of the linear region up to displacement of 25-30 mm; and a final linear response region that extended to the maximum displacement, at which point the load could not be maintained. The tests were terminated at 20 % to 23 % of the micropile shaft diameter. Upon unloading, the piles retrieved up to 70 % of the displacement. This general behaviour is accentuated in Figure 6.8, with the trend line (envelop) curve plotted. The initial linear response represented the resistance of the micropile body with a contribution from the surrounding soil within its linear range, followed by the nonlinear behaviour, which represented the resistance of the surrounding soil progressing towards plastic behaviour. Finally, when full slippage occurred along the side the micropile body and full plastic behaviour of bearing soil, linear behaviour was observed (i.e. constant resistance of the soil).

Figure 6.6 Load-displacement curves for MP1, MP2, MP4, MP6, and MP8
Figure 6.7 Load-displacement curves for MP3, MP5, and MP7

Figure 6.8 Load-displacement curve and envelope load-displacement curve for MP1
The observed general trend of displacement can be explained by noting two deformation mechanisms that took place during the loading: global mechanism, and local mechanism. The global deformation mechanism involves both the hollow bar and grout as one body (i.e. hollow bar and grout moved together due to the load applied at the micropile head), while the local deformation mechanism was associated with the cracking of the grout body. A detailed description for both mechanisms is described below.

The first mechanism involves global behaviour, whereby applying the load on the micropile head causes displacement of the micropile shaft (hollow bar and grout) and, consequently, the surrounding soil. As the lateral displacement increased, this mechanism concluded with the micropile shaft separating from the soil behind the point of the applied load resulting in a gap that started along the centreline of the micropile, and propagated along the circumference of the micropile as the applied load increased (see Figure 6.9a). As the loading continued after this point, the second (local) mechanism started, which was characterized by radial cracks starting from the hollow bar and extending towards the outer surface of the grout as shown in Figure 6.9b. The degree of cracking varied with the strength of the bond between the grout and the hollow bar. El Sharnouby and El Naggar (2011) made similar observations on the lateral performance of helical pull down micropiles.

Comparing the performance of the two micropile groups (shown in Figures 6.6 and 6.7), it is noted that the first group displayed, in general, stiffer response especially in the initial linear region. Also, the second group displayed the nonlinear behaviour sooner (initial linear region extended to only 3-4 mm displacement) and continued to higher displacements (45-45 mm) and the final linear region extended to larger displacement (26-30% of the shaft diameter). This is attributed to the fact that the lateral loading was conducted only three days after completing the axial load testing, which didn’t allow enough time for the soil to regain strength after the axial loading.

Interestingly, MP4 exhibited the stiffest response behaviour, even though it was installed in a backfill layer. However, the direction of loading was pushing against the native stiff clay soil. On the other hand, MP8 showed the softest response due to the disturbed soil in
its vicinity during the preparation for the test. The softening behaviour was in the initial loading stages only. These observations indicated the sensitivity of micropiles lateral response to the properties of soil in the top most soil layer.

6.6 Failure Mechanism and Interpreted Failure Criteria

Micropiles are classified as flexible piles (long piles) due to their small diameter; i.e., the slenderness ratio is very high. The ultimate lateral resistance for long (flexible) piles is defined as the load that propagates a moment equal to the yielding moment of the pile. In contrast, the ultimate lateral resistance for short (rigid) piles is the load that causes failure in the soil mass along the pile shaft. There are several methods of categorizing piles as either rigid or flexible piles. Kasch et al. (1977) defined the pile rigidity using its embedded length-to-diameter ratio, L/d. For flexible piles, L/d > 20 and for rigid piles, L/d < 6. Poulos and Davis (1980) proposed a stiffness ratio K_r as a measure of the pile rigidity, which is given by:
where $E_p = \text{the pile elastic modulus}$, $I_p = \text{moment of inertia of the pile cross-section}$, $E_{\text{soil}} = \text{soil modulus}$, and $L = \text{pile length}$. If the stiffness ratio is less than $10^{-5}$, flexible pile behaviour can be expected, while the pile will be rigid if the stiffness ratio is larger than 0.01. The micropiles used in this study definitely behaved as long flexible piles.

Even though the ultimate lateral resistance is computed based on the ultimate limit state, the serviceability limit state controls the allowable design. Chen and Lee (2010) listed 11 interpretation criteria associated with the lateral capacity of deep foundation based on an acceptable lateral deflection (serviceability) or rotation at the pile head. These methods are based on practical experience and are defined through pile head tests. Seven of these criteria were considered in this study. Tables 6.1 and 6.2 present the methods based on pile head lateral displacements and their corresponding capacity values. The methods were classified based three criteria: the ultimate capacity represented by displacement limits, as defined by McNulty (1956), Walker and Cox (1966), and New York City (1981); and the ultimate capacity defined by the displacement limit as a function of shaft diameter, as reported by Broms (1964), Pyke (1984), and Briaud (1984). Table 6.3 presents the methods based on rotation limits at the pile head, as described Davidson et al. (1982) and the corresponding capacity evaluated considering the micropiles rotations presented in Figure 6.10. The ultimate lateral load capacity is determined in this study as the load corresponding 25 mm lateral displacement at the pile head. According to Chen & Lee (2010), a safety factor of 2.0 can be implemented for drilled shaft design if a load test is performed.
Figure 6.10 Micropile head rotations evaluated from the load tests

Table 6.1 Ultimate lateral capacity: specified deflection limits

<table>
<thead>
<tr>
<th>Method</th>
<th>Pile</th>
<th>MP2</th>
<th>MP4</th>
<th>MP6</th>
<th>MP8</th>
<th>MP1</th>
<th>MP3</th>
<th>MP5</th>
<th>MP7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load at 6.25 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(McNulty, 1956)</td>
<td></td>
<td>9.5</td>
<td>10</td>
<td>7.5</td>
<td>6</td>
<td>10</td>
<td>9.5</td>
<td>8.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Load at 13.0 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Walker &amp; Cox, 1966)</td>
<td></td>
<td>14</td>
<td>20</td>
<td>15</td>
<td>10</td>
<td>17</td>
<td>15</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Load at 25.0 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(New York City, 1981)</td>
<td></td>
<td>20</td>
<td>30</td>
<td>25</td>
<td>20</td>
<td>22.5</td>
<td>24</td>
<td>17</td>
<td>16</td>
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</tbody>
</table>
Table 6.2 Ultimate lateral capacity: deflection limits as a ratio of pile diameter

<table>
<thead>
<tr>
<th>Method</th>
<th>Pile</th>
<th>MP2</th>
<th>MP4</th>
<th>MP6</th>
<th>MP8</th>
<th>MP1</th>
<th>MP3</th>
<th>MP5</th>
<th>MP7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load at 5% D mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Pyke, 1984)</td>
<td></td>
<td>14</td>
<td>20</td>
<td>15</td>
<td>8</td>
<td>17</td>
<td>15</td>
<td>10</td>
<td>8.5</td>
</tr>
<tr>
<td>Load at 10% D mm</td>
<td></td>
<td>20</td>
<td>30</td>
<td>25</td>
<td>16</td>
<td>22.5</td>
<td>24</td>
<td>17</td>
<td>15</td>
</tr>
<tr>
<td>(Briaud, 1984)</td>
<td></td>
<td>34</td>
<td>51</td>
<td>38</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>Load at 20% D mm</td>
<td></td>
<td>34</td>
<td>51</td>
<td>38</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>(Broms, 1964)</td>
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<td></td>
<td></td>
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</tr>
</tbody>
</table>

Table 6.3 Lateral ultimate capacity: specified rotation limits

<table>
<thead>
<tr>
<th>Method</th>
<th>Pile</th>
<th>MP2</th>
<th>MP4</th>
<th>MP6</th>
<th>MP8</th>
<th>MP1</th>
<th>MP3</th>
<th>MP5</th>
<th>MP7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load at 2° head slope</td>
<td></td>
<td>23</td>
<td>30</td>
<td>23</td>
<td>22</td>
<td>25</td>
<td>22</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>(Davidson et al., 1982)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.7 LPILE Analysis and Results

The LPILE computer program (Ensoft, 2006) was used to analyze the lateral response of the micropiles. LPILE is software for analysing the behaviour of piles subjected to lateral loads using the p-y method. The p-y method assumes pile as a beam-column with representing the soil by nonlinear Winkler-type springs. The behaviour of the beam-column is represented by differential equations. The program calculates deflection, bending moment, and shear force and soil response along the pile length using the solution of the differential equations. The program can be used for varies types of soils, pile head conditions, dimensions of piles, and material proprieties that could be vary with the pile length (Ensoft, 2006). However, LPILE software does not offer a built-in section incorporating a hollow bar embedded in a round grout shaft. A round shaft with a permanent casing and hollow core was therefore used but with a modification that simulated a hollow bar micropile: the thickness of the permanent casing was set to zero.
The shaft diameter was set as 1.1 times the diameter of the drill bit; the steel bar had an outer diameter of 76 mm and an inner diameter of 48 mm. The elastic modulus of steel was set at 200,000 GPa, and the grout elastic modulus was set at 15.05 GPa based on results of laboratory tests.

The soil was modeled in LPILE as moderate stiff clay without free water because the tests were conducted at a high loading rate. To enhance the accuracy of the results, the topsoil was divided into sub-layers. The thickness of each layer was double the cross section of the hollow bar micropiles. The parameters required for LPILE modeling include the effective unit weight of the clay, the undrained shear strength parameter for the clay, the strain corresponding to a shear stress level equal to one-half of the shear strength of the material, $\varepsilon_{50}$, and the soil modulus parameter, $k$. The effective unit weight and undrained shear strength parameters that were utilized in the program were derived from the soil investigation. The soil modulus parameter can be calculated as follows (Rodrigo, 2008):

$$ K = \frac{9C_u}{5\varepsilon_{50}B} $$

(6.2)

Tables 6.4 and 6.5 present the recommended values of $\varepsilon_{50}$ and $K$.

**Table 6.4 $\varepsilon_{50}$ values for the clay**

<table>
<thead>
<tr>
<th>Undrained shear strength</th>
<th>$\varepsilon_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>50-100</td>
<td>0.007</td>
</tr>
<tr>
<td>100-200</td>
<td>0.005</td>
</tr>
<tr>
<td>300-400</td>
<td>0.004</td>
</tr>
</tbody>
</table>
Table 6.5 k values for the clay

<table>
<thead>
<tr>
<th>Undrained shear strength</th>
<th>K - static kPa/m</th>
<th>K – cyclic kPa/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-24</td>
<td>8,140</td>
<td>-</td>
</tr>
<tr>
<td>24-48</td>
<td>27,150</td>
<td>-</td>
</tr>
<tr>
<td>48-96</td>
<td>136,000</td>
<td>54,300</td>
</tr>
<tr>
<td>96-192</td>
<td>271,000</td>
<td>108,500</td>
</tr>
<tr>
<td>192-283</td>
<td>543,000</td>
<td>217,000</td>
</tr>
</tbody>
</table>

The LPILE model was calibrated by comparing its predictions with the results of the lateral load field test of MP1 and good match was observed between the calculated and measured responses as shown in Figure 6.11a. Then, the model was verified by comparing its predictions with the results of MP2, and MP6 (i.e. micropiles with 228 mm drill bit) as shown in Figure 6.11a. For further verification, the model was modified to account for the small diameter of MP7 and MP8, which were constructed with 178 mm drill bit. Figure 6.11b compares the calculated and measured responses of MP7 and MP8, which indicate good agreement.

Figure 6.11 Compassion between LPILE results and field test results
After verifying the model, the deflection, moment and shear force profiles obtained from the LPILE analysis of the lateral response of MP1 are plotted in Figure 6.12. The results are presented for graduating load equal to the micropile ultimate capacity of 25 kN, which produced lateral displacement at the pile head of 25 mm. Figure 6.12 indicates that the micropile behaves as a flexible pile. The deflection diagram of the micropile shaft (Figure 6.12a) shows that the top 2.0 m (8-10 D) of the micropile experienced deflections developed, while the lower part didn’t experience any deflection. This implies that the properties of soil along the top 8-10D would have a significant impact on the lateral response of the micropile, and the soil below that depth has little to no effect on micropile lateral behaviour.

**Figure 6.12 Deflection, shear force, and bending moment along MP1 shaft, (LPILE, 2006)**
6.8 Parametric Study

The main objectives of the parametric study are: to examine an innovative grout mix to improve the lateral load capacity of hollow bar micropiles without changing the construction process; and to investigate the effect of head fixity on the behaviour of hollow bar micropiles. To achieve the first objective, different grout-fibres mixes were prepared and cylindrical specimens were tested in order to establish their strength and stiffness properties. These properties are considered along with the verified LPILE model. The second objective complements a parallel study being undertaken at Western, which examines connectivity of micropiles and helical pull-down micropiles to concrete foundations (Diab, 2013). This part of the study was conducted on micropile with 228 mm drill bit as they show higher capacity than those with 178 mm drill bit.

6.8.1 Mechanical properties of fibres

Three different types of fibres were considered for improving the lateral performance and capacity of the hollow bar micropiles under lateral loading, including: plastic fibres (Fibermesh® 650 Synthetic Fibre); basalt fibres (MiniBar); and steel fibres (copper coated micro steel fibre). The lengths of the fibres used were graded for Fibermesh 650, 20 mm for the minibar, and 12 mm for the micro steel fibre. The unit weight of the fibres was 0.9 kN/m$^3$ for the Fibermesh, 1.8 kN/m$^3$ for the MiniBar, and 78.5 kN/m$^3$ for the micro steel fibre. Figure 6.13 shows the fibres used in this study. All grout fibre mixes included fibres dosage of 1 % of grout volume and mixed at water-cement ratio = 0.45.

Three cylindrical samples were tested under compression to determine compressive strength according to ASTM C39 (see Figure 6.14a), and three samples were tested under split tension to determine tensile strength according to ASTM C496 (see Figure 6.14b). In addition, the elastic modulus was obtained for each mix according to ASTM C469.

The mechanical properties are summarized in Table 6.6. The small values of coefficients of variation (COVs) are within the standard limits. However, the COVs for compressive strength are very small compared to COVs for the tensile strength because the fibres affected the tensile strength, but had a small effect on the compressive strength. The COV
for the grout mixed with the micro steel fibre was the smallest, which indicates that the micro steel fibre provides superior distribution along the section. Figures 6.15 and 6.16 shows the test specimens after testing. Figure 6.17 indicates the distribution of fibres in the cross-sectional area for different grout mixes.

Table 6.6 Mechanical properties of grout mixed with fibres

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive Strength MPa (COV)</th>
<th>Split Tension Test MPa (COV)</th>
<th>Modulus of Elasticity MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibermesh 650 + grout</td>
<td>43.73 (2.34)</td>
<td>5.45 (11.78)</td>
<td>20,350</td>
</tr>
<tr>
<td>MiniBar + grout</td>
<td>43.34 (2.82)</td>
<td>5.90 (11.34)</td>
<td>19,950</td>
</tr>
<tr>
<td>Micro steel fibre + grout</td>
<td>53.8 (2.36)</td>
<td>8.45 (4.18)</td>
<td>23,500</td>
</tr>
</tbody>
</table>

Figure 6.13 Different types of Fibres used in this study
Figure 6.14 Compression and tensile tests

a) Compression test  
b) Tensile strength (split tension test)
a) Normal grout

b) Grout + MiniBar 20 mm
c) Grout + Micro Steel fibre 12 mm

Figure 6.15 Cylinders after compression tests
Figure 6.16 Cylinders after tensile tests

a) Normal grout

b) Grout + Fibermesh 650

c) Grout + MiniBar 20mm

d) Grout + Micro steel fibre
Figure 6.17 Fibres distributions
6.8.2 Results and discussion

The mechanical properties of the different grout mixes were used in the calibrated LPILE model and the analyses were conducted to calculate the lateral deflection of hollow bar micropiles with different mixes. The load at lateral displacement of 25 mm at the micropile head was considered to evaluate the increase in the pile capacity. The analysis was conducted assuming free head conditions (i.e. allowing pile head rotation) and the results are shown in Figure 6.18. It can be noted from Figure 6.18 that for free head micropiles, the Fibermesh 650 has and the MiniBar 20 mm fibres improved the lateral capacity of the hollow bar micropiles by about 15%, while using the micro steel fibres increased the lateral capacity by 20%. It is also noted from Figure 6.18 that the load displacement curve for the micropile with neat grout (no fibres) displays a plateau at a load of approximately 25 kN (displacement = 25 mm), i.e., failure occurs. However, micropiles with fibres, especially micro steel fibres, continued to resist more load, implying the performance at higher displacement levels improves significantly. For example, the micropile with micro steel fibres showed almost 50% increase in lateral load resistance at 50 mm displacement. Also, the micro steel fibres increased the maximum bending moment sustained by the micropiles by more than 30%. These results demonstrate the favourable effect of the fibres on the micropile ductility.

In a parallel study at Western, a steel pile cap is developed to connect slender shaft piles (e.g. micropiles and helical pull-down micropiles) to concrete foundations. This steel pile cap ensures the fixity of the slender shaft piles into the concrete foundations. The behaviour of hollow bar micropiles with different grout mixes was evaluated considering fixed head conditions. The results obtained for the fixed head case are shown in Figure 6.19. As expected, Figure 6.19 indicates that for fixed head condition, the micropile shows superior performance compared to the free head condition. The lateral capacity increased by 80% in the case of grout only, and increased by 140% in the case of grout with the micro steel fibres. In addition, Figure 6.19 shows increased lateral resistance at higher displacement and improved ductility using fibres. Similar to the free head case, the
micro steel fibres resulted in the highest improvement in ultimate lateral capacity, bending moment resistance and ductility.

Figure 6.18 Load, and moment versus horizontal displacement (Free head condition)

Figure 6.19 Load, and moment versus horizontal displacement (Fixed head condition)
6.9 Summary

Full-scale lateral load tests were conducted on eight micropiles in a firm to stiff clay. The micropiles consisted of Type BX76 geo-drilled anchors with 76 mm OD and 48 mm ID and a 178 mm or 228 mm carbide bit threaded onto the bar to advance it down the hole using an air/water flushing technique. Based on experimental observations and numerical modeling, a number of conclusions may be drawn.

1. Micropiles with a 228 mm drill bit performed marginally better than micropiles with a 178 mm drill bit. The smaller size of the 178 mm drill bit nozzles resulted in a significant increase in the back pressure during grouting and consequently improved the micropile performance, which compensated for the smaller diameter drill bit.

2. Two failure mechanisms were observed during testing. First, a global mechanism starts with a crack at the interface between the micropile shaft and the soil, behind the point of the applied load and then propagating to a point of separation that begins along the centreline of the micropile. A local mechanism creates a radial crack that starts from the hollow core bar and extends toward the outer surface of the grout.

3. Micropiles behave as long piles due to their high slenderness ratio. The behaviour of hollow bar micropiles is sensitive to properties of soil along the top 8 to 10 times the micropile diameter.

4. Fibres, especially steel micro fibres, can enhance the lateral performance, capacity and ductility of hollow bar micropiles. Using steel micro fibre increased the lateral resistance by 20 %. Ensuring a fixed head condition increased the lateral resistance by 80 % for micropiles with normal grout, which rose to 140 % in the case of steel micro fibre with grout and a fixed head.
6.10 References


Diab, Muhammad, 2013. *Behaviour of helical pull-down micropile connectors for new foundations (PhD. Thesis under preparation)*, Western University (Personal communication)


Chapter 7

Summary, Conclusion, and Future Recommendations

7.1 Summary

The main objective of the research presented in this thesis was to investigate the behaviour of fully instrumented hollow bar micropiles that were constructed using two different sizes of drill bits. The micropiles were subjected to axial monotonic and cyclic loads along with lateral monotonic loading in cohesive soils. Eight hollow core micropiles were installed: six with a 228 mm drill bit diameter (MP1 to MP6) and two with a 178 mm drill bit diameter (MP7 and MP8).

The testing of monotonic axial compression program comprised two stages. First, four micropiles (MP2, MP4, MP6, and MP8) were loaded to the point of failure. The other four micropiles (MP1, MP3, MP5, and MP7) were loaded up to 133% of the design load. The cyclic loading involved quasi-static cyclic load tests on four micropiles (MP1, MP3, MP5, and MP7): three micropiles with a 228 mm (9 in) nominal diameter drill bit and one micropile with a 178 mm (7 in) nominal diameter drill bit.

The monotonic axial uplift tension tests were conducted on four micropiles: three micropiles with 228 mm (9 in) nominal diameter drill bits, and one micropile with a 178 mm (7 in) nominal diameter drill bit. All micropiles (MP1, MP3, MP5, and MP7) were loaded to the point of failure.

Finally, eight micropiles were tested under lateral monotonic loads: six micropiles with a nominal drill bit diameter of 228 mm (9 in), and two with a 178 mm (7 in) nominal drill bit diameter. All micropiles were loaded until lateral displacement reached the point where the lateral load could not be maintained. A numerical model was developed and calibrated/verified using the experimental results. It was then used to conduct a parametric study in order to get better understanding of the lateral behaviour of micropiles. In addition, the parametric study explored the potential benefit of using
innovative grout mix to improve the lateral load capacity of hollow bar micropiles without changing the construction process; and to investigate the effect of head fixity on the behaviour of hollow bar micropiles.

7.2 Conclusions

7.2.1 Monotonic and cyclic axial compression loading results

The experimental results on the axial and cyclic performance and its interpretation revealed the followings:

- In the absence of plunging failure, Fuller and Hoy’s method provides a good estimation of the ultimate capacity.
- The bond strength values proposed by FHWA (2005) for type B micropiles underestimate the ultimate capacity.
- The increase in the diameter of the micropiles ranged from 10% to 20% with drill bit diameters of 228 mm and 178 mm, respectively.
- The ultimate capacity of micropiles installed in stiff clay was mobilized at a head displacement of 5% of the micropile diameter. For micropiles resting on medium dense/dense sand, the ultimate capacity was fully mobilized at head displacement equal to 10% of the micropile diameter.
- The average ultimate skin friction was about 1 to 1.25 times the undrained shear strength for micropiles with a 228 mm drill bit and 1.6 times the undrained shear strength for micropiles with a 178 mm drill bit.
- The 228 mm drill bit performed marginally better than the 178 mm drill bit, but micropiles with a 178 mm drill bit exhibited higher bond strength.
- Hollow bar micropiles are sensitive to the construction technique and the drill bit specifications.
- No stiffness degradation was observed during cyclic loading.
7.2.2 Monotonic axial uplift loading results

The experimental results on the axial tension performance and its interpretation revealed the followings:

- Fuller and Hoy’s method provides a good estimation of the ultimate capacity in the absence of plunging failure.
- The bond strength values proposed by FHWA (2005) for type B micropiles underestimate bond strength for calculating the ultimate capacity.
- The average ultimate skin friction value was about 1.5 times the undrained shear strength for a micropile with a 228 mm drill bit and 1.6 times the undrained shear strength for a micropile with a 178 mm drill bit.
- The 228 mm drill bit performed marginally better than the 178 mm drill bit, although a micropile with a 178 mm drill bit has greater bond strength.
- A final observation is that hollow core micropiles are sensitive to the construction technique and the drill bit specifications.

7.2.3 Monotonic lateral loading results

The experimental results on the axial tension performance and its interpretation revealed the followings:

- Micropiles with a 228 mm drill bit performed better than micropiles with a 178 mm drill bit.
- Two deformation mechanisms were observed during testing. First, a global mechanism starts with the micropile shaft separating from the soil, starting behind the point of the applied load and then propagating along the circumference of the micropile. A local mechanism creates a radial crack that starts from the hollow bar and extends towards the outer surface of the grout.
- Micropiles behave as long piles due to their slenderness.
- Fibres, especially steel micro fibres, increased the lateral capacity of hollow core micropiles. Using steel micro fibre increased the lateral resistance by 20%.
Incorporating a fixed head by rigidly connecting the micropile head to the cap can increase the lateral resistance by 80 %, which rose to 140 % in the case of steel micro fibre with grout.

### 7.3 Future Recommendations

The current research revealed that some further studies on hollow bar micropiles may be needed. The following are recommendations for future research:

- Investigate the behaviour of hollow bar micropiles using the same drill bits (228 mm, and 178 mm), which were used in the study, while maintaining the same nozzle size and pressure at (200 psi).
- Investigate the behaviour of hollow bar micropile using concrete surrounding the hollow core bar instead of grout. The construction technique of this can be explained as filling the hole with aggregate finer than 25 mm and then grouting using cement/sand grout from the bottom of the hole. This can enhance the axial and lateral capacities of hollow core micropiles.
- Enhancing the lateral capacity of hollow bar micropile by using a grout mixed with micro steel fibre at different percentage (not less than 1 % of the grout volume) with maintaining the water-cement ratio used at 0.4-0.45.
Figure A.1 Locations of borehole tests conducted by Atkinson Davies Inc in 2008
Figure A.2 Borehole 1 soil log by Atkinson Davies Inc in 2008
**Figure A.3 Borehole 2 soil log by Atkinson Davies Inc in 2008**
**Figure A.4 Borehole 3 soil log by Atkinson Davies Inc in 2008**
Figure A.5 Borehole 4 soil log by Atkinson Davies Inc in 2008
Figure A.6 Borehole 5 soil log by Atkinson Davies Inc in 2008
**Figure A.7 Borehole 6 soil log by Atkinson Davies Inc in 2008**

**Table: Subsurface Profile**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
<th>Symbol</th>
<th>Number</th>
<th>Type</th>
<th>Plastic Limit %</th>
<th>Unconfined Shear Strength kPa</th>
<th>Natural Water %</th>
<th>Liquid Limit %</th>
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<tr>
<td>0.16</td>
<td>Topsoil</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>Very stiff, brown becoming grey at 3.5m depth, clayey SILT to silty CLAY IR.</td>
<td></td>
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<td></td>
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<tr>
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*End of Borehole. Hole dry at completion.*
Figure A.8 Borehole 7 soil log by Atkinson Davies Inc in 2008
Figure A.9 Borehole 8 soil log by Atkinson Davies Inc in 2008
Figure A.10 Borehole 9 soil log by Atkinson Davies Inc in 2008
Figure A.11 Grain size distribution using laser diffraction at a depth of 1.00 m

Figure A.12 Grain size distribution using laser diffraction at a depth of 3.25 m

Figure A.13 Grain size distribution using laser diffraction at a depth of 4.00 m
Figure A.14 Grain size distribution using laser diffraction at a depth of 4.75 m

Figure A.15 Grain size distribution using laser diffraction at a depth of 7.00 m

Figure A.16 Grain size distribution using laser diffraction at a depth of 7.75 m
### Curriculum Vitae

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