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Monotonic Axial Behaviour of Hollow Core Micropiles

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MONOTONIC AXIAL BEHAVIOUR OF HOLLOW CORE MICROPILES

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ABSTRACT

The use of hollow bars for micropiles has greatly increased over the past ten years. Hollow bar construction -sometimes called “self drilled”- is becoming a popular option because it addresses some installation challenges such as limited spaces and difficult access. This paper presents a field study on the behaviour of single hollow core micropiles in stiff silty clay deposits subjected to monotonic axial loading. Four hollow core micropiles were tested: two in compression and two in tension. All the micropiles were installed using air flushing technique employing specially fabricated large drilling carbide bits. The hollow core micropiles were instrumented using embedded strain gauges installed at the annulus between the mono hollow core all thread rebar and the surface of interface between the grout and the ground. The results of the full-scale loading test performed on the micropiles are presented and analyzed in terms of load displacement curves. Also, the effect of air flushing technique on the nominal grout/ground bond is examined. The results clearly show that the bond values (α_{bond}) suggested by the FHWA implantations manual (2000) is grossly underestimated when considering the hollow core micropile as type B. In addition, the results of the full scale load tests in both tension and compression suggest the potential for using the whole length of the micropiles as a bond zone without the need to install a permanent casing when the design criteria is governed by monotonic axial loading.

1. INTRODUCTION

A micropile is a small diameter (typically less than 300 mm) drilled and grouted pile that is typically reinforced (FHWA 2000). A micropile is constructed by drilling a borehole, placing a steel reinforcing element into the borehole and grouting the borehole by gravity, under pressure methods or by a combination of both (post grouting). Micropiles are advantageous because they can be installed in limited head room areas using small drilling equipment at any angle causing minimal disturbance to the ground and adjacent structures and provide a high grout to soil bond strength. Special drilling equipment is often used to install the micropiles within existing facilities. Micropiles depend on high capacity steel elements to transmit the loads from the super structure to the grout. As well, this steel element transmits the loads to the surrounding soil thought the grout body encapsulating them. The micropile grout to soil bond, known in litterateur as grout/ground bond, transfers the applied load to the surrounding soil through friction.

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The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the interface between the grout and the surrounding soil. Historically, micropiles have been innovative mainly to be used for retrofits and underpinning of structures. The first generation of micropiles were conceived in Italy by Dr. Fernando Lizzie in the 1950's in response to the requirement for the underpinning of historic buildings where access for conventional piling equipment was not possible. The *palo radice* "root pile" was developed and consisted of a small diameter, drilled and grouted reinforced pile. The second generation of micropiles was developed in the 1970's, which were installed by using either an open or cased hole drilling method. That micropile is introduced in North America by 1973 and known as "GEWI-Pile". The GEWI-Pile is typically a pressure grouted pile of small diameter with a central mono all thread bar which is encapsulated in a cement grout body. The huge numbers of projects done successfully using the mono bar system and the demands from contractors encouraged the suppliers of these bars, such as Williams Form and Dywidag, to manufacture large diameter mono bars that nowadays reach 89 mm in diameter.

A new generation of micropiles was devised by Ernst Ischebeck in 1983; and named The Titan Injection Bore (IBO) micropile. A continuously all threaded hollow steel bar is used as the drill steel that can be drilled and grouted simultaneously without the need of a casing during drilling. A sacrificial bit that contains openings that allows for pressure grouting of the surrounding soil is threaded onto the end of the hollow bar and left in place following drilling. The system has historically been known as a "self-drilling anchoring" because the hollow fully-threaded bar serves as both the drill string and the grouted anchor, thus installation is performed in a single operation (William Form – Ground Anchor system 2010). The drilling fluid (air, water, or grout) is introduced through the hollow bar and allows the spoils to flush from the borehole. This also improves the density and support capability of the surrounding soil.

The use of hollow bars for micropiles has greatly increased over the past 10 years. Hollow bar construction became a technique preferred by many contractors in the pilling industry. The success of using hollow bar micropiles in many projects has changed the question about hollow bar micropile from; "what can hollow bar micropiles do?" To "what couldn't hollow bar micropiles do?"

Despite the growing demand on hollow core bar micropiles, little work has been devoted to evaluating the nominal bond strength, (α_{bond}), between the micropile grout and the surrounding soil. In all projects involving this micropile, either called Self-drilling, or Injection Bore (IBO), or Geo-Drill bar, it is classified as Type B grouting according to the (FHWA 2000). Based on many field load tests, the bond strength values suggested by the FHWA (2000) for Type B seem to be conservative for most soil deposits (Gomez et al., 2007; Mitchell et al., 2007). The deviation of the bond strength of Type B and that of hollow core micropiles may be because the classification (which was initially for normal pressure grouting micropiles) didn't take into account the different factors affecting the hollow core micropiles. These factors include: the type of fluid used during installation; the speed and pressure used during installation and during grouting (dynamic grouting); and the effect of all of that on the surrounding ground. The data available in the literature indicates that the dynamic process involved during installing and grouting the micropile could change the phrase grout/ground bond to be grout/improved ground bond, for this type of micropiles. There are many questions regarding the response of micropiles that have not been answered, specially, for micropiles bonded into clayey soils and into rock. Only a few data on the development of grout/soil bond stresses during loading is available. Therefore, there is a need to better understand the performance characteristics of this micropile type with regard to the grout/ground bond, which should be classified as a new grouting category to be added to the four present categories (A, B, C, and D) (FHWA 2000).

A field study on the performance of hollow core micropiles in cohesive soils is presented here. The aim of this study is to evaluate the geotechnical performance of micropiles under monotonic axial load, with a special emphasis on obtaining representative bond strength values between the grout and the surrounding soil. The field study is a part of a comprehensive investigation of the performance of hollow core micropiles under different types of load and in various soil conditions.

2. TEST SITE CONDITIONS

The piles were installed and tested at the University of Western Ontario Environmental Site. This site is located approximately 8 km north of the City of London, Ontario, on a ten hectare parcel of land. The ground surface is very flat and is roughly 200 meters above sea level. The site was selected mainly because of its cohesive nature determined from previously tested piles.

Two boreholes were conducted in October 2009 as part of the current study, within the area where the piles were installed and load tested. The two boreholes are located 16.6 meters apart and both are at the middle of the pile load testing area. Figure 1 shows the logs of the two current boreholes, as well as the SPT field values versus depth. The soil stratigraphy interpreted from the two boreholes and the locations of the tested micropiles are given in Figure 2. The soil deposit from the ground surface to a depth of 5.7m consists of clayey silt to silty clay till. Significant seams of gravel and traces of small cobbles have been observed during soil exploration. A layer of compact to dense sand with seams of silt appeared up until the end of the available bore holes depths (9.0m). The groundwater table was found at depth varying from 3.7 to 4.0 m below the ground surface at the time of boreholes. It should be mentioned that during installation of the reaction piles at the time of tests, the groundwater table was observed at a depth of 1.5 m from the ground surface. This indicates high fluctuation of the groundwater table between the summer and the winter (about 2.5m).

As the piles were loaded in a rapid fashion, and due to the cohesive nature of the soil, a total stress analysis is required to represent the shear strength parameters in such soils. Several attempts were made to extract undisturbed samples from the boreholes using a thick wall Shelby tube at depths up to 5.7m. All the attempts failed in borehole 1 due to the fissured over-consolidation nature of the clayey silt soil. The seams of gravel played a big role in this failure. In borehole 2, the same difficulties were experienced until a depth of 3m. Samples were successfully extracted from depth between 3 to 5.0m. Below the depth of 5.7m, there is no need to extract such samples as the deposit is almost cohesionless and the SPT values is sufficient to represent it.

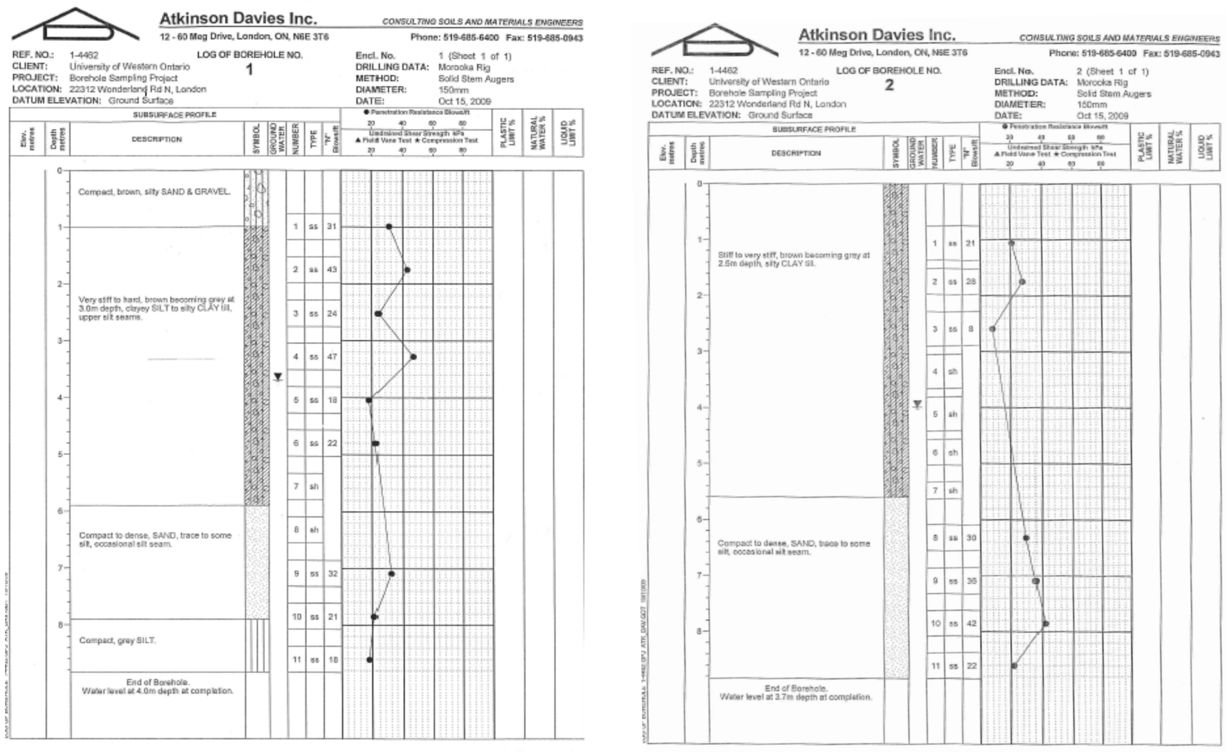

Figure 1. The two borehole logs and the SPT (N_{field}) values Verse depth

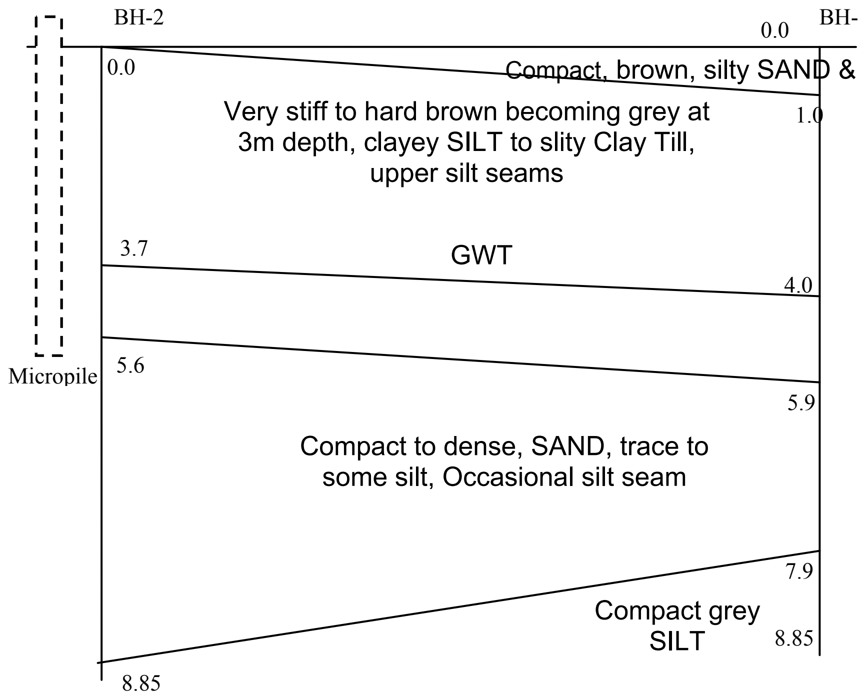

Figure 2. Soil stratigraphy

The samples extracted using the Shelby tubes were tested in a triaxial cell in unconsolidated undrained condition (UU). For the shear strength parameter, C_u, to be representative, it was of great importance to use a loading rate during the UU triaxial tests that is compatible with the rate at which the tested pile is displaced in the soil deposit. Taking into consideration the aforementioned factor, all the triaxial tests were conducted at strain rate equal to 0.0051mm/min. Table 1 summarizes the results of three samples that were successfully extracted from the Shelby tube.

Table 1. Summary of unconsolidated undrained triaxial tests

Depth (m)	Undrained Shear strength parameter, C_u (kPa)	Undrained Secant Modulus, $E_{50\%}$ (MPa)	Failure Strain	Water content
3.0	86	10.1	6.0%	9.5%
3.80	183	15.4	6.0%	10.7%
4.20	174	14.5	5.66%	12.2%

Another attempt was made to obtain the C_u. In this attempt, the SPT field value, N_{field}, obtained from the boreholes is correlated to the C_u through the empirical formula proposed by Terzaghi et al. (1996). The aim of this attempt is to obtain a subsurface profile of the C_u versus depth as the extracted samples seem to be not sufficient for such purpose. Following the procedure given by Sivrikaya and Toğrol (2006), the SPT values is correlated to the C_u thought the following equation;

$$C_u = 6.25\, N_{60} \qquad (1)$$

Where N_{60} is the corrected SPT number (Sivrikaya and Toğrol 2006)
The corrected N_{60} value is related to the SPT filed value, N_{field}, through the equation:

$$N_{60} = (C_B\ C_E\ C_R\ C_S)\, N_{field} \qquad (2)$$

Where:
C_B borehole diameter correction factor =1.05
C_E Energy correction factor, (ER/60) =0.75 for Donut hummer
C_R Rod length correction factor = 0.85 for rod length 4 to 6m, and 0.7 for depth less than 4.0m
C_S Sampler type correction factor, standard sampler without liner =1.2
No correction for the effective overburden pressure is needed, as fine grained soils during penetration are undrained (Sivrikaya and Toğrol, 2006). Introducing these values into Eq. 2 then substituting into Eq. 1 yields:

$$C_u = a_c\, N_{field} \qquad (3)$$

Where a_c = 4.13 (for depth less than 4m) and a_c = 5 (for depth greater than 4m). Figure 3 illustrates the profile for C_u determined from Equation (3) considering the SPT field values obtained from the two available boreholes logs (Figure 1), as well as the results of the three samples tested previously in the triaxial cell.

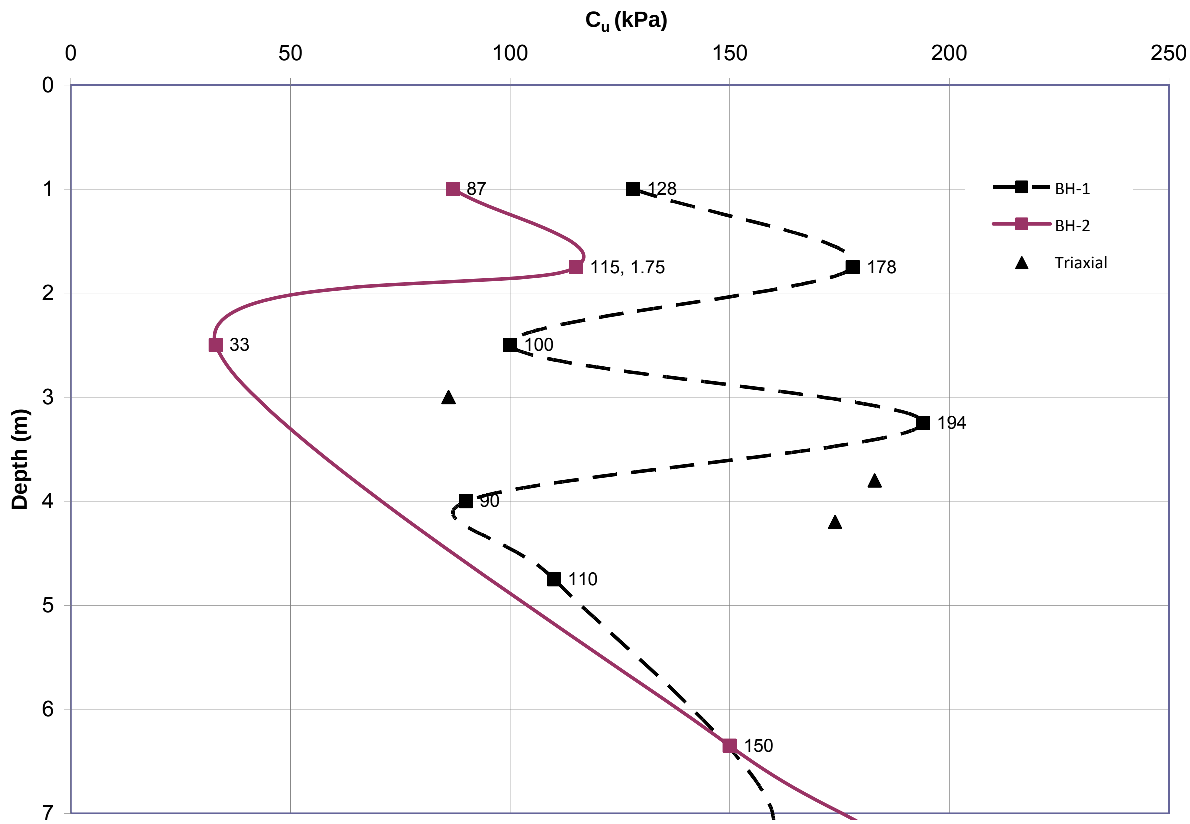

Figure 3. Undrained shear strength, C_u, versus depth

3. MICROPILE MATERIALS AND INSTALLATION

The tested micropiles consist of 6m Geo-drilled injection anchor, B7X1-76, manufactured and supplied by Williams Form Hardware & Rockbolt Ltd (WILLIAMS), shown in Figure 4.The Injection Bar is made of high strength-impact resistant heavy wall steel tubing conforming to ASTM A519 which is continuously threaded over its 10' length with a heavy duty left hand thread/deformation pattern. The thread/deformation pattern of the bar has been shown to exceed the bond characteristics of ASTM A615 reinforcing steel. The thread form is a unique Williams feature that provides a lower thread pitch angle to provide easier coupling disengagement without “locking up”, than conventional rope threads during drilling operations. (Williams Form- Ground Anchor System 2010). The geo-drilled injection bar used had an outer diameter of 76mm, and an inner diameter of 48mm. The hollow core bars were supplied in 3 metre sections and coupled together with 251 mm long geo-drilled anchor coupler, to reach the desired length. The all-thread bar used had a specified yield stress of approximately 580 MPa and a cross-sectional area of 2503 mm^2. A special 176mm diameter tungsten carbide hemispherical button drill bit was used to advance the hollow core bar down the hole. This bit was designed specially for this project by WILLIMS to overcome the gravels and cobbles observed during the soil investigation program. The micropiles were constructed by EBS Engineering and Construction Limited, Breslau, Ontario, using an excavator mounted TE 550 Hydraulic Drifter.

During drilling, the air-flush technique was used to undercut the soils and flush the drill cutting to the ground. Air flushing, rather than the commercial continuous flushing grout technique, was employed in order to examine its ability to advance the hollow core bar down hole with the same efficiency as grout flushing and without any losses in the grout material. The cohesive nature of the deposit should help in successful use of this technique, as a hole in silty clay soils can stand, at least for a short period, without support. During air flushing, the hollow core of the all-thread bar is connected to an XAS 375 JD6 portable air compressor through the swivel at the top of the drilling rig. A pressure of about 0.85MPa was used to advance the hollow core bar downward and flush the debris out from the top of the hole. After reaching the desired depth, the swivel at the top of the drifter was changed and connected to the grout plant.

The bar was grouted continuously to fill the annulus between the hollow core bar and the surrounding soil using a universal post-tensioning grout specially formulated to be pump-able and thixotropic, Master Flow 1341 grout. The filled grout body has water cement ratio of about 0.32 supplied by the grout plant at a pressure of approximately 1.75MPa. The grout cylinders obtained during the installation process were tested after 7 and 28 day for compression and tensile strength. Table 2 shows the results of the tested grout samples. Following the previous procedure, four micropiles were installed in the same day in a square arrangement, and spaced 776mm apart as demonstrated in Figure 5. All the installed micropiles were left in ground for curing after installation and before testing for more than 5 weeks.

4. TEST INSTRUMENTATION AND SETUP

Two compression and two tension load tests were conducted on the four installed micropiles. A maintained load test method was considered in this study, where the load was applied in increments and maintained for a specific period of time. Generally, micropiles are tested in compression in accordance with the ASTM D1143 (1994) quick load test procedure using the setup shown in Figure 6. In tension, they are tested using the setup shown in Figure 7, in accordance to ASTM D-3689 (2007) quick load test procedure. For each load test setup, two helical screw piles were used as reaction piers, and were located at 2.0m (>10 times the micropile diameter) from the center of the tested micropile.

Each micropile was instrumented by five embedded vibrating wire strain gages spaced at 1.5 m from each other. Due to some installations problems, only the top strain gages survived. The two survived gage were located at the top of the micropile and at depth of 1.5m below the pile butt. Four LDTs were used to measure the movement of the pile head. The LDTs are distributed in a square arrange over the pile head using 1 ½ inch plate and mounted on two reference steel extensions supported independently from the loading system. The loading instruments are shown Figure 8.

Figure 4. B7X1-76 Geo-drilled Hollow Core Anchors

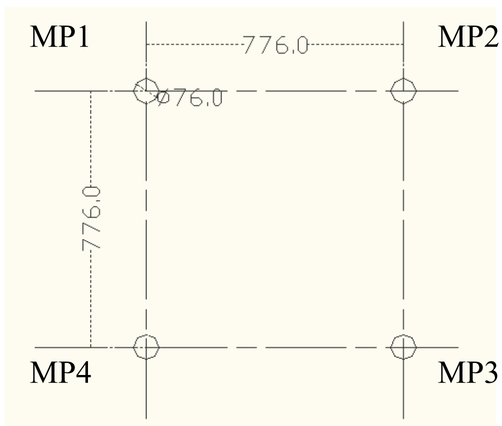

Figure 5. Layout of the Four Installed Micropiles

Table 2. Grout strength

	Compression strength (MPa)	Tensile strength (MPa)
7 days	16.5	4.2
28 days	28.0	6.6

Figure 6. Compression Load test setup

Figure 7. Tension Load test setup

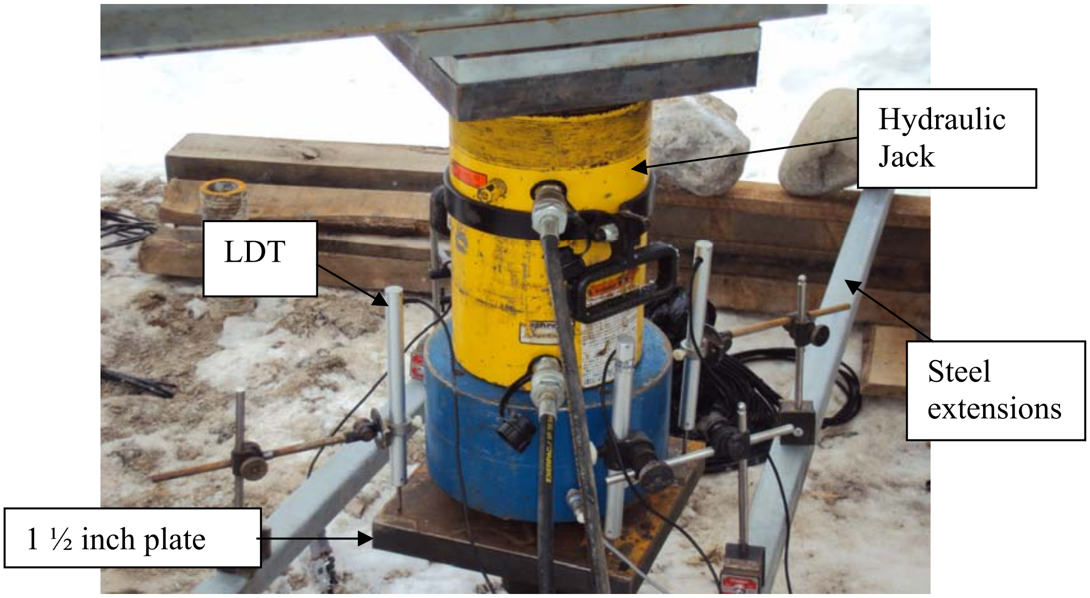

Figure 8. Instrumentation at the pile head

5. TEST PROCEDURE AND RESULTS

One of the main objectives of this study is to highlight the importance of considering the geotechnical failure rather than the structural failure of the micropile. The bond at the bar / grout interface is not an issue for all thread bars used nowadays in micropiles. The thread type used on the bar can achieve high bond performance with the grout, far greater than the obtainable bond at the grout / ground interface. It is always the grout / ground interface that is the limiting factor.

In accordance with the FHWA (2000) Micropile Design Manual, structurally, the installed micropiles can be tested to a load of 1530 kN in compression and a load of 1160 kN in tension using a factor of safety of 1.25. From a geotechnical prospective, FHWA (2000) defines the hollow core micropile as Type B micropile, pressure grouted. Therefore, the nominal bond strength of the stiff silty clay deposit present in the test site is between 70 and 190 kPa. Given the C_u values obtained from the soil investigation program, the highest bond value should be considered. Accordingly, the nominal geotechnical capacity of the micropiles should be 600 kN for a diameter of 176mm for either compression or tension loading. It is particularly important to note that the micropiles in the current phase of testing were not loaded to the point of geotechnical failure during the single pile load test because the testing plan of these piles involves further testing of pairs and 4-pil group. Taking this factor into consideration as well as the values suggested by the FHWA (2000), the maximum load achieved during the micropile load tests reported in this paper was about 500 kN (twice the design load when considering the 600 kN as an ultimate geotechnical resistance of the micropiles).

The two compression load tests were conducted on micropiles MP1 and MP3, while the tension tests were conducted on MP2 and MP4 (see Figure 5). Due to the fact that the micropiles were installed at a relatively close spacing (S/D =4), and because the cohesive nature of the soil deposits, a long testing schedule was followed. The testing schedule incorporated a waiting period of at least 10 days between any two consequent tests to give the soil surrounding the piles some time to rest and regain strength. As mentioned earlier, the piles were loaded in a quick maintained loading procedure, where the load increment was applied and maintained for at least 5 minutes until the maximum load of the test was achieved. When the maximum load is reached, a 10 min creep test was conducted in accordance with the guidelines of the Post-tensioning Institute (2004) to ensure no-geotechnical failure is detected.

Figures 9 and 10 show the load-displacement curve for the two compression and the two tension tests, respectively. The response of the two compression piles under closely the same load seems to be the same, while there is discrepancy between the performances of the two tension piles. Pile MP2 displayed a much stiffere response than MP4 (even stiffer than the compression piles as well). It was also observed that the piles were loaded to a maximum load between 575 and 600 kN with no signs of approaching failure observed in any of them. This clearly demonstrates that the α_{bond} suggested by the FHWA (2000) for Type B micropile underestimates the hollow core micropiles geotechnical capacity.

To examine the possibility of failure of the tested micropiles, the results is examined using two ultimate load criteria; Davisson offset limit (1972) (considered conservative) and NYSDOT (2009) criterion. The Davisson offset limit failure criteria states that the deflection at failure load is:

$$S_f = e_s + 0.15'' + D/120 \qquad (4)$$

Where S_f is the deflection at the ultimate load, e_s is the amount of elastic shortening of the pile, and D is the pile diameter. The amount of elastic shortening of the pile depends on the load transfer mechanism of the pile, i.e., how the pile transfers its load to the surrounding soil. Generally, e_s, is computed from (FHWA 1992):

$$e_s = (Q_p + \alpha_s\, Q_s)\, \frac{L}{A_p E_p} \qquad (5)$$

Where Q_p is the point load transmitted to the pile tip, Q_s shaft friction load, and;
α_s = 0.0 for no load transfer via shaft resistance(End bearing piles), α_s = 0.33 to 0.67 for shaft friction resistance present, and depends on the distribution of shaft friction along the pile (e.g. uniform or linear distribution)

In the case of micropiles, most of the load is transferred to the soil through shaft resistance, relaying on the strong grout/ ground bond developed during installation and grouting, and a micropile is believed to reach a geotechnical failure when reaching an end bearing condition. Accordingly, the value of Q_p herein is assumed to be zero. Due to the overconsolidation behavior of the cohesive soil deposit at the site, a uniform distribution of the shaft friction is assumed, and α_s was taken equal to 0.5.

An important factor that influences e_s is the combined axial stiffness of the micropile, E_pA_p. Axial stiffness of micropiles subjected to tensile loads can be evaluated in a simplified manner by treating it as an anchor, with its single reinforcement and no contribution to the pile stiffness from the grout. For tension test, the axial stiffness is:

$$\Sigma\, EA_t = E_{bar}\, A_{bar} \qquad (6)$$

The composite stiffness of the micropile in compression is far complicated, due to the many factors involved in the installation process, but can be simplified as;

$$\Sigma(EA)_{compression} = (E_{grout} \times A_{grout}) + (E_{bar} \times A_{bar}) \qquad (7)$$

In this study, the elastic modulus values were assigned at $2x10^5$ MPa for the steel and $2.1x10^4$ MPa for the grout.

The NYSDOT (2008) failure criteria states that the slope of the load versus gross settlement curves at twice the design load shall be 0.15mm/ kN, or in other words, the ultimate load is the load where the slope of the load versus gross settlement curve exceeds 0.15/ kN. Figures 11 and 12 demonstrate very clearly that the 600kN load for the compression piles and 580 kN load for the tension piles do not approach neither the failure load nor the nominal geotechnical capacity of the tested micropiles. These results are confirmed by the creep recorded at the pile head presented at Table 3.

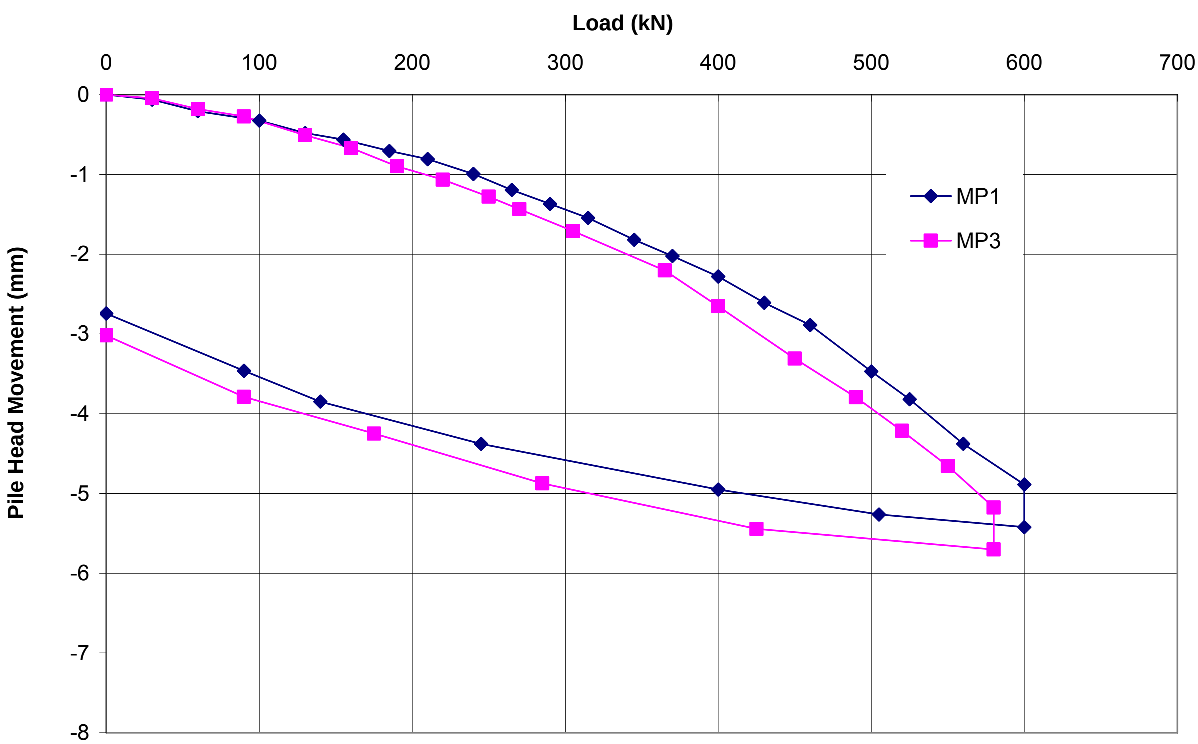

Figure 9. Load –deflection curve for two compression tests

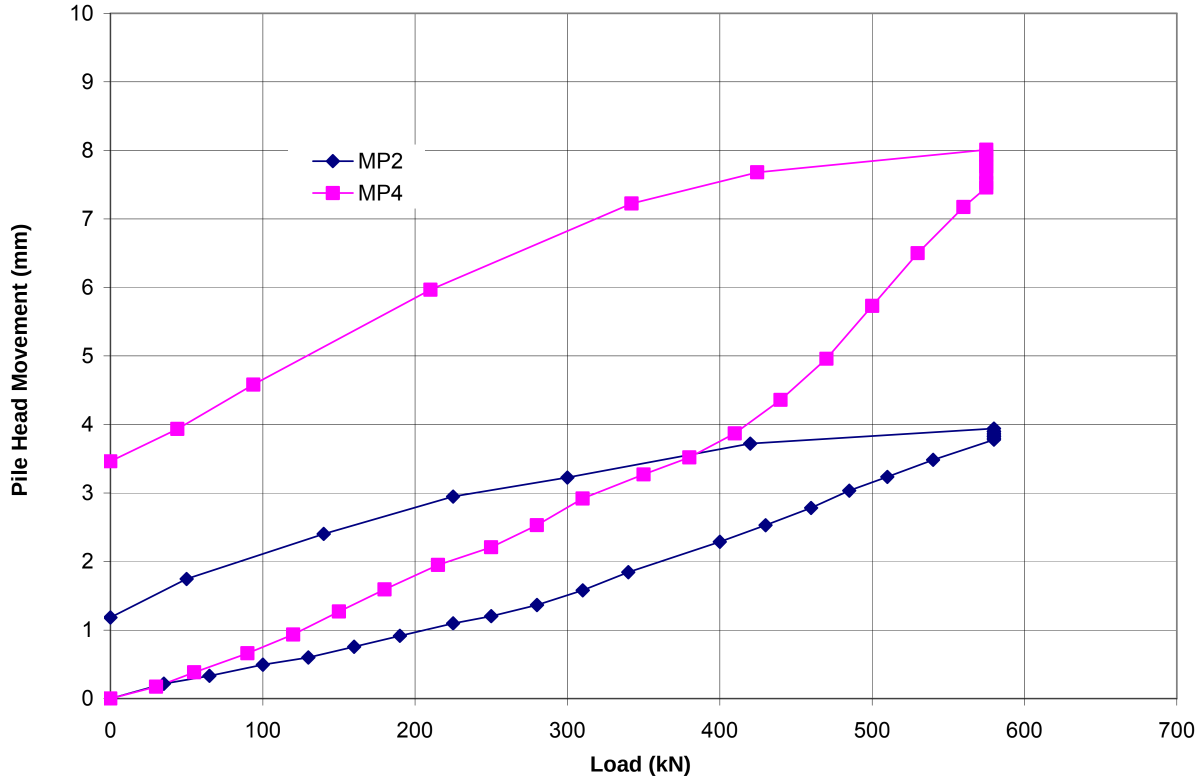

Figure 10. Load –deflection curve for two tension tests

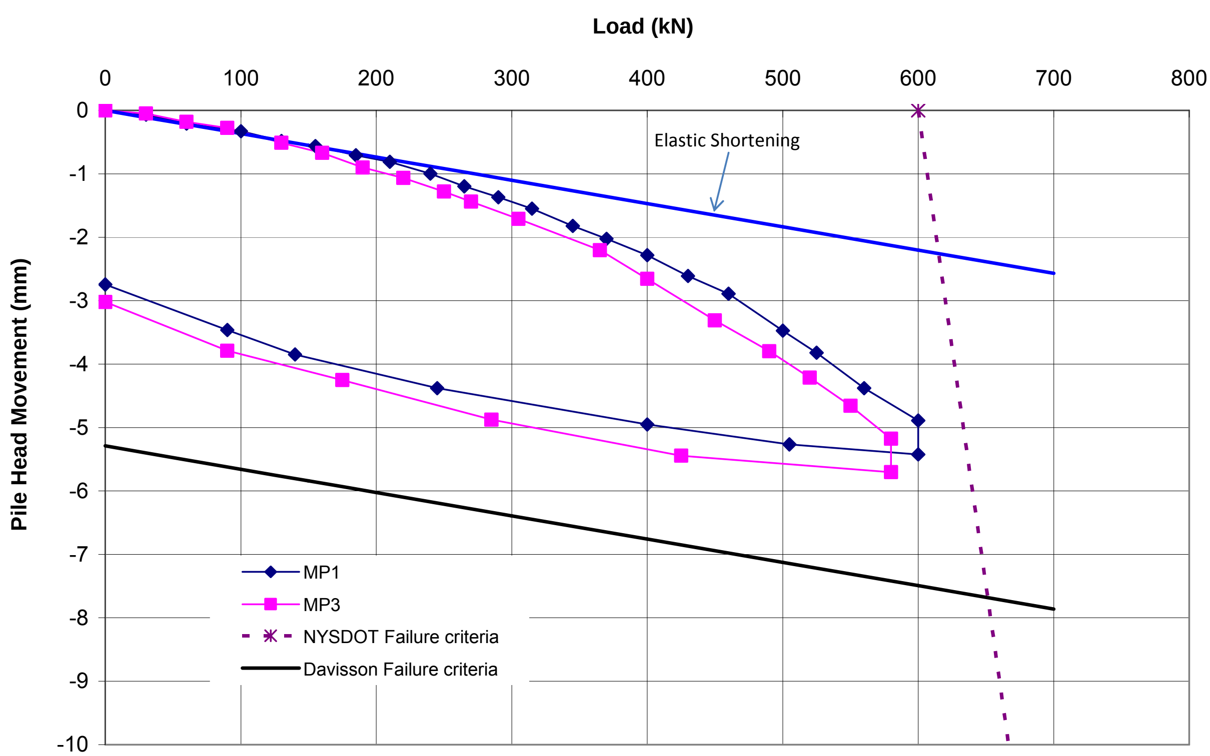

Figure 11. Load –deflection curve for two compression tests with two failure criteria

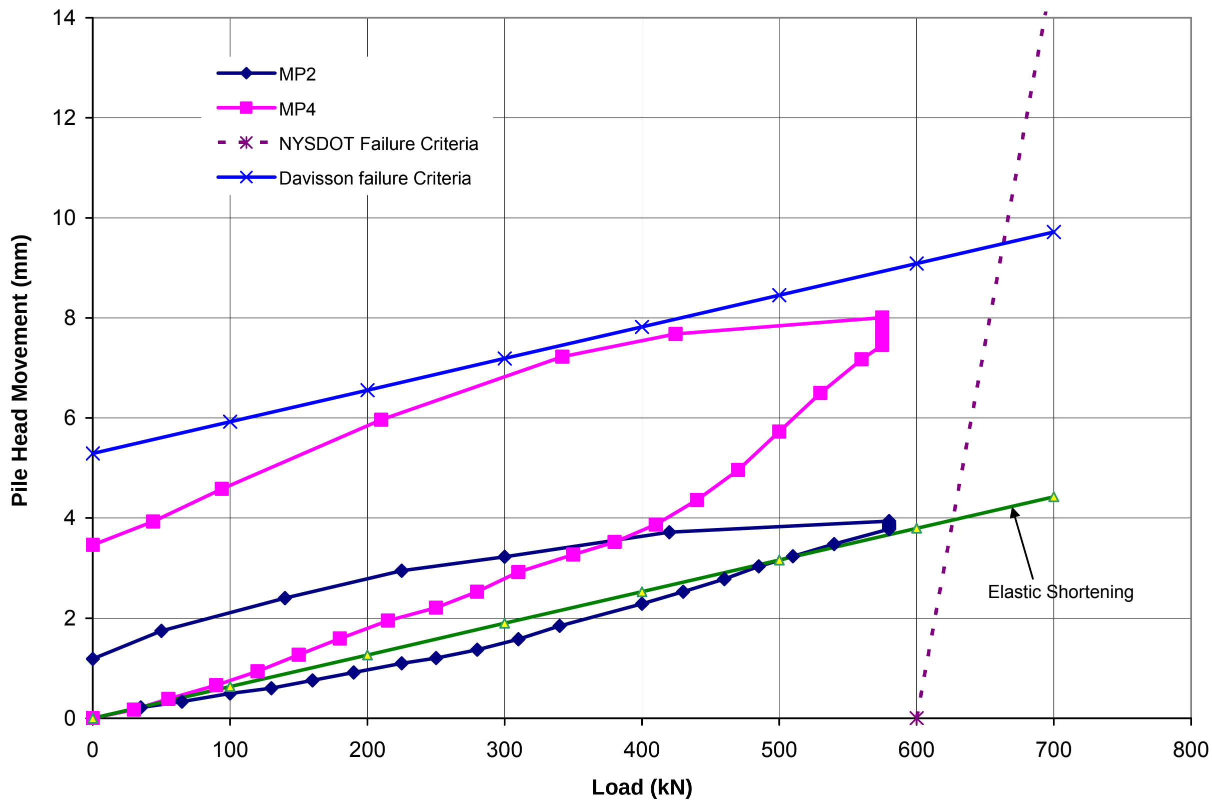

Figure 12. Load –deflection curve for two Tension tests with two failure criteria

Accordingly, the lowest load value of the tested piles, 580 kN, can be considered as twice the design load of such micropiles, therefore, the nominal resistance of the pile will be about 725 kN (1.25 x 2 Design Load (DL) = 2.5 x DL) , and the average bond strength along the micropile length should be 240 kPa at nominal resistance . These values exceed the nominal bond strength suggested by the FHWA (2000) for Type B micropiles installed in silty clay or clayey silt deposit by a factor of about 25%. Even if the hollow core micropile is considered as Type C or D, the evaluated bond strength is still higher than the proposed values. An inspection of the pile diameter enlargement at the base due to installation and grouting process must be accounted for as well. However, the bond values still grossly underestimating the capacity of the micropiles. All of the above results demand that the hollow core micropiles should be treated, geotechnically, as a new type of grouted micropiles, Type E.

Table 3. Micropiles creep at maximum applied load

Micropile	Test	Applied Load (kN)	Creep (mm) from 1 to 10 min
MP1	Compression	600	0.54
MP2	Tension	580	0.17
MP3	Compression	580	0.53
MP4	Tension	575	0.54

Debonding and average bond strength

Bruce et al. (1993) proposed the concept of “elastic ratio”, and showed that the measurement of the elastic deflections can be used to gain an understanding of the length of the pile that is being stressed, and the magnitude and distribution of the load transferred to the ground. The elastic ratio, ER, is defined as the ratio between the elastic deformation of the pile (elastic rebound) and the applied load, that is:

$$ER = \frac{\delta_e}{\Delta P} \qquad (8)$$

Another important parameter that is used to asses the performance of the tested micropiles is the apparent elastic length, L_e, given by:

$$L_e = \frac{\delta_e * \Sigma EA}{\Delta P} \qquad (9)$$

Where; δ_e is the elastic rebound measured or estimated during unloading cycle, ΔP is the magnitude of the unloading calculated as the maximum applied load minus the final load after unloading, and Σ EA is the combined elastic modulus of the micropile section in compression or the elastic modulus of the steel bar in tension. It must be noted that L_e and ER are intrinsically related; one of them can be used to evaluate the other.

The value of δ_e is estimated for a non-instrumented test pile as the total movement minus the residual movement after unloading cycle. Practically, upon unloading, the pile may still have some level of elastic deformation caused by locked in stresses (Gómez et al. 2003). This causes the elastic rebound to be underestimated as well as the load transfer portion of the bond zone, i.e. the apparent elastic length.

For fully bonded micropile, i.e., no casing zone, the value of L_e can be related to the portion of the micropile subjected to significant axial load. Accordingly, it can be used to estimate the ultimate average bond strength acting along a tested micropile where debonding is most probably to occur. Also, It can be used to asses whether an end bearing condition is developed or not. Bruce et al. (1993) explain the development of the end bearing condition as a probability of micropiles failure, which they attribute to the small diameter of the micropiles. This may be the case of a micropile consisting of debonding zone (cased length) and bond zone (uncased length). While in the absence of the cased zone, the problem is far complicated, as the elastic length depends on the amount and distribution of the load being transferred to the soil, type of soil, and method of installation and grouting of the micropile.

Table 4 summarizes the results obtained from the tested micropiles by computing the total, residual and elastic movement as well as the corresponding elastic length calculated using equation 10. For all the tested micropiles, the developed elastic length is less than the total length of the micropiles. This emphasizes that no-geotechnical failure has occurred for any of the tested micropiles. Another important conclusion from Table 4 is the possibility of the increase in the average bond strength. When using a simplified stick-slip response to model the grout-ground interface, the apparent elastic length corresponds to the length where full bond is mobilized and debonding is occurring. By redistributing the load along the developed elastic length (Gómez et al. 2007), the average bond strength will increase from 240 kPa, considering the distribution of the load along the whole length of the micropile, to 280 kPa when considering the load along the developed effective bond length.

Due to the high over-consolidation of the silty clay layer, a post-peak behavior may take place along grout/ground interface at the apparent elastic length rather than full debonding of this portion of the micropile, with the rest of the micropile length still contributing to the grout/ground bond strength. The best way to examine this phenomenon is through cyclic load testing. The current research study involves several one-way cyclic load tests that will be conducted on all micropiles. The results of the cyclic load tests will help in assessing either debonding or softening (post-peak behavior) of the micropiles take place. This may be an important issue for design of micropiles subject to machinery loading, and/or micropiles installed in seismic areas.

Table 4. Summary of the tested micropile results

Micropile	Type of test	P_{max} (kN)	Σ EA (kN) X10^5	Total settlement, δ_t (mm)	Residual settlement, δ_r (mm)	Elastic settlement, δ_e (mm)	Developed Elastic length, L_e (m)
MP1	Compression	600	9.684	5.43	2.70	2.73	3.70
MP2	Tension	580	5.00	3.95	1.18	2.77	2.14
MP3	Compression	580	9.684	5.70	3.02	2.68	3.73
MP4	Tension	575	5.00	8.00	3.46	4.54	3.70

6. CONCLUSION

Full scale pile load tests were conducted to investigate the geotechnical behavior of hollow bar core micropiles. Hollow bars of the type BX76 geo-drilled anchors with 76mm OD and 48mm ID were used. A 176mm carbide bit was thread onto the bar to advance it down the hole. Air flushing technique was used to flush the soil cuttings out the hole. The air flushing was considered successful in getting rid of the soil debris and saving a significant percentage of the grout that is typically used for the grout flushing technique. Two compression and two tension tests were conducted following a quick maintained load procedure. The study indicates that the bond strength at the hollow core micropiles interface is defiantly underestimated by at least 25% in cohesive soils if it is categorized as Type B grouting according to the FHWA (2000). Hollow core micropiles should be treated as a special grouted micropiles, proposed Type E. While considering the hollow core micropiles as type E, factors such as the pressure, speed, and method of installation/flushing, and grouting pressure, must be well documented because of its high influence on the capacity and performance of the micropile. The analysis of the results of the four load tests shows that the elastic ratio, ER, and/or the elastic length, L_e, approaches can well explain the performance of the test micropiles. The apparent elastic length, L_e, may be accomplished with debonding, which is most probably to occur along the apparent elastic length, or post-peak (i.e softening) behavior at the grout/ ground interface. The best way to distinguish between the two behaviors at the grout/ground interface is to test the micropile under cyclic loads and calculate the developed L_e.

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