Use of micropiles in Southern Ontario

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ABSTRACT
Application of gravity-grouted and pressure-grouted micropiles at four sites in Southern Ontario is presented in this paper. The design methods for micropiles are reviewed and discussed. The design load varied from 400 to 800 kN for micropiles with lengths of 11.5 to 25 m and drillhole diameters of 150 to 320 mm. Challenges associated with the micropile installation in saturated cohesionless soils are also addressed. The load testing methods and results are discussed. The bond strength of grout-ground and load transfer back-calculated from the verification/performance tests are correlated to the soil conditions and micropile installation methods. Typical values of micropile bond capacity are recommended.

RÉSUMÉ
On présente dans le présent document l'application de micropieux à quatre sites du sud de l'Ontario. Les méthodes de conception des micropieux ont été examinées et discutées. La charge de conception variait de 400 à 800 kN pour les micropieux avec des longueurs de 11,5 à 25 m et des diamètres de forage de 150 à 320 mm. Le défi lors de l'installation de micropieu dans des sols saturés sans cohésion a également été abordé. Les méthodes de test de charge et les résultats ont été discutés. La force de liaison du coulis au sol et du transfert de charge, calculée à partir des essais de vérification et de performance, était reliée aux conditions du sol et aux méthodes d'installation des micropiles. Les valeurs typiques de la capacité des liaisons micropieu ont été recommandées.

1 INTRODUCTION

A micropile is a small-diameter, drilled and grouted non-displacement pile typically with a single rebar reinforcement. Its diameter is typically less than 320 mm. A micropile is usually constructed by drilling a borehole, placing steel bar reinforcement, and grouting the hole. Another type of micropile employs a hollow-core steel bar with a sacrificial drill bit mounted on the bar tip, used to drill the micropile hole with water, air or grout flush pumped to the bit through the hollow core of the bar. Micropile elements are classified as two cases based on design: (1) the micropile reinforcement directly resists the applied load and (2) the reinforced soil composite reinforced with micropile elements resists applied loads. Micropiles are divided into four types based on the grouting method: (A) grout is placed under gravity head only; (B) grout is placed into the hole under an injection pressure of 0.5 to 1 MPa as the temporary drill casing is withdrawn; (C) primary grout is placed under gravity head and then secondary grout is injected via a sleeved grout pipe without a packer at a pressure of at least 1 MPa prior to hardening of the primary grout (about 15 to 25 minutes); and (D) primary grout is placed under gravity head or under pressure and then secondary grout is injected via a sleeved grout pipe at a pressure of 2 to 8 MPa after hardening of the initially placed grout. For type D micropiles, a packer may be used inside the sleeved pipe so that specific horizons can be treated several times.

Case 1 micropiles have been widely in use for more than 30 years in Canada. Today the use of micropiles represents a common technique for foundation support, earth retention and slope stabilization. In certain design and construction conditions, micropiles offer several advantages over more conventional deep foundations that have resulted in technical benefits such as relatively significant axial load capacity, minimal disturbance to adjacent structures, restrictive access capability, installation at any angle below the horizontal, etc. The major disadvantage of micropiles is relatively high cost in comparison with other deep foundation elements.

Recommendation for design and construction of micropiles are stipulated in the Canadian Foundation Engineering Manual (CFEM) (Canadian Geotechnical Society (CGS), 2006) and Micropile Design and Construction Reference Manual (Federal Highway Administration (FHWA), 2005).

Application of gravity-grouted and pressure-grouted micropiles at four sites in Southern Ontario is presented in this paper. The design methods for micropiles are reviewed and discussed. The design load varied from 400 to 800 kN for micropiles with lengths of 11.5 to 25 m and drillhole diameters of 150 to 320 mm. Challenges associated with micropile installation in saturated cohesionless soils are also addressed. The load testing methods and results are discussed. The bond strength of grout-ground and load transfer back-calculated from the verification/performance tests are correlated to the soil conditions and micropile installation methods. Typical values of micropile bond capacity are recommended.

2 APPLICATION IN SOUTHERN ONTARIO

In Southern Ontario, micropiles are mainly used for foundation support where other types of deep foundation system could not be applied due to site or environmental constraints. Sometimes, micropiles are also used as ground anchors to resist the uplift hydrostatic force. In most cases, a micropile is made up of single 25 to 63 mm, grade 420, 520 or 550 solid thread bar installed within a 100 to 320 mm diameter drill hole. Type D micropiles with post-pressure grout are typically used when a high capacity is required.

Micropile design is similar to a drilled pile or caisson. As micropiles have a relatively small diameter and pressure grouting techniques result in high geotechnical load capacity, the design is usually controlled by structural considerations. High grade steel bar is usually used. In many applications, steel casing is considered for the upper portion of the micropile to increase the capacity of bending moment and lateral resistance, especially in the loose/soft soils. The compressive and tensile loads are resisted through grout to ground bond zone or bond length. The end bearing resistance is ignored for a compressive micropile. The ultimate geotechnical axial capacity of micropiles, R, can be estimated as

$$R = \alpha_{bond}\pi D_b L_b \qquad [1]$$

where α_{bond} is the ultimate bond strength of grout-ground, D_b is the diameter of drill hole, and L_b is the bond length. The ultimate load transfer is defined as R/L_b.

The guidance for estimating the value of α_{bond} is provided in Table 5-3 of FHWA (2005) for Type A to Type D grouting in a variety of ground conditions. The efficiency factors for compressive micropile groups can be taken as 1 for stiff to hard clayey soils or for a pilecap firmly in contact with the ground, and the center-to-center spacing of micropiles greater than three times the diameter of drill hole for cohesionless soils. The potential for the micropile group to fail as a block must also be checked for cohesive soils.

Micropiles are generally designed for working stress and allowable capacity is commonly obtained by dividing the calculated ultimate capacity, R by a factor of safety of 3 without conducting verification or performance tests. A factor of safety of 2 can be used if verification or performance tests are conducted prior to production micropile installation and proof tests are conducted on approximately 5% of production micropiles.

For ultimate limit state (ULS) design, the resistance factor can be taken as 0.6 if verification/performance and proof tests are conducted.

Micropiles can be tested using the same static testing procedure as that for driven or drilled pile based on test methods recommend by ASTM (2013 and 2007). As the micropile capacity is mainly gained from shaft resistance, the micropile can also be tested using the testing procedure for prestressed rock and soil anchors as recommended by Post-Tension Institute (PTI) (2014). The maximum test load for the verification or performance test is usually 2 to 2.5 times the design load. For the proof test, the maximum test load is typically 1.3 to 1.5 times the design load.

The acceptance criterion for verification or performance tests is that the slope of the load versus micropile head movement curve must be less than or equal to 0.15 mm/kN at 2 times the design load as recommend by FHWA (2005). For compressive uncased micropile, the Davisson criteria can be considered (CGS, 2006), in which the micropile head movement, M (unit in mm), at the offset limit is equal to

$$M = \delta + 4mm + 8D_b/1000 \qquad [2]$$

where δ is the elastic deformation of micropile and D_b is the diameter of drill hole in metres. The elastic modulus of rebar is usually used to calculate δ for a conservative approach. For tensile uncased micropiles, a modified offset limit load method may be used to define the maximum movement at the micropile head which is the elastic deformation of micropile plus 4 mm (Kulhawy and Hirany, 1989).

The creep criteria recommended by FHWA (2005) for verification or performance and proof tests are that creep movement should not exceed 1 mm at the test load of 1.3 times design load between 1 and 10 minutes. If this value is exceeded, the creep movement within the period of 6 to 60 minutes should not exceed 2 mm. The creep criteria are similar to those for prestressed rock and soil anchors as recommended by PTI (2014), except that the creep test is conducted at a test load of 1.33 times design load. The creep criterion recommended by ASTM (2013 and 2007) is rarely applied for micropiles, which requires maintaining the maximum test load for 12 hours until the axial movement measured over a period of 1 hour does not exceed 0.25 mm.

Application of micropiles at four sites in Southern Ontario is discussed in following section.

2.1 Pedestrian Bridges in Vaughan

The site was located south of Teston Road and east of Pine Valley Drive in the City of Vaughan, Ontario. Two pedestrian bridges, designated as the South and North Bridges, were proposed to carry trunk sewers across the valley and would also be used by pedestrians and light service vehicles. The South Bridge would be a 219 m long, four span structure with span lengths ranging from 27 to 75 m. The North Bridge would be 172 m long and have three 36 to 75 m spans. In view of the long spans, heavy loads would be expected to be carried by the foundations of the abutments and the piers of the structures.

Subsurface investigation revealed highly variable subsurface conditions throughout the site, the subsurface profile consists of topsoil /fill underlain by cohesionless sand to silt embedded with cohesive silty clay/clayey silt or silty clay/clayey silt till. The cohesive silty clay and clayey silt (till) had low plasticity and firm to hard consistency. Approximate 65% and 80 % of the clayey soils, were, however, very stiff to hard at the South and North Bridge sites, respectively. The compactness conditions of the cohesionless sand to silt ranged from loose to very dense. It was noted that while only 33% of the granular deposits were found to be dense to very dense at the South Bridge site, whereas 75% of these deposits fall into the dense to very dense classes at the site of the North Bridge.

At the North Bridge location, the groundwater table lies between 4.1m and 8.6m below existing ground surface (between elevation 211.2 and 212.1m) as measured in piezometers. At the South Bridge location, the groundwater table lies between 1.1m and 16.3m below existing ground surface (between elevation 195.0 and 201.2m) in the piezometers. Near the creek, the groundwater level lies at or above creek level and flooding may occur in such areas.

For the support of the bridges, consideration was given to use conventional spread footings. Preliminary analyses indicated that suitable bearing strata for the support of the anticipated heavy loads would require excavation depths of 4m or deeper below the existing grades. Consideration of the impact of such deep excavations and the associated dewatering requirements in this environmentally sensitive area would make the use of spread footings less than desirable.

In consideration of the heavy loads at the foundation levels, and the often poor and weak soils found in the upper 4 to 5 m of the subsurface, preference was given to deep foundations as opposed to using spread footings for the support of the structure. Consideration of the environmentally sensitive areas where the piers and some of the abutments would be constructed, of the number of deep foundation alternatives pressure-grouted micropiles were found to be the most environmentally friendly because of the relatively light equipment needed for their installation.

It was therefore proposed that the bridges be supported on 11.5 to 16.5m long (minimum 10 m bond zone), 150 mm diameter pressure-grouted micropiles. The recommended SLS (serviceability limit states) compressive capacity ranged from 400 to 600 kN and factored ULS (ultimate limit states) compressive capacities ranged from 600 to 900 kN. The upper 1.5m of each micropile for the North Bridge and upper 2.0 to 6.5m of each micropile for the South Bridge, where surface fill or weak surface soils exist, were designed to be permanently cased using steel casing. The vertical micropile is not expected to take lateral load. The lateral load was designed to be resisted by raking micropiles. The micropile spacing was more than 1m to avoid the reduction in the shaft friction of micropiles.

The schedule of micropile installation was as follows:

(1) Install a temporary casing using duplex drilling, involving the simultaneous rotation and advancement of inner drill rod and outer drill casing in which the cuttings from the inner drill rod exit the borehole via the annulus between the rod and the casing with the maximum depth of casing below cutting head of 500 mm;
(2) Remove soil cutting using pressured water flush;
(3) Place 50 MPa cement grout through center of drill rod prior to removal of drill rods
(4) Install #18 (57 mm in diameter) of rolled Dywidag thread bar with a yield strength of 517 MPa; and
(5) Perform post pressure grout after first grout has gained its initial strength.

A tensile load test method according to ASTM (2007) was recommended to determine the micropile capacity which is mainly gained from shaft resistance. One tensional performance test to 200% of the design load (SLS) at each abutment of the bridges was recommended. Prior to the installation of the production micropiles, four micropiles were installed for performance tests. Three performance tests were successfully conducted to 195% to 199% of the design load at the east and west abutments of the north bridge and at the east abutment of south bridge. Figure 1 shows the results of one of the performance tests. The movement of the micropile measured at the micropile top is much smaller than that the elongation of the steel bar

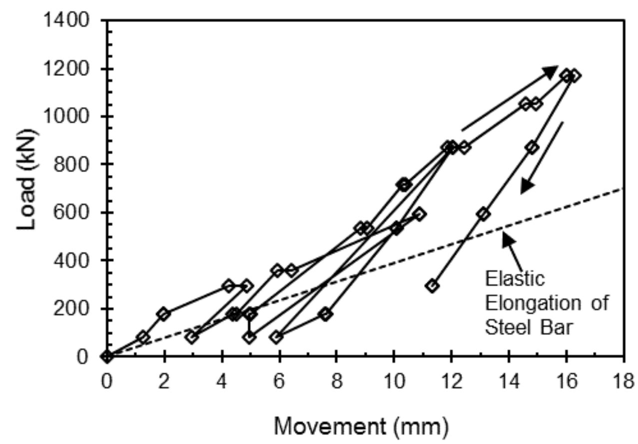

Figure 1. A typical performance test in Vaughan

under the same load. As the performance tests did not reach failure, the load transfer and bond strength calculated from the maximum test load will be smaller than the ultimate values. Thus the performance test confirmed that an ultimate load transfer of greater than 109 to 117 kN/m and the ultimate bond strength of grout-ground (α_{bond}) of greater than 231 to 248 kPa. The value of ultimate load transfer was slightly higher than 100 kN/m for the medium compact sand and silt (Standard Penetration Test (SPT) N-value of 10 to 30), which is higher than 60 kN/m as recommended for hard silt & clay in CGS (2006). The α_{bond} was lower than an average value of 265 kPa for Type D micropile in medium to very dense sand, but higher than the typical values for stiff, dense to very dense silt & clay as recommended in FHWA (2005). Taking a resistance factor of 0.6 as recommended in CSA S6-14 (2014), the factored bond capacity at ultimate limit state (ULS) is greater than 65 kN/m and the factored ULS bond strength is greater than 139 kPa for micropile installed in the firm to hard silty clay to clay silt and compact to vey dense silt to sand.

At the west abutment of the south bridge, the performance test micropile was only performed to 178% of the design working load of 550 kN due to relatively weak soils at this location. The performance test showed an ultimate load transfer of 98 kN/m and α_{bond} of 208 kPa. The α_{bond} was lower than an average value of 265 kPa for Type D micropile in medium to very dense sand as recommended in FHWA (2005), and the ultimate load transfer was also lower than the typical value of 100 kN/m for medium compact sand and silt as recommended in CGS (2006). Taking a resistance factor of 0.6, the factored bond strength at ULS was 56 kN/m for the compact to dense silt to sand. The bond length of micropile was increased from the originally designed 10 m to 11 m.

Production micropiles were successfully installed at all locations except at the south abutment of the south bridge. At least 10 percent of all production micropiles were proof-tested to 150% of the design load (SLS). For the installation of micropiles and to facilitate construction of the south abutment of the south bridge, a working platform was constructed in a shaft using open-cut methods from original sloping ground. During the installation of the second micropile at this location, drilling wash-water blew up through the first micropile which was successfully installed on the same day as well as along the side slope of the shaft. The geotechnical consultant of the contractor considered that the unsuccessful installation of the second micropile was due to a change in ground conditions and therefore the micropile installation was stopped and the constructor required review of the ground condition and micropile design. As the owner's geotechnical consultant, the authors reviewed the subsurface conditions and noted that the soil consisted of saturated sand to silt soils at this location as indicated in the borehole drilled prior to the micropile installation, rather than cohesive clayey silt to silty clay encountered at the other location where the micropile had be successfully installed. The driller did not lower the hydraulic pressure used for water flush to remove soil cuttings. High water pressure resulted in a hydraulic gradient greater than the critical hydraulic gradient as expressed as follows (Holtz et al. 2011)

$$i_c = h/L = \rho'/\rho_w \qquad [3]$$

where h is the excess hydraulic head above the groundwater table in the drill hole for micropile; L is the vertical distance from ground surface to the base of drill hole; $\rho'$is the submerged density of soil; and ρ_w is the water density.

Taking ρ' of 1.9 Mg/m^3 and L of 10 m, h should not be greater than 9 m. The hydraulic pressure added on the drill hole should not be more than 88 kPa.(i.e. 0.88 bar). When the hydraulic pressure is greater than 0.88 bar, a quick condition will occur, i.e. wash-water migration to ground surface.

After two months of discussions, the owner issued a site instruction to ask the contractor to install a test micropile using the same installation process as that used for all other locations with lower water pressure for water flush. The test micropile was successfully installed and tested to 200% of design load under the supervision of owner's geotechnical consultant.

Lessons learned from the project are that the hydraulic pressure used for micropile installation must be selected based on the actual soil conditions and principles of soil mechanics, the micropile specification must be reviewed by the geotechnical engineer who is familiar with the subsurface condition of the site.

2.2 Rail Bridge in Hamilton

The site was located at Desjardins Canal, east of York Boulevard in the City of Hamilton, Ontario. The existing Desjardins Canal Bridge was a single span structure. The railway track lies some 18m above canal bed level. The existing gravity-wall abutments were supported by timber pile foundations. The proposed bridge widening would be located to the west of the existing bridge and consisted of a 3-span structure with two abutments and two middle piers.

Subsurface investigation revealed that the site lies within a deep bedrock valley. Bedrock consisted of reddish-brown shale or mudstone of the Queenston Formation with thin gray limy shale, siltstone or limestone interbeds. The bedrock surface encountered in a borehole at a depth of 66.8 m below existing ground surface (near the track level) was overlain by hard silty clay glacial till. This in-turn was overlain by cohesive soils of generally very stiff to hard silty clay which appear to be of lacustrine origin. These cohesive soils were capped by layers of water-bearing sand, silty sand, sand and silt, sandy silt and silt. The bridge approaches were made up of fill. The fill was predominantly loose to compact silty sand to sand and gravel. Timber was encountered at the bottom of fill. This might represent a timber mat.

The groundwater level in the monitoring wells installed in the boreholes ranged from 14.1 to 14.6 m below existing ground surface, corresponding to the elevations of 74.6 to 74.4 m, slightly higher than the surface water level in the canal at elevation 74.2 m.

Traditional strip/spread footings are not feasible at this site due to only modest bearing capacities available in the native soils. The fact dictated that deep foundation elements be used to support the proposed abutments and piers.

In the boreholes, cohesionless deposits (sand, sandy silt, silty sand and silt) were found, extending to about 40 m below the existing track level and to about 25 m below the canal water level. For the consideration of foundation soil stability and foundation scour protection, any drilled caissons must be founded well below the bottom of the Canal, and thus below the groundwater table. Drilled caissons were not recommended for supporting the structures due to the anticipated problems of caving, boiling and disturbance of the cohesionless deposits below the groundwater level during the installation of caissons.

High-displacement piles such as pipe piles, monotube piles and timber piles may not be suitable for this bridge widening, due to both vibration issues (potentially affecting the existing historical bridge) and given that the pile cap width would be constrained by the existing pile cap and thus closely spaced piles were required.

Driven H-piles would also result in constructability issues. Driving the piles would cause noise and ground vibration problems. Ground vibrations resulting from pile driving might be of concern for the masonry abutments of the existing adjacent bridge. For driven H-piles, a setup period of at least 2 to 3 weeks after the end of initial driving was predicted to be required to allow the increase of pile capacity with time for dissipation of pore pressure generated in the soils during pile driving. In addition, problems could arise for driven piles during the construction to deal with buried obstructions (timber, armour stones, etc.) at the site.

In the boreholes, the soils conditions vary significantly with both location and depth. The native soils above elevation 45 to 50 m generally consist of cohesionless deposits (i.e. silt, sandy silt, sand and silt, silty sand and sand), overlying silty clay deposits. In one borehole, a layer of very dense soils (with SPT 'N' values of over 100 blows per 300 mm penetration) was present between about the elevations of 56 to 65 m, where refusal of driven H-piles would possibly be encountered. However, the dense to very dense cohesionless deposits were not encountered in the other two boreholes, at which practical refusal of driven H-piles might not be reached in the cohesionless deposits. Once a pile is driven to about Elev. 50 m and into the silty clay deposits, the pile driving would not likely reach the required resistance until the pile would be driven well into the clay deposits, possibly to the bedrock (> 65 m in depth). This would be a high potential for overdriving and related concern about maintaining pile shaft straightness. Due to the foregoing constructability issues, driven H-piles were not considered to be the preferred foundation option.

It was recommended that the new bridge be supported by post pressure-grouted micropiles installed within dense to very dense cohesionless deposits. Allowable axial bearing capacity values of 500 to 750 kN per pile were recommended for post pressured-grouted micropiles installed to elevation 50 m.

16.1 m long, 320 mm diameter post pressure-grouted micropiles with 63mm diameter thread bar were designed for the bridge foundation support. The design load was 650 kN and the yield force of the thread bar was 1748 kN.

One test micropile was installed at the north and south abutments respectively. Additional two micropiles were installed to provide reaction force for the test micropiles. The micropiles were tested under axial compressive and tensile loads respectively. The static load testing was performed generally in accordance with CGS (2006) and ASTM (2007 and 2013) using either the standard or the quick load test procedure. The axial compressive load testing was successfully conducted to 200% of the design load and the tensile load testing was successfully tested to 200% to 250% of the design load. At 200% of the design load, the test load was maintained for 6 hours for the compressive load test and 1 hour for the tensile load test.

Figure 2 shows the typical test results for compressive and tensile load tests. The settlement measured at the top of test micropiles ranged from 5.6 mm to 5.8 mm and creep movement ranged from 0.08 mm to 0.09 mm per hour when the micropiles were compressively tested to 200% of the design load. The settlement was much smaller than the estimated elastic deformation of steel thread bar under the same load, which was 32 mm.

The elongation measured at the top of test micropile was 8.4 mm and creep movement was 0.6 mm per hour when the micropiles were tensile tested to 200% of the design load. The elongation was about 150% of the compression under the same test load as shown in Figure 2. This effectively confirms that the tensile load test method to confirm micropile capacity is a conservative approach for the compressive micropile.

The compressive load tests confirmed an ultimate load transfer of greater than 81 kN/m and the α_{bond} of greater than 80 kPa were available. The value of ultimate load transfer was lower than the typical values of 100 to 130 kN/m for medium compact to compact (SPT N-value of 10 to 50) sand and silt as recommended by CGS (2006), and the α_{bond} is also lower than the typical values (145 to 385 kPa) for Type D micropile in medium to very dense sand as recommended by FHWA (2005). This means the bond strength used in the micropile design at this site was conservative.

The tensile load tests confirmed an ultimate load transfer of 101 kN/m and α_{bond} of 100 kPa was available. The value of ultimate load transfer falls within the typical values for medium to compact sand and silt (SPT N-value of 10 to 50) as recommended by CGS (2006), but the α_{bond} is lower than the typical values for Type D micropile in medium to dense sand as recommended by FHWA (2005). Taking a resistance factor of 0.6 as recommended in CSA S6-14 (2014), the factored bond capacity at ultimate limit state (ULS) is greater than 61 kN/m and the factored ULS bond strength is greater than 60 kPa for micropile installed in the dense to very dense silt to sand.

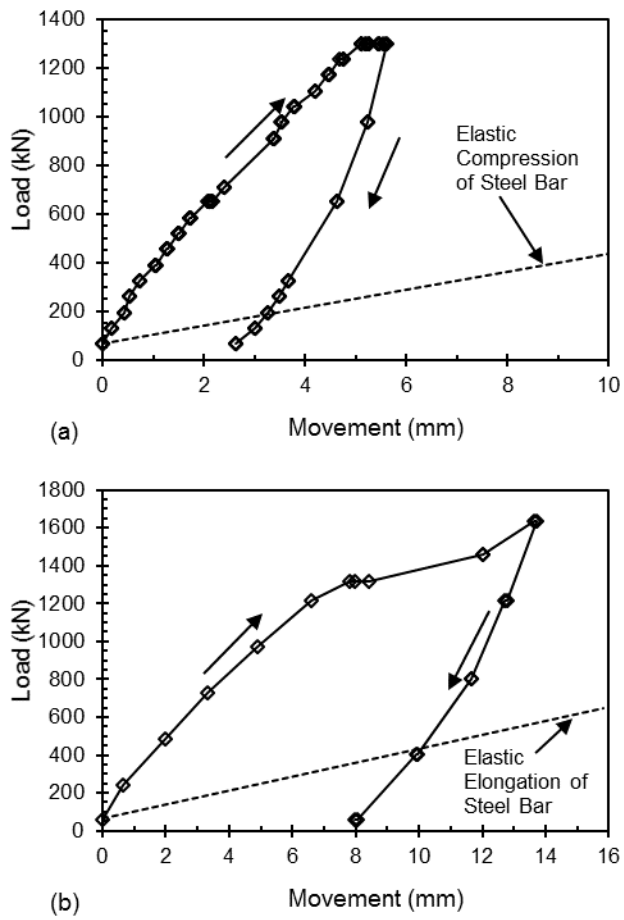

Figure 2. Typical performance tests in Hamilton: (a) compressive load; (b) tensile load

2.3 Waterloo Wastewater Treatment Plant Upgrade

The site was located approximately 200m north of University Avenue East and approximately 100m west of Conestoga Parkway in the City of Waterloo, Ontario. Subsurface investigations indicated that the site stratigraphy was made up of approximately 10m of firm to hard silty clay overlying very dense sand and silt till. The groundwater table was about 4.5m below grade.

The proposed structures would extend below the groundwater table and would, therefore, be subjected to hydrostatic uplift pressures. As the combination of the weight of the structure and the mobilized frictional resistance between the buried portion of the exterior walls and the backfill materials was insufficient to resist the uplift forces during any stage of the construction and/or during the operation of the structure, tie-down anchors were

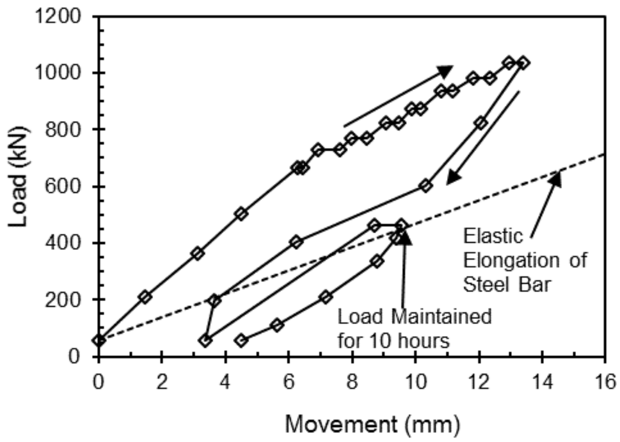

Figure 3. Typical performance tests in Waterloo

proposed to resist buoyancy. Two types of tie-down anchors could be considered: (1) helical pier; and (2) micropile. Since the glacial till deposits at this site contain boulders and cobbles and are in a dense to very dense compactness state, helical piers could not be successfully installed to the design depth. Micropiles were selected as the tie-down anchors.

The micropiles were installed within 178 mm dia. drill holes into very dense sand and silt till and were made up of a 44 mm diameter thread bar. The length of micropile was 12.2 m including a 9.2 m bond length and 3.0 m free length. The micropiles were cement-grouted using the tremie method. The design load for each micropile was 400 kN.

Three performance tests were conducted using the tensile load test method as per the guidelines of ASTM (2007). A 57 mm diameter thread bar was used for the performance test micropiles in order to test the micropile to higher loads.

Figure 3 shows the typical test result. All of three tested micropiles were successfully loaded to 255% of the design load and the test load was maintained 10 minutes. Creep testing was performed in two test micropiles when the loading was released to 115% of design load, under which the test load was maintained for 10 and 48 hours respectively. The elongation of thread bar under 255% of the design load was less than its elastic deformation under the same load. The creep for three tested micropiles was less than 1 mm during the period of 1 to 10 minutes at the applied load of 255% of the design load. The creep was measured to be less than 1 mm for test load at 115% of design load maintained for 10 to 48 hours.

The tensile load tests confirmed an ultimate load transfer of greater than 101 kN/m and α_{bond} of greater than 100 kPa were available. The α_{bond} falls within the typical values (95 to 190 kPa) for Type A micropile in medium to very dense glacial till as recommended by FHWA (2005).

Taking a resistance factor of 0.6 as recommended in CSA S6-14 (2014), the factored bond capacity at ultimate limit state (ULS) is greater than 61 kN/m and the factored ULS bond strength is greater than 60 kPa for micropile installed in the dense to very dense sand and silt till.

Selected production micropiles were successfully proof-tested to 167% of the design work load. The elongation of thread bar under 167% of the design load was less than its elastic deformation under the same load.

2.4 High-rise Building in Toronto

The site was located in the City of Toronto, Ontario. The development consisted of a pair of 16 to 18 storeys of condominium towers with 3 levels of underground parking. Subsurface investigation indicates that the site stratigraphy made up of approximately 2 to 3 m of clayey silt fill over up to 8 m of firm to stiff silty clay to clayey silt till or compact sandy silt till. The glacial till was underlain by a thick deposit of silt, sandy silt to silty sand typical typically in a loose to very dense but generally compact to dense condition. The measured piezometric head varied between 8.6 to 12.0 m below the existing ground in the lower silt, sandy silt to silty sand.

The excavation for the basement construction reached the groundwater table in the silt to silty sand deposit, which is described as being dilatant below the groundwater table, i.e., it has a tendency to expand and exhibit a liverish or jelly-like behaviour when subjected to disturbance or vibration.

Construction dewatering was not properly conducted and as a result, the silt to silty sand dilated at some locations and lost its bearing capacity. Thus, micropiles were proposed to be installed in the disturbed ground.

The drill hole was wash-bored to 25m below the excavated level with 152 mm outside diameter casing. Steel reinforcement was 63.5 mm dia. Williams bar with spacers. Three grout tubes were attached to the bar at 22 m, 16 m, and 10 m depths below grade. The micropile was initially grouted under low pressure and allowed to set, and then post-grouted under high pressure through each of the three grout tubes on the second day after the initial grouting.

One compressive load test was conducted to verify the design load of 800 kN. The test setup followed the guidelines of ASTM (2013). The load was applied to the micropile by a hydraulic jack acting against a reaction frame anchored by four micropiles. The maximum test load was 1200 kN (i.e. 150% of the design load).

Loads were applied in increments of 25% of the design load to reach a total of 150% of the design working load. Cyclic loading was applied at 50%, 75%, 100% and 125% of the design load. At 25% and 50% of the design load, the test loads were maintained for 10 minutes. At 75% of the design load, the test load was maintained for 30 minutes. At 100% and 125% of the design load, the test loads were maintained for 60 minutes. At 150% of the test load, the load was maintained for 30 minutes. On second day, it was found that the load dropped from 1200 kN (150% of the design load) to 1095 kN (137% of the design load) as the load was not maintained. Thus the load was re-applied to 150% of the test load again and maintained for 60 minutes. Then the pile was unloaded in decrements of 25% of the design load to zero with 1 minute between each decrement.

The settlement rate at 100% and 125% of the design load was less than 0.25 mm for 1 hour. When the test load

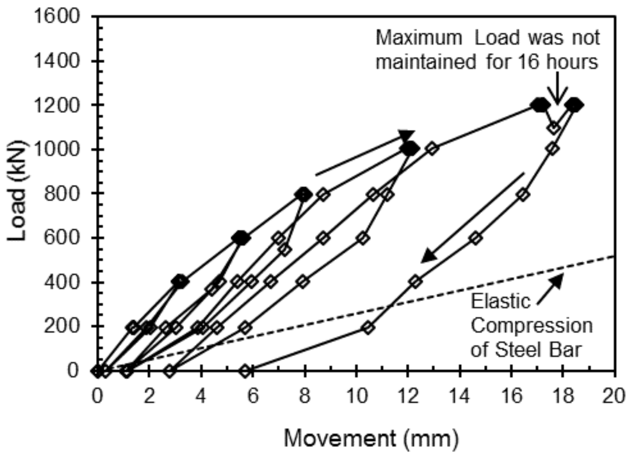

Figure 4. Typical performance tests in Toronto

dropped from 1200 kN to 1095 kN, the settlement was 0.4 mm for 16 hours. When the load was readjusted from 1095 kN to 1202 kN, the settlement was 0.7 mm. The movement of micropile head at the maximum load (150% of the design load) was 18.5 mm, which was 36% of the elastic compression of the steel bar.

Although the compressive test did not reach 2 times the design load due to insufficient reaction force for the test, it was considered that micropile could be rated for the design load of 800 kN as the settlement at the design load was 8 mm, only 27% of the the elastic compression of the steel bar.

The compressive load test confirmed an ultimate load transfer of greater than 48 kN/m and α_{bond} of greater than 100 kPa were available. The value of ultimate load transfer was lower than the typical values (100 to 130 kN/m) for medium to compact sand and silt (SPT N-value of 10 to 50) as recommended by CGS (2006) and the α_{bond} value was lower than the typical values (145 to 385 kPa) for Type D micropile in medium to very dense sand as recommended by FHWA (2005), because the load test was not tested to 2 times design load.

3 CONCLUSIONS

Based on review of application of micropiles at four sites, it is found that relatively low grout-ground bond strength has been generally used in micropile design in Southern Ontario. The following conclusions and recommendations can be made for micropile design and construction:

(1) The tensile load test method to confirm compressive micropile capacity is a conservative approach. The micropile extension under a tensile load was about 150% of the compression under the same load at one site.

(2) It is conservative to design post pressure-grouted micropiles using an ultimate grout-ground strength of 100 kPa for compact to dense silt to sand and to design gravity-grouted micropiles using an ultimate grout-ground strength of 100 kPa for dense to very dense glacial till.

(3) An ultimate strength of grout-ground of 200 kPa and factored ULS strength of grout-ground of 120 kPa can be used for micropile installed in compact to dense silt to sand.

(4) The typical value of ultimate load transfer recommended in CGS (2006) for post pressure-grout anchor may overestimate the strength of grout-ground for micropiles installed in saturated compact to dense silt to sand.

(5) For successful micropile installation in saturated cohesionless deposits, it is critical that the geotechnical engineer be engaged to review contractor’s installation methodology including the flushing pressure used for drilling.

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