Evolution of high capacity micropiles in the Metropolitan Toronto Area

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ABSTRACT
Micropiles are small diameter (≤ 305 mm), drilled, replacement piles consisting of high strength steel and cement grout that transfer axial loading to the earth via grout-to-ground sidewall friction. Several challenging projects within the Metropolitan Toronto Area have been completed using micropiles.

Five case studies selected from among over 100 micropile projects completed by Geo-Foundations Contractors Inc. since 2001 are analysed. The variation of micropile configurations, load capacities and test results are presented to demonstrate the suitability and performance of high capacity micropiles in a variety of settings both above and below surface. All of these projects include at least one static compression load test to at least 200% of micropile design load. Comparisons of the estimated vs. measured grout-to-ground adhesion of various configurations of high capacity micropiles embedded in rock and soil are provided.

The suitability of high capacity micropiles in various soils types and rock is clearly demonstrated based on the evaluation of test results obtained.

RÉSUMÉ
Micropieux sont de petit diamètre (≤ 305 mm), percé, piles de rechange consistant en acier haute résistance et coulis de ciment que le chargement axial transfert à la terre par l'intermédiaire de friction latérale coulis-sol. Plusieurs projets stimulants au sein de la région métropolitaine de Toronto ont été réalisés en utilisant des micropieux.
Cinq études de cas choisis parmi plus de 100 projets réalisés par micropieux Les Entrepreneurs Geo-Fondations depuis 2001 sont analysés. La variation des configurations, des micropieux capacités de charge et les résultats des tests sont présentés pour démontrer la pertinence et la performance de micropieux à haute capacité dans une variété de paramètres à la fois au-dessus et dessous de la surface. Tous ces projets comprennent au moins un test de charge statique de compression d'au moins 200% de la charge la conception des micropieux. Les comparaisons de l'estime vs mesurée coulis-sol d'adhérence de diverses configurations de haute capacité Micropieux intégré dans la roche et le sol sont fournis.
La pertinence de micropieux à haute capacité dans les types de sols différents et le rock est clairement démontré sur la base de l'évaluation des résultats des tests obtenus.

1 INTRODUCTION

Geo-Foundations Contractors Inc. have designed and constructed more than 100 micropile projects across Canada since 2001. The majority of these projects included at least one static compressive load test to 200% of micropile design load. In this paper, five micropile projects constructed in the Metropolitan Toronto Area are analysed to demonstrate the high load capacities achieved using different drilling and grouting installation methods across a wide range of soil conditions.

2 BACKGROUND

Micropiles are small diameter (≤ 305 mm), drilled, replacement piles consisting of high strength steel and cement grout that transfer axial loading to the earth via grout-to-ground sidewall friction. Micropiles are typically used in situations where above-ground or below-ground conditions render the use of conventional deep foundations impractical, too difficult, or too risky to the health of neighbouring structures or services.

Micropiles are classified by numerous sub-types, each determined by the combination of widely varying drilling and grouting installation processes. Drilling methods include double-head duplex, rotary percussion, continuous grout flush, and non-percussive rotary with polymer flush just to name a few. Grouting methods include gravity (tremie) injection, dynamic grouting, pressure grouting during casing retraction, and tremie injection augmented with high pressure post grouting. The specialized drilling and grouting techniques employed allow for high adhesion along the grout-to-ground interface (i.e. bond zone).

The principal structural distinction between micropiles and conventional cast-in-place replacement piles is the primary mode of load carrying: in micropiles, strength is derived primarily by cross sectional area of high strength steel, augmented by neat cement grout; conventional drilled shafts are just the opposite, deriving strength primarily via large cross sectional area of concrete, augmented (or not) by steel reinforcement. In geotechnical terms, end bearing is not considered to contribute to micropile capacity.

Micropile technology has progressed continuously since its inception in Italy by Dr. Fernando Lizzi in the early 1950’s, highlighted in North America by the arrival of micropiles in the United States in the late 1970’s, their progression towards popularity in the 1990’s and their acceptance in the main stream by way of the publication in

2000 of the seminal US Federal Highways Administration (USFHWA) Micropile Implementation Manual. Over the past decade in the Metropolitan Toronto Area, significant advances in drilling and installation methods, coupled with numerous successful full scale load tests, have extended the applicability of micropile techniques beyond the original local market of restricted access, industrial-commercial-institutional (ICI) work into the broader spheres of open access ICI, heavy civil and transportation infrastructure.

Major equipment and process innovations have improved the ability to drill through challenging sub-surface conditions in areas of very limited access, and/or with very limited introduced energy. The significant growth of the micropile market over the past decade is a manifestation of the advances in load testing data from local projects, exposure in technical publications, promotion by trade organizations and excellent quality control measures. Micropiles are considered a reputable construction system throughout the world based on exceptional value and potential (Xanthakos, Abramson and Bruce, 1994); Federal Highway Administration (FHWA, 2000) and Cadden et al. (2004).

High capacity micropiles are becoming competitive with conventional techniques such as driven pile and drilled shaft systems, so much so that micropiles are considered to be a very attractive solution based on cost per kN in North America for even new construction on open sites (Cadden et al., 2004).

Much more so than any other deep foundation system, the geotechnical capacity of a micropile is particularly dependent on the appropriateness and workmanship of the drilling and grouting methods employed. For this reason, full scale load testing of representatively constructed micropiles, often completed on a pre-production basis to verify both the design basis and the quality of workmanship, is a key component of micropile quality assurance.

3 CASE STUDY # 1 – TIP TOP LOFTS, 2003

The former Tip Top Tailors building is one of the most recognizable heritage structures in all of Toronto. The circa-1929 building was transformed from its original, industrial purpose to luxury condominium suites known as *Tip Top Lofts*. The centrepiece of the conversion is a new 6-storey steel tower perched atop the original 5-storey concrete framed structure. Micropiles, constructed within the low headroom basement, were required to support and tie down new elevator cores and new shear walls forming the upgraded framing required to support the vertical expansion.

3.1 Sub-surface Conditions

The site is part of a large block of reclaimed land where Lake Ontario was filled in to extend the limits of Toronto lakeward in the 19th century. Consequently, the site, when measured from finished basement floor, sits atop 6 m of very loose fill deposits (with groundwater present within the first metre) underlain by a thin (1 to 2 m thick) layer of cohesive till with shale fragment inclusions atop weathered shale of typical thickness 1 m transitioning to sound shale of the Georgian Bay formation (typical UCS 10 to 20 MPa).

3.2 Micropile Design Approach

The existing concrete structure is founded on driven timber piles terminated in the weathered shale. With the existing timber piles plumb and the majority of the 76 micropiles battered at 1H:6V, it was virtually certain that there would be interference at some, if not many, of the micropiles. With the basement floor slab preserved as the working grade, typical headroom across the site was 3.3 m. Micropile sizing considerations were governed by the Type I loading condition: 2000 kN compression / 1200 kN tension (service loads acting on the same micropile).

With several constructability factors and high loading to consider, a 2-step drilling process was conceived whereby each hole was first cased to rock by advancing a casing (with butt-welded splices at 1.5 m centre-to-centre spacing) using shoe-drive, single-head, concentric percussive duplex drilling method through the loose fill and high groundwater. Casings were terminated a minimum of 1 m below top of sound rock before drilling the rock socket using open-hole percussion drilling. Given the high tension loading to be resisted, an arrangement of 2-63ø / 550 MPa threaded bars was selected as the micropile reinforcement. Since the bars had to be sufficiently strong to resist the tension loading, it was a small step further to use the bars as the sole reinforcement to resist the compression loading. With the bars capable of taking all the load, the casing's role could be limited simply to a confining shell for the grout and a means by which the rock socket could be drilled and cleaned and inspected, without the casing having to contribute directly to the structural capacity of the micropile. Treating the casing as a left-in-place inclusion, the welded splices did not have to be subject to anything more than perfunctory, duty-based inspections and as such the micropiles were able to be constructed as expeditiously as possible.

3.3 Pre-production micropile

Numerous factors contributed to the decision to perform only one load test at this project. Headroom and access challenges were significant, favouring a lower frequency of testing. Piles were socketed in sound rock, with sockets constructed in such a manner that enabled thorough cleaning and measuring prior to grouting, and this also justified a lower frequency of testing. Depth to bond zone was relatively short (7 m) and the design was conservative (no reliance on the strength or load resisting contributions from the casing) thereby relieving concerns regarding deflection-under-load performance and justifying a lower frequency of testing. Once it was established that only a single test would be required (provided, of course, all acceptance criteria were met) it was important to balance the low frequency of testing with a high magnitude of testing. The test micropile – a battered, Type I micropile at the east elevator core – was loaded to 4000 kN in cycled static compression using two adjacent Type I production piles to anchor the reaction frame. Figure 1 a) shows the set-up for drilling and b) load testing arrangement for the pre-production static compression test. Summarized load test results for the 11 m deep test micropile are presented in Table 1.

Figure 1. a) Low-head room drilling inside the existing basement at Tip Top Lofts; b) Micropile load test arrangement

4 CASE STUDY # 2 – ART GALLERY OF ONTARIO (*TRANSFORMATION AGO*), 2005

The Art Gallery of Ontario's various galleries, collection vaults, and administrative offices are housed within a mix of structures dating from the 1910's, 1920's, 1970's and 1990's. The AGO's most recent major renovation, *Transformation AGO*, required structural modifications to and within structures from each of the AGO's past periods of construction. Micropiles provided a highly capable but sufficiently flexible means for meeting the assorted challenges facing this project's designers. A total of 138 permanent micropiles were constructed, in seven different areas of the property, ranging from the open-air Dundas boulevard space with its buried 6 m deep sewers, to the 1970's sub-basement accessible only by elevator, to the 2.8 m headroom 1920's ground floor space beneath the new South Tower.

4.1 Soil Conditions

Top of sound rock is located at 77.0 m (geodetic) across the entire site. Micropile work was staged from several different elevations, corresponding to prevailing grade or finished floor on grade at the respective work areas. Prevailing grade at the highest elevation work area was at 96.8 m; at the lowest work area 88.0 m. The overburden profile at the highest elevation micropile location consists of fill, soft silty clay till, sand, hard clay till, weathered rock and sound shale bedrock of the Georgian Bay formation. Groundwater elevation was coincident with the top of the sand layer at elevation 85.0 m, 12.5 m above micropile tip elevation.

4.2 Micropile Design Approach

The micropile design was governed by the serviceability requirements of the South Tower pile caps, where the compressive service loads acting at the base of the columns were as great as 6672 kN, and the available footprint in which to construct pile caps limited the quantity of micropiles to a maximum of 3 per cap. Consequently, no consideration of micropile design could begin without first acknowledging that micropiles of individual axial compressive service loading of 2450 kN would be required.

All micropiles were socketed in sound shale bedrock. Although all transfer load above the bond zone was purposely ignored, all micropiles were pressure grouted through the top of the casing in order to engage, to the best degree possible, the competent ground above the bond zone to stiffen the piles' deflection-under-load performance.

With the need for rock-socketing firmly established, constructability considerations guided the micropile structural design. The design had to appropriately address:

- the significant depth to the top of sound rock (18 m) and the character of the overburden soils above the rock (inclusive of a thick sand layer below the water table), which necessitated a drilling process that could ensure hole stability;
- the need for as small a diameter boring as possible so that small, electric-powered drill rigs could be used to construct the many indoor micropiles;
- the severely restricted headroom (2.8 m) found at a high number of micropile locations.

All of these collected challenges were met by selecting thick-walled permanent casing as the primary micropile reinforcement, with threaded bar bundles grouted into the rock sockets (Fig. 2). As a means of keeping the drill hole suitably small, the design was further enhanced by use of 655 MPa threadbars instead of the typical 517 MPa grade, and high performance 50 MPa grout. Also done for the sake of minimizing hole size, sacrificial steel – computation of the cross section of the reinforcement only after removing 1.6 mm from its outer shell – was used as the mode of corrosion protection for both the permanent casing and, most importantly, the rock-socket threadbars.

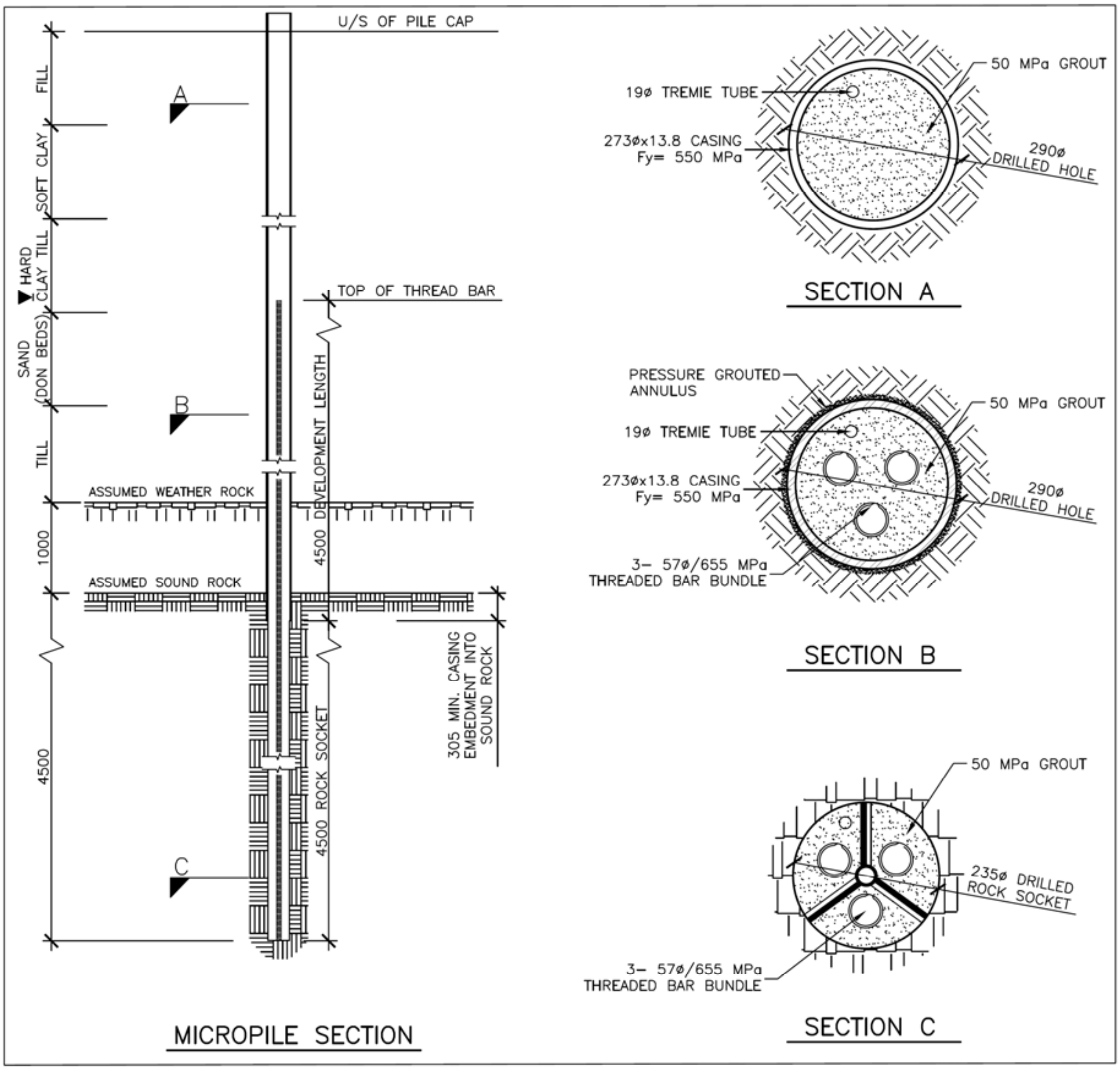

FIGURE 2. Transformation AGO Project - Sub-surface profile and configuration of pre-production micropile

4.3 Sacrificial pre-production micropile

As the micropile design belonged entirely to the micropile contractor, the results of load testing – and the commercial ramifications should there prove to be a problem – were entirely at the contractor's risk. Load testing was at negligible risk of failure due to an overambitious geotechnical design: the geotechnical design basis was well understood, buckling was never truly a concern given the ample shear strengths of the various confining soil layers, and the 1000 kPa design bond stress had been proven countless times via tie back testing and micropile testing in the well known Georgian Bay shale bedrock. This was not necessarily the case for all of the structural aspects of the micropile design however, and the contractor's goals for load testing included taking each test pile as far beyond 200% of design load as proved practical.

The specifications required that there be no fewer than two pre-production load tests performed to at least 200% of design load, in static compression on sacrificial micropiles. Although the specifications called for a seemingly small number of load tests, and the stakes for these tests were disproportionately high given the high magnitude of individual production micropile loading and the paucity of any remotely similar past Toronto installations, this low frequency of load testing was justified in that it limited the cost impact of such expensive testing on the greater project, and in that the risk of performance was entirely borne by the micropile contractor. This low frequency of testing was offset by the micropiles' conservative geotechnical design basis and the fact that the test piles were sacrificial, thereby allowing for higher applied test loads beyond the mandatory 200%. The first load test was carried out by loading a sacrificial pile cap supported by two micropiles designed for axial service compression 1724 kN each. Each micropile consisted of a 20 m deep, 194ø / 550 MPa permanent casing as the sole upper reinforcement overlapping (by 4.5 m) a 9 m long, 2-57ø / 655 MPa threadbar bundle in the rock socket. The second load test was carried out by loading an identically deep, single micropile designed for axial service compression of 2847 kN, consisting of a 273ø / 550 MPa permanent casing as the sole upper reinforcement overlapping a 9 m long, 3-57ø / 655 MPa threadbar bundle in the rock socket. The first test was taken to 8050 kN (233% of micropile design load) at which load the pile cap ruptured but the micropiles were still performing acceptably. The second test was taken to 7004 kN (246%), at which load the test micropile began slowly, but unmistakably, to plunge. Test results are summarized in Table 1.

5 CASE STUDY # 3 – 130 BLOOR STREET WEST, 2007

This project, located in the trendy and popular Yorkville area of midtown Toronto, featured micropiles constructed in the basement of an existing, circa 1950's, 13-storey tower. The existing tower, with 2 basement levels below grade, was retrofitted for a 6-storey vertical expansion to construct high-end luxury residential apartments. Support for the new space was carried downwards via plate-reinforcing of existing steel columns, supported in turn by existing pad footings augmented with micropiles. A total of 46 micropiles were constructed to augment 11 existing pad footings. The new mode of foundation load resistance was a hybrid between bearing at the underside of the footings and supplemental resistance provided by the micropiles.

5.1 Sub-surface Conditions

Relative to the underside of existing footings, the subsurface profile consists of 6 m of Queen's Park Varved Clay, a very stiff, silty clay with thin, dry laminations of very fine sand, underlain by the Queen's Park Stratum, a dense glacial till consisting of a sand silt matrix with gravel and cobble inclusions. From known borings outside of this project's investigations, the depth to groundwater is 9 to 10 m and the depth to bedrock exceeds 30 m.

5.2 Micropile Design Approach

Individual micropile loading was 1000 kN (service) axial compression, at which magnitude the quantity of piles at any one augmented footing could be limited to four. Micropiles were not required to resist uplift or lateral loads. Although headroom restrictions were typical throughout the work and as severe as 2.5 m at some locations, the governing condition, present at half of the micropile locations, was access and working footprint, necessitating the use of a mast-only drill (with remote power pack) fastened to the existing structure. In order to locate the respective pile-to-footing connection structures within the available vertical envelope between the footing and finished floor, the micropiles were drilled inside the respective footprints of the augmented footings. With access limiting the size and capability of the drilling equipment and open-hole drilling being inappropriate beneath the in-service existing footings, non-percussive rotary boring with synthetic mud flush was used to drill the 225 mm diameter micropile holes. Given the relatively high magnitude of loading and the necessity for the micropiles to be slender and soil-bonded, post-grouting using the most advanced method – tube à manchette with double gland packer – was used to post-grout the entire bond zone. With the post-grouting capability established as a key component of the micropile design, combined with the fact that hoisting of heavy components would prove problematic given the cramped quarters in which most of the piles were constructed, the micropile cross section was designed as a 4-32ø / 517 MPa threaded bar arrangement surrounding a 67ø tube à manchette, and the micropiles were designed without any permanent casing.

5.3 Sacrificial Pre-production micropile

It was self evident due to physical constraints that load testing of any production micropile was completely impracticable, and accordingly, a representatively constructed, sacrificial, pre-production test micropile was installed and loaded in cycled static compression in increments of 250 kN to a test load of 3000 kN, or 300% of micropile design load. Post-grouting of the 16 m deep test micropile was performed in two passes, with the first injection taking place between 12 and 16 hours after tremie grouting and the second pass 6 hours after the first. Injection pressures and grout takes were monitored and recorded for every sleeve during both passes. The low frequency of testing, especially in light of the fact that these were soil-bonded micropiles, was offset by the high magnitude of loading. The test micropile succeeded in resisting the test load; load test results are detailed further in Table 1.

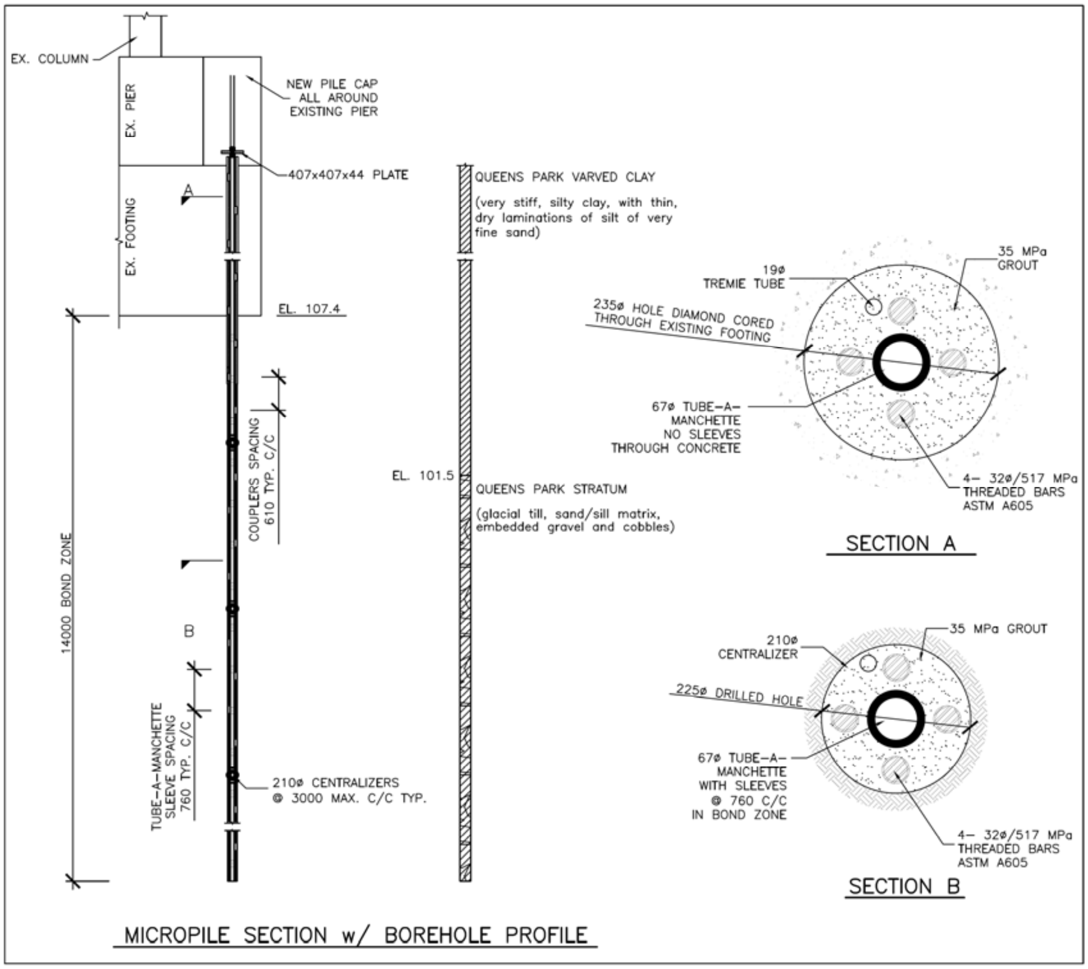

FIGURE 3. 130 Bloor Street West Project - Sub-surface profile and configuration of pre-production micropile

6 CASE STUDY # 4 – SKYY CONDOMINIUM, 2008

Minto Skyy is a glass-clad 23-storey tower that overlooks downtown Toronto from its perch atop the walls of the Don Valley. The tower occupies a site that, before construction, sloped so steeply that the excavation support system varied in terms of height of retention from 17 m at the Broadview Avenue frontage to just 7 m at the northern extent of the Pottery Road frontage. The original foundation design called for spread footings throughout, but a changed condition discovered during excavation necessitated the introduction of large diameter drilled shafts, in groups of three, to support loads in the south and southwest portions of the tower footprint. When the same changed condition necessitated the emergency installation of augmentative, large diameter corner bracing to arrest troubling movements at the southwest corner of the excavation support system, a new restrictive headroom condition was created and the caissons in this area were replaced with micropiles.

6.1 Sub-surface Conditions

The majority of the tower footprint is underlain by dense clay till capable of supporting pad footings of 700 kPa bearing. At the location of the micropiles, the ground conditions varied significantly relative to the balance of the tower footprint, with layers of soft, loose, clayey silt / silty clay as deep as 9 m below underside of pile cap, and the top of the groundwater table within 2 m of underside of pile cap. Relative to underside of pile cap at 105.0 m (geodetic), the depth to the competent soil ranged from 7 to 10 m, after which the soil conditions transitioned very quickly to hard clayey silt till.

6.2 Micropile Design Approach

Beyond the fact that the micropiles would be soil-bonded and not rock-socketed, two key drivers of the micropile design were immediately self evident: restricted headroom and the thick soft soil layer. The presence of the corner bracing meant that at least half of the micropiles would have to be constructed directly beneath an overhead obstruction at 5.5 m above drilling grade. The 1000 kN compression service load and the presence of the 6 to 9 m thick soft layer made certain that the micropiles would all feature a substantial casing over at least their uppermost 8 m of embedment. The final driver of the design was the close pile-to-pile spacing required in order to arrange a sufficient quantity of piles beneath the heaviest loaded pile caps: this factor, combined with the firm to stiff character of the bond zone soils, meant that the process used to drill the bond zones had to be selected to avoid unduly energizing any freshly grouted pile while in the act of drilling its neighbour pile.

6.3 Sacrificial Pre-production micropile

One representatively constructed, 20 m deep, pre-production sacrificial micropile was constructed and load tested to 2000 kN. The installation process involved three distinct steps. For the first step, 10 m of permanent casing was advanced using double-head duplex drilling method, with 245ø / 550 MPa, left-hand threaded-joint casings rotated by the upper drill head and a 203ø drag bit, vertically offset relative to the casing tip in order to maintain a protective soil plug ahead of the drag bit, rotated in right-had rotation by the lower drill head. Cuttings were flushed clean using compressed air delivered through the drag bit. For the second step, the 203ø bond zone was drilled beneath the casing tip using synthetic polymer mud flush. For the third and final step, a single 57ø / 517MPa threadbar, complete with centralizers and tremie grout tube, was installed in the mud-charged hole, tremie grouted until all mud was expunged from the hole and then the pile was pressure grouted through the casing until grout was forced to the surface on the outside of the casing. The sacrificial test micropile was successfully load tested to 2000 kN (200% of design load); load test results are listed in Table 1.

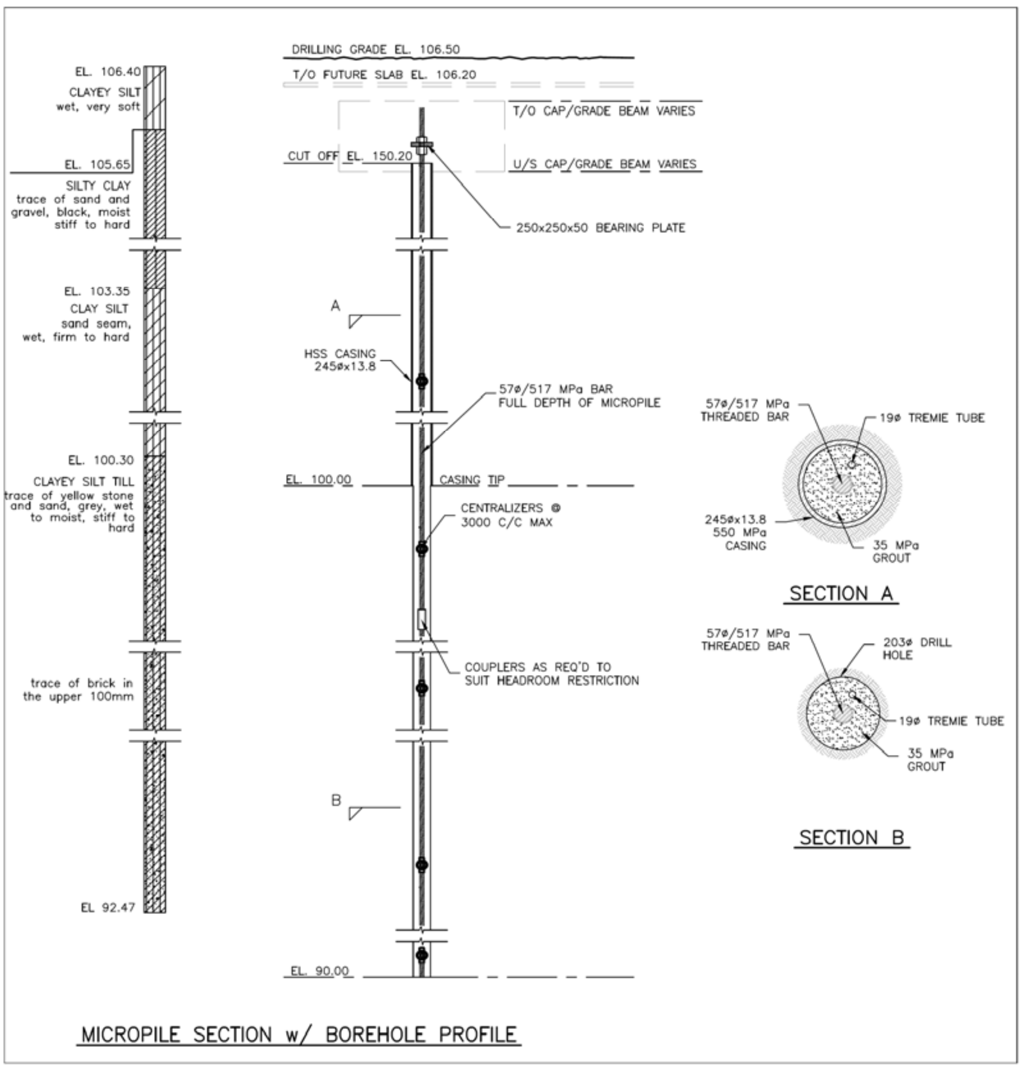

FIGURE 4. Skyy Condominium Project - Sub-surface profile and configuration of pre-production micropile

Scheduled for completion in 2014, the Hullmark Centre project is a large commercial-residential development featuring two office towers, 5 and 7 storeys respectively, and two condominium high rise towers, 37 and 45 storeys, respectively. The northeast corner of the site is co-owned between the developer and the Toronto Transit Commission (TTC), where it overlaps a buried wye railway tunnel and emergency exit building. Both TTC structures were required to remain in service throughout construction. Although street level at this area will become a pedestrian boulevard, a 2-storey mezzanine will overhang this space and be founded on micropiles. Also located at the northeast corner is a new pedestrian tunnel to link the existing TTC Sheppard Station with the Hullmark Centre P3 parking level, and this tunnel will also be founded on micropiles. Beyond knowing of micropiles' existence in the local marketplace, very little was known about micropiles, and soil-bonded micropiles in particular, by the developer, the developer's various consultants, and the TTC. Ultimate approval of the entire proposed development was held up for several months during 2008-2009 by the TTC on account of its lack of comfort with the viability of micropiles at this site, both in terms of available capacity and means by which the micropiles could be constructed to ensure a complete absence of load transfer from the micropiles onto the TTC's several and various existing buried structures. At this juncture, seemingly stalemated, the developer commissioned the design, installation and testing of a prototype micropile.

7.1 Sub-surface Conditions

Micropiles cut off elevations range from 167.2 m (geodetic) to 173.0 m. The undersides of the various TTC structures terminate at 162.0 m. In the vicinity of the wye structure, relative to elevation 171.0 there is 3 to 4 m of fill, underlain by 5 m of dense brown sand, underlain by 7 m of hard grey sandy silt transitioning to clayey silt, underlain by very hard clayey silt till with gravel inclusions.

7.2 Micropile Design Approach

The owner's hope for the prototype micropile was to prove that an 800 kN service load micropile could adequately perform with respect to stiffness and capacity. The micropile contractor set out to prove at least 1200 kN and hopefully as great as 1600 kN allowable compressive loading. In addition to these goals, it was absolutely non-negotiable in order to satisfy the TTC that the micropile must also transfer no load above the horizon coincident with the underside of the TTC's various buried structures. Considering that the site of the proposed permanent micropiles included a hornets' nest of buried services amongst and above the buried TTC structures, there was certain value in proving a higher micropile capacity, because this could be parlayed into reducing the quantity of micropiles, and thereby the number of attempted penetrations through and past the buried elements, required to construct the project. With no bonding allowed above 161 m and some of the cut off elevations as high as 173 m, a thick-walled permanent casing had to be incorporated into the design, both to double as a means of stiffening the unbonded portion of the micropile and, by virtue of the annular space between the outside of the casing and the hole wall, as a means of unbonding the upper reaches of pile using an incompressible but very low shear strength bentonite-cement grout. With the design load possibly as high as 1600 kN, a single 75ø /517 MPa threaded bar was used as the full length reinforcement and, for relatively little added cost and potentially significant benefit, a tube à manchette post grouting system was embedded in the pile for aggressive post grouting of the bond zone. In consideration of the fact that the micropiles would be drilled near and beneath existing structures, the holes were drilled using non-percussive rotary boring with mud flush, with a 305ø hole size to enable a minimum 13 mm annulus after insertion of the 273ø casing.

7.3 Sacrificial Pre-production micropile

Construction and testing of the prototype micropile was undertaken on an early works basis, prior to any other construction activity on the site. As such, the micropile was installed from top of pavement at an operating parking lot, at 174.7 m and the pile was cased to a depth of 15 m to ensure that the 10 m bond zone was drilled in a suitably representative soil layer. Load test results are listed in Table 1. The test was successful in proving both the high capacity of the soil-bonded micropile and the purely elastic compression of the cased portion of the pile under applied loading, which proved that the uppermost portion of micropile could indeed be constructed in a manner that enabled no load transfer to the surrounding soil. The prototype design was adopted for use as the permanent micropile design.

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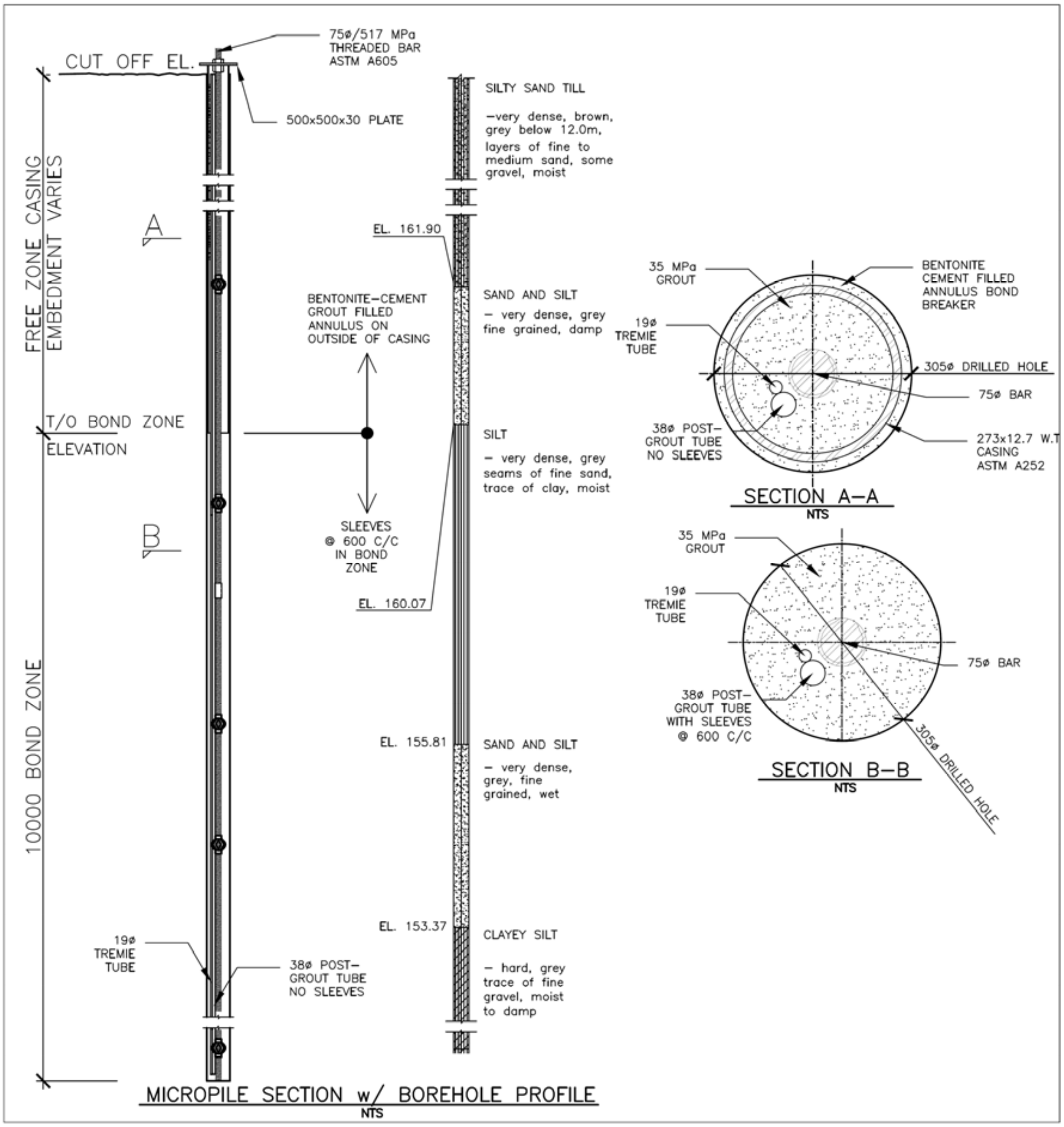

FIGURE 5. Hullmark Centre Project - Sub-surface profile and configuration of pre-production micropile

8 RESULTS

The results presented in Table 1 clearly demonstrate the viability of high capacity micropiles, both rock-socketed and soil-bonded, in the Metropolitan Toronto Area. A significant amount of data has been acquired over the past decade with respect to the performance of various types of micropiles, bonded into various soil and rock types, based on pre-production load testing to high magnitude loading in cycled static compression.

Specialty contractors are acquiring the best available tooling and utilizing most current grouting practices in order to construct micropiles that can be demonstrated to achieve higher than expected grout-to- ground adhesion via successful high magnitude load testing. Although the results shown in Table 1 may seem to demonstrate that the USFHWA estimates of capacity per general soil type are excessively conservative, it must be noted that absolute best practice, in terms of drilling and grouting methods, was employed in constructing the test micropiles whose load test results are detailed in this paper. As such, although the USFHWA estimates may perhaps be somewhat outdated, more time is required for the market to develop further before any change to the published estimates will be warranted.

TABLE 1: Load test results from 5 Toronto micropile projects

PROJECT	TIP TOP	AGO LT1	AGO LT2	130 BLOOR	MINTO SKYY	HULLMARK
Type of load test	Cycled static Compression	Cycled static compression	Cycled static compression	Cycled static compression	Cycled static compression	Cycled static compression
Load test arrangement	Production pile tied down with 2 production piles	Sacrificial pile cap supported by 2 sacrificial micropiles tied down by 4 rock anchors	Sacrificial micropile tied down by 4 rock anchors	Sacrificial micropile tied down by 4 sacrificial tension micropiles	Sacrificial micropile tied down by 4 sacrificial tension micropiles	Sacrificial micropile tied down by 4 post-tensioned rock anchors
Primary reinforcement	2-63 Ø / 550 MPa bars full depth of pile	194 Ø x 10.9 / 550 MPa casing (upper); 2-57 Ø / 655 MPa threadbars (lower)	273Ø x 13.8 / 550 MPa casing (upper); 3-57Ø / 655 MPa threadbars (lower)	4-32 Ø / 517 MPa threadbars	57 Ø / 517 MPa threaded bar	75 Ø / 517 MPa threaded bar
Total Micropile Embedment (m)	11.5	23.5	23.5	16	16.5	25
Bond zone length and diameter (m / mm)	3.5 / 200	4.5 / 152	4.5 / 235	14.0 / 203	10.0 / 203	10.0 / 305
Bond zone medium	Rock socket in shale	Rock socket in shale	Rock socket in shale	Dense glacial till	Hard clayey silt till	Dense glacial till
Grouting method	Gravity (tremie)	Gravity, with limited-quantity pressure grouting through casing	Gravity, with limited-quantity pressure grouting through casing	Gravity grouting, followed 12 hours later by 2 passes of targeted post grouting	Gravity, with limited-quantity pressure grouting through casing	Gravity grouting, followed 12 hours later by 2 passes of targeted post grouting
Design Load (kN)	2000	1724	2847	1000	1000	1200
Max. Test Load (kN)	4000	3448	4700	3000	2000	3600
Max. Test Load (% DL)	200%	233%	246%	300%	200%	300%
Micropile Failure Mode and Load	No pile failure	No pile failure	Plunging at 7004 kN	Creep failure at 3000 kN	No pile failure	No pile failure
Failure Load (% of Design Load)	N/A	N/A	246%	300%	N/A	N/A
Max. bond stress achieved (kPa)	1822	1893	2165	305	317	376
FHWA bond stress values for similar sub-surface conditions (kPa)	205-550	205-550	205-550	120 - 335	50-120	95 - 240
Hold time at Test Load (mins.)	60 (2.0 DL)	60 (2.0 DL)	60 (2.0DL)	60 (3.0 DL)	60 (2.0 DL)	60 (3.0 DL)
Creep (6 to 60 mins) at Test Load (mm)	0.9 (2.0 DL)	0.9 (2.0 DL)	1.04 (2.0 DL)	2.6 (3.0 DL)	1.00 (2.0 DL)	0.65 (3.0 DL)
Gross displacement at 1.0 DL (mm)	4.7	12.6	19.7	5.2	7.7	9.6
Gross displacement at Test Load (mm)	12.7 (2.0 DL)	36.7 (2.0 DL)	44.7 (2.0 DL)	49.4 (3.0 DL)	32.9 (2.0 DL)	38.12 (3.0 DL)

9 DISCUSSION AND CONCLUSIONS

Static compressive load testing of sacrificial micropiles provides data with respect to stiffness and ultimate capacity that simply cannot be gleaned from data compiled during proof testing of production micropiles in tension. Although compressive testing is many factors higher in terms of cost relative to tension testing, there is no comparison with respect to the quality of information obtained. Especially in the case of soil-bonded micropiles, a sound case can be made for justifying the higher cost associated with compression testing, simply by recognizing that the data truly represents how a similarly constructed production micropile will behave when in service, rather than inferring its behaviour with several arbitrarily assigned correction factors.

The technical and economic advantages associated with the use of micropiles have contributed to the increasing popularity of this formerly obscure technology. The load testing successes enjoyed in the local Toronto marketplace have contributed to the acceleration in micropiles' acceptance by large local buyers such as CN, the Ontario Ministry of Transportation and Ontario Power Generation

ACKNOWLEDGEMENTS

The authors would like to acknowledge the contributions of Nadir Ansari and Daniela Xavier of Isherwood Associates during the years 2002 through 2009 to the projects described herein as well as numerous other micropile design, construction and testing endeavours undertaken by the authors during that time.

Our thanks also go to all the drill rig operators, grout plant operators, labourers and field engineers from Geo-Foundations Contractors Inc. who contributed to the safety and successes enjoyed on the projects described in this paper.

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