MICROPILES AT THE TRANSFORMATION AGO PROJECT

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The Transformation AGO project, constructed between 2005 and 2008, featured a foundation trade contract that included design of pile caps and grade beams and construction of 138 permanent micropiles. The new foundation works were constructed within seven different settings – each featuring a host of attributes challenging to constructability – inside and outside the existing Art Gallery of Ontario, all while the gallery remained open for business. Transformation AGO resulted in several firsts for micropiles in the Toronto construction market: this was the first micropile trade contract with a multimillion dollar contract value; it featured the first static compressive load test performed beyond 7000 kN on a single micropile; the first static compressive load test beyond 8000 kN on an experimental pile cap supported by two test micropiles; and it was the first publicly funded project to go to tender with micropiles listed as the preferred foundation option. This paper outlines the successful application of micropiles at the landmark Transformation AGO project.

INTRODUCTION
Toronto’s Art Gallery of Ontario, eager for new exhibit and storage space and a freshening of its image, embarked in 2000 on an ambitious scheme to achieve these goals. Structural engineering work was begun in 2001, a construction manager was hired in 2002 and the foundation design was developed and re-developed until late 2005 when the first trade contract was tendered.

A daunting challenge facing the project’s structural engineers from the start was how possibly to conceive and develop a constructible foundation design, given the maze of existing, mostly shallow-founded, structures in, below and around which the new works would be constructed. This challenge was very well suited to micropiles, but micropiles were, at the outset of project design in 2001, still nowhere near established in the Toronto market. This problem was overcome by regular consultation right up to time of foundation tender between the structural engineer and the eventual micropile contractor, and aided in the meantime by a growing list of successful local micropile projects in Toronto. A second problem facing the team had to do with procurement: in order to qualify for crucial public funding, a minimum of three tenders had to be received for every trade tender, but as of September 2005 there still was only one local foundation contractor truly capable of constructing the type of micropiles required. This problem was overcome by issuing tender drawings depicting two distinct foundation options: a micropile option and a conventional option featuring drilled shafts (outdoors) and hand dug shafts (indoors). As a natural consequence of going to tender with two completely disparate foundation options, the responsibility for design of the network of connecting grade beams – no small task considering the interwoven nature of new and existing foundations – was downloaded onto the foundation trade. These accommodations succeeded in getting three foundation bids tendered and, eventually, the micropile scheme described herein to be awarded.

The eventual project, called Transformation AGO, featured 138 micropiles constructed between October 2005 and June 2006 in a wide variety of challenging physical settings inside and outside the existing gallery.

PROJECT SETTING
Existing Structures
At the outset of the Transformation AGO project, the existing art gallery was housed within a conglomeration of structures dating, respectively, to the 1920s, 1970s and 1990s, all woven within and around one another. The original 1920s era footprint occupied the southern half of the site and consisted principally of a 2-storey, cast-in-place concrete and brick masonry structure on spread footings. The 1970s and 1990s portion occupied the north half
of the property, as well as surrounding the 1920s portion on the southeast and southwest.

Construction of the Transformation AGO works required new, deep foundations to be constructed within all of these existing areas. To assist with identifying the new works, the micropile contractor divided the site into seven designated work areas, each corresponding to its own distinctive elevation or particular physical setting. These designated areas are shown on Figure 1 and their respective physical attributes are listed in Table 1.

![FOUNDATION KEY PLAN](image)

**FIGURE 1: FOUNDATION KEY PLAN**

**Subsurface Conditions**
Top of sound rock is located at Elevation 77.0 msl across the entire AGO site. Micropile work was staged from several different elevations, corresponding to prevailing or finished floor on grade at the respective work areas. Prevailing grade at the highest elevation work area was at 96.8 msl; at the lowest work area 88.0 msl.

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, as shown on Figure 2. Groundwater elevation was coincident with the top of the sand layer at elevation 85.0 msl, 12.5 metres above micropile tip elevation.
### TABLE 1: SUMMARY OF WORK AREA DESIGNATIONS

<table>
<thead>
<tr>
<th>Area</th>
<th>Name</th>
<th>Surface Elev.</th>
<th>Micropile Serviceability</th>
<th>Physical Setting</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Dundas Frontage</td>
<td>96.8 msl</td>
<td>+2028 kN / -490 kN</td>
<td>Battering of micropiles required to avoid multiple existing sewers at Elev. 90.0 msl. Pile arrangement and caps designed on the fly in response to field-generated daylighting of existing sewer positions.</td>
</tr>
<tr>
<td>B</td>
<td>South Tower – outdoors</td>
<td>94.2 &amp; 93.3 msl</td>
<td>+2846 kN</td>
<td>Open headroom, but within the remnants of partially demolished former atrium.</td>
</tr>
<tr>
<td>C</td>
<td>South Tower – indoors</td>
<td>94.2 msl</td>
<td>+2490kN / -600 kN</td>
<td>Headroom restricted to 2.8 metres; work took place within 4.0 m wide hallways, in close proximity to existing spread footings.</td>
</tr>
<tr>
<td>D</td>
<td>Walker Court</td>
<td>94.2 msl</td>
<td>+1335kN</td>
<td>Headroom of 2.8 metres; accessible only via 3.0 m wide hallway</td>
</tr>
<tr>
<td>E</td>
<td>Sub-basement</td>
<td>88.0 msl</td>
<td>+956kN</td>
<td>Accessible via restricted-weight service elevator; headroom 4.5 m</td>
</tr>
<tr>
<td>F</td>
<td>Concourse</td>
<td>92.8 msl</td>
<td>+1579kN / -925 kN</td>
<td>Headroom 3.5 m; accessible only via 3.0 m wide hallway</td>
</tr>
<tr>
<td>G</td>
<td>Stair #8</td>
<td>93.9 msl</td>
<td>+556kN / -556 kN</td>
<td>Work area consisted of 3.0 m x 7.0 m pre-cast concrete stair core, accessible only via 1.5 m x 0.8 m doorway</td>
</tr>
</tbody>
</table>

**FIGURE 2: SUBSURFACE PROFILE**

**Above-grade Conditions**
The area presenting the most challenging above-grade physical setting was Area C, where the available headroom was only 2.8 metres and the micropiles – the largest on this project at $P_{\text{service}} = +2847 \text{ kN}$ – were built within such close proximity to existing spread footings that partial underpinning of some footings was required before the new pile caps could be constructed.

Next in terms of the most challenging setting was Area B, where the micropiles were...
constructed within the remnants of the former south atrium basement. Although all of Area B was open to the sky, the worst of the micropiles in this area had to be drilled just 0.65 m away from the existing, remaining concrete foundation wall retaining a grade separation of 2 metres. Moving from one pile location to the next in this area was always a challenge due to the presence of so much to-be-reused existing superstructure.

In order to access Area E in the sub-basement, all micropile materials and equipment had to be transported via the existing service lift, with a weight restriction of just 7000 kg. Micropiles located in Area F were constructed in headroom of 3.5 m and the area was accessible only by transit through a 3.0 metre wide hallway. Area G was located inside a precast concrete stair core accessible only via a 2.0 m high x 0.9 m wide doorway.

MICROPILE DESIGN CONSIDERATIONS
The micropile design was governed by the serviceability requirements at the South Tower (Areas B and C), where the compressive service loads acting at the base of the columns were as great as 8006 kN, and the available footprint in which to construct pile caps limited the quantity of micropiles to a maximum of three per cap. Consequently, no consideration towards a micropile design could begin without first acknowledging that micropiles of individual axial compressive service loading of > 2700 kN would be required.

Micropile Geotechnical Design
Regardless of the depth to rock, the high magnitude pile loading necessitated a rock-socketed micropile design. In order to keep the geotechnical design basis consistent throughout the site, all piles were socketed in sound shale bedrock with no consideration given to any transfer of load into the weathered rock or soil above the top of the rock socket. Although the plunge length was purposely ignored, the design called for pressure grouting through the top of the casing in order to engage, to the best degree possible, this competent ground for the purpose of stiffening the piles’ geo-structural response to superimposed load. Furthermore, this ignoring of the plunge length resulted in a suitably conservative structural design, in that the structural cross section immediately below the casing tip was capable of resisting the entirety of the service load at the design factor of safety. This aspect conveniently guarded against the possibility of any pile installation where, despite best efforts, negligible pressure grout was able to be placed.

Micropile Structural Design
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 metres) 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 metres) 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, as shown in Figure 3. By employing this approach,

- Single-head percussive duplex with air flush could be selected as the means of constructing a suitably stabilized borehole, installed by suitably small, electric-powered drill rigs since a great deal of the installation energy required was supplied by a remotely located diesel-powered air compressor, hence reducing the amount of torque required from the rotation motor;
- The high cost of having to frequently splice a threadbar bundle could be replaced by the comparatively low cost of frequently splicing a permanent casing instead;
- A proportionally higher cross sectional area of steel could be installed in a reasonably small diameter hole throughout the upper portion of the micropile;
- The process of drilling the micropile hole could be doubled up with the installation
of the primary reinforcement of the upper portion of the micropile, thereby economizing on micropile construction effort and duration;
• The need for retracting casing out of the hole for re-use was eliminated.

The micropile structural design conformed to (for the most part) the recommendations of the USFHA Implementation Manual, with the notable exception of factor of safety on strength of steel in compression, which, at 2.00, was instead in line with local practice.

FIGURE 3: TYPICAL MICROPILE SECTION FOR A SERVICE LOAD OF +2847 kN
As a means of keeping the drill hole suitably small, the design was further enhanced by use of 655 MPa threadbars instead of 517 MPa, and high performance 50 MPa grout. Also done for the sake of minimizing hole size, sacrificial steel – computation of the steel cross sections’ contribution to pile strength 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.

The final, but by no means least significant, aspect of the micropile design to be considered was the anchorage. Remembering that the owner’s only means of tendering the foundation trade contract (with equal consideration given to conventional deep foundation systems) resulted in the design of pile caps and grade beams becoming the responsibility of the micropile contractor, there was decidedly more designer’s license available to the micropile contractor in conceiving a micropile anchorage design. This aspect was seized upon by the micropile contractor as a unique opportunity to buck conventional protocol and submit a micropile design that featured, albeit only for piles with compression-only serviceability, no anchorage other than the 0.15 m projection of the permanent casing (full to the top with 50 MPa grout) into the pile cap. As this design was a clear departure from accepted protocol, a full scale load test to prove the acceptability of the anchorage design was incorporated into the micropile load testing regimen.

MICROPILE CONSTRUCTION

Equipment

Work in outdoor areas was completed using diesel-powered drill rigs with articulating boom masts on all-terrain crawler chassis. Work in indoor areas was completed using a pair of electric-hydraulic mini rigs on fixed frame padded-cleat crawler bases. Eight micropiles within particularly restricted access areas were completed using a remotely located electric-hydraulic power pack and a mast-only apparatus bolted to the existing floor. The electric powered grout plant featured a Colcrete-type high-shear colloidal mixer, 300 litre capacity mechanically agitated hopper and a Moyno 2L6 pump.

Installation Method

Permanent casings were advanced using single-head percussive duplex with concentric underreamer and compressed air flush. With the exception of the particularly restricted access piles in Area G, all of the casings were installed shoeless, rotating in the same direction as the inner drill string with the under-reamer cutting ahead of the casing tip. In Area G, a shoe-drive system installing left-hand threaded casings was employed to enable the use of a significantly smaller rotation motor to turn the drill string. The rock socket threadbars were lowered into place by a light-gauge steel cable twinned to the tremie tube and left in place after grouting. Upon completion of tremie grouting, pressure grouting was performed by injection of grout through the top of the casing until a predetermined volume of 90 litres (the volumetric yield of 5 bags) of pressure grout was injected.

Materials

Three different sizes of permanent casings were used: 273 mm Ø x 13.8 mm wall, 194 mm Ø x 10.9 mm wall, and 141 mm Ø x 9.2 mm wall. All permanent casings were new mill secondary, N80 casings of $F_y = 550$ MPa. Casing splices were mechanical, on 0.9 m or 1.5 m or 3.0 m shoulder-to-shoulder spacing, achieved by torquing together the pin x box API-threadform casing ends. Rock socket reinforcement consisted of single, or bundles of two or three hot rolled, continuously threaded bars of $F_y = 655$ MPa, spliced where required with through-type couplings. Micropiles with dual tension-compression serviceability were outfitted with transition couplers above the top of their development length so that the anchorage was connected to the rock socket by continuous mechanical splicing of the primary tension reinforcement. Grout was mixed on site in small batches as required using water and a pre-packaged, proprietary blend of cement, expansion agent, dry superplasticizer and silica fume, packaged in 30 kg bags and mixed at a water-to-cement ratio (by weight) of 0.3 to yield a 28-day design strength of 50 MPa.

LOAD TESTING

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. The geotechnical design basis was well understood – buckling was never truly a concern and the design adhesion of 1.0 MPa had been proven countless times in the well known Georgian Bay shale formation – and the contractor was highly experienced at grouting, so load testing was at negligible risk of failure.
due to an overambitious geotechnical design. This was not necessarily the case for all of the structural aspects of the micropile design however, as it was very much in question whether pile stiffness, given that high strength steel reinforcement and 50 MPa grout were needed in order to meet the minimum factor of safety, would suffice to meet specification. This was truly the criterion on which the testing would succeed or fail. To this end, the contractor was aided greatly by the project specification regarding load testing, where the acceptance criteria with respect to movement were, thankfully, not overly onerous.

Load Testing Specifications
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. The micropile contractor was motivated to make the load tests confirm dual objectives: first, to meet the specified micropile performance criteria so as to avoid any further investment in load testing beyond the minimum two tests, and second, to prove the acceptability of the contractor’s proposed pile cap and micropile anchorage designs.

Although the specifications called for a seemingly small number of load tests (just two tests for a project that, regardless of whose micropile scheme was to be designed and constructed, would feature well in excess of one hundred micropiles), and the stakes for these tests were disproportionately high given the high magnitude of individual pile 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 piles’ 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 micropile load test acceptance criteria were: (reprinted verbatim from Section 02360 of the project specifications)

a) The slope of the creep displacement versus log time plot must be less than 0.08 inches per log cycle at the load level of 2.0 times the design working load \( T_W \).

b) The elastic deformation must correspond to less than 80% of the pile length.

c) The permanent deformation at 1.0 \( T_W \) shall not exceed 0.40 inches.

d) If the pile movements at \(<2 \ T_W \) shows plunging characteristics the load test is not acceptable.

Although the specifications called for load testing in conformance with ASTM D1143, the micropile contractor successfully applied for relief from two aspects of this standard: the hold periods were modified to suit those suggested in USFHWA Chapter 7, and the minimum offset distance of the tie-down anchors was reduced to enable the use of the micropile contractor’s existing load test framing.

Load Test 1
This load test involved applying cycled, static compressive loading to a pair of micropiles via an instrumented pile cap. Four tie-down rock anchors were used to resist the applied loading; each tie-back was fitted with smooth sheathing over its entire overburden embedment length and proof loaded to 2500 kN prior to being put into service as a tie-down anchor. Figure 4 illustrates the arrangement of Load Test 1.

Pile Cap Load Test
The purpose of including the pile cap as part of pre-production testing was to confirm the appropriateness of both the proposed pile cap design and the proposed anchorage design. Although the proposed pile cap design was a clear departure from that prescribed by The Ontario Building Code, there are provisions within the code to recognize such departures if full scale testing is performed to prove the alternative design basis’ performance. The pile cap was instrumented with strain gauges positioned at regular intervals along several of the pile cap reinforcing bars, and a bottom void form was installed to fully load the two anchorages. Load was applied by a 0.65 m diameter hollow core jack capable of applying as much as 12500 kN to a 0.6 m x 0.6 m steel plate cast into the top of the cap. The pile cap ruptured at 8046 kN, after demonstrating that the proposed anchorage withstood an average bearing stress of more than 450% of the tested compressive strength of the pile cap concrete. A detailed analysis of the pile cap design and load
test results can be found in a paper by Sheffield, et al., submitted to the 7th International Workshop on Micropiles, Schrobenhausen 2006.

**Micropile Load Test**
The two sacrificial micropiles supporting the experimental pile cap were designed for axial service compression 1724 kN each, and each consisted of 194\( \phi \) permanent casing as the primary upper reinforcement overlapping (by 4.5 metres) a 9 metre long, 2.57\( \phi \) threadbar bundle in the rock socket. One of the micropiles was outfitted with a SMART Contractometer. Each micropile was cast into the pile cap in such a way as to simulate the production condition, i.e. with an anchorage consisting of nothing more than the 0.15 m projection of the grout-filled pile with no bearing plate or dowels. The micropiles resisted the applied test load without failure.

![FIGURE 4: LOAD TEST ARRANGEMENT](image)

**RESULTS**
Results of Load Test 1 are summarized in Table 2. The micropiles resisted the maximum applied test load without failure, and performed satisfactorily with respect to all four specified acceptance criteria. The micropiles exhibited gross movement at design load of 12.6 mm. The pile cap ruptured – first with a single failure surface projecting to the top of the cap, then violently – during the 10 minute hold period at 8046 kN. The pile cap performance unquestionably proved the appropriateness of both the proposed pile cap and micropile anchorage designs.

**Load Test 2**
Similar to the tie down anchor arrangement employed in Load Test 1, all four tie-down rock anchors were proof loaded to 2500 kN prior to the commencement of Load Test 2. A single, 273 mm \( \phi \) micropile, designed for axial service compression of 2847 kN, was load tested to
7004 kN, at which load the pile head began slowly, but unmistakably, to plunge. Test results met the specified acceptance criteria and are summarized in Table 2. The tested micropile production micropile installation commenced within days of completion of Load Test 2.

### TABLE 2: SUMMARY OF MICROPILE LOAD TEST RESULTS

<table>
<thead>
<tr>
<th></th>
<th>LOAD TEST 1</th>
<th>LOAD TEST 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>TYPE OF LOAD TEST</td>
<td>Static compression</td>
<td>Static compression</td>
</tr>
<tr>
<td>DESCRIPTION OF TEST PILES</td>
<td>Pile cap supported by 2 piles, each 194 Ø x 10.9 mm wall casing + 2-57 Ø 655MPa threadbar bundle in rock socket</td>
<td>Single pile, 273Ø x 13.8 mm wall casing + 3-57 Ø 655MPa threadbar bundle in rock socket</td>
</tr>
<tr>
<td>LOCATION</td>
<td>Area A (Dundas St. W.)</td>
<td>(Area B) South Tower</td>
</tr>
<tr>
<td>DESIGN LOAD (kN)</td>
<td>1724 (per pile)</td>
<td>2847</td>
</tr>
<tr>
<td>200 % TEST LOAD (kN)</td>
<td>3448 (per pile)</td>
<td>4700</td>
</tr>
<tr>
<td>FAILURE LOAD (%) PER TEST</td>
<td>233 (piles did not fail)</td>
<td>246 (pile plunged)</td>
</tr>
<tr>
<td>FAILURE LOAD (kN) PER TEST</td>
<td>8046</td>
<td>7004</td>
</tr>
<tr>
<td>MAX. GROUT-TO-ROCK ADHESION ACHIEVED (kPa)</td>
<td>1893</td>
<td>2165</td>
</tr>
<tr>
<td>HOLD TIME at 200% (mins.)</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>CREEP (6 to 60 mins at 200%) (mm)</td>
<td>0.9</td>
<td>1.04</td>
</tr>
<tr>
<td>GROSS MOVEMENT AT 100% (mm)</td>
<td>12.6</td>
<td>19.7</td>
</tr>
<tr>
<td>GROSS MOVEMENT AT 200% (mm)</td>
<td>36.7</td>
<td>44.7</td>
</tr>
</tbody>
</table>

### DISCUSSION

The successful application of micropiles at the Transformation AGO project marked a crucial step in the rapidly progressive evolution of the micropile market in Toronto. Micropiles – at least the USFHA-inspired type – were completely absent from the Toronto market prior to 2002, but by early 2005 had gained considerable traction via a handful of projects completed with contract values ranging from $35,000 to $700,000 USD. The redevelopment of the Art Gallery of Ontario had been in conceptual development since 2001 and micropiles were part of the discussion from the outset, even prior to any real, local establishment of their legitimacy, thus providing a constant looming presence over every micropile job that was completed prior to the AGO work going to tender. During the design development phase from 2002 to 2005, the micropile design (always the domain of the eventual micropile contractor, but shared freely and openly with the project’s structural designers) changed several times, and virtually every micropile project undertaken by the contractor was designed, constructed and load tested with as much focus towards AGO as towards the respective project itself.

To this end, the micropile design eventually employed at Transformation AGO was born of the lessons learned by the micropile contractor on multiple predecessor projects and from inspiration taken from the Exton Mall case history (Cadden, et al, 2002) where the benefits of using permanent casing as the primary reinforcement were so adroitly conveyed. The success of the Transformation AGO has since been parlayed to its full value into continued, and extremely successful, promotion of micropiles in the local market, so much so that
micropiles have now begun to regularly be considered by Ontario’s Ministry of Transportation, the market’s single largest owner of structures (both existing and proposed) that could benefit most from micropiles.

CONCLUSIONS
The successful high magnitude pile cap load test verified the technical feasibility of a plateless micropile anchorage design. The success of two high magnitude static compressive micropile load tests demonstrated the appropriateness of the composite, varying-section micropile design employed at Transformation AGO.

The Transformation AGO project provides another example that proves, despite micropiles’ well-deserved reputation for costliness, that there are myriad examples of redevelopment works within and around existing structures where micropiles are not just the best suited technical solution, but can in fact also be the most commercially beneficial solution.

With the Toronto micropile market having sustained significant growth in the years since Transformation AGO, the project proved to be a key contributor to its continuing successful evolution. The 4 year time lag between the first consideration of using micropiles and the project going to tender proved to be particularly beneficial to this market evolution, considering that several key lessons were able to be learned on lower profile projects without having a negative effect on the market evolution that undoubtedly would have occurred if the lessons had instead been learned on Transformation AGO. This aspect enabled the success of micropiles at Transformation AGO to be used enthusiastically ever since in the promotion of micropiles in the local market.

ACKNOWLEDGEMENTS
The authors gratefully acknowledge the role of Isherwood Associates (Nadir Ansari, P. Eng., Daniela Ramirez, P. Eng., and Dan Bateson) as micropile designers of record on behalf of the micropile contractor. Pile cap design and anchorage design were done by Peter Sheffield and Associates. The structural engineer on behalf of the owner was Halcrow Yolles, represented by Kari Valle, P. Eng. and Raef Ghalli, P. Eng. The construction manager of the Transformation AGO project was Ellis Don Corporation.

REFERENCES


