Remedial measures incorporating jet grouting and micropiles for the construction of a new back flow preventer

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ABSTRACT
A new back flow preventer (BFP) was constructed as part of Toronto's West Don Lands (WDL) redevelopment in preparation for the 2015 Pan Am Games. Movement of three adjacent existing bridge piers early during conventional construction necessitated an embargo on dewatering, a change from conventional to innovative techniques as well as remedial measures to construct the new backflow preventer. The site is located within the flood plain of the Don River and is underlain by over 30 metres of weak organic deposits. Overlapping jet grouted columns were installed using the double fluid process to create both a low permeability base plug and a vertical cut-off. Jet grouting parameters were verified by conducting a full-scale pre-production test program. Rock-socketed micropiles were installed through the jet grout base plug to support and tiedown the new chamber. Several challenges were encountered during remedial works, namely revisions to the method and sequence required to mitigate further movement of the adjacent bridge piers. Eventually additional micropiles were installed as part of a foundation retrofitting scheme to permanently transfer the foundations of the three existing bridge piers to rock. Jet grouting was successfully applied to construct a base plug and vertical cut-off to stabilize the ground and enable excavation works to be performed in the dry. Micropiles were successfully applied to construct a foundation through the jet grout base plug as well as replacing the compromised existing bridge pier foundations. Full scale pre-production test programs were conducted for both jet grouting and micropiles. Details of the jet grouting, backflow preventer micropiles and bridge micropiles, including test programs and challenges encountered during construction are outlined in this paper.

RÉSUMÉ
Un nouveau dispositif anti-refoulement a été construit dans le cadre du réaménagement de West Don Lands (BNM) à Toronto, en préparation pour les Jeux panaméricains de 2015. Le déplacement de trois piliers de pont adjacents, tôt durant la construction, a nécessité un embargo sur l'assèchement, une évolution des techniques conventionnelles vers des techniques novatrices, ainsi que des mesures de correction pour construire le nouveau dispositif anti-refoulement. Le site est situé dans la plaine d'inondation de la rivière Don et est recouvert par plus de 30 mètres de dépôts organiques mous. Des colonnes injectées par coulis se chevauchant ont été installées à l'aide du processus à double fluide pour créer à la fois un bouchon de base de faible perméabilité et une coupure verticale. Les paramètres d'injection de coulis ont été vérifiés par la réalisation d'un programme d'essai de préproduction à grande échelle. Des micropieux ancrés dans la roche ont été installés à travers le bouchon de la base d'injection de coulis afin de soutenir et d'arrimer la nouvelle chambre. Plusieurs défis ont été rencontrés lors des travaux de réparation, notamment la révision de la méthode et de la séquence nécessaires pour atténuer davantage le mouvement des piliers de pont adjacents. Éventuellement, des micropieux supplémentaires ont été installés dans le cadre d'un plan de réaménagement des fondations des trois piliers de pont pour les ancrer directement au roc. L'injection par coulis a été appliquée avec succès pour la construction d'un bouchon de base et d'une coupure verticale afin de stabiliser le sol et de permettre d'effectuer les travaux d'excavation à sec. Des micropieux ont été installés avec succès pour construire une fondation à travers le bouchon de base injecté par coulis, ainsi que pour le remplacement des fondations de piliers de pont existantes. Un programme d'essai de préproduction à pleine échelle a été mené à la fois pour l'injection par coulis et pour les micropieux. Les détails de l'injection par coulis, du dispositif anti-refoulement et des micropieux de pont, incluant les programmes d'essai et les difficultés rencontrées, sont présentés dans cet article.

1 INTRODUCTION

The West Don Lands (WDL) is a former industrial area in southeast Toronto transformed into a sustainable residential community featuring 6000 new units, commercial space and 9 hectares of public spaces. A section of the development will be used as the Athletes' Village for the 2015 PanAm / ParapanAm Games and is being developed by Waterfront Toronto in partnership with Infrastructure Ontario. The WDL site is located within the Don River flood plain and extensive flood protection has been implemented into the community design, including a 2-chamber back flow preventer (BFP) intercepting an existing 1650 mm diameter storm sewer. The back flow preventer was constructed beneath the area where the King Street and Queen Street bridges converge just west of the Don River.

Construction of the BFP was originally intended to be completed using an internally braced steel sheet pile excavation support system in conjunction with dewatering to enable excavation around and beneath the existing 1650 mm diameter sewer. The depth of excavation required was 12 m below ground surface, with the south sheet pile wall approaching, in plan, as close as 3 m horizontally to the bridge columns. Work commenced in 2012 but was halted soon thereafter due to movements of adjacent existing bridge columns after installation of sheet piles but before any excavation. The project's consultants decided that the original plan could not proceed. An alternative approach was required that could result in the excavation support system installation – and excavation to depth – being completed without dewatering.

Specialty geotechnical contractor Geo-Foundations was engaged to propose a design approach incorporating jet grouting to improve the existing soils surrounding and below the proposed excavation to the extent necessary that excavation support installation and excavation could proceed without initiating further movement of the adjacent bridge piers, all without dewatering. More specifically, jet grouting was used to construct both a bottom seal and vertical cut-offs at each transverse support wall. Jet grouting was selected due to its versatility and ability to be performed in a surgical manner.

Geo-Foundations was also engaged to construct micropiles for two separate aspects of the project – as the permanent foundation system for the new BFP chamber, and as remedial piles to replace the existing timber piles at the bridge piers that had suffered from movements induced by sheet pile installation.

Figure 1 shows a perspective view of the jet grouting, chamber micropiles and bridge micropiles.

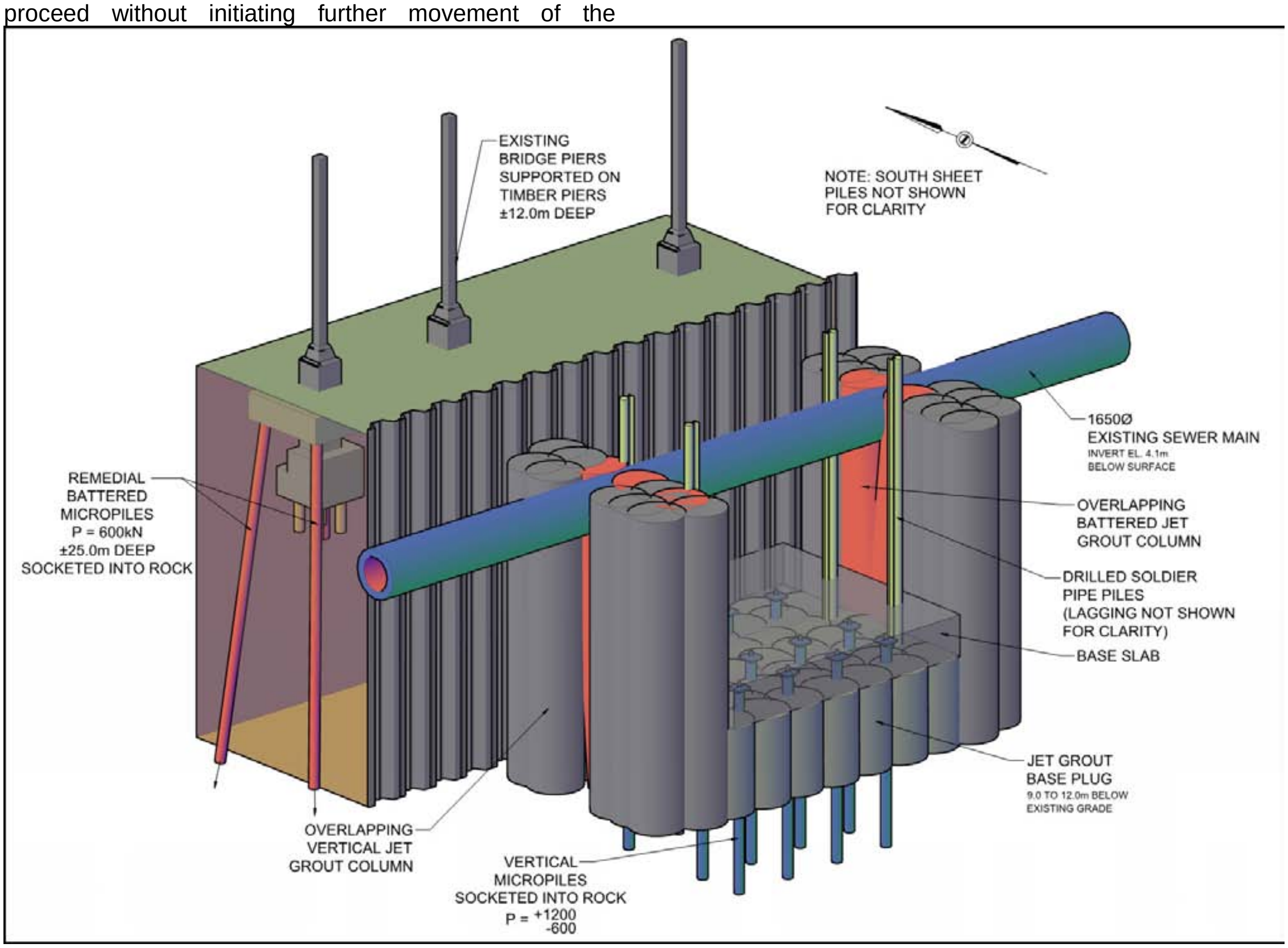

Figure 1: Perspective view of jet grouting, micropiles and soldier piles

2 GEOLOGICAL SETTING

The site is located in the area where the Don River used to flow before being straightened out at the start of the 20th century. The soil profile consists of clayey silt and organic fill from 0-5 m below existing grade. Weak organic silt with SPT 'N' values between 0 and 7 then make up the profile from 5 m to approximately 13 m below existing grade. The remainder of the profile is made up of sandy silt, clayey silt till and highly weathered shale of the Georgian Bay formation at 28 m below grade. Approximately 3 m of weathered shale is present before sound rock is encountered. A dense, wet, sand layer exists at locations closer to the Don River at depths around 20-25 m. Figure 2 shows the subsurface profile at the location of the backflow preventer.

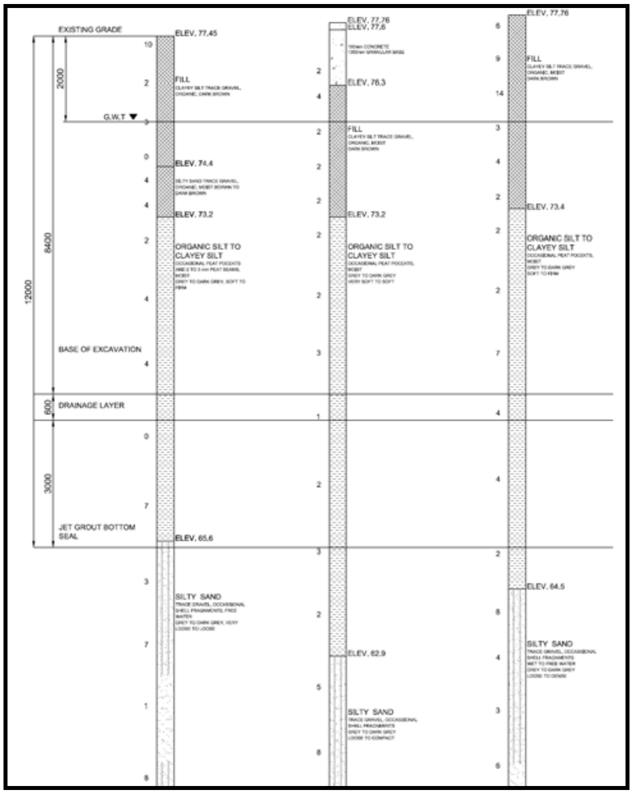

Figure 2: Subsurface profile at work location

3 JET GROUTING

3.1 DESIGN APPROACH

The key driver to the jet grouting design approach was the soft nature of the soils over the entire proposed treatment depth of 15 metres. Coupled with this consideration was the fact that the project was already in crisis, so it was important that the proposed jet grouting method should be something with which the consultants were familiar and with which the contractor had significant local experience (and success). Finally, it was important to use a jet grouting method that could be implemented on a surgical basis to ensure that no further movement of the bridge piers was initiated as a result of jet grouting, and if this did happen, contingency plans could be implemented to continue jet grouting to completion. Based on significant local experience and success, the double-fluid jet grouting method was proposed.

3.2 OVERVIEW

Jet grouting is typically constructed from the bottom upwards. The drill string is advanced to the target depth using non-jetting, typically with water flush or a weak grout mix. The resulting small diameter hole to the bottom of the treated zone sets the stage for jet grouting by creating a passage (upwards through the annular space between the inside of the borehole wall and the outside of the drill string) for evacuation of excess jet grout spoils.

The double fluid process of jet grouting separately supplies grout and compressed air to the bottom of the drill string via separate, concentric passages within the string. Grout is ejected laterally through specially designed nozzles that focus the grout stream for maximum erosive effect. The compressed air meets the grout slurry on the downstream side of the nozzle, shrouding the grout slurry jet (Fig. 3) to further amplify its erosive effect.

Jet grouting parameters such as rotation rate, lift rate, injection pressure and mix design are typically proposed based on the contractors' previous experience in similar ground conditions, before being tested in representative conditions, evaluated for performance and conformance, and eventually selected for, or modified prior to, production jet grouting.

3.3 DESIGN SPECIFIC TO THIS PROJECT

Incorporating the existing sheet pile walls on both long sides of the BFP chamber, the jet grouting layout was designed in order to fully isolate, or "box in", the proposed excavation. Overlapping vertical columns, nominally 12 m high, created cut-off walls on the east and west sides, transverse to the alignment of the existing sewer. These vertical columns were augmented with battered columns arranged to "gouge out" the soils present under the footprint of the existing sewer in order to prevent the possibility of untreated, flowable soil "windows". A 3 m thick base plug, consisting of overlapping jet grout columns installed from 9 mbgs to 12 mbgs, was installed over the entire footprint of the excavation, including intimate contact with the sheet piles and overlapping the vertical and battered cut off columns.

As a preventative measure to avoid basal heave of the existing sewer during jet grouting, PVC sleeve pipes were installed prior to jet grouting at all jet grout locations adjacent to the existing sewer.

The locations and spacing were based on a target jet grout column diameter of 1.8 m and a minimum required column overlap of 150 mm. Target permeability of jet grouted soil, governed by the design requirements for performance of the base plug, was 10^{-5} cm/second.

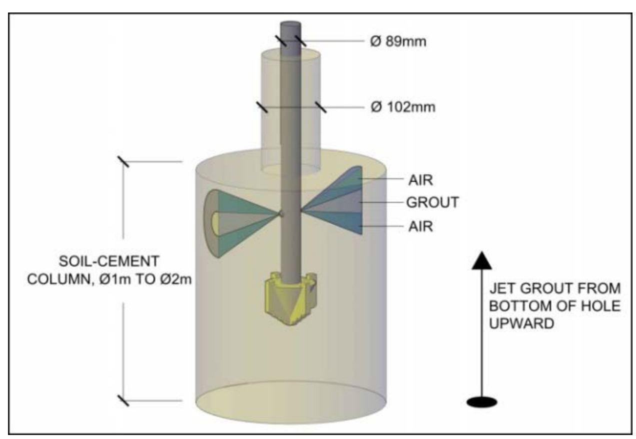

Figure 3: Typical double fluid jet grouting profile

3.4 PRE-PRODUCTION JET GROUT TEST PROGRAM

Pre-production jet grout testing was performed at the site to verify the jet grout parameters that would generate the target column diameter and in situ permeability. A location close to the proposed BFP footprint, but reasonably distant from the now especially movement-sensitive bridge piers, was selected for installation of three overlapping test columns (Fig. 4). The test columns were advanced to a depth of 7 m below ground surface (mbgs) and jet grouting was performed from the bottom upwards from 7 to 2 mbgs.

Alignment surveys were performed on all installed test columns and quality control checks were performed on the grout mix. All jet grouting installation parameters were recorded using the Data Acquisition (DAQ) system on the drill rig.

After a curing period of 48 hours, the columns were exhumed to physically examine the geometric properties within the upper 1m section (i.e. from 2 to 3 mbgs). Core drilling was performed using the PQ-3 system. Core samples were retrieved and subjected to laboratory testing for strength and permeability, and all cored holes were video logged to visually verify the consistency of the borehole wall. A falling head test was performed in the cored hole located at the interstice of the 3 overlapping jet grout test columns.

A summary of the test results is provided in Table 1.0. Satisfied with the results of pre-production testing, the same installation parameters and methodology incorporated into constructing the test columns were utilised for production jet grouting.

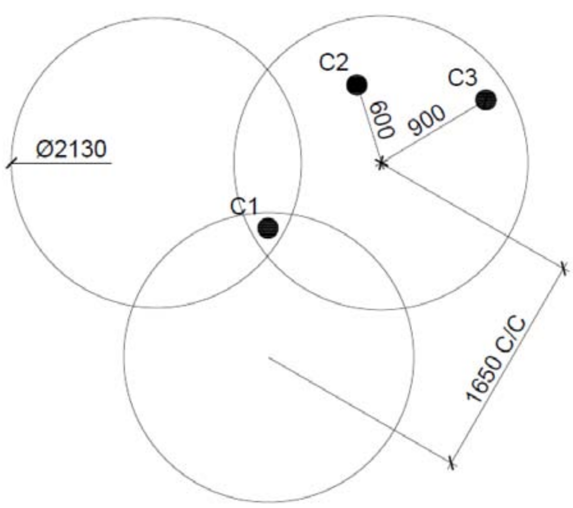

Figure 4: Jet grout test column layout, including coring locations

Table 1.0 : Summary of test results from pre-production jet grout test program

Description	Designed	Average
Column dia. (m)	1.8	2.2
In-situ permeability (cm/s)	1×10^{-5}	1.69×10^{-5}
Unconfined compressive strength after 28 days(MPa)	1	3.2
Specific Energy (MJ/m)	50	50

3.5 JET GROUTING METHODOLOGY

A dynamic process was implemented for establishing the layout of the jet grout columns considering the existing sewer, steel bracing, sheet piles and restricted access. In advance of jet grout installation, all as-built information was incorporated to establish a preliminary scheme to create adequate overlap of the jet grout columns despite the several interferences at various depths throughout the treatment profile. Jet grouting was performed using a specially configured drill rig with on-board data acquisition (DAQ) and control system to perform jet grout installations in a fully automated mode. Grout slurry was batched using a high capacity batch plant and then transferred to the drill string via a high pressure pump capable of producing pressures of up to 100 MPa.

The drill string with the jet grout monitor (the device through which grout is jetted into the ground) was advanced to the target depth and inclination (as required) using a grout slurry and low pressure air as the flushing medium. Installation of each column was completed in a single stroke. Orientation of each drilled hole was surveyed using a Shape Accel Array (SAA) tool. The SAA tool was lowered into the drill rods prior to commencement of jet grouting. Inclination of the drill mast and depth of the drill string were both continuously monitored and recorded by the onboard DAQ system.

After checking the orientation of the hole, jetting commenced. Jetting continued at a typical lift rate of 0.2 m/min, rotation rate of 10 RPM and injection pressure of 40 MPa before being stopped at the top of column elevation.

The section of the hole above the column elevation was tremied with grout. Visual inspection of returning spoils was continuously performed in order to ensure no hydrofracture or hydraulic jacking would occur. All spoils were transferred to a localized containment area by means of a sand guzzler hooked up to the drill rig. Spoils setup to a dense clay consistency within 24 hrs.

3.5.1 Quality Control

Throughout the jet grouting operations, numerous quality control measures were implemented to continuously monitor the jet grouting parameters (i.e. lift, flow, rotation, pressure, air flow, etc.). All DAQ reports from each installed column were reviewed to ensure consistency with the site specific parameters. A typical plot obtained during the jet grouting of a column is shown in Figure 5. Samples of grouting spoils being expelled from the collar of the hole were captured at regular intervals and measured for specific gravity. Spoils samples were also cast into grout cube moulds and sent to an accredited independent testing laboratory for unconfined compressive strength (UCS) testing. Every jet grout column was surveyed and the SAA data was reviewed and plotted in a timely manner to identify potential gaps in the cut-off and base plug. The SAA (Shape Accel Array) tool is a reel mounted unit with 3 MEMS (micro electromechanical systems) in every segment, spaced at 0.5 m intervals. This unit is able to provide real time data of the inclination and orientation with a single shot measurement. A 3-D grouting profile was developed and updated on a daily basis due to the obstructions and complexity in achieving the desired column overlap.

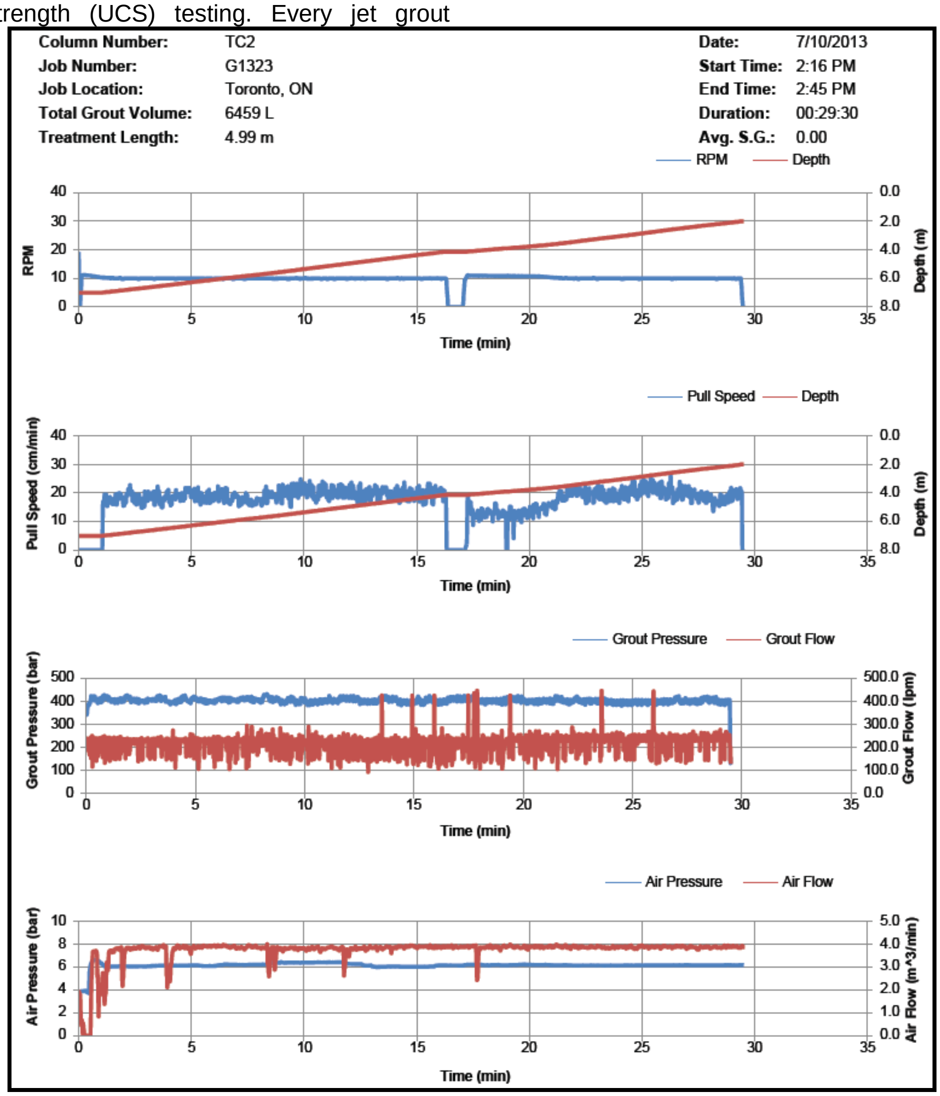

Figure 5: A typical plot obtained from the DAQ system during jet grouting of a column

3.4.2 POST CONSTRUCTION VERIFICATION

Upon completion of the jet grout columns, two P-size cored holes were advanced 11 m below surface to penetrate into the 3 m deep jet grout base plug and falling head permeability tests were performed. The installation procedure of the cored holes was modified from the process used to conduct testing on the pre-production jet grout columns. A P-size casing was advanced 0.5 m into the jet grout base plug and a tremie plug was installed to seal the casing in place. Coring of the 1 m long test section was performed 24 hours after installation of the P-size casing was set in place. The results obtained from the falling head tests performed in the two cored holes are provided in Table 2.0.

Table 2.0: Summary of results from the post-production falling head tests

Hole Location	Permeability (cm/s)
CH-1	2.34×10^{-6}
CH-2	2.75×10^{-7}

4 CHAMBER MICROPILES

When the project first encountered crisis and the decision was reached to change the construction approach to incorporate jet grouting, the project designers had a new problem to tackle in the form of the deep foundation for the cast-in-place concrete BFP chamber. The original scheme of founding the chamber on H-piles driven to rock was no longer feasible given the extreme sensitivity of the bridge piers. The driven H-piles were replaced with twelve rock-socketed micropiles designed to resist both compression and uplift forces, installed from existing grade after jet grouting but prior to chamber excavation.

4.1 Micropile Design Approach

Individual factored loading of each pile was 1200 kN in compression and 600 kN in tension. The poor quality of the overburden soils, and extreme depth to which the poor soils extended, combined with the loading requirements lead to the obvious decision to socket the micropiles in rock. Each micropile was reinforced with a single 57 mm diameter (517 MPa) threaded bar over its entire length concentric to a 178 mm x 13 wall permanent casing extending from surface to a minimum depth of 500 mm into sound rock. The micropiles were installed with a minimum rock socket length (below the tip of the permanent casing) of 5 m in sound rock. Micropiles were designed in accordance with the US Department of Transportation and US Federal Highway Administration, Micropile Design and Construction Guidelines, June 2000, using the load factor design (LFD) method. Given the extreme depth of the poor quality soils beneath the chamber, a buckling check was performed to confirm the adequacy of the casing size and wall thickness based on the governing compression loading case.

4.2 Installation of chamber micropiles

The micropiles were drilled using a double head duplex rotary percussive drilling system. This technique allowed for the retraction of the drill bit into the casing, when required, in order to avoid plugging of the drill rods while in the wet, dense sand layer present at varying depths within the piling profile. The temporary steel casing was advanced up to 500 mm into sound rock to avoid any possible collapse while drilling the rock socket. After the cleaning of the rock socket and casing, the reinforcement was installed with mechanical splices every 7.6 m. Each pile was tremie grouted using a water to cement ratio of 0.45 and once the inside of the casing was full to the top with clean, dense grout each pile was pressure grouted through the top of the casing until the equivalent of 100 litres of pressure grout had been injected. The specific gravity was measured during each pile to ensure a value greater than 1.85 g/cm^3 and grout cubes were taken and sent to an independent accredited testing laboratory for unconfined compressive strength (UCS).

After all 12 production micropiles had been installed, a separate drilling and grouting process was applied to each micropile in order to seal any possibility of leaks through the base plug resulting from each pile's breaching of the plug. Each micropile was over-reamed from surface down to the underside of the jet grout base plug using a 245 mm casing outfitted with ripping teeth and water flush. With the reaming casing in place at the underside of the base plug, grout was injected under gravity head and the casing was retracted. Figure 6 shows the excavation for the construction of the BFP at the target depth.

Figure 6: BFP chamber excavation at target depth

4.3 Proof testing

Balancing the priorities of confirming the micropile design assumptions with the need to keep the project on schedule and keep costs in check, the decision was reached to perform load testing on a production micropile. Given the very poor quality of the soils at existing grade, the presence of numerous obstructions and a freshly constructed jet grout base plug, load testing was performed using cycled static tension on a specially designated production micropile featuring a bond length in rock shortened to half its normal design of 5 m. By using this approach, the magnitude of applied loading could be kept to just 1200 kN while still allowing evaluation of grout-to-rock adhesion at the nominal bond stress at factored compression loading, and the only shortened pile would itself be a proof tested pile (thereby justifying its shorter embedment in rock relative to every other pile). After reaching the test load of 1200 kN, the load was held for 1 hour, over which time it exhibited a creep of 0.61 mm. Unloading of the pile produced non-elastic movement of 4.46 mm. A summary of the test pile and proof test results are presented in Table 3.0.

Table 3.0: Summary of chamber micropile static tension proof test results

Load (kN)	Movement (mm)	Creep from 1 to 10 mins. (mm)
600	9.93	0.07
1200	24.47	0.13

5 REMEDIAL BRIDGE MICROPILES

The final problem arising from the original bridge movements was finding a long term solution that would ensure the integrity of the settled bridge piers. The solution to this problem consisted of constructing battered micropiles connected to new pile caps beneath each of Piers M37, M39 and M42.

5.1 General Approach

During micropile construction the bridge piers continued to be supported on their original timber pile foundations. Consequently, micropile construction had to be regulated and closely observed to ensure the ongoing stability of the bridge, especially during drilling through the sandy layers where the existing timber piles were thought to be terminated. Once the micropiles were completed and ready to resist load, the new pile caps were to be cast in place, but not yet connected to the existing pile caps until such time as the new pile caps developed sufficient strength to take the entire bridge loading. At this time the bridge deck was to be shored in place (using the new pile caps for foundation), the piers disconnected from their original foundations, the piers jacked upwards to restore the deck to its pre-settlement profile, and finally the piers were to be permanently connected to the new micropile-supported pile caps.

5.2 Micropile Design Approach

Each micropile was designed to resist a moderately light compressive load of 600 kN. However, in order to be arranged in a manner that accommodated the existing pile cap, every remedial micropile was battered at an angle to vertical of between 1H:5V to 1H:6V and consequently, due to the extreme depth of very poor soil the micropile design was governed by buckling resistance. The project consultants performed the buckling analysis and mandated that the contractor-designed micropiles have a minimum stiffness of 3860000 kN.

In order to meet the stipulated stiffness, a 273 mm diameter permanent casing with 15.1 mm wall was incorporated into the micropile design. This casing doubled as the means by which the rock socket could be protected from cave in from above, and was socketed a minimum of 500 mm into sound rock. Micropile reinforcement consisted of a 76 mm diameter (517 MPa) threaded bar over the lowermost 6.5 m of micropile, connected mechanically by a transition coupler to a 57 mm diameter (517 MPa) threaded bar extending upwards to the top of the micropile. Micropiles were designed in accordance with the US Department of Transportation and US Federal Highway Administration, Micropile Design and Construction Guidelines, June 2000, using the load factor design (LFD) method.

5.3 Pre-Production Load Test

Recognizing the importance of verifying that the design could meet the strict deflection under load criteria (given that the new micropiles were to take the entirelty of the bridge loading at 3 piers), a sacrificial, pre-production micropile was constructed and load tested in static compression. The test pile was installed vertically to enable the compression loading to safely be resisted by a frame tied down with 4 sacrificial, rock-socketed tension micropiles. The test micropile was socketed 3 m into sound rock and was incrementally loaded to 1800 kN (i.e. 3 times the design load) with acceptable performance across all criteria. A summary of results from the pre-production load test is presented in Table 4.0.

Table 4.0: Summary of pre-production test pile and load test results

Test load (kN)	Movement (mm)		Creep from 1 to 10 mins. (mm)	
	Allowable	Measured	Allowable	Measured
300	6	1.19	1	0.02
600	N/S	3.97	1	0.09
1200	N/S	10.02	1	0.06
1800	N/S	15.43	1	0.01

N/S – not specified

5.4 Installation of remedial bridge micropiles

The production drilling was completed using a low-mast drill rig in order to fit beneath the restrictive headroom and not interfere with the existing bridge piers. A double head rotary percussive drilling system was used to advance the micropile casings through the overburden and into sound rock. Caution was exercised throughout the drilling process to minimize any movement of the bridge piers. A real-time bridge monitoring system was in place during the drilling operations. As soon as any movement was observed, the installation sequence and methodology were modified if necessary. During installation of the majority of the remedial micropiles, particularly while the casing was passing through the dense sandy layer at 15 mbgs, the drilling process was modified, by necessity, to include tremie injection of a head of synthetic polymer drilling mud prior to any attempt to splice on the next segment of casing. This modified process was used in sensitive areas in order to additionally stabilize the hole so that the drill casings could be spliced without risk of native soils piping or boiling into the casing and plugging off the drill bit.

5.5 Connecting the micropiles to the bridge

The original remedial micropile design called for 14 production micropiles – 4 at Pier M37, 4 at Pier M39, and 6 at Pier M42. Despite the best efforts of the micropile crew, piling-induced movements continued and the micropile construction had to be suspended on more than one occasion. During one stoppage in the work, the decision was made to proceed with construction of the new pile caps so that these, despite the fact that not every micropile was yet successfully constructed, could be used to support new temporary shoring connected to the bridge deck. This approach paid dividends as the micropiling was able to be re-commenced and completed without another stoppage in work. An additional unforeseen advantage of this approach was the ability, taking the load test results into consideration, of the project consultant team to reduce the number of remedial micropiles at M42 from the 6 to 4, thereby saving the project from the time and cost of installing the final 2 micropiles.

Figure 7: Installation of bridge micorpiles

6 DISCUSSION AND CONCLUSIONS

Multiple phases of specialty geotechnical construction were performed in conjunction with real time monitoring of sensitive existing structures at the West Don Lands Back Flow Preventer project site. Using this data, movements detected during both jet grouting and micropiling were able to be managed by immediate stoppages of the work and modifications to the sequencing and methodology.

This project included successful completion of overlapping jet grout columns to cut-off potential water inflow and resist basal heave during excavation and construction of the BFP. Quality control testing was performed throughout the installation of the production jet grout columns to monitor the grout mixture, jet grout parameters and compressive strength.

Rock-socketed micropiles were constructed to support the concrete backflow preventer chamber, and successfully done so without resulting in any leakage through the jet grouted base plug.

Remedial rock-socketed micropiles were constructed to replace the compromised foundations at 3 bridge piers that exhibited movement during the construction of the BFP.

All remedial work was able to be successfully completed and allowed the general contractor and project team to complete construction of the BFP. Construction of the BFP was made feasible at this site by the application of specialty geotechnical construction methods. The BFP construction was successfully completed in April, 2015.

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