Full Scale Foundation Testing of Rammed Aggregate Pier® Supported Foundations in the Toronto Waterfront

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ABSTRACT
Construction of a transit maintenance and storage facility in the Toronto waterfront was carried out in deep, uncompacted, and environmentally impacted fills, underlain by compressible peat and organic soils. Ground improvement, consisting of over 7,000 Rammed Aggregate Piers, was used to improve the fill and organics, accelerate grading and surcharge settlement, and support shallow spread foundations, rail slabs, and floor slabs.

Extensive field testing was carried out at the site during construction, including full scale Rammed Aggregate Pier element load tests, full scale foundation load tests, full scale foundation plate load tests, measurement of transmitted vertical stresses below the foundation load test, inclinometer monitoring during ground improvement installation, surcharge settlement monitoring, and long-term foundation settlement monitoring during building construction. The results of the monitoring and testing are presented in this paper.

RÉSUMÉ
Un atelier d'entretien et d'entreposage de véhicules de transports en commun, dans le secteur riverain de Toronto, a été construit sur d'épais dépôts de remblais non-compactés et contaminés, de tourbe compressible et autres sols organiques. L'amélioration du sol a consisté en l'installation de plus de 7 000 colonnes ballastées (Rammed Aggregate Piers), pour améliorer les propriétés des remblais et des sols organiques, accélérer les travaux de terrassement et de tassements en surcharge, et supporter les semelles de fondations isolées, les dalles de rails et les dalles au sol.

Plusieurs essais in situ ont été effectués sur le site durant les travaux de construction, y compris des essais de charge à grande échelle des éléments des colonnes ballastées et des fondations; des essais en charge à grande échelle des fondations et sur plaque, incluant la mesure des contraintes verticales transmises sous les fondations; le suivi à long terme des inclinomètres pendant les travaux d'amélioration du sol; et le suivi à long terme du tassement des fondations durant la construction. Ce document présente les résultats de ces essais et suivi de la qualité des travaux.

1 INTRODUCTION

The Toronto Transit Commission (TTC) is a public transit agency that operates bus, streetcar and subway transit services in Toronto, Canada. It operates the third largest urban mass transit system in North America. Several of its bus and streetcar maintenance and storage facilities are located near the Toronto waterfront. The subsurface conditions typically consist of miscellaneous fills overlying weak organic soils, sand and bedrock, which is typically encountered at depths in the order of about ±15m below grade. Geotechnical investigation and design has traditionally avoided characterization of these weak and variable soils and adopted deep foundation and structural floor slab construction. Based on new ground improvement technologies and enhanced geotechnical investigation tools, construction methods within these soils are changing.

Geotechnical investigation of weak heterogeneous organic soils requires extensive sampling and testing. The use of various in situ measurements allows for predicting settlements and evaluating ground improvement options with confidence. This paper utilizes data gathered and presented in other papers and presents the extensive testing on the ground improvement systems carried out at the site for the new low floor light rail vehicle (LFLRV) yard near the Toronto waterfront.

2 SITE HISTORY

The site is located within the Portlands area of Toronto, Canada, which is characterized by man-made deposits and organic soils overlying bedrock. The Toronto shoreline has been extended since the early 1900s by filling the nearshore area and increasing the available land adjacent to the downtown core. More recently at the site a 12m high mound of excess soil was placed over the majority of the existing hydraulic fills during surrounding site developments since the 1960s.

The geological history of the Toronto area has included several advances and retreats of glaciers of Ionian and Wisconsian age. After the last glaciation, which left a discontinuous veneer of glacial till overlying the shale bedrock and outwash sands overlying the glacial till, the area was inundated by Glacial Lake Iroquois. Recent organic soils accumulated in poorly drained marshy areas, composed of organic silts, organic clays and peat. The groundwater table is generally controlled by the water level of nearby Lake Ontario, about 1 to 2m below existing grade. Figure 1 outlines the area of miscellaneous fills and weak organic soils present near the Toronto waterfront.

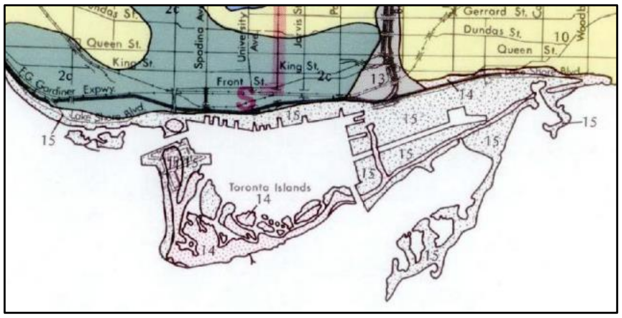

Figure 1. Distribution of fill (denoted area 15) and organic soils (denoted area 13) along the Toronto waterfront (from Sharpe 1980).

Various papers have been published on various in situ soil testing carried out at this site, such as Drevininkas and Sedran (2011), Drevininkas et al (2011), and Drevininkas (2014). This extensive information was used to design the ground improvement system for the project.

3 PROPOSED DEVELOPMENT

The proposed maintenance facility is to house and service the vast majority of the Toronto Transit Commission's fleet of LFLRVs. The facility boasts over 25,000 m^2 of building space and with roughly 75,000 m^2 of additional exterior yard space, including about 8 km of track on concrete slabs. Approximately 120 LFLRVs can be housed on the site, with up to 20 at a time inside the facility and up to 100 outside. Figure 2 presents a photograph of the completed facility.


Figure 2. Photograph of completed facility.

4 GEOTECHNICAL INVESTIGATION

An extensive geotechnical investigation was carried out in support of rail infrastructure, foundations, floor slab-on-grade, underground utilities and the proposed grade raise at the site for the proposed transit maintenance and storage facility. The investigation consisted of conventional borehole sampling, monitoring well installations, in situ testing and geotechnical laboratory testing. Based on the investigation, below the proposed cut grade, the site consists of an overburden thickness of about 16m, comprised of heterogeneous fill, organic soils (organic silts, sands and peat), interstadial sands, and silty clay over shale bedrock. In situ testing within the organic soils and silty clay consisted of field shear vane, piezocone penetration testing (CPTu), seismic dilatometer testing, self-boring pressuremeter and K_0 step blade tests. Geotechnical laboratory tests within these soils consisted of index properties, oedometer and consolidated undrained triaxial tests.

The site stratigraphy from the proposed subgrade elevation of the facility consisted of:

- Miscellaneous Fill – 2 to 6m thick loose to compact Sandy Silt to Silty Sand with debris
- Organic Soils – 5m thick interbedded organic silts and clays
- Peat – discontinuous 1m thick fibrous peat
- Sands – 2 to 6m thick compact to dense
- Glacial Till – 1m thick discontinuous very stiff silty clay
- Shale Bedrock

The groundwater table at the site is at a depth of about 1 to 2m below grade. Figure 3 presents a cross-section across the site before construction showing the variable soil stratigraphy, with the red line indicating the proposed subgrade level. Figure 4 is a photo of the fill with wood debris.

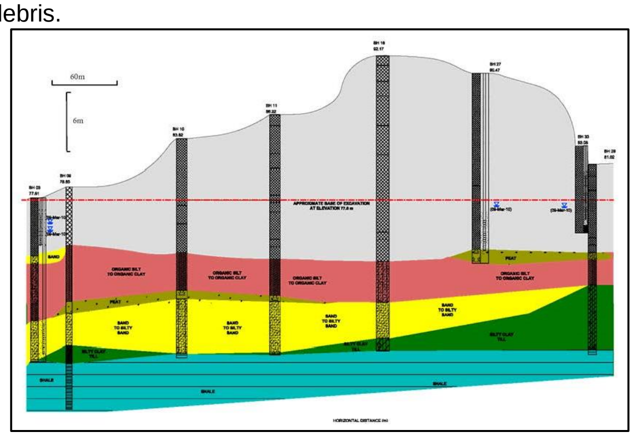

Figure 3. Cross-section at the maintenance facility site.


Figure 4. Photo of exposed fill subgrade with wood debris.

5 RAMMED AGGREGATE PIERS – IMPACT METHOD

An early enabling works package for this site was completed involving the removal and disposal of about 400,000m^3 of environmentally impacted fill and preparation of the subgrade for the subsequent building and yard.

Initially driven piles were considered for the site with associated pile caps, grade beams and structural floor slab, which would have resulted in significant costs for the project. There were also concerns that a grade raise across the site would results in settlement of the proposed track slabs and services on the site. Excavation and replacement of the poor soils below grade were not feasible.

Ultimately, the design team chose the Geopier Rammed Aggregate Pier® (RAP) system, using the Impact® method. The Impact System provided high-bearing capacity while effectively controlling settlement in the soft, highly organic soil profile. The RAP system also provided for radial drainage elements that served the dual purpose of draining the soil during preloading and for providing long-term settlement control and support to the new fill and track slabs. Figure 5 presents the steps involved for the Impact System installation. Figure 6 presents a photograph of the RAP installation process.

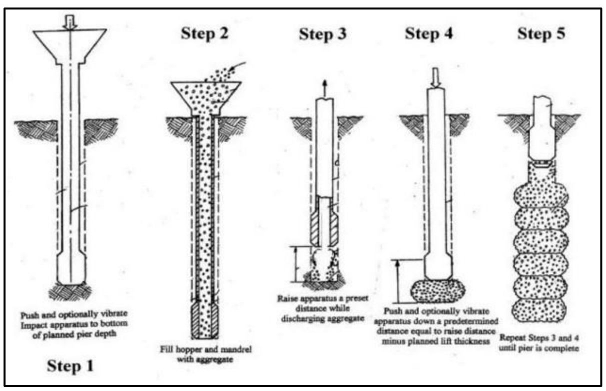

Figure 5. Impact System installation process

A sizeable portion of the site had the 12m high soil mound stockpile on it for decades, which essentially served as a long term and large "pre-load". The remainder of the site did not have the advantage of this historical preloading, and therefore the sub-soils were in a much looser state than the soils that were under the large historic soil pile. This necessitated a different design approach for each of the two zones.

In the zone with no preload, Grouted Geopier elements were employed for footing support, given the high loads and general lack of long-term preload in that area. Ungrouted Geopier elements were also installed to support the structure's floor slab through this zone. In other areas that were "unintentionally" preloaded by the large pile of spoil, no slab support was needed, and standard RAP elements were utilized for footings.

Given the sheer size and scope of the project, specifications dictated a large array of required testing including over 10 individual full-scale Geopier modulus tests, several very large-scale group footing and plate load tests, as well as many others.

In the end, well over 7,000 RAP elements were installed to depths of up to 12m. Rapid installation and system flexibility sped up the project timelines significantly. The preload, which would have otherwise taken several years, was shortened to a few months with the Geopier elements in place as "structural drains."


Figure 6. Photograph of the RAP installation.

6 FOUNDATION TESTING AND MONITORING

A foundation testing program consisting of various types of testing was carried out for this project to confirm the design of the ground improvement system. The various tests carried out as part of the program include the following:

- RAP modulus load tests;
- Full-scale foundation load tests;
- Large plate load tests, and;
- Settlement Monitoring.

6.1 RAP MODULUS TEST RESULTS

Six (6) full-scale modulus load test were conducted on site to verify that the design stiffness of the rammed aggregate pier system had been achieved. The test is set up similar to a pile load test configuration, as shown in Figure 7, and is performed in general accordance of ASTM D-1143. Deflection of the pier during loading are measured at the top of the pier as well as on tell-tales installed at the base of the pier to evaluate the behavior and stiffness of the pier.

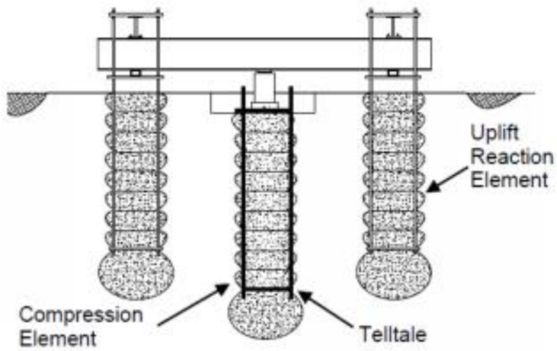

Figure 7. Typical Modulus Test Setup

The modulus load tests were carried out on non-production elements loaded to 150% of the maximum load at various locations on the site. Typically the worst borehole locations are chosen and the highest representative top of pier stress is chosen for the design stress for the test pier. The results of the test are presented in Figure 8 and summarized in Table 1. It can be seen from the results that the maximum deflection at 100 % of the design load was under 12mm. The design modulus of the Geopier elements was 34 MPa/m for ungrouted RAPs and 82 MPa/m for grouted RAPs.

Table 1 Modulus Load Testing Summary

Test RAP	100% Design Stress	Deflection at 100% Design Load	Modulus at 100% Design Load
TP1(un-grouted)	475 kPa	1.3 mm	365 MPa/m
TP6(un-grouted)	475 kPa	3.8 mm	125 MPa/m
TP7(un-grouted)	475 kPa	5.8 mm	82 MPa/m
TP9(grouted)	930 kPa	1.8 mm	517 MPa/m
TP12(un-grouted)	715 kPa	11.9 mm	60 MPa/m

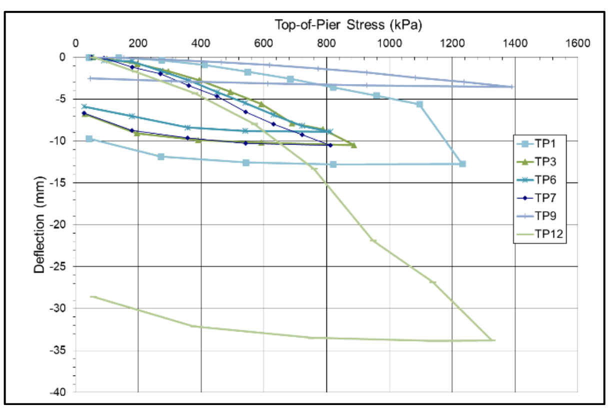

Figure 8. Modulus Load Test Results.

6.2 FULL SCALE FOUNDATION LOAD TESTS

Two (2) full-scale foundation load tests were carried out on improved ground as part of the foundation testing program. Immediately next to each full-scale foundation test one full-scale foundation test was carried out on a 2.1 m by 3.0 m concrete footing constructed over un-grouted RAP elements and the second test was carried out on a 1.5 m by 2.1 m footing constructed over grouted RAP elements.

Next to each of the footing load tests a plate load test was carried out on unimproved ground that had been compacted at surface during site preparation. The plate load tests consisted of loading a 25 mm thick, 2.1 m square steel loading plate next to the un-grouted RAP foundation test and a 25 mm thick 1.5 m square steel loading plate next to the grouted RAP foundation test.

All footing and plate load tests were carried out in general accordance with ASTM D1194. A summary of the tests can be found in Table 1.

At one of the full scale load test locations inclinometers were installed adjacent to the RAPs to measure lateral deformation of the RAPs during loading. Earth pressure cells were installed over the RAPs and subgrade to measure the distribution of stresses below the foundation during loading. The results of the inclinometer and earth pressure cells results are not included in this paper.

Table 2 Foundation Testing Summary

Test Type	Footing /Plate Size	No. of RAPS	SLS Design Load	Final Load
Foundation Load Test (Un-grouted RAPs)	2.1m x 3.0m	8	1,260 kN	2,700 kN
Plate Load Test (Un-grouted RAP area)	2.1m x 2.1m	0	882 kN	1,800 kN
Foundation Load Test (Grouted RAPs)	1.5m x 2.1m	4	630 kN	1,400 kN
Plate Load Test (Grouted RAP area)	1.5m x 1.5m	0	450 kN	980 kN

Results of the load tests are presented in Table 3, and Figures 9 and 10. Figure 11 is a photograph of the foundation load test frame, which consisted of a large gravel box reaction system necessary to generate the reaction force for the large loads applied during testing.

The intention of the testing was to compare the response of the soil in an unimproved state to an improved state and to look at the response of a group of elements in comparison to the results.

The modulus results in Table 3 for the Foundation tests would be a composite of RAP elements and soil where the RAP elements attract more stress than the soil but work in concert with the improved soil in between RAPs. The modulus for the plate load tests is the E modulus of the unimproved soil within the zone of influence.

Table 3 Foundation Testing Summary

Test Type	Settlement at SLS Design Load (design load)	E Modulus at SLS Design Load (E mod type)
Foundation Load Test (on Un-grouted RAPs)	17 mm (1,260 kN)	10 MPa (composite)
Plate Load Test (Un-grouted area)	21 mm (882 kN)	8 MPa (soil only)
Foundation Load Test (on Grouted RAPs)	2 mm (630 kN)	100 MPa (composite)
Plate Load Test (Grouted area)	12 mm (450 kN)	17 MPa (soil only)

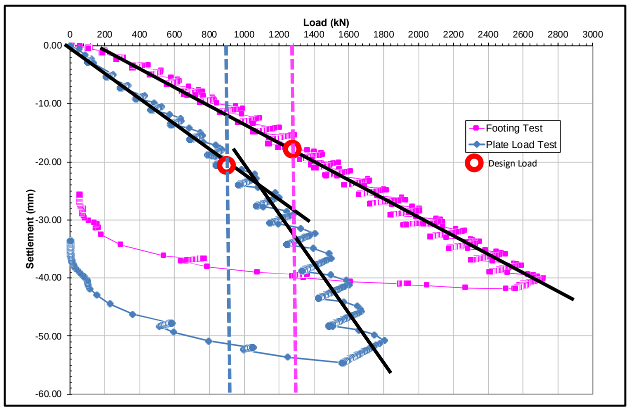

Figure 9. Footing Load Test – Un-Grouted Area.

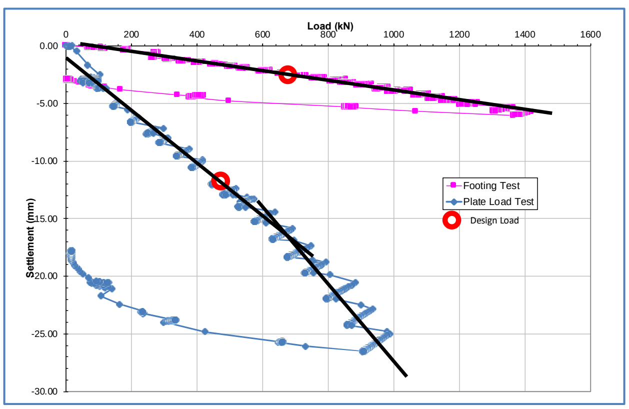

Figure 10. Footing Load Test – Grouted Area.

While the settlement at SLS design load and E modulus at SLS design load do not appear to be greatly improved in the un-grouted RAP area, it should be noted that the total "plate size" and foundation load for the RAP supported footing is 40% larger which results in a deeper zone of influence, carrying the zone of influence through the upper sandy fill and into the organic silt for the RAP supported footing whereas the zone of influence for the plate load test remained largely in the upper sandy fill.

Further it should be noted that the plate load on unimproved fill showed a plastic response in the curve with the break in the stress-strain curve just past the design load (see Figure 9 above). This would indicate a large potential for excessive settlement where the organics in the fill varies in both content and thickness on the site. Conversely, the Group-RAP supported footing test shows a stiff, elastic response well past the design load.

The grouted RAP area shows a more marked difference between the grouted RAPs footing test and the plate load on existing ground. Again, the plate load test exhibits a plastic break in the curve (see Figure 10). It should be noted that the plate load test in the grouted RAP area may have been founded on a layer of preload granular material and the zone of influence may not have adequately engaged the lower problematic unconsolidated organic soil. This area also had the benefit of some preload having taken place already at the time of testing. This may explain the stiffer than expected response from the unreinforced plate load test.

Figure 11. Photo of Foundation Load Test frame.

6.3 LARGE PLATE LOAD TEST RESULTS

In order to facilitate an impermeable geomembrane collection system a design change was needed to the footing support where some footings would not be resting directly on the RAP elements as originally designed and tested in the footing load tests. The new design consisted of a layer of 100 mm of crushed stone (approximately 50 mm diameter) on top of the RAP elements followed by the impermeable geomembrane and 200 mm compacted Granular 'A'. Since this footing support configuration had not been tested in the footing load tests, a series of six (6) large plate load tests were carried out to confirm the design.

The large plate load tests consisted of loading 1.0m to 1.5m wide plates to loads of 467 kN. The plates were founded on 200 mm of compacted Granular 'A' and 100 mm of crushed stone over RAP elements as per the design described above. A summary of the plate load testing is provided in Table 4, and the results are presented on Figure 12.

Table 4 Large Plate Load Testing Summary

Test	Maximum Load	Maximum Pressure	Settlement
PLT#7	467 kN	311 kPa	9.7 mm
PLT#8	467 kN	311 kPa	22.2 mm
PLT#9	467 kN	311 kPa	18.1 mm

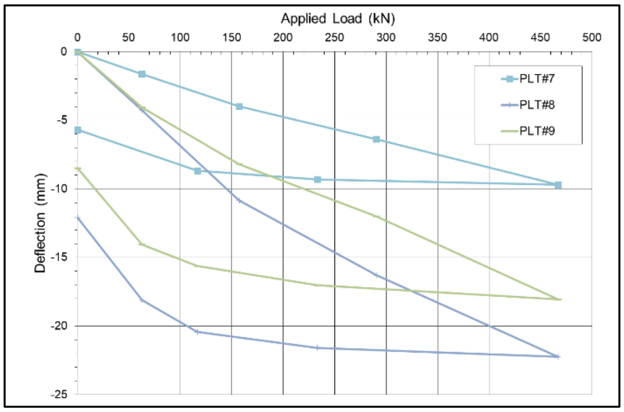

Figure 12. Large Plate Load Test Results.

6.4 COMPARISON OF TESTING RESULTS AND DISCUSSION

Modulus load tests are conducted on individual RAP elements and should generally be predictive for the performance of a footing by calculating a composite E modulus (E_{comp}) using the following formula (Wissmann et. al, 2002):

$$E_{comp} = (1 - R_a)\, E_m + R_a\, E_g \qquad [1]$$

Where E_g is the elastic moudulus of the aggregate pier element, E_m is the elastic modulus of the matrix soil, and R_a is the ratio of the area coverage of the aggregate pier elements to the gross footprint area.

Once composite the elastic modulus (E_{comp}) is established from Equation 1, the settlement in the Upper Zone (s_{uz}) is computed as follows:

$$S_{uz} = \frac{\Delta q\, I_q H_{uz}}{E_{comp}} \qquad [2]$$

Where Δq is the loading, I_σ is the average stress influence factor in the upper zone, and H_{uz} is the thickness of the reinforced upper zone layer.

The zone of influence of the load and the length of the RAP elements results in the entire zone of influence residing in the RAP reinforced zone even with conservative influence factors (i.e calculating settlements to 4B instead of 2B, where B is the width of a square footing). As such, for foundation loads the settlement calculation does not require a lower zone calculation and the summative settlements for a footing would be the results of the upper zone calculation.

Modulus test results provide the E_g component in Equation 1. Using the correct E_m for the soil and the area ratio, Ra, one can use modulus test results to calculate the total predicted settlement.

Using the design modulus of E_g= 34 MPa for ungrouted elements, a calculated E_{comp} would be about 10 MPa within the organic soils, with a settlement of 25mm. As shown in Table 3 for Foundation load test in the ungrouted RAP area, the actual E_{comp} measured in the group footing test was similar to the calculated E_{comp} and the actual measured settlement is lower than the calculated settlement. This demonstrates a reasonably conservative prediction, which is appropriate for this type of soil.

The plate load tests on engineered fill from Table 4 and Figure 12 demonstrate similar settlement results to the group footing tests done directly on the Geopier elements. In attempting to compare the modulus test results (from section 6.1), the plate and group footing test results (from section 6.2) and the large plate load tests on a layer of granular above the elements (from section 6.3), the site variability could play a negative role in drawing direct comparisons as the tests were done at different locations on the site and the site is heterogenous and variable by nature.

What is apparent in the comparison of these three test results is that the improved ground resulting from the installation of RAP elements, provides for the necessary performance for the conventional construction of spread and strip footings and slab on grade and that 200 kPa SLS was an achievable bearing capacity for footings using ground improvement for this poor soil condition.

6.5 SETTLEMENT MONITORING

Settlement monitoring was carried out during temporary surcharge at the site and on foundations during construction of the structure.

Temporary Surcharge

Twenty ground settlement rods were placed on the cut grade surface prior to the 2m high preloading or surcharging at the site. The settlement rods consisted of metals rods welded to a flat steel plate placed on grade and separated from the surrounding preload material by encasing the settlement rods in a corrugated metal pipe. Figure 13 presents the results of two typical ground settlement points at the site. The settlement plate over the organic soils resulted in 89mm settlement with an interpreted C_h of 0.03m^2/day, while the settlement plate over the silty clays resulted in 66mm settlement with an interpreted C_h of 0.10m^2/day. Soil conditions were variable in the north-south direction across the site.

Foundation Settlement Monitoring

In order to monitor the performance of the foundations during building construction, 15 settlement monitoring points were installed on existing column foundations. These settlement monitoring points were surveyed by total station on a weekly to monthly basis for a period of up to

13 months as the building was constructed. At end of construction, with a foundation design geotechnical reaction at Serviceability Limit State of 200 kPa, column foundation settlements ranged from 4 to 34mm, with an average of 20mm.

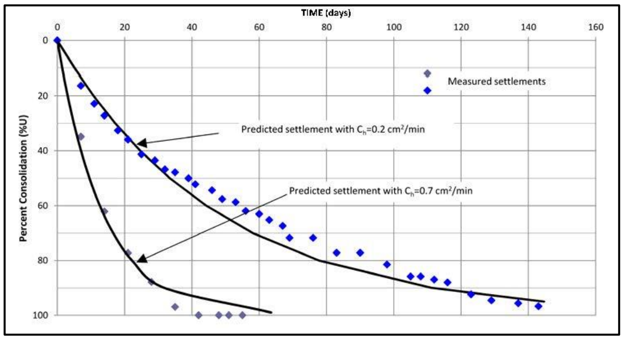

Figure 13. Results of settlement monitoring for the 2m high preload.

7 CONCLUSIONS

The TTC transit maintenance and storage facility is founded on variable, weak and soft subsoils, requiring the use of ground improvement to support the foundations of the facility. The RAP ground improvement system was proven and effective foundation support technique though extensive load testing and construction monitoring, including full scale foundation testing and settlement monitoring. While the results of these tests varied to a certain extent, this was anticipated given the variability of the subsoil and all results exceeded the design requirements.

8 ACKNOWLEDGEMENTS

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