Design and Monitoring of a 23 Storey Condominium Using Rammed Aggregate Pier Support

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ABSTRACT
A 23-storey tall residential condominium tower is currently under construction in Waterloo, Ontario. The condominium consists of a 23-storey tower and two attached 3-storey podiums. The combination of the tall tower and shorter podiums resulted in extreme differential loading between the tower raft foundation spread footings under the podium. This presented risk of excessive settlement across the raft footprint and between the tower raft and podium footings. In order to limit the total and differential settlements, Geopier Rammed Aggregate Pier® ground improvement was used under the raft foundation. The spacing of the pier elements varied to provide a variable stiffness profile, greatly reducing the potential for differential settlement. This paper will present the design and construction methodologies along with results of verification testing of the ground improvement system used for this project and will review initial settlement monitoring data to corroborate the design.

RÉSUMÉ
Une tour d'habitation de 23 étages est actuellement en construction à Waterloo, en Ontario. Le condominium se compose d'une tour de 23 étages et de deux podiums de trois étages. La combinaison de la grande tour et des podiums plus courts a entraîné une fondation de radeau sous la tour et des semelles étalées sous le podium, ce qui présente un risque de tassement excessif sous le radier et de tassement différentiel à l'intérieur du radeau et entre les podiums et la tour. Afin de limiter les tassements totaux et différentiels, l'amélioration du sol de Geopier Rammed Aggregate Pier® a été utilisée sous la fondation du radier. L'espacement des éléments de la pile variait pour fournir un profil de rigidité variable, réduisant considérablement le potentiel de tassement différentiel. Ce document présentera les méthodologies de conception et de construction ainsi que les résultats des tests de vérification du système d'amélioration du sol utilisé pour ce projet et passera en revue les données initiales de surveillance du peuplement pour corroborer la conception.

1 INTRODUCTION

The Caroline Street Private Residences in Waterloo, Ontario consist of the development of a residential condominium with a 23-storey tower and two -storey podiums with one level of underground parking. The residential condominium is to be connected to an existing residential structure at the site on the basement level. The foundation design of the structure consists of a raft foundation to support the tower and spread footings to support the podiums. Contact pressures ranging from 200 to 465 kPa at serviceability limit states (SLS) were exerted by the tower's raft foundation, and column loads ranging from 430 kN to 2,950 kN at SLS on spread footings designed for 180 kPa SLS {270 kPa ULS] were used to support the podium portion of the development.

During the design phase, differential settlement between the spread footings supporting the podium and the raft slab supporting the tower were identified as a design issue. Unreinforced differential settlement across the tower raft was estimated by the project geotechnical engineer to be on the order of 86 mm to 148 mm, and differential settlements between the raft and tower foundations exceeded 65 mm. These values were considered to be excessive. The design team evaluated several foundation solutions, including caissons, driven piles, and Geopier Rammed Aggregate Pier® (rammed aggregate pier) ground improvement. The aggregate system was selected because it provided outstanding settlement control and the ability to easily provide variable stiffness below the different building areas to match settlements across the structure. The aggregate pier system also was the lowest cost option, which the easiest foundation system (raft and spread footing) to design and construct on site.

A differential settlement criterion of 19 mm between the spread footings and raft was initially required by the structural engineer. During construction, the differential tolerance was reduced to less than 13 mm between the edge of the raft and the podium. It was also determined that the differential settlement within the raft needed to be optimized to reduce the total cost of the raft. The use of the rammed aggregate Pier system allowed highly variable composite stiffness design, which substantially reduced bending of the tower raft. When combined with staged construction (constructing the tower ahead of the podium), the podium-to-tower differential settlement criteria to be met.

The development is still under construction. Settlement monitoring of the tower foundation has been ongoing since the on-set of construction. The preliminary results of this monitoring will be discussed within this paper.

1.1 Geotechnical Conditions

A total of thirteen (13) boreholes were advanced within the footprint of the condominium with eleven (11) of the boreholes advanced within the vicinity of the tower and two (2) boreholes advanced within the podium footprint only. The borehole depths ranged from 10 m to 46 m below existing grade. In addition to the boreholes pressuremeter testing (PMT) was completed at two (2) locations within the tower footprint.

The boreholes generally encountered a 1 to 3 m thick fill layer underlain by an upper cohesionless deposit followed by a cohesive deposit and a lower cohesionless deposit. A simplified soil profile of the geotechnical conditions encountered at the site are shown in Figure 1.

The upper cohesionless deposit was generally encountered below the fill to elevation 311.5 to 301.5 m (13.5 to 23.5 m below underside of raft). SPT 'N' values of 9 to greater than 50 blows per 300 mm of penetration were measured within the deposit corresponding to a compact to very dense relative density.

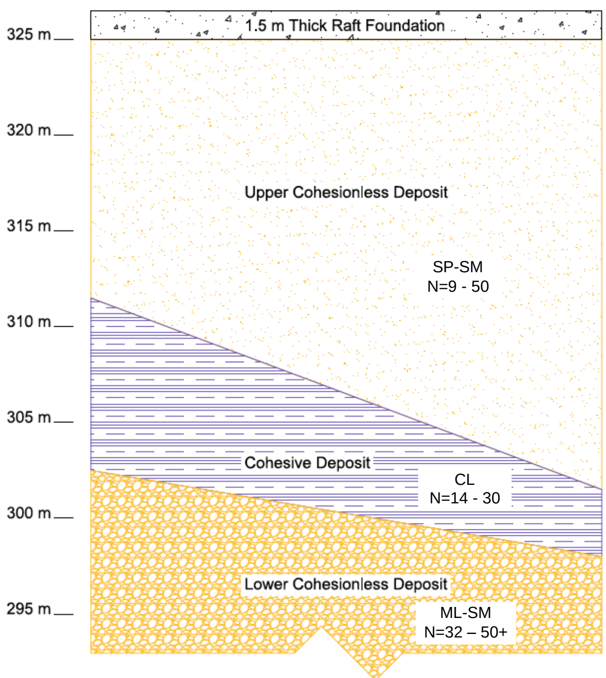

Figure 1. Soil Profile

The cohesive layer consists of clay to silty clay extending to elevation 302.5 to 298 m (22.5 to 27 m below underside of raft). SPT 'N' values of 14 to greater than 30 blows per 300 mm of penetration were encountered within this deposit, corresponding to a very stiff to hard consistency.

The deep boreholes were terminated within the lower cohesionless deposit which consisted of silt to silty sand and borehole. SPT 'N' values of 32 to greater than 50 blows per 300 mm of penetration were encountered within the deposit, corresponding to a dense to very dense relative density.

A total of seven (7) PMT tests were completed at the site at two (2) different locations. The results of the test indicate PMT moduli (E_{pmt}) of 12.1 to 93.2 MPa. A summary of the PMT results are provided in Table 1, below:

Table 1. Summary of PMT Results

Elevation (m)	E_{pmt} (MPa)	p_y (kPa)	p^*_L (kPa)	p^*_L/p_y
96.4	12.1	396	1383	3.7
91.1	32.3	1005	2906	2.9
87.4	46.8	1341	4355	3.2
82.1	23.3	655	2050	3.1
94.9	24.0	409	1599	3.9
90.5	24.3	606	2384	3.9
85.1	93.2	1546	4170	2.7

2 FOUNDATION SOLUTION

Total and differential settlements in the top 20 to 25 meters of the profile exceeded the project tolerances. Deep foundations were considered, but high costs associated with foundation installation as well as pile caps, grade beams, and structural slabs made those solutions cost prohibitive. Ground improvement with simple raft and spread footing construction proved to be the most cost effective, and the rammed aggregate system was selected to minimise differential settlement within the raft slab and between the tower and podium foundations. The flexibility of the rammed aggregate system allowed the application of higher stiffness at locations of higher applied pressure, resulting in a cost effective raft design.

2.1 Rammed Aggregate Pier System

The displacement rammed aggregate pier system is installed by first driving a specially designed 200 to 300 mm outer-diameter hollow mandrel with a 350 to 400 diameter foot into the soil using a large static force augmented by dynamic vertical impact energy. The rammed Aggregate Pier elements typically extend between 1.5m to 15.5m (5 to 50 feet) below grade, but may extend deeper depending on project requirements. After driving to the design depth, the hollow mandrel serves as a conduit for the placement of open-graded aggregate. The aggregate typically ranges from 19 mm to 35 mm stone. The aggregate is placed inside the hopper and mandrel and the mandrel is raised, leaving a continuous lift of aggregate. The mandrel and foot are raised and then driven back down, forming a one- to two-foot thick compacted lift. Compaction is achieved through the static crowd force and dynamic impact energy from the hammer. The hammer densifies aggregate vertically and the tamper foot forces aggregate laterally into cavity sidewalls. Subsequent lifts are performed using the same approach of raising and lowering the mandrel and tamper. Figure 2, provided below, provides a schematic sketch of the rammed aggregate pier system.

The rammed aggregate pier process results in lateral stress increase and densification in the matrix soil and combined with the high stiffness aggregate pier, increase settlement control and stiffness of the founding soil.

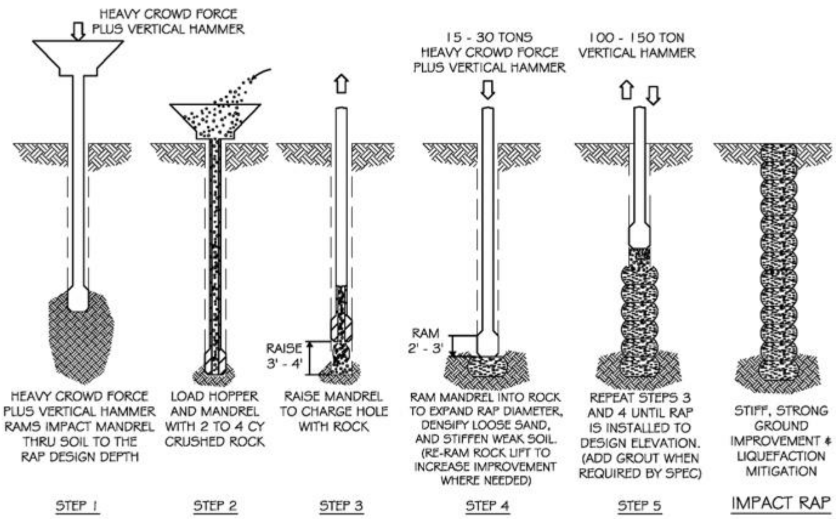

Figure 2. Rammed Aggregate Pier Installation

2.2 Design Approach

The raft slab was designed to support the high tower loads, resulting in variable foundation contact pressures at the base of the raft. In order to limit the differential settlement, shear stresses, and bending moments within the slab, a variable spacing of rammed aggregate pier elements was designed. The varied pier spacing results in variable stiffness under the slab which can be tailored to match the pressure envelopes for the different load conditions required in design. Changes in subgrade reaction modulus (i.e. pier spacing) induce changes in the structural analysis of the superstructure. This results in changes in the contact pressure, which necessitate revisions to the pier spacing. As such, iterations between the rammed aggregate pier designer and the structural engineering team are required to converge on the most efficient raft design and pier spacing.

2.3 Design for Settlement

Design for settlement control was carried out using the method proposed by Wissmann, et al., 2001, where the settlement is analysed in layers including the reinforced zone (upper zone) and unreinforced zone (lower zone). Each of these zones are then further sub-categorized based on the matrix soil type and stiffness.

The reinforced zone settlement is analysed using a composite elastic modulus approach. The rammed aggregate pier installed is stiffer than the matrix soil, therefore attracting a larger percentage of the imposed stress. The composite elastic modulus (E_{comp}) is calculated using equation 1, below:

$$E_{comp} = (1 - R_a)E_m + R_aE_g \quad [1]$$

Where, E_g is the elastic modulus of the rammed aggregate pier, E_m is the stiffness of the matrix soil, and R_a is the area ratio of rammed aggregate pier to the footprint.

Following the determination of E_{comp} the settlement within the reinforced zone, S_{uz}, is calculation using equation 2, below:

$$S_{uz} = \frac{qI_fH_{uz}}{E_{comp}} \quad [2]$$

Where, q is the foundation bottom stress, I_f is the stress influence factor, and H_{uz} is the thickness of the layer.

Settlements for the unreinforced zone are calculated using standard geotechnical methods. For the Caroline Street Private Residences project, the unreinforced zone settlements were calculated using an elastic solution and boussinesq stress influences, where the lower zone settlement, s_{lz}, is calculated using equation 3, below.

$$S_{lz} = \frac{qI_fH_{lz}}{E_{lz}} \quad [3]$$

Where, q is the foundation bottom stress, I_f is the stress influence factor, H_{lz} is the thickness of the layer, and E_{lz} is the elastic modulus of the layer.
The total estimate settlement, s, is calculated by summing the settlements of the unreinforced and reinforced zone. Settlement parameter values used for this project are presented in Table 2, below:

Table 2. Design Parameter Values

Parameter		Value
Reinforced Zone	Rammed Aggregate Pier Modulus, E_g	250 to 300 MPa
	Matrix Soil Elastic Modulus, E_m	60 MPa
Unreinfoced Zone	Elastic Modulus, E_{lz}	70 to 140 MPa

Stress at various depths were estimated using the commercially available programme, Settle3D by Rocscience. Settle3D calculates the stress at a given depth and location based on boussinesq stress influence equations. The computed stresses were then used to calculate the estimated settlement and corresponding subgrade reaction values using the method described above.

Figures 3 through 5 depict the calculated subgrade reaction values, applied bearing pressures, and settlements at serviceability limit states (SLS) based on the analysis.

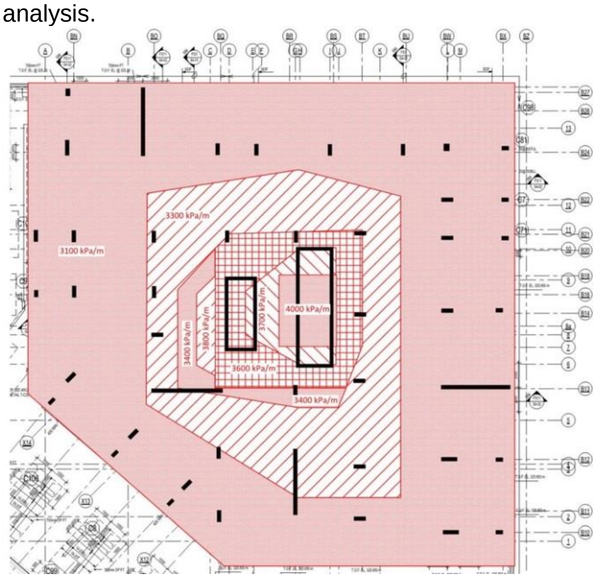

Figure 3. Calculated Subgrade Reaction Values at SLS

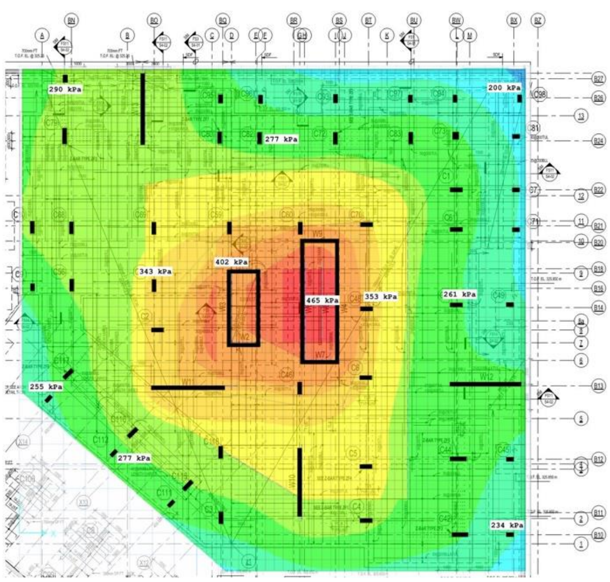

Figure 4. Calculated Applied Bearing Pressures at SLS

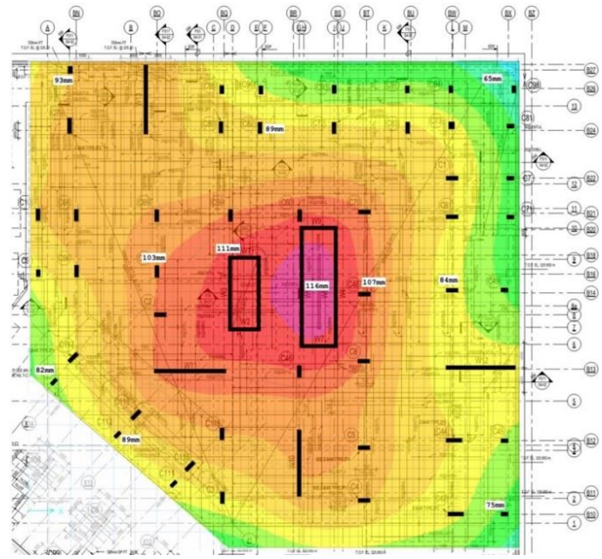

Figure 5. Calculated Settlements at SLS

3 QUALITY CONTROL AND TESTING

Full time quality control of the ground improvement system was provided by a technician who documents the construction of each pier, including depth, location, and volume of aggregate. During installation, a select number of piers are subjected to a Bottom Stabilization Test (BST) to field check geotechnical conditions and pier stiffness for uniformity across the site. Additionally, a full-scale modulus load test was completed on a non-production pier, and PiezoCone Penetration testing (CPT) was conducted between installed piers to verify densification of the matrix soil.

3.1 Modulus Load Test

A full-scale modulus load test was 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 6, 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.

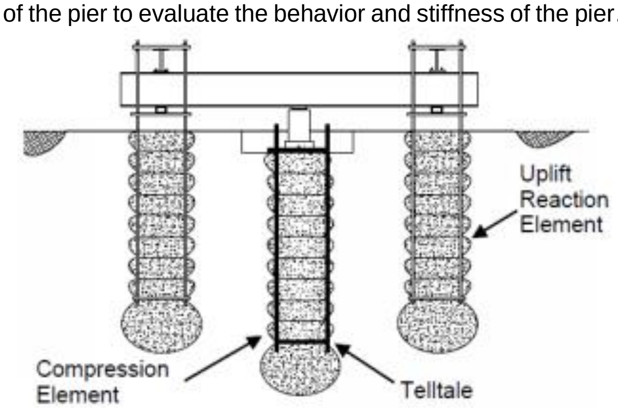

Figure 6. Typical Modulus Test Setup

3.1.1 Modulus Load Test Results

Modulus load testing for this development was carried out a non-production element that was loaded to 150% of the maximum load calculated for any pier installed. The results of the test are presented in Figure 7. At the 100% load increment of 1,400 kPa (29,240 psf) the measured top and bottom of pier displacements of 13 mm and 0 mm, respectively, and at the 150% load increment of 2,200 kPa (45,950 psf), the deflections were measured to be 27 mm and 0 mm for the top and bottom.

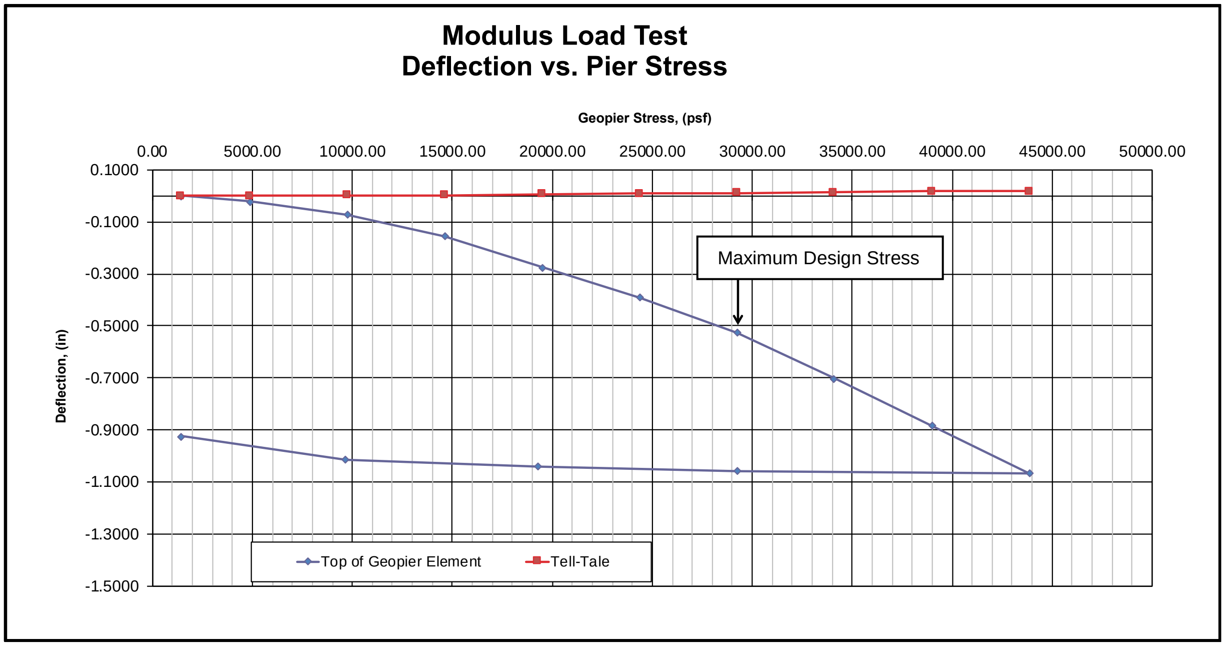

Figure 7. Modulus Load Test Results

3.2 Cone Penetration Testing (CPT)

CPT was carried out between installed piers at 6 locations to determine the densification of the matrix soil due to the ground improvement. From the results of the CPT the constrained modulus (M), which is generally equivalent 90% of the Young's Modulus (E) can be interpreted (In-Depth Geotechnical Inc., 2016). At all locations cone refusal was encountered at depths of 0.9 to 6.3 m, indicating that the densification achieved had surpassed the design assumptions. Table 3, below, presents the results of the CPT.

Table 3. Summary of CPT Results

Test	Depth (m)	Mean qt (MPa)	Mean M (MPa)
CPT-1	0 to 6.3	12.0	82.6
CPT-2	0 to 2.1	15.0	105.0
CPT-3	0 to 3.2	14.2	98.3
CPT-4	0 to 3.5	14.7	102.2
CPT-5	0 to 0.9	10.4	77.3
CPT-6	0 to 4.5	10.8	74.7

Settlement Monitoring

Settlement monitoring of the tower raft slab has been ongoing since the raft foundation was constructed. The settlement monitoring shows minimal movement of the slab since the start of construction. A graph showing the maximum measured settlement to date is shown in Figure 8 and a graph showing the average measured settlement is shown in Figure 9. The maximum recorded settlement was measured at a value of 9 mm within the core of the raft and 7 mm along the raft edge. The benchmark points used by the surveyor were checked to an external benchmark off site which indicated that as much as 3 mm of downward monitoring. Accounting for an instrument error of +/- 5 mm, the maximum deflection of the tower may be up to 17 mm and the core and 15 mm along the edge. The current that the substantial improvement of the matrix soil indicated in the CPT results, which exceeded the ground improvement design expectations, is responsible for the small settlement observed to date. At the time of this paper, the complete load of the structure has not yet been applied.

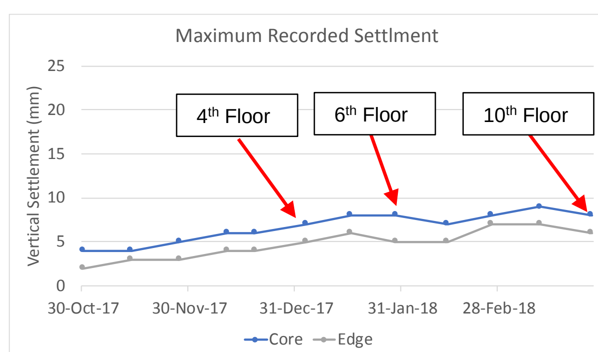

Figure 8. Maximum Settlement Monitoring Results to Date.

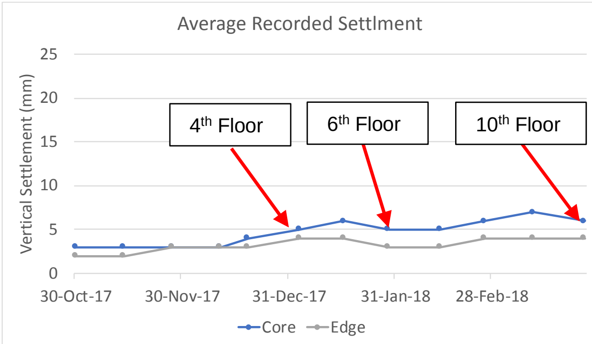

Figure 9. Average Settlement Monitoring Results to Date.

4 CONCLUSIONS

The rammed aggregate pier ground improvement system was selected based on the low cost, ease of installation, and ability to provide a highly varied differential support solution that met the demanding variable load application of the project. Very high contact pressures were supported on the rammed aggregate pier system.

During installation, the quality control testing (including the BSTs, modulus test, and CPTs) indicated that the rammed aggregate pier system exceeded the design. The pier elements were constructed with higher stiffness than used for design, and the matrix soil was improved more than expected in the design. As such, the building construction began with high confidence that settlement estimates would be more favorable than predicted during the design phase of the project.

The interim results of the settlement monitoring show less than anticipated settlement. This may be due to several factors including the following:

1. Stiffer in-situ soil than indicated by the geotechnical investigation
2. The dead weight of the raft accounts for a significant portion of the structural loading and settlement monitoring was not carried out prior to or during the placement of the raft and settlement due to the dead weight of the raft was not captured during monitoring due to lack of baseline
3. Greater ground improvement than anticipated during design.

5 REFERENCES

Barksdale, R.D. and Bachus, R.C. 1983. Design and Construction of Stone Columns, *FHWA/RD 83/026,* Vol. I. Report No. 1, Federal highway Administration, 210 pp.

In-Depth Geotechnical Inc. 2016. In-Situ Pressuremeter Testing, 155 Caroline St. South, Waterloo, Ontario.

NAVFAC. 1982. Soil Mechanics Design Manual 7.1, *Department of the Navy, Naval Facilities Engineering Command,* Alexandria, VA.

Sola Engineering Inc. 2016. Supplementary Geotechnical Investigation, Proposed High-Rise Development, 155 Caroline Street South, Waterloo, Ontario.

Terzaghi, K., Peck, R.B. and Mesri, G. 1996. *Soil Mechanics in Engineering Practice*, Third Edition, John Wiley & Sons, Inc., New York, NY, USA.

Wissmann, K.J., FitzPatrick, B.T., White, D.J., and Lein, B.H. 2002. Improving global stability and controlling settlements with Geopier soil reinforcing elements, *Proceedings, 4th International Conference on Ground Improvement*, Kuala Lumpur, Malaysia, 26 – 28 March.