RSPile Analysis of Two Osterberg Cell Load Tests on Post-Grouted and Conventionally Installed Caissons in Vaughan, Ontario

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Abstract. Three towers (35–57 storeys) are proposed in Vaughan, Ontario. The preferred foundation approach is bored concrete piles (caissons) bearing in soil at about 46 m depth, with Osterberg Cell (O-Cell) load testing to maximize design capacity. Several conventional O-Cell load tests have been designed and analyzed by Grounded Engineering in the vicinity. An alternative novel approach was recommended at this site: a two-caisson test investigating the feasibility of caissons with post-grouted pile tips compared with conventional tremie concrete caissons. Using load cell pressure as well as telltale, LVDT, and strain gauge data from both fully instrumented caissons, t-z and Q-z curves were input into RSPile to numerically model both conventional and post-grouted load tests to verify soil parameters. Once the model was verified, the loading was switched from bidirectional at the base to top-down loading to estimate the Equivalent Top Load (ETL) for caissons of different diameters using RSPile. Settle3 was used to model the change in effective vertical stress from the test condition to the final building condition, including excavation unload, dewatering, and 3D effects, over the full pile depth profile. The resulting stress reduction was built into the RSPile production pile ETL model. A table of capacities for various production caisson diameters, for both conventional and post-grouted options, was produced using RSPile. A significant cost savings is anticipated for either option when compared to conventional parameters, due to the innovative test design and numerical modelling.

Keywords: Site investigations · foundation engineering · deep foundations · Osterberg Cell · load test · pile modelling

1 Introduction

Three high rise towers (35–57 storeys) are proposed for a site in the Vaughan Metropolitan Centre (VMC) area of Vaughan, Ontario. The towers will also consist of low-rise podiums and three levels of underground parking. The anticipated tower column loads are anticipated to range from about 17 MN to 32 MN.

Bedrock elevation at the site would require 70 ± m long bored cast-in-place concrete piles (caissons). The native soils in the VMC area are competent for the support of

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caissons acting in both side shear and end bearing. The economical solution in this area is to use high-capacity caissons founded in soil, utilizing both side shear and end-bearing resistance, and designed and verified using Osterberg Cell Load Testing.

This paper provides a case study of Osterberg Cell load testing, interpretation, and comparison of a conventional caisson, and a caisson with a post-grouted tip. The importance of full-scale load testing on deep foundations is discussed, from the perspective of risk mitigation as well as economy. The benefits of numerical analysis (modelling) of load test results using RSPile are discussed. The study is limited to the analysis of a single pile; group effects are not discussed.

1.1 Osterberg Cell Load Testing

The Osterberg Cell (O-Cell) is a practical method to measure the full bearing capacity (both in end bearing and side shear) of a cast-in-place concrete pile (caisson). The O-Cell is a bi-directional sacrificial load cell consisting of a hydraulic jack mounted between two bearing plates and cast into concrete inside a drilled shaft which is also fully instrumented with strain gauges Linear Variable Differential Transformers (LVDTs), and tell-tales for measuring displacements (see Fig. 1). The O-Cell concurrently tests side shear (also known as "skin friction" or "shaft resistance") by pushing up, and end bearing plus side shear (below the O-Cell) by pushing down. Measurements made during the test include the pressure in the load cell, displacement of both load cell plates, and a full profile of loads and displacements along the shaft. As the hydraulic pressure in the jack is increased it exerts equal upward and downward loads on the caisson, using the upwards side shear to resist downward force. This allows the load test to be completed without the need for expensive, large, and dangerous reaction frames resisted by additional sacrificial caissons. Osterberg Cell load test instrumentation, testing, and factual reporting are provided by Fugro Loadtest ("Loadtest").

1.2 Post-Grouted Caissons

It is believed that high-capacity caissons bearing in soil in the Greater Toronto Area can be improved beyond the current state of practice. There is an opportunity to innovate with technology that is used internationally but is relatively new to the Greater Toronto Area market. The technology in questions involves post-grouting caisson bases using a 2$^{\text{nd}}$ round of high pressure grouting after the first concrete pour has set. Base post-grouting procedures and previous experience have been documented extensively by Dapp and Brown [1] and Mullins et al. [2].

The opportunity at the subject site is to conduct a second O-Cell test on a post-grouted caisson, and to compare both ultimate capacity and strain response with a conventionally constructed and O-cell tested caisson. It is hypothesized that post-grouting the base will have the following effects, which are described in detail by the authors above as well as Boeckmann et al. [3]:

- Post-grouting the base will create upward load and displacement. As side shear is now pre-engaged in the reverse direction of top-down loading, this should result in more side shear capacity before plastic deformation begins to occur. Side shear resistance

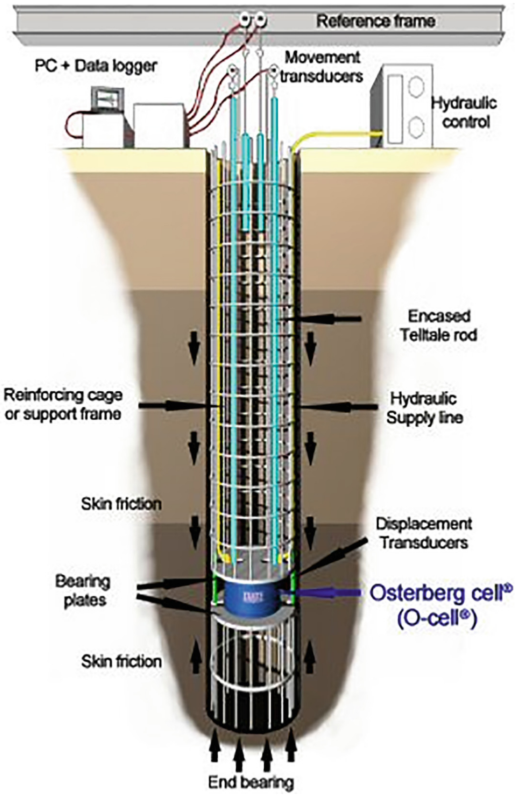

Fig. 1. Typical O-Cell Test Setup (Loadtest.com)

should therefore be improved. As a conventional top-down load test is required to observe and quantify this behaviour, it is out of scope for this study.

- Base stiffness may be increased by densifying any loose soil cave or sediment that collects at the base of each caisson prior to the concrete being tremied. Because high-capacity caissons are installed without casing to maximize side shear, it is possible for soils from the sidewall to slough off and collect at the base. Some sedimentation is also possible at the base of the caisson due to concrete wait times. This has been observed at other sites. Post-grouting acts to consolidate and densify loosened material at the pile base. Improvement in base stiffness arising from use of post-grouting can be measured and compared using O-Cell testing.
- For the end-bearing ("base") layer of dense sands and silts at this site, the use of post-grouting to improve reliability and mitigate risk during construction may be the greatest benefit of the technique, by ensuring that any unanticipated or unobserved loose soil at the pile tip is remediated on every caisson.
- Post-grouting should not result in an improved ultimate end-bearing capacity for pile tips made in very dense sands, as it is not possible to improve the soils further. The pressure exerted by post-grouting will preload the base. As post-grouting mobilizes unrecoverable strain at the base, the base will now be closer to ultimate end-bearing

failure. As such, ultimate end-bearing capacity may be lower for post-grouted caissons when compared to conventional construction.

1.3 Site Investigation

A detailed site investigation was carried out prior to the design of the load test. Thirteen (13) geotechnical boreholes were advanced by Grounded at the site, including nine (9) deep boreholes to over 55 m depth with pressuremeter testing for deep foundation design. A summary of index properties is provided in Table 1. The following stratigraphic units are interpreted for engineering purposes (see Fig. 2):

- A dense to very dense "cohesionless till" unit (matrix of sandy silt to sand and silt) underlies the earth fill. The base of this unit varies from about 5 to 15 m depth.
- Underlying the cohesionless till, a very stiff to hard "silts and clays" unit is observed. The base of this unit varies from about 40 to 43 m depth. The undrained shear strength of this unit is inferred to be over 200 kPa for present purposes. This is not an input parameter for test design, and is inferred based on SPT N-values.
- Underlying the silts and clays unit, a very dense wet "lower sands" unit (silty sand, to sand and silt) is observed. The base of this unit varies from about 49 to 52 m depth.

The groundwater table is shallow at this site, and in the area in general. Monitoring wells screened in the upper soil units observed the groundwater table as high as 1 m below grade. A monitoring well was also installed in the lower sands and observed a lower piezometric head within that unit at about 20 m depth.

2 O-Cell Caisson Test Design

The design and installation methodology of the load-tested caisson becomes the specification for all production caissons. Caisson diameter can be varied to a certain extent (as long as side shear and end bearing are corrected for diameter), but end-bearing elevation and installation procedure remain fixed. Therefore, it is critical that the sacrificial load test caisson be designed to anticipate what the final production caissons will be.

Table 1. Summary of Index Properties for Interpreted Soil Units

Soil Unit	SPT N-values (bpf)	Moisture Content (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	PMT Young's Modulus (MPa)
Cohesionless till	12 – 50 + (Avg. 54)	6 – 18 (Avg. 11)	n/a	n/a	n/a	n/a
Silts and Clays	13 – 80 (Avg. 45)	11 – 34 (Avg. 19)	15 – 23	34 – 55	17 – 32	66 – 162
Lower Sands	> 50	13 – 38 (Avg. 21)	n/a	n/a	n/a	249 – 292

Table 2. Production capacities for single caissons modelled by RSPile using O-cell data.

Caisson Diameter (mm)	Conventional Caisson ETL Capacities (Q_{ULS}/Q_{SLS}, MN)	Post-Grouted Caisson ETL Capacities (Q_{ULS}/Q_{SLS}, MN)
1200	16.8/15.5	16.8/16.3
1350	20.4/17.7	20.4/18.9
1500	24.0/20.3	24.0/21.6
1676	28.8/23.6	27.6/24.9
1830	33.6/26.5	32.4/28.0
1981	38.4/29.7	37.2/31.1
2286	49.2/36.3	45.6/37.8

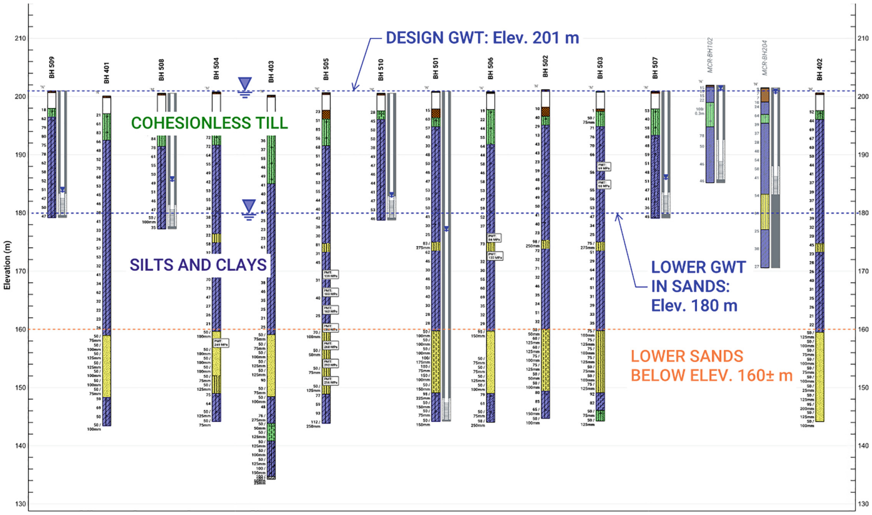

Fig. 2. A simplified stratigraphic profile of the site for engineering purposes.

A typical O-cell test is designed using a single caisson to balance upward (side shear) and downward (side shear plus end bearing) resistances. Balancing the test requires that the O-cell elevation within the caisson be specified. Ideally, both directions are equally mobilized, and measurements of maximal side shear and end bearing are made. In practice, balancing a test using desktop analysis is rarely achievable, even for experienced engineers with prior information.

The goal of the O-cell investigation at this site was to test post-grouted caissons and compare them with conventional capacities. The Developer also required that the use of conventional caisson construction not be ruled out by the test. Two O-cell load tests were therefore required: one on a conventionally installed caisson, and one on a post-grouted caisson. Because two caissons were to be tested, balancing each O-Cell test was not a prerequisite.

Column loads of about 17 to 32 MN were anticipated at preliminary design. A 1.5 m (±) diameter production caisson is anticipated on this basis, using assumed preliminary side shear and end-bearing parameters. It is common practice to test a production-sized caisson for side shear, whereas pile tip capacities can be more easily scaled for caissons of different sizes. The post-grouted test caisson was designed as a 1.5 m diameter test caisson to provide more side shear area to resist the larger anticipated resistance from a post-grouted pile tip, and to provide side shear parameters for both post-grouted and conventional construction options. This assumption is valid because post-grouting is not expected to improve shaft capacity, and is conservative in that side shear resistance is expected to be improved by post-grouting (although this is typically only quantified with a with a top-down load test) [3].

The 1.5 m diameter post-grouted caisson O-cell test was designed for about 25% more upward resistance, slightly weighted toward achieving end-bearing failure but also mobilizing enough side shear to get good side shear versus downward movement (t-z) data. It will provide end-bearing resistance versus downward displacement (Q-z data) for a post-grouted tip, as well as side shear resistance (t-z data) applicable to both conventional and post-grouted options.

The conventional (not post-grouted) caisson is designed to provide the maximal Q-z plot and ultimate end-bearing resistance for a conventional pile tip. The O-cell is placed near the bottom of this caisson, and the caisson diameter is minimized to 0.9 m, so that upward resistance is about 3 times higher than downward resistance. The goal was to mobilize the end-bearing resistance only, anticipating incomplete side shear resistance data.

The Canadian Foundation Engineering Manual (CFEM) [4] provides the following conventional equation for estimating the geotechnical axial capacity of a single pile (R):

$$R = \sum_{z=0}^{L} Cq_s \Delta z + A_t q_{ult} - Wp \tag{1}$$

Where C is the pile circumference, L is the embedded length subdivided into segments of length Δz, A_t is the pile toe area, and W_p is the pile weight.

Effective vertical stress is a factor in both side shear and end bearing resistance. Per the NAVFAC Design Manual [5], it is assumed that the vertical effective stress increases up to a limiting depth of embedment of 20B, where B is the pile tip diameter. Below this depth, the effective vertical stress is limited for the purpose of side shear and end bearing calculations. The pore water pressure profile is interpreted from monitoring wells nested in both units. Monitoring wells screened in the lower sands observed a lower piezometric head at Elev. 180 ± m, and the pore water pressure (u) profile is adjusted accordingly (Fig. 3).

2.1 End Bearing Resistance

The CFEM provides the following conventional equation for estimating the bearing capacity of a pile toe (q_{ult}) in cohesionless soils:

$$qult = Nt\sigma' t \tag{2}$$

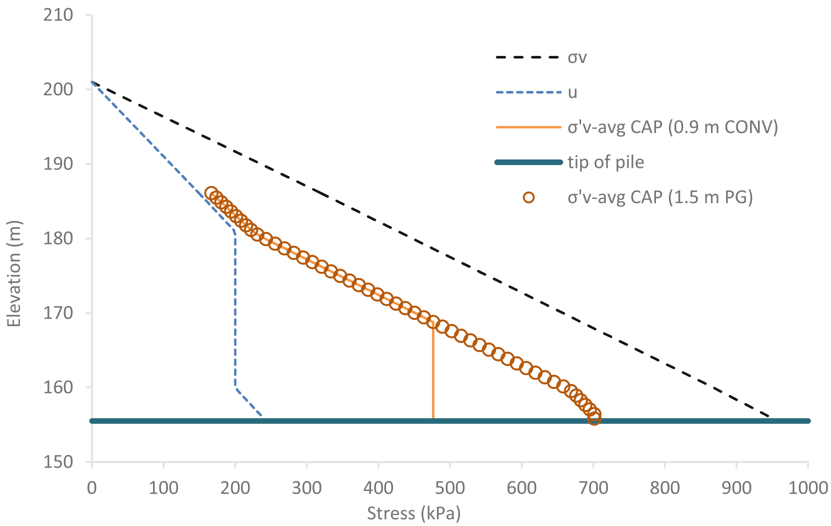

Fig. 3. Effective vertical stress profiles used for 0.9 m dia. Conventional ("0.9 m CONV") and the 1.5 m dia. Post-grouted ("1.5 m PG") caisson design.

where N_t is the bearing capacity factor (dimensionless) and σ'_t is the effective vertical stress acting on the pile tip (kPa).

A range of values for N_t for different soils are provided in the CFEM for preliminary purposes. For very dense sands, the CFEM suggests that N_t may be between 50 to 100. For the proposed 46 m long caissons at this site, a minimum q_{ult} of 34 MPa is calculated using a 20B-limited vertical effective stress and an N_t of 50. This value is unrealistic as it is more reflective of a bedrock end-bearing ultimate capacity. The CFEM preliminary parameters therefore may not apply to a 46 m long caisson in very dense sands, and caution is warranted.

Previous O-cell tests in the VMC area provided acceptable design values, but did not fully mobilize end-bearing ultimate resistance. When the maximum end-bearing results from previous VMC O-cell tests on the very dense lower sands are used to back-calculate an the mobilized end bearing factor N_t, the values of N_t are in the range of 8 to 10. Since end-bearing ultimate failure was not observed in those tests, the values of N_t at ultimate limit state must be higher. It is hypothesized that the actual N_t of a caisson loaded to ultimate limit state (i.e. failure) in these soils at this depth could be in the range of 15 to 20.

2.2 Side Shear Resistance

The CFEM provides the following conventional equation for estimating the unit shaft friction (q_s) in cohesionless soils:

$$qs = \beta\sigma'v \tag{3}$$

where β is the combined shaft resistance factor (dimensionless) and σ'_v is the effective vertical stress adjacent to the pile at depth z (kPa). β is a function of the lateral earth pressure at depth z, and the angle of friction between the soil and the caisson. The CFEM recommends using effective stress analysis (β method) for pile analysis in cohesive soils with an undrained shear strength of over 100 kPa, which is the case at this site. A range of values for β for various soils are provided in the CFEM for preliminary purposes. Based on the CFEM and prior experience in the area, a preliminary β value of about 0.4 was adopted for the very stiff to hard clay and silt unit, which will provide most of the side shear resistance. A β value of 0.3 was conservatively adopted for the lower sand.

Because side shear resistance is critical in the design of both the O-cell test and production caissons, all caissons are to be installed in uncased holes with polymer drilling methods to support the side walls. The construction method must be clearly and intentionally specified for both the O-cell test and production drilling, and carefully monitored by the Geotechnical Engineer of Record.

2.3 Final Test Design

According to the investigation objectives outlined above, desktop analysis was used to design the following two-caisson O-cell deep foundation load test program:

- One 900 mm dia. Conventional caisson (no post-grouting), with the O-Cell positioned at 0.5 m above pile tip elevation.
- One 1500 mm dia. Caisson with post-grouted pile tip, with the O-Cell positioned at 0.5 m above pile tip elevation.

All instrumentation was installed on an H-pile carrying frame. Both caissons were founded at Elev. 156 ± m and installed uncased with polymer drilling methods. Base cleaning and probing is specified in accordance with Sect. 3.7.3 of the ADSC Drilled Shaft Inspector's Manual [6]. SonicCaliper logging was specified to verify drilled shaft diameter and verticality and to identify zones of sloughing in the sidewalls.

3 O-Cell Results and Interpretation

The O-cell load testing specified above was completed successfully on site in September 2022. Instrumentation, testing, and factual data reporting was carried out by Loadtest. Both tests were terminated successfully at maximum system pressure. Grounded observed the installation and testing in the field.

3.1 End Bearing Results

In order to evaluate the end bearing component of a 900 mm diameter test caisson scaled up to a 1500 mm diameter production caisson, Grounded followed the Loadtest recommended procedure to scale the end bearing settlement using the simple theory of elasticity of the settlement of a rigid disk subjected to uniform pressure on an elastic medium with a consistent Young's Modulus and Poisson Ratio, as follows:

$$S_{Foundation} = S_{Test}\left[\frac{D_{Foundation}}{D_{Test}}\right] \tag{4}$$

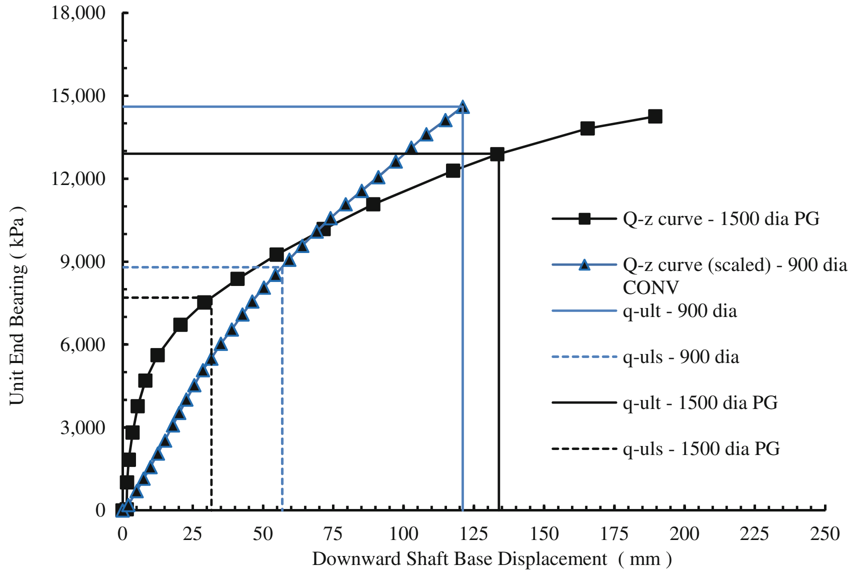

Fig. 4. Unit end bearing data for both the 900 mm dia. Conventional caisson test (scaled up to a 1500 mm dia. Equivalent caisson) and the 1500 mm dia. Post-grouted caisson, showing interpreted ultimate and ULS parameters. The Figure clearly demonstrates the differences in behaviour between the two caisson types. The post-grouted tip exhibits a stiffer initial response, but also results in a slightly lower ultimate bearing capacity due to the "pre-load" from post-grouting.

where $S_{foundation}$ is the predicted settlement of the foundation, S_{test} is the measured settlement in the load test, $D_{foundation}$ is the diameter of the proposed foundation, and D_{test} is the diameter of the plate/caisson in the load test.

The end-bearing resistance data (Q-z curves) are shown in Fig. 4 for equivalent 1500 mm dia. Caissons, to provide a direct comparison. q_{ULT} was evaluated using several different interpolation methods, including final pressure, the Davisson Offset Limit Load and the De Beer Yield Load [7], and the 6% diameter criterion. For the 900 mm dia. Conventional caisson, there was no significant creep displacement of the toe at maximum pressure, indicating that ultimate end-bearing failure had not yet occurred. For the 1500 mm dia. Post-grouted caisson, downward apparent creep was observed at about 29 MN of downward force indicating that ultimate capacity was exceed for this caisson. A reduction factor $\Phi_R = 0.6$ (per the CFEM for a static in situ load test) was applied to q_{ULT} calculate q_{ULS}, the factored geotechnical end-bearing capacity of each pile at Ultimate Limit State (ULS).

The final Q-z curves were input directly into RSPile for analysis.

3.2 Side Shear Results

Side shear data is acquired by strain gauges instrumented along the length of the pile. Each strain gauge level has an elevation, and each zone refers to the pile length between two strain gauge levels. All strain gauges are located above the O-cell elevation, where the caisson is resisting upward loading. The strain gauge level layouts are shown in Fig. 5.

The resulting unit side shear (t-z) data for both tests is shown in Figs. 6 and 7. The horizontal axis ("upper average zone movement") indicates that all shear zones are moving upwards, as the O-cell is positioned at the pile tip for both tests in this study. It was found that side shear capacities were significantly higher within Zones 1 and 2 (below about Elev. 162 ± m) in the post-grouted caisson. Within these lowest zones, measured side shear resistance was about twice as much as that measured in the conventional caisson test. Further, post-peak strain softening behaviour is evident in the Zone 1 curve, whereas this was not the case in the conventional test. Although a direct comparison is not possible due to the different caisson sizes, this indicates that the grout bulb from the post-grouting procedure extended up the side of the pile tip within these zones, indicating a secondary and, to-date, not well documented, benefit of post-grouting.

Due to this apparent and unanticipated improvement in side shear caused by pile tip post-grouting, side shear data from the 900 mm diameter conventional caisson test was

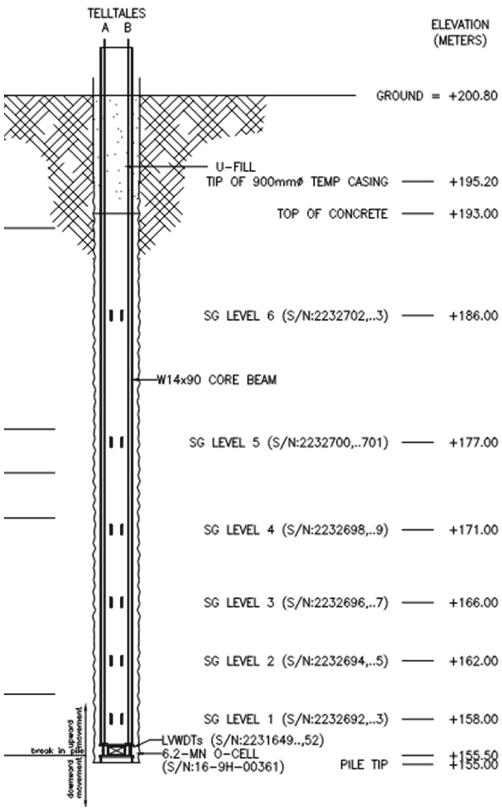

Fig. 5. Strain gauge levels and elevations used for the 900 mm dia. Caisson test (and identical for the 1500 mm dia. Caisson test), from Loadtest.

used for Zones 1 and 2 when analyzing conventional caisson capacities. The final t-z data were input directly into RSPile for analysis. A separate "material" was defined in RSPile according the elevations of each strain gauge zone, and the t-z curve for each zone was entered accordingly.

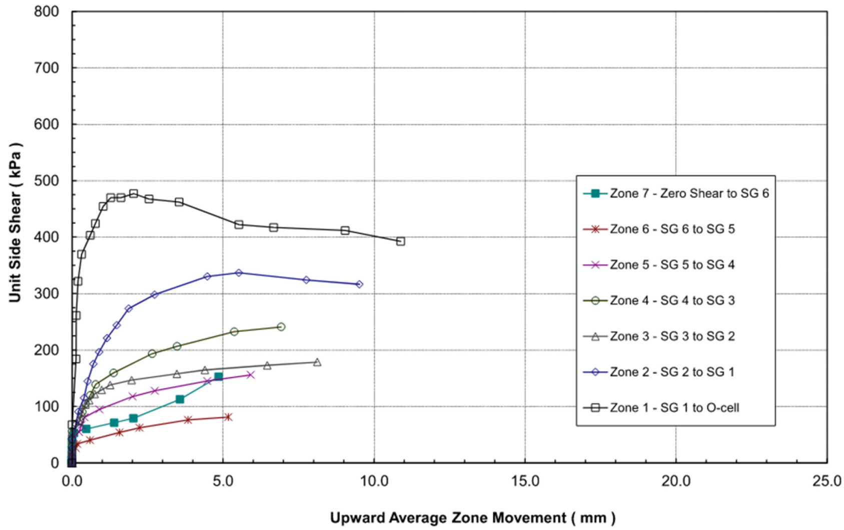

Fig. 6. Unit side shear (t-z) data for the 1500 mm dia. Post-grouted test, to be utilized for the design of both post-grouted and conventional caissons.

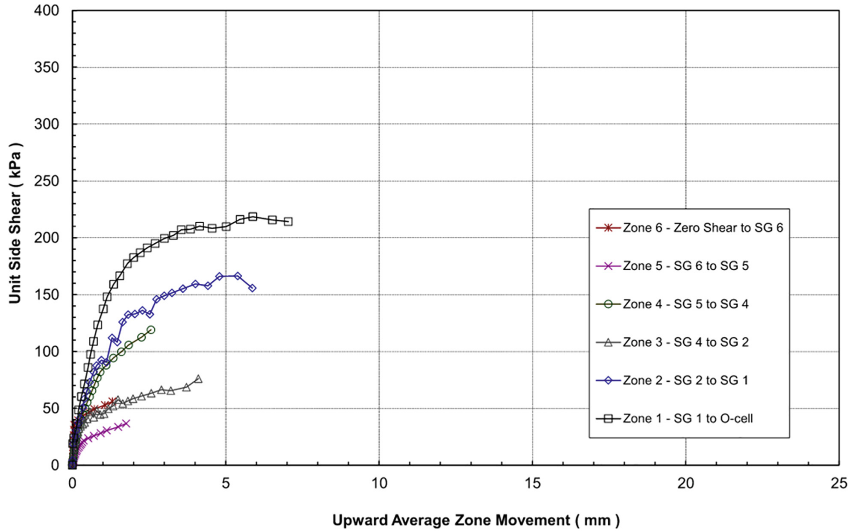

Fig. 7. Unit side shear (t-z) data for the 900 mm dia. Conventional test, showing a clear difference in side shear behaviour in the lower Zones 1 and 2.

3.3 RSPile Modelling

When a caisson is loaded by a bidirectional O-cell, it measures upward and downward resistance based on the position of the O-cell. However, a production pile is loaded from the top, and in that case all movement is down. Uplift design is not covered in this study). Equivalent Top Load (ETL) curves estimate pile reactions under top-down loads, and can be derived from bidirectional O-cell testing. Loadtest can estimate an approximate ETL for each test using a basic closed-form additive approach and simplified assumptions on pile elasticity and behaviour under loading.

A 46 m long caisson will compress and will not act as an infinitely rigid element. As such, top-down axial resistance is dependent on the stiffness of the pile. ETL stress-strain response also depends on the points at which side shear and end bearing start to behave plastically, which is related to how the caisson sheds load with depth. To properly account for this complex behaviour, RSPile (version 3.015) was used to numerically model production piles of different diameters for axial capacity under a top-down loading scenario.

RSPile is a general pile analysis software package that can compute the stress-strain relationship of an axially loaded pile. It assumes three mechanisms: axial deformation of the pile, soil skin friction ("side shear" in this study) along the shaft, and soil end-bearing (see Fig. 8). RSPile discretizes the pile into elements, and uses finite element analysis of the discretized pile to solve the governing differential equation using the "t-z curve method", which allows for non-linear stress-strain behaviour in soil by employing t-z and Q-z curves. Stiffness at each iteration is computed using the solved displacement values [8]. Corrected t-z and Q-z curves from the load tests are input directly into RSPile as "user defined soil models". When the soil displacement exceeds the last entered displacement value on each curve, the soil resistance is assumed to be the last entered resistance.

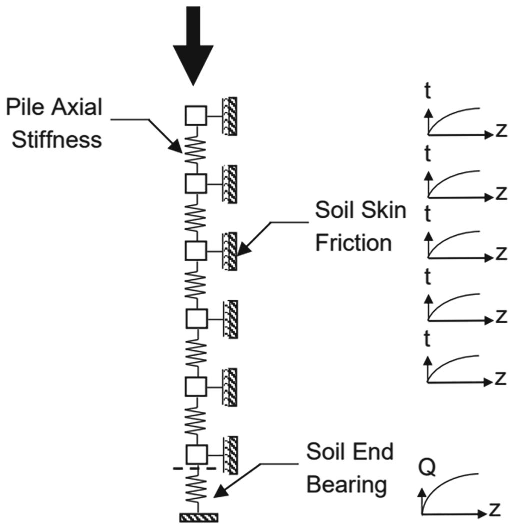

Fig. 8. The three different loading mechanisms that can describe the stress-strain relationship of axially loaded bored piles (from Rocscience).

The first modelling step is to set up an RSPile model to "reproduce" the O-cell test, to validate modelling parameters. The test caissons themselves are modelled with the actual steel reinforcement (core beam) as well as the concrete strength and stiffness reported by Loadtest. RSPile computes the structural pile stiffness directly.

RSPile was used to numerically model both O-cell load tests, assuming bidirectional loading at O-cell elevation. Equivalent Top Load (ETL) curves were then modelled in RSPile for both tests (see Fig. 9), which are then used to verify the ground modelling parameters.

It was found that the Convergence Tolerance (CT) modelling parameter plays a large role in simulating ETLs with RSPile. This parameter describes how closely finite element modelling iterations need to converge before the modelling is deemed "complete". A smaller CT produces more modelling iterations. If the numerical model is adequately converging, output should not change substantially with more iterations (i.e. with a lower CT).

RSPile simulated the Loadtest ETLs most closely at a CT of 0.01. However, RSPile ETLs changed significantly as more iterations were used (i.e. a lower CT), as shown above. Modelling results were not sensitive to CTs lower than 0.0001, which therefore became the modelling parameter used for all subsequent modelling.

The numerically modelled RSPile ETLs are more conservative (there is more displacement per load step) than the closed-form approximation provided by Loadtest. As RSPile's finite element model is more robust than the simplified closed-form desktop ETL estimate, it is adopted on this basis. A larger comparison of desktop versus numerically modelled ETLs could warrant further study.

3.4 Effective Vertical Stress Corrections

The geotechnical axial capacity of a single pile is a function of effective vertical stress along the length of the pile (limited at a depth of 20B). The O-cell tests were done from ground surface. The future development will have three basements (P3) set about 12 m below existing grade. As the basement will be drained, the groundwater table will be lowered to just below the P3 elevation, implying a slight increase to effective vertical stress. Production piles will therefore be loaded from a P3 elevation of about 189 ± m, which implies bulk excavation of about 12 m of soil and a resulting net reduction in effective vertical stress. This will have the effect of reducing pile capacities in both side shear and end bearing. Because O-cell data is measured under the existing higher stress regime, it needs to be corrected to account for a change in effective vertical stress when modelling production piles.

The net reduction in effective vertical stress is assessed along a 46 m caisson. Settle3 (version 5.018) was used to model effective vertical stress change before excavation (σ'_{v1} as reflected in the O-cell data) and after construction (σ'_{v2} as it would apply to production piles). Settle3 accounts for the 3D effects of the excavation as well as changes to the groundwater table elevation. For each zone, a multiplier is calculated to reduce side shear, as follows:

$$SideShearMultiplier = \sigma' v2/\sigma' v1 \tag{5}$$

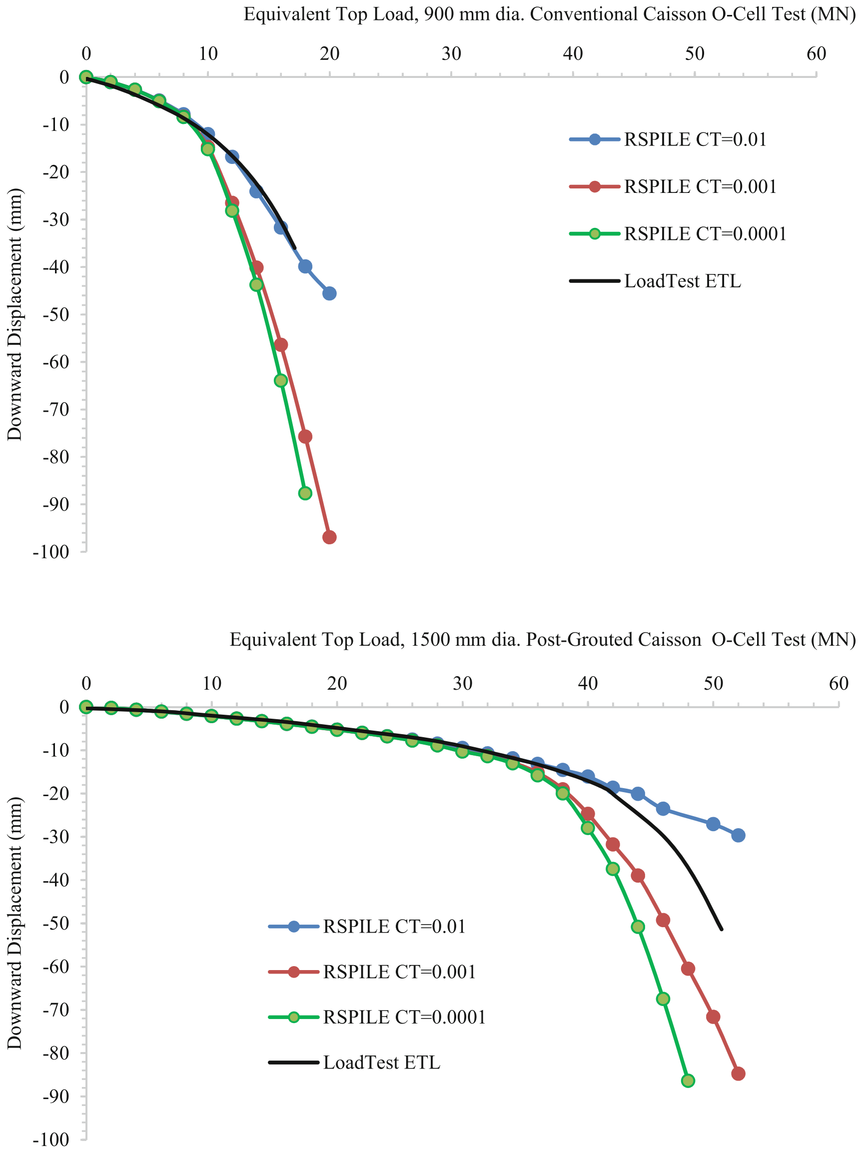

Fig. 9. Equivalent Top Load Curves for both O-cell tests reported by Loadtest, compared with RSPile output for various convergence tolerances.

Settle3 modelling shows that the excavation did not significantly change the effective vertical stress in the lower sand. As such, the end-bearing (Q-z) results for both pile types were determined to be unaffected by excavation.

3.5 Production Pile Modelling

Once the ground model was verified, RSPile was used to model production piles of different diameters using both conventional and post-grouted bases, corrected t-z and Q-z data, and a sufficient Convergence Tolerance (CT). ETL curves were modelled by RSPile for 7 different production pile diameters with both conventional and post-grouted tips (14 scenarios). The data are shown in Fig. 10.

Ultimate limit state pile capacity was assessed from the RSPile ETL modelling data by determining which ETL load step causes plastic behaviour to fully develop in both side shear and end-bearing (i.e. large changes in displacement to occur). The ultimate capacity (R, or Q_{ULT}) is factored by 0.6 per the CFEM to determine the factored geotechnical axial capacity of each pile at ULS (Q_{ULS}).

The net geotechnical reaction at Serviceability Limit State (SLS) for this project, Q_{SLS}, was defined as the ETL that produces a downward deflection of 25 mm at the top of each pile. The modelled top-down deflection considers pile stiffness, nonlinear load distribution along the length of the shaft, and plastic behaviour assessed separately in each side shear zone and at the base. To verify the Q_{SLS} loads, RSPile was used to assess the induced end-bearing pressure on each pile at Q_{SLS}. It was found that SLS bearing pressures at the pile tips were all in the range of 2100 to 3700 kPa, and that pile tips moved as much as 18 mm as modelled. When checked against the Boussinesq closed-form solution for deflection under a circular load and using the pressuremeter data in the very dense sands, 9 to 20 mm of deflection at the base is estimated under these loads, which further confirms the modelling (Table 2).

As 17 to 32 MN column loads (SLS) were anticipated during preliminary O-cell design, a variety of caisson diameters are available to make caissons as efficient as possible. Once the detailed caisson layout is available for review, group effects will be evaluated.

ETLs from conventional and post-grouted caisson of similar diameters and depths can be compared to evaluate the effectiveness of post-grouting. Q_{SLS} is improved by about 4–7% because of post-grouting the bases, which supports the initial hypothesis that post-grouting would produce stiffer ground support. ETLs for selected caisson diameters are compared in Fig. 11.

4 Conclusions

The RSPile analysis of the corrected O-cell data provides the following conclusions:

- Because side shear and end bearing reactions do not engage simultaneously or reach plasticity uniformly, numerical modelling supported by O-cell testing is recommended for determining ETLs for highly loaded caissons supported in soil.
- Q_{ULS} is not improved by post-grouting the bases in these very dense sands, which supports the initial hypothesis.
- Q_{SLS} is improved by about 4–7% as a result of post-grouting the bases, which supports the initial hypothesis that post-grouting would produce stiffer ground support.

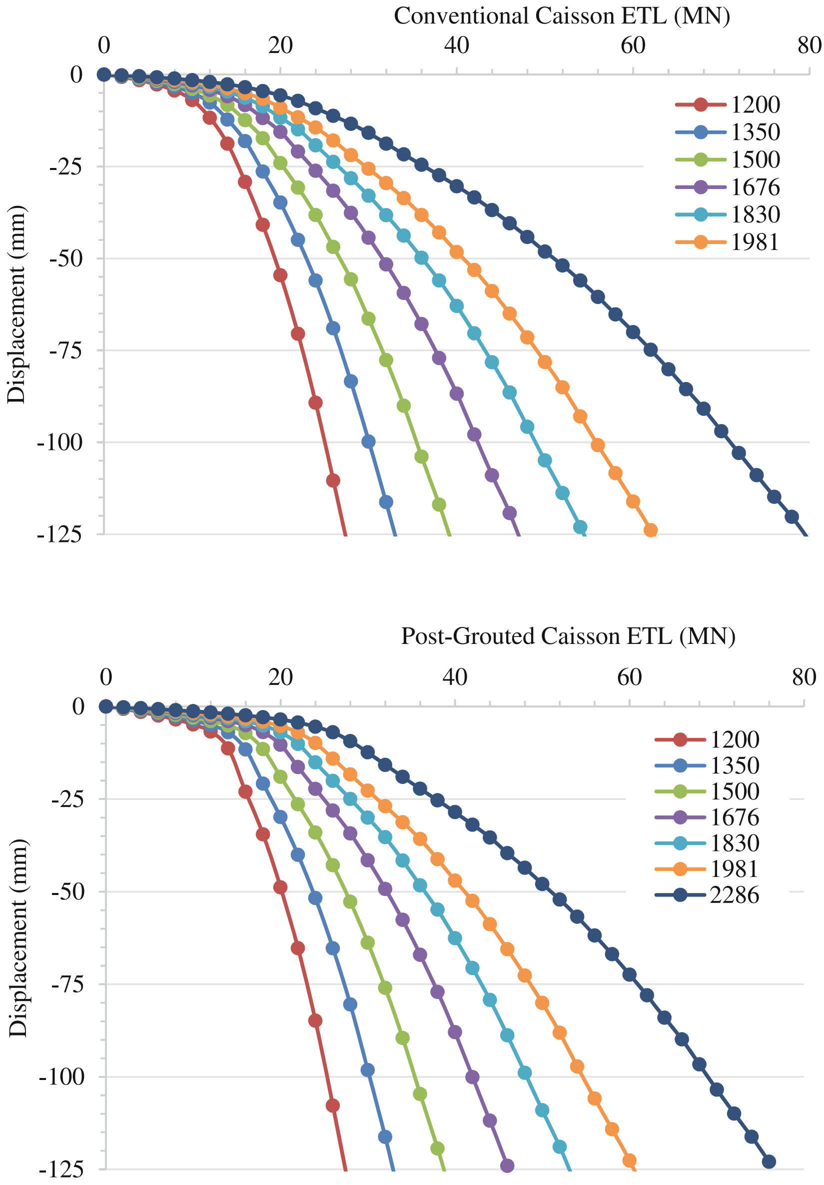

Fig. 10. RSPile Equivalent Top Load (ETL) Curves for Both Conventional and Post-Grouted Caissons of Different Diameter

- Back-calculating the O-cell results to obtain an equivalent N_t (per the CFEM) suggest values of about 18 for the post-grouted caissons, and about 20 for the conventional caisson (noting that failure was not observed in that test), which supports the initial

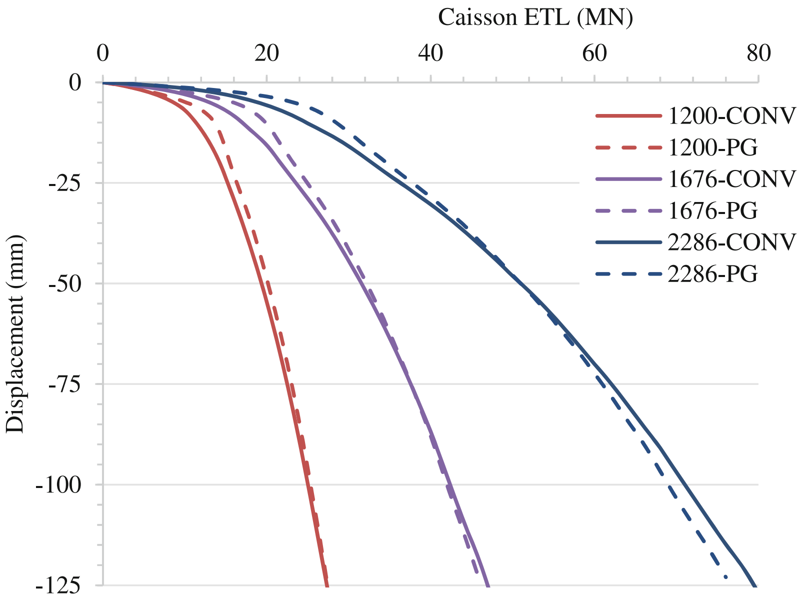

Fig. 11. Direct Comparison of Conventional ("CONV" vs. Post-Grouted ("PG") Caissons using RSPile ETLs.

hypothesis. Caution is therefore warranted when applying typical CFEM N_t parameters. Effective overburden pressure should also be limited at a depth of 20B, per NAVFAC Design Manual 7.02.

- Post-grouting provides an important but non-quantifiable benefit of improving reliability in caisson design, as it mitigates the risk of any base loosening that may occur due to production pile installation deficiencies.
- The O-cell data in this study indicates that post-grouting of the base also improved the unit side shear capacity close the base. This was contrary to the initial hypotheses that post-grouting would not act on the shaft, and that side shear improvement could not be observed with an O-cell test.
- The caisson pile tips support about 17 to 38% of the total SLS load in end bearing. The relationship is nonlinear and depends mainly on diameter, plasticity in the side shear zones, and the corrected t-z and Q-z data.
- The majority of the SLS load (62 to 83%) is supported by side shear resistance, which are therefore critical to final performance. Specialized caisson drilling methodology is required to maximize side shear, which must be carefully observed and recorded by the Geotechnical Engineer of Record.
- RSPile modelling is sensitive to Convergence Tolerance. An assessment of the appropriate convergence tolerance must be used to determine the final CT used in design.
- Production pile modelling is sensitive to reduction in effective vertical stress (i.e. bulk excavation). This must be accounted for when correcting t-z and Q-z curves for the final excavated condition.

- The conventional desktop approach to constructing ETLs appears to become unconservative at higher loads, when compared with ETLs numerically modelled by RSPile using an acceptable CT. This topic warrants further study.

The post-grouting of caisson bases is an excellent risk mitigation measure for deep caissons. The increased cost of installation must be evaluated against the increase in axial capacity, and the design team's risk tolerance.

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