Static axial load test on strain gauge instrumented concrete piles

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ABSTRACT
The Edmonton New Remand Center (ENRC) will be a multi-building complex located in Edmonton, Alberta. The foundation system for the ENRC comprised more than 2,600 continuous flight auger (CFA) piles. The subsurface conditions at the project site consisted of lacustrine deposits of clay and silt underlain by highly variable strata of glacial clay till and clay shale bedrock. A pile load testing program was undertaken to optimize the design of the piles by determining the shaft resistance being developed along the length of the test piles within the different subsoil strata. The load testing program consisted of a total of four axial compressive load tests on strain gauge instrumented CFA concrete test piles. The pile load tests permitted a significant increase in the design shaft friction parameters, which provided substantial cost savings for the project foundations.

RÉSUMÉ
Le Nouveau Centre de Détention d'Edmonton (NCDE) sera un complexe de plusieurs bâtiments situé à Edmonton, en Alberta. Le système de fondation pour le NCDE comprend plus de 2600 pieux de vols tarière continue (VTC). Les conditions de souterraine au site du projet se composait de dépôts d'argile et de limon lacustres sous-tendus par les couches très variables d'argile de blocaux jusqu'au socle rocheux de schiste argilleuse. Un programme des essais de chargement de pieu a été entrepris afin d'optimiser la conception des pieux par la détermination de la résistance des fûts qui se développait le long des pieux d'essai dans les couches de sols différentes. Le programme de chargement de pieu comprenait un total de quatre essais de chargement en compression axiale sur la jauge de déformation des pieux d'essais en béton VTC. Les essais de chargement de pieu ont permis une augmentation importante des paramètres de conception des fûts, qui ont fourni des économies substantielles pour les foundations du projet.

1 INTRODUCTION

The Edmonton New Remand Center (ENRC) is a multi-building complex under construction in the north end of Edmonton, Alberta. The complex will include a central structure comprised of a main building area as well as healthcare and institutional services facilities. Several pods will be connected to the main structure via links. The subject site is approximately 40 acres including the proposed parking area, site accesses and roadways. The construction is expected to be completed in 2010. Figure 1 shows the approximate location of the subject site.

Continuous flight auger (CFA) concrete piles were used for the ENRC project. A total of 2,602 friction piles were installed, with pile diameters ranging from 406 mm to 914 mm and pile lengths ranging from approximately 5.2 m to 18.8 m.

A pile load testing program was undertaken in the early stages of pile installation. The objective of the pile load tests was to optimize the design of the piles by identifying the shaft resistance being developed along the length of the test piles. The testing program consisted of the construction of four dedicated test piles, installation of instrumentation for each test pile, monitoring pile behaviour during loading, data reduction and analyses, and design implications of the test results. Figure 2 is a ENRC site layout showing the locations of the four test piles.

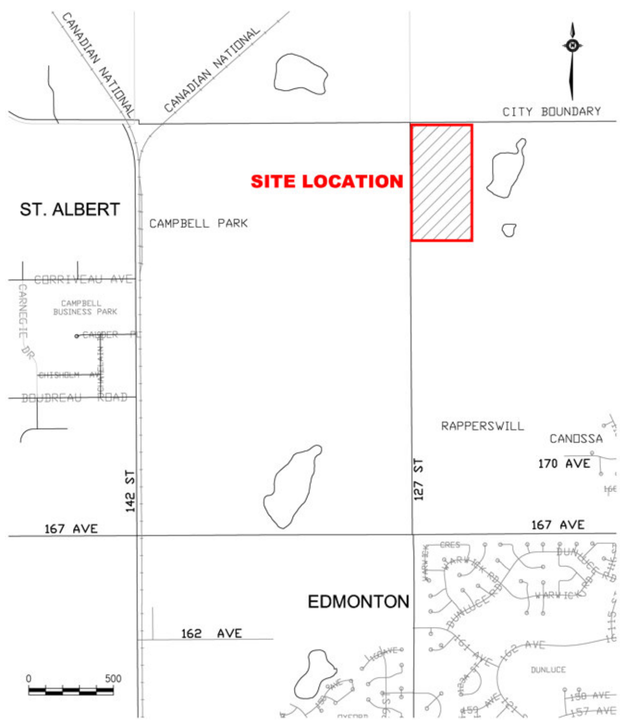

Figure 1. ENRC site location plan

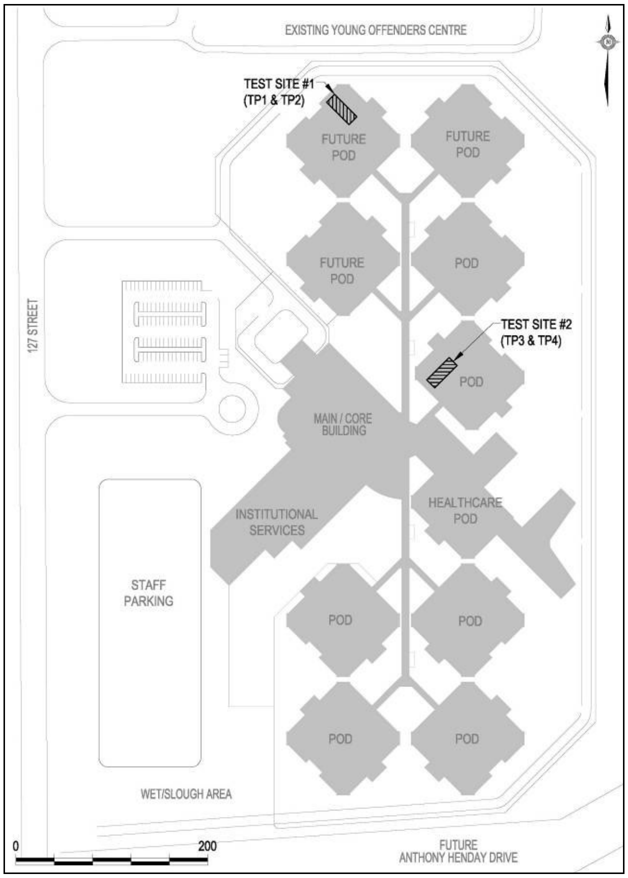

Figure 2. ENRC site layout showing test pile locations

2 SUBSURFACE CONDITIONS

In general, the subsurface conditions across the subject site are highly variable. A surficial layer of lacustrine clay and silt deposits are underlain by highly variable strata of glacial clay till and bedrock. The upper surfaces of the clay till and bedrock strata are variable. The clay till is discontinuous at some regions of the project site, while the bedrock is ice rafted at most locations. Discontinuous seams or pockets of water-bearing sand were encountered within or overlying the clay till. Wet coal seams were encountered within the bedrock.

A sand and gravel pad approximately 1.5 m thick was placed on the project site before the pile installation. The sand and gravel fill was well graded with a compact to dense consistency.

2.1 Test Site #1

A simplified borehole log describing the subsurface soil conditions at Test Site #1 is presented in Figure 3.

Alternating deposits of clay and silt were encountered underlying the sand and gravel fill material to depths of 12.7 m below grade. The clay is silty, moist to wet and saturated, medium to high plastic with a firm to very stiff consistency. Wet to saturated silt seams were encountered within the clay. The silt has some clay, is wet to saturated, low to medium plastic with medium to rapid dilatancy. Clay layers were encountered in the silt.

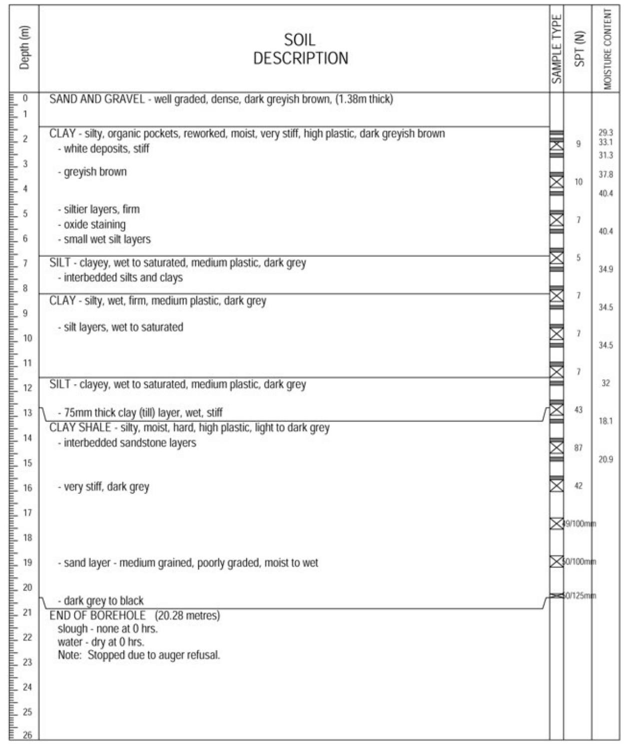

Figure 3. Simplified borehole log at Test Site #1

The native lacustrine deposits were underlain by clay shale bedrock to the termination depth of the borehole (20.3 m). The clay shale is silty, moist, high plastic and of hard consistency. Thin sandstone layers were encountered within the clay shale.

2.2 Test Site #2

A simplified borehole log describing the subsurface soil conditions at Test Site #2 is presented in Figure 4.

Native lacustrine clay was encountered underlying the sand and gravel fill material to a depth of 6.1 m below grade. The clay is silty, moist to wet, medium to high plastic with a soft to very stiff consistency. Wet to saturated silt seams were encountered within the clay. Silt was encountered underlying the clay to a depth of 9.5 m below grade. The silt is clayey, wet, medium plastic with medium to rapid dilatancy. Clay and sand layers and coal inclusions were encountered within the silt.

Interbedded clay till and clay shale bedrock were encountered underlying the silt to the termination depth of the borehole (24 m). The clay till is silty with some sand, moist, medium plastic with stiff to very stiff consistency. Coal and clay shale inclusions were encountered within the clay till. The clay shale is silty, moist, high plastic with a very stiff to hard consistency. Interbedded layers of sandstone, sand, till, and coal seams were encountered within the clay shale.

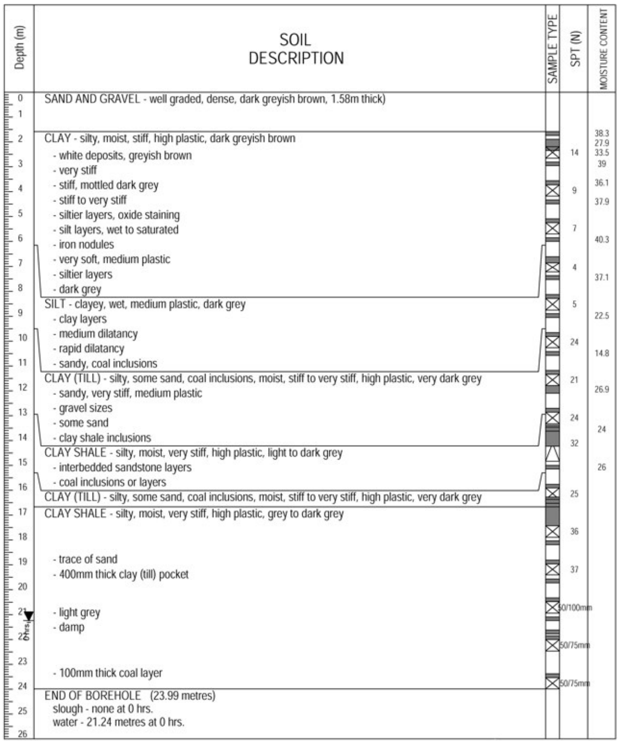

Figure 4. Simplified borehole log at Test Site #2

3 TEST PILE INSTALLATION

The pile load testing program consisted of axial compression load tests of four CFA piles at two test sites. The test pile layout consisted of two test piles (TP) and six reaction piles (RP) at each test location, as shown in Figure 5.

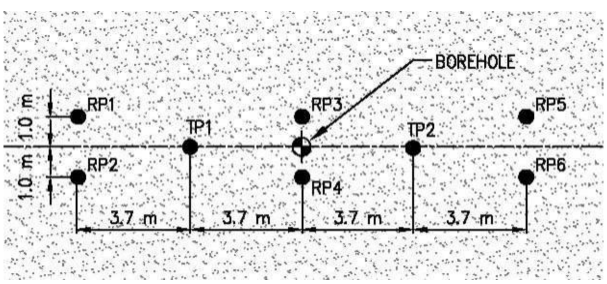

Figure 5. Typical test pile layout

The test piles were installed to approximate depths of 10.0 m and 19.8 m below grade. TP1 to TP3 had pile diameters of 400 mm, and TP4 had a pile diameter of 600 mm. Table 1 summarizes the test pile dimensions at the two test sites.

During the CFA pile installation program, the piling rig computer monitored and recorded the installation details for each test pile, including the as-built pile geometry, pile shaft profile, concrete pressure, auger drilling rate and lifting rate.

Table 1. Test pile properties

Test Site	#1		#2	
Test Pile No.	TP1	TP2	TP3	TP4
Depth Drilled (m)	19.8	10	19.8	10.3
Pile Stick-up (m)	0.6	0.5	0.8	0.4
Total Pile Length (m)	20.4	10.5	20.6	10.7
Pile Diameter (mm)	400	400	400	600
Design Capacity (kN)	1000	260	1000	400

4 PILE LOAD TEST PROGRAM

4.1 Strain Gauges in Test Pile

Due to the variable stratigraphy of the project site comprising lacustrine clay and silt, clay till, and clay shale bedrock, each of the strata will develop different shaft friction. Each test pile was instrumented with vibrating wire strain gauges at select levels in each test pile to obtain information about the shaft friction developed within the different subsoil strata.

A Vibrating Wire Rebar Strain Meter (Model 4911), or "sister bar," supplied by Geokon Incorporated, was used in the pile load testing program. A sister bar consists of a 1.38 m long steel bar with a vibrating wire strain gauge sensor fixed axially inside the central length of the steel bar. The rebar extensions on either side of the strain gauge are long enough to ensure good contact with surrounding concrete. It is assumed that the measured strains inside the steel bar are equal to the strains in the surrounding concrete.

The sister bars were installed by tying them alongside an existing length of rebar to the reinforcement steel cage before test pile installation. Figures 6 and 7 illustrate the strain gauge installations on the rebar cage.

The strain gauges were installed at eight levels for the 19.8 m long piles (i.e., TP1 and TP3), and at four levels for the 10 m long piles (i.e., TP2 and TP4). At each specific level, a total of three strain gauges were installed.

Figure 6. "Sister bars" installed on the rebar cage (note: leads bundled to vertical rebar)

Figure 7. “Sister bars” offset from the vertical rebar

4.2 Test Pile Setup

Each static pile load test consisted of one CFA test pile, four CFA reaction piles, a steel reaction beam and two load transfer beams at each end. The ends of the load transfer beams were secured to the reaction piles using a 32 mm diameter Dywidag bar embedded the full length of each reaction pile.

A 4500 kN hydraulic jack was used to apply load to the test piles. A load cell, as shown in Figure 8, was placed between the reaction beam and the hydraulic jack to accurately monitor the load applied by the hydraulic jacking system. A pair of hemispherical bearing plates was installed to minimize eccentric loading. Pile settlement was measured at the pile head using one displacement transducer and two dial gauges that were mounted on two fixed reference beams.

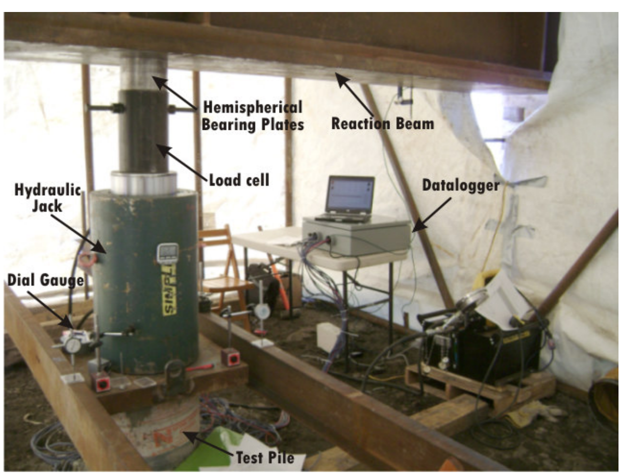

Figure 8. Test setup and instruments

The readouts from each of the strain gauges, load cell and displacement transducer were monitored and recorded by a datalogger with a sampling rate of every 30 seconds. A laptop computer was connected to the data logger for real-time monitoring of all the instruments on the computer screen. The sampling frequency of the load cell readout on the computer screen was every 2 seconds, which enabled a prompt display of the load on the screen and allowed for the load to be adjusted, if necessary. The output of all the instruments (load cell, displacement transducer, and strain gauges) was converted into engineering units by applying corresponding calibration factors for each of the instruments.

4.3 Pile Loading Procedure

The loading procedure for each pile load test was conducted generally in accordance with ASTM D1143M-07 using Procedure B (Section 8.1.3). During each of the pile load tests, the loading increments were based on 25% of the design load. Load increments were not increased until the specified settlement criteria (0.25 mm per hour) was achieved at each load level. The minimum and maximum holding times at each load increment were 20 min and 60 min before 200% of the design load level was achieved. The 200% design load level was then held for a minimum of 12 hours, followed by an unload-reload cycle. The loading decrements and increments in the unload-reload cycle were based on 50% of the design load. After the unload-reload cycle, the loading increments applied were 25% of the design load, until completion of the load test.

5 PILE LOAD TEST RESULTS

Results from two test piles are presented in this section and subsequent sections. The measured load displacement curves for pile tests TP1 and TP4 are presented in Figures 9 and 10 respectively. The unload-reload cycle of each pile load test is included in each graph.

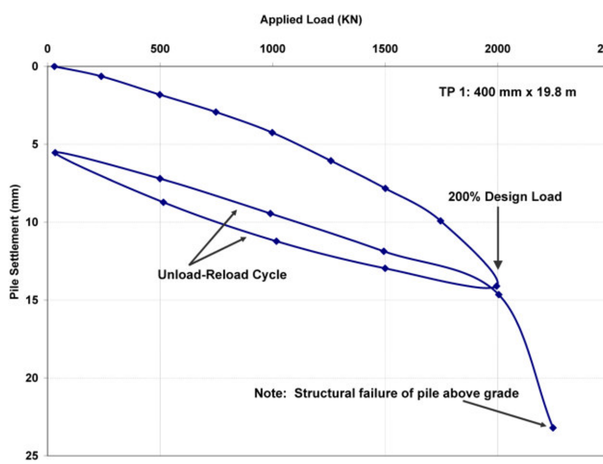

Figure 9. Load displacement curve (TP1)

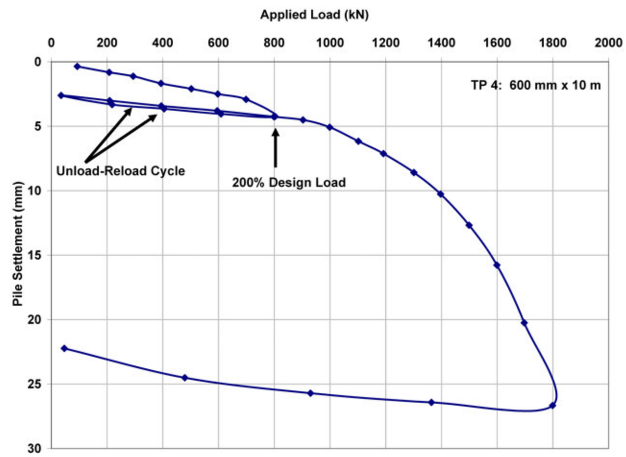

Figure 10. Load displacement curve (TP4)

The pile settlement at a specific load level may be interpreted from the load displacement diagrams. For TP1 (19.8 m in length and 400 mm in diameter), the pile settlements at 100% and 200% of design load (1000 kN and 2000 kN) are estimated to be less than 4 mm and 14 mm, respectively. For TP4 (10 m in length and 600 mm in diameter), the pile settlements at 100% and 200% of design load (800 kN and 1600 kN) are estimated to be less than 2 mm and 5 mm respectively.

The load/displacement performance of the test piles allowed the design team to confirm the pile capacity of these two pile lengths and diameters. Note that TP1 experienced structural failure when the last few load increments were applied. Surface cracking and pile head concrete deformation were noticed. If TP1 had not failed structurally, it may have been possible to achieve higher capacities in the pile. The structural failure of the test pile also complicated the interpretation of the strain gauge readings and shaft resistance calculation.

6 INTERPRETING STRAIN GAUGE MEASUREMENTS

6.1 Strain Gauge Analysis Methodology

Typically, the loads in the pile at the plane of the strain gauges are computed from the measured strains and an estimated modulus using the following equation:

$$P = \varepsilon A E \qquad [1]$$

Where:
P = Load (kN)
ε = measured strain from gauges (microstrain)
A = composite cross-sectional area of concrete and steel (m^2)
E = composite modulus of concrete and steel (kPa)

The determination of the composite modulus of concrete and steel is complicated due to the uncertainty of the concrete modulus (Hayes and Simmonds 2002). The modulus of steel is constant; however, the modulus of concrete varies and is a function of the imposed load. As a result, the pile's composite modulus is a linear rather than a constant function of the imposed strain (Fellenius et al. 2000). The "tangent modulus" analytical method (proposed by Fellenius 2001) was adopted to convert the measured strain into load in the pile at each gauge level.

6.2 Strain Gauge Data Results

Plots of the applied load versus measured strain at each strain gauge level are presented in Figures 11 and 12 for pile tests TP1 and TP4, respectively. Higher strain values were measured at shallower gauge locations (i.e., the first several rows of strain gauges from the pile head), whereas less strain was measured with increased distance from the gauge levels to the pile head. The unload-reload cycle is also plotted on each graph.

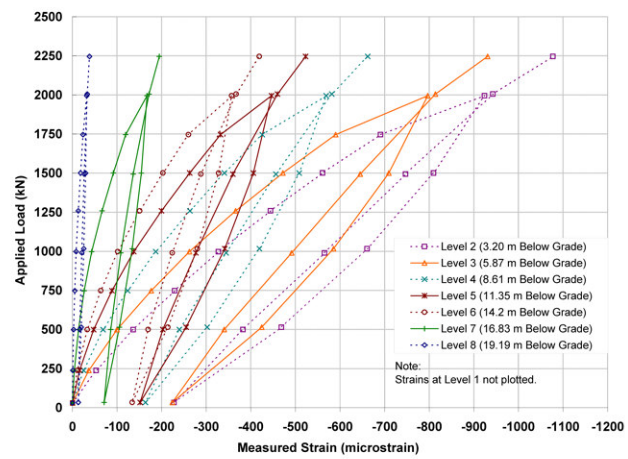

Figure 11. Applied load vs. strains (TP1)

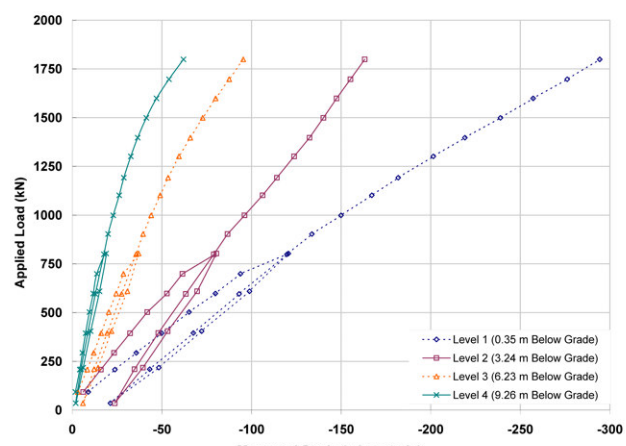

Figure 12. Applied load vs strains (TP4)

6.3 Estimating Shaft Resistance

The load in the test pile was calculated from the strains at each gauge level by using the procedure described in Section 6.1. The load distribution for each loading increment is presented in Figures 13 and 14 for pile tests TP1 and TP4, respectively.

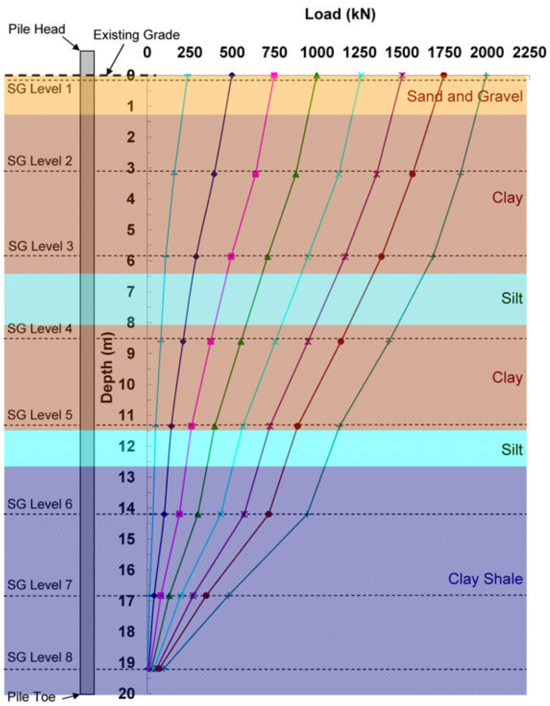

Figure 13. Calculated Load Distributions (TP1)

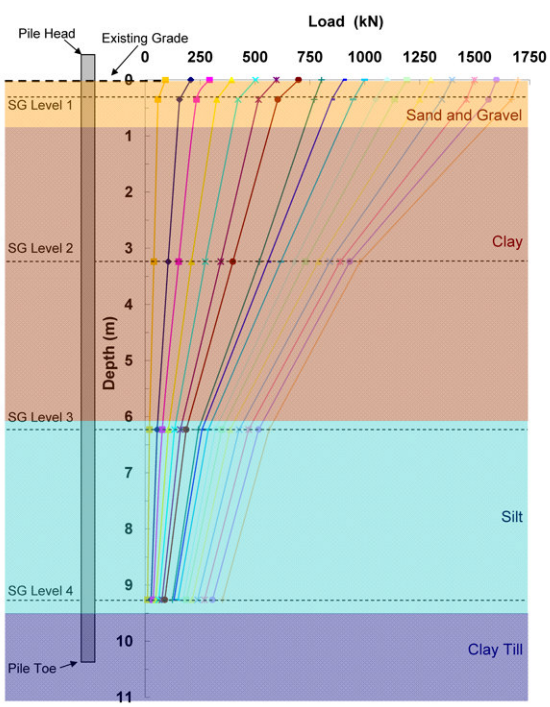

Figure 14. Calculated Load Distributions (TP4)

In the load distribution diagrams (Figures 13 and 14), as the increment of the applied load increases, the load distribution curve gradually moves to the right, indicating an increase in the load at each gauge level. The slope of a load distribution curve at any gauge level reflects the shaft friction along the pile. Theoretically, between two adjacent gauge levels, the change of load divided by the shaft circumferential area (between the two gauge levels) provides the shaft resistance along the pile within that zone.

Shaft resistance is estimated using the calculated load distribution diagram of each pile test, based on the assumption that the pile diameter and the cross-sectional area along the pile length are constant. The skin friction resistance at varying depths is summarized in Table 2. The estimate allows the shaft resistance to be safely increased by 50% from the original values recommended without pile load tests.

Table 2. Summary of the skin friction resistance

Depth Below Existing Ground (m)	Allowable Skin Friction Resistance (kPa)
Zone of New Fill	0
0.0 to 1.5	0
1.5 to 10.0	21
10.0 to 15.0	52
Below 15.0	67

The structural failure of TP1 near the end of the test complicated the interpretation of the pile load test result, as the sister bars at select depth levels might not have been measuring strains that reflected the true load in the pile at that level.

End bearing was not considered in the design due to the uncertainty associated with the quality of cleaning the pile base, which would significantly impact the tip resistance.

7 DISCUSSION

The assumption that a constant pile diameter and cross-sectional area are used to estimate shaft resistance, presented in Section 6.3, is likely not true in reality. The as-built CFA pile shaft profile monitored by the rig computer should be considered approximate. A sensitivity analysis indicates that even a slight difference (± 5% range) in pile diameter makes a substantial difference in the calculation of the shaft resistance along the pile length.

Back analysis of loads from strain gauge readings are challenging when testing concrete piles because of the difficulty of knowing the concrete modulus and the fact that the concrete modulus varies with compressive strength and strain. The analysis of the data is further complicated because the shaft diameter is not uniform throughout the pile length.

The fact that one test pile failed structurally creates some uncertainty regarding the strain gauge readings. If TP1 had not failed structurally, it may have been possible to achieve higher capacities in the pile and possibly adopt even a higher shaft friction value. Regardless, the pile load tests permitted a significant increase in the design shaft friction parameters, which is considered to provide substantial cost savings for the project foundation installation.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

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