Field load testing and integrity tests of CFA piles in sandy deposits

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ABSTRACT
CFA piles were adopted to support a multi-storey residential building with one level of basement. The soils below the basement floor consisted of a 6 m thick layer of weak silty clay overlying dense to very dense sandy deposits. The test pile of 0.51 m in diameter and 12.7 m in length was loaded to 3800 kN. The bearing capacity of the test pile is significantly higher than the theoretical estimation. During the installation of the CFA piles, it was observed that the amount of grout used was typically 1.4 to 1.6 times the theoretical nominal volume of the augered holes. The extra grout is considered to have infiltrated into the sandy deposits, making the effective pile diameter larger than the nominal. Post-installation pile integrity tests (PIT) of production piles also indicated enlargement of the pile shaft diameter.

RÉSUMÉ
Les piles de CFA ont été adoptées pour soutenir un bâtiment résidentiel de plusieurs étages avec un niveau de sous-sol. Les sols au-dessous du sous-sol consistaient en une couche d'argile limoneuse de 6 m d'épaisseur recouvrant des dépôts sableux denses à très denses. La pile d'essai de 0,51 m de diamètre et de 12,7 m de longueur a été chargée à 3800 kN. La capacité portante de la pile d'essai est significativement plus élevée que l'estimation théorique. Lors de l'installation des piles CFA, il a été observé que la quantité de coulis utilisée était typiquement de 1,4 à 1,6 fois le volume nominal théorique des trous taraudés. On considère que le coulis supplémentaire s'est infiltré dans les dépôts sableux, rendant le diamètre effectif de la pile supérieur au diamètre nominal. Les tests d'intégrité du pieu (PIT) post-installation des pieux de production ont également indiqué une augmentation du diamètre de l'arbre du pieu.

1 INTRODUCTION

Continuous flight auger (CFA) piles, i.e. auger-cast piles, were adopted to support a multi-storey residential building with one level of basement in Markham, Ontario. The piles were designed for bearing capacity values of 1400 kN/pile at serviceability limit state (SLS) and 1900 kN/pile at factored ultimate limit state (ULS). Field static load test was carried out to confirm the design capacity of the piles. Pile integrity tests (PIT) were conducted for selected production piles across the site as a part of quality assurance of the pile foundations.

1.1 CFA Piles

CFA piles are cast-in-place piles. During construction, the pile is drilled to the target depth using a continuous flight auger, while the flights of the auger are filled with soils. Then, the auger is withdrawn from the hole, and at the same time cement grout is placed by pumping through the hollow centre of the auger pipe to the base of the auger. The most important consideration in CFA pile construction is matching the rate of concrete pumping with the rate of withdrawal of the augers such that 'necking' of the pile does not occur. Steel reinforcement is then placed into the hole filled with fluid concrete shortly after the withdrawal of the auger. The diameter of CFA piles generally ranges from 0.3 to 0.9 m and the length of CFA piles is typically up to 30 m.

Compared to conventional drilled shafts (drilled caissons), CFA piles do not require the use of casing or slurry to temporarily support the hole in unstable soils, such as cohesionless sandy soils below the groundwater table. A main disadvantage of CFA piles is that the available quality assurance (QA) methods to verify the structural integrity and pile capacity are less reliable or more costly than for drilled caissons and driven piles. In favourable circumstances, CFA piles have significant advantages such as construction speed and economy, provided that careful construction practices are followed. CFA piles have been used for support of various structures including buildings, bridges and retaining structures.

1.2 Bearing Capacity of CFA Piles

In the literature, various methods are available for estimating bearing capacities of CFA piles (FHWA, 2007) and drilled shafts (CGS, 2006).

The ultimate (total) capacity (R_u) of a pile consists of skin friction capacity (R_s) and toe capacity (R_b), expressed as:

$$R_u = R_s + R_b \quad [1]$$

For a pile installed in a number of soil layers, the total skin friction capacity is obtained by adding the contribution of all soil layers:

$$R_s = \sum(f_s \pi B L_i) \quad [2]$$

In the above equation, f_s is the unit skin friction in kPa, B is the pile diameter, and L_i is the thickness of soil layer.

The toe capacity of the pile can be estimated using the following equation:

$$R_b = q_p A_b \quad [3]$$

In the equation, q_p is the unit toe bearing capacity and A_b is the cross-sectional area of the pile at the base.

The skin friction capacity is fully mobilized with relatively small pile settlement, typically 5 to 10 mm. The settlement required to fully mobilize toe capacity can be assumed to be approximately 5% of the pile diameter (Reese and O'Neil, 1988; AASHTO, 2006).

1.2.1 Cohesive Soils

According to FHWA (2007), the unit skin friction (f_s) and unit toe capacity (q_p) of CFA piles in cohesive (i.e. clay) soils can be estimated using:

$$f_s = \alpha C_u \quad [4]$$

$$q_p = N_{c^*} C_u \quad [5]$$

In the above equations, C_u is the undrained shear strength of cohesive soil, α is a reduction factor for unit skin friction, and N_{c^*} is the bearing capacity factor.

The N_{c^*} values range from approximately 6.5 to 9, increasing with soil C_u values varying from 25 kPa to 200 kPa (O'Neil and Reese, 1999).

According to Reese and O'Neil (1988) and O'Neil and Reese (1999), the reduction factor (α) is

$$\alpha = 0.55 \qquad \text{for } C_u \leq 150 \text{ kPa} \quad [6]$$

For C_u values varying from 150 kPa to 250 kPa, the α value varies linearly from 0.55 to 0.45, decreasing with increasing C_u value.

Other methods for estimating α values based on C_u values are also available in the literature (Coleman and Arcement, 2002).

1.2.2 Cohesionless Soils

The unit toe capacity (q_p) of CFA piles in cohesionless (i.e. sandy) soils can be estimated using (FHWA, 2007):

$$q_p \text{ (kPa)} = 57.3 N_{60} \qquad \text{for } N_{60} \leq 75 \quad [7]$$

$$q_p = 4300 \text{ kPa} \qquad \text{for } N_{60} > 75 \quad [8]$$

In the equations, N_{60} is the SPT-N value at 60% hammer efficiency.

The unit skin friction of CFA piles in cohesionless soils can be obtained using:

$$f_s = \beta \sigma_{v'} \quad [9]$$

In the equation, $\sigma_{v'}$ is the effective vertical stress in soil. The β value is related to the soil strength and in situ lateral stress conditions and is limited to $0.25 \leq \beta \leq 1.2$. The β value can be calculated using:

$$\beta = 1.5 - 0.135 Z^{0.5} \qquad \text{for } N \geq 15 \quad [10]$$

$$\beta = (1.5 - 0.135 Z^{0.5})\, N/15 \qquad \text{for } N < 15 \quad [11]$$

In the equations, N represents uncorrected SPT-N values, and Z is the depth to the middle of the soil layer.

1.3 This Study

A total of 204 CFA piles were installed to support a multi-storey building with one level of basement in Markham, Ontario. Field static load testing of a test pile was carried out to a maximum load of 3800 kN. Pile integrity tests (PIT) were conducted for selected production piles for quality assurance.

This paper presents the field testing results and relevant analyses and discussion.

2 SUBSURFACE CONDITIONS

In the proposed building area, the excavation base to the basement level was approximately 3 m below ground surface. Boreholes drilled at the site revealed that the soils below the basement level consisted of a 6 m thick layer of firm silty clay deposits overlying dense to very dense sandy deposits.

The groundwater table was at the excavation base level, i.e. at approximately 3 m below the existing grade.

2.1 Silty Clay Deposits

Field vane shear tests were conducted in the weak silty clay deposits. The measured undrained shear strength (C_u) of the silty clay deposit ranged from 24 to 52 kPa, with an average C_u value of 37 kPa, indicating a generally firm consistency. The SPT-N values measured in the weak silty clay deposit typically ranged from 3 to 6 blows per 300 mm of penetration. The water contents measured in samples of this deposit ranged from 17% to 26%.

2.2 Sandy Deposits

Underneath the weak silty clay deposit, cohesionless (sandy) soils were encountered, extending to depths of 17 to 20 m. The cohesionless deposits generally consisted of sandy silt to silty sand, with some layers of sand and gravel. The cohesionless deposits were dense to very dense. Grain size analyses of the cohesionless samples indicate that the deposits contain 2 to 12% gravel, 30 to 55% sand, 15 to 25 % silt and 10 to 12% clay particles.

2.3 Shear Wave Velocity

Field shear wave velocity measurement was carried out at the site.

The investigation included both the multi-channel analysis of surface waves (MASW) and the micro-tremor array measurements (MAM) methods to generate a shear-wave velocity profile. The test results are presented in Figure 1. Based on the shear wave velocity testing results, the subject site for the proposed building can be classified as "Class C" for seismic site response.

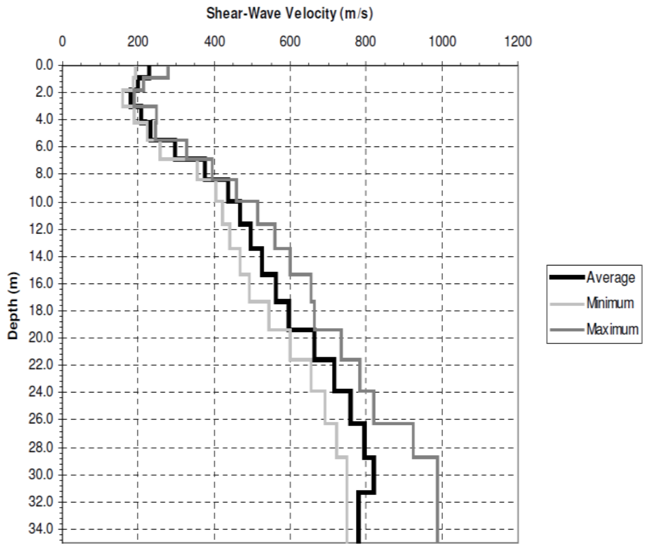

Figure 1. Shear wave velocity testing results

3 FIELD STATIC LOAD TESTING

In order to confirm the availability of the design capacity of the piles, field static load test was conducted on a pre-production (sacrificial) test pile. Details of the test pile installation, test procedures, test results and discussions are presented as follows.

3.1 Pile Installation

The test pile of 0.51 m in diameter and 12.7 m in length was installed from the basement level. The auger was advanced to the target depth of 12.7 m. An initial grout head of 1.5 m was created by pumping approximately 0.3 m^3 of grout. During the withdrawal of the auger and pumping of grout, positive (clockwise) rotation was maintained at all times during the placement of grout. It was important to coordinate the rate of auger withdrawal and grout injection to maintain minimum 1.5 m grout head at all times.

The grout head observed at the surface was 3.4 m. The total grout injected was approximately 140% of the nominal volume of the auger hole of the test pile.

The grout consisted of 1 part of Portland cement, 2 parts of sand, and 2 parts of water. The strength of the grout at 7 days was measured at 35 MPa.

3.2 Field Load Test

The maximum load applied to the pile was 3800 kN. The pile was loaded and unloaded in a number of load increments of 475 kN, generally in accordance with the Quick Test Procedure, ASTM D1143/1143M-07, Standard Load Test Methods for Deep Foundations under Static Axial Compressive Load. The load was applied to the test pile using a hydraulic jack, and the settlement of the pile was measured using two dial gauges. Load testing was carried out by jacking against a horizontal steel beam which was attached to 4 reaction piles. The hydraulic jack pressure readings (in psi) and the dial gauge readings were recorded with time during the load testing of the pile.

Each load increment was maintained for approximately 10 minutes, except at the peak load of 3800 kN which was maintained for 18 hours. The settlement of the pile at each load increment was measured using 2 dial gauges, attached to independent reference beams.

3.3 Load and Settlement

The applied load and the measured settlement at the top of the test pile were obtained. The relationships between load and settlement are shown in Figure 2.

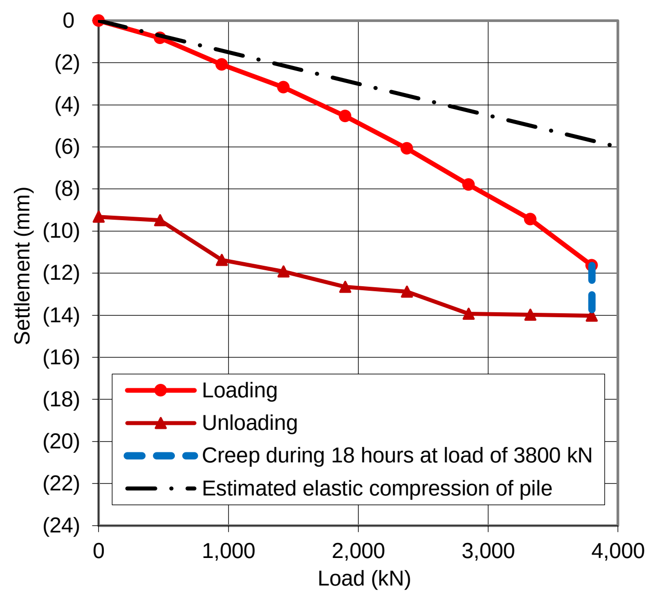

Figure 2. Load versus settlement of test pile

During loading, the measured settlement of the test pile was approximately 4.5 mm at 1900 kN (100% of ULS load) and was approximately 11.6 mm at 3800 kN (200% of ULS load). The pile head settlement at the SLS load (1400 kN) was 3.2 mm.

During the constant loading at 3800 kN, the creep settlement of the pile was approximately 3.2 mm over a period of 18 hours (Figure 3).

The settlement of the test pile at design load is small and meets the design requirement. The test results indicate that the test pile is capable of supporting the design load of 1400 kN at SLS and 1900 kN at ULS.

3.4 Creep Rate

During the load test, the maximum applied load of 3800 kN was maintained for about 18 hours while the settlement at the top of the pile was measured. The increase of the measured settlement with time is shown on Figure 3. The creep settlement of the pile was approximately 3.2 mm over a period of 18 hours. The creep settlement was approximately 1.7 mm within 1 hour. The increment of creep was approximately 0.9 mm from 1 hour to 6 hours, and was 0.6 mm from 6 hours to 18 hours. The creep rate decreased with time.

As shown on Figure 3, more than 80% of the creep settlement occurred within the initial 6 hours after the application of the load of 3800 kN.

The creep settlement plotted against the logarithm of time is presented in Figure 4. At the load of 3800 kN, the creep rate is approximately 1.1 mm per log-cycle time. At the design SLS load of 1400 kN, the creep rate is expected to be much lower than 1.1 mm per log-cycle time.

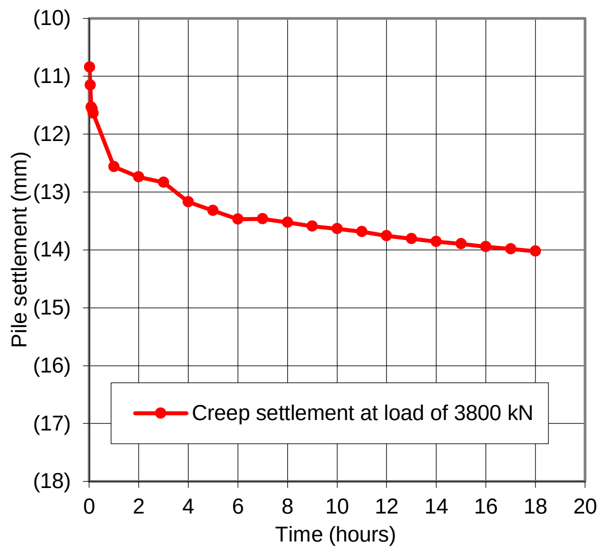

Figure 3. Settlement with time of test pile

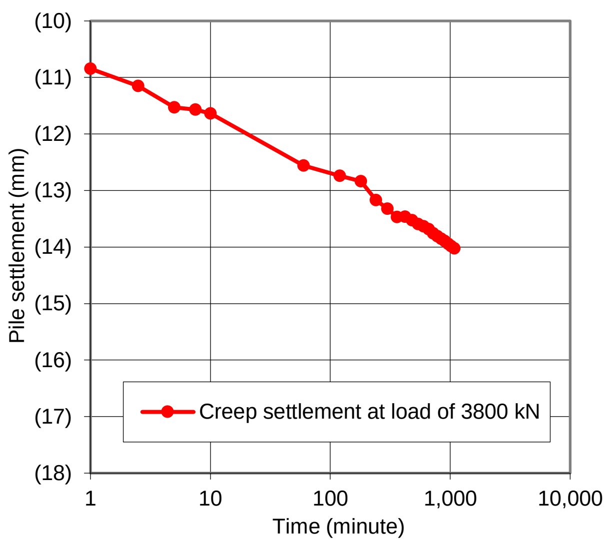

Figure 4. Settlement with time of test pile

3.5 Measured Bearing Capacity

During loading, the measured pile head settlement was 11.6 mm at 3800 kN at which the pile had still not failed in terms of bearing capacity. The test results indicate that the available bearing capacity values of the CFA piles installed at the site are much higher than the theoretical estimation.

During the installation of the production piles, it was observed that the amount of grout used was typically in the range of 1.4 to 1.6 times the theoretical nominal volume of the augered holes for the production piles. It is assumed that the extra grout (beyond the volume of the augered holes) have infiltrated into the sandy deposits at the lower portions of piles, making the effective pile size (diameter) and roughness larger than the nominal, and resulting in significant increase of the bearing capacity of the piles. This assumption is supported by the PIT testing results that indicate enlargement of pile size (diameter) within the lower portions of the piles.

4 PIT TESTING

After the field static load testing of a test pile that confirmed the availability of the design bearing capacity of 1900 kN at ULS, a total of 204 CFA piles were installed at the site under full-time supervision by SPL Consultants Limited (now WSP Canada Inc.). In order to confirm the acceptability of the installed piles, post-installation pile integrity tests (PIT) were conducted on selected production piles as a part of quality assurance of the pile foundations.

The PIT tests were conducted using the Pulse Echo method (PEM). This low strain test method uses a hand-held hammer to impact the top of a pile to generate compressive stress waves in the pile. The stress wave reflections from non-uniformities (defects) and from the pile toe are measured at the pile top using an accelerometer. The pile integrity test device records the pile top velocity with time. The recorded signals are processed and interpreted by the test engineer to evaluate the integrity of the pile. In the past decades, PIT testing has been widely used to assess the integrity of concrete piles (Rausche et al. 1988; Steinbach, 1975).

PIT testing is a quick and inexpensive method for assessing pile integrity. However, there are a number of factors affecting reliable judgement, such as experience of the test engineer, knowledge of the test pile, including concrete strength, length and shape of the pile and soil conditions along the pile shaft. There are also limitations on sizes of defects/anomalies that can be detected. As a rule of thumb, interpretation of the test results will be more and more difficult for a pile with a length of more than 30 times its diameter.

PIT testing was conducted on 38 selected production piles across the site. The test piles were approximately 12.5 to 13 m in length. Typical PIT testing results from a test pile are shown in Figure 5. Based on the test results, no defects (i.e. reduction of pile cross section, or necking) were detected in the test piles. The test piles were considered acceptable in terms of pile integrity.

In most test piles, the results indicate enlargement of pile size (diameter) within the lower portions of the piles, as impedance increase at the lower portions of the piles was observed from the PIT test results. Based on the borehole information, the upper portions of the piles were installed in the relatively weak clay soils, and the lower portions of the piles were installed in the dense to very dense cohesionless soils of sand, sandy silt to sand and gravel. The expanded sizes of the piles were considered due to infiltration of grout into the sandy soils around the pile shaft.

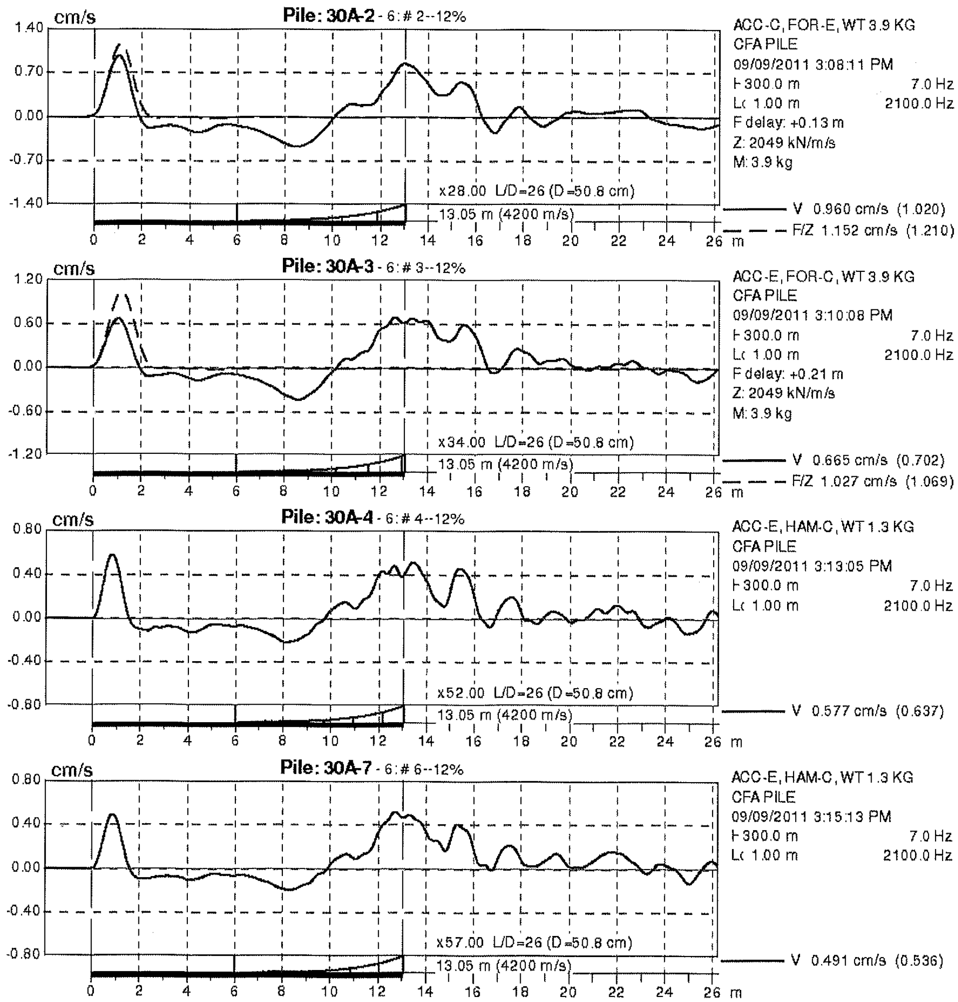

Figure 5. Typical PIT testing results

5 DISCUSSION

Based on the borehole information and according to the methods presented in Section 1.2, the estimated theoretical ultimate bearing capacity of a single CFA pile of 0.51 m in diameter and 12.7 m in depth is about 2100 kN.

The test pile was loaded to 3800 kN at which the settlement of the pile was approximately 12 mm. As the pile had still not failed at the load of 3800 kN, the ultimate capacity of the test pile is greater than 3800 kN, which is much greater than the theoretically estimated ultimate capacity of 2100 kN.

It is believed that the high capacity of the piles is mainly due to the expanded effective sizes of the piles. During the installation of the production piles, it was observed that the amount of grout used was typically in the range of 1.4 to 1.6 times the theoretical nominal volume of the augered holes. The extra grout would have infiltrated into the sandy deposits at the lower portions of piles, making the effective pile diameter and roughness larger than the nominal, resulting in significant increase in the bearing capacity of the piles. Post-installation pile integrity tests (PIT) of production piles also indicated enlargements of pile diameter within the lower portions of the tested piles. In addition, the effective depth of the piles would also be deeper than the bottom of the augered holes.

The actual expanded sizes of the piles were unknown. It was anticipated that the extra grout would have mainly infiltrated in the sandy deposits at the lower portion (6 m) of the pile.

In order to roughly estimate the expanded sizes of the piles, it is assumed that the grout infiltrated horizontally and downward for an equal distance. Within this distance, the water in the void of soil is completely displaced by the grout. Assuming a void ratio of 0.5 for the dense to very dense sandy soils, the estimated distance of grout infiltration beyond the pile shaft and below the pile bottom is approximately 0.21 m. That is, the effective pile depth is increased by 0.21 m, and the effective pile diameter at the lower portion (6 m) is increased from 0.51 m to 0.93 m.

For the expanded pile with an effective diameter of 0.93 m for the lower portion of the pile shaft, the estimated ultimate bearing capacity of the pile using the methods presented in Section 1.2 is approximately 4900 kN. This bearing capacity value is probably near the actual ultimate capacity of the pile, as the test pile had still not failed at the maximum applied load of 3800 kN.

This method for estimating the expanded effective sizes of CFA piles is very conceptual and does not take into account the change in pile shaft roughness. As such, it should not be used directly in practice without further supporting test data and engineering judgement.

6 SUMMARY AND CONCLUSIONS

This paper presents the results of the field static load test and PIT testing of CFA piles at a site in Markham, Ontario.

During loading, the measured settlement of the test pile was approximately 4.5 mm at 1900 kN (100% of design ULS load) and was approximately 11.6 mm at 3800 kN (200% of design ULS load). The pile head settlement at the design SLS load (1400 kN) was 3.2 mm.

At the maximum load of 3800 kN, the creep settlement of the pile was approximately 3.2 mm over a period of 18 hours. The creep rate of the test pile was approximately 1.1 mm per log-cycle time.

The production piles with similar sizes were installed using the same procedures as the test pile. PIT testing was conducted on 38 production piles. The test results indicate enlargement of pile diameter within the lower portions of the piles in the sandy horizon.

During the installation of the production piles, it was observed that the amount of grout used was typically in the range of 1.4 to 1.6 times the theoretical nominal volume of the augered holes. It is assumed that the extra grout infiltrated into the sandy soils around the lower portion of the piles, making the effective sizes of the piles much larger. It is believed that the enlargement of the pile sizes and possible enhanced shaft wall roughness have resulted in high bearing capacity of the CFA piles.

Based on the field static load test and PIT test results and review of the installation records of all production piles, the CFA piles installed at the site are considered to be satisfactory and capable of supporting the design bearing values of 1400 kN per pile at SLS and 1900 kN per pile at ULS.

ACKNOWLEDGEMENTS

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