Case history: Continuous flight auger (cfa) piling at coastal region in the island of Madagascar

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ABSTRACT
Continuous Flight Auger (CFA) pile foundations were used to support a major new nickel mine process plant located at the east coast of the island of Madagascar in the Indian Ocean. The site is situated on a relatively level coastal strip with near surface soils in the area generally comprised of alluvial and dune deposited fine to medium grained sands. The site experiences frequent cyclones and high winds resulting in significant design lateral and uplift loads. Over six thousand piles, 16 m long, were constructed in 21 months from September 2007 to May 2009. Static pile load tests were conducted prior to and during pile construction to ensure that the required pile capacities were achieved. This paper presents the outline of the pile construction, quality control and quality assurance (QA/QC) measures employed during installation and outcomes of the pile load testing program.

RÉSUMÉ
Des fondations sur pieux forés à la tarière continue ont été utilisées pour supporter la nouvelle usine principale de nickel située sur la cote est de l'ile de Madagascar dans l'Océan Indien. Le site est situé sur la cote relativement plat avec des sols de surface composés généralement de dépôts alluviaux et des dunes de sables fins à moyens. Le site reçoit fréquemment des cyclones et de forts vents créant de forces latérales et de d'arrachement très élevées. Plus de 6,000 pieux de 16 m de long ont été construits en 21 mois, de Septembre 2007 à Mai 2009. Des essais de chargement statiques ont été conduits avant et pendant la construction pour assurer que les capacités des pieux ont été atteintes. Cet article présente la construction des pieux et les divers moyens de contrôle et d'assurance de qualité qui ont été employés pendant l'installation des pieux et les résultats du programme d'essais de chargement.

1 INTRODUCTION

The process plant site of Ambatovy Nickel Project is located at the east coast of the island of Madagascar. The plant site, approximately 200 hectare, accommodates numerous units and large equipment for mining (Nickel and Cobalt) process. Figure 1 presents a view of one of the major pieces of equipment. Major structures required deep foundation due to heavy loadings and small total and differential settlement tolerances. Relatively loose ground condition and high wind loads due to frequent cyclones also contributed to the deep foundation selection. Several deep foundation alternatives were considered during detail design stage. CFA (Continuous Flight Auger) / ACIP (Augered Cast-In-Place) piles were selected since they provided a cost and schedule effective foundation system, and allowed construction on multi fronts within the facility. Piles were 16 m long with diameters of 600 and 750 mm. A specialized international piling contractor with experienced operators was selected. Pile installation was monitored continuously.

An extensive static pile load testing program was planned and carried out prior to and during pile construction to ensure that required pile capacities were achieved.

In total well over six thousand piles were successfully constructed from September 2007 to May 2009 for various facilities within the plant site.

This paper first presents design background and outline of the pile loading test program. The design pile capacity is examined in the light of the loading test results followed by a discussion on pile construction and quality control/assurance (QA/QC) measures.

Figure 1. Autoclave Installed at the Process Plant

2 SITE CONDITIONS

The process plant site is located approximately 2 km from the Indian Ocean on the east coast of Madagascar. The site comprises a relatively level coastal strip consisting of a flat wetland area with low undulations and elevations ranging between 4 m and 8 m above mean sea level. The undulating topography consists of ancient dunes that run parallel to the coastline. The site was generally developed by removing top soil and vegetation, and levelling the site using a cut and fill approach.

Geotechnical site investigations prior to detail design stage determined that the near surface soils in the area generally comprise alluvial and dune deposited fine to medium grained sands. Soils at depth mainly consist of coastal deposited interbedded fine to medium grained sands and lagoonal silts. The bedrock surface generally consists of residual gneiss with weathered corestones in a residual gneiss matrix. The depth to the bedrock surface slopes from 30 m to 100 m from the westward to eastward ends of the process plant site. The major geological units at the location of the plant site are summarized in Table 1. Top two units, Unit 1 Upper Coastal Deposit and Unit 2 Upper Lagoonal Deposit, have their lower contact depths from 14 to 43 m, and considered to dominate foundation design. Those two units consist of fine to medium sand with little fines (Figure 2). The SPT 'N' value profile (Figure 3) shows that upper about 7 to 8 m, likely Upper Coastal Deposit, is in loose to compact condition. Then, the SPT 'N' value increases with depth up to an average value of 40 indicative of a denser condition at approximately 15 m. The SPT 'N' value becomes constant below 15 m, and slightly drops below 25 m.

The groundwater level is relatively shallow, within about 2 to 3 m below ground surface. Near surface sand horizons are uniformly graded and almost cohesionless, resulting in highly permeable conditions allowing for rapid drainage of surface water and horizontal migration of groundwater.

The site climate is considered to be warm and wet with an annual average temperature of 25°C and an annual average precipitation of 3330 mm. The site experiences an average of about four tropical cyclones each rainy season between November and April.

A basic wind speed considered in the design with 3 second gust, 50 years return period and 10 m above ground open terrain is estimated to be 274 km/h.

Based on a site specific seismic hazard assessment, the site is located within a relatively homogeneous region of low seismicity, and consistent with the IBC (2006) design requirements a Peak Ground Acceleration value of 0.08 g (at Site Class C/B interface) and an earthquake magnitude of 6.0 were used as design parameters. With the exception of shallow loose sand layers, the soils were not considered liquefiable for the given seismic design criteria. Site improvement works such as ground water control, regrading and compaction within upper few meters reduced any chances of liquefaction in the shallow loose sand layers.

Table 1. Generalized Subsurface Condition

Unit	Geological Unit	Description	Lower Contact Depth (m)
1	Upper Coastal Deposit	Sand, trace clay, loose to dense	10.0 to 28.0
2	Upper Lagoonal Deposit	Sand, trace to some clay, compact to dense	14.0 to 43.0
3	Middle Lagoonal Deposit	Clay, trace to some sand, very stiff to hard	35.5 to > 70.95
4	Middle Coastal Deposit	Sand, compact to very dense	17.5 > 70.95
5	Lower Lagoonal Deposit	Sandy Clay, organic rich, very stiff to hard	52 to 70.95
6	Alluvium	Clayey Sand, some gravel, cobbles and boulders, compact to dense	59.5 to 69.5
7	Lower Coastal Deposit	Clayey Sand, hard, slightly cemented	64.2 to >70.95
8	Weathered Bedrock Surface	Weathered corestones with residual gneiss matrix	> 70

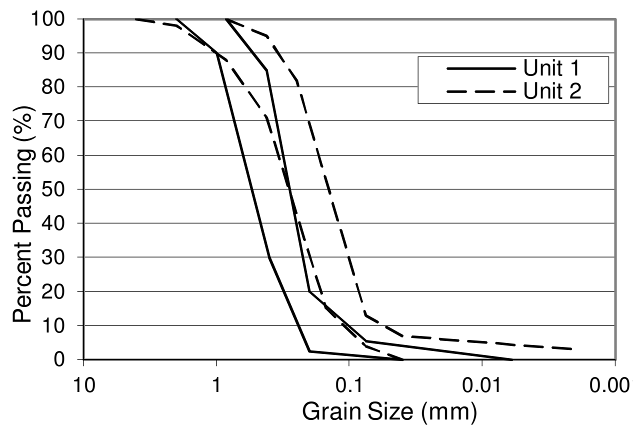

Figure 2. Range of Grain Size Distribution of Major Geological Units

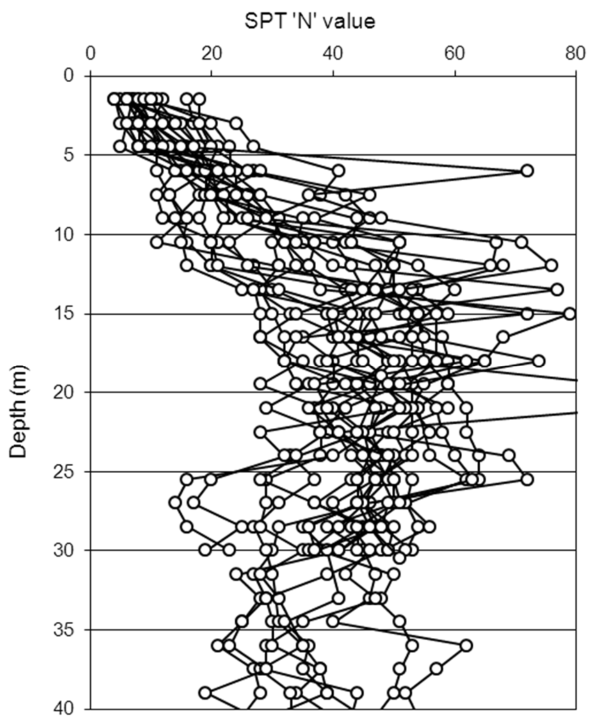

Figure 3. SPT 'N' value Profile

3 FOUNDATION DESIGN

The pile capacities, both in compression and uplift, were estimated using the method proposed by Zelada and Stephenson (2000). The ultimate pile capacity in compression, Q_u, was obtained as a sum of end bearing, Q_b, and shaft friction, Q_s, capacities using equations below:

$$Q_u = Q_s + Q_b \quad [1]$$

$$Q_b = q_b A_b \quad [2]$$

$$Q_s = \Sigma f_{sz} A_{sz} \quad [3]$$

where A_b, pile toe area; f_{sz}, unit shaft friction at any depth z; A_{sz}, pile circumferential area at depth z; q_b, unit bearing capacity of the pile toe estimated by Eq. [4]:

q_b = 0.16 N MPa ≤ 7.2MPa (for tip movement of 0.1D) [4]
D = Pile diameter; N = SPT 'N' value.

To account for possible decompression (rebound) and loosening of soil at pile tip, q_b was limited to 4 MPa which is consistent with several other methods in practice reported by Zelada and Stephenson (2000).

f_{sz} was estimated by Eq. [5]:

$$f_{sz} = \beta p'_0 \quad [5]$$

where p'_0, vertical effective stress at depth z; $\beta = 1.2 - 0.108 z^{0.5}$ $(0.2 \leq \beta \leq 0.96)$.

Typical piles were designed to be of 16 m embedded length with diameters of 600 and 750 mm. The calculation of the pile capacities for a pile with the diameter of 750 mm is described below.

Bearing capacity of pile toe, q_b was estimated using a SPT 'N' value of 33 at pile tip (z = 16 m) referring to SPT 'N' value profile in Figure 3:

q_b = 0.16 x 33 = 5.28 MPa > 4.0 MPa, and Q_b = 4.0 x 1000 x 0.442 = 1770 kN [6]

The shaft friction averaged along 16 m pile, $f_{s\ (avg)}$ was estimated to be 51 kPa and Q_s = 1920 kN, assuming ground water level to be at 1 m below ground surface. Therefore, ultimate pile capacity, Q_u, and allowable capacity, Q_a, become:

$$Q_u = 1770 + 1920 = 3690 \text{ kN} \quad [7]$$

$$Q_a = 3690 / 3 = 1230 \text{ kN} \quad [8]$$

where Q_a was obtained by Q_u divided by factor of safety, FS = 3.

The allowable uplift capacity, Q_t, was taken as the combination of 80% of the calculated skin friction in compression and effective self weight of pile, W_p:

$$Q_t = (0.8 \times Q_s) / 3 + W_p = 510 + 100 = 610 \text{ kN} \quad [9]$$

Typical design of pile is shown in Figure 4. The pile included 8 #M25 reinforcing steel bars for the top 6 m. The piles subjected to tension force were constructed with one D32 Dywidag bar.

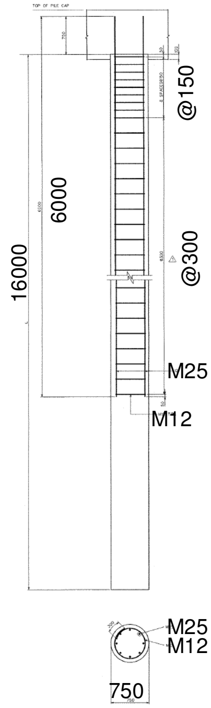

Figure 4. Typical Pile Design (not to scale)

4 PILE LOAD PROOF TESTS

A pre-production pile (test pile) conventional head-down load test program was planned and carried out in order to verify geotechnical capacities of CFA pile with various dimensions, and to establish the construction and monitoring procedures to be implemented during the construction of the production piles. Eight test piles with same dimension as the production pile were installed at various locations of the site prior to and during construction. Five compression tests and three uplift tests were carried out on the test piles, as listed in Table 2. Additionally, five compression tests were performed on five production piles (Table 3).

Figure 5 shows the loading system used for the compression test. The load was applied by a hydraulic jack supported by the loading frame. Four anchor piles were installed for each test pile. Relevant measured items were the loading on top of pile, the settlement of pile head, the displacement of tell-tale (if available) and the movement of anchor piles. Tell-tales made of steel bar were installed at the pile tip of some test piles. The load applied on top of pile was calculated based on reading of a pressure gauge of the hydraulic jack, which was verified by calibrated load cell utilized for some of the compression tests. The pile head settlement was measured by four dial gauges equally spaced and supported by two independent steel beams. The system was modified for the uplift test so that the hydraulic jack pulls up test pile instead of compressing it.

Figure 5. Loading Frame

Table 2. Summary of Test Piles

No	Pile Test ID	Test Type	Tell-Tale
1	TP6	Compression	
2	TP8	Compression	x
3	CCD TP1	Compression	x
4	CCD TP3	Compression	
5	Ore Leach TP1	Compression	x
6	TP10	Uplift	
7	CCD TP2	Uplift	
8	Ore Leach TP4	Uplift	

Table 3. Summary of Loading Test on Production Piles

No	Pile Test ID	Diameter (mm)	Test Type
1	WPT1	600	Compression
2	WPT2	750	Compression
3	WPT3	750	Compression
4	WPT4	750	Compression
5	WPT5	600	Compression

The load increments varied from 200 kN to 400 kN. The maximum load was achieved after 11 to 14 steps depending on the maximum load. Each loading was held constant for 5 minutes. At the maximum test load and approximately 40 and 60 % of the estimated ultimate pile capacity, the load held constant for one hour. After each of these loads the piles were unloaded and loaded again (two cycles of loadings before the maximum load).

Outcomes of the tests are presented and discussed next.

4.1 Test Pile Compression Test

The pile head settlement and loading curve from the compression test on TP8 is shown in Figure 6 as a typical compression test result. TP8 was equipped with a tell-tale at the tip of the pile, thus displacement of the tell-tale and loading curve was recorded as shown in Figure 6.

Both the pile head and the tell-tale loading curves show smooth hyperbolic shape. After applying Davisson failure criteria (Eq. [10] (Canadian Geotechnical Society (2006)), the pile capacity was estimated to be 3750 kN for a corresponding pile head settlement of 14.5 mm.

$$d = (QL/AE) + (4 + 8A_b)10^{-3}\ (m) \qquad [10]$$

where d, is the movement of the pile head at the offset limit load elastic shortening of the pile; Q, is the load applied on top of pile; A, is the cross sectional area of the pile (A is assumed to be equal to A_b in this paper); E, is Young's modulus of piling mix concrete (assumed to be 30 GPa) and L, is the length of pile.

The factor of safety obtained from this pile capacity becomes 3.0 (3750kN / 1230kN), which verified the design described in the previous section.

Pile head settlement and loading curves from the five test piles are shown in Figure 7. Loading curves in Figure 7 as well as those in subsequent figures are shown excluding unloading and reloading portion of curve for simplification. Despite the same dimensions of the piles and relatively uniform subsurface condition, the test results varied considerably. Pile capacities based on Davisson failure criteria are summarized in Table 4. The capacity ranges from 2100 to 3750 kN with pile head settlement from approximately 13 to 15 mm. The range of the pile capacity is equivalent to factors of safety from 2.3 to 3.0, except for TP6, (it was confirmed that a factor of safety greater than 2.0 was achievable on site and this production performance was accepted by design engineer).

To eliminate any future uncertainty from the piling operation, the presence of a qualified inspector on each rig was required to monitor installation of each production pile.

The skin friction and the respective axial force at failure point defined by Davisson criteria were estimated on three piles, TP8, TP1 at CCD area and TP1 at Ore Leach area, utilizing the displacement from the tell-tales. The results are summarized in Table 4. The skin friction was approximately 75 to 90 % of the design value (conversely 40 to 45% of pile capacity), while the pile tip capacity in compression exceeded 100% of the design value (up to 130 %). It is not clear if the skin friction was over-estimated by the design or it was not fully-mobilized at failure as defined by the Davisson criteria. Future studies regarding mobilization of skin friction and pile tip capacity are suggested to clarify this issue.

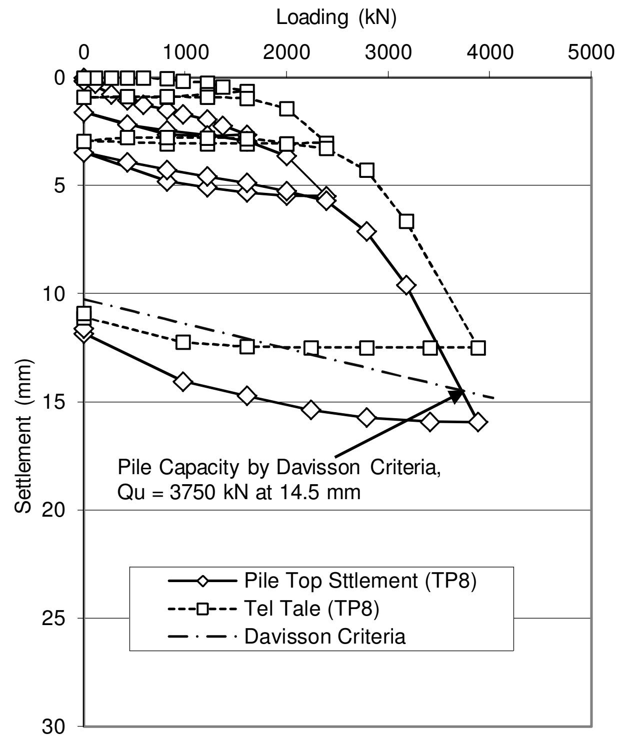

Figure 6. Typical Compression Test Result

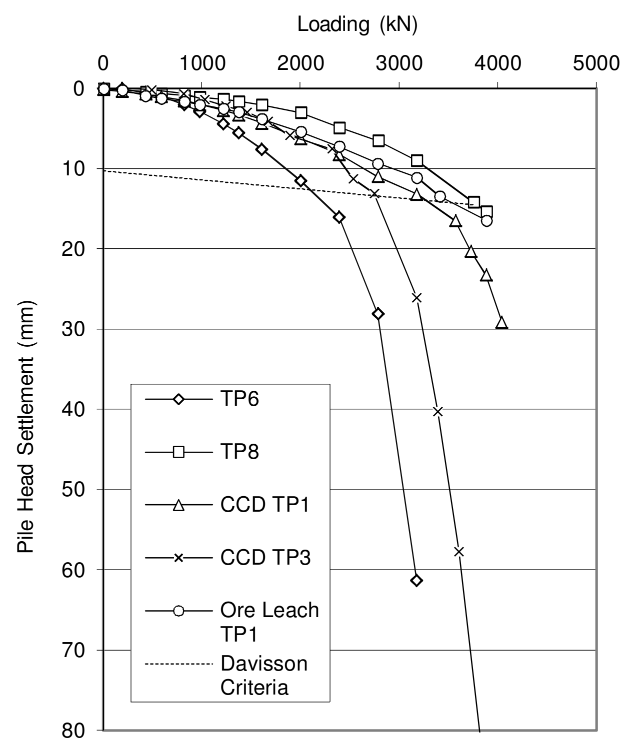

Figure 7. Summary of Compression Test

Table 4. Summary of Loading Test Results based on Davisson Failure Criteria

	Pile Test ID	Pile Capacity (kN)	Settlement at Failure (m)	FS [1]	Pile Tip (kN) [2]	Skin Friction (kN) [2]
Test Pile	TP6	2100	12.8	1.7		
	TP8	3750	14.5	3.0	2280	1470
	CCD TP1	3350	14.3	2.7	1870	1480
	CCD TP3	2780	13.4	2.3		
	Ore Leach TP1	3700	14.6	3.0	2000	1700

[1] Q_a = 1230 kN for D=750mm.
[2] Estimated from tell-tale at the tip of pile.

4.2 Uplift Test

Pile top displacement (upward) and uplift loading curves from three load tests are shown in Figure 8, along with three lines specifying FS = 1, 2 and 3 based on the design value. The results seem to vary as the compression tests. However, a FS = 2 was achieved at uplift displacement less than 10 mm in all tests. Thus it was confirmed that a satisfactory uplift capacity was achievable.

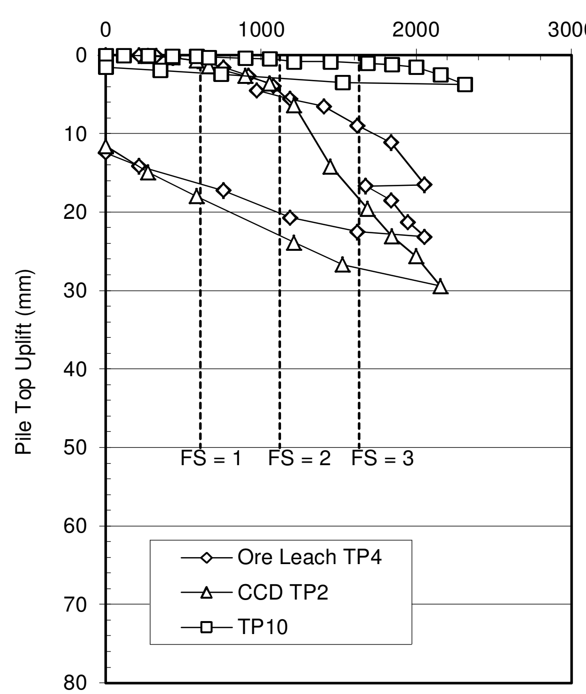

Figure 8. Summary of Uplift Tests

4.3 Load Test on Production Pile

Load tests were also conducted on five production piles throughout the period of pile production as summarized in Table 3. The maximum applied load was approximately 150% of the designed load, that is 2000kN for piles with D (diameter) = 750 mm and 1500 kN for piles with D = 600 mm. Figure 9 shows the pile head settlement versus applied load curves from three tests on the piles with D = 750 mm along with the test piles discussed earlier.

The curves falls within the range of those of the test piles, i.e. the production piles performed as expected based on the behaviours of the test piles, and their performances were considered to be satisfactory. This provided further confidence on the pile production.

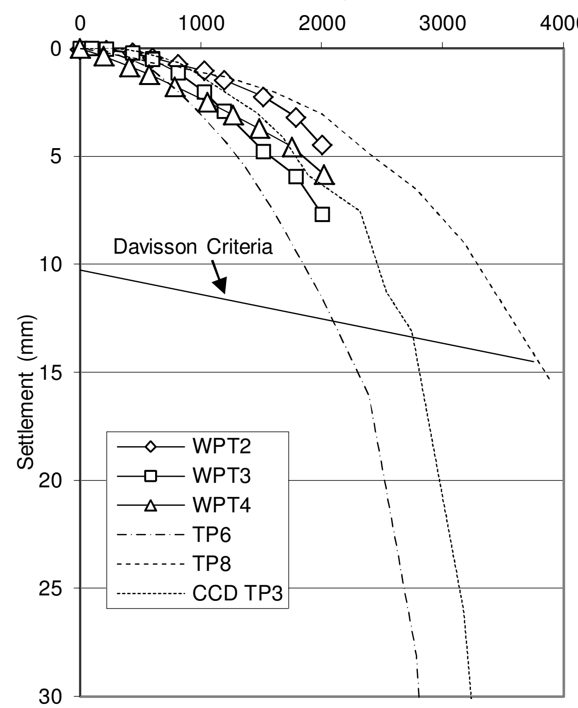

Figure 9. Summary of Compression Test on Production Piles

5 CONSTRUCTION

5.1 Outline

Pile construction commenced in September 2007, and completed in May 2009. Over six thousand piles were constructed at various areas of the plant site. One cyclone hit the plant site during the period of piling, which halted piling activity for a couple of weeks due to wet ground conditions. Four pile rigs worked simultaneously at peak time. All four rigs were imported into the island of Madagascar. The pile rigs were equipped with monitoring device and recorded various aspects of piling, e.g. concrete pressure, auger lifting rate, concrete flow rate, auger torque, auger rotation rate and drill rate. One of the CFA piling rigs utilized for this project is shown on Figure 10.

Figure 10. Typical CFA Piling Rig

Piling mix concrete of 30 MPa compression strength was prepared at batch plant located within the plant site and delivered to the rigs upon request. Most material such as cement, steel reinforcing bars and Dywidag bars were imported into the country except for aggregate and sand which were transported from a local quarry.

An average of 6.3 piles were constructed per day per rig during peak time. The average concrete over consumption was 31 %.

5.2 Quality Assurance (QA) Program

A site engineer representing SNC-Lavalin Inc., (the EPCM contractor for the project), provided quality assurance program through a specialized international geo-engineering consultant. The inspectors recorded various aspect of piling, e.g. dimension of steel cage, slump test, concrete delivery time, etc. Pile rig monitoring output and inspection data were reviewed by the site engineer on daily basis. In addition to pile installation monitoring, the consultant carried out a pile integrity test program to monitor the integrity of installed piles, while the contractor also conducted its own integrity test program. The test results were reported to the site engineer and reviewed on a regular basis. The production piles on which the loading test was carried out were chosen according to these information to verify their quality, as well as the validity of the quality assurance program.

Some piles were disregarded after/during construction due to various reasons, e.g. concrete supply was halted before pile installation completion due to pipe blockage; the reinforcing cage failed to be fully installed, etc. In such case, an alternative pile location was selected for the contractor and another pile was constructed immediately after the site condition allowed to do so. No significant quality issues were found on the completed production piles.

6 SUMMARY & CONCLUSIONS

1) Over six thousand piles were successfully constructed, utilizing the outcomes of pre-production loading tests and the daily implemented quality assurance program.
2) The implementation of static pile load testing program was successful, and brought useful additional confirmatory data for designers and constructors. Loading tests on production pile provided further confidence on the reliability of the pile installation program. Piles were evaluated, and as result some additional piles were added to the project confirming a safe design.
3) The skin friction observed by the loading tests was approximately 75 to 90 % of design value, while the pile tip capacity achieved 100 to 130 % of the design value. Future investigation on the mobilization of pile capacities is recommended, preferably utilizing fully instrumented piles, a variety of methods and the collected pile loading test results.

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