Behavior of Instrumented Continuous Flight Auger Piles in Sedimentary and Residual Soils

Comportamiento de pilotes hélice continua instrumentados en suelo sedimentario y residual

P.J.R. de Albuquerque, D. de Carvalho
The State University of Campinas – Brazil

C.T.D.B. Alledi
Federal Center for Technical Studies of Espirito Santo – Brazil

U.F.Polido
Geoconsult - Brazil

ABSTRACT

The behavior of instrumented continuous auger piles was studied through load tests. Piles were instrumented along the shaft with electric strain gauges and tested in two different types of soil. The first was a typical tropical soil from the South-Eastern region of Brazil (lateritic, collapsible and unsaturated) and the second was a typical sedimentary soil from the marine regions of Brazil (saturated sands and clays). In the first soil, slow maintained load (SML) and quick maintained load (QML) tests were performed in three piles with 0.4m diameter and 12m length. In the second soil, slow maintained load (SML) and quick maintained load (QML) tests were performed in two piles with 0.4m diameter, one with 8m length and another with 12m length. In all cases one electrical strain gauge was installed at the mid-length and two close to the tip of the piles. Field tests (SPT-T, DMT, CPT) were performed in parallel to the load tests in both areas. From the results obtained it was possible to determine side shear and tip resistances for all piles tested. Comparisons were done for load distribution along the piles. The increase in pile response due to a second loading cycle was also evaluated for each case tested. At the end, the authors present recommendations for shaft instrumentation of continuous auger piles.

RESUMEN

Se presenta un estudio del comportamiento de pilotes hélice continua instrumentados a través de las pruebas de carga. Se instrumentaron los pilotes a lo largo del fuste con sensores eléctricos y se ensayaron en dos tipos diferentes de suelo. El primero era un suelo tropical típico de la región sur oriental de Brasil (lateritico, colapsible y no saturado) y el segundo era un suelo típico sedimentario de las regiones marinas de Brasil (arenas y arcillas saturadas). En el primer tipo de suelo, se ejecutó prueba de carga lenta (SML) y la prueba rápida (QML) en tres pilotes con el diámetro de 0.4m y 12m longitud. En el segundo tipo de suelo, se ejecutó la prueba de carga lenta (SML) y la prueba rápida (QML) en dos pilotes con diámetro de 0.4m, uno con 8m y otro con 12 m de longitud. En todos los casos se instalaron sensores eléctricos a media longitud y dos cerca de la punta de los pilotes. Pruebas del campo tipo SPT-T, DMT, CPT se realizaron en ambas áreas. De los resultados obtenidos fue posible determinar la resistencia por fuste y de la punta de todos los pilotes ensayados. Se presentan comparaciones de las diferentes distribuciones de carga a lo largo de los pilotes. El aumento en la respuesta del pilote debido a un segundo ciclo de carga, también se presenta para cada elemento ensayado. Finalmente los autores presentan las recomendaciones para la instrumentación del fuste de los pilotes tipo hélice continua.

Keywords: continuous flight auger piles, instrumentation, sedimentary soil, residual soil

Palabras-Clave: pilote hélice continua, instrumentacion, suelo sedimentario, suelo residual

1 INTRODUCTION

With continuous growth of the urban centers, availability of new sites for construction has become limited. The option is then the vertical growth, which requires development of a better technology for foundations and their execution. The use of driven piles has become problematic due to noise and vibration they cause to existing nearby constructions. In many cases, the continuous flight auger piles (CFA) has become more viable because they reach project requirements and need a reduced execution time. However, there is a lack of knowledge of soil parameters for both unsaturated residual soils and also quaternary sedimentary soils. As a consequence, economical foundation designs with CFA piles are hard to accomplish. The present paper brings a contribution to this lack of knowledge, presenting parameters obtained from load tests performed on CFA piles.

The focus of the work is on the compressive behavior of monitored CFAs that were tested in two different experimental sites with two different sets of geotechnical characteristics. The first site was located in Vitoria, at the coastal region of Espirito Santo State. The region suffered continuous sedimentation during the quaternary period, where layers of sandy alternate with layers of clay soils of low resistance. The second site was in Campinas, Sao Paulo, a region typically with diabase residual soils. Both sites were in Brazil.

In both sites, the piles were instrumented with electrical strain-gauges at the shaft and close to the tip. The objective of the work was to investigate the load transference from the piles to the adjacent soil along the depth.

2 EXPERIMENTAL SITES

The geotechnical characteristics of each experimental site are presented below.

2.1 *Vitoria/ES – Experimental Site*

The Vitoria Experimental Site is located at the continental coastal region of the city. The geotechnical profiles have basically sedimentary soils from the Quaternary Period. The Brazilian sedimentary soils from the coast were formed mainly due to episodes of sea level variations, which occurred during that period. Soft clays present there have typical characteristics of SFL, according to what was defined by Massad (1999).

At the Vitoria site, an extensive geotechnical investigation was performed, involving field (SPT, CPT e DMT) and laboratory tests. Figure 1 present the average characteristics of the soils.

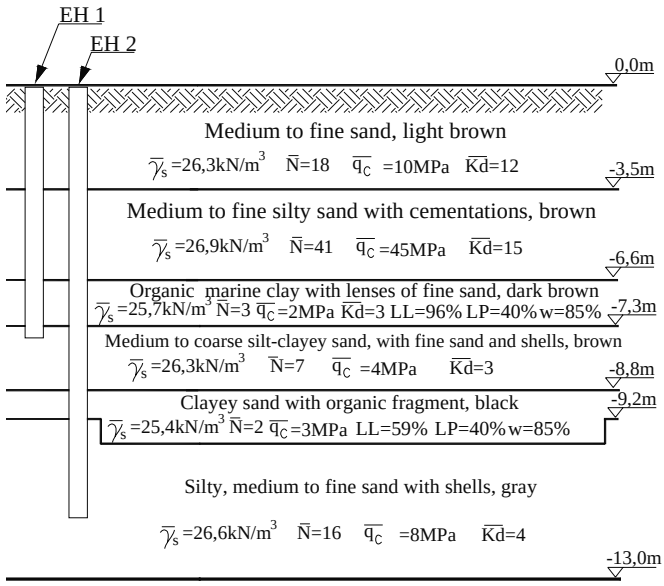

Figure 1 – Average Geotechnical Characteristics for the Vitoria-ES Experimental Site.

2.2 *Campinas/SP – Experimental Site*

The Campinas experimental site is located at the Unicamp campus (The State University of Campinas). The site has suffered extensive investigation programs, with several field (SPT, CPT, PMT, DMT etc) and laboratory tests.

The subsoil profile is constituted of diabase residual soils, mainly. It presents a 6m thick surface layer of silty-sandy porous collapsible clay, followed by clayey-sandy silt until 19m depth. The water table is below 17m. Figure 2 present the average characteristics of the soils at the Campinas experimental site.

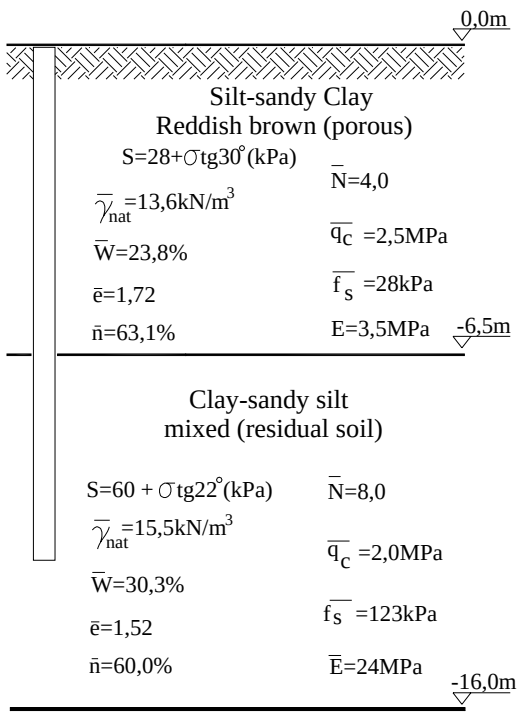

Figure 2 – Average Geotechnical Properties for the Campinas-SP Experimental Site.

3 PILE CHARACTERISTICS

3.1 *Vitoria/ES Experimental Site*

For this site, the piles had 0,4m diameter by 8m and 12m length. The longitudinal steel reinforcement was constituted of 4 bars of ϕ16mm diameter and 4m long and stirrups of 6,4mm diameter and spaced at 20cm.

Local experience in the Vitoria region dictated that, in sandy soil, fast loss in concrete fluidity occurs. This makes made steel reinforcement introduction difficult. To avoid this problem, a pumped concrete was dosed with high cement consumption, according to the adopted procedure described Almeida Neto (2002). Further details about the pile construction can be found in Alledi (2004) e Alledi et al (2006).

3.2 *Campinas/SP – Experimental Site*

Three test CFA piles were executed with 0.40m diameter and 12m length. Inside the piles a galvanized steel tube and the longitudinal steel reinforcement were introduced. The reinforcement steel was 4 bars of 16mm diameter and 6m length, and stirrups of 6,4mm diameter spaced at 20cm. Further constructive details can be found in Albuquerque (2001).

3.3 *Instrumentation Installation*

The instrumentation was installed through a galvanized tube (42.3mm diameter and 3mm wall thickness) running longitudinally at the center of each pile. The tube was placed though the auger axis before pumping in the concrete (Albuquerque, 2001).

4 REACTION SYSTEM

In both experimental sites the reaction system was constituted of reaction piles, Dywidag bar anchors type ST 85/105 with 32mm diameter, longitudinal steel reinforcements and steel reaction beams. In Vitoria (Alledi, 2004) the steel beams were cross shaped and in Campinas (Albuquerque, 2001), they had two supports.

5 INSTRUMENTATION

The instrumentation systems that were used in both sites are presented below.

5.1 *Instrumentation preparation*

CA-50 bars (12.5mm diameter and 0.60m length) were instrumented with electric strain gauges. The bars were attached together and placed through the galvanized tube that was placed inside each pile. The instrumented bars were then consolidated to the tubes by injecting cement paste.

Table 1 presents the positions of the instrumentation installed in the piles.

Table 1 – Instrumentation Position along Pile Shafts.

Campinas (3 piles)	**Vitória (2 piles)**	
ϕ=0.4m e L=12m	ϕ=0.4m e L=8m	ϕ=0.4m e L=12m
Reference section		
5,0m	3.7m	5.85m
11.1m	6.8m	10.95m
11.7m	7.4m	11.55m

6 LOAD TESTS

A total of ten load tests were performed: six in Campinas and four in Vitoria. The tests were performed by applying a first load cycle using a slow maintained load (SML), and then after five days, the second load cycle was done with a quickly maintained load (QML) in each of the five test piles.

7 RESULTS

7.1 *Instrumentation at the top of the piles*

Table 2 presents results for the two piles tested at the Vitoria site: test piles EH1 and EH2. Table 2 specifies the ultimate loads and maximum displacements reached in the slow and quickly maintained load tests (SML and QML, respectively). Figure 3 shows load *vs* displacement curves for both slow and quick load tests.

Table 2- Ultimate loads and maximum displacements reach by the load tests at the Vitoria site.

Pile	**Test**	**Ultimate Load (kN)**	**Settlement (mm)**
EH1 L=8m	SML	720	41,99
	QML	756	40,79
EH2 L=12m	SML	1100	69,97
	QML	1150	47,84

The increase in the pile response from the first load cycle (SML) to the second (QML) was of 5% for the EH1 pile and of 4.5% for the EH2 pile. This shows that the use of quick or slow load application rate had little influence on the bearing capacity of the two piles tested. Load tests were conducted until the measured displacements at the top of the piles were greater than 10% of the pile nominal diameters. Both piles reached their maximum capacities with displacements of in average 7.4% of their diameter, about 30mm.

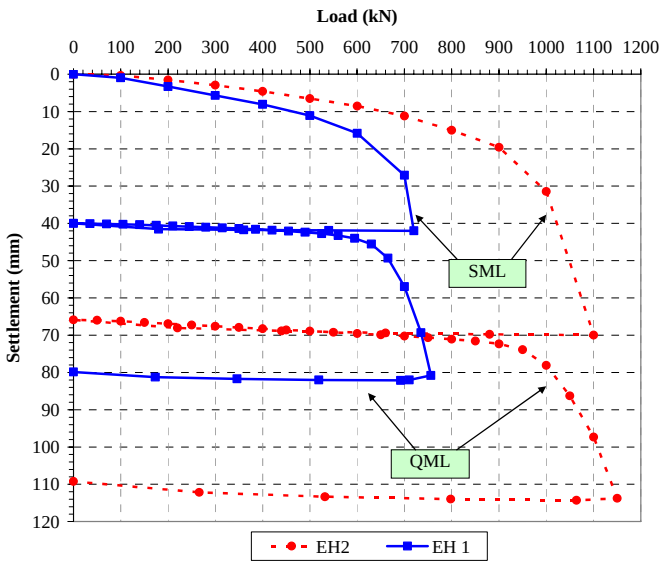

Figure 3 – Load × Settlement Curves (Vitoria).

Figure 4 presents the curves load *vs* displacement for the piles tested at the Campinas site for both, slow and quickly maintained load tests. Table 3 specifies the ultimate loads and maximum displacements reached in the tests performed.

Table 3 – Ultimate loads and maximum displacements reached by the load tests at the Campinas site.

Pile	Test	Ultimate Load (kN)	Settlement (mm)
CFA1	SML	960	80,24
	QML	810	70,48
CFA2	SML	975	85,62
	QML	915	62,77
CFA3	SML	720	88,23
	QML	683	62,09

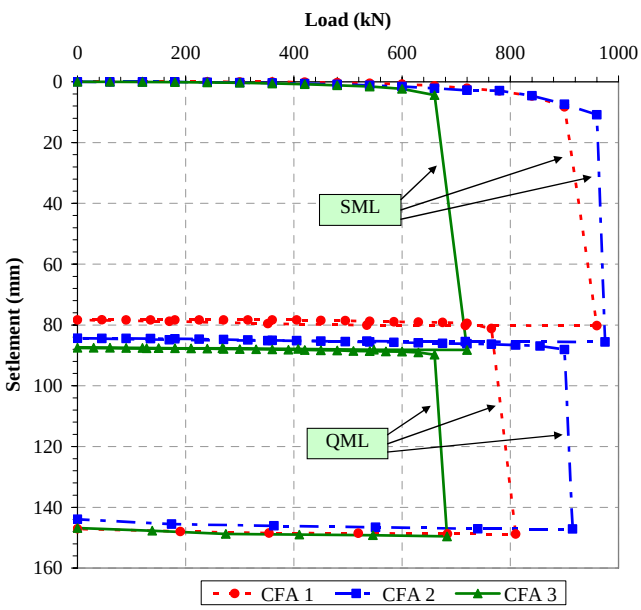

Figure 4 – Load × Settlement (Campinas).

From Table 3 and Figure 4, for both load cycles, CFA 3 presented a smaller ultimate load than the other two piles (CFA 1 and 2). In the slow load cycle, CFA 3 ultimate load was 26% smaller than the average obtained for CFA 1 and CFA 2, and in the quick load cycle, it was 21% smaller. The reason for this behavior could be related to a reduction in the lateral friction, a tip deficiency or even due to soil heterogeneity. All piles reached failure with displacements of about 4.8% of their diameter in average (19mm).

7.2 *Pile Shaft Instrumentation*

Table 4 presents the elasticity modules and the respective coefficients of determination for all piles tested.

Table 4 – Elasticity modules for the piles and coefficients of determination.

Site	Pile	SML		QML	
		E (GPa)	R^2	E (GPa)	R^2
Vitoria	EH1	26,7	0,999	24,7	0,999
	EH2	20,7	0,993	26,7	0,999
Campinas	CFA1	23,0	0,999	24,2	0,999
	CFA2	22,2	0,997	21,1	0,997
	CFA3	15,2	0,993	11,2	0,999

Table 4 shows that the elasticity modules for the piles varied greatly in both experimental sites. This is due to the properties of the concrete used, as well as the concrete execution processes. The elasticity modules determined for pile CFA 3 (Campinas) was smaller than for the other two piles tested at that same site. CFA 3 also presented the smaller ultimate loads of the three piles tested in Campinas.

The coefficients of determination indicate a linearity of the curves. The average values of the elasticity modules were, in general, within the limits expected, indicating that the instrumentation worked well.

7.3 *Load transfer along the depth*

Tables 5 and 6 present for the slow and quick load cycles, respectively, the maximum unit skin friction observed in each instrumentation level. Tables 5 and 6 also present values for the tip resistance obtained in each experimental site.

Table 5 – Maximum unit shaft friction obtained from slow and quick load tests (Vitoria).

Pile	Level – fs (max)		Tip(kN)
EH1	0 – 3,7m	3,7 – 7,7m	---
SML	64kPa	70kPa	91
QML	85kPa	52kPa	138
EH2	0 – 5,85m	5,85 – 11,85m	---
SML	58kPa	63kPa	206
QML	42kPa	59kPa	402

For pile EH1, skin friction accounted for most of the pile response to the load applied at the top: 88% for the slow test and 84% for the quick. The same was observed for pile EH2, with skin friction responding to 90% of the load applied in the slow test. The small decrease in the skin friction response could be explained by a possible densification of the sandy soil at the tip region after large displacements imposed by the tests, which made the soil more resistant and less compressible.

For pile EH1, skin friction responded to most of the load applied at the top: 88% for the SML test and 84% for the QML test. The same was observed for pile EH2, with skin friction responding for 90% of the load applied in the SML test. The small decrease in skin shear response could be explained by a possible densification of the sandy soil at the tip region after the large displacements imposed by the first load cycles, which made the soil more resistant and less compressible.

For pile EH1, the maximum skin friction for the SML test occurred at a displacement of 26.7mm, which corresponds to approximately 6.4% of the pile nominal diameter. For the QML test, the displacement corresponding to the maximum skin friction was 16.5mm (4.0% of the pile diameter). For pile EH2, for the SML test the displacement was 30.2mm (7.2% of the diameter) and for the QML test, 11.1mm (2.7% of the diameter).

The tip resistance, for pile EH1, responded to 13% of the total load applied in the SML test and to 18% in the QML test. For pile EH2, these results were 19% and 35%, respectively. In both cases, an increase of the tip resistance was observed for the second load cycle (QML test), indicating a larger mobilization of the tip resistance after a second load cycle was applied.

Table 6 – Maximum unitary skin friction obtained from slow and quick tests (Campinas).

Pile	**Level – fs (max)**		**Tip(kN)**
CFA 1	0 – 5m	5 – 12m	---
SML	80kPa	47kPa	102
QML	68kPa	42kPa	67
CFA 2	0 – 5m	5 – 12m	---
SML	80kPa	53kPa	71
QML	72kPa	51kPa	65
CFA 3	0 – 5m	5 – 12m	---
SML	69Pa	36kPa	23
QML	65kPa	35kPa	18

The average maximum unitary skin friction obtained at the Campinas site for piles CFA1, CFA2 and CFA3 at the SML tests were 60kPa, 63kPa e 49kPa, with displacements of approximately 7,8mm (2,0% of the pile diameter); 7,0mm (1,8% of the diameter) e 3,9mm (1,0% of the diameter), respectively. The maximum skin friction was observed at small displacements and results for the two first piles were relatively close. For the SML tests, the skin friction had a small reduction, with results of 52kPa, 59kPa and 46kPa, and displacements of approximately 2,5mm (0,6% of the diameter); 3,1mm (0,8% of the diameter) e 1,7mm (0,4% of the diameter), for piles CFA1, CFA2 and CFA3, respectively. There was a smaller tip resistance in the QML test than in the SML test, indicating a smaller mobilization at the tip resistance after a second load was applied.

8 CONCLUSION

The conclusions listed below should be used with caution once they were reached after the analyses of only two SML tests at the Vitoria and three at the Campinas experimental site.

8.1 *Main Conclusions for the Vitoria Experimental Site*

From the instrumentation installed at the tip of the piles, it was observed that:

- Curves load × displacement for the SML tests did not present a well defined failure, despite the large displacement reached (≥ 10% of pile diameters). The maximum load for pile EH1 was defined as 700kN and for pile EH2 was 1000kN.
- The ultimate loads reached by the piles were similar for both SML and QML tests. The increase in pile response from the slow to the quick test was of 5% for pile EH1 and 4.5% for pile EH2. This shows that the rate of load application had a minor influence in pile response.
- For the second load cycle (QML), there were small displacements for load increments until close to failure. This shows that the displacements were mainly plastic. This was also observed during the unloading cycles.
- Piles reached the ultimate load, which was defined mathematically, with displacements close to the values usually observed in driven piles. Therefore, considering the displacements necessary to reach the ultimate resistance, the tested piles had a behavior closer to driven piles than to cast-in-place piles.

Considering the instrumentation installed along the piles, it was observed that:

- The applied load was transferred mostly to skin friction. For pile EH1, skin friction responded for 88% of the applied load in the SML test and for 84% in the QML test. For EH2, the skin friction responded for 90% in the SML test and for 67% in the QML test. The observed decrease in skin friction

during the second load cycle is possibly related to a densification of the soil in the tip region, which could have happened during the first load cycle.
- A progressive mobilization of the tip reaction was observed in the two tested piles. The ultimate tip resistance occurred for accumulated tip displacements of 19.3% of the pile diameter for EH1 pile and 44.8% for the EH2 pile.

8.2 *Main Conclusions for the Campinas Experimental Site*

From the instrumentation installed at the tip of the piles, it was observed that:
- Curves load × displacement for the SML and QML load tests presented a clear definition of failure.
- The average ultimate load was approximately 885kN. The reduction in the average ultimate load observed in the second load cycle (from 885 to 803kN) can be associated to the shaft geometric characteristics, such as "vaulting" at the top part of the piles, which were confirmed after pile extraction. One of these geometric characteristics was a decrease in the contact soil-pile observed at the top soil layer at the end of the first load cycle.

Considering the instrumentation installed along the piles, it was observed that:
- The functions of load transfer presented a good definition, both for skin friction and tip reaction.
- Small displacements were necessary for the whole mobilization of the skin friction (6.2mm). The average tip resistance was 7% of the total load applied. This number is similar to what was observed for a precast pile tested at the same site and with the tip resting at the same depth (Albuquerque, 2001). In general, cast-in-place piles have loose and disturbed soil at the tip and, for this reason, tip resistance is expected to be low. The tip resistance of the piles tested in the present work had an intermediate behavior between driven precast and cast-in-place piles.
- Most of the skin friction distributed along the the upper part of the pile shaft, due to "vaulting" effect. This effect was responsible for the decrease in skin friction in the SML tests..

8.3 *Placement of pile instrumentation*

Considering the instrumentation used in both experimental sites:
- There was a consistency in the results obtained for the product E.A. (elasticity modules *vs* pile section) This confirms that the use of instrumented CA-50 steel bars using "complete bridge" (electric linking) and installed through a metal tube placed longitudinally and at the centre the piles is a good technique to monitor load transfers in deep foundation elements.
- From the experience gained with the present work, it is recommended that the instrumentation for CFA piles (instrumented bars inside a metallic tube) be installed when the auger reaches the desired depth. The tube and the instrumented bars should be installed through the auger axis, before concrete is pumped in.

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