Behavior of continuous flight auger piles subjected to uplift load tests in unsaturated diabasic soil

P.J.R. Albuquerque
State University of Campinas (Unicamp), Campinas, Brazil.

J.A. Paschoalin Filho
Nove de Julho University (Uninove), São Paulo, Brazil.

D. Carvalho
State University of Campinas (Unicamp), Campinas, Brazil.

ABSTRACT: This paper presents the behavior of three continuous flight auger piles conducted in unsaturated diabasic soil submitted to uplift forces. The piles were built at the site for Experimental Studies in Soil Mechanics and Foundations at Unicamp, located in the city of Campinas, Brazil. Field tests have already been conducted at the site (SPT, CPT, DMT and PMT), as well as laboratory tests using sample soils taken from a well up to 17m deep. The water table is not checked until a depth of 17m. In order to check the behavior of the piles when submitted to uplift forces, slow static load tests were carried out as per the recommendations of NBR 12.131/92. The carrying capacity of these piles was also provided by means of theoretical methods, appropriate for uplift forces, and through semi-empirical methods appropriate for compression forces, considering only the portion of lateral resistance. The values estimated using the methods considered were compared to those obtained by means of load tests. One of the tested piles was extracted from the soil to be the subject of a study on its geometry.

1 INTRODUCTION

During the last few years, there have been great advances in the development of building with deep foundations, due to ever greater requirements for productivity and the constant increase in loads to be transferred to the subsoil. Because of this, foundation engineering has had to closely track this growth, developing new techniques to build deep foundations using "in situ" molded piles.

The Continuous Flight Auger (CFA) pile, built using a continuous helix auger, was first used in the United States during the fifties. In Europe, this type of pile was introduced in the seventies. In Brazil, the use of CFA piles has become a constant in medium-to-large size jobs, principally in those located in the state of São Paulo, where the largest number of companies building this type of foundation is to be found.

As the use of this pile is increasing, it becomes imperative to understand its behavior.

The interior of the state of São Paulo, where Campinas is situated, where economic growth is marked, a large number of medium-to-large construction jobs are generated, bringing about an increase in the use of these kinds of foundations, principally because the jobs are mostly industrial, where time is essential in defining the construction method.

1.1 *Continuous Flight Auger Piles – An historical review.*

From the time of their introduction in the USA until the present day, significant investment has been made in CFA piles and presently they can be built up to 32m deep, in diameters up to 1200mm, at an available torque of up to 390kN.m.

The CFA pile was introduced into Brazil in 1987, using locally manufactured equipment, based on foreign models, allowing the construction of 275mm, 350mm and 425mm diameter piles, at a maximum depth of 15m. In 1993, due to changes in importation laws, it was already possible to build piles of up to 1000mm diameter and lengths of up to 25m.

CFA piles became very popular in the eighties, due to their technical advantages, combined with relatively low cost (Brons & Kool, 1988). However, theses authors warn that the production process should receive special attention, especially with pile column continuity, subsoil disturbance due to auger extraction and failure in weak soils due to high applied pressures that can lead to significant consumption of concrete. Moreover, they warn that the increasing competition between construction companies, with the consequent cost reduction, can lead to a reduction in quality when producing these piles.

Operator sensitivity to controlling the construction of CFA piles is the most severe limitation of these piles (Bottiau, 1993). A great deal of attention must be exercised during the entire installation process, including excavation, auger extraction and positioning the reinforcements. This author also mentions that the CFA pile was developed with the aim of eliminating one of the most important disadvantages of the bored piles: soil decompression. In field studies, using a Marchetti dilatometer, before and after building a pile, it was seen that the construction process did not cause this decompression. Bottiau (1993) emphasizes that another important advantage of the CFA pile is the possibility of continuous monitoring, which furnishes documentation on the pile´s construction.

1.2 *Uplift Capacity of deep foundations*

Uplift Capacity of piles depends on several factors such as: a) Different types, characteristics and properties of soil; b) types of piles, their executive and geometric characteristics; c) types of loading (accidental, permanent and cyclic). Orlando (1999).

Design of piles to resist uplift forces is very common in Foundation Engineering. There are many situations in which this kind of force is primarily considered, such as: foundations of electrical transmission towers, foundations that cross over extensive soils; foundations of light structures submitted to the forces of wind; etc.

There are many different theoretical methods to estimate the uplift capacity for piles. However the use of these methods is very limited. This occurs because the parameters involved with these methods are very difficult to obtain.

In addition, in many situations, these methods also present very optimistic or very conservative estimated uplift capacity values.

In Brazil, estimating uplift capacity for piles using semi-empirical methods developed for compressive forces is a common practice among foundation engineers. It is assumed in these cases that the uplift capacity should be a percentage of the total skin friction resistance of the pile when it is submitted to compression forces.

There are many different methodologies to obtain the uplift capacity of a pile. The description of these methods is recounted by many pieces of research, such as Danziger (1983), Carvalho (1991), Paschoalin Filho *et al* (2008a, 2008b), etc.

2 EXPERIMENTAL FIELD CHARACTERIZATION.

The Experimental Field for Studies in Soil Mechanics and Foundations is situated at the State University of Campinas, within the boundaries of the city of Campinas, in the interior region of the state of São Paulo, Brazil. The experimental field has a total area of 400m^2.

Several field tests, such as SPT, CPT, CPTU, Cross Hole, Marchetti Dilatometer, Refraction Seismics, Vertical Electric Investigation, were carried out in this experimental field.

Laboratory tests using disturbed samples (characterization tests) and undisturbed samples (triaxial, simple compression, odometer, permeability tests, etc.) were also carried out. Static and Dynamic Load tests in different kinds of piles, such as precast concrete piles, steel piles, root piles, omega piles, bored piles, etc. were also performed.

The local subsoil is basically composed of migmatites, in which intrusive rocks occur, from the Serra Geral Formation (diabasic), covering 98km^2 of the Campinas region, about 14% of its area. Diabasic bodies are also found, incrusted in the Itararé Formation and in the Crystalline Complex, as “sills” and “dykes”. At the outcrops, it may be observed that the diabasic soil is quite fractured, with the formation of small blocks; the fractures are usually open or filled with clayey material.

In the experimental field a diabasic soil is present, presenting a superficial layer approximately 6.5m thick, composed of high porosity silty-sand clay, followed by a clayey-sandy silt to a depth of 19m; the water table is reached at 17m. The soil of the first layer is weaker than the lower layer and is collapsible, presenting collapse ratios ranging from 2.4% to 24%, depending on the applied pressure, according to Vargas (1978). Some geotechnical characteristics of the experimental area are presented in the following tables.

Table 1. Average results of the field tests.

Soil	Depth (m)	Nspt	qc (kPa)	fc (kPa)
Reddish brown silty-sandy clay	1	3.0	392	28
	2	2.7	589	19
	3	3.0	883	36
	4	4.0	1324	63
	5	5.0	1864	85
	6	6.5	2502	130
Clayey-sandy silt mix (residual soil)	7	6.2	2453	168
	8	7.0	2256	193
	9	7.6	2158	204
	10	9.2	2009	221
	11	10.2	2551	254
	12	10.2	2404	238
	13	10.0	2600	265
	14	10.4	2551	224
	15	9.5	2354	198

Key: qc and fc are, respectively, the point of resistance and lateral friction from CPT (Cone Penetration Test).

Table 2. Average geotechnical parameters obtained by laboratory tests.

Depth (m)	ρ_{nat} kN/m^3	ρ_s kN/m^3	e	c (kPa) *	ϕ^o *	R_c (kPa)
1	13.4	29.7	1.77	5	31.5	26.2
2	13	29.1	1.76	11	31.5	48.0
3	13	29.5	1.79	2	30.5	40.7
4	13	30.1	1.86	0	26.5	11.2
6	15.4	30.1	1.44	18	18.5	54.1
7	15.4	29.1	1.40	31	22.5	76.1
8	14.8	29.5	1.56	18	25.5	59.7
9	15	30.1	1.6	64	14.5	50.6
10	15.1	30.1	1.6	78	22.8	67.0
12	16.1	29.6	1.46	87	18.3	145.1
14	16.4	30.6	1.48	76	19.1	185.4
16	16.7	30.1	1.51	55	22.0	218.7

Key: ρ_{nat}, ρ_s, and, c*, Ø*, Rc are, respectively, natural specific weight, grain specific weight, void ratio, total cohesion, total friction angle, simple compression resistance.

3 TEST PILES AND REACTION SYSTEM

Three Continuous Flight Auger piles were conducted at the area under study. The CFA pile dimensions are as follows: nominal diameter 0.40m and length 12m. The piles followed a pre-defined alignment and spacing between them was 4.80m. Three pile head blocks were also built with dimensions of 0.70 x 0.70 x 0.70m^3 for each pile studied.

For the execution of the CFA piles, a MAIT HR-200 drill press with a depth capacity of 32m was used. The equipment´s torque ranged from 220kN.m to 380kN.m; this variation is a function of the rotation speed and diameters used.

The concrete used in piles ($\pm$ 240mm slump and transportable by pump) consumed cement at a rate of 400kgf/m^3 and aggregates (sand and fine crushed rock). For the pile head blocks, concrete with fck=25MPa was used.

The longitudinal reinforcement of the piles was composed of 4Ø 16mm (approximately 8.0cm^2), 6m in length and stirrups of Ø=6.4mm every 20cm. The steel used was of the CA-50A type.

The reaction system was composed of a reaction beam, double "I" section, designed to support loads applied on its axis, 5.3m in length and by a steel tie-rod system composed of ST85/105 (Dywidag) special bars, 32mm in diameter, nuts, plates and steel sleeves, all manufactured with the same material.

4 STATIC LOAD TESTS

For each CFA pile, a static load test was carried out with load maintenance. The static load tests were carried out in accordance with the guidelines established by Brazilian Standards (NBR 12.131/92).

The load applications were applied in steps of 120kN, up to the load at which the displacements indicated a rupture of the pile. Unloading was performed in four stages.

5 THEORETICAL AND SEMI-EMPIRICAL METHODS USED FOR UPLIFT CAPACITY ESTIMATING.

The carrying capacity of the studied piles was also provided by means of theoretical methods appropriate for uplift forces. The methods considered were: Meyerhoff (1973), Levacher & Sieffert (1984) and University of Grenoble according to Martin (1973).

Semi-empirical methods were also used, appropriate to compression forces, considering only the portion of lateral resistance. The methods studied were the following: Décourt & Quaresma (1978); Aoki & Velloso (1975, CPT and SPT); Antunes & Cabral (1996) and P.P.Veloso (1981).

The values estimated using the methods under consideration were compared to those obtained by means of the load tests.

6 RESULTS OF THE LOAD TESTS

The load vs. settlement curves for all CFA piles studied are shown in figure 1. The values for ultimate load and maximum displacement for each pile are presented in table 3.

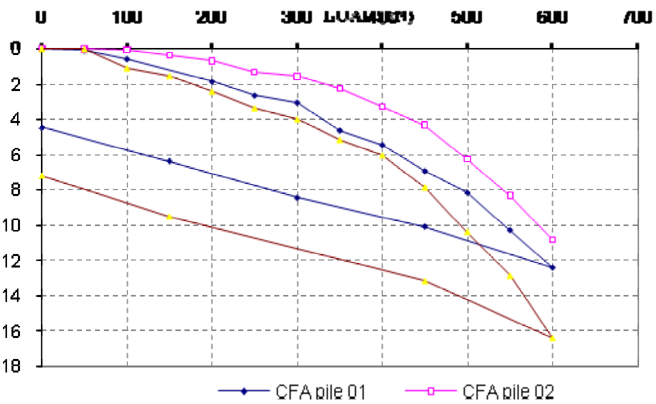

Figure 1. Load vs. settlement curves for all piles studied.

Table 3. Ultimate loads and maximum displacements reached in the loading tests.

Pile	Ultimate Load (kN)	Maximum Displacement (mm)
CFA 01	600	12.4
CFA 02	600	10.8
CFA 03	600	16.4

The three load tests were stopped prematurely because of the inadequacy of the reaction system. So the ultimate loads were estimated by the Van der

Veen method (1953). The values of the estimated ultimate loads are presented in table 4.

Table 4. Estimated ultimate loads using Van der Veen´s Method (1953)

Pile	Estimated ultimate load (kN)	Estimated ultimate load mean (kN)	Standard deviation
CFA 01	700		
CFA 02	600	667	58
CFA 03	700		

7 LATERAL RESISTANCE VALUES ESTIMATED BY SEMI-EMPIRICAL METHODS

The values for lateral resistance estimated by semi-empirical methods are presented in table 5.

Table 5. Lateral resistance values estimated using the studied methods.

Method	Pile	Estimated ultimate load (kN)	$RL_{est}/P_{load\ test*}$	Mean
D&Q	CFA 01	413	0.6	
(1996)	CFA 02	413	0.7	0.63
	CFA 03	413	0.6	
A&V	CFA 01	230	0.3	
(SPT,	CFA 02	230	0.4	0.33
1975)	CFA 03	230	0.3	
A&V	CFA 01	320	0.45	
(CPT,	CFA 02	320	0.5	0.47
1975)	CFA 03	320	0.45	
A&C	CFA 01	260	0.37	
(1996)	CFA 02	260	0.4	0.38
	CFA 03	260	0.37	
P.P Vel	CFA 01	442	0.63	
(1981)	CFA 02	442	0.74	0.67
	CFA 03	442	0.63	

Key: D&Q=Décourt & Quaresma (1978); A&V=Aoki &Velloso (1975); A&C=Antumes & Cabral (1996); P.P Vel=P.P. Veloso (1981); Pload test*=Ultimate load estimated by Van der Veen´s Method (1953).

The comparison between the mean values obtained of RL_{est}/P_{load}, determined by each semi-empirical method considered, is presented in figure 2.

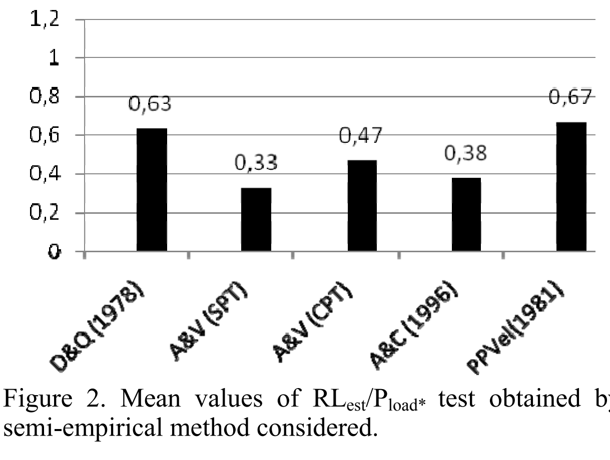

Figure 2. Mean values of RL_{est}/P_{load*} test obtained by each semi-empirical method considered.

In accordance with figure 2, the method of P.P Velloso (1981) presented the mean value of $RL_{est}/P_{load\ test*}$ closest to unity compared with the other methods. The method of Aoki & Velloso (SPT) presented the most conservative value of $RL_{est}/P_{load\ test*}$ of all the methods studied. All methods projected lower values than those obtained through the load tests.

8 UPLIFT CAPACITY VALUES ESTIMATED BY THEORETICAL METHODS

The values for estimated uplift load capacity using the theoretical methods studied are presented in table 6. The comparison between the mean P_{est}/P_{load**} values obtained, as determined by each theoretical method considered, is presented in figure 3.

Table 6. Uplift capacity values estimated by the studied methods.

Method	Pile	Estimated ultimate load (kN)	$P_{est}/P_{load\ test**}$	Mean
Meyerhoff[1]	CFA 01	1063	1.52	
(1973)	CFA 02	1063	1.77	1.6
	CFA 03	1063	1.52	
Meyerhoff[2]	CFA 01	1208	1.72	
(1973)	CFA 02	1208	2.0	1.81
	CFA 03	1208	1.72	
Levacher	CFA 01	844	1.2	
&	CFA 02	844	1.4	1.27
Sieffert	CFA 03	844	1.2	
(1984)				
	CFA 01	908	1.3	
Grenoble[#]	CFA 02	908	1.5	1.37
	CFA 03	908	1.3	
	CFA 01	981	1.4	
Grenoble[*]	CFA 02	981	1.6	1.47
	CFA 03	981	1.4	

Key: [1]= ca=0.8c, δ=0.95Ø and Ku=1.0; [2]= ca=c, δ=Ø and Ku=1.0; [#]=λ=0; *= λ=-Ø/8; Pload test**= Ultimate load estimated by Van der Veen´s Method (1953).

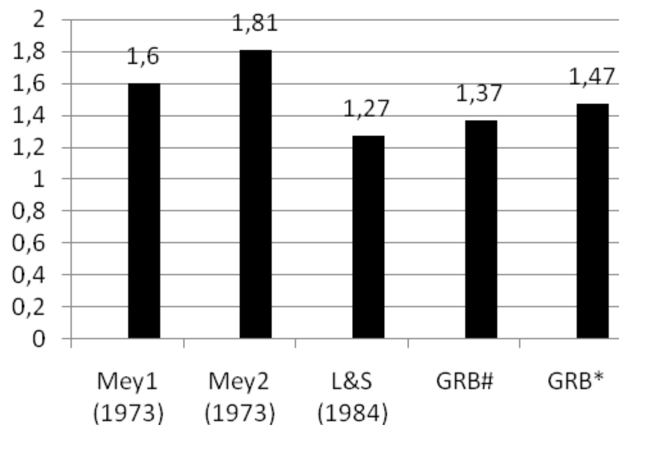

Figure 3. Mean values of P_{est}/P_{load**} test obtained by each theoretical method considered.

According to figure 3, the method of Levacher & Sieffert (1984) presented the mean value of $P_{est}/P_{load\ test**}$ closest to unity compared with the other theoretical methods considered. The method of Meyerhoff (1973), assuming ca=c, δ=Ø, presented the mean $P_{est}/P_{load\ test**}$ value which was furthest from unity. All the theoretical methods employed presented $P_{est}/P_{load\ test**}$ values, on average, at least 27% higher than the actual values obtained in the load tests.

9 EXTRACTION OF A PILE

After performing the load tests, a pile was extracted, with the objective of knowing its geometric characteristics. For this work to be possible, a complete study was required on the possible ways of extracting the pile. Several field works were performed to make the extraction viable. All steps taken at this stage of the research are described below.

9.1 *Removal of the pile head block*

For the pile removal to be possible, the pile head block had to be demolished, to reduce the mass to be hoisted and also not to jeopardize the soil excavation along the shaft.

9.2 *Device to fix the hoist*

To permit hoisting the piles, it was seen that the most adequate means would be to fix a split metallic ring at the top of the pile. To build the ring, it was necessary to determine the pile perimeter with the objective of obtaining its diameter with maximum accuracy, so that each ring would fit perfectly onto the pile. The ring was fixed onto the pile by uniting its two parts and filling the pile-ring interface with cement slurry, to assure the connection between them. Ring details are presented in Figure 4.

Figure 4. Arrangement to fix the pile.

9.3 *Pile extraction*

Thirty-three days after the ring was fixed, pile removal was performed. For this, it was necessary to manually excavate around its shaft. To lift the piles, an appropriate hoist was employed, since it should lift the pile at least 1 m above the ground (Figure 5).

Figure 5. Extracted pile.

9.4 *Post-extraction analysis of the pile*

A complete examination of the pile was performed, revealing important data of the shaft surface, its geometry and point shape.

a) The shaft surface showed some crimps, formed by the drill, throughout the lenght of the shaft (Figure 6). Detail of the pli tip is shown in Figure 7.

b) It was been a survey of the pile perimeter and thus to get the average diameter. It was possible verify an increase of the diameter from 1.5 to 3.0m in length (Figure 8).

c) The shaft perimeter may also be determined, thus obtaining its average diameter; it was verified that the actual diameter (40,4cm) was on average 1% greater than the nominal diameter (40cm).

Figure 6. Crimps along the shaft.

Figure 7. Detail of the CFA pile tip.

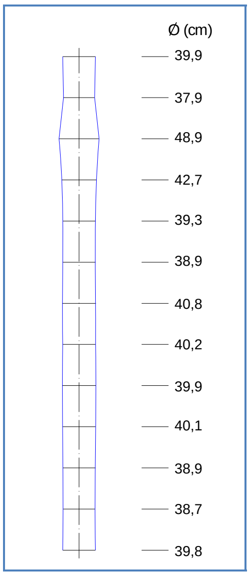

Figure 8. Detail of the CFA diameter in depth.

10 CONCLUSIONS

Observing the parameters obtained through this research, the following conclusions may be reached:

a) The semi-empirical methods used in this research led to estimated values lower than the values obtained by the load tests. The method with the closest values obtained average values of 67% of the values obtained in the load tests.

b) The theoretical methods considered led to estimated uplift capacity values higher than the uplift loads obtained with the load tests.

c) The use of the theoretical methods depends on the adoption of parameters like: ca, δ, Ø, etc. This situation, due to the lack of good soil value data along the length of the pile shaft, may lead to significant mistakes in uplift capacity estimates.

d) The helix auger has left crimps along the shaft. There was an increase in pile diameter between 1.5m and 3m depths, may have been caused by the pressure of concrete on the layer of soil weaker

11 ACKNOWLEDGEMENTS

The authors would like to express their gratitude to the University of Campinas (Unicamp), Nove de Julho University (Uninove) and FAPESP by the sponsorship of this research, Brazil.

REFERENCES

Antunes, W.R., Cabral, D. A. 1996. Load capacity of continuous flight auger pile. In: Seminar of Engineering of Special Foundations. 3rd, 1996, São Paulo-Brazil. Proceedings… São Paulo: ABMS, v.2, 1996.p.105-109.

Aoki, N.; Velloso, D.A. 1975. An approximate method to estimate the bearing capacity of piles. In: Pan-American Congress of Soil Mechanics and Foundation Engineering, 5th, 1975, Buenos Aires-Argentina. Proceedings…p.367-376.

Bottiau, M. 1993. Pile walling with the PCS Method. In: I.G.S.D.F.B.A.P., 2nd, 1993, Ghent-Belgiun. Proceedings…Rotterdam: A.A. Balkeman, 1993. p.185-190.

Brazilian Technical Standards Association: Static load tests: NBR 12.131/92. Rio de Janeiro: A.B.N.T., 1992. (In Portuguese).

Brons, K.F., Kool, A.F.1988. Methods to improve the quality of auger piles. In: International Geotechnical Seminar on Deep Foundations on Bored Auger Piles, 1st , 1998, Ghent-Belgium. Proceedings…Rotterdam: A.A.Balkema, 1988, p.269-272.

Carvalho, D. 1991. Analysis of uplift loads of instrumented excavated piles built at experimental Field of São Carlos. 204p. Thesis. University of São Paulo. São Carlos. 1991. (In Portuguese).

Danziger, F.A.B. 1983. Load capacity of foundations submitted to vertical uplift forces. 331 p. (Dissertation) Federal University of Rio de Janeiro. 1983. (In Portuguese).

Décourt, L.; Quaresma, A.R. 1978. Capacidade de carga de estacas a partir de valores de SPT. In: Brazilian Congress of Soil Mechanics and Foundation Engineering, 4th , 1978, Rio de Janeiro-Brazil. Proceedings…São Paulo: ABMS, 1978, p.367-376. (In Portuguese).

Levasher, D.R.; Sieffert, J.G. 1984. Tests on model tension piles. Journal of Geotechnical Engineering. A.S.C.E., v.110, n12, p1735-1747. Dec. 1984.

Martin, D. 1973. Calcul des pieux et foundations à dalle des pylônes de transport d'énergie électrique. Etude théorique et resultats d'essais en laboratorie et in-situ. Annales De L'Institut Technique du Bâtiment et des Travaux Publics, v. 307-308, p. 105-130, juil.-oct.,1973

Meyerhoff, G.G. 1973.The uplift capacity of foundations under oblique loads. Canadian Geotechnical Journal, v10 p64-70, 1973.

Paschoalin Filho J.A. Albuquerque, P.J.R.A,Carvalho, D., Nogueira, R.C.R. 2008a. Behavior of root piles submitted to uplift loads performed in diabasic soil, typical of the region of Campinas, Brazil. In: Congresso Luso-Brasileiro de Geotecnia, 4th, 2008, Porto. Portugal. Proceedings…Porto: ABMS, 2008, cd-rom (In Portuguese).

Paschoalin Filho J.A. Pereira, M.; Carvalho, D.; Nogueira, R.C.R; Albuquerque, P.J.R. 2008b. Behavior of excavated piles built in diabasic soil submitted to uplift forces. In: Brazilian Congress of Agricultural Engineering, 36th, 2008, Bonito. Brazil. Proceedings…Jaboticabal: Sbea, 2008, cd-rom (In Portuguese).

Van der Veen, C. 1953. The bearing capacity of a pile. In: Procedures of Third International Conference of Soil Mechanics and Foundation Engineering. Vol 2 , pp 84-90, Zurich. 1953.

Vargas, M. 1978. Soil Mechanics. São Paulo, McGraw Hill – Brazil Ltda. 1978.509p. (In Portuguese).