Evaluation of DMT and CPT Parameters to be used in Numerical Modeling of Piles

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ABSTRACT: The behavior of a foundation element is directly linked to geotechnical parameters of the surrounding soil. We can point our in-situ tests to determination parameters of soil, Marchetti Dilatometer Test (DMT) and Cone Penetration Test (CPT), which reliably provide a greater number of soil parameters. In this sense, the soil resistance parameters are estimated based on the results of these tests by means of correlations found in the geotechnical literature and used in numerical analyses. This way, the behavior of an instrumented continuous flight auger pile (ϕ = 0.40 m and L = 10 m) is analyzed. The pile was executed at the Unicamp Experimental Foundation Site located in the city of Campinas/SP/Brazil, where the profile of the subsoil is comprised of diabase, with an approximate 6.5 m thick surface layer, constituted of high-porosity silty-sandy clay, followed by a layer of clayey-sandy silt down to 19 m. The water level is found at 17 m. The aforementioned pile was tested via slow maintained load (SML) test and the results obtained were compared to those obtained by means of three-dimensional numerical modeling which utilizes the method of finite elements based on parameters estimated by the DMT and CPT tests.

1 INTRODUCTION

The DMT and CPT are in-situ tests widely used in the design of deep foundations. However, this technique is not yet used as a routine by Brazilian geotechnical engineers. The reason could be the doubt concerning the possibility of using the available correlations in the international literature to estimate geotechnical parameters and bearing capacity, especially when lateritic, high porosity and unsaturated soils are being studied.

The load transfer response of the pile is a very important parameter in order to estimate the bearing capacity of the foundation system when the point resistance is not the main resistance to take into account. For that reason, it could be very important to get a reliable value either from a in-situ test or from another tool. The results from CPT and DMT tests provide stratigraphic identification of soil profiles, and help estimate their mechanical properties based on empirical and semi-empirical correlations to be applied in several areas of geothecnics, particularly to estimate the capacity of load and settlements of foundations (Budhu, 2006; Barnes, 2000 and Chen, 2002).

In this paper the results from in-situ tests (DMT and CPT) conducted at the Experimental Site of Unicamp (Campinas/SP, Brazil) will be the basis to get the soil parameters to be used in in Cesar-LCPC software to assess the behavior of a 12 m long, 0.40 m diameter continuous flight auger pile instrumented in depth, which was submitted to a slow static load test.

2 EXPERIMENTAL SITE

The research was performed at the Soil Mechanics and Foundations Experimental Site, at the Unicamp Campus, in Campinas, SP, Brazil (Carvalho et al., 2000). Several in-situ tests as well as laboratory tests both on disturbed and undisturbed samples collected from a 16 m deep well have already been performed at this location.

The local subsoil is composed of basic migmatites, where intrusive rocks from the Serra

Geral Formation (diabasic) occur, covering 98 km^2 of the Campinas region, about 14% of its total area. Diabasic bodies are also found incrusted into the Itararé Formation and in the Crystalline Complex, as “sills” and dikes. At the outcrops, it may be seen that the diabase is quite fractured, with the formation of small blocks; the fractures are usually either open or filled with clayey material.

The profile of the subsoil of the experimental site is formed of residual diabase soil, presenting an approximately 6.5 m thick surface layer composed of high porosity silty-sandy clay, followed by a clay-sandy silt to the depth of 19 m; the water level is reached at 17 m. The soil of the first layer is collapsible, presenting collapse ratios ranging from 2.4 % to 24 %, depending on the pressure applied, according to Vargas (1978).

3 ESTIMATE OF GEOTECHNICAL PARAMETERS

3.1 *Marchetti Flat Dilatometer*

The soil parameters were derived from DMT-based empirical and semi-empirical correlations found in the geotechnical literature. According to the classification of soil layers as a function of their mechanical characteristics indicated by the results of the Marchetti Flat Dilatometer (DMT), the equations shown below were used.

In order to determine the deformability modules of soil (E) layers that were characterized, the equation determined by TC16 (2001) was used:

$$\mathrm{E} = 0{,}8 \cdot M \quad \text{and} \quad \mathrm{M} = R_m \cdot E_D \tag{1}$$

where:
M - Vertical Drained Constrained Modulus;
E_D - Dilatometer Modulus.

And, at-rest earth pressure coefficient:

$$K_0 = \left(\frac{K_D}{1{,}5}\right)^{0{,}47} - 0{,}6 \tag{2}$$

In order to estimate the undrained strength of clayey soils from the DMT based on Ladd et al. (1977) and Mersi (1975), the following equation proposed by Lunne & Lacasse (1989) was used:

$$S_u = 0{,}20 \cdot \sigma'_{v0} \cdot (0{,}5 \cdot K_D)^{1{,}25} \ (\text{kPa}) \tag{3}$$

$$\mathrm{c} = s_u * 0{,}5 \ (\text{kPa}) \tag{4}$$

where: c - cohesion

The values of friction angle (ϕ) were determined from the correlations developed by Marchetti (2001), as shown below:

$$\phi'_{DMT} = 28^{\circ} + 14{,}6^{\circ} \log K_D - 2{,}1^{\circ} \log^2 K_D \tag{5}$$

3.2 *Cone Penetration Test*

The soil parameters were derived from CPT-based empirical and semi-empirical correlations found in the geotechnical literature. According to the classification of soil layers as a function of their mechanical characteristics indicated by the results of the Cone Penetration Test (CPT), the equations shown below were used.

The estimate of geotechnical parameters from the formulations based on in-situ tests made it possible to obtain the behavior of the load-settlement curve produced by the numerical method (Garcia et al., 2013, Garcia & Albuquerque, 2014).

In order to determine the elastic modules of soil layers characterized by the mechanical behavior of sand, the equation determined by Trofimenkov (1974) for Soviet sands is used:

$$Ei = 130 + 3{,}4 \cdot q_c \ (\text{in kgf/cm}^2) \tag{6}$$

The ratio between the deformability modulus (E) and the net cone resistance (q_c - σ_{vo}) for different soils follows the following ratio developed by Kulhawy & Mayne (1990):

$$Ei = 8{,}25 \cdot (q_c - \sigma_{v0}) \ (MPa) \tag{7}$$

For soil layers indicated as having clayey behavior, the deformability modulus is determined by Barata (1986) for clays of the regions of Campinas-SP:

$$Ei = \alpha \cdot q_c \ \ (MPa) \quad where\ 5.2 < \alpha < 9.2 \tag{8}$$

The values of friction angle (ϕ) were determined from the correlations developed by Kulhawy & Mayne (1990) for sands, as shown below:

$$\emptyset_{CPT} = tan^{-1} \cdot \left(0{,}1 + 0{,}381\, log\left(\frac{q_c}{\sigma_{v0}}\right)\right) \qquad (9)$$

where:
q_c – cone resistance;
σ_{v0} – effective vertical stress

In order to estimate the undrained strength of clayey soils from the CPT, the following equation proposed by Lunne et al. (1997), can be used, with N_K assuming values that vary from 15 to 20.

$$s_u = \frac{q_c - \sigma_{v0}}{N_K} \quad (kPa) \qquad (10)$$

where: N_K – cone factor
(N_K = 20 was used in this study)

$$c = s_u * 0{,}5 \quad (kPa) \qquad (11)$$

where: c - cohesion

To estimate geotechnical parameters of the local subsoil, mechanical CPT and DMT were performed. The CPT results are shown in Fig. 1. The CPT and DMT measurements are used to get the soil mechanical parameters required for numerical analyses such as cohesion, friction angle and deformability modulus.

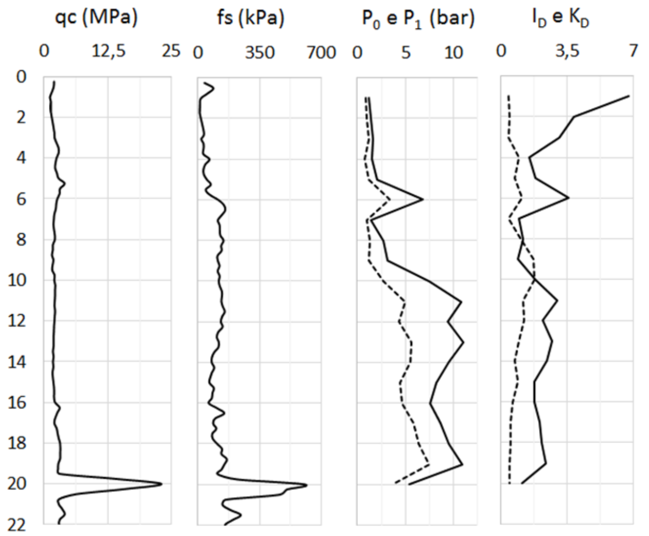

Fig. 1. Results of DMT e CPT Tests (Fontaine, 2004).

Based on the results from the mechanical CPT, it was possible to verify that the subsoil consists of diabase soil, with a surface layer of high porosity silty-clay approximately 6.5 m thick, followed by a layer of clayey-sandy silt down to 9 m. The water level was at 17 m (Table 1). Using the CPT charts of Robertson et al (1986), the soil is classified as sandy silt to silty clay, unlike the behavior obtained based on visual classification (Fontaine, 2004).

Table 1. Parameters obtained by CPT and DMT.

Soil layer	CPT			DMT	
	qc	fs	Rf	I_D	K_D
0 - 6m	2,1	50,5	2,28	0,7	3,4
7 14m	1,9	123,3	6,46	1,1	1,9
15 - 22m	3,9	164,7	4,53	0,6	1,9

qc - tip resistance (MPa); fs - skin friction (kN/m²); Rf - friction ratio; I_D - Material Index; K_D - Horizontal stress Index.

Tables 2 and 3 show the values of geotechnical parameters used in numerical analyses based on the CPT and DMT results.

Table 2. Parameters for numerical modeling obtained by correlations CPT.

Soil layer	γ	c	φ	ν	Ei
0 - 6m	13.5	40	33	0.4	18
7 - 14m	15.5	58	29	0.3	15
15 - 22m	16.5	91	27	0.3	32

γ - Specific weight (kN/m³); c - cohesion (kN/m²); φ - friction angle (°); ν - Poisson coefficient; Ei – Deformability modulus (MPa).

Table 3. Parameters for numerical modeling obtained by correlations DMT.

Soil layer	γ	c	φ	ν	E
0 - 6m	13.5	8	34	0.4	14
7 14m	15.5	16	31	0.3	23
15 - 22m	16.5	24	32	0.3	48

γ - Specific weight (kN/m³); c - cohesion (kN/m²); φ - friction angle (°); ν - Poisson coefficient; E - deformability modules of soil (MPa).

4 INSTRUMENTED CONTINUOUS FLIGHT AUGER PILE

The continuous flight auger pile is formed in situ where the soil is excavated using a continuous auger, with blades around a hollow center tube. After the auger is introduced in the soil down to the specified depth, the auger is extracted while concrete is injected through the hollow tube. As the auger is removed, the soil confined between the blades is removed.

A 12 m deep continuous flight auger pile measuring 0.40 m in diameter was made. The longitudinal frame of the pile included four 6 m long steel bars with a 16 mm diameter (≅8 cm^2), and stirrups with 6.3 mm diameter at every 20 cm (CA-50 steel), made with a MAIT HR-200 drill. The torque of the equipment is between 220 kN.m and

380 kN.m. This variation is due to the rotation speed and the diameter used (Albuquerque, 2001).

The pile was instrumented along the shaft in the following depths: 0.30 m (reference section); 5.0 m; 11.1 m and 11.7 m. To obtain the Young's modulus of the completed pile, a section near the top of the pile was used, where a measuring instrument was placed (instrumented bar). The soil was excavated around this section to prevent any influence on the reading of the instruments. This is referred to as the reference section.

5 RESULTS FROM THE LOAD TESTS

A slow load test was made following the prescriptions of NBR 12.121/92, adopting slow loading. The maximum load for the pile was 960 kN, with maximum displacement of 80.22 mm. Figs. 2, 3 and 4 show the load vs. displacement curve, the graphs of variation in load at each level vs. strain and distribution of the load along the depth.

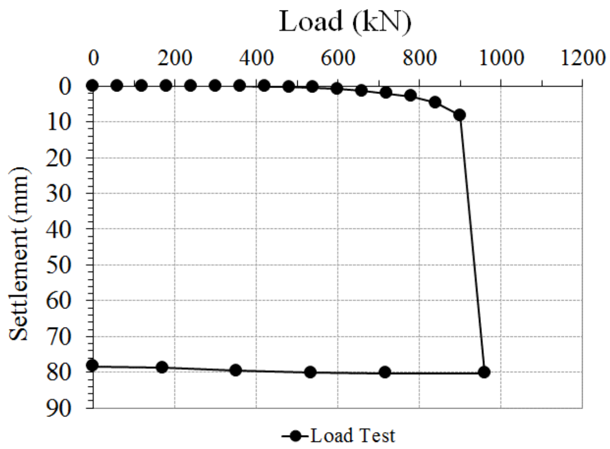

Fig. 2. Load-displacement (Albuquerque et al, 2011)

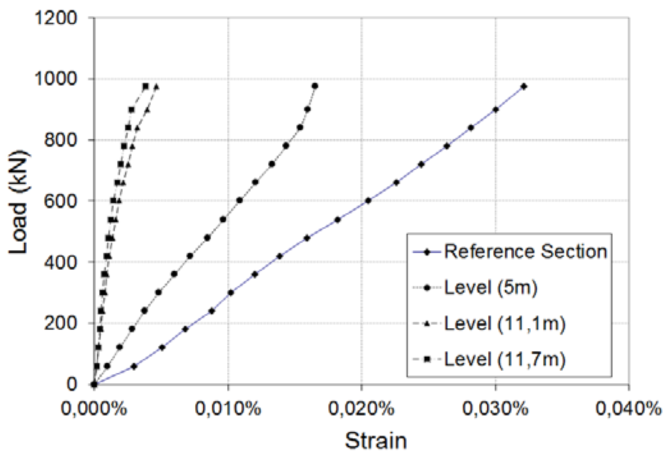

Fig. 3. Load vs. strain (Albuquerque et al, 2011)

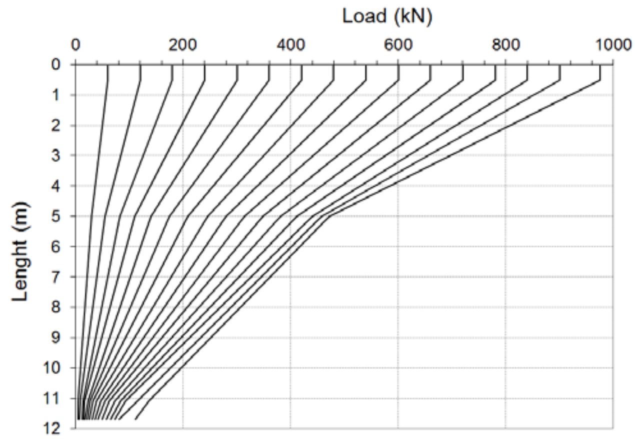

Fig. 4. Load transfer (Albuquerque et al, 2011)

6 NUMERICAL MODELING

Numerical modeling is suitable to refine the mesh close to the pile, particularly at the tip, since this area is critical for results of load capacity of the pile. If possible, a sensitivity analysis of the mesh must be carried out for each problem to enable direct assessment of the influence of the numerical type (Diaz-Segura, 2013).

The modeling was performed from ¼ of the problem under analysis due to the symmetry along the pile shaft, which resulted in a rectangular block of 10 m x 10 m section with variable depth as a function of the length of the pile under analysis, but at least 10 m below the pile tip. These dimensions were determined based on tests performed to ensure that the surrounding conditions attributed at the far margins of the model could be considered as no displacement or had very low displacements and, as a consequence, could not affect the results of the analyses. An elasto-plastic model was used, which varied depending on the stresses applied, following a model of non-linear behavior. The mesh of finite elements was composed of triangular-shaped elements of quadratic interpolation, which were extruded at every meter in depth.

The properties attributed to the different layers of soil followed the Mohr-Coulomb criteria, i.e., values of specific weight (γ), cohesion (c), friction angle (ϕ), deformability modulus (E) and Poisson coefficient (ν) from Table 3 were used. For materials with a fragile behavior (Parabolic Model), such as concrete and injection mortar, values of resistance to compression, traction (Rt), specific weight, strain modulus and Poisson coefficient were attributed.

7 NUMERICAL RESULTS

Fig. 4 shows the results obtained from the pile load test compared to those from the numerical analyses. In the numerical modeling with the parameters obtained at the DMT parameters, a maximum load of 960 kN was obtained for a total settlement of 70.9 mm, whereas in the analyses with parameters obtained via semi-empirical correlations, the simulation reached the maximum value of 1,100 kN and total settlement of 75.90 mm.

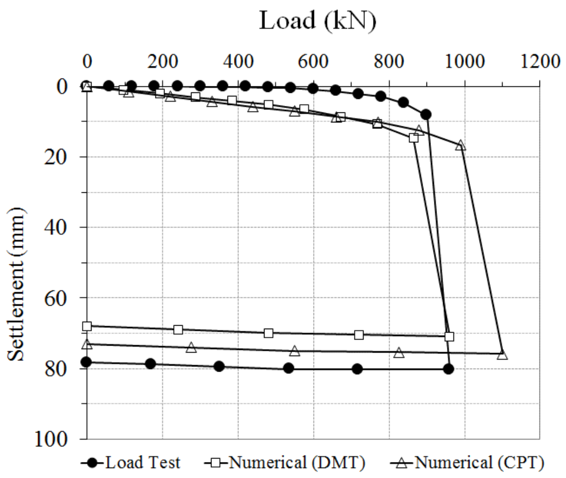

Fig. 4. Load vs. settlement curves: numerical and experimental

Fig. 5 compares the results for the load distribution along the pile length. The same figure also shows that, in the experimental test, the portion of the tip was 6 % and 16 % for half of the maximum load (½ Q_{max}) and for the maximum test load (Q_{max}), respectively. However, the numerical results with DMT parameters led to participation of the tip of 6% in comparison to ½ Q_{max} and 10 % for Q_{max}. For the numerical analysis with parameters estimated by empirical correlations through CPT data, the portion of the tip of 7 % and 13 % was obtained for ½ Q_{max} and Q_{max}, respectively.

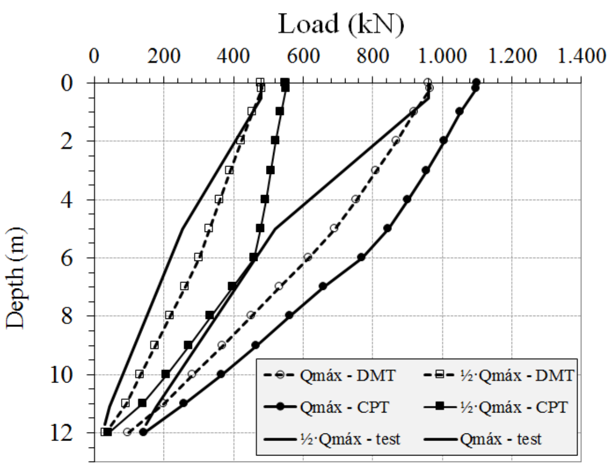

Fig. 5. Load distribution: numerical and experimental.

8 CONCLUSIONS

- The use of CPT and DMT tests proved to be appropriate to get soil parameters in order to use the numerical analysis tool, but attention must be paid to the correlation to be used, since the results vary largely.
- The load vs. settlement curves obtained did not display the same rigidity up to the rupture as the rigidity provided by the load test. This difference may be associated to the characteristics of the local soil, which is non-saturated and lateritic.
- The tip loads fell within an appropriate interval when compared to the experimental results. However, it was noted that the lateral load transfer took place in a differentiated manner, which can be once more due to the characteristics of the local soil.
- The numerical tool used proved to be appropriate for analysis and understanding of the behavior of a pile when submitted to static axial load, when using soil parameters obtained from correlations of CPT and DMT tests.

9 REFERENCES

Albuquerque, P. J. R. 2001. “Estacas escavadas, hélice contínua e ômega: Estudo do comportamento à compressão em solo residual de diabásio, através de provas de carga instrumentadas em profundidade”. *Tese de Doutorado*, Escola Politécnica/USP, São Paulo. 297p.

Albuquerque, P.J.R.; Massad, F.; Fonseca, A.V.; Carvalho, D.; Santos, J.; Esteves, E. 2011. “Effects of the construction method on pile performance: evaluation by instrumentation”. Part 1: experimental site at the State University of Campinas. *Soils & Rocks*, v. 34:1, p. 35-50.

Barata, F. E. 1986. “Recalques de Edifícios sobre Fundações Diretas em Terrenos de Compressibilidade Rápida e com a Consideração da Rigidez da Estrutura”. *Tese de Concurso* para Professor Titular do Departamento de Construção Civil, Escola de Engenharia da UFRJ, Rio de Janeiro.

Barnes, G. 2000. “Soil Mechanics: Principles and Practice”, Palgrave Macmillan.

Budhu, M. 2006. “Soil Mechanics and Foundations”, John Wiley & Sons Inc.

Carvalho, D., Albuquerque, P.J.R, Giacheti, H. “Campo experimental para estudos de Mecânica de Solos e Fundações em Campinas-SP”. *SEFE IV*. São Paulo. 2000.

Chen, W.F. 2002. “The Civil Engineering Handbook”, CRC Press Inc.,U.S..

Diaz-Segura, E. G. 2013. “Método simplificado para la estimación de la carga última de pilotes sometidos a carga vertical axial en arenas”. *Revista Dyna*, 179, 109-115.

Fontaine, E. B. 2004. “Utilização de ensaios geotécnicos especiais de campo, cone elétrico e pressiométrico, em solos do interior do estado de São Paulo”. *Tese de Doutorado*, Feagri, Unicamp, Campinas, 256p.

Garcia, J. R., Albuquerque, P. J. R., Perez, N. B. M. 2013. “Análise experimental e numérica de estaca pré-moldada embutida em solo de diabásio”. In: VII Seminário de Engenharia Geotécnica do Rio Grande do Sul, In: *VII SEMINÁRIO DE ENGENHARIA GEOTÉCNICA DO RIO GRANDE DO SUL. Santa Maria*: ABMS/RS, 1, 237-244.

Garcia, J. R.; Albuquerque, P. J. R. “Assessment of soil parameters via CPT in numerical modeling of a continuous flight auger pile”. In: *3rd International Symposium on Cone Penetration Testing*, 2014, Las Vegas, Nevada, USA. CPT Aplications. Las Vegas: OmniPress, 2014. v. 1. p. 781-787.

Kulhawy, F. H. & Mayne, P. H. 1990. “Manual on Estimating Soil Properties for Foundation Design”. *Electric Power Research Institute*, EPRI, 1990.

Ladd et al., C.C., Foot, R., Ishihara, K., Poulos, H. G. and Schlosser, F. 1977: “Stress-deformation and strength characteristics” *Proc. 9th ICSMFE, Vol. 2*, State-of-the-Art-Paper, pp. 421-494.

Lunne, T.; Lacasse, S.; Rad, N. S. (1989) “SPT, CPT, pressuremeter testing and recent developments on in situ testing of soils”. General Report Session. *Proceedings of the 12th Int. Conf. Soil Mech. Found. Eng.*, p. 2339-2404.

Lunne, T.; Robertson, P.K.; Powell, J. J. M. 1997. “Cone penetration testing in geotechnical practice”. London (England), Blackie Academic & Professional, Chapman &Hall., 1st ed., 312 p.

Marchetti, S. (2001). “The flat dilatometer test (DMT) and its applications”. Available at http://www.geotech.se/Dilatometer (DMT)_guideline.html. Accessed on Dec. 15, 2014.

Mersi, G. (1975). Discussion on “New design procedure for stability of soft clays”. *J. Geotech. Eng. Div., ASCE*, v.101, p.409-412.

Robertson P.K. and Campanella R.G. (1986). “Estimating Liquefaction Potential of Sands Using the Flat Plate Dilatometer”. *ASTM Geot. Testing Jnl*, Mar, 38-40.

TC16 (2001) "The DMT in Soil Investigations", A Report by the ISSMGE Committee TC16, 41 pp. (pdf format or doc format 1 MB).

Trofimenkov, J. G. 1974. “Penetration Test in URSS-State-of-the-Art-Report.” In: *EUROPEAN SYMPOSIUM ON PENETRATION TESTING, 1, 1974*, STOCKHOLM. Proceedings… Stockholm National Swedish Building Research, p. 147-154.

Vargas. M. “Introdução à mecânica dos solos”. Editora McGraw-Hill do Brasil Ltda. São Paulo. 1978.