Evaluation of the use of Different Approaches for Interpretation of Pressuremeter Test Results in Wind Turbine Foundation Projects in Northeast Brazil

Alfran Sampaio Moura
DSc, Professor at the Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail: alfransampaio@gmail.com

Marcos Fábio Porto de Aguiar
DSc, Federal Institute of Ceara (IFCE) / University of Fortaleza, /CE, Brazil
e-mail: marcosfpa@hotmail.com

Fernando Feitosa Monteiro
Eng, Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail: engffmonteiro@gmail.com

Alexandre Miranda Mont'Alverne
DSc, Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail: montalverne@ufc.br

ABSTRACT

This article presents a study that aims to evaluate the influence of adopting different approaches to interpretation of the data from pressuremeter tests (PMTs) in wind turbine foundation projects from a study conducted in sandy soil of the coastal dunes of Northeastern Brazil. For this study, PMTs were performed as well as standard penetration tests (SPTs), with energy and torque measurement. Data interpretation of pressuremeter tests was done in a traditional (ASTM, 1987) and rational way (CUNHA, 1994). The variations obtained in determinations of soil parameters by using the two approaches were quite expressive. In determining the bearing capacity of the shallow foundation of a wind turbine located on the coast of Northeast Brazil, there was no possibility of evaluating influence of the interpretive approach as the determination method used was based only on the parameters provided by the traditional approach. The determination of the natural frequency of vibration (f_n) from the use of soil parameters obtained by the rational approach was limited by the fact that the methods utilized to determine the maximum shear modulus (G_{max}) mostly only required parameters obtained with the use of the traditional approach. The evaluation of shear deformation of each specific problem. In the case of wind turbines, one must use the soil maximum shear modulus (G_{max}). When it is impossible to conduct seismic tests, the pressuremeter is a particularly useful tool for estimation of the frequencies of vibration of wind turbine foundations.

KEYWORDS: Pressuremeter Tests; Soil Parameters; Shallow Foundation; Wind Turbine.

INTRODUCTION

The most common approach to measure shear strength of residual soils is through field and laboratory tests. Commonly in-situ tests consist of standard penetration tests (SPT), cone penetration tests (CPT or CPTU), vane shear tests, and pressuremeter tests [1]. Meanwhile, the pressuremeter is a powerful tool for geotechnical investigation which is being increasingly used for understanding soil-bearing capacity in shallow foundations of wind turbines. In this context, we highlight the large number of wind farms that are currently being installed in Northeast Brazil due to the favourable physical characteristics of the region.

The study site is located in São Gonçalo do Amarante, which is limited to the east by the Fortaleza metropolitan area and lies about 60 km from the capital city of the state, Fortaleza. The site under consideration is located over a large strip of dunes lying on the “Barreiras” formation.

TESTS PERFORMED

The study of the sandy soil dunes’ behaviour was carried out from a programme of laboratory tests and a campaign of field tests. The programme of laboratory tests consisted of a battery of characterization tests and a battery of special tests. The characterization was performed using grain size analysis, specific gravity tests, Atterberg limits, and determination of the minimum and maximum index void ratios. The special tests performed were consolidation and direct shear tests [2]. Regarding the field tests, a campaign of standard penetration tests (SPTs) was carried out with measurement of the energy and torque, and borehole pressuremeter tests (PMT) were conducted using Ménard-type equipment positioned in the vicinity of a wind turbine in one of the wind power stations in the region, according to the illustration in Figure. 1.

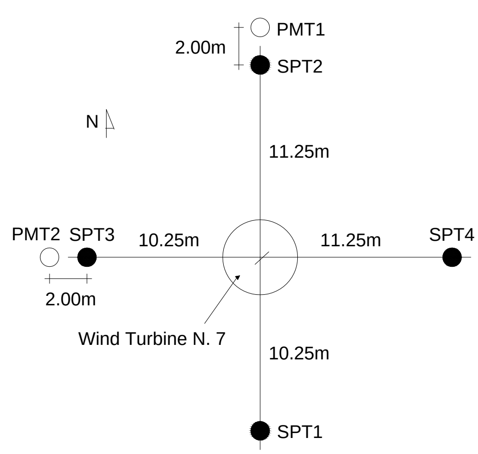

Figure 1: Location of field tests performed

FIELD TEST RESULTS

Standard Penetration Tests (SPT)

The values of standard penetration resistance (N-value or NSPT) obtained along the soil profile studied were quite high and increased with depth, ranging from 20 to 70 blows. The studied soil consists of fine dune sand which is compact to very compact. In two of the SPTs, torque (T) measurements were taken. The ratio of the resulting value of (T/NSPT) of the studied soil ranged from 0.90 to 1.07 on average. Additionally, energy measurements were performed using an that the efficiency of the system used is of the order of 64%.

Results of Pressuremeter Tests (PMTs)

The equipment used for pressuremeter testing is of the Ménard type (model GC, Apageo) consisting of a pressure source, a pressure and volume unit (CPV), and a cylindrical load cell 45 cm long and 5.9 cm in diameter. The connection between the cylindrical load cell and the control unit is made using a flexible coaxial pipe with an outer diameter of 11 mm and length of 25 m. A total of 14 pressuremeter tests were performed on two holes up to the maximum depth of 7 m using an auger-type spade with a diameter of 60 to 70 mm.

After completing the necessary corrections for pressure, volume, and hydrostatic pressure, curves of pressure versus volume were created. For the same depths of the holes PMT1 and PMT2, it was found that the curves practically coincided, preliminarily indicating the high uniformity of the ground studied.

Using the recommendations of American standards (ASTM, 1987) [3], profiles of horizontal earth pressure at rest (σ_{ho}), limit pressure (p_l), effective limit pressure (p_l*), coefficient of earth pressure at rest (K_o), pressuremeter deformation modulus (E_i), pressuremeter shear modulus (G_i), and pressuremeter unload–reload shear modulus (G_{ur}) were determined. Table 1 shows the results of the parameters determined from the pressuremeter test results of hole PMT1 made by the traditional method.

Table 1: Summary of parameters obtained for the test hole PMT1 by the traditional method (ASTM, 1987).

Parameter	Depth (m)						
	1	2	3	4	5	6	7
σ_{ho} (kPa)	50.00	44.00	70.00	75.00	50.00	50.00	120.00
E_i (MPa)	4.90	12.19	15.18	15.84	20.90	19.68	19.23
G_i (MPa)	1.84	4.58	5.70	5.96	7.86	7.40	7.23
p_l (MPa)	0.74	1.89	2.56	2.72	3.30	–	3.40
p_l* (MPa)	0.69	1.84	2.49	2.64	3.25	–	3.28
E_{ur} (MPa)	–	135.40	–	169.18	–	–	184.90
G_{ur} (MPa)	–	50.90	–	63.60	–	–	69.51
K_o	2.95	1.30	1.38	1.11	0.59	0.49	1.01

The estimated values of horizontal stress at rest (σ_{ho}) varied from 44 to 120 kPa in hole PMT1 and from 47 to 145 kPa in hole PMT2. The estimated values of the coefficient of earth pressure at rest (K_o) ranged from 0.49 to 2.95 for hole PMT1 and 1.18 to 2.77 for hole PMT2. The estimates of effective pressure limit (p_l*) ranged from 0.69 to 3.28 MPa in hole PMT1 and

1.30 to 3.56 MPa in hole PMT2 and had a progressively increasing trend with depth in both holes. The values of shear deformation modulus (G_i) ranged from 1.84 to 7.86 MPa in hole PMT1 and 2.86 to 8.79 MPa in hole PMT2. It was also observed that the G_i values of holes PMT1 and PMT2 were very close, confirming the trend of homogeneous behaviour already preliminarily perceived through the stratigraphic profiles and standard penetration tests (SPT) performed.

To obtain the pressuremeter parameters in the rational way, a curve fitting technique was applied Cunha (1994) [4], in which the experimental curve provided by the test was compared with a theoretical curve generated with the use of cylindrical cavity expansion theory.

The analysis consisted of varying (ϕ, ϕ_{cv}, σ_{ho}, G_e, and G_{pl}) to obtain an agreement between the theoretical and field test curves. The Poisson ratio (ν) was considered to be constant and equal to 0.33 and the friction angle was set at 40°. The friction angle was considered at constant volume (ϕ_{cv}) 5° below ϕ and the shear plastic deformation modulus (G_{pl}), twice the corresponding elastic shear modulus (G_e). The soil cohesion was considered to be equal to 5 kPa. A good fit was obtained from data obtained experimentally in hole PMT1 corresponding to the test performed at a depth of 1 m. Table 2 shows the results of the parameters determined from the results of borehole pressuremeter tests in hole PMT 1 using the rational method.

Table 2: Summary of the parameters obtained for hole PMT1 by the borehole pressuremeter using the rational method (CUNHA, 1994).

Parameter	Depth (m)						
	1	2	3	4	5	6	7
σ_{ho} (kPa)	50.00	50.00	80.00	100.00	105.00	80.00	120.00
G_e (MPa)	11.00	70.00	92.00	85.00	110.00	–	105.00
ϕ (°)	40	40	40	40	40	–	40
E_p (MPa)	29.26	186.20	244.72	226.10	292.60	–	279.30
K_o	2.95	1.47	1.57	1.47	1.24	0.79	1.01

In order to verify the differences between the parameters obtained using the traditional method and the adjusted curve technique, graphs of the parameters K_o, K_i, G_e, E_i, and E_e versus depth were made for boreholes PMT1 and PMT2 (Figure 2 and 3).

In Figure 2a, it is observable that the value of K_0 tends to converge to a value near the depth unit of 7 m. However, the variations obtained in the parameters using the traditional method or using the curve fitting technique are considerable throughout almost all of the depths tested. The smallest differences were obtained at 7 m and the largest at 1 m. It is believed that the difficulty in obtaining the horizontal earth pressures at rest (σ_{ho}) using the traditional method is responsible for the differences found.

In Figure 2b we observe differences of up to an order of magnitude between the shear deformation modulus obtained in the conventional way (G_i) and the rational way (G_e). Differences of this magnitude are attributed to the effects of disturbance of the soil present when obtaining the referred parameter using the traditional and rational forms, and are considered using cavity expansion theory.

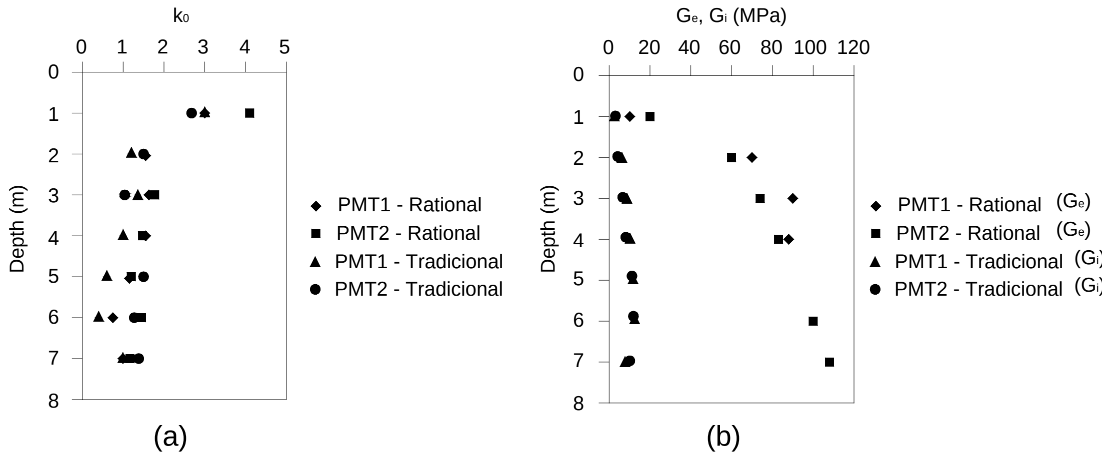

Figure 2: Comparison between the parameter values obtained using the traditional method and the rational method: (a) parameter K_0, (b) parameters G_e and G_i.

Figure 3a shows the comparison between the elastic shear modulus (G_e) obtained using the rational form and the unload–reload shear modulus (G_{ur}) obtained in the traditional way for the drilling depth of PMT1. Figure 3b shows the relation of G_e/C_{ur} with depth.

In Figure 3a we observe that the elastic shear modulus (G_e) is of the same order of magnitude as the unload–reload shear modulus (G_{ur}), with values which reach fifty percent higher. According to previous reports, the elastic shear modulus (G_e) obtained by the rational form reduces the effect of the disturbance of the soil during the test. In this way, as expected, values near those of the unload–reload shear modulus (G_{ur}), obtained using the traditional form. On the other hand, the values obtained for Ge, which were superior to those for Gur, are attributed to the poor unload–reload path obtained in the tests, the simplification of the theory methodology, and the various levels of deformation. Figure 3b shows that the relation G_e/G_{ur} for the drilling hole PMT1 varied from 1.34 to 1.51, presenting an average value of 1.41. For the drilling hole PMT2, the relation Ge/Gur reached values of up to 2.96.

Cunha (1996) compared the elastic shear modulus (G_e) of a sandy soil with the unload–reload shear modulus (G_{ur}), obtaining a value of 1.3 for the relation G_e/G_{ur}. The discrepancies found were attributed to the different deformation amplitudes imposed by the unloading–reloading cycle [5].

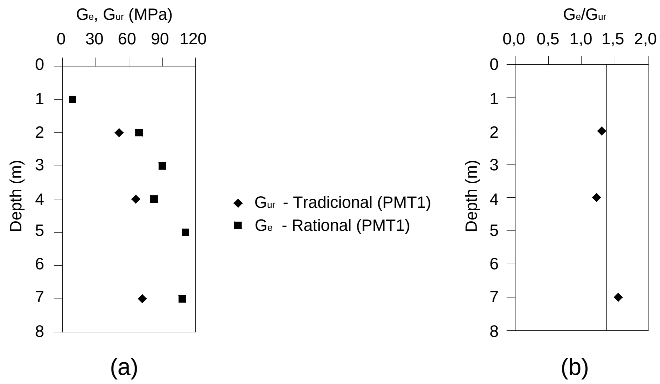

Figure 3: A comparison between the values of the parameters obtained using the traditional method and the rational method along the depth of the drilled borehole PMT1: (a) elastic shear modulus (G_e) and unload–reload shear modulus (G_{ur}); (b) the relation G_e/G_{ur}.

ESTIMATES AND COMPARISONS

In order to evaluate the effect of adopting different methods of interpretation of the data of pressuremeter tests in wind turbine foundation projects, estimates of the bearing capacity of the settlements and the frequency of vibration of the foundation of a wind turbine located near to the test locations were made.

It is important to report information regarding the geometric characteristics of the previously mentioned wind turbine. The wind turbine presents a nominal power of 500 kW, a rotor diameter of 4.2 m, and an axle height of 46.2 m and has an active control path angle such that the propellers pass in front of the tower in a clockwise rotation. Each wind turbine has three propellers 18.9 m long and weighing 13 kN. The propellers are made of fibreglass reinforced with epoxy. The generator has a horizontal axle and weighs 136 kN. The square shallow foundation is made of reinforced concrete with sides of 9 m, a height of 1.5 m, and a depth of 1.5 m, and each foundation weighs 3,038 kN (303.8 tf). Adding to the weight of the superstructure gave a total estimated weight of 3700 kN (370 tf).

Estimates of Allowable Stress for the Soil

According to the pressuremeter tests, the bearing capacity of the foundation soil was obtained such that [6]:

$$q_{rup} = k_p . p_{le}^* + \sigma_{vo} \tag{1}$$

where q_{rup} is the rupture stress of the soil, K_p = 1.3 is a factor of bearing capacity obtained by cupping on the basis of dependence on the shallow depth and width H_c/B and the type of soil [7].

σ_{v0} = 25.5 kPa is the total vertical stress at the foundation level and P^*_{le} = 2381 kPa is the average equivalent effective pressure limit of the soil in the zone influenced by the foundation. Wherein:

$$p_l^* = \left(p_{l1}^* p_{l2}^* . p_{l3}^* .... p_{\ln}^* \right)^{1/n} \tag{2}$$

where P^*_{ln} are the average effective pressure limits between the drilling holes PMT1 and PMT2 found in the zone influenced by the foundation.

Considering a security coefficient equal to 3 and using average values of the limit pressure along the depth, an allowable stress of 1040 kPa was estimated from the PMT. Comparing this value with estimates made from the two semi-empirical processes based on the SPT (TERZAGHI AND PECK, 1967) [8] and (MEYERHOF, 1965) [9], we can verify that the estimated value obtained from the PMT is about two times higher (Table 3).

Table 3: Comparison of the estimates of the allowable stress of the soil made based on the SPT and the PMT.

Method	Type of data used	σ_{adm} (kPa)
Terzaghi and Peck (1967)	SPT	506
Meyerhof (1965)	SPT	410
Ménard (1975)	PMT	1040

For this given analysis, it is relevant to note that there was no way to evaluate the influence of the type of interpretation method, since the determination of the bearing capacity is affected by the pressure limit (Pl) wich is determined by the traditional method Ménard (1975) [6].

Estimates of the Settlements

The prediction of settlements, according to the results of the PMTs, was done using the following expression [10]:

$$s = \frac{2q'.B'}{9E_d}.\left(\lambda_d . \frac{B}{B'} \right)^{\alpha_p} + \frac{\lambda_c .q'.B}{9.E_c}.\alpha \tag{3}$$

where q'= 199.9 kPa is the applied liquid stress, E_c and E_d are the pressiometer module in the zones of the spherical tensor and deviator, respectively, $\lambda_c = 1.10$ and $\lambda_d = 1.12$ are the factors for the spherical form and deviators, $\alpha_p = 1/3$ is the rheological factor, and B = 9 m and B' = 0.60 m are the widths of the foundation and the reference.

Utilizing the pressiometer module (E_c = 15482 kPa and E_d = 15482 kPa) obtained by the traditional method, a total settlement (S) of 9.1 mm was estimated. In this case, the settlement was estimated from the average of these pressiometer module of the drilling holes PMT1 and PMT2.

The parameters determined by the rational interpretation method led to an estimated settlement of only 0.7 mm. The reduced value obtained is attributed to the fact that Equation 3 uses values of the initial pressiometer modulus (E_p) which do not correspond to the elastic pressiometer modulus (E_c). According to the theory of elasticity, the shallow foundation settlement is given by:

$$s = q_a.B.\frac{1-\upsilon^2}{E}.I_s.I_d.I_h \qquad (4)$$

where q_a = 137.04 kPa is the applied average stress, B = 9 m is the dimension of the shallow foundation, υ = 0.33 is the Poisson's ratio, I_s = 0.99, Id = 1.0, and I_h = 1.0 are factors of form, depth, and layer thickness, respectively, and E is the elasticity modulus. From the theory of elasticity expression (Equation 4) associated with the value of the elastic pressiometer modulus (E_c) obtained by the rational interpretation approach, the settlement was estimated to be 7.4 mm, which is in good agreement with estimates done previously.

The settlements estimated using the proposals of Burland e Burbidge (1985) [11] and Schmertmann (1970) [12] were 5.9 and 7.5 mm, respectively. These two methods were carried out using the SPT data of the standard penetration test (SPT) in order to better evaluate the size order of the settlements estimated using pressiometer parameters. In Table 4 the estimates of the settlements obtained are compared.

Table 4: Comparison of estimated values for settlement of the wind turbine foundation

Reference	Method of obtainment	Settlement (mm)
Schmertmann (1970)	SPT	5.9
Burland and Burbidge (1985)	SPT	7.5
Ménard and Rousseaud (1962)	PMT (ASTM, 1987)	9.1
Ménard and Rousseaud (1962)	PMT (Cunha, 1994)	0.7
Theory of elasticity	PMT (Cunha, 1994)	7.4

Estimates of the Natural Frequencies of Vibration

The estimates of the natural frequencies of vibration of the foundations of the wind turbine analyzed were made using two methods that consider the soil as an elastic semi-space: the methods proposed by Lysmer and Richart (1966) [13] and by Nagendra and Sridharan (1981) [14].

The shear deformation modulus (G) of the soils decreases with increasing deformation level, and for very low values of shear deformation (γ), which is the case of the wind turbines, the secant modulus, G, becomes equal to the maximum modulus, G_{max}. According to the proposal of [15], the maximum deformation modulus can be determined by the expressions below:

$$G_{\max} = 138.p_l^{1,42} \qquad (5)$$

$$G_{\max} = 45.G_i \qquad (6)$$

where p_l is the pressure limit and G_i is the modulus of initial pressure shear deformation.
From Equation 5, proposed by Kalteziotis et al. (1990) [15], the determination of G_{max} is a function only of the pressure limit (Pl), which is determined only by the traditional method. In the same way, if the determination of G_{max} is affected by Equation 6, it can be observed that the

determination of G should be done using the traditional method of interpretation (ASTM, 1987) [3].

Another proposal for the determination of G_{max} for sand is that of Byrne et al. (1990) [16]. In this case, the determination of G_{max} is done using the function of the unload–reload shear modulus (G_{ur}), which can be estimated by the traditional form of interpretation [3] or, alternatively, using the elastic shear modulus (G_e), which can be obtained through the rational form.

Figure 4 shows that the range of variation of the estimates of natural frequency was wide, varying from 575 to 1991 RPM. The lowest values were estimated by the proposal of Byrne et al. (1990) [16] and the highest values by the proposal of Kalteziotis et al. (1990) [15].

Comparing the estimates of f_n made only using the proposal of Lysmer and Richart (1966) [13], it was observed that use of the soil parameters obtained by the rational interpretation form [4] was possible only when the method of determination of G_{max} proposed by Byrne et al (1990) [16] was used. In this case the variation of f_n was 13% higher than when G_{ur} was used in the same method. The determination of the natural frequency vibration (f_n) from the use of parameters of the soil obtained by the rational approach was limited by the fact that the great majority of the methods used to determine the maximum shear modulus (G_{max}) required parameters obtained only by using the traditional approach.

Comparing now the estimates of f_n determined by the methods of Lysmer and Richart (1966)[13] and Nagendra and Sridharan (1981) [14], for the same form of obtaining parameters and determination of G_{max}, we observed that the variations obtained were the same and were around 11.4%.

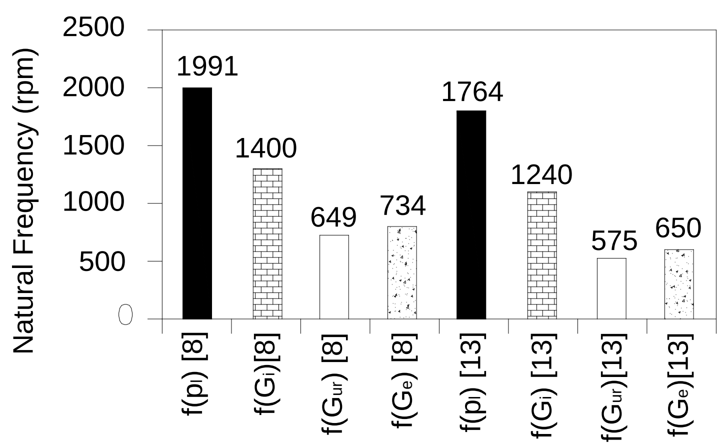

Figure 4: Comparison of the natural frequency vibration (f_n) of the foundations of the wind turbine.

It is important to mention that many factors affect the shear modulus of the soils (G), particularly the value of shear deformation imposed. In this way the evaluation of the shear deformation modulus should always be rendered compatible with the level of deformation of the analyzed situation.

CONCLUSION

This study allowed us to establish the following conclusions:

- The pressuremeter data are useful for calculating the estimates of the bearing capacity, settlements, amplitude, and vibration frequency of the wind turbines' superficial foundations.

- The variations obtained by the determinations of the soil parameters using the traditional and rational approaches were very expressive.

- In determining the foundation's bearing capacity in the analysis, there was no way to assess the influence of the type of interpretation method because, according to the proposal of Ménard (1975), the determination of bearing capacity is affected by the pressure limit (P_i), which is determined only by the traditional method.

- The settlements determined by Ménard and Rousseaud (1962) using the method of rational interpretation led to very low estimates when compared to the rest of the estimates made.

- The use of the rational method of interpretation of the data for PMT gave estimates of settlements that were only coherent with the expression of the theory of elasticity (Equation 4) associated with the value of elastic shear modulus (G_e).

- The lowest values of natural frequency vibration (f_n) were estimated by the proposal of Byrne et al. (1990) and the highest values by the proposals of Kalteziotis et al. (1990).

- The natural frequency vibration (f_n) found by using parameters of the soil obtained by the rational approach was limited by the fact that the methods used to determine the maximum shear modulus (G_{max}) used, for the most part, require parameters obtained only by the traditional method.

- The variations found upon estimating the natural frequency vibration (f_n) by the methods of Lysmer and Richart (1966) and Nagendra and Sridharan (1981), using the same approach to obtain the parameters of the soil and to determine G_{max}, were practically the same.

- When it is not possible to conduct seismic tests, the pressiometer is a viable alternative for estimating the natural frequency vibration (f_n) of wind turbine foundations.

- An evaluation of the shear deformation modulus should be compatible with the level of deformation for each specific problem. For cases such as foundations of rotating machines, which is the case of wind turbines, the maximum shear modulus (G_{max}) should be used.

ACKNOWLEDGEMENTS

The authors of this paper thank, CAPES for the financial support, the Federal University of Ceará (UFC) for the extremely relevant support; the Federal University of Paraiba (UFPB) for the use of the Ménard pressuremeter, and Professor Erinaldo Hilário Cavalcante, from the Federal University of Sergipe (UFSE), for field testing.

REFERENCES

[1] Rohaya Alias, Anuar Kasa, Mohd Raihan Taha (2014) “Effective Shear Strength Parameters for Remolded Granite Residual Soil in Direct Shear and Triaxial Tests” *Electronic Journal of Geotechnical Engineering*, 2014(19):4559-4569. Available at ejge.com

[2] Moura, A. S. (2007) Avaliação de Metodologias de Projeto de Fundações Superficiais de Aerogeradores Assentes em Areia de Duna. Tese de Doutorado, Departamento de Engenharia Civil, Universidade de Brasília, 323 pp.

[3] American Society for Testing of Materials (1987) ASTM D4719/87. Standard Test Method for Pressuremeter Testing in Soils. Annual Book of ASTM Standards, New York , vol. 04.08.

[4] Cunha, R. P. (1994) Interpretation of self-boring pressuremeter tests in sand. Ph.D. thesis, Department of Civil Engineering, The University of British Columbia, Vancouver.

[5] Cunha, R. P. (1996) A new cavity expansion model to simulate self-boring pressuremeter tests in sand. *Solos e Rochas*. São Paulo. vol. 19, no 1, pp. 15–27.

[6] Ménard, L. (1975) The Ménard Pressuremeter, Interpretation and Application of Pressuremeter Test Results to Foundation Design, no. 26.

[7] Briaud, J. L. (1992) The Pressuremeter, Trans Tech Publications, Rotterdam: A. A. Balkema, 322 p.

[8] Terzaghi, K.; Peck, R. B. (1967) Soil Mechanics in Engineering Practice, 2nd edition, John Wiley & Sons, New York.

[9] Meyerhof, G. G. (1965) Shallow foundations, JSMFD, ASCE, vol 91, no. SM2 (March), pp. 21–31.

[10] Ménard, L.; Rousseau, J. (1962) L’Evaluation dês Tassements – Tendences Nouvelles.

[11] Burland, J. B.; Burbidge, M. C. (1985) Settlements of foundations on sand and gravel, Proceedings of the Institution of Civil Engineers, part 1, vol. 78.

[12] Schmertman, J. H. (1970) Static cone to compute settlement over sand. Journal of the Soil Mechanics and Foundation Engineering, ASCE, vol. 96, pp. 1011–1043.

[13] Lysmer, J.; Richart, F. E. (1966) Dynamic response of footings to vertical loading. Journal of the Soil Mechanics and Foundations Division, Proc. ASCE, vol. 92, no. SM1, January, pp. 65-91.

[14] Nagendra, M. V.; Sridharan, A. (1981) Response of circular footings to vertical vibrations. Journal of the Geotechnical Engineering Division, Proc. ASCE, Technical Notes, vol. 107, no. GT7, July, pp. 989–995.

[15] Kalteziotis, N. A.; Tsiambaos. G.; Sabatakakis, N.; Zervogiannis, H. S. (1990) Prediction of dynamic parameters from pressuremeter and other in-situ tests. In: International Symposium on Pressuremeters, 3. Proceedings, British Geotechnical Society, pp. 391-400.

[16] Byrne, P. M.; Salgado, F. M.; Howie, J. A. (1990) Relationship between the unload shear modulus from pressuremeter tests and the maximum shear modulus for sand. 3rd ISPMA, Oxford Univ., pp. 231–241.

Editor's note.

This paper may be referred to, in other publications, as:

Moura, Alfran Sampaio; Marcos Fábio Porto de Aguiar, Fernando Feitosa Monteiro, Alexandre Miranda Mont'Alverne: "Evaluation of the use of Different Approaches for Interpretation of Pressuremeter Test Results in Wind Turbine Foundation Projects in Northeast Brazil" Electronic Journal of Geotechnical Engineering, 2016 (21.10), pp 3845-3856. Available at ejge.com.