Comparative study of several methods of axial design of piles located in a thick clay deposit: Pierre-de-Saurel wind farm case

Merouane Menaa & Noureddine Ghlamallah
Englobe Corp., Laval, Quebec, Canada

ABSTRACT
As part of a wind farm construction project, several methods of piles axial design were used to optimize pile costs. These methods include: Beta method (effective stress analyses) of the Canadian Foundation Engineering Manual, CFEM, 2013; Alpha method (total stress analyses) of the CFEM; a method based on piezocone tip resistances; "U.S. Army Corps of Engineers" method (effective stress analyses); and finally, a method based on the results from the pressuremeters. In order to confirm the results, dynamic loading and pull out tests were carried out on three test piles. The results of this study showed that the Beta method, recommended by the CFEM, is very sensitive to the choice of the piles design parameters. The piezocone-based method gives the closest resistance values to those measured by the dynamic loading test.

RÉSUMÉ
Dans le cadre d'un projet de construction d'un parc éolien, plusieurs méthodes de dimensionnement axial de pieux ont été utilisées afin d'optimiser les coûts relatifs aux pieux. Ces méthodes sont : méthode Béta (en contrainte effective) du Manuel canadien d'ingénierie des fondations, MCIF, 2013; méthode Alpha (en contrainte totale) du MCIF; méthode basée sur les résistances en pointe du piézocône, MCIF; méthode de « U.S. Army Corps of Engineers» (en contrainte effective), et finalement, méthode basée sur les résultats des pressiomètres. Afin de valider les résultats, des essais de chargement dynamique et d'arrachement ont été réalisés sur trois pieux tests. Les résultats de cette étude montrent que la méthode Béta, recommandée par le MCIF, est très sensible aux choix des paramètres de conception des pieux. La méthode basée sur le piézocône donne les valeurs de résistances les plus proches aux valeurs mesurées par l'essai de chargement dynamique.

1 INTRODUCTION

The methods used to determine the axial capacity of piles are mostly empirical. These methods are widely used in the design phase, particularly the effective stress method so-called the Beta method (CFEM 2013), given its simplicity of use. In geotechnical practices, it is recommended to use different methods for the design of piles. In the design phase, Engineers introduce a geotechnical resistance factor of 0.4 to the axial capacity calculated with these empirical methods in accordance with the various codes, namely, the CHBDC (2017) and the NBCC (2015). To confirm the axial capacity calculated with the empirical methods and increase the resistance factor, load tests during the design phase are recommended.

Indeed, the various codes require the application of a geotechnical resistance factor of 0.5 if dynamic loading tests are carried out during the design phase, representing an axial capacity gain of 25%. In case, static loading tests are carried out, a geotechnical resistance factor of 0.6 can be applied which represents a gain of 50% in axial capacity.

It is very important to note that these loading tests are not tests for quality control of the geotechnical resistance of the piles but a validation of the geotechnical pile design.

This technical paper presents the methods of pile design and the load test validation; used for the construction of 12 wind turbines at Pierre-de-Saurel in the municipality of Yamaska in Quebec. The geotechnical conditions of the site are characterized here by a silty clay deposit of 25 m thick while the depth of the bedrock is between 36 and 38 m. Optimization of the geotechnical design of the piles was an important economic issue for the realization of the project.

To optimize the pile design, five static analysis methods were used to calculate skin resistance and end bearing of a single pile driven to refusal. To validate the results obtained through these methods, dynamic loading and pull out tests were carried out on three test piles.

2 SUBSURFACE CONDITION

Pierre-de-Saurel wind farm project is in the municipality of Yamaska (Quebec). The Municipality is in the region of Montérégie located south of Montreal Island. Considering the scale of the project, a major investigation campaign was carried at the locations of the 12 future wind turbines. The following surveys were completed: 12 conventional boreholes, two undrained shear vane test profiles using a Nilcon brand field scissometer, three Cone Penetration Test (CPT) profiles using a piezocone brand Hogentogler and two pressuremeter profiles using a Menard type G-AM pressuremeter. This campaign allowed to determine the following stratigraphy from the soil surface: loose to medium silt, sand and clay in varying proportions (soil classification according to Unified Soil Classification

System - USCS: SM, ML to CL) averaging 5 m in thickness, a soft to stiff silty clay deposit with very high plasticity (CH) with an average thickness of 25 m, a 6 m thick till composed of sandy silt (SM probable) deposit of dense compactness, and finally, the bedrock encountered at depths between 36 m and 38 m. The level of groundwater was close to the surface and was measured in the boreholes at an average depth of about 1 m.
Figures 1 to 5 present the investigation results at the location of wind turbine no. 6 where dynamic loading and pull out tests were performed. Figure 1 shows the undrained shear strength (Su) profile measured with the Vane test and through the CPT test by the following correlation (Eq. 1):

$$S_u = \frac{(q_{t} - \sigma_{vo})}{N_k} \qquad [1]$$

Where q_t is the corrected tip resistance, σ_{vo} is the in situ total soil pressure and N_k a cone factor.

Figure 1 shows that both (Su) curves increase with depth. The consistency of the clay is medium until elevation 0 m where the clay became stiff. Figure 1 also shows that the values of (Su) measured with the vane test and those extrapolated from the CPT match very well except between elevations -8 m to -14 m. At these elevations the CPT test was able to show the presence of a transition between the silty clay deposit and the till deposit. The vane test did not detect this transition. The difference between the vane test and the CPT test regarding the presence of the transition between the silty clay deposit and the till deposit influences the results depending on the methods used. The piezocone tip resistance (qc) shown in Figure 2 has the same tendency as (Su).
Figures 3 and 4 present values of the pressuremeter modulus (E) and the limit pressure (P_L) measured at 2 boreholes located 2.5 m apart from each other. Figure 5 presents the values of (Su) measured with the vane test and those extrapolated from the pressuremeter test with the equation 2 bellow.

$$S_u = 0.67\, P_L^{*0.75} \qquad [2]$$

Where P_L^* is the net limit pressure described in section 3.1.5.

Although, the curves of (Su) from the vane test and from the pressuremeter presents an offset, it can be noticed that the curves have the same trend.
The interpretation of the results of the vane, CPT and pressuremeter tests allowed for the validation of the reliability of the in-situ tests as well as the values of the geotechnical parameters that will be used in pile design. The results highlight the presence of a transition between the silty clay deposit and the till deposit at elevations -8 m to -14 m. This transition was detected by the CPT and not by the vane test. The pressuremeter has detected a higher value at elevation -10 m; however, CPT is a continuous test and gives better interpretation and detection of the transition. The fact that the transition was detected by the CPT and not by other in-situ tests will later have an impact on the results depending on the methods used to determine pile axial capacity.

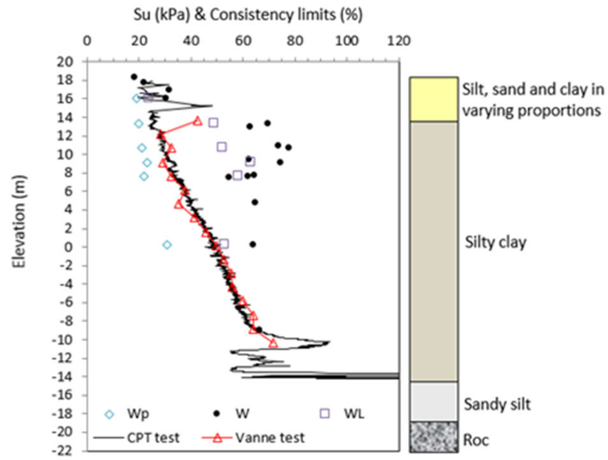

Figure 1. Undrained shear strength (Su) and consistency limits at the wind turbine no. 6 location

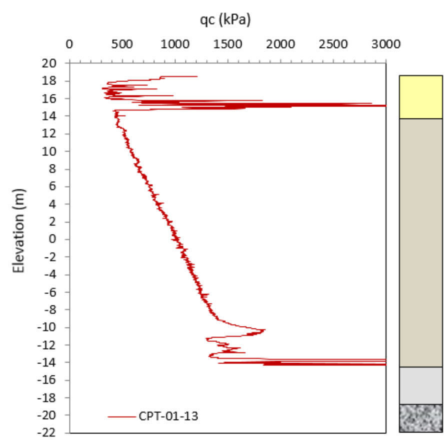

Figure 2. CPT tip resistance (qc) at the wind turbine no. 6 location

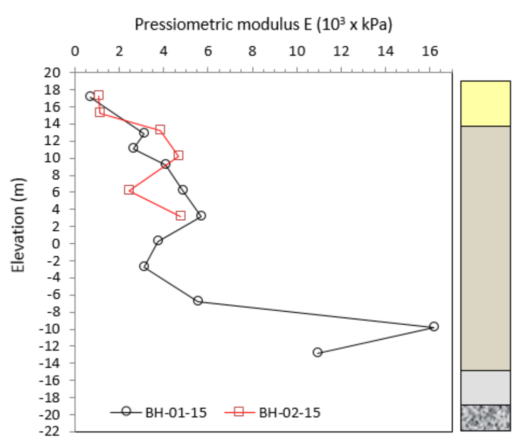

Figure 3. Pressiometric modulus (E) at the wind turbine no. 6 location

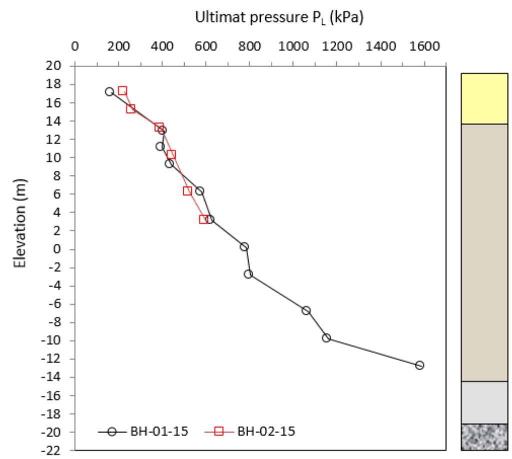

Figure 4. Limite Pressure (P_L) at the wind turbine no. 6 location

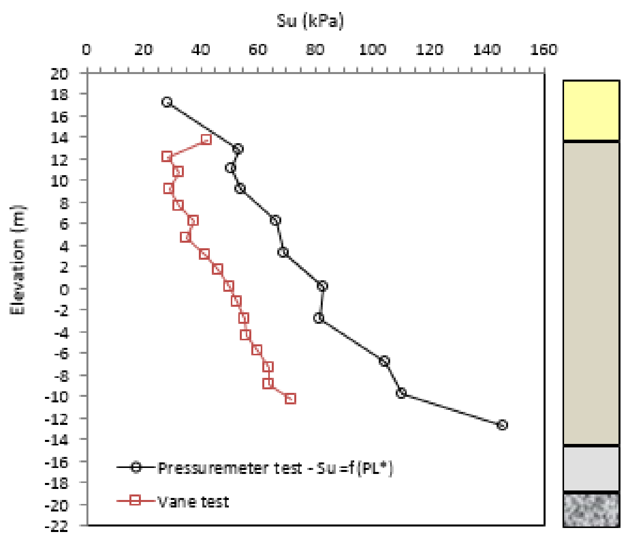

Figure 5. Su from van test Vs from pressuremeter test

3 DESIGN OF PILES

3.1 Presentation of the methods

Five methods were used to determine the skin resistance and the end bearing of a single pile. The ultimate geotechnical axial resistance of a single pile (R_u) is given by equation 3 below.

$$R_u = \left(\sum_{z=0}^{L} C \,.\, q_s \,.\, \Delta z \right) + A_t \,.\, q_t - W_p \qquad [3]$$

Where C is the pile circumference, Δz is the thickness of the layer, A_t is the area of the pile toe, W_P is the weight of the pile, and qs and qt are the unit skin friction and the end bearing, respectively.

3.1.1 Effective stress analyses (Beta method)

For granular soils, the unit skin friction and the end bearing are given by equations 4 and 5 below:

$$q_s = \sigma'_v \cdot K_S \cdot \tan \delta = \beta \cdot \sigma'_v \qquad [4]$$

$$q_t = N_t \cdot \sigma'_t \qquad [5]$$

Where σ'_v and σ'_t are the effective stress at depth z and at the base of the pile, respectively, k_s is the coefficient of lateral earth pressure and δ is the friction angle at the soil-pile interface. Fleming et al. (1992) provided a detailed discussion of the parameters ks and δ. In the MCIF (2013), ks and δ are combined into a single factor of friction resistance β whose values for granular soils are given in the first column of Table 1. N_t, represents the bearing resistance factor that depends on the soil composition, its particle size distribution, its compactness and other factors. For granular soils, the values given in the MCIF (2103) are given in the second column of Table 1.

The bracket in the values given in Table 1 is quite large. In the case of driven piles, it varies between 67 and 167% for the factor β and between 20 and 167% for the factor N_t. This method is therefore very sensitive to the choice of the design parameters of the piles. In addition, β and N_t do not represent conventional geotechnical parameters for which geotechnical engineers have developed a judgment based on theory, regional experience and investigation methods. In this context, the choice of β and N_t, that has a direct impact on the axial capacity of the pile depends essentially on the precautions that the engineer is willing to take.

For cohesive soils, Burland (1973) discussed the values of the coefficient β with values ranging from 0.25 to 0.32. Skempton (1951) and Ladanyi (1963) discussed the N_t factor values with values ranging from 3 to 10.

Table 1. β and N_t factors for driven piles (from MCIF. 2013)

Soil type	β	N_t
Silt	0.3 – 0.5	20 – 40
Loose sand	0.3 – 0.8	30 – 80
Medium sand	0.6 – 1.0	50 – 120
Dense sand	0.8 – 1.2	100 – 120
Gravel	0.8 – 1.5	150 – 300

3.1.2 Total stress analyses (Alpha method)

This method is applicable for driven piles in cohesive soils with undrained shear strength S_u <100 kPa. Although this method is not recommended because of correlations, not very reliable between S_u and unit skin resistance and end bearing, the method it is still presented in the MCIF (2013) and still used by engineers. For the calculation of the unit skin resistance, the equation 6 is used.

$$q_s = \alpha \cdot S_u \qquad [6]$$

Where α is a coefficient of adhesion varying between 0.5 and 1.0. End bearing is usually neglected in the case of driven piles in cohesive soils. Figure 6 shows the coefficient α as a function of the undrained shear strength S_u.

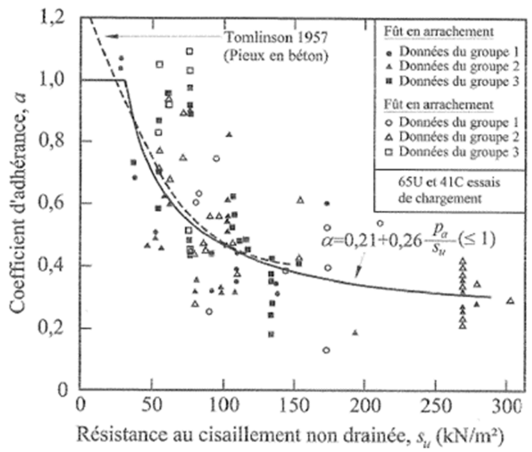

Figure 6. Coefficient of adhesion α as a function S_u (from MCIF. 2013)

3.1.3 U.S. Army Corps of Engineers method (USACE)

In its general formulation, the USACE (1991) effective stress method is similar to the Beta method. The major difference between the two methods lies in the approach for the determination of the parameters.

For granular soils, the unit skin resistance and the bearing resistance are given by equations 7 and 8 below:

$$q_s = \sigma'_v \cdot K \cdot \tan\delta \qquad [7]$$

$$q_t = N_q \cdot \sigma'_v \qquad [8]$$

Where σ'_v is the vertical effective stress, K is the coefficient of earth lateral pressure, δ is the friction angle at the soil-pile interface and N_q is the bearing capacity factor.

The angle of friction at the soil-pile interface δ is a parameter that depends on the internal friction angle of the soil ∅ and the pile material. The values given by the USCA are presented in Table 2. Friction angle δ values are also available in the Naval Facilities Engineering Command design manual (NAVFAC DM 7.02, 1986).

For the bearing capacity factor N_q, the MCIF suggests equation 9 below (Meyerhof, 1963). The values suggested by the USCA (1991) are quite identical.

$$N_q = e^{(\pi \tan\emptyset)} \tan^2 (45 + \emptyset / 2) \qquad [9]$$

The coefficient of earth lateral pressure, K (Kc in compression and Kt in tension) must be chosen carefully. The values given by the USCA (1991) and presented in Table 3 are given as general guidance. This value is not applicable to pre-drilled piles or installed with a vibrating hammer. On the other hand, displacement piles produce higher K values than non-displacement piles. To take the maximum value of the coefficient K, it is suggested to carry out loading tests and to specify the value of the coefficient K by a back calculation.

For cohesive soils, the formulation used by the USACE (1991) total stress method is identical to the Alpha method. Here too, the main difference between the two methods lies in the approach for determining the parameters. In the USACE method, the adhesion coefficient α is given by the values shown in Figure 7. For undrained shear strength Su, the USACE method recommends values determined by unconsolidated and undrained triaxial shear tests.

Table 2. Angle of friction δ (from USCA, 1991)

Pile material	δ
Steel	20 – 40
Concrete	30 – 80
Timber	50 – 120

Table 3. Coefficient of earth lateral pressure, K (from USCA, 1991)

Soil type	k_c	k_t
Sand	1.0 to 2.0	0.5 to 0.7
Silt	1.0	0.5 to 0.7
Clay	1.0	0.7 to 1.0

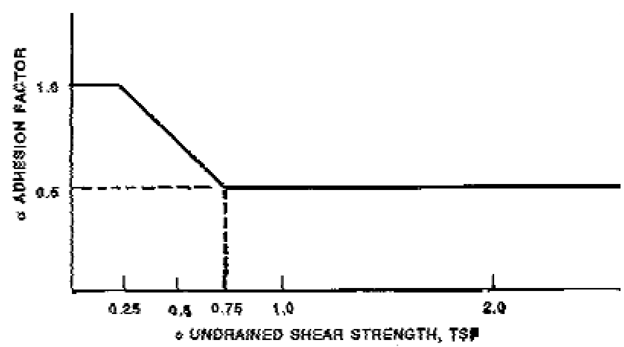

Figure 7. Coefficient of adhesion α as a function S_u (from USCA, 1991)

3.1.4 Method based on piezocone tip resistances

This method, described in the MCIF (2013), recommends the use of the tip resistance of the piezocone qc to evaluate the unit skin resistance and the end bearing of a single pile given by the equations 10 and 11 below:

$$q_s = \frac{q_c}{\alpha} \qquad [10]$$

$$q_t = k_c \cdot q_{ca} \qquad [11]$$

Where k_c is the bearing capacity factor based on the type of pile and type of soil in place and, α is the coefficient of friction and q_{ca} is the equivalent peak strength at the base of the pile determined by the Bustamante method &

Gianeselli (1982). Tables 4 and 5 present values of k_c and α.

This method is based on pile loading tests carried out in France (Bustamante & Gianeselli, 1982) and in North America (Robertson & al., 1988, Briaud & Tucker, 1988). The parameters kc and α, given in the MCIF (2013), consider the scale effect and the effect due to the installation method, between the piezocone and the pile.

Table 4. Bearing capacity factor Kc (from MCIF. 2013)

Soil type	q_c (MPa)	Kc factor for category IIB
Soft clay and mud	< 1	0.5
Moderately compact clay	1 to 5	0.45
Silt and loose sand	≤ 5	0.5
Compact to stiff clay and compact silt	> 5	0.55
Moderately compact sand dans and gravel	5 to 12	0.5

Table 5. Coefficient of friction α (from MCIF. 2013)

Soil type	q_c (MPa)	α Coefficient for category IIB
Soft clay and mud	< 1	30
Moderately compact clay	1 to 5	80
Silt and loose sand	≤ 5	120
Compact to stiff clay and compact silt	> 5	120
Moderately compact sand dans and gravel	5 to 12	200

3.1.5 Method based on the pressuremeter

In this method, the unit skin resistance q_s is calculated from a correlation with the net limit pressure P^*_L and the bearing resistance q_t is calculated with P^*_L and a bearing capacity factor k. The net limit pressure P^*_L is calculated from the pressuremeter limit pressure P_L (Menard, 1963) by Eq 12 below.

$$\mathrm{P}_{\mathrm{L}}^{*} = \mathrm{P}_{\mathrm{L}} - \mathrm{P}_{\mathrm{o}} \qquad [12]$$

Were P_o is the at rest total horizontal stress at the testing depth.

The bearing resistance is given by equation 13:

$$q_t = \mathrm{k} \, . \, \mathrm{P}_{\mathrm{Le}}^{*} + \mathrm{q}_{\mathrm{o}} \qquad [13]$$

Were q_o is the at rest total vertical stress at the pile toe depth and $\mathrm{P}_{\mathrm{Le}}^{*}$ is the equivalent net limit pressure. $\mathrm{P}_{\mathrm{Le}}^{*}$ is given by equation 14:

$$\mathrm{P}_{\mathrm{Le}}^{*} = \sqrt[n]{\mathrm{P}_{\mathrm{L1}}^{*} x \, \mathrm{P}_{\mathrm{L2}}^{*} \, x \ldots x \, \mathrm{P}_{\mathrm{Ln}}^{*}} \qquad [14]$$

Where $\mathrm{P}_{\mathrm{L1}}^{*} x \, \mathrm{P}_{\mathrm{L2}}^{*} \, x \ldots x \, \mathrm{P}_{\mathrm{Ln}}^{*}$ are the net limit pressures obtained from tests performed within the zone near the pile toe (+1.5B to -1.5B) with B being the diameter of the pile.

Several methods are proposed to determine the bearing capacity factor k. The one recommended by the Eurocode (2012) is presented in Table 6.

Table 6. Bearing capacity factor k (from fascicle 62, Eurocode 7, 2012)

Soil type	k
Clay/Silt	1.35
Sand/Gravel	3.1
Chalk	2.3
Altered and fragmented rock	2.3

The unit skin resistance q_s is calculated from a correlation with the net limit pressure P^*_L given by Eq 15.

$$q_s = \alpha \times (a \times P_L^* + b)\left(1 - e^{-c \times P_L^*}\right) \qquad [15]$$

Where α is a coefficient that considers the pile and soil type and a, b, c parameters that consider the type of soil. Tables 7 and 8 present values of the parameters α , b and c.

Table 7. α coefficient (from fascicle 62, Eurocode 7, 2012)

Soil type	α
Clay/Silt	0.8
Sand/Gravel	1.2
Chalk	0.4

Table 8. a, b, c parameters (from fascicle 62, Eurocode 7, 2012)

Soil type	a	b	c
Clay/Silt	0.003	0.04	3.5
Sand/Gravel	0.01	0.06	1.2
Chalk	0.007	0.07	1.3
Altered and fragmented rock	0.01	0.08	3

3.2 Parameters used in this project

The analyses were performed for a 406 mm internal diameter single pile driven to refusal on the silty sand till deposit.

Based on the in-situ test analysis, the parameters used in Beta method and USACE method are shown in Tables 9 and 10 while for Alpha method, the adhesion factor α was calculated using Figure 6. The values of α used in the Alpha method are presented in Figure 8. The parameters used in the CPT method are presented in Table 11. Table 12 presents the parameters used in the Pressuremeter method.

Table 9. Parameters used in the Beta method

Soil type	γ' kN/m³	β	N_t
Silt, sand and clay (top deposit)	9,0	0.3	30
Silty clay	6,5	0.3	3 to 5
Silt and sand (lower deposit)	10,0	1	62

Table 10. Parameters used in the USCA method

Soil type	γ' kN/m³	Φ'	S_u kPa	Nq
Silt, sand and clay (top deposit)	9,0	28	-	15
Silty clay	6,5	-	Figure 1	-
Silt and sand (lower deposit)	10,0	36°	-	38

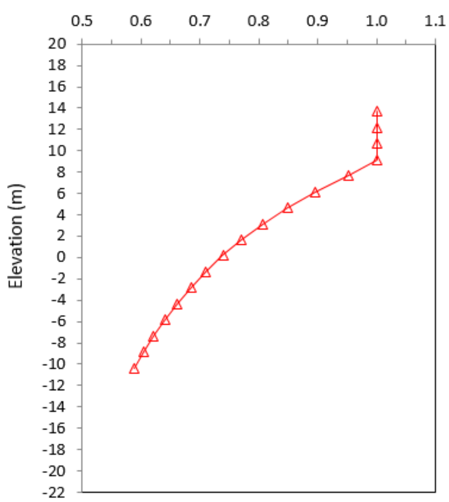

Figure 8. Values of adhesion factor α used in the Alpha method

Table 11. Parameters used in the CPT method

Soil type	γ' kN/m³	α	Kc
Silt, sand and clay (top deposit)	9,0	30 for (qc <1 MPa) 80 for (qc >1 MPa)	0.5 for (qc <1 MPa) 0.45 for (qc >1 Mpa)
Silty clay	6,5	30 for (qc <1 MPa) 80 for (qc >1 MPa)	0.5 for (qc <1 MPa) 0.45 for (qc >1 Mpa)
Silt and sand (lower deposit)	10,0	200	0.4

Table 12. Parameters used in the Pressuremeter method

Soil type	γ' kN/m³	k	α	a	b	c
Silt, sand and clay (top deposit)	9,0	1.35	0.8	0.003	0.04	3.5
Silty clay	6,5	1.35	0.8	0.003	0.04	3.5
Silt and sand (lower deposit)	10,0	3.1	1.2	0.01	0.06	1.2

4 PILE LOADING TESTS

A static pull-out test and a dynamic loading test were carried out on a 406 mm internal diameter test pile installed at the wind turbine at location no. 6. Table 13 shows the sequence of work for the realization of these tests.

Table 13. Sequence of work for the realization of the pile loading tests

Date[1]	Work description
8 to 14 Sep.[2]	Realization of 2 boreholes for the installation of pneumatic piezometers
25 Sep.	Driving three (3) test piles of 406 mm diameter
25 Sep. to 15 Nov.[3]	Follow-up of the dissipation of interstitial pressures following test piles driving
15 Nov.	Performing a static pull-out test
30 Nov.	Performing a dynamic loading test

[1]works completed in 2015 [2]September [3]November

4.1 Test piles installation

Three test piles were driven with a 4,536 kg (10,000 lb) hammer falling from a height of 1.53 to 1.83 m (5 'to 6'). The three piles were spaced 2.5 m in a single alignment. The refusal criterion was a pile vertical displacement of 12.7 mm (0.5 ') for 8 blows. Piles no. 1, 2 and 3 found their refusal in the dense till deposit at depths of 34.5 m (elevation -15.84 m), 34.6 m (elevation -15.94 m) and 34.98 m (elevation -16.32 m), respectively.

4.2 Static pull-out tests

The pull-out test was conducted according to ASTM D 3689/D3689M on pile no. 2 located in the center of the three test piles. As shown in Figure 9, the load in traction is applied by means of two hydraulic jacks placed directly on piles no. 1 and 3. The load is transmitted to pile no. 2 via a reaction beam welded to the latter. The deformations of the pile are recorded by means of two deflectometers placed on either side of pile no.2.

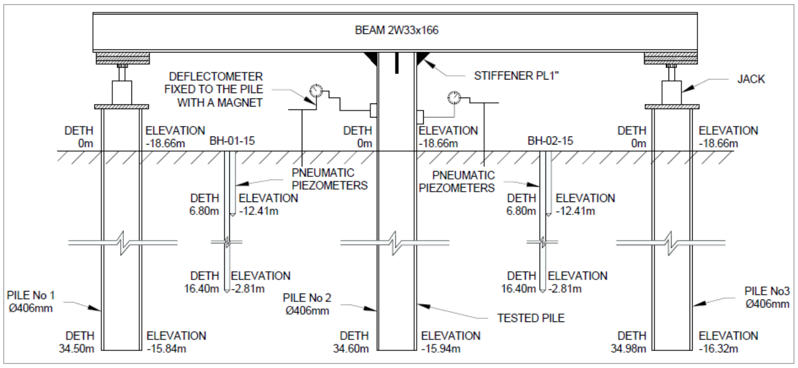

Figure 9. Pull-out test set- up

4.3 Dynamic load test

Dynamic load test was performed on pile no. 3 according to ASTM D 4945-12. Impact on the piles was applied using a 2 540 kg (5 600-lb) hammer falling from a drop height of 7.62 m.

5 RESULTS ANALYSIS

5.1 Axial capacity in compression

Figures 10 to 12 show the skin friction, the end bearing and the total capacity calculated with the five analysis methods. The results obtained from the loading test is also shown in these figures.

Calculations were made for a driven pile of 406 mm diameter with a pile tip at elevation -15 m in the dense silty sand till. Generally, a critical depth for tip resistance and skin friction is introduced in pile analyses and is considered between 10 to 20 times the diameter of the pile (USCA, 1991). The critical depth is characterized by the fact that the tip resistance and skin friction follow the effective stress to a certain depth called the critical depth. Below this depth, the resistances are constant and equal to the value calculated at the critical depth. In this analysis, the critical depth was first introduced in the Beta and USCA methods. The results regarding the end bearing were found very underestimated compared to the end bearing measured with the dynamic loading test and calculated through CPT test. The critical depth was then not considered in the analyses since it was found that the end bearing calculated with the Beta and USCA methods are more consistent as shown in Figure 11. Figure 11 shows that end bearing calculated with the CPT method is closer to the dynamic load test than the other methods that underestimate the end bearing.

Figure 10 shows the accumulated skin friction curves obtained from the five static methods and from the dynamic test. It can be noticed in Figure 10 that the Alpha and the USCA curves match together. Note that the main deposit is a silty clay and that the USCA methods can be considered in this context as a total stress method like the Alpha method. It was, therefore, expected that the Alpha and the USCA curves match together.

Except the two later methods, Figure 10 shows that all the methods diverge from each other and from the dynamic load test. It becomes, therefore, very important to carry out dynamic load test during the design phase to calibrate the analytical methods. Except the CPT method, the other methods overestimate the skin friction.

In can also be noticed on Figure 10 that the CPT method match with the dynamic load test until elevation -6 m after which the curves diverge. This elevation corresponds to the beginning of the transition between the clay and the till deposit that was detected by the CPT test. It is important to mention that the CPT curve on Figure 10 was calculated before the dynamic load test. The parameter used then were those suggested by the literature as shown in Table 11. Other analyses were carried out after the dynamic loading test to caliber the parameter of the CPT method. A coefficient of friction α of 30 was taken through the whole silt clay deposit. For the silty sand till, α coefficient and kc factor were taken as 120 and 0.6, respectively. Figure 13 shows that the total capacity obtained from the CPT test matches very well with the dynamic load test.

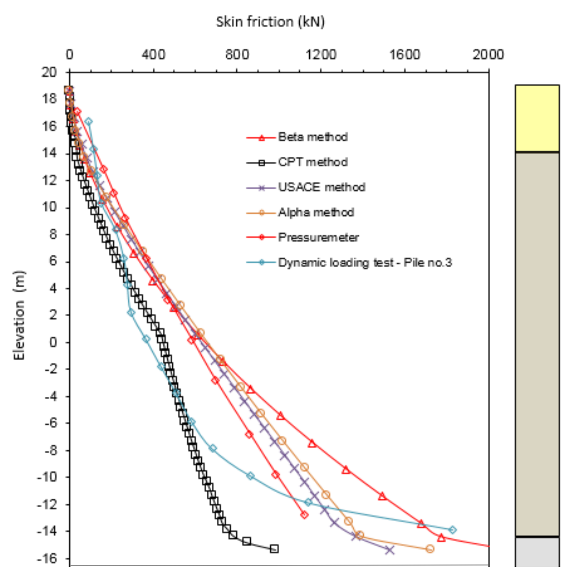

Figure 10 Skin friction capacity

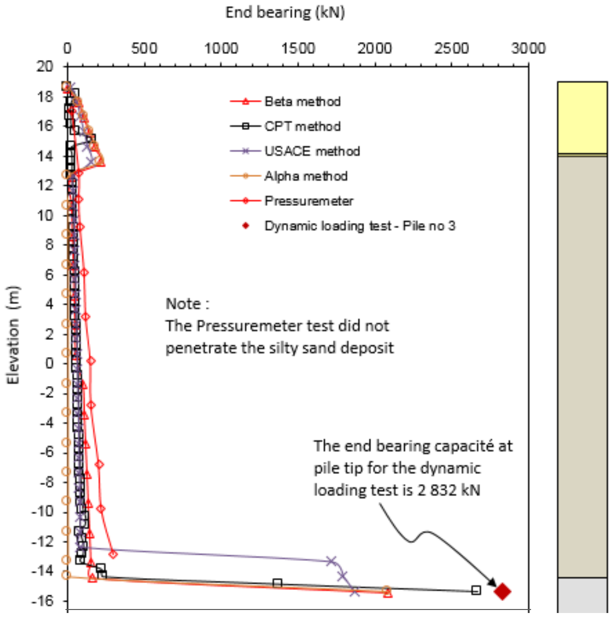

Figure 11. End bearing capacity

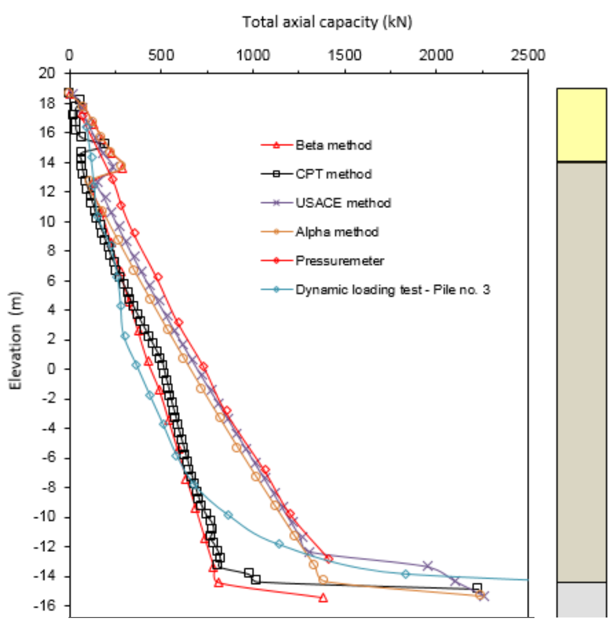

Figure 12 Total axial capacity

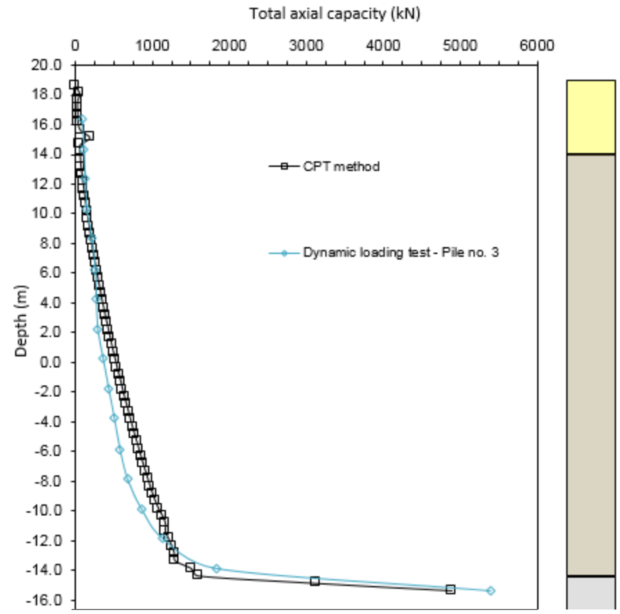

Figure 13 Total axial capacity calculated with CPT method and calibration of the parameters Vs dynamic loading test

5.2 Axial capacity in traction

When performing the pull-out test, the loading applied to the pile reached a maximum of 1 506 kN (338.67 kips). From this latter load, the subsequent loading stages tended to decrease during the test. As a result, tensile geotechnical resistance at the ultimate limit state was established at 1506 kN.

The accumulated skin friction measured through the dynamic load tests at pile no.1 and pile no. 3 are 2 120 kN and 2 563 kN, respectively. Thus, tensile geotechnical resistance at the ultimate limit state correspond to 60-70% of accumulated skin friction.

6 CONCLUSION

Five methods were used to determine unit skin friction and bearing resistance of a single pile driven to a dense silty sand deposit and through a thick silty clay deposit. These methods were Beta method (effective stress analyses); Alpha method (total stress analyses); a method based on piezocone tip resistances; "US Army Corps of Engineers" method (effective stress analyses); and a method based on the results from the pressuremeters. Dynamic loading and pull out tests were carried out on 406 mm diameter piles driven to refusal at elevation -15 m in the dense silty sand till. The results of the analyses showed that these methods are very sensitive to the parameter used and, therefore, depend essentially on the experience of the engineer and the precautions that are to be taken. In this context, it becomes very important to carry out a dynamic load test during the design phase to calibrate the analytical methods.

Moreover, the analyses show that the method based on the CPT gives the best fit to the dynamic load test provided that the method is calibrated. This can be explained by the fact that CPT is a continuous test and can detect the transition between the silty clay and the till deposits and also because the CPT probe reacts as a pile when pushed into soil. The other methods overestimate the skin friction and underestimate the end bearing.

The pull-out and dynamic loading tests showed that the tensile geotechnical resistance at the ultimate limit state corresponds to 60-70% of accumulated skin friction.

7 REFERENCES

American Society of Testing Materials, 2013. Standard Test Methods for Deep Foundations Under Static Axial Tensile Load, No. D 3689/D3689M*, ASTM, West Conshohocken, USA.*

American Society of Testing Materials , 2012. Standard test method for high-strain dynamic testing of deep foundations, No. D 4945-12*, ASTM, West Conshohocken, USA.*

Briaud, J. L., Tucker, L. M. 1988. Measured and Predicted Axial Response of 98 Piles, *Journal of Geotechnical Engineering, Vol. 114 (9), 984-1001.*

Briaud, J. L. 1992. *The Pressuremeter*, Balkema, Roterdam, Netherlands.

Burland, J. B. 1973. Shaft Friction of Piles in Clay - A Simple Fundamental Approach, *Ground Engineering*, 6(3), 30-42.

Canadian Geotechnical Society, 2006. *Canadian foundation engineering manual*, *4th ed*., Bi Tech Publishers Ltd*.,* Canada.

Canadian Standards Association, 2017. *Canadian Highway Bridge Design Code*, *CSA S6-14, 11th ed.,* CSA international, Rexdale, Ontario, Canada.

Fleming, W. G. K., Weltman, A. J., Randolnh, M. F. & Elson, W. K. 1992. *Piling Engineering*, 2nd ed., New York. Halsted Press.

Ladanyi, B.1963. Expansion of a Cavity in a Saturated Clay Medium, *Journal of the Soil Mechanics and Foundations* Division, 1963, 89(4):127-164.

Meyerhof, G.G. 1963. Some Recent Research on the Bearing Capacity of Foundations, *Canadian Geotechnical Journal*, 1(1):16-26.

National Building Code of Canada 2015. Associate Committee on the National Building Code. National Research Council of Canada.

U.S. Navy, 1986. *Foundations and Earth Structures, Design Manual No. 7.02,* Naval Facilities Engineering Command, Alexandria, USA.

Skempton, A.W. 1951, The Bearing Capacity of Clays, *Building Research Congress*, London, Div. I, 180-189.

Bustamante, M., and Gianeselli, L. 1982. Pile Bearing Capacity Predictions by Means of Static Penetrometer CPT, *Proceedings of the 2nd European Symposium on Penetration Testing (ESOPT II),* Amsterdam 2: 493-500.

Robertson, P.K., Campanella, R.G., Davies, M.G. and Sy, A. 1988. Axial Capacity of Driven piles in Detail Soils Using CPT, *International Symposium on Penetration Testing (ISOPTI)*, Orlando. 2:919-928.

Scientific and Technical Center for Construction, 2012. *Justification Geotechnical Works - Standard NF P 94-262*, National Application Standards of the Eurocode.

U.S. Army Corps of Engineers, 1991. *Design of Pile Foundation – Engineer Manual No. 1110-2-2906,* USACE, Washington, DC, USA.