Deep Foundation Design and Optimisation: A Case Study

Bintou Keïta, Raluca Opris & Michel Küntz
WSP Canada Inc., Montreal, Quebec, Canada

ABSTRACT
As part of a foundation design for a structure supporting a new fire protection system at the Norcan Oil Terminal, a geotechnical investigation campaign satisfying the needs of the project was carried out. Following the campaign that included different sampling and testing methods (drilling, field vane test and SCPTu), the interpretation of field and laboratory data results was undertaken to enable the design of deep foundations. The dimensioning of the piles supporting the fire protection structure was carried out and optimized based on the structural load cases and ground conditions. Pile load tests were also carried out and their results were used to validate the design. All the steps taken, validation of the calculations using in situ tests and the dimensioning methods used are presented in this paper.

RÉSUMÉ
Dans le cadre de la conception de fondations pour une structure supportant un nouveau système de protection incendie au terminal pétrolier Norcan, une campagne d'investigation géotechnique satisfaisant les besoins du projet a été réalisée. À la suite de la campagne qui incluait différentes méthodes d'échantillonnage et d'essais (forages, scissomètre et SCPTu), l'interprétation des données de terrain et de laboratoire a été complétée et a permis la conception de fondations profondes. Le dimensionnement des pieux supportant la structure du système de protection incendie a pu être effectué et optimisé basé sur les cas de chargement structuraux et les conditions souterraines. Des essais de chargement sur pieu ont également été réalisés et ont confirmé la conception. L'ensemble des démarches entreprises, la validation des calculs à l'aide des essais in situ et les méthodes de dimensionnement mises en œuvre sont présentés.

1 INTRODUCTION

A detailed investigation was carried out by WSP at the Norcan Oil Terminal in Montreal (Quebec) for the construction of a new fire protection system that required foundation design recommendations. Figure 1 shows a sketch of the proposed structure (pipe-rack) supporting the fire protection system, for which pile foundations were intended. Deep foundations were preferred by the client considering the space limitations as well as construction schedule and duration.

Figure 1. Proposed structure to be constructed

This paper summarizes the geotechnical investigation data interpretation and the design methodology using conventional methods and complementary 2D modelling of single piles. This paper also discusses the design methods proposed in the literature and their validity based on the CAse Pile Wave Analysis Program (CAPWAP®) results. The seismic and liquefaction concerns are not covered in this article.

2 GEOTECHNICAL INVESTIGATION

According to a previous geotechnical investigation carried out in other areas of the Terminal, the stratigraphy is relatively consistent across the Terminal. The layers below were encountered in the previous study, ordered by of increasing depth, from existing ground level:

- Fill
- Stiff silty clay
- Compact to dense sandy silt / silty sand
- Firm clayey silt / silt with some clay
- Dense silty sand (till)

The water table was identified to be located at approximately 5.60 m depth.

Based on these observations and the anticipated soil conditions, three types of *in situ* testing were performed: Standard Penetration Test (SPT), Field Vane Test (FVT), and Seismic Cone Penetration Tests (SCPTu) to validate the nature and properties of the existing subsoil and groundwater conditions in the proposed fire safety system area. The soundings were performed near the location of the pipe-rack foundations as per design plans.

2.1 Sampling & Standard Penetration Test (SPT)

A total of two (2) boreholes near the location of the proposed structure, identified as 18F02 and 18F03, were carried out to depths of 30.49 m and 20.57 m respectively in which Standard Penetration Testing (SPT) was performed. Soil samples were collected at regular intervals during borehole drilling, using the standard B caliber (51 mm diameter) split spoon samplers (SS). During sampling, standard penetration "N" values were measured according to the ASTM D1586 standard. The samples were classified according to the ASTM D2487-00 unified soil classification system standard. Figure 2 shows the interpreted stratigraphy and the variation of SPT N values.

Bedrock samples were recovered between depths of 23.62 m and 30.49 m in borehole 18F02 to validate the bedrock quality. The bedrock consists of black shale of poor quality at its surface becoming of excellent quality with depth.

Based on the information and data available from the client, numerous wells were installed in the Terminal for environmental purposes and for the groundwater depth measurement. For this reason, no additional wells were installed as part of this geotechnical investigation. The groundwater table estimated from the available data is shown in Figure 2.

2.2 Field Vane Test (FVT)

A field vane test was conducted between depths of 1.50 m and 6.50 m in the clayey deposit identified during the sampling activities. The purpose of this test is to determine the clay's intact and remolded undrained shear strengths, thus its sensitivity. The test was conducted alongside the borehole 18F02 according to the ASTM D2573 / D2573M standard and the results are shown in Figure 2.

2.3 Seismic Cone Penetration Test (SCPTu)

A total of three (3) SCPTu, identified as 19SCPTu-02 to 19SCPTu-04 were performed until refusal on dense soils near 22.5 m depth. These *in-situ* soundings were performed according to the ASTM D-5778-95 standard using 10 tons pushing rig and a 10 cm^2 cross section *Vertek* cone system. The latter allowed the real-time logging of the:

- Tip resistance (q_c);
- Sleeve friction (f_s);
- Pore pressure (u_2).

The shear waves velocities (V_s) were measured each meter for each SCPTu. The tests 19SCPTu-02 and 19SCPTu-03 were respectively located alongside the boreholes 18F02 and 18F03 for calibration and comparison purposes.

All the SCPTu data were interpreted using the CPeT-IT v.3.0 software by GeoLogismiki. The results indicate a consistent stratigraphy with the one obtained in the boreholes near the location of the proposed structure. The same soil layer sequence and similar thicknesses were found in the different soundings.

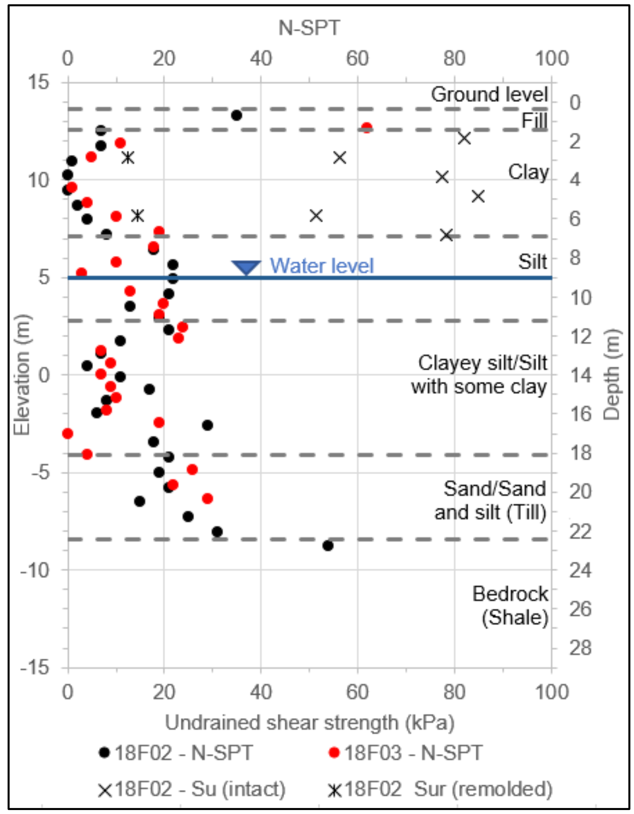

Figure 2. Stratigraphy, N values and FVT Results – Boreholes 18F02 and 18F03

3 FIELD DATA VALIDATION

The parameters (N_{60}, S_u, V_s) estimated with the SCPTu tests were compared to results obtained with the SPT, the FVT and the measured shear wave velocities. The consistency and the representativeness of all the information gathered was validated. To undertake this exercise, the boreholes 18F01, 18F02 and 18F03 were respectively compared to the soundings 19SCPTu-01, 19SCPTu-02 and 19SCPTu-03. The results set for the 18F02 and SCPTu-02 pair are shown in Figures 3 to 5.

Generally, the estimated parameters using the SCPTu test results correlate well with the measured data. The estimated N_{60} values were obtained from the N-SPT calculated with a factor of 0.75 as per the *Canadian Foundation Engineering Manual* (CFEM) (2013). They were also obtained from CPeT-IT based on the ratio "n" ($n = q_c/N_{60}$) that is dependent on the soil behaviour type index (I_c) as shown in Lunne et al. (1997) and Jefferies and Davies (1993) (GeoLogismiki, 2014).

Due to the variability of soil properties for the same soil type, it is well known that it is difficult to estimate parameters based on empirical correlations for CPT and SPT (Urmy & Ansary, 2017). It can be noticed the estimated and measured N_{60} values follow a similar trend as shown in Figure 4. However, the estimated N_{60} from N-SPT is generally three (3) times smaller than N_{60} obtained from SCPTu (when averaged over 0.5 m to allow the comparison). This factor of 1/3 is observed in both granular and cohesive deposits and in all three soundings' locations.

The use of this factor on the estimated N_{60} from the SCPTu data brings it to a percentage difference of approximately 30% from the SPT results. The comparison results for SCPTu-02 and borehole 18F02 are shown in Figure 4.

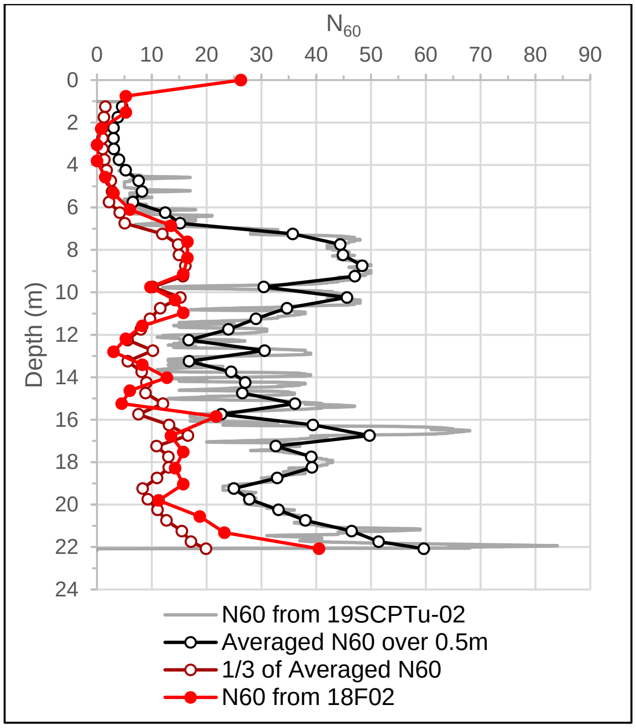

Figure 3. SPT and CPT correlation validation for 18F02 and 19SCPTu-02

The undrained shear strength of the upper clayey deposit was obtained with a Field Vane Test near 18F02. The S_u profile was compared to the estimated S_u values calculated in CPeT-IT with the 19SCPTu-02 data. The software calculates this parameter based on q_t (total tip resistance), the total stress at the depth of the measurement and the undrained shear strength cone factor for clays N_{kt}. This factor is recommended to be obtained by calibrating SCPTu values with laboratory of FVT tests. A value of 14 was used which is within the typical range for Champlain sea clays (Hébert et al. 2016). A S_u value of 0 is given for soils belonging to the SBTn zones 5, 6, 7, 8 as per Roberston (1990) chart (GeoLogismiki, 2014). In order to compare the FVT data to the estimated S_u from SCPTu, the latter was averaged over a thickness of 1 m and only non-zero values were considered. Also, the peak S_u values measured right above the 0 values layers were neglected. The high peaks are probably influenced by the underlaying soil and would highly affect the calculated average. The comparison between FVT and SCPTu results is shown in Figure 5. As it can be noticed, the two testing methods give similar results with an average percentage difference of 21%. The values obtained from the SCPTu are underestimated in the first 4.5 m (classified as clay as per the SBTn) and overestimated in the last two meters (classified as clay & silty clay).

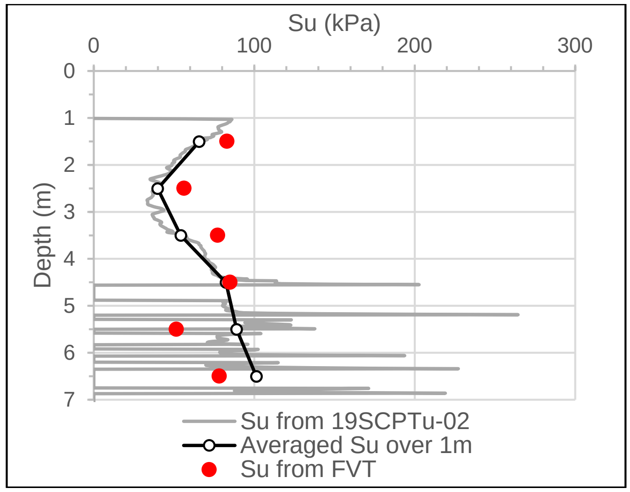

Figure 4. FVT and CPT correlation validation for 18F02 and 19SCPTu-02

The V_s readings of the SCPTu were measured at an interval of 1m over 22m depth. This data was compared to the estimated V_s values calculated in CPeT-IT. The software estimates V_s based on the small strain shear modulus G_o that is dependent on q_t, total stress, and the soil behaviour type index (I_c) (GeoLogismiki, 2014). Figure 6 shows the estimated V_s compared to the measured ones in SCPTu-02. This comparison was done based on an averaged estimated V_s value over 1m. It can be noticed the general tendencies of both curves are similar and an average error of 14% is obtained. A concern when estimating V_s from q_t is that they are small strain and large strain measurements respectively. The behavior at small and large strains may not be the same (Amoroso, 2013). This could explain the differences obtained with the V_s measurements.

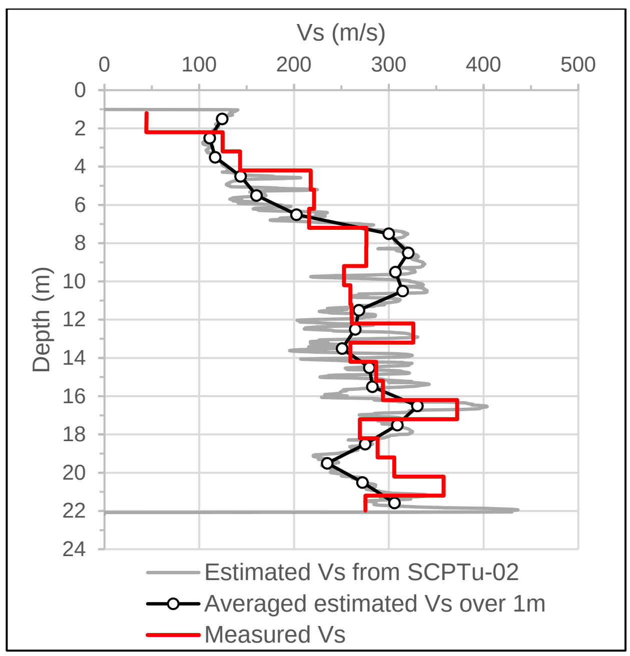

Figure 5. CPT correlation validation for estimating V_s for 19SCPTu-02

4 FOUNDATION DESIGN

The proposed design for the foundation consisted in a single pile per column of the pipe-rack. Considering the distance between each pile, no group reduction effect was expected.

4.1 Load Conditions

The factored load cases considered in the preliminary analysis consisted of the worst-case scenario as provided by the structural engineer responsible for the fire protection structural design. Given the nature and properties of the subsoil as well as the amplitude of tensile and horizontal forces applied on the foundations, it was confirmed that conventional shallow foundations would not have been a viable option.

In order to further optimize the pile properties, the critical load cases were identified as follows:

Table 1. Critical Factored Load Cases

Load Cases	Compressive Load (kN)	Tensile Load (kN)	Horizontal Load (kN)
A	210	350	150
B	780	250	150
C	510	75	75

4.2 Preliminary Design

Considering the consistency between the different soundings, the evaluation of the ultimate geotechnical axial capacity of each pile was done in accordance with the method described in the CFEM (2013) based on SCPTu data. This calculation method is already implemented in the software CPeT-IT. Circular steel pipes with diameters ranging between 200 mm and 400 mm and with a closed end were considered. It was assumed the piles will be driven to refusal at approximately 22.0 m depth (Category IB, from Table 18.4 of CFEM, 2013).

Given the piles structural integrity is assured by the structural engineer and the piles are to be driven to refusal on dense till or bedrock (high bearing capacity), the resistance in traction of the piles was considered to be the limiting factor in the axial capacity design. Based on the critical factored load case (in traction) from Table 1, the minimum pile diameter providing sufficient friction resistance was determined in accordance to the CFEM (2013) recommendations. The axial bearing capacity of the piles were also evaluated based on this reference.

4.3 Pile Modelling & Optimisation

Considering the nature and directions of the applied loads on the piles as well as the spacing between each other, it was assumed the foundation can be represented as single piles in a 2D model. Numerical analyses were performed using the 2D software LPile 2016 by Ensoft Inc. to validate and optimize the foundation design.

LPile is commonly used to compute deflection, shear, bending moment in the pile as well as the soil response to specific lateral and vertical load cases applied at the pile head (Isenhower, 2016). The software generates internally the nonlinear response of the soil in the form of p-y curves for lateral loading and solves the response of the pile.

The stratigraphy, soil parameters and the water level used in the model were based on the available data and the literature. The structural parameters were obtained from the structural engineer and the pile's cross-section consisted of a circular concrete pile with a permanent steel casing. Predefined soil models from LPile as well as the calculated soil-spring stiffnesses were used in the analyses.

The soil-spring stiffness have been evaluated according to the method described in the CFEM (1994) that is based on Terzaghi (1955) and Davisson (1970). The method considers the pile as a flexible beam and the soil as a spring assembly with a stiffness K_s (kPa/m) around the pile. The K_s values were calculated for pile diameter ranging between 200 mm and 400 mm.

The modelling allowed the optimisation of pile diameters with respect to the allowable deflections, shear and bending moment set by the structural engineer.

4.4 Final Design

The final design of the piles was based on the results of the LPile analyses as well as the axial requirements. In most cases (all but one), the lateral resistance of the piles controlled the size of the pile and the thickness of the steel casing. The resistance in traction was the governing factor for the pile subjected to the critical factored load case A from Table 1.

The pile sizes show in Table 2 were obtained following the optimisation process:

Table 2. Final Concrete Pile Design

Final Pile Diameter (mm)	Thickness of Steel Casing (mm)	Maximal Factored Lateral Loads (kN)
245	8.9	75
356	9.5	150
406	9.5	150*

** The load in traction of 350 kN controlled the design for this pile*

5 DYNAMIC PILE TESTING

5.1 Test Procedure

Following pile installation, one (1) Dynamic Pile Testing was performed per pile diameter in accordance with the ASTM D4945-17 standard using the Pile Driving Analyzer (PDA) system and the CAPWAP®.

Before each test, accelerometers and strain transducers were installed on the pile head and connected via data signal cables to the PDA. A hammer was dropped onto the pile head from a predetermined height generating two impact records collected by the sensors and transmitted to the PDA: a force and an acceleration. The measured acceleration was then used as an input to a pile model, with estimates of soil resistance, quake and damping parameters to calculate a force-time signal which was compared to the measured force-time signal. Using the CAPWAP® program, the estimated parameters were adjusted until agreement between the measured and calculated signals (FHWA, 2006). The static component of the resistance that provided the satisfactory match was assumed to be the static bearing capacity of the pile (FHWA, 2000) which can also be obtained with the Case Method, used in Dynamic Pile Monitoring Testing.

5.2 CAPWAP® Tests Results

Tests were considered conclusive if the bearing capacity obtained with the Case Method was greater than the associated critical factored compressive load for each pile. The bearing capacity obtained simply corresponds to the sum of the shaft and toe resistance multiplied by the geotechnical resistance factor as per the CFEM (2013) recommendations. The results of the tests are summarized in Table 3.

Table 3. Dynamic Pile Testing Results

Diameter (thickness) (mm)	CAPWAP® capacity (kN)			Bearing capacity* (kN)	Factored Compression load (kN)
	Shaft	Toe	Total		
245 (8.9)	1,977	188	2,165	1,083	510
356 (9.5)	1,357	807	2,164	1,082	780
406 (9.5)	1,519	921	2,440	1,220	210

***** *includes a geotechnical resistance factor of 0.5*

6 DISCUSSION

6.1 Design Validation with SCPTu and CAPWAP®

The objective of the Dynamic Pile Testing was to validate the design by measuring the capacities along the shaft and at the toe of the tested piles. The data acquired during this testing was compared to the cumulative friction (Q_s) along the shaft and the toe resistance estimated with the CFEM (2013) method for the SCPTu data. For each pile diameter, the comparison of the toe resistance is shown in Table 4 and the friction capacity along the shaft is shown in Figures 6 to 8.

It can be noticed for the toe resistance that a smaller value was obtained from the CAPWAP® than the SCPTu for the 245 mm diameter pile with a percentage difference of 81% (overestimation with SCPTu). For the 356 mm and 406 mm diameter piles, a percentage difference of 12% and 23% was found respectively (conservative results with SCPTu).

It can also be noticed for the 245 mm diameter pile, the cumulative friction obtained with the CAPWAP® test and the SCPTu first correlate well in the clay layer, then the two curves start to diverge. The percentage difference between the SCPTu and the CAPWAP® results in the clay layer is on average 16%. The CAPWAP® indicates a greater cumulative friction; up to 3.5 times higher than the SCPTu results below the clay layer. The average percentage differences of 26% and 9% are obtained between the calculated and measured friction along the shaft for the 356 mm and 406 mm diameter piles respectively. Overall, the friction resistance estimated with the cone penetration test design method from the CFEM (2013) provides conservative results for the 245 mm and 356 mm diameter piles and relatively accurate results for the 406 mm diameter pile.

Based on these observations, it appears the diameter of the pile could have an influence on the correlation based on the SCPTu data with regards to the cumulative friction and toe resistance. The average percentage differences between the CAPWAP® and the SCPTu results tend to decrease as the diameter increases. Considering these observations were done based on only three tests, it is difficult to guarantee that the pile diameter is the only factor affecting the differences in the results. It is well known the SCPTu is one of the most effective *in-situ* tests for pile design due to its geometrical analogy with piles (M. Hassan Baziar and al., 2015). Also, studies show the CAPWAP® test is reliable and provides consistent results that are not dependent on the operator (B.H., Fellenius, 1988). Therefore, more tests on various pile diameters should be performed in order to clearly identify the cause of this difference.

Table 4. Toe Resistance Evaluated with the SCPTu and CAPWAP® Analyses

Diameter (mm)	Toe resistance (kN)		Difference relative to CAPWAP® capacity (%)
	CAPWAP® capacity	SCPTu estimate	
245	188	339	81
356	807	708	12
406	921	708	23

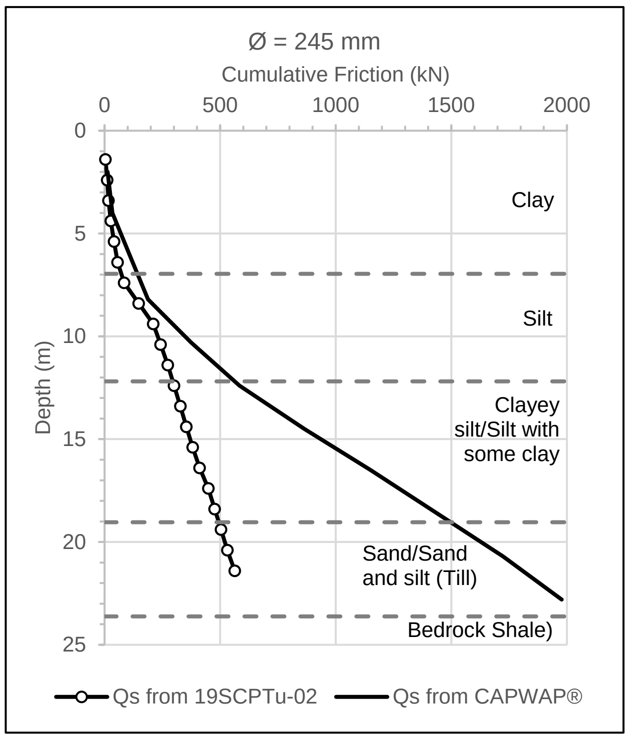

Figure 6. Q_s correlation validation for 19SCPTu-02 and CAPWAP® analysis - Ø=245 mm

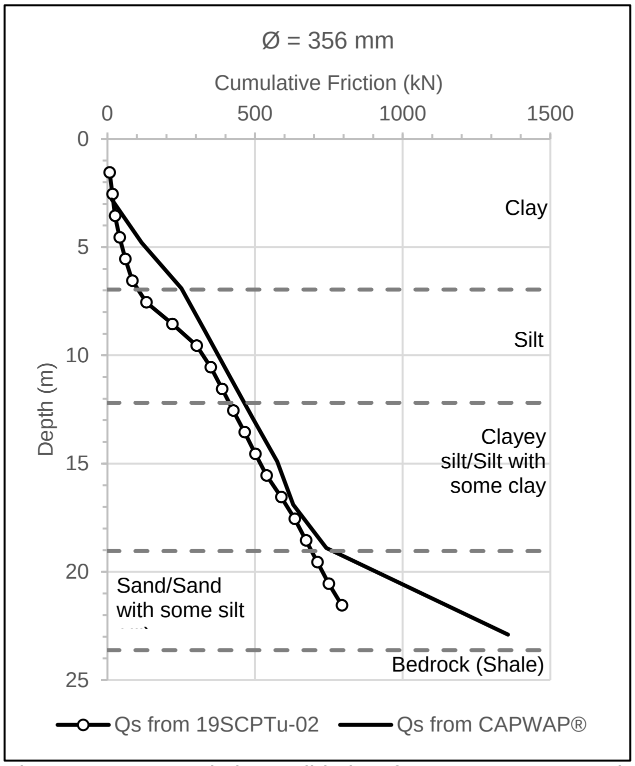

Figure 7. Q_s correlation validation for 19SCPTu-02 and CAPWAP® analysis - ∅=356 mm

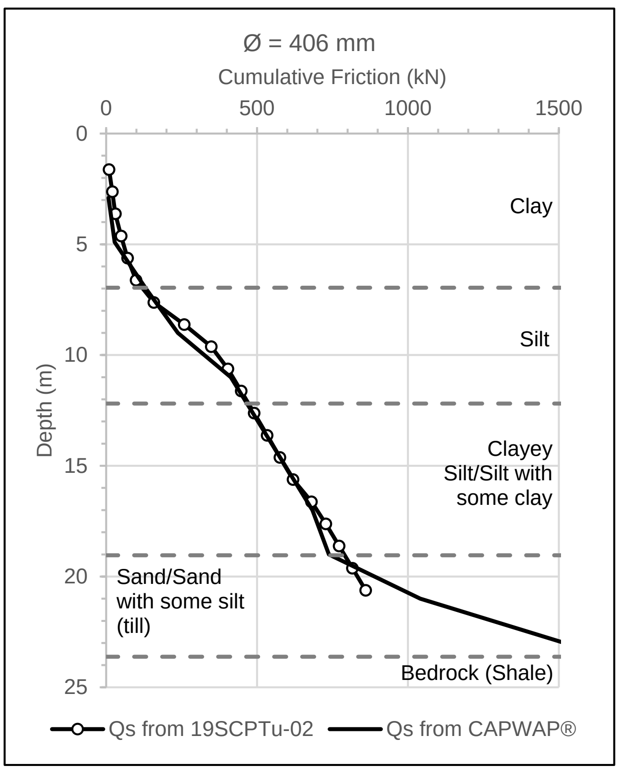

Figure 8. Q_s correlation validation for 19SCPTu-02 and CAPWAP® analysis - ∅=406 mm

6.2 Design Validation with the Effective Stress Approach and CAPWAP®

The cumulative friction and the toe resistance were estimated using the effective stress approach described in the CFEM (2013) for sublayers of 2m. The borehole data is used to determine the parameters β and N_t for the calculation of the shaft and toe resistances respectively. The intervals of these parameters (β_{min}, β_{max}, $N_{t\ min}$, $N_{t\ max}$) provided in the CFEM (2013) were used in the comparison with the CAPWAP® results. For each pile diameter, the comparison of the toe resistance and the friction along the pile is shown Table 5 and in Figures 9 to 11.

The estimated values of the toe resistance with the CFEM method are generally 3 to 6 times greater than the measured ones, depending on the diameter (overestimation with CFEM method).

For the 245 mm diameter pile, the measured friction along the shaft and the ones estimated with the CFEM (2013) correlate well in the clay layer and are found between the upper and lower bounds. Below that layer, the CAPWAP® results start to diverge from the β_{min} curve and become up to 1.5 times higher than the estimated friction using β_{max} in the clayey silt and sand layers (conservative results with CFEM method).

The measured friction for the 356 mm diameter is also found between the bounds of the CFEM curves for the clay and silt layers. However, for this pile size, when the curves start to diverge, the results indicate the measured friction reach values that are 1.3 times smaller than the estimated friction using β_{min} in the clayey silt and till layers (overestimation with CFEM method).

Regarding the 406 mm diameter, the results indicate the measured friction in the clay layer is, on average 4.5 times lower than the ones estimated with the CFEM method with β_{min}. The curve remains below the β_{min} bound and the measured friction reaches values that are 1.4 times lower than the estimated friction using β_{min} (overestimation with CFEM method).

Based on these observations, it appears the diameter of the pile could have an influence on the reliability of the CFEM method for the estimation of both toe resistance and shaft friction. More specifically, if the design had been done with this method, the calculated shaft friction would have been conservative for the 245 mm diameter pile but overestimated for the 356 mm and 406 mm diameter piles. The toe resistance would have been overestimated for all three pile diameters.

Again, considering these observations were done based on only three tests, it is difficult to guarantee that the pile diameter is the only factor affecting the differences in the results. More tests on various pile diameters should be performed in order to clearly identify the cause of these differences.

Table 5. Toe Resistance Evaluated with the CFEM and CAPWAP® Analyses

Diameter (mm)	Toe resistance (kN)			Minimum ratio
	CAPWAP® capacity	CFEM estimate		
		$N_{t\ min}$	$N_{t\ max}$	
245	188	1186	1423	6.3
356	807	2512	3015	3.1
406	921	3278	3934	3.5

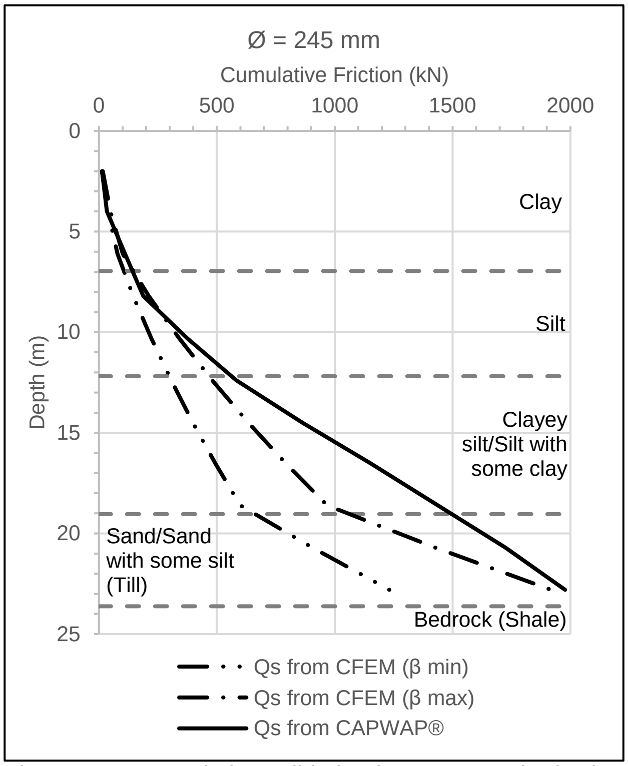

Figure 10. Q_s correlation validation for CFEM method using SPT and CAPWAP® analysis - ∅=245 mm

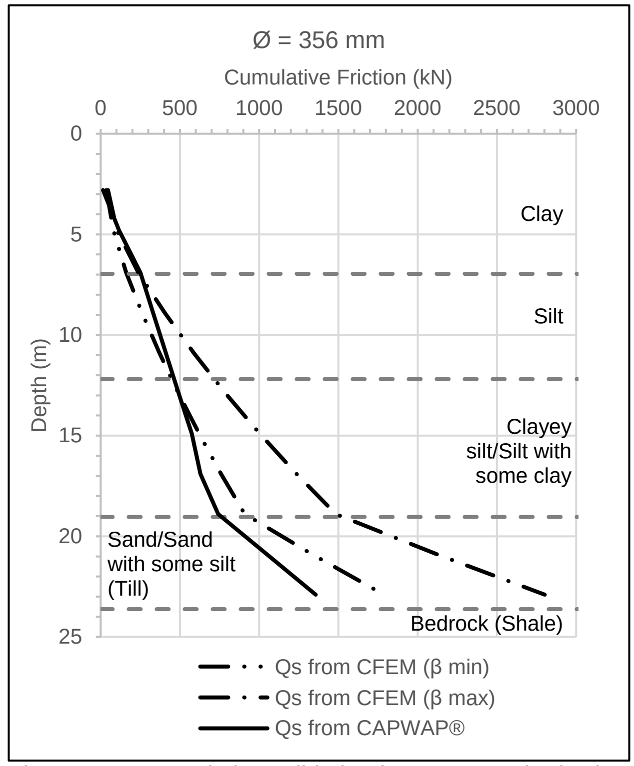

Figure 11. Q_s correlation validation for CFEM method using SPT and CAPWAP® analysis - ∅=356 mm

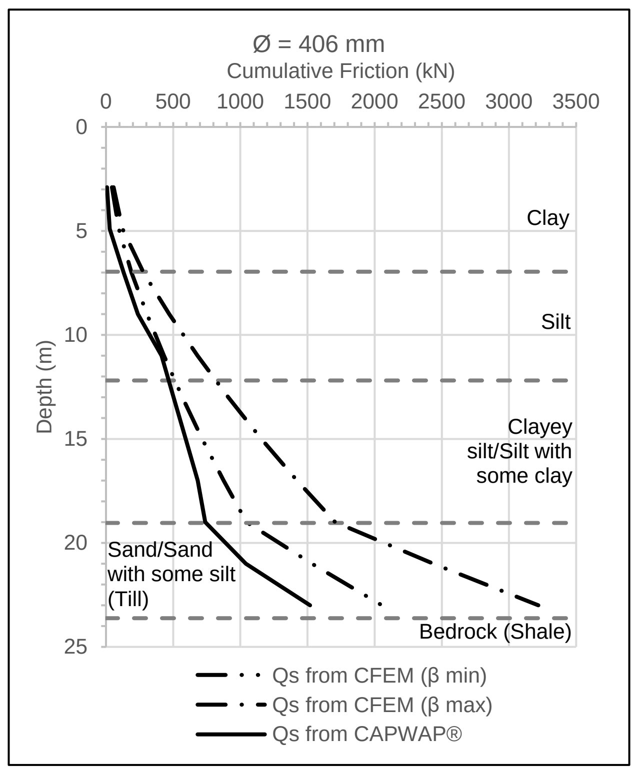

Figure 12. Q_s correlation validation for CFEM method using SPT and CAPWAP® analysis - ∅=406 mm

7 CONCLUSION

This case study provides an overview of the close relationship between geotechnical investigation and design as well as the effectiveness of a multidisciplinary approach.

The geotechnical investigation included SPT, FVT and SCPTu soundings and allowed the comparison and calibration of various design parameters. This comparison showed a similarity in the observed stratigraphy as well as undrained shear strength in the upper clay layer. However, when comparing N_{60} values obtained from N-SPT to the ones estimated with SCPTu data, a factor of 1/3 was observed in both granular and cohesive deposits. More comparisons between these sounding methods should be performed in order to confirm the applicability of a factor for different soil types.

This paper also discussed the design methods proposed in the literature and their validity based on the CAPWAP® results. Overall, the results indicate the shaft friction calculated with the SCPTu method from the CFEM (2013) correlates well with the CAPWAP® results except for the 245 mm diameter pile. The toe resistance is underestimated with this method for the 356 mm and 406 mm diameter pile and overestimated for the 245 mm diameter pile. The shaft friction calculated with the effective stress approach from the CFEM (2013) is generally overestimated even when using β_{min} except for the 245 mm diameter pile. For the latter, the shaft friction is underestimated even when using β_{max}. The toe resistance calculated with $N_{t\,min}$ is also overestimated for all three piles diameters.

The difference observed between the CAPWAP® results for the 245 mm diameter pile and the other two sizes suggests there could have been an issue with the test done on the 245 mm diameter pile diameter. This indicates one CAPWAP® test per pile size might not to be sufficient to compare and validate measured and estimated values.

Considering the effective stress approach is widely used in the industry, the observations made in this paper are critical and should be further explored. Based on the results of this study, it would be preferable to use β_{min} and $N_{t\,min}$ for resistance estimations.

Finally, lateral load pile testing would have permitted to further validate the design by comparing the field results to LPile model. Such comparisons have been done in the past and can be found in the literature.

ACKNOWLEDGMENTS

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