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Driven pile capacity for wind farms in western Ontario, Canada

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ABSTRACT
For some wind farm projects in Western Ontario (Canada), piled foundations are required to support 80 m to 90 m high wind turbines due to the presence of thick soft soil strata. The wind farms constructed with piled foundations are located in Bruce County and Essex County in Ontario. In the last few years, approximately 2,000 steel H piles were driven to support about 60 wind turbines of which the majority were located in Bruce County. The piles had to be driven to achieve the design pile axial capacities in both compression and tension.

During the design phase, the pile capacities were evaluated by static pile capacity analysis using borehole information obtained at each wind turbine location. The subsurface soil conditions were investigated by Standard Penetration Test (SPT), Dynamic Cone Penetration Test (DCPT), in-situ field vane shear test and laboratory test. The minimum pile embedment depth at each wind turbine location was calculated in order to achieve the design pile compression and tension capacities. Pile driving criteria based on Hiley formula were then developed and used for driving piles.

During pile driving, Pile Driving Analyzer (PDA) was utilized to evaluate the performance of each pile driving hammer and the capacities of piles for both the end of initial drive and the restrike. The results of the PDA were then used to confirm the pile driving criteria and the design pile capacities. The results of the PDA were back-analyzed to determine the soil strength parameters applicable for static pile capacity analysis. The soil strength parameters back-analyzed include adhesion factor for pile shaft resistance in cohesive soils and bearing capacity factor for pile end bearing resistance in non-cohesive soils. The degree of pile capacity increase due to soil set-up is also obtained from the PDA results. The soil strength parameters and the soil set-up obtained from the PDA results presented in this paper will be useful for designing and driving piles in Bruce County and Essex County of Western Ontario (Canada).

RÉSUMÉ
Pour certains projets de parc éolien développés en Ontario (Canada), dû à la présence des épaisses strates de terre molles, les fondations sur pieux sont requises pour supporter des turbines éoliennes à haute vitesse, mesurées de 80m à 90m d'hauteur. Les parcs éoliens construits avec fondations sur pieux sont situés aux comtés de Bruce et d'Essex en Ontario. Dans les dernières années, approximativement 2000 pieux en acier de type H sont utilisés pour supporter environ 60 turbines éoliennes dont la majorité est située au comté de Bruce. Les pieux ont dû enfoncés bien profondément dans le sol à l'aide des puissants marteaux d'enfoncement jusqu' à ce que leurs capacités axiales de conception - en compression ainsi qu'en tension, soient atteintes.

Pendant la phase de conception, les capacités des pieux ont été évaluées par l'analyse de capacité de pieu statique en utilisant les données de forage obtenues à chaque location où les turbines éoliennes se situent. Les conditions sous-terraines ont été investiguées par les méthodes d'Essai de Pénétration Standard (SPT), Essai de Pénétration Dynamique au Cône (DCPT), Essai au Scissomètre de Chantier (VST) et les tests en laboratoire. À chaque location où les turbines éoliennes se situent, la profondeur d'ancrage minimale du pieu a été déterminée afin que les capacités de conception, en compression ainsi qu'en tension, soient atteintes. Les critères d'enfoncement du pieu basés sur la formule de Hiley ont été en suite développés et utilisés pour l'enfoncement des pieux.

Pendant l'enfoncement du pieu, Pile Driving Analyzer (PDA) a été utilisé pour évaluer la performance de chaque marteau de fonçage et les capacités des pieux à la fin du fonçage initial et de rebattement. Les résultats obtenus du PDA ont été utilisés en suite pour confirmer les critères de fonçage et les capacités de conception du pieu. Les résultats du PDA ont été analysés à l'inverse pour déterminer les paramètres de la force du sol applicable à l'analyse de la capacité statique du pieu. Les paramètres de la force du sol qui ont été analysés à l'inverse incluent la facteur d'adhésion par résistance au long du fût du pieu dans les sols cohésifs et la facteur de capacité portante en pointe à la base du pieu dans les sols incohésifs. La degrée d'augmentation de capacité du pieu grâce à la configuration du sol est aussi obtenue des résultats du PDA. Les paramètres de la force du sol et la configuration du sol obtenue des résultats du PDA présentées dans cet article seront utiles pour la conception et le fonçage des pieux aux comtés de Bruce et d'Essex de l'Ontario de l'Ouest.

1 INTRODUCTION

In the last few years, wind farm projects have been designed and built in Bruce County and Essex County in Western Ontario, Canada (Figure 1). Each wind turbine installed is, in general, 1.65 MW to 1.80 MW in capacity and 80 m to 90 m in hub height. The wind turbines are located approximately a few hundred metres apart, such that each wind farm project site covers a large area of up to about 10 km by 10 km. An example of the wind turbines already erected at one of the project sites is shown in Photograph 1. Due to the presence of thick soft-to-stiff clayey soil strata in some areas, about 60 wind turbines have to be supported by piled foundations, the pile lengths of which exceed 30 m in some turbine locations. Each piled foundation (Figure 2) has been designed by considering the pile group behaviour to support the high moment on the foundation caused by the wind load, resulting in the number of piles for each foundation to be less than 40 piles. Approximately 2000 piles have been installed for the wind farm projects located in Bruce County and Essex County.

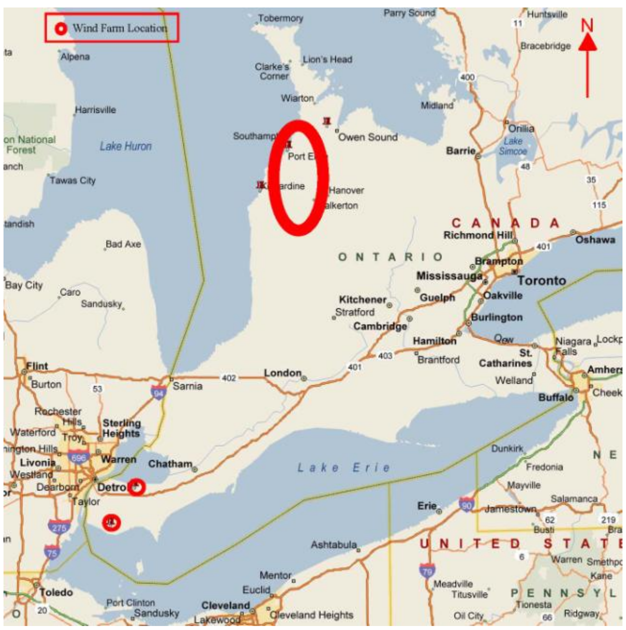

Figure 1. Site Location Plan

In order to limit the type and size of pile for construction control, H 310 x 110 and H 310 x 79 steel piles are selected, based on their availability and common usage in Ontario. The design allowable pile capacity for H 310 x 110 is 900 kN in compression and 350 kN in tension, and for H 310 x 79 is 900 kN in compression and 250 kN in tension. Pile driving criteria are based on a minimum pile length to achieve the design pile capacity in tension (established by static pile analysis) and Hiley's Formula to achieve a driven pile capacity of 2,700 kN in compression. Pile Driving Analyzer (PDA) is used to determine the pile driving energy for every pile driving rig used, revise set criteria by Hiley's Formula (if necessary), and verify the pile capacities in both compression and tension.

This paper describes the results of back analysis conducted to determine the adhesion factor for pile shaft resistance in cohesive soils, shaft resistance factor in non-cohesive soils, and bearing capacity factor for pile end bearing resistance in non-cohesive soils. The back-analyzed soil parameters will be useful for estimating pile capacities and pile lengths in Bruce County and Essex County, including other locations with similar soil conditions.


Photograph 1. Example of Wind Turbine Erection

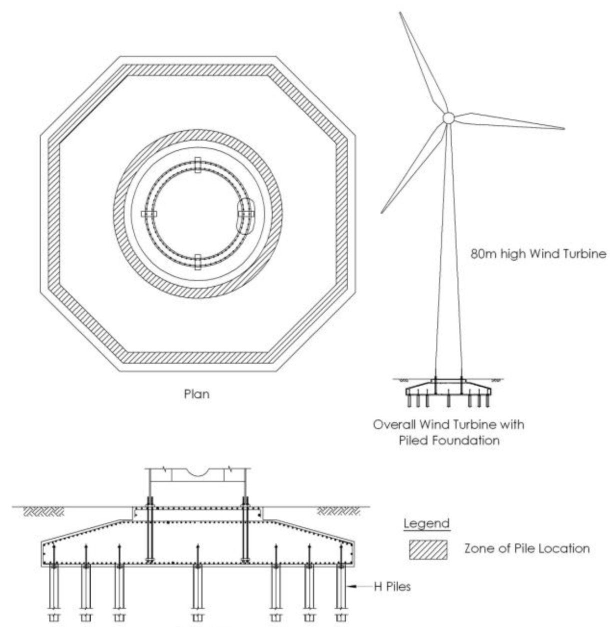

Figure 2. Piled Foundation for Wind Turbine

2 SUBSURFACE SOIL CONDITIONS

The geotechnical investigation for a wind farm project is typically carried out by the Project Owner in order to provide subsurface soil information for a design-build contract. As such, this information is in general sufficient for deciding the type of foundation (i.e., shallow or deep), but it may not be sufficient for detail foundation design. Additional geotechnical investigation may later be carried out by the design-build team during the detail design phase, depending on the adequacy of the available information.

For the wind farm projects considered in this paper, each wind turbine location was investigated by a minimum of one borehole drilled to a minimum depth of 18 m in order to establish the subsurface soil conditions for foundation design. Each borehole was drilled typically by hollow-stem augering whenever a soft soil stratum was encountered. Standard Penetration Test (SPT – ASTM D 1586) was carried out at regular depth intervals, together with field vane shear testing in soft clay strata. If the soil stratum at 18 m depth was not competent to support deep foundation, a dynamic cone penetration test (DCPT) using the same hammer and the same 0.75 m drop height as SPT (CFEM, 2006) was conducted below the 18 m depth through the hollow stem augers until refusal to cone penetration (100 blows/0.3 m) was reached. Alternatively, SPT was continued until a competent soil stratum was reached.

The results of the subsurface soil investigation can be categorized broadly, with respect to the soil conditions that require piles to support the wind turbines, as follows:

2.1 Thick Clayey Soil Overlying Very Dense Stratum ("Soft Soil Profile")

A typical soil profile where a thick soft-to-stiff clayey soil overlies a very dense stratum is shown in Figure 3. Such a soil profile is referred to as "soft soil profile" in this paper, in relation to pile driving effort. At this borehole location, the field vane strength of the firm-to-stiff clay ranges from about 50 kPa to 60 kPa with the sensitivity in the range of 1.6 to 1.8. The liquid and plastic limits of the clay are 25 and 15 respectively, with its natural water contents varying generally from 16 % to 22 %. DCPT conducted below the 18 m depth through hollow stem augers reaches refusal (100 blows per 0.3 m) at a depth of about 31 m.

The majority of the "soft soil profiles" encountered at the sites in Bruce County and Essex County fall in this category.

2.2 Clayey Soil Overlying Very Dense to Compact Sand ("Soft to Dense Soil Profile")

Figure 4 shows an example of a soil profile where a soft-to-stiff clay overlies a relatively-thick, very dense sand which subsequently becomes less dense (i.e., 'compact'). Such a soil profile is referred to as "soft to dense soil profile" in this paper, with respect to pile driving effort. In this example, a 7.5 m thick, firm clay with a field vane strength of 29 kPa is underlain by a 6 m thick, very dense sand with SPT 'N' values of higher than 50 blows per 0.3 m. However, the very dense sand becomes less dense below a depth of about 13.5 m, without the evidence of being loosened due to groundwater inflow to the hollow stems during drilling.

The minority of wind turbines are located in this "soft to dense soil profile" category.

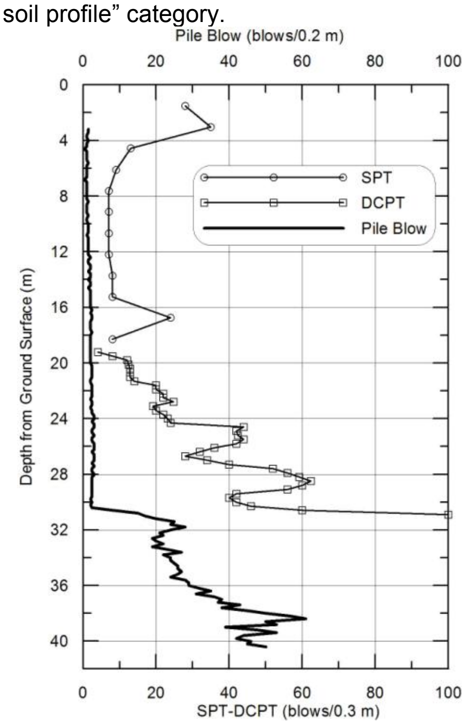

Figure 3. Example of Thick Clayey Soil Overlying Very Dense Stratum ("Soft Soil Profile")

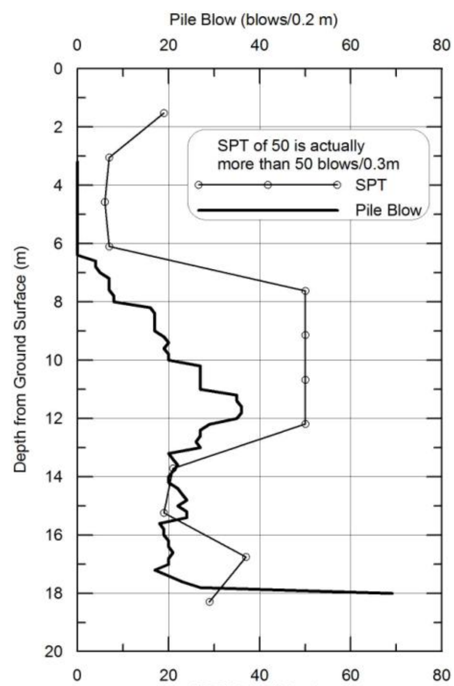

Figure 4. Example of Clayey Soil Overlying Very Dense to Compact Sand ("Soft to Dense Soil Profile")

3 CALCULATION OF PILE CAPACITY

To calculate the pile capacity in compression of a single pile using static analysis, the following equation is provided in Canadian Foundation Engineering Manual (CFEM, 2006):

$$R = \sum Cq_s \Delta z + A_t q_t - W_P \quad [1]$$

where
- R = geotechnical axial capacity of a single pile
- C = pile circumference
- q_s = shear stress along pile shaft
- Δz = pile shaft section length
- A_t = pile toe area
- q_t = bearing capacity of pile toe
- W_P = pile weight

For cohesionless soil, the unit shaft friction at any depth z along the pile is expressed by

$$q_s = \sigma'_v K_s \tan \delta = \beta \sigma'_v \quad [2]$$

and the bearing capacity of the pile toe is

$$q_t = N_t \sigma'_t \quad [3]$$

where
- β = a combined shaft resistance factor
- K_s = coefficient of lateral earth pressure
- σ'_v = vertical effective stress adjacent to the pile at depth z
- δ = the angle of friction between the pile and the soil
- N_t = bearing capacity factor
- σ'_t = vertical effective stress at the pile toe

For shaft resistance in clays with undrained shear strength less than 100 kPa, the ultimate shaft resistance of a single pile using total stress analysis is determined from

$$q_s = \alpha s_u \quad [4]$$

where
- α = adhesion coefficient ranging from 0.5 to 1.0
- s_u = undrained shear strength

For driven piles, the ranges of β coefficients and N_t factors are provided in CFEM (2006) as a function of soil type. The relationship between the adhesion factor α and undrained shear strength is also given in CFEM (2006).

Back analyses of the piles driven for the wind farm projects considered in this paper are carried out by using the results of the Pile Driving Analyzer (PDA – ASTM D 4945) which estimate the pile capacities in terms of shaft resistance and toe resistance. Although the pile capacities determined by PDA have not been compared with those evaluated by static pile loading tests, PDA is normally used to determine pile capacities in Ontario. As such, the use of PDA results in this paper will directly be useful for other projects that assess driven pile capacities by PDA. The results of the back analyses are compared with the ranges of β, N_t and α given in CFEM (2006) so that the ranges of these values (referred to as "soil strength parameters" in this paper) applicable to Western Ontario can be established for future piling projects.

4 BACK ANALYSIS OF PILE CAPACITY

The following procedures/considerations are used to back analyze the capacities of the piles driven for the wind farm projects:

a. From the borehole log where the pile is tested by PDA, the soil profile is simplified by two soil layers, i.e., a thick soft-to-stiff clay layer overlying a very dense sand stratum ("soft soil profile" in Figure 3) or a soft-to-stiff clay layer overlying a thick layer of very dense to compact sand ("soft to dense soil profile" in Figure 4), based on the actual soil conditions encountered at the wind turbine location tested by PDA.
b. For the clayey soil, the average undrained shear strength is obtained based on the average SPT-N values measured for the clayey soil, and subsequently determined from a range of undrained shear strength corresponding to SPT-N values shown in CFEM (2006).
c. The mobilized shaft friction and end bearing of the driven pile are obtained from the PDA tests and CAPWAP analysis.
d. To calculate the shaft friction obtained by PDA, the adhesion coefficient (α) for the clayey soil and the shaft resistance factor (β) for the sandy soil are estimated by trial and error.
e. The circumference area of the H pile is considered as fully-plugged (i.e., rectangular section - □ covering the H cross-sectional area) and unplugged (i.e., H section) as illustrated in Figure 5.
f. To calculate the end bearing obtained by the PDA and CAPWAP, the bearing capacity factor (N_t) is estimated by using both rectangular and H sections.

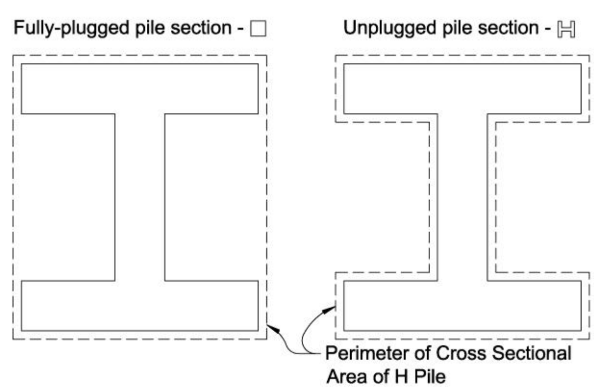

Figure 5. Cross Sectional Area and Circumference of H Pile Used in Back-analysis

5 RESULTS OF BACK-ANALYZED SOIL STRENGTH PARAMETERS

More than 30 driven piles were tested by PDA for both "end of initial drive" (EOID) and "beginning of restrike" (BOR). All piles were initially driven to the set criteria based on Hiley Formula with a factor of 3 for the design pile axial resistance in compression (i.e., 900 kN design resistance vs 2700 kN driven resistance). The set criteria were subsequently revised, if necessary, by using PDA on the first few driven piles prior to driving production piles. The results of PDA and CAPWAP shown in Figure 6 indicate that the majority of the mobilized pile resistances in compression were approximately 2400 kN which were derived from both shaft resistance and toe resistance. These piles were therefore not totally end-bearing piles.

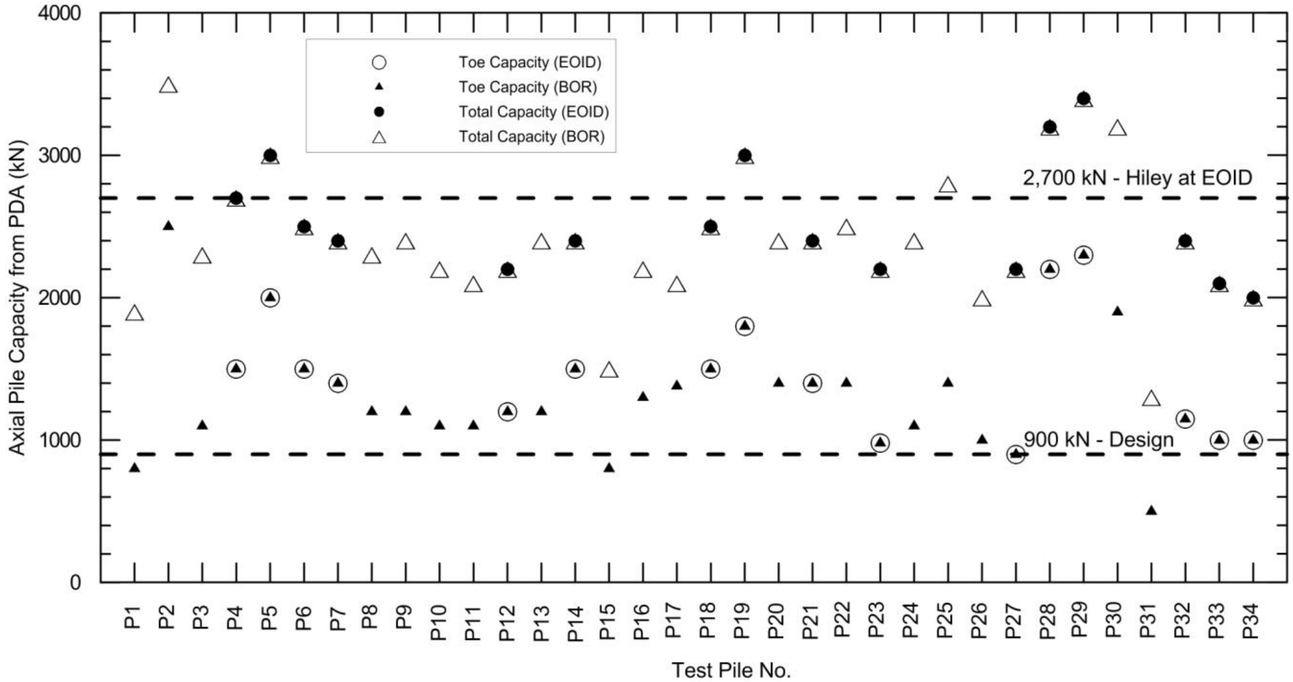

Figure 6. Mobilized Pile Resistance in Compression Obtained by PDA / CAPWAP

5.1 Back-analyzed Values of α, β and N_t

The back-analyzed results lead to the mobilized values of adhesion (α), shaft resistance factor (β) and bearing capacity factor (N_t) for each pile as shown in Figure 7.

The mobilized values of β for non-cohesive soils obtained from back analysis are, in general, much less than the β values provided in CFEM (2006) for medium to dense sand. The difference in the circumference of the H piles (i.e., □ or H section) for the β values is relatively small. Such a difference is much more pronounced in the values of N_t for end bearing with the fully-plugged section (□ section) being in relatively-good agreement with the lower range of the N_t values provided in CFEM (2006). If the N_t values provided in CFEM (2006) are for ultimate pile capacity, the end bearing capacity of the driven piles tested may have nearly reached or reached its ultimate value.

The adhesion (α) of clay shown in Figure 7 is further plotted in relation to the corresponding undrained shear strength (s_u) in Figure 8, together with the representative relationship shown by CFEM (2006). The values of the adhesion (α) shown in Figure 8 consider the ranges of the undrained shear strength (s_u) derived from the SPT N values and the measured field vane shear strength. The mobilized adhesion (α) values for the clayey soil in Bruce County and Essex County are evidently lower than those shown as the representative values in CFEM (2006), i.e., solid and dashed lines in Figure 8. For the majority of the undrained shear strength (s_u) which range from 40 kPa to 80 kPa, the range of the adhesion factor (α) is between 0.1 and 0.3 as compared to 0.5 and 0.8 for the representative values of α shown in CFEM (2006). The difference could be due to the possibility that the driven piles may have been driven close to failure but not yet to failure (i.e., close to the ultimate shaft resistance). Other contributing factors may include the possible effects of the combination of the two interface surfaces (i.e., "steel against clay" and "clay against clay" surfaces [Figure 5], instead of "concrete against clay" surface shown in Figure 8), and the plug or unplug conditions of H piles (Figure 5).

5.2 Soil Set-up

The driven piles were tested by PDA for both "end of initial drive" (EOID) and "beginning of restrike" (BOR) for piles driven a few days earlier. The variations of mobilized pile capacity with time are shown in Figure 9a for the "thick clayey soil overlying very dense stratum" (Figure 3 – "soft soil profile") and Figure 9b for the "clayey soil overlying very dense to compact sand" (Figure 4 – "soft to dense soil profile").

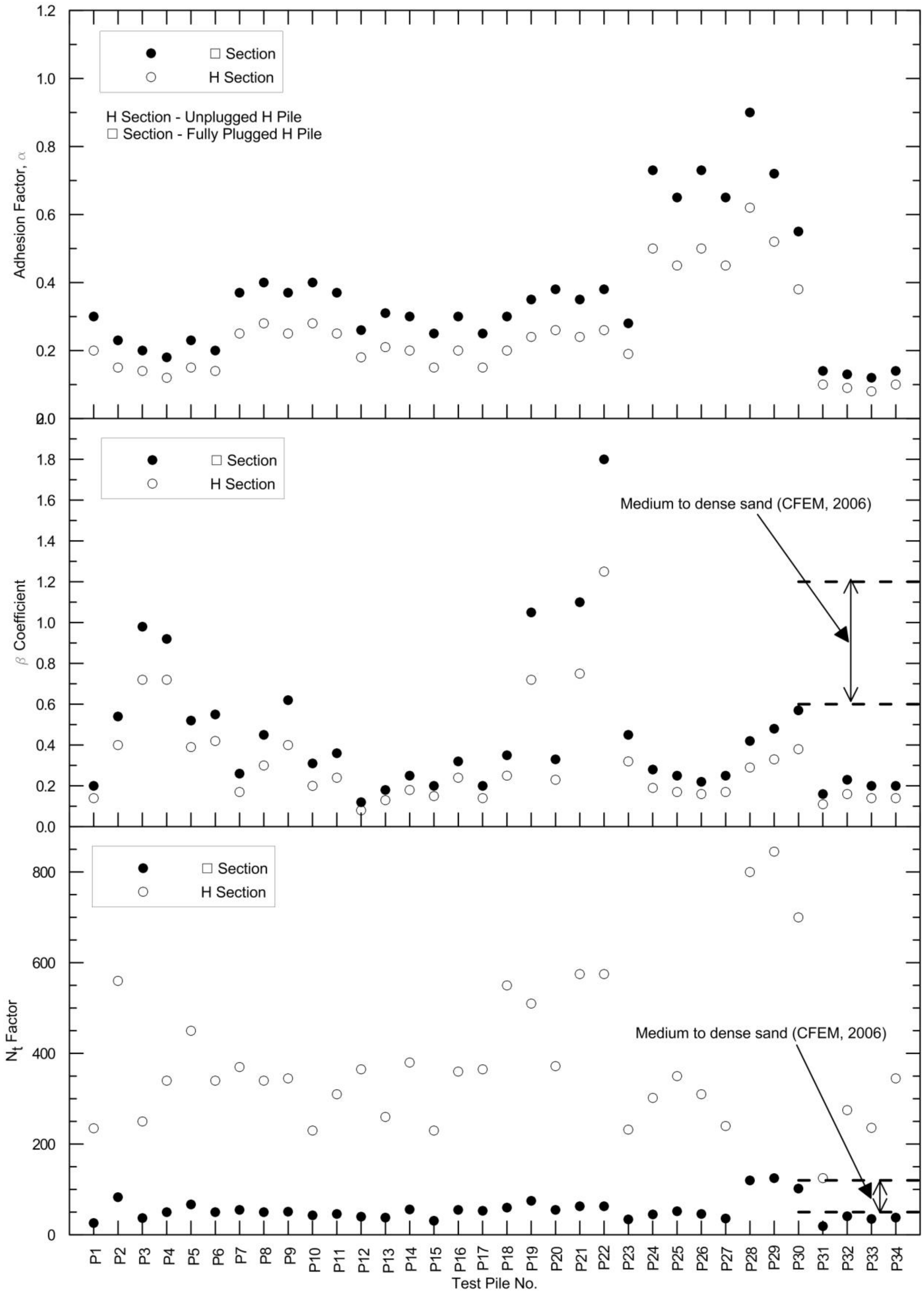

Figure 7. Mobilized Values of α, β and N_t

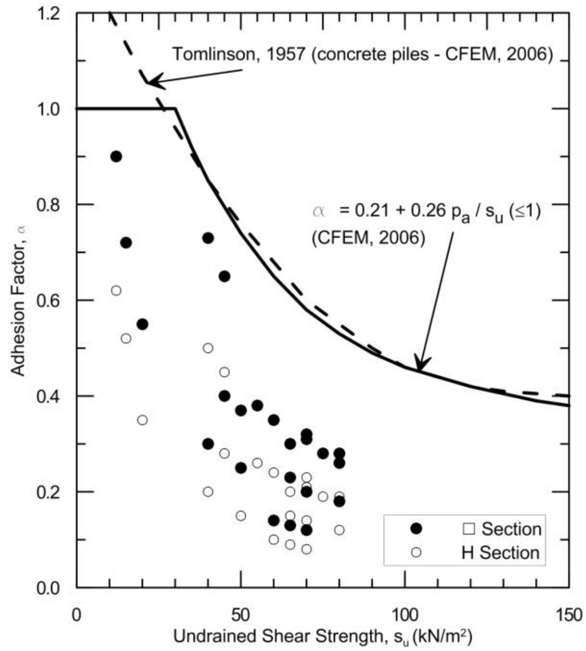

Figure 8. Relationship of Adhesion Factor (α) and Undrained Shear Strength (s_u)

For the “soft soil profile” in Figure 9a, both the shaft resistance and the toe resistance show small variation with time, indicating no significant gain of pile capacity by soil set-up with time. As for the “soft to dense soil profile” in Figure 9b, the variation of shaft resistance with time is small while that of toe resistance is rather large and inconclusive. The insignificant or inconclusive soil set-up gain of pile capacity in Bruce County and Essex County could be due to the low sensitivity of the clayey soils and probably low excess porewater pressure caused by pile driving. It should however be noted that the driven pile capacities as reported in this paper were measured on different piles by PDA over a maximum period of 11 days after the piles were driven. There could be some significant gain in pile capacity after the 11 day period.

From the results of EOID and BOR, the pile capacity gain from soil set-up, if any, should not be taken into account in design and pile driving criteria.

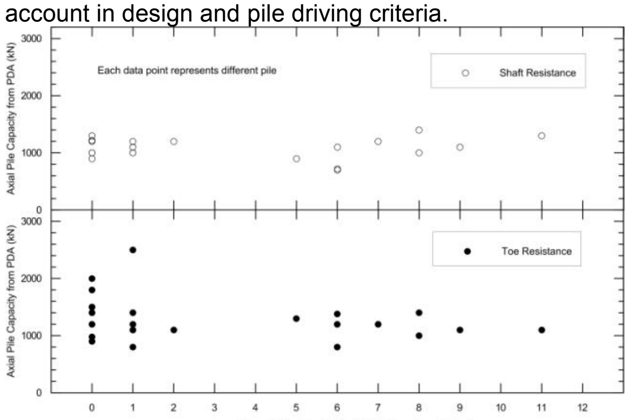

Figure 9a. Variation of Pile Resistance with Time (Thick Clayey Soil Overlying Very Dense Stratum – “Soft Soil Profile”)

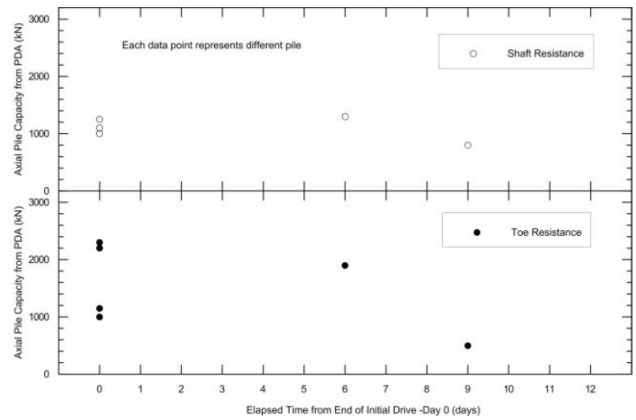

Figure 9b. Variation of Pile Resistance with Time (Clayey Soil Overlying Very Dense to Compact Sand – “Soft to Dense Soil Profile”)

6 CONCLUSIONS

The results of back analyzing the pile capacities obtained by PDA for more than 30 driven piles in Bruce County and Essex County (Ontario, Canada) lead to the following conclusions:

- For piles driven into dense sandy soils, the bearing capacity factor (N_t) shown in CFEM (2006) provides a reasonable end bearing capacity for pile design when used with the assumption that the cross sectional area for steel H piles is fully-plugged.
- For estimating the shaft capacity of steel H piles, the shaft resistance factors (α and β) shown in CFEM (2006) should be selected from the lower range.
- Using the Hiley Formula with 3 times the design pile capacity in compression to drive steel H piles to the design static compression capacity will provide an actual pile capacity that is higher than the design pile capacity, but possibly lower than 3 times the design pile capacity. In other words, the factor of safety for the design pile capacity in compression may be less than 3.0, if this is required. The actual pile capacity driven by the Hiley Formula should always be verified by pile loading tests.
- Soil set-up for steel H driven piles may not be significant, in other words, the capacity of driven steel H piles may not increase with time after being driven.

ACKNOWLEDGEMENTS

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