Correlation of Cone Penetration Test (CPT) Pile Design Methods to Measured Shaft Resistance of a Driven Steel H-Pile in Silty Clay

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ABSTRACT
Numerous cone penetration test (CPT) methods exist for estimating pile capacity and shaft resistance. All CPT methods are developed from empirical correlations to load tests and are most relevant for the geological conditions and pile type for which they were derived. Therefore, it is beneficial to develop correlations for specific geographic areas and pile types. It is not yet known which methods are most accurate for driven steel H-piles in Lake Agassiz silty clay near Winnipeg, Manitoba. Data from CPT tests near Winnipeg was used to estimate shaft resistance using four CPT/CPTu methods. The estimated shaft resistance was compared to measured shaft resistance from static load testing of an instrumented steel H-pile. It was found that each method greatly overestimated the unit shaft resistance in the upper 2 m. The LCPC method provided the best fit overall. The UniCone method was the least accurate but was calibrated for an improved fit.

RÉSUMÉ
Il existe de nombreuses méthodes de l'épreuve du pénétromètre cône pour estimer la capacité et le frottement latéral des pieux. Toutes ces méthodes sont développées à partir de corrélations empiriques avec des essais de charge et sont les plus pertinentes selon les conditions géologiques et les types de pieux desquels elles ont été dérivées. Par conséquent, il est avantageux de développer des corrélations pour des zones géographiques et des types de pieux spécifiques. Les méthodes les plus précises pour les pieux d'acier en H battus dans l'argile limoneuse du lac Agassiz, près de Winnipeg, au Manitoba, ne sont pas encore connues. À l'aide de quatre méthodes, les données des épreuves pénétromètres cône ont été utilisées pour estimer le frottement latéral des pieux près de Winnipeg. Le frottement latéral estimé a été comparé au frottement latéral mesuré à partir d'essais de charge statique d'un pieu d'acier en H instrumenté. Il a été constaté que chaque méthode a considérablement surestimé le frottement latéral des deux mètres de la partie supérieure. La méthode LCPC a dans l'ensemble satisfait au mieux. La méthode UniCone était la moins précise, mais a été calibrée pour mieux servir.

1 INTRODUCTION

The Cone Penetration Test (CPT) and piezocone (CPTu) test are economic, efficient, and supply relatively quick continuous readings of soil behaviour with depth (Mayne 2015). Estimation of pile capacity was one of the earliest applications of CPT data (Eslami and Fellenius 1997). Numerous CPT methods have been developed for estimation of pile capacity which involve calculating and summing shaft capacity (Q_s) and tip capacity (Q_t). One limitation of CPT methods are that they ignore effective stresses, development of excess pore pressures, and dilatancy effects (Eslami and Fellenius 1997). This limitation is avoided in CPTu methods by incorporating the measured pore pressure. The term "cone test" will be used in this paper to inclusively refer to the CPT and CPTu.

All cone test methods are derived from empirical correlation between cone data and measured resistance from load tests of piles (Fellenius 2021). Therefore, these methods are most applicable for the geological conditions and the pile type(s) for which they were developed. The correlation between cone data and load test results are influenced by the interpretation of load test data. This includes the definition of pile capacity for the load test, and whether or not residual stresses were considered in evaluating the shaft versus tip resistance. Therefore, no single cone test method is expected to be accurate for all scenarios because of these limitations. Site-specific correlations may be necessary to improve the accuracy of cone test methods.

Often, deep foundations are used for scenarios where soft, compressible soils overly dense, competent soils or bedrock. This describes the typical conditions in Winnipeg, Manitoba where the stratigraphy generally consists of glaciolacustrine clay deposited from glacial Lake Agassiz, overlying till which can be very dense, over carbonate sedimentary bedrock (Baracos et al. 1983, Skaftfeld 2014). The cone test methods are limited in their applicability for these ground conditions if piles are installed to a greater depth than the cone can be advanced. In these scenarios, the cone data can't be used to interpret a pile capacity reliably. Estimating the unit shaft resistance (q_s) for the upper portions of the pile is still useful information for evaluating pile settlement and drag force for a pile under serviceability conditions. Steel H-piles are a preferred pile type in the Lake Agassiz basin for bridge foundations where accurate interpretation of the resistance distribution of the pile is particularly important for evaluating downdrag and drag force. However, there are no published records of cone test methods for estimating shaft resistance for driven steel H-piles in the Winnipeg region.

The objective of this study was to compare several cone test methods for estimating shaft resistance to determine a suitable existing method for driven steel H-piles in the Lake Agassiz clay near Winnipeg, Manitoba and other locations with consistent stratigraphic conditions. Four CPTu tests were performed at a study site west of Winnipeg. The shaft resistance was calculated using several common CPT methods including the Schmertmann and Nottingham method (Nottingham 1975, Schmertmann 1978), the LCPC method (also known as the French method) (Bustamante and Gianeselli 1982), and the European method (DeRuiter and Beringen 1979). A single CPTu method, the UniCone method (Eslami 1996, Eslami and Fellenius 1997), was also used to estimate shaft resistance. The estimated shaft resistance from these methods was then compared to the measured shaft resistance from static load testing of an instrumented driven steel H-pile at the site.

2 BACKGROUND

The study site is located approximately 2.5 km west of Winnipeg in the Rural Municipality of Headingley, Manitoba. The stratigraphy at the site generally consists of approximately 0.5 m of fill, overlying high plastic glaciolacustrine silty clay to approximately 7.5 m depth, over sandy silt till. The fill layer consisted of crushed limestone and clay. The glaciolacustrine silty clay was heavily over consolidated, stiff, and contained trace amounts of sand and gravel. The till layer is very dense below depths of approximately 8.8 m. The till deposit in the region is highly variable in relative density and permeability and consists of particles ranging from clay to boulder sizes, though is predominantly comprised of silt (Baracos et al. 1983).

The four CPTu tests conducted at the study site included two in a 2019 testing program and two in a 2020 testing program. The four tests are referred to as CPTu 2019-01, CPTu 2019-02, CPTu 2020-01, and CPTu 2020-02. The cone was advanced to approximately 8.8 m depth for each test to the very dense till, after which the cone could not be penetrated further. The profile of the cone tip resistance (q_c), sleeve friction (f_s), friction ratio (R_f), and pore pressure measured at the cone shoulder (u_2) are shown in Figure 1 for each CPTu test. A greater increase in u_2 below 4 m depth in the silty clay was observed from the 2020 testing program than the 2019 program data. It is possible that the filter stone of the cone was not completely saturated for the 2019 testing.

The soil classification based on CPTu data is shown in Figure 2 based on the UniCone method (Eslami 1996, Eslami and Fellenius 1997) and the normalized soil behaviour type (SBTn) (Robertson 1990). Both the fill and till are highly variable in their classification. The glaciolacustrine silty clay layer is generally classified as “clay” or “silty clay, stiff clay and silt” from the UniCone method and “clay to silty clay” or “clayey silt to silty clay” from the SBTn. A thin layer of sand or gravel was identified at approximately 6.5 m depth of CPTu 2020-01. The transition zone between clay and till can occasionally have till lenses in the clay (Baracos et al. 1983). This layer was not identified at the other CPTu test locations.

The test pile for the study consisted of a steel HP 310x94 pile. The pile was instrumented with arc-weldable vibrating wire strain gauges (RST Instruments model VWSG-A). The strain gauges and their cables were protected with a steel angle welded along the flange of the pile. The pile was driven with a Junttan HHK 5S hydraulic hammer and was installed to be end bearing on the very dense till. The pile was driven to 9.1 m depth with an energy of 14 kJ at end of initial driving. Details on the instrumentation program are described by (Bartz and Blatz 2021).

The test pile program was also developed for measuring the performance of piles when subject to ground settlement. The test pile was installed in September, 2019 and an embankment was constructed surrounding the pile in October, 2019 to induce ground settlement. The embankment was constructed of crushed limestone, was approximately 1.5 m thick, and approximately 8 m by 9 m at the crest with 2 horizontal : 1 vertical side slopes. A static

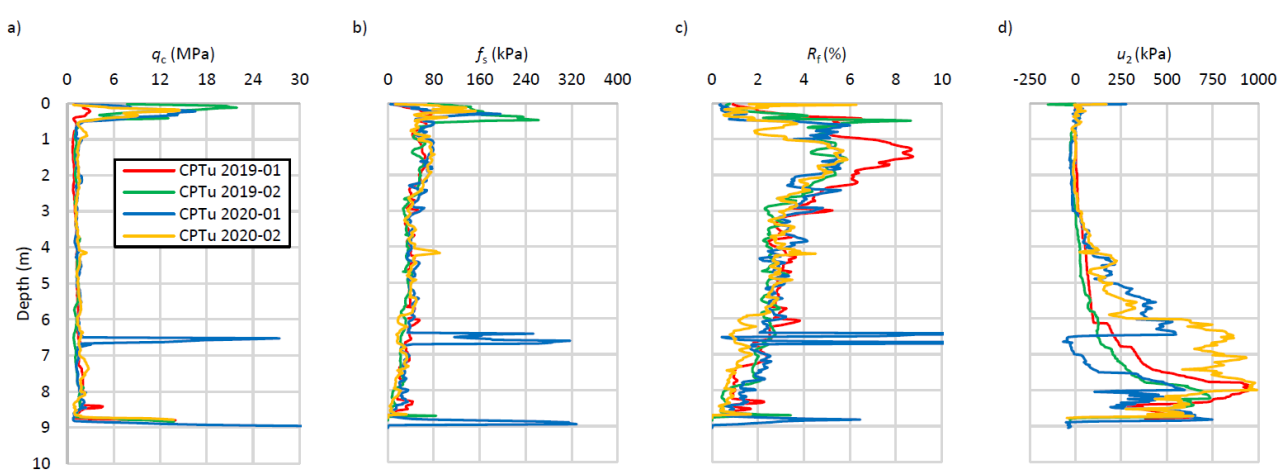

Figure 1: Basic CPTu data. (a) Cone tip resistance. (b) Sleeve friction. (c) Friction ratio. (d) Pore pressure at cone shoulder.

load test was performed in October, 2020. Observations of the performance of the test pile when subject to ground settlement are summarized by Bartz (2021). The construction of the embankment is relevant to this study comparing CPT/CPTu methods because the embankment resulted in a change of effective stress. CPTu 2019-01 and CPTu 2019-02 were conducted prior to pile installation. CPTu 2020-01 and CPTu 2020-02 were conducted after construction of the embankment and outside the footprint of the embankment.

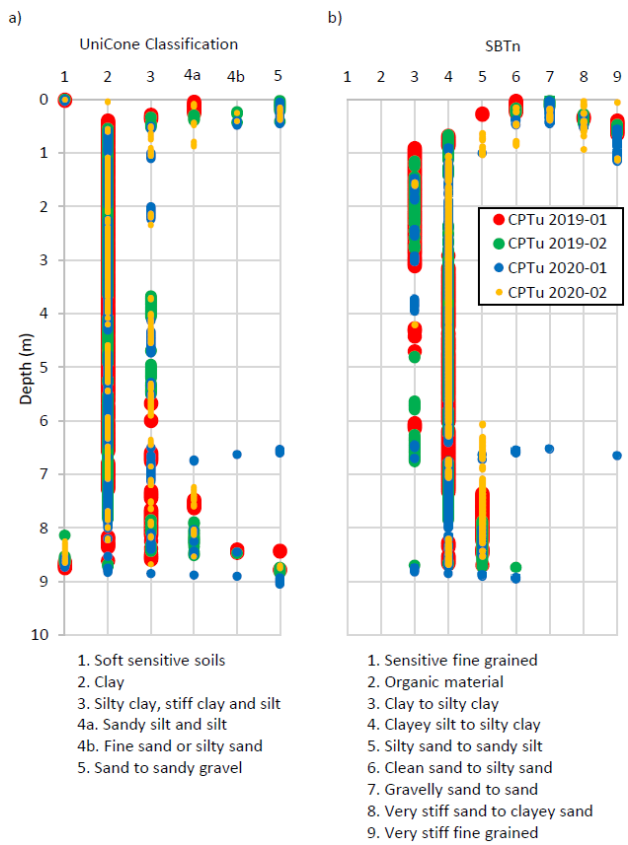

Figure 2: CPTu soil classification. (a) UniCone method. (b) Normalized soil behaviour type (SBTn).

3 CPT AND CPTU PILE DESIGN METHODS

Several cone test methods were selected for comparison to the actual measured shaft resistance from static load testing. The comparison in this study is not exhaustive of all methods, rather several common methods were compared. Also, the comparison is limited to estimation of shaft resistance only. Most methods require data from several pile diameters below the pile tip elevation to estimate tip resistance. The cone could not be penetrated deep enough into the very dense till to employ calculations of tip resistance.

The value of q_s is calculated from the Schmertmann and Nottingham method as:

$$q_s = K f_s \quad [1]$$

where K is a dimensionless coefficient. An upper limit of q_s of 120 kPa is imposed. For clays, K ranges from 0.2 to 1.25 and depends on pile material and f_s. In sand, K is a function of the embedment ratio (d/b) (where d is depth and b is pile diameter). The value of K is linearly interpolated from 0 at ground surface to 2.5 at depth of 8 d/b, then decreases with depth from 2.5 to 0.891 at depth of 20 d/b. Alternatively for sands, q_s can be calculated as:

$$q_s = C q_c \quad [2]$$

where C is a dimensionless coefficient ranging from 0.008 to 0.018 depending on pile type.

The value of q_s is calculated from the LCPC method using Equation 2. The value of C is dependent on q_c, the soil type, and pile type and ranges from 0.005 to 0.033. Upper limits of q_s are imposed ranging from 15 kPa to 120 kPa depending on q_c and soil type.

The European method calculates q_s in sand as the smaller value of f_s and $q_c/300$. For clays, q_s is calculated from:

$$q_s = \alpha s_u = \alpha (q_c / N_k) \quad [3]$$

where α is an adhesion factor, s_u is undrained shear strength, and N_k is a dimensionless coefficient usually equal to 20. The value of α is equal to 1.0 for normally consolidated clay and is equal to 0.5 for overconsolidated clay. An upper limit for q_s of 120 kPa is imposed for the European method.

The UniCone method incorporates the measured pore pressure in the analysis. The value of q_s is calculated from:

$$q_s = C_s q_E \quad [4]$$

where q_E is an "effective" cone resistance ($q_E = q_t - u_2$, where q_t is the cone stress adjusted for pore pressure on the cone shoulder). C_s is a shaft correlation coefficient ranging from 0.004 to 0.08 depending on the soil type determined from the UniCone profiling method. The soil type is classified from q_E and f_s as the following types: 1) soft sensitive soils; 2) clay; 3) silty clay, stiff clay and silt; 4a) sandy silt and silt; 4b) fine sand or silty sand; and 5) sand to sandy gravel.

4 RESULTS

The calculated shaft resistance for the Schmertmann and Nottingham method, LCPC method, European method, and UniCone method are shown in Figures 3 through 6, respectively. Also shown in these figures for comparison is the actual shaft resistance from the static load test at the

site. The pile was installed to approximately the same depth that the CPTu tests were advanced so a direct comparison can be made. The static load test results were presented by Bartz (2021). The cumulative shaft resistance is presented in Figures 3 through 6 and Q_s can be interpreted from the cumulative shaft resistance at the pile tip. Also shown is q_s with depth. Lastly, an equivalent beta coefficient (β) is presented to correlate q_s to the vertical effective stress (σ'_z). β was calculated according to:

$$\beta = q_s / \sigma'_z \quad [5]$$

The distribution of σ'_z was not consistent along the pile shaft during the static load test compared to σ'_z with depth at the time of CPTu testing because of construction of the embankment. The surcharge loading was considered to calculate σ'_z with depth to calculate β for the static load test results. The surcharge load was calculated using Boussinesq stress theory and considering the location of the pile within the footprint of the surcharge load. Boussinesq stress theory was also used to calculate surcharge stress with depth for CPTu 2020-01 and CPTu 2020-02 because these tests were completed following construction of the embankment. These CPTu tests were completed outside the footprint of the embankment and the surcharge stress was relatively minor.

The CPT methods compared in this study each separate soil type into clay or sand. The fill and till at the study site are not accurately described as either clay or sand soil type. For the purpose of this study, q_s was calculated according to the equations for a sand type for the fill and till because cohesionless soil was present in these soil layers. The transition of stratigraphic layers was interpreted from the UniCone and SBTn classifications. The UniCone soil classification was used for calculating q_s for the UniCone method. The cone data was not filtered or smoothed prior to calculating q_s. The randomly distributed extremes in the data have a minor effect on calculating shaft resistance and attempts to filter or smooth the data are subjective (Fellenius 2021).

Q_s is approximately equal to 270 kN as measured from the static load test. This was calculated by assuming that q_s measured from the bottom two strain gauge elevations is constant to the pile tip. The pile tip is at 9.1 m below original grade and the bottom strain gauges are 0.6 m and 0.9 m from the pile tip. The range of Q_s calculated for each method from the four cone tests is summarized in Table 1. Q_s was underestimated with the LCPC method and was overestimated with the other methods. The estimated Q_s using the UniCone method was greater than two times the actual magnitude. The shaft resistance distribution of the LCPC method shown in Figure 4a most closely resembles the actual distribution.

Table 1. Shaft capacity from CPT/CPTu methods for four tests.

Method	Shaft Capacity (kN)	
	Range	Average
Actual (static load test)	270	270
Schmertmann and Nottingham	330 - 380	350
LCPC	190 - 250	220
European	280 - 400	330
UniCone	590 - 680	620

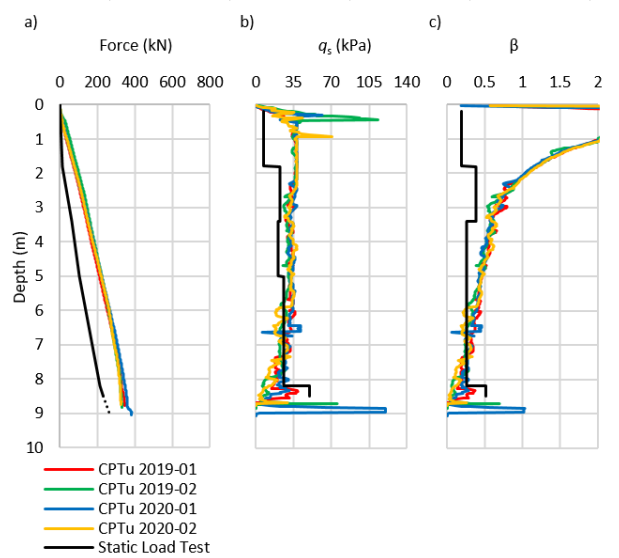

Figure 3: Calculated shaft resistance using Schmertmann and Nottingham method. (a) Cumulative shaft resistance. (b) Unit shaft resistance. (c) Equivalent beta.

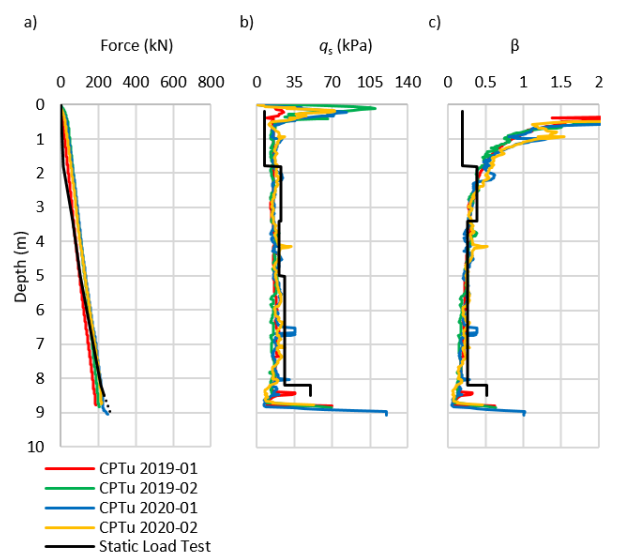

Figure 4: Calculated shaft resistance using LCPC method. (a) Cumulative shaft resistance. (b) Unit shaft resistance. (c) Equivalent beta.

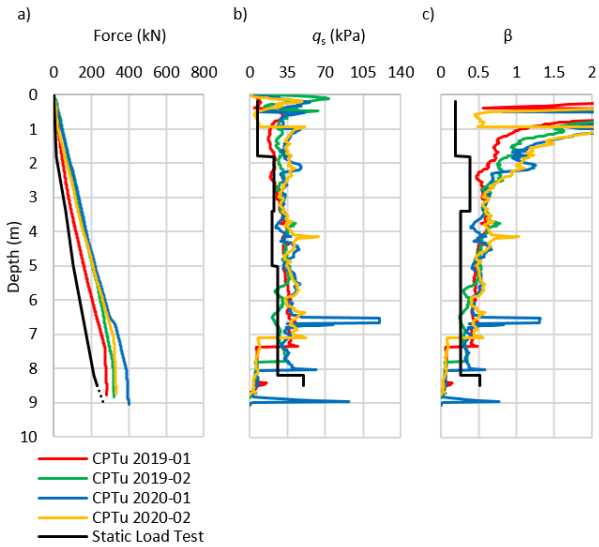

Figure 5: Calculated shaft resistance using European method. (a) Cumulative shaft resistance. (b) Unit shaft resistance. (c) Equivalent beta.

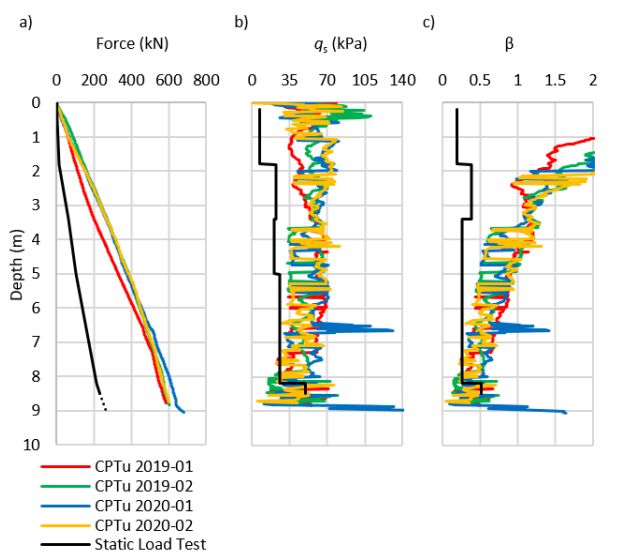

Figure 6: Calculated shaft resistance using UniCone method. (a) Cumulative shaft resistance. (b) Unit shaft resistance. (c) Equivalent beta.

The calculated q_s and β were over predicted in the upper 2 m for each cone test method compared to the actual measurements from the static load test. The fit improved for the CPT methods below the upper 2 m. q_s and β were overestimated with the UniCone method in the silty clay. Also, q_s and β fluctuate within the silty clay due to fluctuating classification of soil type from the UniCone classification. The ratio of calculated q_s to measured q_s was evaluated to further compare the accuracy of the cone test methods. A histogram of this ratio, calculated for each data point of CPTu 2020-02 between 2m to 7.1 m depth, is shown in Figure 7. This representative data set was

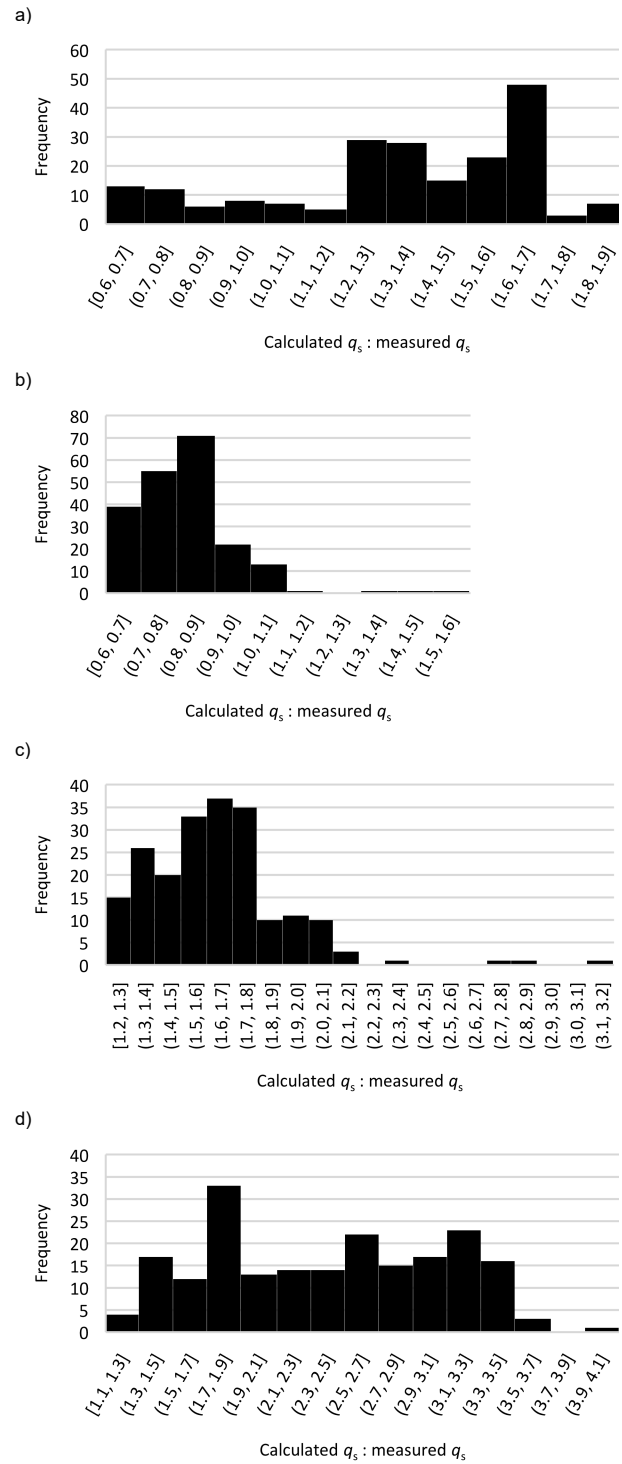

Figure 7: Ratio of calculate q_s to measured q_s for silty clay between 2 m and 7.1 m depth from CPTu 2020-02. (a) Schmertmann and Nottingham method. (b) LCPC method. (c) European method. (d) UniCone method.

selected to compare the accuracy of the methods in the silty clay layer. The upper 2 m was not accurate for any method and was neglected in Figure 7. The soil type

transitioned to till around approximately 7.1 m depth as interpreted for the CPTu data. The mean ratio of calculated q_s to measured q_s is nearest to 1.0 for the LCPC method. The LCPC method generally under predicts q_s and the other methods generally over predict q_s.

5 DISCUSSION

The purpose of this study was to compare existing cone test methods for estimating shaft resistance of a driven steel H-pile in silty clay near Winnipeg, Manitoba. The calculated Q_s was underestimated using the LCPC method and was overestimated with the other CPT methods and the UniCone method. The calculated q_s was overestimated with each method for the upper 2 m of the pile length. Below this upper 2 m, the estimated q_s for the CPT methods was improved. The estimated q_s using the UniCone method was highly variable due to the varying soil classification.

The actual q_s ranged from approximately 20 to 26 kPa from 1.8 m to 8.2 m depth from strain gauge readings during the static load test. The measured q_s in the upper 1.8 m was notably lower with an average of approximately 7 kPa. A corresponding lower q_c and f_s was not observed at these shallower depths however, resulting in overestimating q_s near ground surface. The high plastic glaciolacustrine clay near Winnipeg is known to undergo significant volume change with changes in water content (Baracos et al. 1983, Skaftfeld 2014). In local practice, the upper portion of shaft resistance is often neglected in pile design to account for the potential of shrinkage and a loss of shaft resistance (Skaftfeld 2014). It is possible that the silty clay near ground surface experienced shrinkage due to desiccation prior to the static load test resulting in a low q_s relative to the q_c and f_s cone data. The groundwater table was approximately 2.5 m below ground surface at the time of the static load test. Therefore, the upper 2.5 m could be expected to undergo changes in water content due to seasonal effects. Seasonal changes in water content were not measured to confirm this theory.

The UniCone method overestimated q_s in the silty clay and there was significant fluctuation in q_s because of the fluctuating soil classification. The UniCone classification in this stratigraphic layer straddled between "clay" and "silty clay, stiff clay and silt" which have C_s of 0.05 and 0.025, respectively. Fellenius (2021) recommends applying a constant C_s to a soil layer where the CPTu data is grouped together on a classification chart and straddles a boundary between soil types. The "silty clay, stiff clay and silt" description is more representative of this stratigraphic layer based on observations from a conventional drilling, sampling, and lab testing program. Therefore, a single C_s for the soil layer is expected to be closer to 0.025. A site-specific C_s of 0.018 was found to provide a close fit for q_s and β for this study. The calculated q_s and β are shown in Figure 8 for the silty clay layer. A value for C_s of 0.018 was applied below the fill layer to the top of the till layer. Erroneous estimates are expected around 6.5 m depth for CPTu 2020-01 where a sand or gravel layer was identified. Figure 9 shows a histogram of the calculated q_s to measured q_s for CPTu 2020-02 for the silty clay layer between 2 m to 7.1 m depth. The mean ratio of 1.028 is near unity, indicating this calibrated analysis resulted in an improved estimated of q_s. Site specific coefficients could similarly be created to improve the correlation of the CPT methods. This exercise was only completed for the UniCone method because it has the advantage of incorporating the pore pressure.

Eslami and Fellenius (1997) compared their UniCone CPTu method for calculating pile capacity to static load test results for 102 case histories. This comparison included 24 case histories where Q_s and Q_t were separated in the static load test analysis for detailed correlation of q_s and q_E. The C_s reported for the "silty clay, stiff clay and silt" soil type ranged from 0.0206 to 0.028. The site-specific C_s of 0.018 for driven steel H-piles in Lake Agassiz silty clay is lower than the range reported by Eslami and Fellenius (1997). The 24 case histories analyzed by Eslami and Fellenius (1997) did not include examples of H-piles in the "silty clay, stiff clay and silt" soil type. It is reasonable that a low-displacement pile such as an H-pile could have a lower q_s, and therefore a lower C_s for correlation to q_E.

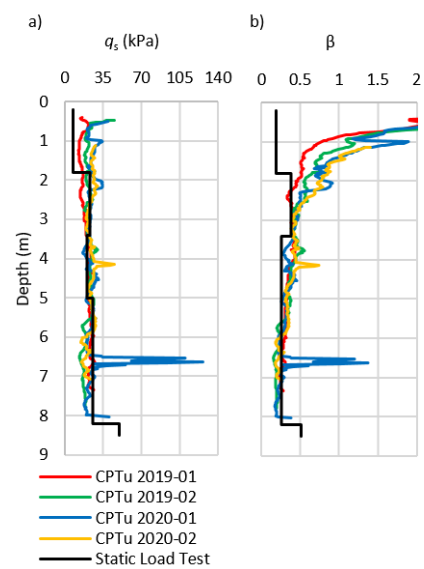

Figure 8: Calibrated UniCone method for silty clay. (a) Unit shaft resistance. (b) Equivalent beta.

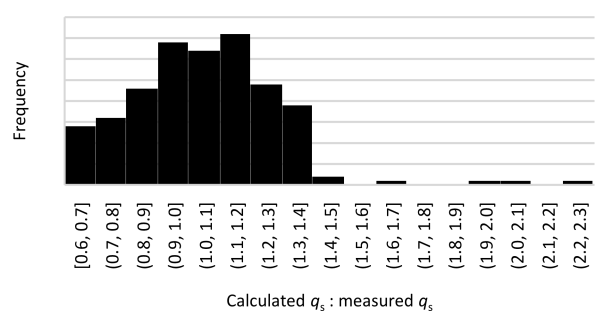

Figure 9: Ratio of calculated q_s to measured q_s using calibrated UniCone method for silty clay between 2 m and 7.1 m depth from CPTu 2020-02.

6 CONCLUSIONS

Three CPT methods and one CPTu method were compared to the measured shaft resistance from static load testing of a driven steel H-pile in Lake Agassiz glaciolacustrine silty clay near Winnipeg, Manitoba. The following key conclusions are drawn from this study:

1. All the methods overestimated q_s in the upper 2 m of the pile. This is possibly due to shrinkage of the silty clay above the water table resulting in a decrease in q_s during the static load test.
2. The LCPC method provided the best estimate of q_s and the shaft resistance distribution of the select cone test methods compared in this study. The UniCone method considerably overestimated Q_s with an estimate greater than two times of Q_s measured from the static load test.
3. The UniCone method provided a good match of q_s in the silty clay by applying a constant C_s of 0.018 throughout the entire soil layer.

This study demonstrates the suitability of several CPT methods for driven steel H-piles in Lake Agassiz silty clay and provides recommendations for calibrating the UniCone CPTu method. The study demonstrated that cone test methods can overestimate shaft resistance near ground surface of expansive clays. It is important to be aware of the potential to overestimate shaft resistance near ground surface as this may impact calculations of pile capacity. Also, an improved and more accurate interpretation of the shaft resistance is useful for estimating pile settlement and drag force under serviceability conditions. These findings are beneficial for pile design in the Winnipeg region and similar geological conditions.

7 ACKNOWLEDGEMENTS

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