Correlation between the PDA-Derived Pile Side Resistance and Undrained Shear Strength of Clays in Alberta

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ABSTRACT
For many centuries, piles have been used to support structures; however, challenges still arise to accurately predict the ultimate axial capacity of a pile. The main challenges are due to the inconsistent soil conditions, pile installation effects, and errors by site investigation techniques. The goal of this study was to improve the accuracy of design methods for the pile side resistance in cohesive soils by proposing a new approach. The new design method attempts to empirically consider the pile length and load-transfer effects. A database of 128 driven pipe piles was collected from a site in Alberta and applied in this study. The piles were subjected to Pile Driving Analyzer (PDA) tests, and the surrounding soil was investigated with borehole drilling and pocket pen measurements. The soil stratigraphy at this site is relatively consistent, composed of sand underlain by stiff clay (till). A study was conducted to correlate the side resistance obtained from PDA tests to the undrained shear strengths measured by pocket pens. A new design method was proposed with a coefficient of determination (R^2) of 0.81 between the measured and predicted unit side resistances, and it offered significant improvements compared to existing methods. The findings will assist practitioners to deliver more reliable pile designs, particularly in Alberta soils.

RÉSUMÉ
Pendant des siècles, les pieux ont été utilisés pour supporter des structures; Cependant, il reste encore du mal à prévoir avec précision la capacité axiale ultime d'un pieu. Les principales difficultés sont dues aux conditions de sol incohérentes, aux effets de l’implantation des pieux et aux erreurs des techniques d’investigation sur site. Le but de cette étude était d’améliorer la précision des méthodes de calcul de la résistance latérale des pieux dans les sols cohérents en proposant une nouvelle approche. La nouvelle méthode de conception a pris en compte de manière empirique les effets de longueur de pieu et de transfert de charge. Une base de données de 128 pieux tubulaires entraînés a été collectée sur un site en Alberta et utilisée dans cette étude. Les pieux ont été soumis à des tests de Pile Driving Analyzer (PDA) et le sol environnant a été étudié à l'aide de sondages de forage et de mesures au stylet de poche. La stratigraphie du sol sur ce site est relativement consistante, composée de sable reposant sur une argile raide (till). Une étude a été menée pour corréler la résistance latérale obtenue à partir des tests PDA aux résistances au cisaillement non drainées mesurées par les stylos à poche. Une nouvelle méthode de conception a été proposée, avec un coefficient de détermination (R^2) de 0,81 entre les résistances latérales de l'unité mesurée et prévue, et elle offre des améliorations significatives par rapport aux méthodes existantes. Les résultats aideront les praticiens à fournir des conceptions de pieux plus fiables, en particulier dans les sols de l'Alberta.

1 INTRODUCTION

Although pile foundations have been used commonly to support structures, a lot of uncertainties and variabilities in design still exist to estimate the axial capacity. These uncertainties reduce the reliability of a design and have an economic impact during construction (Hannigan et al. 2016; Kraft et al. 1981). They are contributed by the soil conditions and installation effects, which can change with time. Existing design methods have been developed from various soil conditions, setup times, and pile types (Kraft et al. 1981; Kulhawy and Jackson 1989; Meyerhof 1976; Semple and Rigden 1986; Vijayvergiya and Focht 1972). Due to the lack of costly and time-consuming tests with instrumented piles, most design methods were generated with the average soil conditions as conventional top-down pile tests were investigated. Site conditions, however, are rarely consistent, especially if heterogeneous glacial tills are present. Using the average soil conditions also does not fully consider the influence of parameters, such as the shear strength, effective stress, and soil plasticity, with the pile length. Despite decades of research, a single universal formula is not available in practice due to the complexity of the pile-soil interaction.

This paper presents a correlation study that was conducted on data from site investigations and a total of 26 piles tested with high-strain dynamic-loads, particularly Pile Driving Analyzer (PDA) tests at the end of initial drive (EOID). PDA testing is faster, more cost efficient, and more reliable than traditional static load tests, and the method estimates the static pile capacity and resistance distribution (Hannigan et al. 1997). In this paper, the side resistance from PDA tests were correlated with the soil conditions from a site in Alberta, Canada to propose a new design method for clays. Since the side resistance distribution is simulated, PDA testing becomes a practical technique to verify design assumptions for local soil conditions and can help designers to limit the uncertainty.

2 RESEARCH BACKGROUND

2.1 Pile Capacity from a Pile Driving Analyzer

The ultimate capacity (Qu) of a pile under compression loading is composed of two mechanisms: the side resistance and tip resistance.

$$Qu = q_s A_s + q_p A_p \qquad [1]$$

Where A_s is the skin area of the pile, A_p is the area of the pile tip, and q_s and q_p are respectively called the unit side resistance and the unit tip resistance of the pile.

From a PDA test, the static pile resistances can be estimated. The PDA test is conducted by applying a large mass onto the top of the pile foundation and measuring the applied force and velocity by use of accelerometers and strain transducers attached to the pile (Hannigan et al. 1997). The pile resistances are then determined by matching simulated stress waves to the measured stress waves (Hannigan et al. 1997). The testing process is quick which allows the pile capacity to be estimated immediately after pile installation (End of Initial Drive, EOID) and several days after the pile is installed (Beginning of Restrike, BOR). For this paper, the focus was on the q_s from EOID measurements; thus, the short-term conditions were considered, and the set-up effects were ignored.

2.2 Current Pile Design Methods

Design methods that correlate the q_s with soil parameters are divided into total stress methods and effective stress methods. The total stress methods, or *α* Methods, are intended for the short-term conditions of cohesive soils (Kulhawy and Jackson 1989; Karlsrud et al. 2005).

$$q_s = \alpha\, C_u \qquad [2]$$

Where α is an adhesion factor and C_u is the undrained shear strength. The effective stress methods are usually called *β* Methods and are for cohesionless soils or the long-term conditions of cohesive soils (Meyerhof 1976; Vesic 1977).

For α Methods, numerous approaches have been developed over the years, and they include a range of soil properties and design situations. Kulhawy and Jackson (1989) proposed an α Method that is a function of C_u, and it was later adopted by the Canadian Foundation Engineering Manual (CFEM) (Canadian Geotechnical Society (CGS) 2006). Semple and Rigden (1986) and Kolk and Van der Velde (1996) improved the method by including the effect of the pile length and strength ratio (Cu/σ'), where σ' is the effective stress. Karlsrud et al. (2005) provided correction factors for the pile geometry and noticed soils with a lower plasticity index (PI) had a significantly lower α compared to Kulhawy and Jackson (1989). The development of these design methods shows that the soil-pile interface is dependent on many factors.

2.3 Influence of the Pile Length

The influence of the pile length may not be very distinct, but it is related to several factors: disturbance of the soil fabric, 3-D pile movements, and site variability. While a pile is being driven, the soil fabric towards the pile top is likely more disturbed than the soil at the pile base. Depending on the variation of the soil stiffness with depth, the load-transfer and mobilized side resistance will vary along the pile (Kraft et al. 1981). 3-D pile movements include the lateral movement, or “whipping”, of the pile during the driving process, which can weaken the surrounding soil (Semple and Rigden 1986). In addition to land formation and soil stratigraphy, soil properties may vary spatially due to seasonal changes or soil-drag down (Kraft et al. 1981). Particularly for offshore foundations, design methods have been progressing to include the influence of the pile length. One of the famous approaches is the λ Method by Vijayvergiya and Focht (1972), which is a hybrid between total and effective stress methods. Unfortunately, many of the developed design methods have been based on the average soil conditions and likely idealize the soil properties.

3 RESEARCH METHODOLOGY AND RESULTS

3.1 Overview of the Methodology

The goal of this research was to develop a new design method for the side resistance of driven steel pipe piles for axial compression loads. The methodology will involve 4 steps. (1) PDA EOID test results and soil conditions from a site in Alberta were collected from the database by Peter Kiewit Sons ULC. (2) The quality of data was checked, and the PDA-derived q_s was matched with the soil conditions from the nearest borehole logs. (3) A correlation study was conducted to predict q_s from the Cu measured from pocket pens. (4) The accuracy and variability of the developed design method was compared to existing design methods.

3.2 Tested Piles

Table 1 shows the list of open-ended steel pipe piles that were selected for this study. The outside pile diameters range from 406 mm to 762 mm, and the embedment lengths vary from 15 m to 22 m with an average length of 19 m.

3.3 Soil Conditions of Test Site

Figure 1 shows an example of soil conditions determined from borehole logs. The soil stratigraphy at this site is relatively consistent and is composed of alternating layers of sand underlain by firm clay. A stiff clayey till is located under the firm clay.

Several borehole logs were collected for the geotechnical site investigation. This includes the shear strength measurements by pocket pens for cohesive soils and soil descriptions and classifications according to the modified Unified Soil Classification System (modified USCS). The main difference of the modified USCS to the classical USCS is the plasticity chart and the addition of an intermediate plastic clay (CI), which has a liquid limit between 30 to 50. The clays ranged from CL to CH. The tills were commonly classified as CI. The sands were usually silty (SM) or poorly graded (SP). Atterberg and sieve testing was occasionally conducted at the site, and the groundwater table was usually high with a depth between 1 m to 3 m from the ground surface.

3.4 Criteria for Data Selection for the Analysis

The correlations between variables will highly depend on the data. The data was selected for the analysis if they met the following conditions.

- During the PDA test, the dropped mass should apply a large enough load to generate mobilization at the pile tip. Test results were selected if the simulated load-displacement response at the pile tip did not remain in the elastic condition and developed sufficient plunging displacement.
- Cu measurements greater than 250 kPa were not available as the pocket pen could not measure values that exceeded this strength.
- Pile tests were matched with nearby borehole logs if the borings were less than 40 m away. Although more borehole logs and pile tests were available, the spatial variability can be limited by relying on nearby borehole logs.
- The PDA-derived q_s was commonly reported for every 1 m of depth along the pile, and soil conditions from the borehole logs were matched to the q_s if the vertical distance was within 0.5 m.

3.5 Relationships with the Side Resistance

The α factor can be calculated by Equation 2 since the unit side resistance is known by the PDA results. After comparing Cu to α in Figure 2, the level of variability at the site can be seen. The adhesion factor appears to decrease with increasing Cu, as seen by many references. The soils are separated by their classifications to further help identify trends. The CL have the lowest α on average, and the CI and tills usually have the highest α. Karlsrud et al. (2005) suggested the side resistance is a function of plasticity, but the side resistance does not appear to be heavy influenced by the PI due to the scatter in the data. In addition, the CI and tills, which are also a CI, should overlap. In Figure 2,

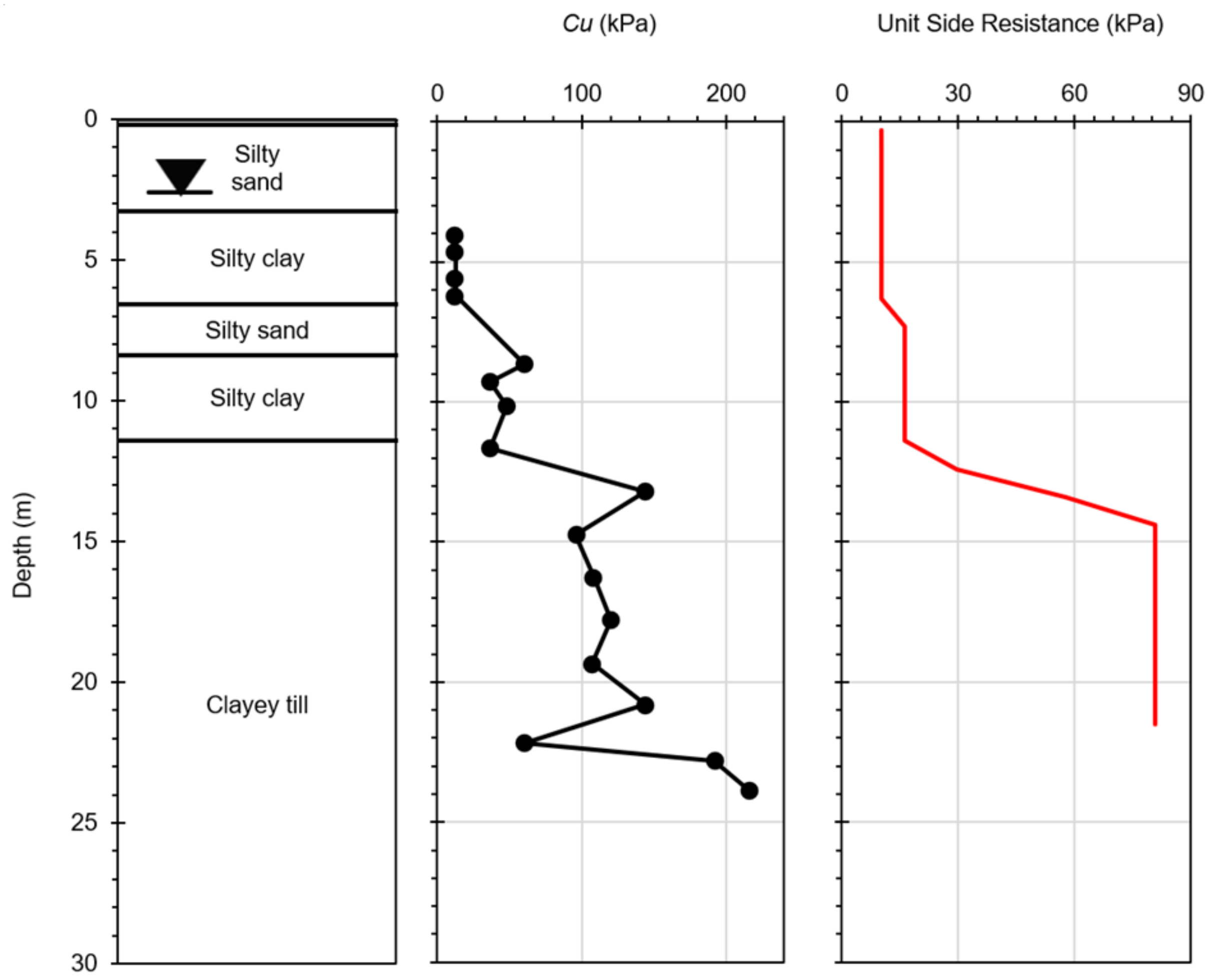

Figure 1. Example of Site Conditions and PDA-Derived Unit Side Resistance

Table 1. Capacity and Geometry of Piles Used in This Study

Pile No.	Pile Size[1] (mm)	Embedment Length (m)	Shaft Capacity[2] (kN)
1	406 x 9.5	16.8	1314
2	762 x 15.9	21.8	1990
3	762 x 15.9	21.5	2123
4	610 x 12.7	20.2	2316
5	610 x 12.7	20	1345
6	406 x 9.5	16.3	806
7	508 x 12.7	18	1246
8	508 x 12.7	22	1513
9	508 x 12.7	20.3	2390
10	610 x 12.7	20	1294
11	610 x 12.7	20	1855
12	610 x 12.7	16.5	1047
13	762 x 15.9	20.8	2070
14	610 x 12.7	18	1158
15	762 x 12.7	16	1190
16	762 x 12.7	18.3	1686
17	508 x 12.7	21.3	2037
18	610 x 12.7	21	1779
19	508 x 12.7	20	1137
20	508 x 12.7	18.5	1045
21	762 x 12.7	20	1677
22	508 x 12.7	16.3	901
23	762 x 12.7	20	940
24	762 x 15.9	18.3	1405
25	610 x 12.7	18.5	1053
26	406 x 9.5	15.3	988

[1]Dimensions are outside diameter x wall thickness; [2]PDA derived

the α Method adopted by CGS (2006) seems to offer an average trend. The method will underestimate the resistance in the till but overpredict the resistance in the other clays.

The differences between the α values is attributed to the soil location, as shown previously in Figure 1, and the level of disturbance on the soil fabric. The clays, other than the tills, are closer to the ground surface, while most piles begin to penetrate the till stratum. Figure 3 shows the normalized unit side resistance (q_s/q_{max}) to the normalized depth (z/Lcr), where q_{max} is the maximum side resistance along the pile, z is the depth, and Lcr is the length along the pile from the ground surface to reach q_{max}. For a z/Lcr ratio less than 0.5, the soil possesses a unit side resistance that is usually less than 20% of the maximum. The depth of Lcr roughly corresponds to a penetration of 1 m into a stiff soil stratum, which is the glacial till. The distance between Lcr to the pile base ranges from 0.3 m to 7.1 m. The shortest piles with shallow penetrations into the till have their q_{max} very close to the pile base. Once Lcr (z/Lcr =1) is reached, q_s remains constant near q_{max}.

Figure 4 again shows the relationship between α and Cu, but the soils have been classified by their location above ($z/Lcr < 1$) or below ($z/Lcr \geq 1$) Lcr. The adhesion values are likely lower for the clays above Lcr because the peak shear strength or critical state has been achieved, and the post-peak soil strength then resists the applied pile

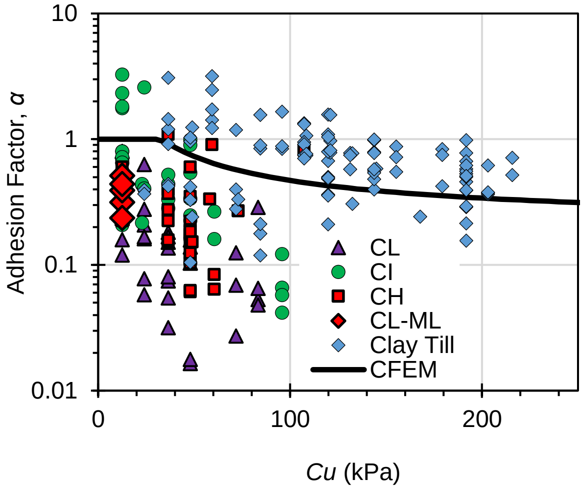

Figure 2. Relation between Adhesion Factor and Undrained Shear Strength by Soil Type

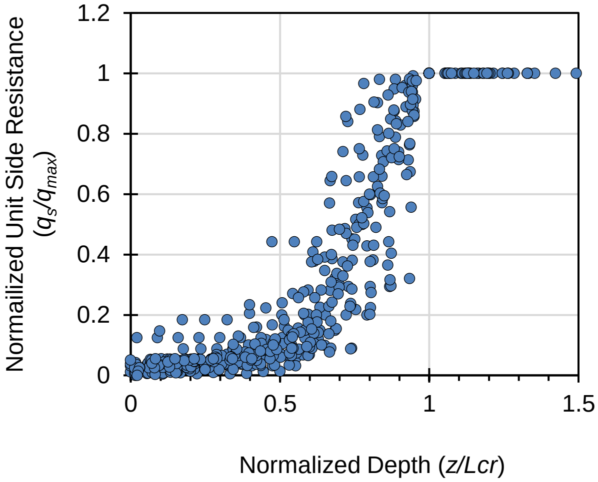

Figure 3. Relation between Normalized Unit Side Resistance and Depth

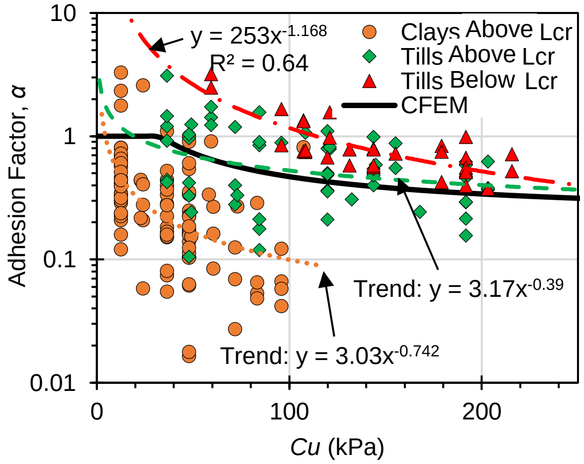

Figure 4. Relation between Adhesion Factor and Undrained Shear Strength by Location to Critical Depth

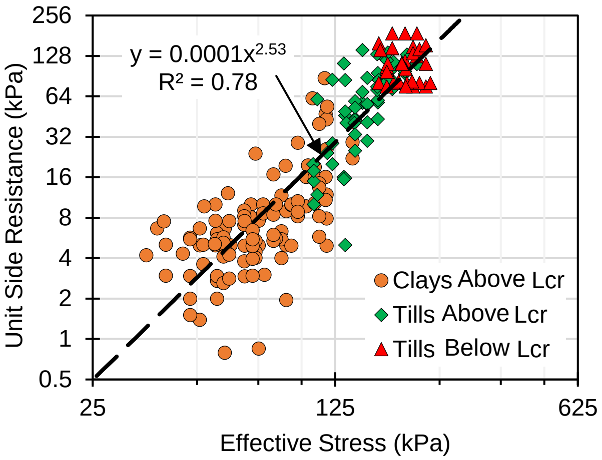

Figure 5. Relation between Side Resistance and Effective Stress by Location to Critical Depth

load. Due to the consistent stratigraphy at the site, tills were the only soil located below Lcr, and they usually had an α near a value of one, which may indicate that the peak shear strength was not surpassed. Thus, the side resistance likely depends on the soil-pile stiffness and load transfer conditions.

For Cu less than 150 kPa, the α values sometimes exceeded a value of one. This usually occurs with weaker clays that are located at greater depths and exposed to greater confinement stresses. Figure 5 shows the unit side resistance has a strong positive relationship with the effective stress, especially for soils above Lcr. The q_s rarely surpasses the effective stress.

From nonlinear least-squares regression, the proposed design method was created by combining the most influential parameters:

$$q_s = 49.88 \cdot \operatorname{atan}(0.018 \cdot ((z/Lcr) \cdot Cu^{0.039} \cdot \sigma' - 128)) + 58 \qquad [3]$$

Several combinations and relations, such as power, logarithmic, and simple arithmetic operations, with the various parameters were considered to predict q_s. The best fit for q_s was achieved with an arctangent relationship with z/Lcr, which is similar to Figure 3. For q_s and the three influential variables, the proposed equation passes through the origin (0,0), and the q_s will increase as either z/Lcr, Cu, or σ' increases. Equation 3 is mostly sensitive to z/Lcr and σ'.

3.6 Comparison between Proposed and Existing Methods

The performance of the proposed method was compared to existing methods. The studied existing methods, namely the α Methods from CGS (2006), Kolk and Van Der Velde (1996), and Karlsrud et al. (2005), apply various parameters and were selected since they could be applied to heterogeneous soils. The equations and details of the design methods are in Table 2.

The α Method adopted by CGS (2006) does not consider the disturbance of the soil fabric or load-transfer within the pile. Figure 6 shows the accuracy of the design method. The black dashed lines indicate a ratio of 0.5:1 (underprediction) and 2:1 (overprediction) between the measured (q_{sm}) and predicted (q_{sp}) unit side resistances. A 1:1 ratio is represented with a solid black line. As shown in the figure with the dashed blue trendline, the design method mainly overpredicts the weaker clays with an average ratio of the predicted to measured unit side resistance (q_{sp}/q_{sm}) of 5.11, and the coefficient of variation (COV) was 126 % between the measured and predicted results. Table 3 shows the details on the accuracy of the design methods with the average q_{sp}/q_{sm}, COV, and

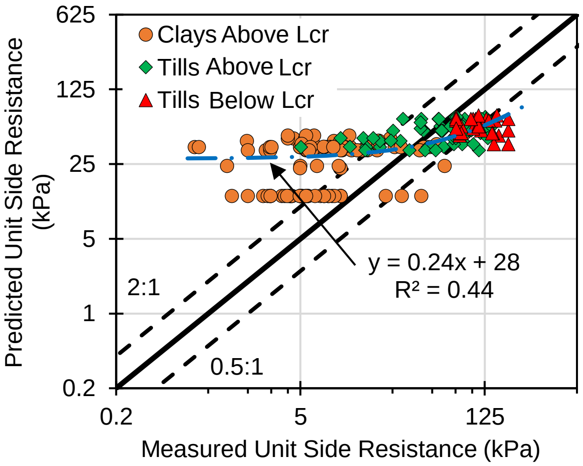

Figure 6. Comparison of Predicted and Measured Unit Side Resistance by CGS (2006) Method

Table 2. Studied Design Methods for the Unit Side Resistance

Reference	Condition	Equation for α
CGS (2006)		$\alpha = 0.21 + (26/C_u) \leq 1$
Karlsrud et al. (2005)[1]	Case 1 (C_u/σ' < 0.25):	$\alpha = 0.32\,(PI - 10)^{0.3} \leq 1$
	Case 2 (C_u/σ' > 1):	$\alpha = 0.5\,(C_u/\sigma')^{-0.3} F_{Tip}$
	Case 3:	Linear interpolate between Case 1 and 2
Kolk and Van Der Velde (1996)[2]		$\alpha = 0.9[(L - z)/D]^{-0.2}(\sigma'/C_u)^{0.3} \leq 1$

[1] F_{Tip} is a factor for the pile tip condition. For uncapped pipe piles, $F_{Tip} = 1$. [2] z is the depth that corresponds to the midpoint of a soil layer along a pile.

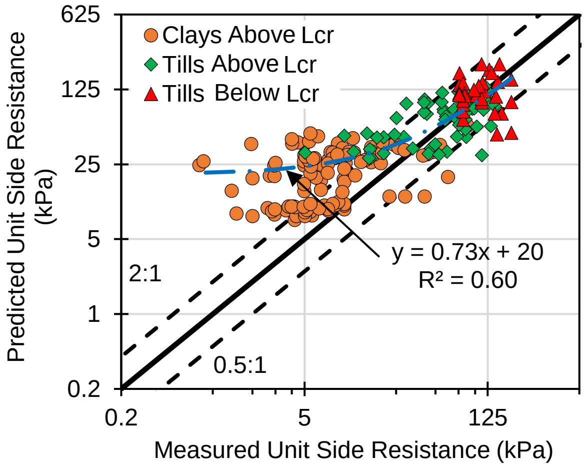

Figure 7. Comparison of Predicted and Measured Unit Side Resistance by Kolk and Van Der Velde (1996) Method

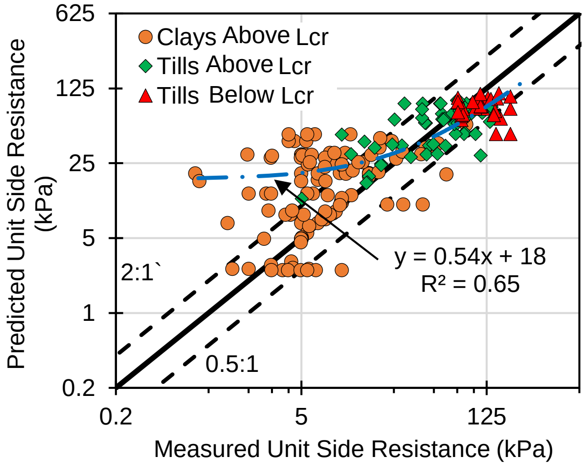

Figure 8. Comparison of Predicted and Measured Unit Side Resistance by Karlsrud et al. (2005) Method

number of samples (n). The clayey tills were underpredicted with an average q_{sp}/q_{sm} and COV of 0.54 and 32.5 %, respectively. For all the soils, the coefficient of determination (R^2) is poor at 0.44. The method also overpredicts by a factor of 3.23 and has a COV of 162 %. Yet, if the average shear strengths were applied, the method would likely be able to predict the total side capacity with a reasonable tolerance.

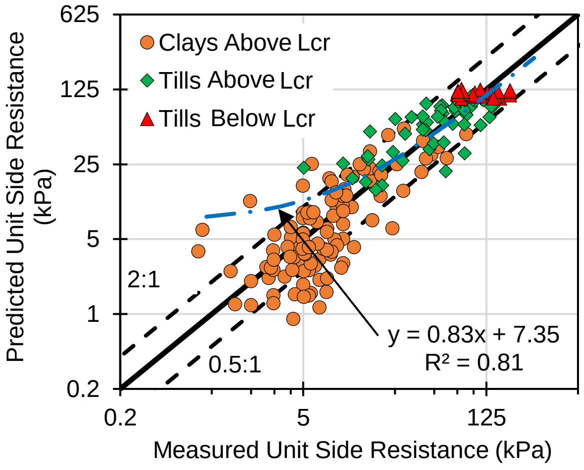

Figure 9. Comparison of Predicted and Measured Unit Side Resistance by Proposed Method

The Kolk and Van Der Velde (1996) Method considers the pile length to calculate the α factor. Figure 7 shows it performs much better than the CGS (2006) Method, particularly for the clays above Lcr and the tills below Lcr. The method still overpredicts the clays above Lcr. It has the second-best average q_{sp}/q_{sm} of 1.14 for the tills below Lcr, but the COV is the highest among the methods at 41 %. Overall, the method provides good results with an average q_{sp}/q_{sm} and COV of 2.73 and 137 %, respectively, for all soils.

In order to calculate the method by Karlsrud et al. (2005), the PI was required. Although PI was not measured for each Cu measurement, reasonable ranges were assumed based on the soil classification and the few Atterberg index tests that were available. It should also be noted that the method does not recommend an α value for a PI less than 10, and low-plastic clays, such as silty clays, would use the suggested curve for a PI of 10.

This method provides mixed results. It has one of the best average q_{sp}/q_{sm}, which is 2.13 for all the soils, but it

Table 3. Accuracy of Predicted Unit Side Resistance by Design Methods

Soil Type	Descriptive Statistic	Proposed Method	CGS (2006)	Kolk and Van Der Velde (1996)	Karlsrud et al. (2005)
Clays Above Lcr (z/Lcr $<$ 1)	Mean	1.11	5.11	3.87	3.04
	COV	94.1	126.4	121.4	126.0
	n	109	109	109	109
Tills Above Lcr (z/Lcr $<$ 1)	Mean	1.27	1.22	1.50	1.20
	COV	55.4	92.5	70.4	62.6
	n	57	57	57	57
Tills Below Lcr (z/Lcr $\geq$ 1)	Mean	1.09	0.54	1.14	0.79
	COV	29.9	32.5	41.1	31.9
	n	31	31	31	31
All	Mean	1.15	3.23	2.73	2.13
	COV	76.0	162.3	137.4	142.4
	n	197	197	197	197

also has one of the highest COVs, as shown in Figure 8. The low average q_{sp}/q_{sm} is likely because the method is more conservative than CGS (2006) by offering lower α values for weak, low to medium plastic clays.

In Figure 9, the predicted and measured unit side resistance is compared for the proposed method. Variability is mainly found with the clays found above Lcr. This is expected among all the methods as the clays near the ground surface were weaker, and the soil disturbance and the uncertainty during installation will be the greatest towards the ground surface. Generally, the COV decreases with depth. For all the soils, the average q_{sp}/q_{sm} and COV was 1.15 and 76 %, respectively. As shown in Table 3, the proposed method performs much better than the existing methods by usually having the average q_{sp}/q_{sm} closest to one and having the lowest COV.

4 CONCLUSIONS AND DISCUSSIONS

In order to reduce the uncertainties of designs, this preliminary investigation proposed a new design method for the side resistance of pipe piles driven in cohesive soils. Equation 3 shows the proposed relationship. This analysis was based on 26 piles with EOID results, and the influence of spatial variability was limited by selecting piles that were less than 40 m away from a borehole log. The number of piles may seem small, but PDA tests can help by considering the pile-soil interaction as it offers a quick way to obtain the distribution of the side resistance immediately after installing a pile.

In addition to the Cu, the proposed method indicates that the side resistance is highly influenced by σ' and the pile-soil stiffness, which was represented by the normalized depth (z/Lcr). Once the Lcr is reached, the unit side resistance should be limited to a constant value. A similar idea, called the critical depth, has been proposed by Vesic (1977) and Meyerhof (1976) for cohesionless soils. For example, Vesic (1977) recommends the critical depth is roughly equal to 15 to 20 pile diameters. However, Lcr is likely not a function of the pile diameter or length but a reflection of the soil profile. For a pile, it is recommended to determine Lcr as the depth to the stiffest stratum. In this case, it was the clayey till. Properly determining Lcr is important as the z/Lcr was empirically fitted based on the trend in Figure 3. Lcr may also be influenced by soil plugging within the pile and soil drag-down during driving, but these factors are difficult to investigate as they were not measured.

For clayey tills that were located below Lcr ($z/Lcr \geq 1$), the following formula from Figure 4 may be used to estimate α:

$$\alpha = 253Cu^{-1.168} \quad [4]$$

In general, for all soils, the predicted unit side resistance should be less than the effective stress. The α values should be less than one, especially for clays located at a z/Lcr less than 0.5. Exceptions may be made for weak clays found at a large depth near Lcr.

There are several limitations to be addressed in the future. Since the pile capacity is a function of the load transfer and the site stratigraphy was consistent, the proposed relationships may vary for other sites. The PDA-simulated load-transfer parameters may be considered for the pile-soil stiffness. In the future, the restrike PDA tests can be included to investigate influence of setup. The plasticity did not appear to influence the side resistance during the analysis, but site conditions can change between regions. It would be ideal to add test results from other sites. A PDA test may not be completely compatible to a static load test, but one of the challenges for a geotechnical engineer is the approach to consider spatial variability. Soils and profiles are not always going to be homogeneous. As demonstrated in this investigation, PDA testing, due to its efficiency, can be a valuable technique to reduce uncertainty in designs for local site conditions.

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