CPT/CPTu Pile Capacity Prediction Methods – Question Time

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ABSTRACT: Numerous CPT/CPTu methods are available for predicting pile capacity. Pile capacity is not, however, a single value, because it is not only linked to the selection of the failure criterion in the load-movement curve, but it is also time-dependent (setup and relaxation). This latter condition is a very important perspective which is, unfortunately, missing from most CPT/CPTu capacity prediction methods. A case history is presented of a pile testing program where pile capacity was determined during initial pile installation, and again periodically for up to 3 years. This testing showed a continued increase in pile capacity with time due to soil setup. A seismic piezocone sounding was performed at the site, and the results were used to predict pile capacity using various published CPT/CPTu methods. As expected, the results indicate differing predicted capacities. Potential modifications to the CPT/CPTu prediction methods are discussed to address the 100-day reference time capacity goal.

1 INTRODUCTION

What is the capacity of a driven pile? Why is there usually a wide range of calculated capacities between various pile capacity prediction methods? Numerous CPT/CPTu methods are available for predicting pile capacity. Pile capacity is not, however, a single value, because it is not only linked to the selection of the failure criterion in the load-movement curve (for example by one of the following methods: s/d = 10%, Davisson, Chin, or Decourt), but it is also time-dependent (setup and relaxation). This latter condition is a very important perspective which is, unfortunately, missing from most CPT/CPTu capacity prediction methods. These methods are generally based on the results of pile static loading tests performed 30 days, or more, after initial pile installation, but no means are usually given to quantify the time frame in the prediction methods, apart from sometimes stating "after pore pressures have dissipated." When high strain dynamic pile testing is used to verify the design pile capacity, the field capacity is determined during initial driving and again during pile restrike, usually only about 1 to 3 days after initial driving. Longer time intervals for restrikes and static loading tests, if performed, are usually limited, for economic and constructability reasons, to only about 7 to 14 days after initial driving, which is within a time period where full pore pressure dissipation within the surrounding soil often has not yet occurred. Therefore, on first inspection, it might appear that the capacity prediction by the selected CPT/CPTu method may have either over-predicted or under-predicted the pile capacity when compared to actual test results, when in reality, the predicted capacity and tested capacity need to be correlated based on time effects.

2 TIME EFFECT ON PILE CAPACITY

Time has an important effect on the capacity of piles installed in soil and some rock types. Experience has shown that pile capacity can either increase (setup) or decrease (relaxation) with time, with setup being more common than relaxation. Soil setup can occur in most soil types, but is most predominant in cohesive soils. During pile installation, the soil surrounding the pile experiences plastic deformations, remolding, and pore pressure changes. With time, pore pressures return to equilibrium. Where positive pore pressures are generated, there is a reduction in effective stress. As the pore pressures return to equilibrium, the effective stress increases, and in cohesive soils consolidation occurs around the pile shaft, resulting in strength gain. In low permeability cohesive soils that lack any lensing of more permeable soils, the time for pore pressure equilibrium can be many days; however, soil setup typically tends to begin almost immediately upon completion of initial pile installation. After the consolidation phase is complete, further capacity increase is then due to soil aging effects.

One well-known relationship between time after initial pile installation, t, and axial pile capacity, Q, is that described by Skov and Denver (1988), and is expressed by the following time function equation:

$$Q = Q_0\left[1 + A\ \log_{10}\left(\frac{t}{t_0}\right)\right] \qquad (1)$$

Where Q_0 is the reference capacity measured at the reference time t_0, and A is a dimensionless setup factor. Because the setup factor corresponds to a ten-fold increase in time, the factor A will be denoted herein as Δ_{10} which follows the nomenclature suggested by Augustesen (2006). The resulting setup factor is dependent on the choice of the reference capacity, and thereby the reference time. There is no consensus, however, in the published literature on the choice of t_0, with suggested values of 0.1 day (Svinkin and Skov, 2000) and 1 day (Bullock, 2005a, 2005b), to 100 days (Augustesen, 2006). Therefore, when comparing setup factor values suggested in literature, it is important that the setup factors be converted to the same reference time.

3 CASE HISTORY

A test pile program was performed to evaluate long-term soil setup. A seismic CPTu sounding was performed at the test pile site, and the results are used to evaluate published CPT/CPTu-based pile capacity prediction methods, particularly with respect to soil setup.

3.1 *Site and Interpreted Subsurface Conditions*

The test pile site was located in the city of Wayzata, about 20 km west of downtown Minneapolis, Minnesota. Several glacial advances had combined to form the geologic setting in the area. The soils were deposited during the late Wisconsinan glacial epoch that had occurred between 10,000 to 25,000 years BP. The two glacial advances that contributed most of the deposition in the area were the Superior and Des Moines glacial lobes. Numerous lakes in the area were formed when the glaciers last receded, leaving behind buried ice blocks that melted to form large basins, or “kettles.” These kettles filled with water; however, many also filled with organic sediment which became peat bogs. The bedrock at the site is Ordovician-age sandstone which is present at a depth exceeding 50 m below grade.

A seismic piezocone penetration test (SCPTu) was performed at the specific test pile location. The piezocone had a 15-cm^2 conical tip, and was advanced by a dedicated 200 kN SCPTu truck. Penetration pore pressures were measured along the shoulder at the u_2 position. Down hole seismic shear wave velocities (V_s) and pore pressure dissipation tests were also measured at specific depths during the advancement of the piezocone. Figure 1 presents basic SCPTu results.

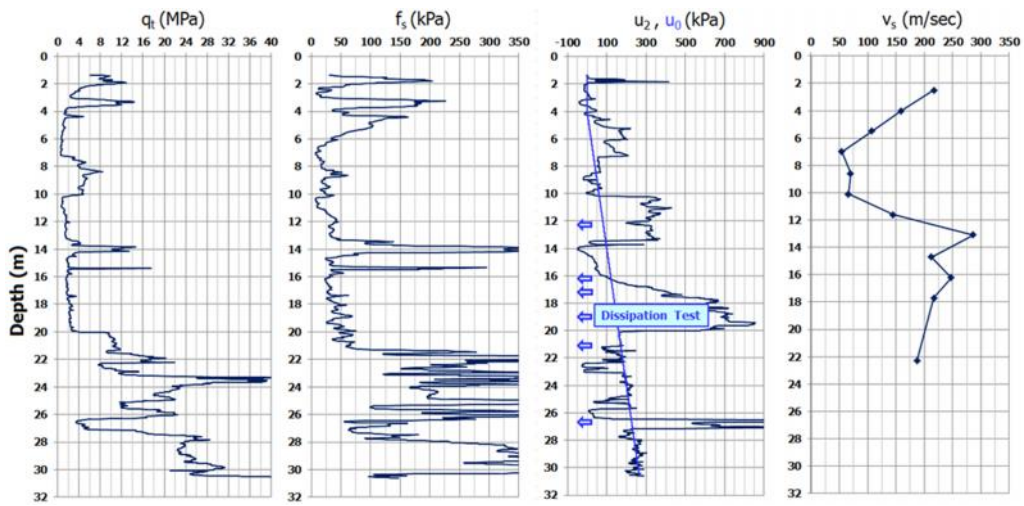
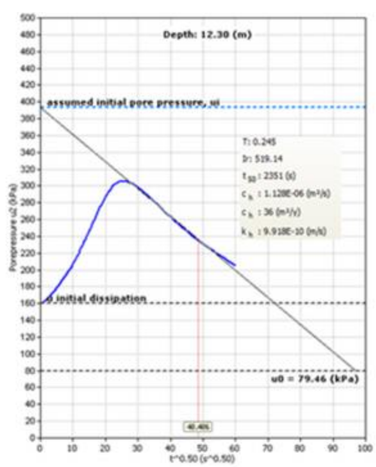


Figure 1. SCPTu sounding results.



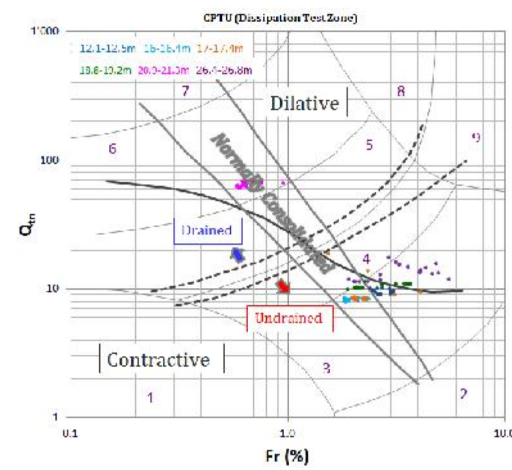


Figure 2. Typical dissipation test result

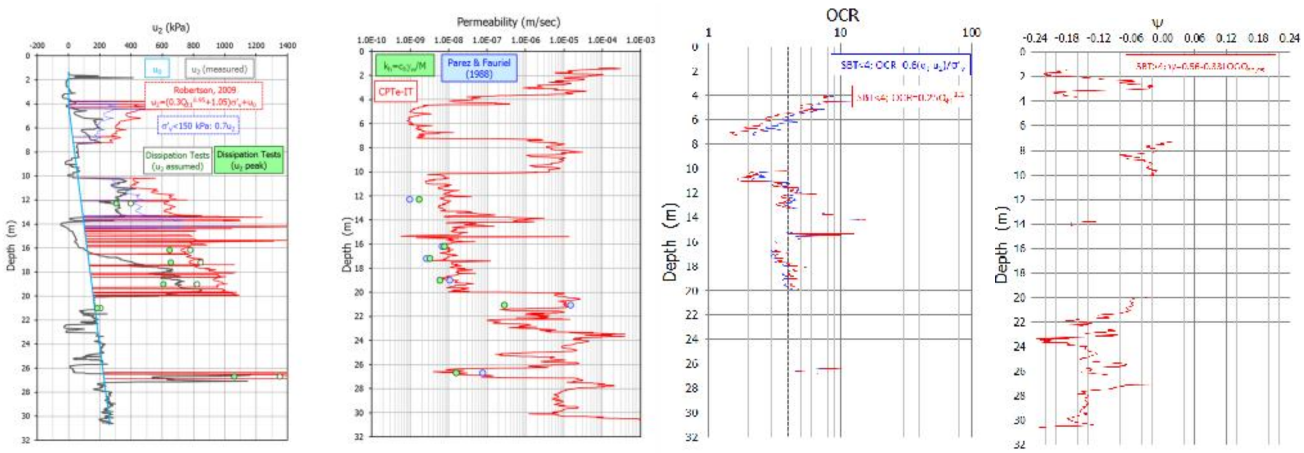
Figure 3. Soil behavior chart

The hydrostatic groundwater level is at a depth of about 4 m below grade. The pore pressure dissipation test depths are shown in Figure 1. Figure 2 presents a typical test result. All of the dissipation plots had a bell-shaped curve, with a temporary increase in pore pressure followed by a subsequent decay, typical of dilative soil behavior associated with overconsolidated clays. The CPTu data at the dissipation test depth intervals are plotted on the Robertson (2012) chart in Figure 3, which shows approximate boundaries between dilative-contractive behavior, and drained-undrained CPT response.

The u_2 plot in Figure 1 shows a negative response suggesting dilative behavior during advancement of the cone; however, instrument behavior (cavitation) could also explain some of this response. To account for this, an adjusted penetration pore pressure was calculated by using equation 2 (Robertson, 2009), which follows the relationships developed by Schneider, J.A. et al (2008) for insensitive clays.

$$u_2 = (0.3Q_{t1}^{0.95} + 1.05)\sigma_v' + u_0 \qquad (2)$$

The results are plotted in Figure 4a. Also plotted in Figure 4a is a plot of 70% of the calculated u_2 at an effective overburden pressure of less than 150 kPa. Figure 4b presents calculated and derived permeabilities. The criterion suggested by Robertson (2012), for separating contractive-dilative soil behavior assumes that interpreted sand-like soils with a state parameter, ψ, less than -0.05 and clay-like soils with an OCR > 4 are dilative at large strains (Figures 4c and 4d).



A. B. C. D.

Figure 4. a.) Penetration pore pressure, u_2, measured and derived from CPTu data, b.) Permeability, calculated from dissipation tests and derived from CPTu data, c.) Overconsolidation Ratio (OCR), and d.) State Parameter (ψ).

A soil boring that was drilled in the area, but not part of the test pile program, has allowed the stratigraphic and lithological comparison shown in Figure 5 which highlights similar, but not identical, soil types (because the boring was not drilled at the same location as the SCPTu sounding).

Figure 5 also presents the Soil Behavior Type index with the cut-off between clay-like soil and sand-like soil at I_c=2.58 following Ku et al. (2010). This figure also presents the Apparent Fines Content, FC, plotted as the mean value from methods described by Yi (2010), Idriss & Boulanger (2008), and Robertson & Wride (1998). The plots in Figure 5 assist in identifying the overall thickness of these strata, as a way to explain the significance of the measured setup phenomenon and also to assist in developing a method to predict the development and magnitude of the setup.

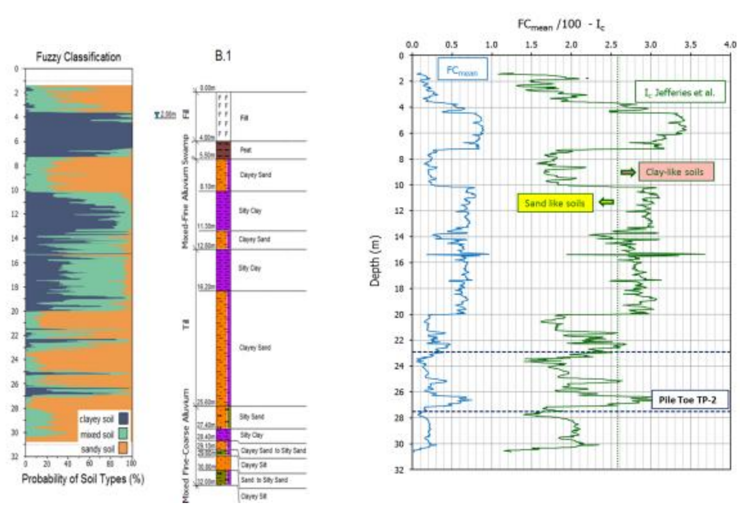


Figure 5. Stratigraphic and lithological comparison between CPTu and nearby boring results (left and middle), and Apparent Fines Content and Soil Behavior Type index (right).

Summarizing, the test pile site is covered by about 4.4 m of fill, which in turn overlies swamp-deposited peat to a depth of about 7.2 m. The fill was predominantly sand and silty sand, with some clayey sand and lean clay. A review of the site history indicated that the fill had been in place for over 40 years; therefore, the underlying peat had been compressed by the weight of the fill; however, the peat was still in a relatively soft condition and is, by definition, normally consolidated. Below the peat was non-organic glacio-alluvial and glacial till soils consisting predominantly of interbedded strata of over-consolidated sand, silty sand, silty clay, and clayey sand.

3.2 *Test Pile Program*

Two test piles were driven at the Wayzata site. Both were 0.178 m diameter, closed-ended steel pipe piles; one (TP-1) was driven to a depth of 22.9 m and the other (TP-2) was driven to a depth of 27.5 m below grade. Using the SCPTu data from Figure 4, the soil along the pile shafts has been divided into clay-like soil and sand-like soil with total thicknesses along the pile shafts as presented in Table 1.

Table 1. Soil-type thickness along the pile shaft

Test Pile	Pile Length (m)	Clay-like Soil (m)	Sand-like Soil (m)
TP-1	22.9	12.6	10.4
TP-2	27.5	13.2	14.3

High strain dynamic testing was performed with a Pile Driving Analyzer on each of the piles during initial driving, and again during restrike at approximately 1, 6, 27, and 1250 days after initial driving. Signal matching analyses by the Case Pile Wave Analysis Program (CAPWAP) were performed on the data. The results of the measured mobilized total (shaft and toe) pile capacity, Q_m, are presented in Figure 6, which shows that pile capacity continued to increase for both piles throughout the 1250 day period due to soil setup.

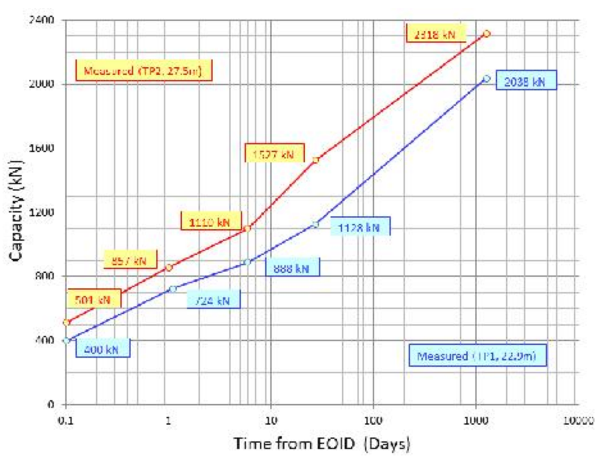


Figure 6. Total capacity, Q_m, plotted as a function of the logarithm of time for test piles TP-1 and TP-2.

By scaling the piezocone dissipation test time by the square of the pile/piezocone diameter ratio (Randolph, 2003) it is estimated that the time for 90% consolidation of the soil surrounding the piles is on the order of 90 days. Therefore, any pile capacity increase beyond this time is interpreted to be attributed to aging effects.

4 PILE CAPACITY PREDICTION

Numerous direct and indirect methods are available for predicting pile capacity based on CPT and CPTu results. The published methods used to evaluate the pile capacity at the test pile site were the LCPC method (Bustamante, 1982), the Eslami and Fellenius (1997) method, the KTRI method (Takesue et al., 1998), the NGI-05 method (Clausen et al., 2005, and Karlsrud et al., 2005), the UWA-05 method (Lehane et al. 2005, and Schneider, et al., 2007) and the Togliani (2008) method. The time period between initial pile driving and the loading tests that the methods are based upon is presented in Table 2.

Table 2. Time frame referenced in the CPTu pile capacity prediction methods.

Method	Time Frame
LCPC	Not described
Eslami and Fellenius	Not described
KTRI	"at least one month"
NGI-05	100 days
UWA-05	35 and 60 days*
Togliani	30 days

*Schneider modification for layered sand, silt, and clay based on the results of two static loading tests performed in Tokyo Bay.

Predicted total pile capacity, Q_p, using the Wayzata CPTu data for the various published methods are presented in Table 3. No modifications were made to the original methods when calculating the predicted capacities.

Table. 3. Predicted total pile capacities, Q_p.

	Method and Capacity (kN)					
Pile	LCPC	Eslami & Fellenius	KTRI	NGI-05	UWA-05	Togliani
TP-1 (22.9 m)	650	1089	1194	720	1243	1290
TP-2 (27.5 m)	1037	1296	1621	1043	1681	1649

There is a wide range of Q_p for the two test piles at this site, although the KTRI, UWA-05, and Togliani methods predict capacities relatively close to one another. The wide range of Q_p is not unusual, and is typically the case, particularly when applying these methods at sites for which they were not specifically developed. The question then becomes: What would be the final, single predicted capacity of each of the test piles?

5 ANALYSIS OF RESULTS

It has been shown that pile capacity is usually not constant, and it changes with time due to soil setup. It has also been shown that capacity prediction by various published pile capacity methods can produce a wide range of results; therefore, to accept or dismiss a particular method based on a single pile test result performed at one particular instance of time is not correct. Time is an important variable that must be incorporated in pile capacity prediction.

One method is to use the time function (equation 1) to correct Q_p for time. At this site, the reference time, t_0, is chosen to be 100 days; a time where the consolidation phase of setup had been completed. This results in a Δ_{10} of 0.25, with a corresponding 100-day Q_0 of 1600 kN for TP-1, and a Δ_{10} of 0.24, with a corresponding 100-day Q_0 of 1830 kN for TP-2. These values for Δ_{10} compare very well with the value of 0.24 for Δ_{10} as recommended by Augustesen (2006) for t_0 of 100 days, and also compare well with the setup coefficient of 0.26 calculated by Doherty and Gavin (2013) at the Belfast harbor tests, which also used t_0 of 100 days.

Figure 7 presents plots of the measured pile capacity with time, along with a capacity plot using Δ_{10}. Also included on these figures are the CPT/CPTu pile capacity predictions plotted along the time line. As can be seen, the capacity predictions are not necessarily incorrect, but are each correct for a specific period of time after end of initial driving.

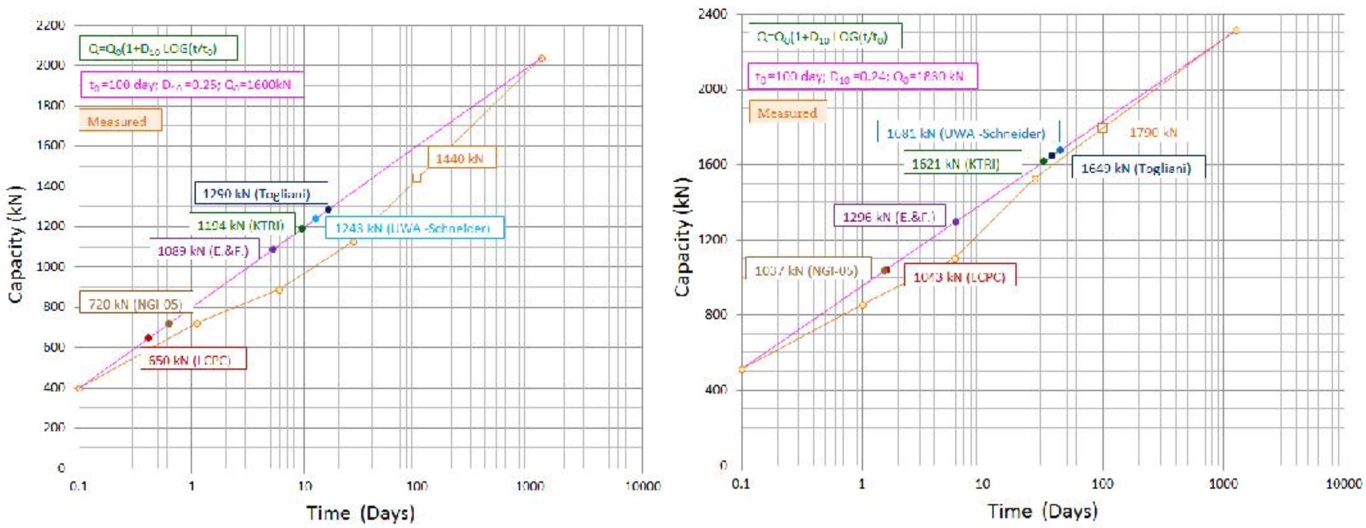


Figure 7. Graphical results of the pile capacity analyses for TP-1 (left) and TP-2 (right).

6 MODIFICATIONS TO PREDICTION METHODS

The following are attempts to predict the 100-day capacity by modifying the CPTu methods used during the initial pile capacity predictions. Table 1 indicates that approximately half of the pile length is embedded in "clay-like" soil; therefore, the modifications focus on adjustments to parameters most likely affected by the clay-like soil.

6.1 *KTRI Method*

The KTRI method is largely based on the measured f_s, while being strongly influenced by the measured au_2. Negative penetration pore pressures were measured during the piezocone penetration, as seen in Figure 1. Because of this, an adjusted penetration pore pressure was calculated by using equation 2, which then allowed a re-calculation of the pile shaft resistance using the adjusted u_2. This adjustment results in Q_p of 1393 kN for TP-1 (Q_p/Q_m = 0.87) and 1984 kN for TP-2 (Q_p/Q_m = 1.08).

6.2 *Eslami and Fellenius Method*

The Eslami and Fellenius method uses a correlation coefficient, C_s, for calculation of pile shaft resistance. C_s is a function of soil type and is the ratio of the values of pile unit shaft resistance to the average effective cone resistance. Table 4 presents the C_s values recommended by Eslami and Fellenius for differing soil types, based on their reported range of values in each soil category.

Table 4. Shaft correlation coefficient, C_s, for the Eslami and Fellenius method

Soil type	C_s (%) - Method	C_s (%) - Wayzata Site	Setup Ratio
Soft sensitive soils	8.0	24.0	3.0
Clay	5.0	12.5	2.5
Stiff clay and clay/silt mix	2.5	5.0	2.0
Silt and sand mix	1.0	1.5	1.5
Sand	0.4	0.5	1.25

Table 4 also presents adjusted C_s values for the Wayzata site for calculation of a 100-day setup capacity. More change in C_s was given in the cohesive soils to reflect greater soil setup than that which occurs in granular soils. Using the modified C_s values results in a predicted 100-day capacity of 1572 kN for TP-1 (Q_p/Q_m = 0.98) and 1857 kN for TP-2 (Q_p/Q_m = 1.01).

6.3 *Togliani Method*

The shaft resistance calculated by the Togliani method, f_{pNC}, is modified by OCR using equation 3:

$$f_p = f_{pNC} OCR^{0.13} \tag{3}$$

Using the modified f_p values result in predicted 100-day capacities of 1420 kN for TP-1 (Q_p/Q_m = 0.89) and 1803 kN for TP-2 (Q_p/Q_m = 0.99).

6.4 *All Methods*

To modify the existing capacity prediction methods for prediction of the 100-day capacity, a "virtual" pile model was first established based on the percentage of soil types most likely to contribute to soil setup, that is, clay soils being given larger weight than sand soils, as shown in Table 5. The clay multipliers follow values suggested by Hotstream and Schneider (2012).

Table 5. Stratum thickness multipliers

Ic	f_s (kPa)	Multiplier
> 2.58	<10	3
	10-20	2
	20-30	1.5
	>30	1.2
2.58-2.1	--	1.1
< 2.1	--	1.05

The stratum thickness corresponding to the particular soil behavior type is multiplied by the factors in Table 3 to produce a virtual pile length, L_V, which is greater than the actual pile length, L_A. A final multipier factor, F_m, is then calculated by equation 4:

$$F_m = \left(\frac{L_V}{L_A}\right)\left[\varsigma + \left(\frac{c}{L_A}\right)^{\frac{L_A}{c}}\right] \tag{4}$$

Where c is the total length of pile embedded in clay-like soil, and ς is a numeric variable which is different for each of the considered capacity methods. The "uncorrected" capacity calculated from the individual method is then multiplied by F_m to approximate the 100-day pile capacity. The "ς" variable was determined by trial and error for TP-2 (27.5 m pile) to establish the 100-day capacity, as seen in

Figure 8. These same "ς" variables were then applied to TP-1 (22.9 m pile). This method, as expected, increased the predicted capacity of all the methods; however, only the modified Eslami & Fellenius and Togliani methods again closely approximate the 100-day capacity for TP-1.

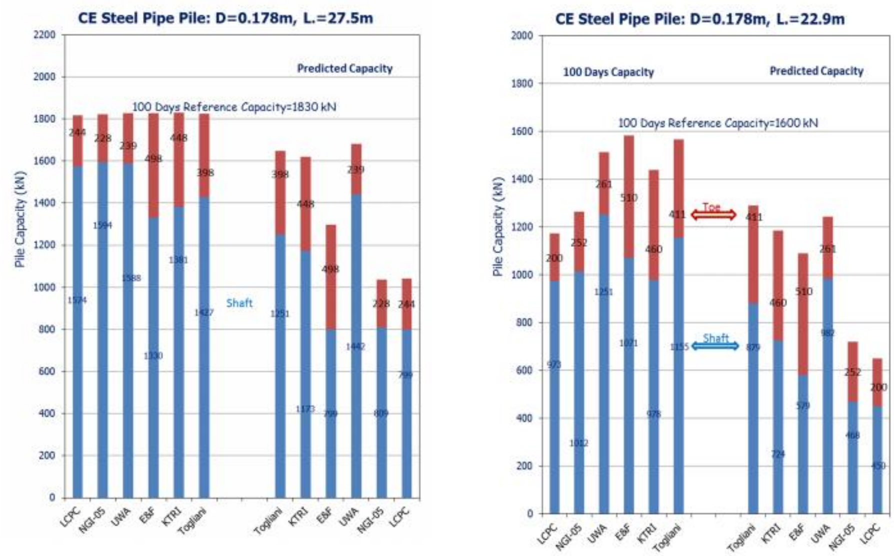


Figure 8. Predicted capacity by CPTu methods and 100-day prediction modifications for TP-2 (27.5 m) and TP-1 (22.9 m).

7 AGING

Karlsrud et al. (2005) proposed that the time effect due to aging could tentatively be accounted for by use of the following expression:

$$Q(t) = Q(100)[1 + \Delta 10 Log(t/100)] \qquad (3)$$

where: $\Delta 10 = 0.1 + 0.4(1 - \frac{PI}{50})OCR^{-0.8}$

The plasticity index, PI, and OCR are averages along the pile shaft. Assuming PI= 20 (typical of the low plasticity soils in the area) and an average OCR= 4.7 for TP-1, and 4.8 for TP-2 from the SCPTu data processing, the following capacities are obtained for the date of the long-term restrike which compare very well with the measured capacities:

TP-1: Q(t) = 1600[1+0.2496 log(1257/100] = 2039 kN (Q_m = 2038 kN)

TP-2: Q(t) = 1830[1+0.2484 log(1253/100] = 2329 kN (Q_m = 2318 kN)

8 CONCLUSIONS

Pile capacity is not a single value and changes with time. Modifications were made to published CPT/CPTu-based prediction methods in an attempt to predict the 100-day reference pile capacity for the test piles at the Wayzata site. These modifications may not be applicable to other sites, but it is the authors' intent to highlight the need to consider time when using CPT/CPTu-based capacity prediction methods and to form a basis for discussion for establishing methods to account for the time-dependent change in capacity. The authors' also agree with other researchers that the reference pile capacity prediction should universally be referred to 100 days, which would then allow a direct comparison between calculated setup factors and validity of the different pile capacity prediction methods.

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