UPDATING FLORIDA DEPARTMENT OF TRANSPORTATION'S (FDOT) PILE/SHAFT DESIGN PROCEDURES BASED ON CPT & DTP DATA

By

ZHIHONG HU

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To my wife for her strong support, endless encouragement, and help to make this happen
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LIST OF ABBREVIATIONS

$\alpha$  
Empirical factor to calculate tip resistance in Zhou et al. method

$\alpha_C$  
Penetrometer to pile friction ratio in clay in Schmertmann method

$\alpha_S$  
Penetrometer to pile friction ratio in sand in Schmertmann method

$\alpha_s$  
Empirical factor to calculate skin friction in Philipponnat method

$A_S$  
Surface area for the calculation of skin friction

ASTM  
American society for testing and materials

$\beta$  
Adhesion factor in De Ruiter and Beringen method and Zhou et al. method

$\beta_T$  
Target reliability index

$C_S$  
Pile skin friction factor in Eslami and Fellenius method

CPT  
Cone penetration test

COV (Q)  
Load coefficients of variation

COV (R)  
Resistance coefficient of variation

COV$_{Q_D}$  
Dead load coefficient of variation

COV$_{Q_L}$  
Live load coefficient of variation

$\delta_f$  
Pile-soil interface friction angle at the maximum shear stress

$\delta_{cv}$  
Constant volume interface friction angle between sand and pile

$D$  
Diameter or side length of pile

$D_{CPT}$  
Diameter of cone penetrometer

$D_{int}$  
The internal diameter for pipe pile

DTP  
Dual tip penetrometer

$\Delta r$  
Interface dilation

$\Delta \sigma_{rd}$  
The net dilatant component

$\Phi$  
LRFD resistance factor

$F_b$  
Empirical factor to calculate tip resistance in Aoki and De Alencar method
\( F_s \)  Empirical factor to calculate skin friction in Aoki and De Alencar method

\( F_S \)  Empirical factor to calculate skin friction in Philipponnat method

FDOT  Florida department of transportation

FORM  First Order Reliability method

FOSM  First Order Second moment

\( f_L \)  Loading coefficient

\( f_S \)  Pile unit skin friction

\( f_{sa} \)  Average CPT sleeve friction

\( \phi \)  Friction angle

\( G \)  The sand shear modulus, failure equation in terms of random variables

\( G_{\text{max}} \)  Maximum shear modulus

\( \gamma_D \)  Dead load factor

\( \gamma_L \)  Live load factor

\( h \)  The height above the pile tip

I-295  Interstate 295

I-95  Interstate 95

\( K_c \)  Earth pressure coeffient after equilization

\( k \)  Pile tip resistance factor in MTD method

\( k_1 \)  Pile skin friction factor in Almeida et al. method and Powell et al. method

\( k_2 \)  Pile tip resistance factor in Almeida et al. method and Powell et al. method

\( k_b \)  Pile tip resistance factor in Prince, Wardle method Philipponnat method, LCPC method, and the Proposed method

\( k_S \)  Pile skin friction factor in Prince and Wardle method

\( L \)  Pile embedment length

LCPC  Laboratoire central des ponts et chaussées
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<th>Acronym</th>
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<td>Load and resistance factor design</td>
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<tr>
<td>$\gamma_i$</td>
<td>Load factors</td>
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<tr>
<td>$\lambda_{QD}$</td>
<td>Dead load bias factor</td>
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<tr>
<td>$\lambda_{QL}$</td>
<td>Live load bias factor</td>
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<tr>
<td>$\lambda_R$</td>
<td>The mean bias</td>
</tr>
<tr>
<td>$\lambda_{Ri}$</td>
<td>The ratio of measured to predicted pile capacities</td>
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<tr>
<td>MTD</td>
<td>Marine technology directorate</td>
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<tr>
<td>MN/m$^2$</td>
<td>Mega Newton per square meters</td>
</tr>
<tr>
<td>m</td>
<td>Pile skin friction factor in Tumay and Fakhroo method</td>
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<tr>
<td>N</td>
<td>The number of cases, or SPT blow count</td>
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<tr>
<td>$N_C$</td>
<td>Bearing capacity factor</td>
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<td>$N_k$</td>
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<td>$N_{kt}$</td>
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<tr>
<td>$N_s$</td>
<td>Cone factor to estimate sensitivity</td>
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<tr>
<td>NCHRP</td>
<td>National cooperative highway research program</td>
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<tr>
<td>OCR</td>
<td>Over consolidation ratio</td>
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<td>PI</td>
<td>Plasticity index</td>
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<td>Pa</td>
<td>The atmosphere pressure</td>
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<td>PPC</td>
<td>Precast-prestressed-concrete</td>
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<td>PL-AID</td>
<td>Pile load settlement analysis from <em>in-situ</em> data</td>
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<td>psi</td>
<td>Pound per square inches</td>
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<td>Q</td>
<td>Random variable for load</td>
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<td>$Q_{c/N}$</td>
<td>The ratio of CPT tip resistance to the SPT blow count</td>
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<td>$Q_D$</td>
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**Q_{Davisson}**  Davisson capacity

**Q_L**  Live load ratio

**Q_D/Q_L**  Dead to live load ratio

**Q_S**  Pile total tip resistance

**Q_{S,ult}**  Ultimate pile skin friction

**Q_{T,ult}**  Ultimate pile tip resistance

**q_c**  CPT tip resistance

**q_{ca}**  Average CPT tip resistance

**q_e**  The average of the effective cone resistance within the calculation layer

**q_{eg}**  The geometric average of the effective cone resistance

**q_{eq}**  Average of tip resistance within 1.5 D above and 1.5 D below the pile tip after eliminating abnormal data (out of the range of $\pm 30\%$ of average value)

**q_t**  Pile unit tip resistance

**R**  The diameter of the pile in MTD method, random variable for resistance

**R_{cla}**  The pile’s center-line-average roughness

**R_{design}**  Design capacity

**R_{mi}**  Measured capacity from load test data

**R_{ni}**  Predicted capacity form CPT data

**R_n**  Nominal resistance

**R.D.**  Relative density

**SMO**  State material office

**SPT**  Standard penetration test

**S_t**  Clay sensitivity

**S_u**  Undrained shear strength

**\sigma_R**  The standard deviation of $\lambda_{Ri}$
\( \sigma_{rc} \)  
Radial effective stress on side after equalization

\( \sigma_{rf} \)  
Radial effective stress at maximum shear stress

\( \sigma_{v0} \)  
The total overburden stress

\( \sigma'_{v0} \)  
The effective overburden stress

T1  
DTP first tip resistance

T2  
DTP second tip resistance

ts\(\text{sf}\)  
Tons per square feet

UF  
The University of Florida

UWA  
The university of Western Australia

VAR  
Variance

YSR  
Yield stress ratio

\( y \)  
The distance between the surface and the skin friction calculating point

\( \eta \)  
Load modifier for importance, redundancy and ductility
The Florida Department of Transportation (FDOT) began developing a geotech-material-construction database that includes information on piles and drilled shafts, specifically, *in-situ* data (SPT, CPT, etc.) and load test data in the early 1990s. More recently, FDOT sponsored novel research to develop a new *in-situ* device, the dual tip penetrometer (DTP), to identify cemented soils and thereby contribute to the database data. My research evaluated current pile design methodologies (Schmertmann, LCPC, etc.) using CPT, DTP, and modified current methods and proposed a new method to improve future driven pile design. My research also involves identifying cemented soils using the DTP, since cementation is a critical issue in pile design procedures.

My research explores 14 pile-capacity-design methods based on cone penetration test (CPT) and assesses load and resistance factor design (LRFD) resistance factors for each using 21 cases from Florida and 28 from Louisiana. The resulting resistance factors were not satisfactory for any of the methods. A new design method was proposed, taking into account cementation and other issues. The LRFD resistance factor was also assessed for this new method. DTP tests were performed at cemented-soil sites to verify the cemented-soil identification (T2/T1, [Tip
1/Tip 2], and Friction Ratio) from DTP. The ratio T2/T1 was finally determined to be an excellent identifier in locating cemented sand.

My new method provides better LRFD resistance factors for both Florida soil and Louisiana soils. It could be a promising method to improve pile design in the future. From the DTP tests in cemented soils, it was concluded that the DTP could be an efficient tool to identify cemented sand and thereby better predict pile capacity.
CHAPTER 1
INTRODUCTION

From the 1960s to the 1980s, FDOT sponsored research at the University of Florida to evaluate the methods used for calculating static pile capacity based on the CPT (cone penetration test). After years of research, Dr. John H. Schmertmann (1978) proposed the method which was later named after him. This method was put into the FDOT pile design software and termed, PL-AID. This method has been used successfully in the districts of Florida on 16” to 18” precast-prestressed-concrete (PPC) piles. However, during the last two decades, the size of the pile has increased to 24” to 30”, primarily due to higher strength concretes and steel as well as larger pile driving equipment. The Schmertmann method became conservative in evaluating pile capacity based on the comparison between the predictions and static load test results.

Compare to drilled shaft, driven piles have their own advantages. There are significant reductions of lateral capacity for drilled shaft due to torsional loading (Hu et al. 2006). Since driven piles are usually in group and there will be no reduction of lateral capacity due to torsional loading. That is the reason why more and more CPT based pile capacity prediction methods have been proposed. During the 1980s, many methods were developed around the world. These included the Aoki and de Alencar method (1975), Penpile method (Clisby, M.B., et al. 1978), de Ruiter and Beringen method (1979), Philipponnat method (1980), Bustamante and Gianeselli method (LCPC) (1982), Price and Wardle method (1982), and Tumay and Fakhroo method (1982). Most of these methods were generated by matching the CPT and load test database in the local area. None of the methods have been evaluated in Florida soils. The Louisiana Transportation Research Center evaluated these methodologies in predicting the axial ultimate capacity of square PPC piles driven into Louisiana soils. Based on the result, the de Ruiter and Beringen and Bustamante and Gianeselli (LCPC) methods showed the best
performance in predicting the pile capacity. However, that may or may not be the case in Florida.

From 1990 until the present, many new methods have been proposed. They include the Almeida et al. method (1996), Jardine and Chow method (1996), Eslami and Fellenius method (1997), Powell et al. method (2001), and UWA-05 method (Lehane, B.M., et al. 2005). Most of these methods take into account the pore pressure from CPTU to improve their accuracy. The Zhou et al. method (1982) was proposed in 1982 using load test data and CPT performed in the eastern China. However, it had not been evaluated in other areas outside of this region. Again, none of these methods have been evaluated in Florida. Details of these methods are given in the following chapters.

There are a total of 21 cases (load test data with CPT close to them) in Florida and 28 in Louisiana. All previous methods were evaluated by using these cases. The LRFD resistance factor for each method was calculated and compared. One of the most accurate and simplest methods, the Philipponnat method, was chosen and modified to form the proposed UF method. Both Florida and Louisiana soil data were used to validation.

One of the more challenging soil types in Florida is cemented sand. Cementation in sands improves strength, but the strength increase depends on the degree of cementation. The degree of the bond strength in the cemented sands should be considered when designing foundations on or in cemented sands. Since this material cannot be identified by a CPT test, none of the methods mentioned above had taken into account the cementation issue. This means they may overestimate the pile capacity, which may produce a serious design flaw. Recently, the FDOT funded the University of Florida to develop a new cone penetrometer, the dual tip penetrometer (DTP), to be able to identify cemented sands. UF’s new proposed design method takes into
account the cementation issue and results in the highest $\Phi/\lambda_R$ (the ratio of resistance factor to the mean bias - 0.62 for Florida soil and 0.67 for Louisiana soil) among all the methods as well has the lowest coefficient of variation (0.27 for Florida soil and 0.23 for Louisiana soil).
Cemented Sand

The term “cemented sand” is a general term used for a wide variety of soils. R.W. King (Lunne et al. 1997) proposed a classification system for a variety of cemented carbonate soils. One of the main problems of this system is that degree of cementation is only a function of penetrometer resistance $q_c$, and does not take into account the relative density of sand. For example, non-cemented dense sand may have cone resistance $q_c$ higher than 10 MN/m$^2$, with no cementation. It will be very useful for design engineers, if better identifiers or parameters for cemented sand can be found.

Cemented sands exist in many areas of the United States, including California, Texas, Florida, and along the banks of the lower Mississippi River. They also exist in Norway, Australia, Canada, and Italy (Puppala et al. 1995). Calcareous cemented sands are a feature of warm water seas mainly due to the sedimentation of the skeletal remains of marine organisms (Lunne et al. 1997).

Cemented sand as the name implies, is a cohesionless material in which a calcium-carbonate chemical bond develops - to some extent. This chemical bond is the result of the deposition of calcium at the particle-to-particle contacts and the chemical reaction between calcium and sand over time. The strength of these chemical bonds depends on the degree of cementation as well as the distribution. This kind of cementation leads to a significant increase in modulus (Briaud). Ahmadi et al. used computer modeling for CPT penetrating process and found that the modulus of sand is the key factor in CPT tip resistance (Ahmadi et al. 2005). This is why cementation tends to increase tip resistance. This phenomenon was also found by Puppala et al (Puppala et al. 1995).
From a mechanical point of view, cemented sand belongs to an intermediate class of geomaterials placed between classical soil mechanics and rock mechanics. Often, no physical or mathematical models are able to integrate this kind of material in a consistent and unified framework (Gens and Nova 1993). During loading, cemented sand shows a very stiff behavior before yielding, which is governed by cementation. After stress reaches yielding stress, it suddenly changes into a ductile material. Leroueil and Vaughan (1990) discovered that the structure of chemical bonds and its effects on soil behavior is a very important factor in determining the soil stress-strain behavior as well as other factors, such as the relative density, over consolidation ratio etc. However, the structure of chemical bonds is an unpredictable and very difficult to identified, let alone quantify.

An understanding of the effect of a low degree of cementation on the sand’s strength is increasingly important in geotechnical engineering design and analysis. In the current design procedure, the effect of cementation is often neglected because cementation often improves the strength. However, preliminary studies indicate that light cementation increases the tip and friction resistances while decreasing the friction ratio of CPT (Rad and Tumay 1986). This could be explained by the bonds increasing the resistance during the penetration. However, the bonds tend to break during pile driving, and the CPT could not “sense” this reduction of strength.

The degree of cementation in sands can be an issue for geotechnical engineers. For well cemented sands, the strength can be so high that engineers will neglect the cementation issue. However, in these cases, even though the CPT could not totally break the chemical bond within the sand particles, large diameter driven piles may indeed do so, resulting in a much lower pile capacity than that predicted by the CPT results. For lightly cemented sand, a breakdown of “cohesion” bonds can occur from a disturbance such as an earthquake, cone penetrating, or pile
driving. One such example is when the Loma Prieta San Francisco earthquake caused slope failures along the cemented northern Daly City bluffs (Puppala et al. 1995). Other similar slope failures have occurred due to earthquakes and heavy rains (Rad and Tumay, 1986).

If the sands tested with a CPT test are not known to be cemented, the high bearing readings may be misinterpreted as being due to high relative densities. This can lead to an underestimation of the liquefaction potential of the soil and an overestimation of the ultimate pile capacity. None of the CPT prediction methods evaluated to date take into account the reduction of strength in cemented sand. The concern was proved legitimate by the CPT and DTP testing performed at Port Orange Relief Bridge in Port Orange Florida. A comparison between the predictions using the prediction methods and the load test result shows that most of the methods over-predict the pile capacity (some by over 100%).

Researchers have performed laboratory tests on cemented sands obtained in the field. Both Clough et al. (1981) and Puppala et al. (1998) have tested naturally cemented sands in triaxial tests and unconfined compressive strength tests. The cemented sands were obtained by trimming samples using an SPT split spoon sample. The retrieval of undisturbed, lightly cemented sands was quite difficult since the bonds tended to break under light finger pressure.

Due to the difficulty in sampling, in-situ testing has become a more popular method of testing naturally cemented sands. The CPT test is a popular device for testing the cemented sands. The CPT test has been used to test both naturally occurring cemented sands in the field (Puppala et al., 1998) and artificially cemented sands in calibration chambers (Rad & Tumay, 1986; and Puppala et al. 1995). In both the 1985 and 1996 calibration chamber studies, Monterrey No. 0/30 sand was cemented with 1% and 2% Portland cement. An attempt was made to relate the tip bearing and friction sleeve values to the sand properties, including cement
content, relative density, confining stress and friction angle. Puppala (1995) did this by using the bearing capacity equations of Durgunoglu & Mitchell (1975) and Janbu & Senneset (1974). To include the effect of cementation or cohesion on tip bearing, the other parameters that affect the tip bearing, mainly relative density and confining stress, needed to be known and included in the equations proposed by Puppala. Even though the calibration chamber study is time-efficient and makes it easy to control the cementation ratio, there are several drawbacks; first, the cementation structure in the nature is almost impossible to simulate in the chamber and, as is discussed above, this is a very important issue to determine a soil’s strength. Secondly, the stress state in the field is not the same as those in the calibration chamber, especially for deeply occurring cemented soil, due to size limitation of the chamber. Therefore, the literature indicates that the best approach in dealing with this issue is to test materials in-situ (e.g., CPT test) and somehow identify when cementation is present.

Since the main problem of cemented sands is providing “cohesion” on tip bearing resistance, if its effect could be removed from the tip bearing resistance it may be possible to obtain more accurate bearing capacity predictions. One way to accomplish this would be to design an in-situ device that could measure the bearing strength of the cemented sands both before and after the cohesive bonds have been broken up. This is the rationale that led to the development of the dual tip penetrometer developed at the University of Florida. Since it is simply an enhanced CPT, a brief history of this versatile instrument is provided below.

**Cone Penetration Test**

The cone penetration test is considered one of the most cost-effective and reliable method for soil classification. The CPT (Figure 2-1) test pushes a cone into the soil at a constant rate by means of cylindrical rods that are connected in series with the cone located at the base of the string of rods. During the test, the sleeve friction and tip resistance are measured and recorded.
These two parameters are used to classify soil and to estimate strength and deformation characteristics of soils.

In 1917, the Swedish railways introduced the CPT. Ten years later, Danish railways started to use CPT. The first apparatus was simply a cone and a string of outer rods. In 1936, the Dutch Mantle cone was introduced. This cone has an area of 10 cm² and an apex angle of 60°, which is similar to the currently ones in use. But the cone was pushed by hand and there was a limitation on the capacity and penetration depth. In addition, it could not penetrate very dense sand or cemented soils.

In the 1940s and early 1950s, hydraulic jacks were introduced that allowed for much more reactive force being applied, thereby increasing penetration depths. This advancement dramatically increased CPT usage. In 1948, the first electric cone penetrometer was developed. Strain gages were used to measure the soil resistance, which increased its accuracy dramatically, since the bridge circuit made it more sensitive to small changes in soil resistance. The most important feature of electric CPT is that it can provide a continuous reading of a soil’s resistance during the test (typically logged every 5 cm). This provides a wealth of subsurface information for geotechnical engineers.

One of the most important improvements of the CPT was made in 1953. Begemann proposed the use of a separate sleeve located just behind the tip that allows the penetrometer to measure both tip resistance $q_c$ and sleeve friction resistance $f_s$. The friction sleeve has an area of 150 cm² and was used in conjunction with the traditional Begemann mechanical cone in the late 1950s. In 1968, an electric cone penetrometer with the friction sleeve was developed in Australia.
The first ASTM standard (ASTM D-3441-75T) for the cone penetrometer was published in 1975. In 1979 and 1986, ASTM D-3441-79 and ASTM D-3441-86 were published to revise the previous standard. In 1988, an international reference test procedure was developed by the International Society of Soil Mechanics and Foundation Engineering.

Currently, there are two diameters for the cone: 1.41 in (10 cm$^2$ cross section) and 1.71 in (15 cm$^2$ cross section) with both having a 60° angle. The first one is the most commonly used.

**CPT Based Pile Capacity Prediction Methods**

Using CPT data for design is considered one of the most promising methods to predict the pile capacity for the following reasons:

1. The shape of a cone penetrometer is very similar to a cylindrical driven pile except at the bottom. However, during the ultimate failure of a pile, the soil under the pile tip is densified and forms a cone-shaped failure envelop similar to the cone penetrometer’s 60° tip.

2. The soil state during penetration is comparable to that during pile driving.

3. The testing process is quasi-static, which is more representative of a static load test compared to other in-situ tests.

4. Because the cone penetrometer actually penetrates the soil, causing an ultimate failure (punching failure) condition, it should be possible to predict the ultimate failure of the pile - including ultimate skin friction and ultimate tip resistance. These two predictions can also be useful during pile driving in order to prevent damage during the driving process.

5. The speed of conducting a test allows for more CPT soundings at a particular site and coupled with load test data make it possible to generate improved pile capacity prediction methods.

There are also issues involved in the prediction of pile capacity using CPT which have to be solved by empirical correlations:

1. The scale effect caused by the difference between the diameter of the penetrometer and that of the piles. This will influence the soil densification. The larger the diameter, the more densification the soil can achieve. It will also influence the size of the “stress
ball” which will influence the zone of resistance soil near the pile tip. If the soil is not uniformly distributed and the soil layer is not horizontal, one CPT may not be able to represent the pile - particularly large diameter piles.

2. The CPT can not be used to identify cemented soils. It tends to misidentify the material as simply a denser soil state, due to the high $q_c$ values.

3. When a CPT is performed in saturated clayey soils, especially with low permeability, high excess pore pressure will be generated during penetration and will cause higher $q_c$ value. However, when a pile is driven into the same soil, much higher excess pore pressure will be generated and will dissipate slowly depending on the permeability.

In order to propose a better method to predict pile capacities, many existing methods have been investigated. An extensive literature research was conducted, specifically looking for axial pile prediction methods based on CPT cone soundings. The following methods were identified as those used by a number of DOTs, consultants or contractors: the Schmertmann method, the de Ruiter and Beringen method, the Penpile method, the Price and Wardle method, the Tumay and Fakhrroo method, the Aoki and De Alencar method, the Philipponnat method, and the LCPC (Bustamante and Gianeselli) method. Most of the above methods were developed in 1980’s. From 1990 until now, many new methods have been proposed. They are the Almeida et al. method, the MTD (Jardine and Chow) method, the Eslami and Fellenius method, the Powell et al. method, and the UWA-05 method. The Zhou et al. method was proposed in 1982 using load test data and CPT performed in eastern China. A discussion of each of the methods is presented in the following section of this chapter.

**Schmertmann Method**

This method was first proposed by Schmertmann in 1978. It uses both tip resistance and sleeve friction to predict the pile capacity. The pile’s unit tip capacity is calculated by the minimum path rule. Schmertmann set an upper limit of 150 tsf for the unit tip capacity.

The pile’s unit skin friction:
In clay: \[ f_s := \alpha_c \cdot f_{sa} \leq 1.2sf \]
where: \( \alpha_c \) is a function of \( f_{sa} \).

In sand: \[ Q_s := \alpha_s \left( \sum_{y=0}^{8D} \frac{y}{f_{sa} \cdot A_s} + \sum_{y=8D}^{L} f_{sa} \cdot A_s \right) \]
where: \( \alpha_s \) is a function of pile depth to width ratio.

**De Ruiter and Beringen Method**

This method is proposed by de Ruiter and Beringen from their study of the soil near the North Sea. It uses both tip resistance and sleeve friction to predict the pile capacity.

The pile’s unit tip capacity:

In clay: \[ q_{t} := N_c \cdot S_u(tip) \]
where: \( N_c = 9 \), constant, bearing capacity factor;

\( q_c \) (tip) is the average cone tip resistance around the pile tip - similar to Schmertmann method (minimum path rule);

\( N_k = 15 \sim 20 \), constant, cone factor, (20 was used in the current study since it yielded better results).

In sand: \[ q_{t} := \frac{q_{c1} + q_{c2}}{2} \leq 150 \text{tsf} \]

The calculation of tip capacity is similar to Schmertmann method.

The pile’s unit skin friction:

In clay: \[ f_s := \beta \cdot S_u(side) \]
where: \( \beta \) constant, adhesion factor, 1 for N.C., 0.5 for O.C., (1 was used in the current study).

\( q_c \) (side) is the average cone tip resistance within the calculated layer along the pile.
In sand:  \[ f_s := \min \left[ f_{sa}, \frac{q_c(side)}{300} \text{(compression)}, \frac{q_c(side)}{400} \text{(tension)}, 1.2 \text{tsf} \right] \]

where:  \( f_{sa} \) is the average sleeve friction within the calculated layer along the pile.

**Penpil Method**

This method was invented by Clisby et al. for the Mississippi Department of Transportation. It uses both cone tip resistance and sleeve friction to predict the pile’s axial capacity.

The pile’s unit tip capacity:

In clay:  \( q_t := 0.25q_{ca} \)

In sand:  \( q_t := 0.125q_{ca} \)

where:  \( q_{ca} \): the average of three cone tip resistances close to the pile tip.

The pile’s unit skin friction:

where:  \( f_{sa} \): the average sleeve friction within the calculated layer along the pile.

\( f_s, f_{sa} \) are expressed in psi (lb/in\(^2\)).

**Prince and Wardle Method**

This method uses both the CPT tip resistance, \( q_c \), and sleeve friction, \( f_s \), to predict the axial pile capacity.

The pile’s unit tip capacity:  \( q_t := k_b \cdot q_{ca(tip)} \leq 150 \text{tsf} \)

The pile’s unit skin friction:  \( f_s := k_s \cdot f_{sa} \leq 1.2 \text{tsf} \)

where:  \( k_b \) and \( k_s \) are factors that depend on pile type;

\( k_b = 0.35 \) for driven pile, \( 0.3 \) for jacked pile;

\( k_s = 0.53 \) for driven pile, \( 0.62 \) for jacked pile, and \( 0.49 \) for drilled shaft;
\( q_{ca}(\text{tip}) \) is the average CPT tip resistance within 4D below and 8D above the pile tip (there is no reference about the influence zone, therefore for better results, 4D below and 8D above were chosen).

**Tumay and Fakhroo Method**

This method was proposed by Tumay and Fakhroo for estimating pile capacity in clayey soil. In order to see how this method performed for Florida soil, it was also evaluated. It uses both tip resistance and sleeve friction to predict the pile capacity.

The pile’s unit tip capacity:

\[
q_t := \frac{q_{c1} + q_{c2}}{2} \leq 150 \text{tsf}
\]

The calculation is similar to Schmertmann method except letting \( y \) equal to 4.

The pile’s unit skin friction:

\[
f_s := m f_{sa} \leq 0.72 \text{tsf}
\]

\[
m := 0.5 + 9.5 \cdot e^{-9 \cdot f_{sa}}
\]

where: \( f_{sa} \) is the average sleeve friction within the calculated layer along the pile with the unit tssf (ton/ft²).

**Aoki and De Alencar Method**

This method only uses the CPT tip resistance to predict the pile capacity.

The pile’s unit tip capacity:

\[
q_t := \frac{q_{ca}(\text{tip})}{F_b} \leq 150 \text{tsf}
\]

The pile’s unit skin friction:

\[
f_s := q_{ca}(\text{side}) \cdot \frac{\alpha_s}{F_s} \leq 1.2 \text{tsf}
\]

where: \( F_b, F_s \) are empirical factors that depend on pile type, \( \alpha_s \) is a function of soil type.

\( q_{ca}(\text{tip}) \) is the average CPT tip resistance within 4D below and 8D above the pile tip.
**Philipponnat Method**

This is another method which uses tip resistance, $q_{ca}$, to predict the axial pile capacity.

The pile’s unit tip capacity:  
$$q_t := k_b \cdot q_{ca}(tip)$$

The pile’s unit skin friction:  
$$f_s := q_{ca}(side) \cdot \frac{\alpha_s}{F_s} \leq 1.27 \text{tsf}$$

where:  
$q_{ca} (tip)$ is the average tip resistance of 3D below and 3D above the pile tip;  
$k_b$ and $F_s$ are functions of soil type;  
$\alpha_s$ is determined by pile type, $= 1.25$ for precast prestressed concrete piles.

**LCPC (Bustamante and Gianeselli) Method**

This method only uses cone tip resistance for predicting axial pile capacity. It was proposed by Bustamante and Gianeselli for the French Highway Department after the study of 197 piles in Europe. It is also called the French method.

The pile’s unit tip capacity:  
$$q_t := k_b \cdot q_{eq}(tip)$$

where:  
$q_{eq} (tip)$ is the average of tip resistance within 1.5 D above and 1.5 D below the pile tip after eliminating abnormal data (out of the range of $\pm 30\%$ of average value);  
$k_b$ is a function of soil and pile type.

The pile’s unit skin friction is a function of pile type, soil type and cone tip resistance.

**Almeida et al. Method**

This method was proposed by Almeida et. al. based on the analysis of 43 load tests on driven and jacked piles in clay in Norway and Britain. Most of the load tests were performed in
tension and only 4 tests in compression. The parameters used in this prediction method are penetrometer tip resistance and overburden stress.

The pile’s unit skin friction:

\[
f_s := \frac{q_c - \sigma_{v0}}{k_1} \quad k_1 := 11.8 + 14.0 \log \left( \frac{q_c - \sigma_{v0}}{\sigma_{v0}'} \right)
\]

where: \(q_c\) is CPT tip resistance, with pore pressure correction for piezocones,
\(\sigma_{v0}\) is the total overburden stress,
\(\sigma_{v0}'\) is the effective overburden stress.

In order to calculate the effective overburden stress, hydrostatic pressure has been used. A reduction in \(k_1\) needs to be applied if \(L/D > 60\). The reduction factor is recommended by both Semple and Rigden (1948) and is included in the procedure suggested by Randolph and Murphy (1985).

The pile’s unit tip capacity:

\[
q_t := \frac{q_c - \sigma_{v0}}{k_2}
\]

where: \(k_2\) is a function of both pile type and material \((k_2 = 2.7\) for driven pile =

\(1.5\) for jacked pile in soft clay, and \(=3.4\) for jacked pile in stiff clay).

In order to prevent a nonrealistic result in sand, a limitation of highest unit skin friction is set at 1.2 tsf.

**MTD (Jardine and Chow) Method**

The method proposed by Jardine and Chow is from intensive field tests using 4 inch (102 mm) diameter, closed-ended instrumented piles at two sand sites in France. In addition, data acquired from field tests on high-quality instrumented displacement piles in a large range of clay
soils performed by MIT, Oxford University, NGI and Imperial College over 15 years was utilized.

The pile’s unit tip capacity:

In clay:  \[ q_t := k \cdot q_{ca} \]

where:  \( k = 0.8 \) for drained loading, =1.3 for undrained loading.  \( q_{ca} \) is the average cone tip resistance within 1.5D above and 1.5D below the pile tip.

In sand:  \[ q_t := \left( 1 - 0.5 \log \left( \frac{D}{D_{CPT}} \right) \right) \cdot q_{ca} \]

where:  \( D \) is the diameter of the pile;

\( D_{CPT} \) is the diameter of cone penetrometer which is 1.4 inch (36 mm).

\( q_t \) has the lower bound value of 0.13* \( q_{ca} \) when \( D \) is greater than 6.56 ft (2 m).

The pile’s unit skin friction:

In clay:  \[ f_s := f_L \cdot K_c \cdot \frac{\sigma_v}{\sigma_v^0} \cdot \tan(\delta_f) \]

\[ K_c := \left( 2.2 + 0.016\text{YSR} - 0.870 \log(S_t) \right) \cdot \text{YSR}^{0.42} \cdot \left( \frac{h}{R} \right)^{-0.20} \]

where:  \( f_L \): loading coefficient, = 0.8;

\( K_c \): earth pressure coefficient after equilization;

\( \text{YSR} \): yield stress ratio (yield stress determined in an oedometer test divided by the vertical effective stress). In case the YSR is not available, Lehane et.al (2000) provides the following relationship between YSR and cone tip resistance;

\[ \text{YSR} := 0.04427 \left( \frac{q_c}{\sigma_v^{0.1667}} \right) \]
$S_t$: clay sensitivity, and in case the $S_t$ is not available, Robertson and Campanella (1983) proposed the relationship between $S_t$ and friction ratio;

$$S_t := \frac{10}{R f(\%)}$$

$h$: the height above the pile tip. In order to prevent too large a $K_C$ value, $h/R \geq 8$;

$R$: the diameter of the pile;

$\sigma'_{vo}$: effective over-burden pressure;

$\delta_f$: pile-soil interface friction angle at the maximum shear stress.

Because the large variations are possible, it is recommended by Lehane et.al (2000) to use a ring shear test to obtain the direct measurement; in case direct measurement is not available, Jardine and Chow proposed a relationship between $\delta$ and clay plasticity index for steel pile. $\delta_f$ will be between peak ($\delta_{peak}$) and ultimate ($\delta_{ultimate}$) depending on relative displacement between pile and soil.

If the PI of the deposit is not available for determining $\delta_f$ for clay, Schmertmann (1978a) suggests assuming an average normally consolidated ratio of 0.33 for most post-pleistocene clay which is corresponding to the PI equal to 0.59. From the relationship between PI and $\tan(\delta_f)$, 0.2 was determined to be $\tan(\delta_f)$.

In sand:

$$f_s := \sigma_{rf} \tan(\delta_f)$$

$$\sigma_{rf} := \sigma_{rc} + \Delta \sigma_{rd}$$

For compression pile

$$\sigma_{rf} := 0.8 \sigma_{rc} + \Delta \sigma_{rd}$$

For tension pile
\[ \sigma_{rc} := 0.029 q_c \left( \frac{\sigma_{v0}}{\text{Pa}} \right)^{0.13} \left( \frac{h}{R} \right)^{0.38} \]

\[ \Delta \sigma_{rd} := \frac{4G R_{\text{cla}}}{R} \]

\[ G := q_c \left( 0.0203 + 0.00125 \frac{q_c}{\sqrt{\text{Pa} \cdot \sigma_{v0}}} - 1.216 \times 10^{-6} \frac{q_c^2}{\text{Pa} \cdot \sigma_{v0}} \right)^{-1} \]

where:

- \( \sigma_{rf} \): radial effective stress at maximum shear stress;
- \( \sigma_{rc} \): radial effective stress on side after equalization;
- \( \Delta \sigma_{rd} \): the net dilatant component;
- \( \text{Pa} \): the atmosphere pressure;
- \( G \): the sand shear modulus;
- \( R_{\text{cla}} \): the pile’s center-line-average roughness. It is qual to \( 10^{-5} \) for steel pile, \( 10^{-4} \) for very rough casing of concrete pile and \( 3 \times 10^{-5} \) for prestressed concrete pile;
- \( \delta_f \): pile-soil interface friction angle at the maximum shear stress. It is recommended by Jardine and Chow (1996) to use an interface-direct or a ring-shear test with the same roughness and hardness as the pile material and same effective normal stress as the field; in case direct measurement is not possible, Jardine and Chow recommended to use the relationship between \( \delta_{cv} \) (critical state interface friction angle) and sand mean particle size \( (d_{50}) \) for steel pile and assume \( \delta_f \) is equal to \( \delta_{cv} \). From correspondence with the authors, it was found that they are currently conducting sets of
interface shear tests on sands sheared against concrete but have not finished yet. For now, it is recommended to assume $\delta_f$ between concrete pile and sand is not so different from $\delta_f$ between steel pile and sand. Since D50 of Florida soil is somewhat between 0.1 mm and 0.3 mm, it was decided to use 30° as $\delta_f$.

**Eslami and Fellenius Method**

This method was proposed by Eslami and Fellenius from the study of 102 cases around the world. This is the method that uses cone tip resistance ($q_c$) and pore pressure ($u$) to predict the axial pile capacity. CPT sleeve friction is only used to identify the soil type.

The pile’s unit tip capacity: 

$$q_t = q_{eg}$$

where: $q_{eg}$ is the geometric average of the effective cone resistance. The effective cone resistance is calculated by subtracting the hydrostatic pressure from the cone resistance if pore pressure data is not available.

The influence zone proposed by the Eslami and Fellenius is as follows:

- 2D above and 4D below the pile tip when the pile is installed through a dense soil into a weak soil.
- 8D above and 4D below the pile tip when the pile is installed through a weak soil into a dense soil.

The pile’s unit skin friction:

$$f_s = c_s * q_e$$

where: $c_s$ is functions of soil type.

$q_e$ is average of the effective cone resistance within the calculation layer.
The effective cone resistance is calculated by subtracting the hydrostatic pressure from the cone resistance if pore pressure data is not available.

**Powell et al. Method**

This method was proposed by Powell et al. from the study of 63 steel driven or jacked piles. The soil condition ranged from soft normal-consolidated clay to stiff over-consolidated clay and two sand sites. The parameters used by this method are cone tip resistance \( q_c \) and pore pressure \( u \), undrained shear strength \( s_u \), and a soil profile to predict the axial pile capacity.

The pile’s unit skin friction:

\[
f_s := \frac{q_c - \sigma_v}{k_1} \quad k_1 := 10.5 + 13.3 \log \left( \frac{q_c - \sigma_v}{\sigma_v} \right)
\]

where: \( q_c \) is CPT tip resistance, with pore pressure correction for piezocones, \( \sigma_v \) is the total overburden stress, \( \sigma_v' \) is the effective overburden stress.

In order to calculate the effective overburden stress, hydrostatic pressure has been used. A reduction in \( k_1 \) needs to be applied if \( L/D > 60 \). The reduction factor is recommended by both Semple and Rigden (1948) and is included in the procedure suggested by Randolph and Murphy (1985).

The pile’s unit tip capacity:

\[
q_t := \frac{q_c - \sigma_v}{k_2} \quad k_2 := \frac{N_{kt}}{9}
\]

where: \( N_{kt} \) is cone factor, range from 10 to 20 based on local experience. 15 was used in current study.
UWA-05 Method

This method was proposed by Lehane et al. in 2005. This method is especially used to predict pile ultimate capacity in sand.

The pile’s unit tip capacity:

\[ q_t := q_{ca} \left[ 0.15 + 0.45 \left( 1 - \frac{D_{int}}{D^2} \right) \right] \]

where: \( q_{ca} \) is calculated by minimum path rule. \( D_{int} \) is the internal diameter for pipe pile, \( D \) is the outer diameter of the pile.

The pile’s unit skin friction:

\[ f_s := 0.03 q_c \left( 1 - \frac{D_{int}}{D^2} \right)^{0.3} \left[ \max \left( \frac{h}{D}, 2 \right) \right]^{0.5} + 4G \frac{\Delta r}{D} \tan(\delta_{cv}) \]

\[ G := q_c \cdot 185 \cdot \left\{ \frac{q_c}{\text{Pa}} \right\}^{0.7} \cdot \left( \frac{\sigma'_{\text{vo}}}{\text{Pa}} \right) \]

where:
- \( q_c \): average CPT tip resistance within the calculated soil layer;
- \( \text{Pa} \): the atmosphere pressure;
- \( G \): the sand shear modulus;
- \( h \): the height above the pile tip. In order to prevent too large a \( K_c \) value, \( h/R \geq 8 \);
- \( \sigma'_{\text{vo}} \): effective over-burden pressure;
- \( \Delta r \): interface dilation, 0.02 mm was used by current study;
- \( \delta_{cv} \): constant volume interface friction angle between sand and pile.
Since the large variations are possible, it is recommended to use ring shear test to obtain the direct measurement; In the absence of lab tests, the trends between $\delta_{cv}$ and $D_{50}$ recommended by ICP-05 with the upper limit 0.55 for the $\tan(\delta_{cv})$ are considered reasonable. Since $D_{50}$ of Florida soil is somewhat between 0.1 mm and 0.3 mm, it was decided to use 29° as $\delta_{cv}$, which will give the $\tan(\delta_{cv})$ 0.55.

**Zhou et al. Method**

This method was proposed by Zhou et al. after the study of 96 pre-cast driven concrete piles in several eastern Chinese provinces. It provides a satisfactory predictions (80% of the predicted errors are within 20% of the true load test results). The soil condition ranged from sand to clayey soil. The parameters used by this method are cone tip resistance ($q_{c}$) and sleeve friction ($f_{s}$) to predict the axial pile capacity. One of the interesting points about this method is that it predicts limit load capacity in stead of ultimate load capacity. The limit load is defined as the load near the starting point of the straight line portion on the load test curve, the point where the shaft resistance of pile would be fully mobilized, while the end resistance only partially mobilized. If the point is not obvious form the data, they recommend using the load at a relative settlement of 0.4 – 0.5.

The pile’s unit tip capacity:

$$q_{t} = \alpha \cdot q_{ca}$$

where: $q_{ca}$ is average CPT tip resistance within 4D above and 4D below the pile tip;

$\alpha$ is the function of soil type and $q_{ca}$, use the following equations to calculate $\alpha$ value;

- Soil Type I: $\alpha := 0.71 \cdot q_{ca}^{0.25}$
- Soil Type II: $\alpha := 1.07 \cdot q_{ca}^{0.35}$
Soil type is defined as:  

Soil Type I: \( q_{ca} > 2 \text{ MPa} \) and \( f_{sa} / q_{ca} < 0.014 \)

Soil Type II: other than Soil Type I.

The pile’s unit skin friction:

\[ f_s := \beta \cdot f_{sa} \]

where: \( f_{sa} \) is average CPT sleeve friction along the calculated soil layer;

\( \beta \) is the function of soil type and \( f_{sa} \), use the following equations to calculate \( \beta \) value.

Soil Type I: \( \beta := 0.23 f_{sa}^{0.45} \)

Soil Type II: \( \beta := 0.22 f_{sa}^{0.55} \)
Figure 2-1. Regular cone penetrometer
CHAPTER 3
MATERIALS AND METHODS

Dual Tip Penetrometer (DTP)

The DTP is the latest version of a series of devices developed at the University of Florida intended for identifying cemented sands. Daniel Hart developed the first version of the device in 1996 while he was a graduate student at the University of Florida. The device at that time had a lip welded onto the top of the friction sleeve. However, this meant that the bearing reading measured by the lip was added to the frictional component measured by the friction sleeve strain gauge. The lip also made it difficult to remove the cone from the ground. In 1998, Randell Hand eliminated the welded lip and welded a bearing annulus onto the friction reducer coupler. The annulus was therefore located about 20 inches above the top of the cone’s friction sleeve. Strain gages were used to measure resistance in the annulus. The voltage output was translated into a second “q_c” (tip resistance) reading. Steve Kiser and Hogentogler & Co. Inc. improved on this design and came up with the dual tip penetrometer in 1999. In 2004, Hogentogler & Co. Inc. converted the DTP from analog into digital cone, which is the latest version of DTP equipped in the cone truck at the State Material Office.

The DTP is similar to conventional cone penetrometers except for a second tip (actually an annulus) just above the friction sleeve as shown in Figure 3-1. The second tip has the same angle (60°) and bearing area (10 cm²) as the regular cone. The first tip was originally designed to break down the cohesive bonds of cemented sand while the second tip was meant to measure the residual or broken-up bearing resistance. Based on the relationship between Tip 1 and Tip 2 along with the soil profile, through a large number of experiments, cemented sand identifiers would be identified.
Locate Cemented Sites

Cemented sands exist in many areas of the United States, including California, Texas, Florida, and along the banks of the lower Mississippi River. Lightly cemented sands are usually misidentified in the CPT test, which can cause design problems. SPT borings log are one resource used to identify cemented sand. However, if the cementation is very weak and can be easily broken by finger pressure, it might not be noticed by the field technicians.

In order to incorporate cemented sands into the new proposed design method, strongly cemented and lightly cemented sand sites were identified using two databases, FDOT and UF. There are hundreds of in-situ and load test data in these two databases. Both SPT data and boring logs were searched to identify cemented sand sites. CPT data were also searched and combining with SPT N (Qc/N) to locate cemented sand sites. All previous projects reports were reviewed to find soil and load test data. In case of sites where load test and SPT data were available, but no CPT data, the sites were flagged for future CPT and DTP testing.

There were a total of 21 cases where load test, SPT and CPT (DTP in some cases) are all available. Figure 3-2 shows the relative locations of these 21. These cases were used to calibrate the ultimate pile capacity prediction methods and their corresponding LRFD resistance factors $\Phi$.

Axial Ultimate Pile Capacity Prediction Methods

A total of 14 prediction methods have been analyzed in my research. They are: the Schmertmann method, de Ruiter and Beringen method, Penpile method, Price and Wardle method, Tumay and Fakhroo method, Aoki and De Alencar method, Philipponnat method, LCPC (Bustamante and Gianeselli) method, Almeida et al. method, MTD (Jardine and Chow) method, Eslami and Fellenius method, Powell et al. method, UWA-05 method, and Zhou et al. method. Because most of the methods involve complicated calculation and digital CPT data
make these calculations time intensive, a MathCAD program was used to calculate each of predictions. For each method there is one MathCAD program that was presented in the appendix.

Figure 3-3 shows the program for the Philipponnat method. The colored fields are inputs. The CPT data input is an Excel file formatted with four columns (depth, tip resistance, sleeve friction, and pore pressure). Other inputs are diameter or edge length of pile, embedment length, layer depths. After all required fields have been inputted, the program will calculate ultimate tip resistance, ultimate skin friction and Davisson capacity (1/3 of ultimate tip resistance + ultimate skin friction).

**Load and Resistance Factor Design (LRFD)**

Over the past two decades, Load and Resistance Factor Design (LRFD) have been incorporated in structural and geotechnical designs. One of the benefits of LRFD is its consistent reliability in design practice. Many state DOT’s, including FDOT, are now implementing AASHTO (American Association of State Highway and Transportation Officials) LRFD Specifications. Hence, the object of my research is to update FDOT’s pile/shaft design procedure based on CPT and DPT data and access the LRFD resistance factor for each pile capacity prediction method. Even though LRFD requires both load and resistance factors, the resistance factor is considered as a variable for each static pile capacity prediction method whereas load factors are typically constants, based on local experience.

**Modified First Order Second Moment (FOSM) Approach**

The LRFD approach used in my research is the modified FOSM (First Order Second Moment Approach). The modification was developed at the University of Florida (Styler, 2006) due to the difference between FORM (First Order Reliability method) and FOSM resistance factor in NCHRP Report 507. The modified portion in FOSM is the term COV (Q). The
previous FORM assumes \( \text{COV}(Q) = \text{COV}(Q_D) + \text{COV}(Q_L) \). It was found that this equation was incorrect and a modified \( \text{COV}(Q) \) was formulated as:

\[
\text{COV}(Q) := \sqrt{\frac{Q_D^2 \lambda_{QD}^2 \text{COV}_{QD}^2 + Q_L^2 \lambda_{QL}^2 \text{COV}_{QL}^2}{Q_D^2 + 2Q_D^2 \lambda_{QD} \lambda_{QL} \lambda_{QL} + Q_L^2}}
\]

where: \( Q_D/Q_L = \) Dead to live load ratio, varies from 1.0 to 3.0 (spans, \( L = 57-170 \) ft. Since it is not very sensitive, a value of 2.0 used herein),

\( \lambda_{QD}, \lambda_{QL} = \) Dead load and live load bias factors, \( \lambda_{QD} = 1.08, \lambda_{QL} = 1.15 \) (recommended by AASHTO 1996/2000),

\( \text{COV}_{QD}, \text{COV}_{QL} = \) Dead load and live load coefficients of variation,

\( \text{COV}_{QD} = 0.128, \text{COV}_{QL} = 0.18 \) (recommended by AASHTO).

Based on this modified FOSM, it was concluded that the difference between FORM and FOSM was little if any. Since FORM involves complicated calculation process, the modified FOSM is the approach used in my research.

The following section provides a detailed deviation of LRFD resistance factor using modified FOSM approach.

**Limit state equation**

In this approach, load and resistance are assumed to be lognormal distribution. Therefore, the limit state equation is as follows:
\[ G := \ln(R) - \ln(Q) \]

\[ E(G) := E(\ln(R)) - E(\ln(Q)) \]

\[ E(\ln(R)) := \ln(E(R)) - \frac{1}{2} \cdot \ln\left[1 + (\text{COV}(R))^2\right] \]

\[ E(\ln(Q)) := \ln(E(Q)) - \frac{1}{2} \cdot \ln\left[1 + (\text{COV}(Q))^2\right] \]

\[ E(G) := \ln\left[ \frac{E(R) \sqrt{1 + (\text{COV}(Q))^2}}{E(Q) \sqrt{1 + (\text{COV}(R))^2}} \right] \]

It is assumed that \( R \) and \( Q \) are statistically independent, therefore \( \sigma_G \):

\[ \sigma_G := \sqrt{\text{VAR}(\ln(R)) + \text{VAR}(\ln(Q))} \]

\[ \sigma_G := \sqrt{\ln\left[1 + (\text{COV}(R))^2\right] \cdot \ln\left[1 + (\text{COV}(Q))^2\right]} \]

where: \( \text{COV}(R), \text{COV}(Q) \) = Resistance, load coefficient of variation.

**Reliability index**

\[ \beta := \frac{E(G)}{\sigma_G} \]

\[ \beta := \frac{\ln\left[ \frac{E(R) \sqrt{1 + (\text{COV}(Q))^2}}{E(Q) \sqrt{1 + (\text{COV}(R))^2}} \right]}{\sqrt{\ln\left[1 + (\text{COV}(R))^2\right] \cdot \ln\left[1 + (\text{COV}(Q))^2\right]}} \]

\[ E(R) := E(Q) \cdot \exp\left[ \beta \sqrt{\ln\left[1 + (\text{COV}(R))^2\right] \cdot \ln\left[1 + (\text{COV}(Q))^2\right]} - \frac{1 + (\text{COV}(Q))^2}{\sqrt{1 + (\text{COV}(R))^2}} \right] \]

**Resistance factor \( \Phi \)**

The LRFD equation:

\[ \phi R_n \geq \sum \eta \gamma_i Q_i \]
where: $\Phi = \text{Resistance factor}$

$R_n = \text{Nominal Resistance}$

$\eta = \text{Load modifier for importance, redundancy and ductility}$

$\gamma_i = \text{Load factors}$

$Q_i = \text{Force effects}$

Load modifier $\eta$ is set equal to 1 which leaves all uncertainty on the resistance factor $\phi$.

For driven pile design, two load effects, dead load $Q_D$ and live load $D_L$ are considered.

Therefore, $\gamma_D$ and $\gamma_L$ are considered as load factors for dead load and live load, respectively. The LRFD equation becomes:

$$\phi \frac{R_n}{\eta} \geq \gamma_D Q_D + \gamma_L Q_L$$

$$\phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{R_n}$$

Since:

$$E(R) := \lambda_R \cdot R_n$$

where: $\lambda_R = \text{Mean bias (mean of resistance bias factor)}$,

$$\lambda_R := \frac{1}{N} \sum_{i=1}^{N} \lambda_{R_i}$$

$$\lambda_{R_i} := \frac{R_{mi}}{R_{ni}}$$

where: $R_{mi} = \text{Measured capacity from load test data}$

$R_{ni} = \text{Predicted capacity form CPT data}$

$N = \text{The number of cases}$

$$\phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{E(R)} \cdot \frac{\lambda}{\lambda_R}$$
By inserting $E(R)$ into the above equation, the following equation can be derived:

$$\phi \geq \frac{\lambda_R (\gamma_D Q_D + \gamma_L Q_L) \sqrt{1 + (\text{COV}(Q))^2}}{E(Q) \cdot \exp \left[ \beta \cdot \sqrt{\ln \left[ \frac{1 + (\text{COV}(Q))^2}{1 + (\text{COV}(R))^2} \right]} \right]}$$

Since the dead load and live load are considered as statistically independent, $E(Q)$ can be expressed as follows:

$$Q := \gamma_D Q_D + \gamma_L Q_L$$

$$E(Q) := \lambda Q_D Q_D + \lambda Q_L Q_L$$

where: \(\lambda_{QD}, \lambda_{QL} = \text{Dead load and live load bias factors;}\)

\(\lambda_{QD} = 1.08, \lambda_{QL} = 1.15 \) (recommended by AASHTO 1996/2000)

By inserting $E(Q)$ into the equation for resistance factor, and divide both numerator and denominator by $Q_L$, the following equation results:

$$\phi \geq \frac{\lambda_R (\gamma_D Q_D + \gamma_L Q_L) \sqrt{1 + (\text{COV}(Q))^2}}{\left(\lambda_Q D Q_D + \lambda Q_L Q_L\right) \cdot \exp \left[ \beta \cdot \sqrt{\ln \left[ \frac{1 + (\text{COV}(Q))^2}{1 + (\text{COV}(R))^2} \right]} \right]}$$

This equation was traditionally used to calibrate the resistance factor using FOSM in AASHTO’s specifications by assuming $\text{COV}(Q)^2 = \text{COV}(QD)^2 + \text{COV}(QL)^2$. However, as mentioned previously, this assumption is incorrect. The correct derivation is as follows:

$$\text{VAR}(Q) := Q_D^2 \cdot \text{VAR}(\gamma_D) + Q_L^2 \cdot \text{VAR}(\gamma_L)$$

$$Q_D := \gamma_D Q_D$$

$$Q_L := \gamma_L Q_L$$
\[
\text{COV} (QD)^2 := \frac{\text{VAR} \left( \gamma_D \right) \cdot Q_D^2}{\lambda \cdot Q_D^2 \cdot Q_D^2}
\]
\[
\text{COV} (QL)^2 := \frac{\text{VAR} \left( \gamma_L \right) \cdot Q_L^2}{\lambda \cdot Q_L^2 \cdot Q_L^2}
\]
\[
\text{VAR} \left( \gamma_D \right) := \lambda \cdot Q_D^2 \cdot \text{COV} (QD)^2
\]
\[
\text{VAR} \left( \gamma_L \right) := \lambda \cdot Q_L^2 \cdot \text{COV} (QL)^2
\]
\[
\text{VAR}(Q) := Q_D^2 \cdot \lambda \cdot QD^2 \cdot \text{COV}(QD)^2 + Q_L^2 \cdot \lambda \cdot QL^2 \cdot \text{COV}(QL)^2
\]
\[
\text{COV}(Q)^2 := \frac{\text{VAR}(Q)}{E(Q)^2}
\]
\[
\text{COV}(Q)^2 := \frac{Q_D^2 \cdot \lambda \cdot QD^2 \cdot \text{COV}(QD)^2 + Q_L^2 \cdot \lambda \cdot QL^2 \cdot \text{COV}(QL)^2}{Q_L^2 \cdot \lambda \cdot QD^2 + 2 \cdot \frac{Q_D}{Q_L} \cdot \lambda \cdot QD \cdot QL + \lambda \cdot QL^2}
\]

By inserting \text{COV}(Q) into the resistance equation, the following equation can be obtained:

\[
\phi \geq \lambda_R \left( \gamma_D \frac{Q_D}{Q_L} + \gamma_L \right) \exp \left( \beta \cdot \ln \left[ 1 + (\text{COV}(R))^2 \right] \right) \left( \lambda \cdot \frac{Q_D}{Q_L} + \lambda \cdot QL \right) \exp \left( \frac{\frac{Q_D^2 \cdot \lambda \cdot QD^2 \cdot \text{COV}(QD)^2 + \lambda \cdot QL^2 \cdot \text{COV}(QL)^2}{Q_L^2 \cdot \lambda \cdot QD^2 + 2 \cdot \frac{Q_D}{Q_L} \cdot \lambda \cdot QD \cdot QL + \lambda \cdot QL^2}}{1 + (\text{COV}(R))^2} \right)
\]
where: $\gamma_D = \text{Dead load factor (1.25, recommended by AASHTO (1996/2000))}$,

$\gamma_L = \text{Live load factor (1.75, recommended by AASHTO 1996/2000)}$,

$Q_D/Q_L = \text{Dead to live load ratio, varies from 1.0 to 3.0 (spans, L = 57-170 ft,}

\text{is not very sensitive, a value of 2.0 used herein)}$

$\lambda_R = \text{Mean of resistance bias factor,}$

$\text{COV (R) = Resistance coefficient of variation,}$

$\lambda_{QD}, \lambda_{QL} = \text{Dead load and live load bias factors,}$

$\lambda_{QD} = 1.08, \lambda_{QL} = 1.15 \text{ (recommended by AASHTO 1996/2000),}$

$\text{COV}_{QD}, \text{COV}_{QL} = \text{Dead load and live load coefficients of variation,}$

$\text{COV}_{QD} = 0.128, \text{COV}_{QL} = 0.18 \text{ (recommended by AASHTO)}$

$\beta_T = \text{Target reliability index, AASHTO and FHWA recommend values from}$

$2.0 \text{ to 3.}$

Of major importance in estimating LRFD resistance factor, $\Phi$, are the resistance’s bias

factor ($\lambda_R$), and the resistance’s coefficient of variation (COV(R)). Both are computed from: 1) the nominal predicted resistance, $R_{ni}$ (predicted pile capacity), and 2) the measured pile capacity, $R_{ni}$, i.e. load test results. Based on the ratio of measured to predicted pile capacities, $\lambda_{Ri}$, for each of the sites, the mean, $\lambda_R$, standard deviation, $\sigma_R$, and coefficient of variation, COV(R), were determined for all 14 CPT methods. Using these inputs, the LRFD resistance factors, $\Phi$, were determined with a reliability index, $\beta$, of 2.5 for each method. Because driven piles are usually in groups and redundancy will require lower reliability, 2.5 was used in my research.

**Differentiate Ultimate Skin Friction and Tip Resistance from Load Test Data**

All of the load tests were conventional top-down static load tests performed for which load-settlement data exist (see Figure 3-4.). Using FDOT’s load testing protocol (i.e.
Specification Section 455), the Davisson Capacity was assessed for each load versus settlement curve. The Davisson Capacity, referred to as the measured capacity, $R_m$, generally occurs for settlements that are tolerable (i.e. less than 1”), under service loading conditions. These values were subsequently used with the predicted capacities, $R_n$, (where $R_n = Q_{S_{ult}} + \frac{1}{3} * Q_{T_{ult}}$) to assess the LRFD resistance factor, $\Phi$ for each of the fourteen CPT methods.

However, the load versus settlement curve does not provide the ultimate skin friction and tip resistance. In order to evaluate the accuracy of each prediction method for ultimate skin friction and tip resistance separately, each needs to be estimated using this load test curve. Figure 3-5 shows the proposed process that was developed to separate the two attributes. For the majority of the tests, the piles were not loaded sufficiently to induce a plunging failure. While the ultimate skin friction is likely fully mobilized at small displacements (0.1”), the tip resistance is not. Thus, it is not possible to determine the ultimate tip resistance (i.e., the tests ended after one inch displacement was reached, whereas two inches of displacement or approximately $D/10$ [D is in inches] is typically needed to induce a plunging failure) unless the load test curve is extrapolated to the two inch value. The idea of extending the load test curve follows from deBeer’s method for determining pile capacity.

The proposed differentiate method is as follows (see Figure 3-5 for an example of the process):

a. The load test is plotted in log-log space.

b. Two straight trend lines are drawn among the data points, with the second line extended or extrapolated to two inches.

c. Two distinct loads are indentified: the first occurs at the intersection of the two sloped lines and the other at the presumed displacement of two inches.

d. Since the first load typically occurred at a displacement of approximately 0.1 to 0.2 inches, (i.e., 5% to 10% of the extrapolated value), it was assumed that 5% of
the ultimate tip resistance was mobilized. Therefore, the first load is assumed to be the sum of the ultimate skin friction and 5% of the tip resistance, while the second load is the ultimate skin plus tip resistance. Therefore, the separate contributions of ultimate tip and skin friction can be calculated.

**The Proposed UF Method**

The proposed method uses the following equation to estimate the ultimate pile unit tip resistance, $q_t$, from the CPT tip resistance, $q_c$:

$$q_t = k_b \cdot q_{ca} \text{(tip)} \leq 150 \text{ tsf}$$

where: $k_b$ is a factor that depends on the soil type as shown in Table 3-1.

The soil type was determined using the soil classification chart for the standard electronic friction cone (Robertson et al, 1986) which includes tip resistance and sleeve friction. Soil cementation was determined by SPT samples, DTP tip2/tip1 ratio or SPT $q_c/N$ ratio (>10).

$q_{ca} \text{(tip)}$: the average CPT tip resistance, which is calculated as follows:

$$q_{ca} \text{(tip)} = (q_{ca \text{ above}} + q_{ca \text{ below}}) / 2$$

$q_{ca \text{ above}}$: average $q_c$ measured from the tip to 8 D above the tip;

$q_{ca \text{ below}}$: average $q_c$ measured from the tip to 3 D below the tip for sand or 1D below the tip for clay;

Impose the condition: $q_{ca \text{ above}} \leq q_{ca \text{ below}}$, which means if $q_{ca \text{ above}} \geq q_{ca \text{ below}}$, let $q_{ca} \text{(tip)}$ be equal to $q_{ca \text{ below}}$.

The proposed method uses the following equation to estimate the ultimate skin friction resistance of the pile, $f_s$, from the CPT tip resistance, $q_c$:

$$f_s = q_{ca} \text{(side)} * 1.25 / F_s \leq 1.27 \text{ tsf}$$
where: \( F_s \): friction factor that depends on the soil type as shown in Table 3-2.

The following criterion was used to determine the sand state based on relative density: loose sand (R.D. < 40 \%), medium dense sand (40 \% < R.D. < 70 \%), and dense sand (R.D. > 70 \%).

\( q_{ca \ (side)} \): the average \( q_c \) within the calculating soil layers along the pile.
Table 3-1. Ultimate unit tip resistance factor $k_b$

<table>
<thead>
<tr>
<th>Well Cemented Sand</th>
<th>Lightly Cemented Sand</th>
<th>Gravel Sand</th>
<th>Slit</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.15</td>
<td>0.35</td>
<td>0.4</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Table 3-2. Ultimate unit skin friction empirical factor, $F_s$

<table>
<thead>
<tr>
<th>Well Cemented Sand</th>
<th>Lightly Cemented Sand</th>
<th>Gravel and Dense Sand</th>
<th>Medium Dense Sand</th>
<th>Loose Sand</th>
<th>Silt, Sandy Clay, Clayey Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>250</td>
<td>200</td>
<td>150</td>
<td>100</td>
<td>60</td>
<td>50</td>
</tr>
</tbody>
</table>
Figure 3-1. Dual tip penetrometer
Figure 3-2. The locations of 21 sites with load test data and CPT data
Figure 3-3. MathCAD program for Philipponnat method
Figure 3-4. Static load test, Apalachicola Bay Bridge (pier 3)
Example: \[ S + 5\% T = 249 \text{ tons} \]
\[ S + T = 352.5 \text{ tons} \]
\[ S = 243.5 \text{ tons} \]
\[ T = 109 \text{ tons} \]

\( S \) : Ultimate Skin Friction
\( T \) : Ultimate Tip Resistance

Figure 3-5. Separate the ultimate skin friction and tip resistance
CHAPTER 4
RESULTS

This chapter includes two parts. The first provides a synopsis of the DPT and CPT results from various sites and the discussion of the cemented sand identifier. The second part is the evaluation of current axial ultimate pile capacity prediction methodologies using LRFD statistical method and the proposed method, including its verifications.

Identification of Cemented Sand

Cemented sands exist in many areas of the United States, including California, Texas, Florida, and along the banks of the lower Mississippi River. They also exist in Norway, Australia, Canada, and Italy (Puppala et al. 1995). If the sands found with a CPT test are not known to be cemented, the high bearing readings may be misinterpreted as being due to high relative densities. This can lead to an underestimation of the liquefaction potential of the soil and overestimate the ultimate pile capacity. It is a very challenging soil type for geotechnical engineers. It is therefore important to identify such soil before the design of foundation. In previous practice, engineers used SPT borings and CPT (Qc/N) to identify cemented sand. However, there are two drawbacks: firstly, lightly cemented sand may not be able to be identified by field technicians since the strength between sand particles is very weak, and secondly, the SPT provides a discontinuous data so that engineer judgment is required to estimate the N value between data points for calculating Qc / N ratio. It would be very beneficial if the DTP could be used to identify cemented sand.

DTP & CPT Test Data at FDOT Bridge Sites

Many FDOT’s bridge sites have been revisited and had DTP and CPT tests performed. DTP and CPT tests were usually performed in pairs for comparison. All field tests were conducted by personnel from the State Material Office (SMO). The sites were chosen so that
load tests and SPT data were available, and several sites also had cemented sand presented. The sites include: Archer Landfill (Archer, FL), I-95 at Edgewood Avenue (Jacksonville, FL), Apalachicola River Bridge, Pier 3 (Apalachicola, FL), West Bay Bridge, Pier 20 (Bay County, FL), Port Orange Relief Bridge (Port Orange, FL), University of Central Florida (Orlando, FL), I-295 at Blanding Blvd (Jacksonville, FL), I-295 at Normandy Blvd. (Jacksonville, FL), and White City Bridge, Pier 5 and Pier 8 (White City, FL).

**Archer Landfill Site**

This site is one of the first sites used for DTP testing. The Archer landfill is an excellent site for initial scrutiny. Located in Archer, Florida, this site contains a very clean, fine sand deposit peripheral to the outside of the clay liner boundary. This area has been used for many years as a borrow area, where clean sands were mined for daily cover of the landfill debris.

No SPT data is available for this site. Figure 4-1 shows the CPT tip resistance, DTP Tip 1 and Tip 2 resistances. The Tip 1 and CPT tip resistance are very close to each other. Therefore, the second tip (the one above the friction sleeve) does not influence the reading of the first tip (the one at the same location as the CPT). This phenomenon was found in all paired CPT and DTP tests, which means the distance between these two tips is sufficient enough to eliminate the interaction between each other. Another finding is that the relative magnitudes between DTP Tip 2 reading (T2) and Tip 1 reading (T1) for the clear fine sand equal 0.5.

Figure 4-2 shows the comparison between friction ratios of CPT and DTP. From the plot, it can be found that the friction ratio of the DTP is much higher than that of CPT for sand (~ 3 times). The main reason for this increase is the normal stress adjacent to the friction sleeve due to the second tip. Therefore, the shear stress near the friction sleeve is also increased. The discrepancy in friction ratios between the DTP and CPT does not allow for soil classification unless a new soil classification chart for the DTP is created. However, the main purpose of DTP
is to identify cemented sand using cemented sand identifiers (T2/T1 ratio). This ratio is presented below in the following analysis.

**I-95 at Edgewood Avenue Site**

This site is located in Jacksonville and is a FDOT bridge site. Figure 4-3 shows the CPT tip resistance, DTP Tip 1 and Tip 2 resistances. The Tip 1 and CPT tip resistance are very close to each other. It can be found that the T2/T1 ratio varies with depth. Since the depth is function of soil type, T2/T1 is a function of soil type, depth, or both. Through later test results, it was found that the T2/T1 ratio is solely a function of soil type.

Figure 4-4 shows the comparison between the friction ratios of the CPT and DTP. From the plot, it can be observed that the DTP’s is again higher than the CPT’s for sandy soil (about 3 times), but it is close to the CPT’s for clayey soil. Since the friction ratio of DTP is different from that of CPT and hence could not be used to classify the soil, the friction ratio of following cases will not be presented.

Figure 4-5 shows the comparison between the T2/T1 ratio and Qc/N ratio with depth. Generally speaking, when the Qc/N ratio is less than 3, the soil is considered as a clayey, silty soil. With Qc/N ratios between 3 to 10, the soil is considered a regular sand, and above 10, cemented sand may be encountered. For the I-95 at Edgewood site, the soil profile is as follows: loose to medium dense sand with a thin layer of silt (0 feet – 27 feet), clay (27 feet – 36 feet), and dense, silty fine sand (36 feet – 59 feet). This was confirmed by the Qc/N ratio. The T2/T1 ratio is 0.5 for loose to medium dense sand, greater than or equal to 1 for clayey silty soil, and greater than 1 for very dense, silty sand.

**Apalachicola River Bridge Site**

This site is also a FDOT bridge site located in Apalachicola, FL. Figure 4-6 shows the CPT tip resistance, DTP Tip 1 and Tip 2 resistances. The Tip 1 and CPT tip resistance are very
close to each other. The soil profile in this site is as follows: two lightly cemented sand layers (0 feet – 25 feet), sandy silt (25 feet – 45 feet), silty clay (45 feet – 60 feet), and lightly cemented sand layer (under 60 feet). Figure 4-7 gives the comparison between the T2/T1 ratio and Qc/N ratio with depth at the Apalachicola River Bridge site. The T2/T1 ratio is less than 0.5 for lightly cemented sand, between 0.5 to 1 for sandy silt and silty clay soil.

**West Bay Bridge Site**

This site is a FDOT bridge site located in Bay County, FL. Figure 4-8 shows two CPT tip resistances, DTP Tip 1 and Tip 2 resistances. The reason for presenting two CPTs is to show that there is significant spatial variability at this site. These two CPTs are within 100 feet and are totally different. The DPT Tip 1 and CPT1 Tip resistances are very close under 68 feet but DPT T1 is much higher than CPT1 tip resistance above 68 feet. However, DPT T1 is very similar to the CPT2 tip resistance at the depth above 64 feet but significant smaller at the depth below 64 feet. In this site, there are two lightly cemented sand layers and one well cemented sand layer. Figure 4-9 provides a comparison between T2/T1 ratio and Qc/N ratio along the depth at this site. The T2/T1 ratio is less than 0.5 for lightly cemented sand, and close to 1 for well cemented sand.

**Port Orange Relief Bridge**

This site is a FDOT bridge site located in Port Orange, FL. Figure 4-10 shows CPT tip resistance, DTP Tip 1 and Tip 2 resistances. The DPT Tip 1 and CPT1 Tip resistances are very close to one another. At this site, there are two well cemented sand layers with one clayey silt and one lightly cemented sand layer in the middle.

Figure 4-11 shows a comparison between T2/T1 and Qc/N with depth. The T2/T1 ratio is less than 0.5 for lightly cemented sand and clayey silt and close to 1 for well cemented sand. The interesting finding in this figure is that the Qc/N ratio is higher for lightly cemented sand than that for well cemented sand. The lightly cemented sand is loose to medium dense sand with
very low N blow count; therefore, it does not require a high tip resistance ($q_c$) to acquire very large $Q_c/N$ ratio. Therefore, this ratio cannot indicate just how strong the cementation is.

**University of Central Florida**

This site is located at the University of Central Florida, Orlando, FL. Figure 4-12 shows CPT tip resistance, DTP Tip 1 and Tip 2 resistances. The DPT Tip 1 and CPT1 Tip resistances are similar. At this site, there are two lightly cemented sand layers ($T_2/T_1 < 0.5$) and four well cemented sand layers ($T_2/T_1 \approx 1$) with one clayey silt layer with shell (38 feet – 53 feet) and two clay layers (4 feet – 6 feet, 30 feet – 34 feet).

Figure 4-13 gives a comparison between $T_2/T_1$ and $Q_c/N$ with depth. The $T_2/T_1$ ratio is less than 0.5 for lightly cemented sand, close to 1 for well cemented sand, and less than 0.5 for clay and clayey silt. The interesting finding at this site is that the $T_2$ values in clay and clayey silt are very low and even negative. That means that the clay is highly sensitive and loses its shear strength dramatically after the disturbance of the first tip.

**I-295 at Blanding Blvd Site**

This is a site in Jacksonville, FL. Figure 4-14 shows CPT tip resistance, DTP Tip 1 and Tip 2 resistances. At this site, there are two sand layers (0 feet – 32 feet, 53 feet – 58 feet) with four lightly cemented sand layers within ($T_2/T_1 < 0.5$) and one clayey silt layer (32 feet – 53 feet) with one lightly cemented sand within. Figure 4-15 shows the comparison between $T_2/T_1$ and $Q_c/N$ with depth. The $T_2/T_1$ ratio is less than 0.5 for lightly cemented sand, close to or greater than 1 for clayey silt.

**I-295 at Normandy Blvd Site**

This is another site in Jacksonville, FL. Figure 4-16 shows CPT tip resistance and DTP Tip 1 and Tip 2 resistances. At this site, there are three lightly cemented sand layers ($T_2/T_1$
<0.5). Figure 4-17 shows a comparison between T2/T1 and Qc/N with depth. The T2/T1 ratio is less than 0.5 for lightly cemented sand.

White City Bridge Site (Pier 5)

This is a site in White City, FL. Figure 4-18 shows CPT tip resistance and DTP Tip 1 and Tip 2 resistances. At this site, there are three lightly cemented sand layers, two silty sand layers and three clay layers. Figure 4-19 indicates the comparison between T2/T1 and Qc/N with depth. The T2/T1 ratio is less than 0.5 for lightly cemented sand, is between 0.5 – 1 for silty sand and close to or greater than 1 for clay.

White City Bridge Site (Pier 8)

This is the second pier at White City Bridge site in White City, FL. Figure 4-20 shows CPT tip and DTP Tip 1 and Tip 2 resistances. There are three lightly cemented sand layers, one silty sand layer, one clay layer, and one sand layer. Figure 4-21 gives the comparison between T2/T1 and Qc/N with depth. The T2/T1 ratio is less than 0.5 for lightly cemented sand, is between 0.5 and 1 for silty sand, greater than 1 for clay and 0.5 for sand.

Identifier for Cemented Sand Summarized from DTP Data

Based on the data collected to date, the T2 /T1 ratio appears to be a reasonable predictor or identifier for cemented sand. The following ranges for each soil type are based on the limited data collected in Florida. It may be different elsewhere and more data needs to be collected to verify these values.

T2/T1:

\[ \approx 0.5 \quad \text{Non-Cemented Sand (Loose and Medium Dense Sand)} \]

\[ = 0.3\sim0.5 \quad \text{Lightly Cemented Sand} \]

\[ \approx 1 \quad \text{Strongly Cemented Sand} \]

\[ = 0.5\sim1 \quad \text{Silty Sand, Sandy Silt and Silty Clay} \]
< 0.5      Highly Sensitive Clay

> 1         Dense Silty Sand

≥1          Clayey Silt, Silty Clay and Clay

The reasons for the identifier having different values for different soil types may seem ambiguous. However, the following is one possible explanation that will have to be verified by additional testing.

For non-cemented sand (loose and medium dense sand), T2 is less than T1 due to the compression of the sand after the first tip penetrates the soil. The value 0.5 for the identifier is function of the location of the second tip. Therefore, it is the unique character for the DTP used in my research. However, for dense sand, due to the dilation of sand particles after the disturbance of the first tip, T2 is greater than T1. For highly sensitive clay, the disturbance of the first tip dramatically reduces the shear strength of clay that results in the T2/T1 of less than 0.5. For lightly cemented sand, the cementation bonds are totally broken after penetration of the first tip so that T2 is the same as that for non-cemented sand. However, T1 for lightly cemented sand is higher than that for non-cemented sand, which causes T2/T1 ratio to be less than 0.5. For well cemented sand, the cementation bonds are so strong that they cannot all be broken or are only partially destroyed. Therefore, T2 is close to T1 for well-cemented sand. For low sensitive clay, clayey silt and silt clay, the penetration of the first tip may reduce the shear strength of the soil and the reduction depends on the strength of clay. However, the excess pore pressure generated by the intrusion of the first tip also increases T2. The resultant effects cause T2/T1 ratio to be less than, close to or greater than 1. For a mixed soil such as silty sand and sandy silt, T2/T1 is between 0.5 and 1 due to the combining effects.
Assessing LRFD Resistance Factors, $\Phi$, Based on Reliability (Risk)

Two set of data were used for the LRFD assessment. They include 21 Florida cases and 28 in Louisiana. The data for Florida and Louisiana are analyzed separately to see how each method works for their respective soil type (Florida: predominantly sand, Louisiana: predominantly clay).

Criteria Used to Quantify Pile Capacity from Load Test Data

The criteria used to quantify pile capacity from load test data are the same as provided in FDOT specification for 21 cases in Florida. It is defined as the load that causes a pile tip deflection equal to the calculated elastic compression plus 0.15 inches plus $1/120$ of the pile width in inches for piles 24 inches or less in width. For piles greater than 24 inches, it is equal to the calculated elastic compression plus $1/30$ of the pile width (FDOT Specification 2007, 455-2.2.1). The criteria are similar to Davisson capacity except for piles greater than 24 inches. All methods predict the ultimate pile tip and skin capacity, therefore the nominal capacities are calculated by the sum of ultimate skin capacity and $1/3$ of ultimate tip capacity ($R_n = Q_{S \text{ult}} + 1/3 * Q_{T \text{ult}}$). The ratio of measure capacity (FDOT failure load) to the nominal capacity is used to assess LRFD resistance factor.

Since clay is the predominant soil type in Louisiana, a typical load test curve is shown in Figure 4-22. The ultimate pile capacity is less than the peak load. Therefore, it is not appropriate to compare the predicted nominal capacities ($R_n = Q_{S \text{ult}} + 1/3 * Q_{T \text{ult}}$) with Davisson capacity ($Q_{\text{Davisson}}$). Thus it was decided to compare the predicted ultimate pile capacity ($R_{\text{ult}} = Q_{S \text{ult}} + Q_{T \text{ult}}$) with the ultimate capacity from the load test (load at 2 inches of displacement). According to FDOT specification, the allowable load of any pile tested must be either 50 % of the maximum applied load or 50 % of the failure load, whichever is smaller.
LRFD Resistance Factor, \( \Phi \), for Florida Soil

Using the 14 CPT methods with the cone data from the 21 load test sites in Florida, - see Tables 4-1, 4-2, and 4-3 - the ultimate skin friction, ultimate tip resistance, and Davisson capacity were determined for each test pile. Also shown in these tables are the measured ultimate skin friction, ultimate tip resistance, Davisson Capacity (from load test curve and the proposed process to separate skin and tip). The highlighted cases represent cemented sand sites. Shown in Figure 4-23 is a plot of the ratio (measured Davisson capacity/predicted Davisson capacity) for each method for all 21 piles. In this plot, the cases are sorted by diameter. No correction between the ratio \( \lambda \) and diameter. It was attempted to sort the case by length and there still was no correction between the ratio and length. The methods above the value of one are conservative. Next, the LRFD resistance factors, \( \Phi \), were assessed for each method.

Based on the ratio of measured to predicted pile capacities, \( \lambda_{Ri} \), for each of the sites, the mean, \( \lambda_{R} \), standard deviation, \( \sigma_{R} \), and coefficient of variation, \( \text{COV}_{R} \), were determined for all 14 CPT methods. Using the computed mean, \( \lambda_{R} \) and coefficient of variation, \( \text{COV}_{R} \), the LRFD resistance factors, \( \Phi \), were determined with a reliability index, \( \beta \) of 2.5 (recommended for redundant foundation elements) for each method. Tables 4-4, 4-5, and 4-6 are the LRFD resistance factors for ultimate skin friction, ultimate tip resistance, and Davisson capacity. The last method is the one proposed by UF and was created by modifying the most promising method, the Philipponnat method (1980).

Evident from Tables 4-4, 4-5, and 4-6, many of the methods result in higher resistance factors, \( \Phi \). However, if the method has a substantial amount of scatter (a high \( \text{COV}_{R} \)), Table 4-4 (e.g. Schmertmann, Prince, etc.), then the LRFD resistance factor, \( \Phi \), will be adjusted downward.
Of strong interest is a ranking of the 14 CPT methods investigated. The latter may be determined as follows:

\[ R_{\text{design}} = \Phi R_n \] (i.e. predicted capacity from individual method)

However \[ \lambda_R = R_m \] (i.e. Davisson) / \( R_n \) (predicted capacity)

Substituting the above equation into the \( R_{\text{design}} \) equation for \( R_n \)

\[ R_{\text{design}} = \left( \frac{\Phi}{\lambda_R} \right) R_m \]

The term \( \left( \frac{\Phi}{\lambda_R} \right) \) in the above equation identifies the percent of measured Davisson capacity that is available for design. Obviously, the higher the \( \left( \frac{\Phi}{\lambda_R} \right) \) term, the better the method. Shown in Tables 4-4, 4-5, and 4-6 are \( \left( \frac{\Phi}{\lambda_R} \right) \) terms for each method. Clearly, Philipponnat & LCPC are the better methods. Interestingly, both the Philipponnat & LCPC methods use just the CPT tip resistance, \( q_c \), to predict axial pile capacity. Finally, the proposed UF method gives the value of 0.617 for \( \Phi/\lambda_R \), which means 61.7% of load test measured capacity can be used for design.

**LRFD Resistance Factor, \( \Phi \), for Louisiana Soil**

Tables 4-7, 4-8, and 4-9 show the ultimate skin friction, ultimate tip resistance, and ultimate pile capacity using the 14 CPT methods with cone data from 28 load test sites in Louisiana. Shown in Figure 4-24 is a plot of the ratio (measured ultimate capacity/predicted ultimate capacity) for each method. Again, methods above one are conservative. Tables 4-10, 4-11, and 4-12 show the LRFD resistance factors for ultimate skin friction, ultimate tip resistance, and ultimate pile capacity.

Shown in Tables 4-10, 4-11, and 4-12 are the \( \left( \frac{\Phi}{\lambda_R} \right) \) terms for each method. From each table, it is found that the ranking of prediction methods are different from the Florida data. The
Jardine & Chow (MTD) & Philipponnat methods are the better methods. However, both the Jardine & Chow (MTD) and Philipponnat methods use only the CPT tip resistance, $q_c$, to predict axial pile capacity. The proposed UF method provides a value of 0.673 for $\Phi/\lambda_R$, which is second best method, and only slightly less than the MTD method.

In summation, it appears that the proposed UF method works very well for both sands (Florida soil) and clays (Louisiana soil).

**Evaluate the Prediction Methods Using the Bootstrap Method**

After calculating the $\lambda_i$ (Measured/Predicted) for each method and performing an LRFD resistance factor analysis, one question remains: how representative is the data to the population? Is there sufficient data to predict the pile capacity with a certain level of confidence? Fortunately, a very powerful tool is available, termed the Bootstrap method.

This method estimates the sampling distribution of an estimator by resampling with replacements from the original sample. For instance, there are total of 21 cases involving Florida soil. We have 21 $\lambda$s which are original samples from the whole population of $\lambda$s. We want to know how good the samples’ mean and standard deviations are.

There are three steps involved in the method. The first is to resample with replacement 21 $\lambda$s from our original samples, which means picking $\lambda$ one by one and after each pick it is placed back into the pool for the next pick. Therefore, in this new set of samples, some $\lambda$'s may appear more than once, but some not at all. The second step is to calculate the mean and standard deviation of the new sample set. The last step is to repeat the first and second steps numerous times (at least 1,000). Finally, one obtains the distribution of the mean and standard deviation of the new sample sets. From these one can determine how representative the samples are to the population.
Figures 4-25, 4-26, and 4-27 show the results of the Bootstrap analysis for Florida soils. It can be seen that the standard deviation of the resampled mean and resampled standard deviation are quite small (0.061, 0.042 respectively). Figures 4-28, 4-29, and 4-30 show the analysis for the proposed method for Louisiana soil. It also shows very small standard deviation (0.042, 0.027) for the resampled mean and standard deviation. Therefore, the proposed method does provide a higher level of quality predictions.

Figures 4-31, 4-32, and 4-33 show the frequency diagram of the original sample, resampled mean and resampled standard deviation for Schmertmann method for Florida soils. As it shows, the standard deviation of resampled mean and resampled standard deviation are somewhat larger (0.148, 0.243), which means the sample may not be representative of the population. In other words, in order to perform more accurate analysis to Schmertmann method, more data is required.
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<th>Aldi &amp; de Alencar</th>
<th>Penpile</th>
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Table 4-4. LRFD resistance factors, \( \Phi \), for CPT methods (ultimate skin friction, Florida soil)

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<th>( \text{COV}_R )</th>
<th>( \Phi )</th>
<th>( \Phi/\lambda_R )</th>
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<td>1.619</td>
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Table 4-5. LRFD resistance factors, \( \Phi \), for CPT methods (ultimate tip resistance, Florida soil)

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<th>( \text{COV}_R )</th>
<th>( \Phi )</th>
<th>( \Phi/\lambda_R )</th>
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Table 4-8. Predicted ultimate tip resistance for 14 CPT methods (Louisiana soil)

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<th>Length (ft)</th>
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<th>7</th>
<th>Prince &amp; Turmanj &amp; Almeida</th>
<th>Espali &amp; Chow</th>
<th>UWA-05</th>
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### Table 4-10. LRFD resistance factors, $\Phi$, for CPT methods (ultimate skin friction, Louisiana soil)

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<td>0.476</td>
<td>0.593</td>
</tr>
<tr>
<td>Almeida et al. (1996)</td>
<td>0.734</td>
<td>0.282</td>
<td>0.436</td>
<td>0.594</td>
</tr>
<tr>
<td>Eslami &amp; Fellenius (1997)</td>
<td>0.713</td>
<td>0.259</td>
<td>0.448</td>
<td>0.628</td>
</tr>
<tr>
<td>Jardine &amp; Chow (MTD) (1996)</td>
<td>1.145</td>
<td>0.240</td>
<td>0.752</td>
<td>0.657</td>
</tr>
<tr>
<td>Powell et al. (2001)</td>
<td>0.684</td>
<td>0.275</td>
<td>0.413</td>
<td>0.604</td>
</tr>
<tr>
<td>UWA-05 (2005)</td>
<td>2.983</td>
<td>0.306</td>
<td>1.667</td>
<td>0.559</td>
</tr>
<tr>
<td>Zhou et al. (1982)</td>
<td>0.614</td>
<td>0.290</td>
<td>0.357</td>
<td>0.582</td>
</tr>
<tr>
<td><strong>Proposed Method (2006)</strong></td>
<td><strong>1.040</strong></td>
<td><strong>0.248</strong></td>
<td><strong>0.670</strong></td>
<td><strong>0.644</strong></td>
</tr>
</tbody>
</table>

### Table 4-11. LRFD resistance factors, $\Phi$, for CPT methods (ultimate tip resistance, Louisiana soil)

<table>
<thead>
<tr>
<th>Analysis method</th>
<th>$\lambda_R$</th>
<th>COV $r$</th>
<th>$\Phi$</th>
<th>$\Phi/\lambda_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmertmann (1978)</td>
<td>1.073</td>
<td>0.678</td>
<td>0.241</td>
<td>0.224</td>
</tr>
<tr>
<td>de Ruiter and Beringen (1979)</td>
<td>1.869</td>
<td>1.000</td>
<td>0.209</td>
<td>0.112</td>
</tr>
<tr>
<td>Bustamante and Gianeselli (LCPC) (1982)</td>
<td>1.342</td>
<td>0.728</td>
<td>0.269</td>
<td>0.200</td>
</tr>
<tr>
<td>Aoki &amp; de Alencar (1975)</td>
<td>1.133</td>
<td>0.910</td>
<td>0.152</td>
<td>0.135</td>
</tr>
<tr>
<td>Penpile Method (Clisby et al. 1978)</td>
<td>4.024</td>
<td>0.727</td>
<td>0.807</td>
<td>0.201</td>
</tr>
<tr>
<td>Philipponnat (1980)</td>
<td>1.519</td>
<td>0.856</td>
<td>0.229</td>
<td>0.151</td>
</tr>
<tr>
<td>Prince and Wardle (1982)</td>
<td>1.864</td>
<td>0.937</td>
<td>0.237</td>
<td>0.127</td>
</tr>
<tr>
<td>Tumay and Fakhroo (1982)</td>
<td>0.876</td>
<td>0.844</td>
<td>0.136</td>
<td>0.155</td>
</tr>
<tr>
<td>Almeida et al. (1996)</td>
<td>2.721</td>
<td>1.163</td>
<td>0.224</td>
<td>0.082</td>
</tr>
<tr>
<td>Eslami &amp; Fellenius (1997)</td>
<td>1.168</td>
<td>0.913</td>
<td>0.156</td>
<td>0.134</td>
</tr>
<tr>
<td>Jardine &amp; Chow (MTD) (1996)</td>
<td>0.941</td>
<td>0.590</td>
<td>0.261</td>
<td>0.277</td>
</tr>
<tr>
<td>Powell et al. (2001)</td>
<td>1.710</td>
<td>1.176</td>
<td>0.137</td>
<td>0.080</td>
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<tr>
<td>UWA-05 (2005)</td>
<td>1.794</td>
<td>0.678</td>
<td>0.403</td>
<td>0.225</td>
</tr>
<tr>
<td>Zhou et al. (1982)</td>
<td>0.889</td>
<td>0.578</td>
<td>0.253</td>
<td>0.285</td>
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<tr>
<td><strong>Proposed Method (2006)</strong></td>
<td><strong>1.107</strong></td>
<td><strong>0.447</strong></td>
<td><strong>0.435</strong></td>
<td><strong>0.393</strong></td>
</tr>
</tbody>
</table>
Table 4-12. LRFD resistance factors, $\Phi$, for CPT methods (ultimate pile capacity, Louisiana soil)

<table>
<thead>
<tr>
<th>Analysis method</th>
<th>$\lambda_R$</th>
<th>COV $R$</th>
<th>$\Phi$</th>
<th>$\Phi/\lambda_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmertmann (1978)</td>
<td>0.838</td>
<td>0.320</td>
<td>0.452</td>
<td>0.539</td>
</tr>
<tr>
<td>de Ruiter and Beringen (1979)</td>
<td>0.613</td>
<td>0.257</td>
<td>0.387</td>
<td>0.631</td>
</tr>
<tr>
<td>Bustamante and Gianselli (LCPC) (1982)</td>
<td>0.776</td>
<td>0.250</td>
<td>0.498</td>
<td>0.641</td>
</tr>
<tr>
<td>Aoki &amp; de Alencar (1975)</td>
<td>1.417</td>
<td>0.280</td>
<td>0.845</td>
<td>0.596</td>
</tr>
<tr>
<td>Penpile Method (Clisby et al. 1978)</td>
<td>1.740</td>
<td>0.312</td>
<td>0.958</td>
<td>0.551</td>
</tr>
<tr>
<td>Philipponnat (1980)</td>
<td>0.985</td>
<td>0.231</td>
<td>0.663</td>
<td>0.673</td>
</tr>
<tr>
<td>Prince and Wardle (1982)</td>
<td>1.376</td>
<td>0.372</td>
<td>0.652</td>
<td>0.474</td>
</tr>
<tr>
<td>Tumay and Fakhroo (1982)</td>
<td>0.698</td>
<td>0.263</td>
<td>0.434</td>
<td>0.622</td>
</tr>
<tr>
<td>Almeida et al. (1996)</td>
<td>0.766</td>
<td>0.266</td>
<td>0.473</td>
<td>0.618</td>
</tr>
<tr>
<td>Eslami &amp; Fellenius (1997)</td>
<td>0.676</td>
<td>0.246</td>
<td>0.438</td>
<td>0.648</td>
</tr>
<tr>
<td>Jardine &amp; Chow (MTD) (1996)</td>
<td>0.971</td>
<td>0.222</td>
<td>0.668</td>
<td>0.687</td>
</tr>
<tr>
<td>Powell et al. (2001)</td>
<td>0.678</td>
<td>0.260</td>
<td>0.424</td>
<td>0.626</td>
</tr>
<tr>
<td>UWA-05 (2005)</td>
<td>2.317</td>
<td>0.276</td>
<td>1.396</td>
<td>0.603</td>
</tr>
<tr>
<td>Zhou et al. (1982)</td>
<td>0.597</td>
<td>0.286</td>
<td>0.351</td>
<td>0.587</td>
</tr>
<tr>
<td><strong>Proposed Method (2006)</strong></td>
<td><strong>0.964</strong></td>
<td><strong>0.230</strong></td>
<td><strong>0.649</strong></td>
<td><strong>0.673</strong></td>
</tr>
</tbody>
</table>
Figure 4-1. CPT and DTP test data from Archer Landfill site
Figure 4-2. Friction ratio of CPT and DTP rest from Archer Landfill site
Figure 4-3. CPT and DTP test data from I–95 at Edgewood Avenue site

- Slightly Silty Fine Sand to Fine Sand
- Clay
- Dense Silty Fine Sand
Figure 4-4. Friction ratio of CPT and DTP test from I–95 at Edgewood Avenue site
Figure 4-5. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test from I-95 at Edgewood Avenue site
Figure 4-6. CPT and DTP test data from Apalachicola River Bridge site
Figure 4-7. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test from Apalachicola River Bridge site
Figure 4-8. CPT and DTP test data from West Bay Bridge site
Figure 4-9. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test from West Bay Bridge site
Figure 4-10. CPT and DTP test data from Port Orange Relief Bridge site
Figure 4-11. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test from Port Orange Relief Bridge site
Figure 4-12. CPT and DTP test data at the University of Central Florida site
Figure 4-13. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test at the University of Central Florida site.
Figure 4-14. CPT and DTP test data at I-295 at Blanding Blvd site
Figure 4-15. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test at I-295 at Blanding Blvd site
Figure 4-16. CPT and DTP test data at I-295 at Normandy Blvd. site
Figure 4-17. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test at I-295 at Normandy Blvd. site
Figure 4-18. CPT and DTP test data in White City site (pier 5)
Figure 4-19. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test in White City site (pier 5)
Figure 4-20. CPT and DTP test data in White City site (pier 8)
Figure 4-21. Tip2/Tip1 ratio and Qc/N ratio of CPT and DTP test in White City site (pier 8)
Figure 4-22. Typical load test curve in Louisiana soil

Q Davisson

Bayou_Boeuf_East_Approach_F4

Load (tons)

Displacement (in)
Figure 4-23. Comparisons (ratio: measured/predicted) for 14 methods using Florida soil. A) Y-scale from 0 to 4. B) Y-scale from 0 to 2.
Figure 4-24. Comparisons (ratio: measured/predicted) for 14 methods using Louisiana soil. A) Y-scale from 0 to 4. B) Y-scale from 0 to 2.
Figure 4-25. Frequency of $\lambda$ of the proposed method for Florida soil

Figure 4-26. Frequency of the sample means using Bootstrap method for Florida soil (100,000 re-sampling runs, the proposed method)
Figure 4-27. Frequency of the sample standard deviations using Bootstrap method for Florida soil (100,000 re-sampling runs, the proposed method)

Mean = 0.278
Stdev = 0.042

Figure 4-28. Frequency of $\lambda$ of the proposed method for Louisiana soil

Mean = 0.963
Stdev = 0.224
Figure 4-29. Frequency of the sample means using Bootstrap method for Louisiana soil (100,000 re-sampling runs, the proposed method)

Figure 4-30. Frequency of the sample standard deviations using Bootstrap method for Louisiana soil (100,000 re-sampling runs, the proposed method)
Figure 4-31. Frequency of $\lambda$ of the Schmertmann method for Florida soil

Figure 4-32. Frequency of the sample means using Bootstrap method for Florida soil (100,000 re-sampling runs, Schmertmann method)
Mean = 0.631
Stdev = 0.243

Figure 4-33. Frequency of the sample standard deviations using Bootstrap method for Florida soil (100,000 re-sampling runs, Schmertmann method)
CHAPTER 5
CONCLUSION

Conclusion

A total of 21 cases (load test data with CPT close to it) in Florida and 28 cases in Louisiana have been used to assess LRFD resistance factor, $\Phi$, for 14 pile axial capacity prediction methods based on CPT results. One of the best methods, the Philipponnat method, was chosen and modified to form the proposed UF method. Ten field sites were revisited with paired CPT and DTP tests performed. This was an attempt to develop identifiers for cemented sand, which is the one of the most challenging soil types for geotechnical engineers.

Based on the analysis of the test data and resistance factors of each method, the following conclusions can be drawn:

- Current pile axial capacity prediction methods using CPT results overestimate pile capacities in cemented soil. This issue needs to be addressed with additional testing.

- The proposed procedure for separating skin friction and tip resistance is a useful tool to better evaluate the pile capacity prediction methods.

- Of the fourteen pile axial capacity prediction methods analyzed, the Philipponnat method, LCPC method and MTD method were shown to have the highest $\Phi/\lambda$ value, indicating more accurate results.

- The proposed UF method has the highest $\Phi/\lambda$ for Florida soil and second highest for Louisiana soil (0.62 for Florida soil and 0.67 for Louisiana soil) among all the methods. In addition, it has the lowest coefficient of variation for Florida soil and second lowest for Louisiana soil (0.27 for Florida soil and 0.23 for Louisiana soil). Therefore, the proposed UF method works very well for both sands (Florida soil) and clays (Louisiana soil). (Hu et al. 2007a)

- The dual tip penetrometer can be a potential tool to identify cemented soil by using the identifier, $T_2/T_1$. The ranges of $T_2/T_1$ for different soil type are shown below, based on limited test data (10 cases) and more data need to be collected to confirm these figures. (Hu et al. 2007b)

  \[
  \begin{align*}
  \approx 0.5 & \quad \text{Non-Cemented Sand (Loose and Medium Dense Sand)} \\
  = 0.3\sim0.5 & \quad \text{Lightly Cemented Sand} \\
  \approx 1 & \quad \text{Strongly Cemented Sand}
  \end{align*}
  \]
Using the bootstrap method on field data shows that the standard deviation of the resampled mean and resampled standard deviation are quite small for the proposed method. This means that the proposed method provides higher quality predictions.

**Future Work**

There still needs work to confirm the above conclusions. For example:

- Since the sleeve friction reading of DTP is not identical to CPT and it hinders the ability of DTP to identify soil type, a redesign of DTP is recommended to eliminate the influence of the second tip to the sleeve friction.

- More bridge sites should be revisited and tested with both the CPT and DTP. Table 5-1 shows the bridge sites where load test data are available.

- Validate the cemented sand identification values from the DTP (T1/T2), and compare them with SPT & CPT data.

- Evaluate the proposed pile capacity method based on a larger CPT, DTP, and load test database. This will provide confidence in recommending a realistic LRFD resistance factor, $\Phi$, and in turn further the stage of geotechnical engineering practice.
### Table 5-1. Bridge sites where load test data are available

#### Cemented Sand Site

<table>
<thead>
<tr>
<th>Load Test No.</th>
<th>Site Name</th>
<th>Project Number</th>
<th>Pier (Bent/Slab) No.</th>
<th>Station</th>
<th>Pile Embedded Length (ft)</th>
<th>Desired CPT Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>T-1</td>
<td>N1533290</td>
<td>E631461</td>
<td>46</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>T-6</td>
<td>N1533447</td>
<td>E631541</td>
<td>67</td>
<td>72</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>T-7</td>
<td>N153325</td>
<td>E631568</td>
<td>46.2</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>T-14</td>
<td>N1533572</td>
<td>E631617</td>
<td>36</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>TP-2</td>
<td>N/A</td>
<td>33</td>
<td>38</td>
<td></td>
<td></td>
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<tr>
<td>6</td>
<td>TP-10</td>
<td>N/A</td>
<td>63</td>
<td>68</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>3B</td>
<td>N/A</td>
<td>46</td>
<td>53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>3C</td>
<td>N/A</td>
<td>60</td>
<td>65</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>B-20</td>
<td>N2909 W6492</td>
<td>64</td>
<td>70</td>
<td></td>
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</tr>
<tr>
<td>10</td>
<td>B-4 (Pier-4)</td>
<td>264+27.23</td>
<td>52.6</td>
<td>59</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>B-9 (Pier-9)</td>
<td>271+25.67</td>
<td>45.8</td>
<td>52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>C-2 (Pier-2)</td>
<td>1981+87.41</td>
<td>43</td>
<td>49</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Pier #2</td>
<td>N/A</td>
<td>47.3</td>
<td>54</td>
<td></td>
<td></td>
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<tr>
<td>14</td>
<td>Pier #3</td>
<td>N/A</td>
<td>103.6</td>
<td>114</td>
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<tr>
<td>15</td>
<td>Pier #4</td>
<td>N/A</td>
<td>49.2</td>
<td>58</td>
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<td></td>
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</table>

#### Non-Cemented Sand Site

<table>
<thead>
<tr>
<th>Load Test No.</th>
<th>Site Name</th>
<th>Project Number</th>
<th>Pier (Bent/Slab) No.</th>
<th>Station</th>
<th>Pile Embedded Length (ft)</th>
<th>Desired CPT Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TP-1</td>
<td>240+10</td>
<td>41.2</td>
<td>50</td>
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<tr>
<td>2</td>
<td>TP-38</td>
<td>378+29</td>
<td>23.6</td>
<td>32</td>
<td></td>
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<td>3</td>
<td>Pile #2</td>
<td>N/A</td>
<td>64</td>
<td>68</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Pile #3</td>
<td>N/A</td>
<td>49.2</td>
<td>58</td>
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<tr>
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<td>Pile #4</td>
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<td>30</td>
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<tr>
<td>6</td>
<td>N/A</td>
<td>N/A</td>
<td>47.3</td>
<td>54</td>
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<td></td>
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<tr>
<td>7</td>
<td>N/A</td>
<td>N/A</td>
<td>103.6</td>
<td>114</td>
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<tr>
<td>8</td>
<td>Pier 1R</td>
<td>216+14.25</td>
<td>85.7</td>
<td>94</td>
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<tr>
<td>9</td>
<td>Bent 2R</td>
<td>N/A</td>
<td>61.3</td>
<td>70</td>
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<td>10</td>
<td>Pier 1</td>
<td>494+00</td>
<td>53.4</td>
<td>64</td>
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<td>11</td>
<td>Pier 2E</td>
<td>244+05</td>
<td>89.3</td>
<td>100</td>
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<td>45+00</td>
<td>61</td>
<td>67</td>
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</tr>
<tr>
<td>15</td>
<td>N/A</td>
<td>N/A</td>
<td>52.3</td>
<td>61</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX
MATHCAD PROGRAM

Pile Capacity Prediction Methods

Schmertmann Method (1978)

Schmertmann Method (1978)

exchange = 0.16/0.42

Ultimate Pile Capacity Based on CPT data Schmertmann Method (1978)

O = 1

Input Data:

\[ CPT = \text{C:\Users\CPT xls} \]

\[ D = 15 \text{ in} \] Diameter or Edge Length of the Pile

\[ L = 45 \text{ ft} \] Embedment of the Pile

Average \( q_{0} \) over a distance of \( yD \) below the pile tip: \( \text{ave}_{\text{below}}(CPT, D, L, y) \)

\[
q_{\text{ave}_{\text{below}}}(CPT, D, L, y) = \frac{n \times \text{exchange}}{L}
\]

\[
i = 0
\]

\[
q_{\text{ave}_{\text{below}}} = 0
\]

\[
\text{while } CPT_{i+1} < \frac{L + yD}{n}
\]

\[
q_{\text{below}} = q_{\text{below}} + CPT_{i+1}
\]

\[
i = i + 1
\]

\[
\text{counter} = i
\]

\[
q_{\text{below}} = 0
\]

\[
q_{\text{below}_{\text{min}}} = CPT_{i+1}
\]

\[
\text{while } i > 0
\]

\[
\text{if } q_{\text{below}_{\text{min}}} < CPT_{i+1}
\]

\[
q_{\text{below}} = q_{\text{below}} + q_{\text{below}_{\text{min}}}
\]

\[
i = i + 1
\]

\[
\text{otherwise}
\]

\[
q_{\text{below}_{\text{min}}} = CPT_{i+1}
\]

\[
q_{\text{below}} = q_{\text{below}} + q_{\text{below}_{\text{min}}}
\]

\[
i = i - 1
\]

\[
q_{\text{ave}_{\text{below}}} = \frac{q_{\text{below}} + q_{\text{below}}}{2(\text{counter} + 1)}
\]

return \( q_{\text{ave}_{\text{below}}} \)

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.18</td>
<td>23.09</td>
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<td>16</td>
<td>2.48</td>
<td>107.29</td>
<td>0.738</td>
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</table>
\[
q_c(L) \triangleq y \leftarrow 0
\]
\[
\text{for } y \in 0.7, 0.71, \ldots
\]
\[
\begin{align*}
q_c & \leftarrow q_{\text{case below}}(CPT, D, L, y) \\
q_c & \leftarrow q_{\text{case below}}(CPT, D, L, y)
\end{align*}
\]
\[
y \leftarrow y
\]
\[
\text{return } \begin{pmatrix} q_c \\ y \end{pmatrix}
\]
\[
q_c(L) = \begin{pmatrix} 121.258 \\ 3.75 \end{pmatrix}
\]

\[
q_c(L) :=
\]
\[
\begin{align*}
&\text{return } "L \text{ is too short}" \text{ if } L \leq 8D \\
&n \leftarrow \text{floor} \left( \frac{L}{\text{exchange}} \right) \\
&cn \leftarrow q_c \left( \frac{L}{\text{exchange}} \right)
\end{align*}
\]
\[
\begin{align*}
q_c & \leftarrow y \leftarrow 0 \\
q_c & \leftarrow q_{\text{case below}}(CPT, D, L, 0.7) \\
\text{for } y \in 0.7, 0.71, \ldots
\end{align*}
\]
\[
\begin{align*}
&\begin{align*}
&\text{if } q_c \geq q_{\text{case below}}(CPT, D, L, y) \\
&q_c & \leftarrow q_{\text{case below}}(CPT, D, L, y)
\end{align*}
\end{align*}
\]
\[
y \leftarrow y
\]
\[
q_{\text{below min}} \leftarrow i \leftarrow 0
\]
\[
\text{while } CPT_{\text{low}, i+1} \leq \frac{L + q_c D}{\text{exit}}
\]
\[
\begin{align*}
&b_{i+1} \leftarrow CPT_{\text{low}, i+1} \\
i & \leftarrow i + 1
\end{align*}
\]
mix(b)
\[
q_{\text{above min}} \leftarrow q_{\text{below min}}
\]
\[
q_{\text{above}} \leftarrow 0
\]
\[
i \leftarrow 0
\]
\[
\text{while } CPT_{\text{high}, i+1} \geq \frac{L - 8D}{\text{exit}}
\]
\[
\begin{align*}
&\text{if } q_{\text{above min}} \leq CPT_{\text{high}, i+2} \\
&q_{\text{above}} & \leftarrow q_{\text{above}} + q_{\text{above min}} \\
i & \leftarrow i + 1
\end{align*}
\]
\[
\text{otherwise}
\]
\[
\begin{align*}
&q_{\text{above min}} \leftarrow CPT_{\text{high}, i+2} \\
&q_{\text{above}} & \leftarrow q_{\text{above}} + q_{\text{above min}} \\
i & \leftarrow i + 1
\end{align*}
\]
\[
q_{\text{case above}} \leftarrow \frac{q_{\text{above}}}{i}
\]
\[
\text{return } q_{\text{case above}}
\]

\[
q_c(L) = 67.393
\]
Calculation of the average cone tip resistance in Schunertmann method:

\[ q_{c1} = 121.28 \]
\[ q_{c2} = 67.393 \]
\[ q_{c1} = \frac{q_{c1} \sqrt{q_{c1} + q_{c2}}}{2} \leq 150, \quad \frac{q_{c1} + q_{c2}}{2} \leq 150 \]
\[ q_{c1} = 54.575 \text{ kN/m}^2 \]

Tip resistance:
\[ A = D^2 \quad A = 2.25 \pi r^2 \]
\[ Q_{c1} = A \cdot q_{c1} \quad Q_{c1} = 212.785 \text{ kN} \]

Side Friction (Schunertmann method):

Calculate \( a_e \) for Concrete & Timber Piles:

\[ n = \frac{L}{D} \quad n = 296 \]
\[ x_0 = 0.2549 \left( \frac{D}{G} \right)^{5/2} - 1.1770 \left( \frac{D}{G} \right)^{7/2} + 2.1379 \left( \frac{G}{D} \right)^{7/2} - 1.3322 \left( \frac{G}{D} \right)^{3/2} - 0.7543 \left( \frac{G}{D} \right)^{3/2} + 1.25 \]
\[ x_0 = 0.014 \]

Calculate \( a_e \) for concrete pile:

\[ a_e(L) = \left[ \frac{L}{D} < 35, -9 \times 10^{-5} \left( \frac{L}{D} \right)^2 + 0.0061 \left( \frac{L}{D} \right) - 0.15 \frac{L}{D} + 2.1320, 0.83 \right] \]

\[ a_e(L) = 0.83 \]

Calculate Ultimate Side Friction: \( Q_s \)

\[ Q_s(L1, L2, SeuType) = \begin{cases} \text{if } L1 + i \times \text{exchange} \leq L2 \\ Q_s &= 0 \text{ kN} \\ A_q &= 4 \times D \times \text{exchange} \\ i &= 0 \end{cases} \\
\text{if } \text{SeuType} = 2 \\
\ y &= CFT_{b+1,1} \cdot f \\
\ f &= \text{if } \left( f \leq \frac{D}{G}, a_e(L1) \cdot \frac{L}{G} \cdot CPT_{b+1,3} \cdot a_c(L1) \cdot CPT_{b+1,3} \right) \frac{\tan}{\pi^2} \\
\ f &= \text{if } \left( f \leq 1.2 \frac{\tan}{\pi^2} \cdot f, 1.2 \frac{\tan}{\pi^2} \right) \\
\ Q_s &= Q_s + f \cdot A_q \\
\ i &= i + 1 \\
\text{otherwise} \\
\ f &= \text{if } \left( a_c \right) \left( \frac{CPT_{b+1,3}}{CPT_{b+1,3} \leq 1.2, a_c \left( \frac{CPT_{b+1,3}}{CPT_{b+1,3} \leq 1.2, a_c} \right) \frac{\tan}{\pi^2}} \\
\ Q_s &= Q_s + f \cdot A_q \\
\ i &= i + 1 \\
\text{return } Q_s 
\]
De Ruiter and Beringen Method (1979)

exchange = 0.1640428

Ultimate Pile Capacity Based on CPT data de Ruiter and Beringen Method (1979)

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<td>2.46</td>
<td>107.53</td>
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<td>0.688</td>
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</table>

CPT = DATA input
C:\\ACPT.xls

D = 15 in
Length of the Pile

Average qc over a distance of yD below the pile tip: qc ave_below(CPT,D,L,y)
\text{qave\_below}(\text{CPT}, D, L, y) := \begin{align*}
n &\left\lceil \frac{L}{\text{exchange}} \right\rceil \\
&i \leftarrow 0 \\
&\text{qbelow\_1} \leftarrow 0 \\
\text{while } &\text{CPT}_{n+i,1} \leq \frac{L + y \cdot D}{ft} \\
&\text{qbelow\_1} \leftarrow \text{qbelow\_1} + \text{CPT}_{n+i,2} \\
&i \leftarrow i + 1 \\
&i \leftarrow i - 1 \\
&\text{counter} \leftarrow i \\
&\text{qbelow\_2} \leftarrow 0 \\
&\text{qbelow\_min} \leftarrow \text{CPT}_{n+i,2} \\
\text{while } &i \geq 0 \\
&\text{if } \text{qbelow\_min} \leq \text{CPT}_{n+i,2} \\
&\text{qbelow\_2} \leftarrow \text{qbelow\_2} + \text{qbelow\_min} \\
&i \leftarrow i - 1 \\
&\text{otherwise} \\
&\text{qbelow\_min} \leftarrow \text{CPT}_{n+i,2} \\
&\text{qbelow\_2} \leftarrow \text{qbelow\_2} + \text{qbelow\_min} \\
&i \leftarrow i - 1 \\
\text{qave\_below} &\leftarrow \frac{\text{qbelow\_1} + \text{qbelow\_2}}{2(\text{counter} + 1)} \\
\text{return } &\text{qave\_below} \\
\end{align*}

\text{qcl}(D) := \begin{align*}
&yy \leftarrow 0.7 \\
&qcl \leftarrow \text{qave\_below}(\text{CPT}, D, L, 0.7) \\
&\text{for } y \in 0.7, 0.71 \ldots 4 \\
&\text{if } qcl \geq \text{qave\_below}(\text{CPT}, D, L, y) \\
&qcl \leftarrow \text{qave\_below}(\text{CPT}, D, L, y) \\
&yy \leftarrow y \\
\text{return } \begin{pmatrix} qcl \\ yy \end{pmatrix} \\
\end{align*}
qc2(L) :=

return "L is too short" if L ≤ 8D

n ← floor\left( \frac{L}{\text{exchange}} \right)

nn ← ceil\left( \frac{L}{\text{exchange}} \right)

qcl ← \ yy ← 0.7

\quad qcl ← qcave_below(CPT,D,L,0.7)

\quad \text{for } y ∈ 0.7,0.71,\ldots,4

\quad \text{if } qcl ≥ qcave_below(CPT,D,L,y)

\quad \quad qcl ← qcave_below(CPT,D,L,y)

\quad yy ← y

qbelow_min ← i ← 0

\quad \text{while } CPT_{mn+i,1} ≤ \frac{L + qcl \cdot D}{\text{ft}}

\quad \quad b_{i+1} ← CPT_{mn+i,2}

\quad \quad i ← i + 1

\quad \min(b)

qabove_min ← qbelow_min

qabove ← 0

i ← 0

\text{while } CPT_{n-i,1} ≥ \frac{L - 8 \cdot D}{\text{ft}}

\quad \text{if } qabove_min ≤ CPT_{n-i,2}

\quad \quad qabove ← qabove + qabove_min

\quad \quad i ← i + 1

\quad \text{otherwise}

\quad \quad qabove_min ← CPT_{n-i,2}

\quad \quad qabove ← qabove + qabove_min

\quad \quad i ← i + 1

qcave_above ← \frac{qabove}{i}

return qcave_above

qc2(L) = 67.893
Calculation of the average cone tip resistance in Schmertmann method:

\[ q_{c1}(L) = 121.238 \]
\[ q_{c2}(L) = 67.993 \]

**SoilType = 2** 1 for clay and 2 for sand

\[ N_c = 9 \]

\[ N_k = 20 \]

\[ q_c(L) = \frac{q_{c1}(L) + q_{c2}(L)}{2} \]

\[ q_c(L) = \frac{1}{\pi^2} \tan^{-1} \left( \frac{q_c(L)}{q_{c1}(L)} \right) \]

\[ q_c(L) = 94.775 \frac{1}{\pi^2} \text{ton} \]

Tip resistance:

\[ A = D^2 \quad A = 2.29 \pi^2 \]

\[ Q_c(L) = A \cdot q_c(L) \quad Q_c(L) = 212.795 \text{ton} \]

Side Friction (de Ruiter and Boringan method):

Calculate Ultimate Side Friction \( Q_s \)

**FileType := 1** 1 for compression pile, 2 for tension pile

\[ Q_s(L, L_2, SoilType, beta) := \]

\[ n \leftarrow \text{cal} \left( \frac{L_2}{\text{exchange}} \right) \]

\[ m \leftarrow \text{from} \left( \frac{L_1}{\text{exchange}} \right) \]

\[ A_L \leftarrow 4 \cdot D \cdot (L_2 - L_1) \]

\[ q_{sum} \leftarrow 0 \]

**for** \( j \in n, m \)

\[ q_{sum} \leftarrow q_{sum} + CPT_{j,2} \]

\[ q_{axis} \leftarrow \frac{q_{sum}}{m-n+1} \]

\[ f_s \leftarrow \frac{\beta \cdot q_{axis} \leq 1.2}{N_k} \]

\[ \text{if} \ \text{SoilType} \neq 1 \]

\[ f_s \leftarrow \tilde{q}_{sum,1,2,2} \]

\[ Q_s \leftarrow \frac{f_s \cdot A_L}{\pi^2} \]

**return** \( Q_s \)
Penpile Method (1980)

$\text{exchange} := 0.164042\text{ft}$

Ultimate Pile Capacity Based on CPT data Penpile Method (Clisby et al. 1978)

$\text{ORIGIN} = 1$

Input Data:

$\text{CPT} := \text{DATA input}$

$D := 18\text{in}$

$L := 48.5\text{ft}$

Soil Type: 1 for cohesive soil, 2 for cohesionless soil

$L_1 = 4\text{ft}$  $L_2 = 5.33\text{ft}$  $L_3 = 27.9\text{ft}$  $L_4 = 36.5\text{ft}$  $L_5 = 1$

Layer 1:

Soil Type = 2

$Q_1 = Q_0(0.01A, L_1, \text{Soil Type}, 1) = 5732\text{ton}$

Layer 2:

Soil Type = 1

$Q_2 = Q_0(L_1, L_2, \text{Soil Type}, 1) = 12.96\text{ton}$

Layer 3:

Soil Type = 2

$Q_3 = Q_0(L_2, L_3, \text{Soil Type}, 1) = 30.996\text{ton}$

Layer 4:

Soil Type = 1

$Q_4 = Q_0(L_3, L_4, \text{Soil Type}, 1) = 13.841\text{ton}$

Layer 5:

Soil Type = 2

$Q_5 = Q_0(L_4, L_5, \text{Soil Type}, 1) = 20.062\text{ton}$

$Q_s = Q_1 + Q_2 + Q_3 + Q_4 + Q_5$

$Q_s = 83.591\text{ton}$

$Q_{\text{Devisson}} = Q_s + \frac{Q_0(L_5)}{3}$

$Q_{\text{Devisson}} = 134.523\text{ton}$
Calculation of the average cone tip resistance using Penfield Method (Clarby et al. 1978)

the average of the three tip resistance close to the tip

\[
q_{\text{ave tip}}(D, L) = \begin{cases} 
\alpha = \text{real} \left( \frac{L}{\text{exchange}} \right) \\
q_{\text{ave}} \leftarrow \frac{CPT_{n-1,2} + CPT_{n-2,2} + CPT_{n+1,2}}{3} \quad \text{if } \alpha \leq 0.5 \\
q_{\text{ave}} \leftarrow \frac{CPT_{n-2,2} + CPT_{n-1,2} + CPT_{n,2}}{3} \quad \text{otherwise}
\end{cases}
\]

return \( q_{\text{ave}} \)

\[
q_{\text{ave tip}}(D, L) = 143.153
\]

\text{SoilType} := 2 \quad 1 \text{ for clay, 2 for sand}

\[
\lambda_0 = \phi(\text{SoilType} = 1, 0.23, 0.125)
\]

\[
q(T) = q_{\text{ave tip}}(D, L) \cdot \lambda_0 \cdot \frac{\tan \theta}{n^2}
\]

Tip resistance:

\[
A := D^2 \quad A = 2.25B^2
\]

\[
Q(T) = A \cdot q(T) \quad Q(T) = 40.362 \text{ton}
\]

Side Friction Penfield Method (Clarby et al. 1978):

\[
L1 := 0.16 \quad \text{Input the soil layer}
\]

\[
L2 := 0.6
\]

\[
f_{\text{ave side}}(L1, L2) = \begin{cases} 
\alpha = \text{real} \left( \frac{L1}{\text{exchange}} \right) \\
i \leftarrow 0 \\
k \leftarrow 0 \\
f_s \leftarrow 0 \\
\text{while } CPT_{n+i, 1} \leq \frac{L2}{n} \\
f_s \leftarrow f_s + CPT_{n+i, 3} \\
i \leftarrow i + 1 \\
f_{\text{ave}} \leftarrow \frac{f_s}{i} 
\end{cases}
\]

return \( f_{\text{ave}} \)
\[ f_{ave_{side}}(L_1, L_2) = 0.406 \]

\[
f(L_1, L_2) = \frac{f_{ave_{side}}(L_1, L_2)}{1.5 + 0.1 \cdot \frac{f_{ave_{side}}(L_1, L_2)}{0.072}} \frac{\text{ton}}{\text{ft}^2} \]

\[ f(L_1, L_2) = 0.197 \frac{\text{ton}}{\text{ft}^2} \]

Calculate Ultimate Side Friction: \( Q_s \)

\[ Q_s(L_1, L_2) = f(L_1, L_2) \cdot 4 \cdot D \cdot (L_2 - L_1) \]

\[ Q_s(L_1, L_2) = 42.402 \text{ ton} \]

\begin{tabular}{cccccc}
L_1 &=& 4\text{ft} & L_2 &=& 5.8\text{ft} & L_3 &=& 27.9\text{ft} & L_4 &=& 36.5\text{ft} & L_5 &=& L \\
\end{tabular}

Layer 1:

\[ Q_{s1} := Q_s(0.01\text{ft}, L_1) \]

\[ Q_{s1} = 3.976 \text{ ton} \]

Layer 2:

\[ Q_{s2} := Q_s(L_1, L_2) \]

\[ Q_{s2} = 3.837 \text{ ton} \]

Layer 3:

\[ Q_{s3} := Q_s(L_2, L_3) \]

\[ Q_{s3} = 26.877 \text{ ton} \]

Layer 4:

\[ Q_{s4} := Q_s(L_3, L_4) \]

\[ Q_{s4} = 3.895 \text{ ton} \]

Layer 5:

\[ Q_{s5} := Q_s(L_4, L_5) \]

\[ Q_{s5} = 14.669 \text{ ton} \]

\[ Q_s := Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5} \]

\[ Q_s = 55.074 \text{ ton} \]

\[ Q_{Davisson} = Q_s + \frac{Q_t(L)}{3} \]

\[ Q_{Davisson} = 68.495 \text{ ton} \]
Prince and Wardle Method (1982)

\[
\text{exchange} := 0.1640/\text{ft}
\]

Ultimate Pile Capacity Based on CPT data Prince and Wardle (1982)

\[
\text{ORIGIN} = 1 \\
\text{For Clayey Soil Only}
\]

Input Data:

\[
\text{CPT} = \text{DATA input} \\
\text{D} = 18\text{in} \\
\text{L} = 48.5\text{ft}
\]

Diameter or Edge Length of the Pile

Length of the Pile

Calculation of the average cone tip resistance using Prince and Wardle (1982):

\[
q_{\text{ave, side}}(L_1,L_2) = \left[ n = \text{ceil} \left( \frac{L_1}{\text{exchange}} \right) \\
i = 0 \\
i = i + 1 \\
q_v = 0 \\
\text{while } CPT_{x+i,1} \leq \frac{L_2}{\text{ft}} \\
one \leftarrow q_v + CPT_{x+i,2} \\
i \leftarrow i + 1 \\
q_{\text{ave},v} = \frac{q_v}{i} \\
\text{return } q_{\text{ave},v}
\]

\[
q_{\text{ave, tip}}(D,L) = \frac{q_{\text{ave, side}}(L-SD,D)+q_{\text{ave, side}}(L+4D,L)}{2} \quad \text{Influence zone is } SD \text{ above and } 4D \text{ below the pile tip}
\]

\[
q_{\text{ave, tip}}(D,L) = 112.72\text{t}
\]

\[k_b = 0.35 \quad 0.35 \text{ for driven pile, } 0.3 \text{ for jacked pile}
\]

\[
q_t(L) = 0.35 \left[ q_{\text{ave, tip}}(D,L) \right]_{k_b \leq 0.5, q_{\text{ave, tip}}(D,L) \geq 0.15} \cdot \frac{\tan \theta}{n^2} \\
q_t(L) = 39.452 \cdot \frac{1}{n^2} \text{ ton}
\]

Tip resistance:

\[A = D^2 \quad A = 2.25\text{ft}^2 \]

\[Q_t(L) = A \cdot q_t(L) \quad Q_t(L) = 88.768 \text{ ton}\]
Side Friction (Prince and Wardle (1982)):

\[
L_1 = 3.8\text{ft} \\
L_2 = 11.2\text{ft}
\]

\[
f_{\text{ave, side}}(L_1, L_2) =:\begin{align*}
n &\leftarrow \text{ceil} \left( \frac{L_1}{0.463} \right) \\
j &\leftarrow 0 \\
k &\leftarrow 0 \\
f_i &\leftarrow 0
\end{align*}
\]

\[
\text{while } CPT_{i+1,1} \leq \frac{L_2}{3} \\
\{ \\
fy &\leftarrow fy + CPT_{i+1,3} \\
j &\leftarrow j + 1 \\
f_{\text{ave, side}} &\leftarrow \frac{fy}{i}, \\
\}
\]

\[
f_{\text{ave, side}}(L_1, L_2) = 0.578
\]

Calculate Ultimate Side Friction, Qs

\[
\kappa = 0.53 \\
\kappa = 0.53 \text{ for driven piles, 0.62 for jacked piles, and 0.49 for bored piles.}
\]

\[
f_y(L_1, L_2, k_2) = \frac{f_{\text{ave, side}}(L_1, L_2) \times L_2}{1.2} \times 1.5 \\
f_y(L_1, L_2, k_2) = 0.578 \times \frac{10n}{s^2}
\]

\[
Q_s(L_1, L_2, k_2) = f_y(L_1, L_2, k_2) \times D \times (L_2 - L_1)
\]

\[
Q_s(L_1, L_2, k_2) = 13.592 \text{ton}
\]

\[
L_1 = 4\text{ft} \quad L_2 = 5.8\text{ft} \quad L_3 = 27.9\text{ft} \quad L_4 = 36.5\text{ft} \quad L_5 = 1\text{ft}
\]

Layer 1:

\[
Q_s[0.01\text{ft}, L_1, k_2] = 7.272 \text{ton} \\
Q_s[0.01\text{ft}, L_1, k_2] = Q_s[0.01\text{ft}, L_1, k_2]
\]

Layer 2:

\[
Q_s[L_1, L_2, k_2] = 6.395 \text{ton} \\
Q_s[L_1, L_2, k_2] = Q_s[L_1, L_2, k_2]
\]

Layer 3:

\[
Q_s[L_2, L_3, k_2] = 29.74 \text{ton} \\
Q_s[L_2, L_3, k_2] = Q_s[L_2, L_3, k_2]
\]

Layer 4:

\[
Q_s[L_3, L_4, k_2] = 3.262 \text{ton} \\
Q_s[L_3, L_4, k_2] = Q_s[L_3, L_4, k_2]
\]

Layer 5:

\[
Q_s[L_4, L_5, k_2] = 16.264 \text{ton} \\
Q_s[L_4, L_5, k_2] = Q_s[L_4, L_5, k_2]
\]

\[
Q_s = Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5}
\]

\[
Q_s = 62.022 \text{ton}
\]

\[
Q_{\text{Deformation}} = Q_s + \frac{Q_d(L)}{3} \\
Q_{\text{Deformation}} = 92.212 \text{ton}
\]
Tumay and Fakhroo Method (1982)

\[
exchange := 0.164 \text{in/ft}
\]

Ultimate Pile Capacity Based on CPT data Tumay and Fakhroo Method (1982)

**For Clayey Soil Only**

\[ORIGIN = 1\]

Input Data:

\[CPT := \text{DATA input}\]

\[D := 18\text{in}\]

\[L := 48\text{ft}\]

Diameter or Edge Length of the Pile

Length of the Pile

Average \(q_e\) over a distance of \(yD\) below the pile tip: \(q_e\_ave\_below(CPT, D, L, y)\)

\[
q_{\text{ave\_below}}(CPT, D, L, y) := \left(\frac{1}{exchange}\right)
\]

\[i := 0\]

\[q_{\text{below\_1}} := 0\]

while \(CPT_{n+1,1} \leq \frac{L + yD}{n}\)

\[q_{\text{below\_1}} := q_{\text{below\_1}} + CPT_{n+1,2}\]

\[i := i + 1\]

\[i := i - 1\]

\[\text{counter} := i\]

\[q_{\text{below\_2}} := 0\]

\[q_{\text{below\_min}} := CPT_{n+1,2}\]

while \(i \geq 0\)

if \(q_{\text{below\_min}} \leq CPT_{n+1,2}\)

\[q_{\text{below\_2}} := q_{\text{below\_2}} + q_{\text{below\_min}}\]

\[i := i - 1\]

otherwise

\[q_{\text{below\_min}} := CPT_{n+1,2}\]

\[q_{\text{below\_2}} := q_{\text{below\_2}} + q_{\text{below\_min}}\]

\[i := i - 1\]

\[q_{\text{ave\_below}} := \frac{q_{\text{below\_1}} + q_{\text{below\_2}}}{2(\text{counter} + 1)}\]

return \(q_{\text{ave\_below}}\)
\( q_2(L) = q_{\text{ave below}}(CPT, D, L, 4) \)

\[
q_2(L) :=
\begin{align*}
\text{return } "L is too short" & \text{ if } L \leq 3D \\
n & \left\lfloor \frac{L}{\text{exchange}} \right\rfloor \\
n_n & \left\lceil \frac{L}{\text{exchange}} \right\rceil \\
q_1 & \left\lfloor y \right\rfloor \\
q_1 & \left\lfloor q_{\text{ave below}}(CPT, D, L, 0.7) \right\rfloor \\
& \text{for } y \in 0.7, 0.71, \ldots 4 \\
& \text{if } q_1 \geq q_{\text{ave below}}(CPT, D, L, y) \\
& q_1 \left\lfloor q_{\text{ave below}}(CPT, D, L, y) \right\rfloor \\
y & \left\lfloor y \right\rfloor \\
q_{\text{below min}} & \left\lfloor i \right\rfloor \\
& \text{while } CPT_{n+i, 1} \leq \frac{L + q_1 \cdot D}{\text{f}} \\
& \text{if } CPT_{n+i, 1} \leq \frac{L - 8 \cdot D}{\text{f}} \\
& \text{if } q_{\text{below min}} \leq CPT_{n-i, 2} \\
& q_{\text{above}} \left\lfloor q_{\text{above}} + q_{\text{below min}} \right\rfloor \\
& i \left\lfloor i \right\rfloor + 1 \\
& \text{otherwise} \\
& q_{\text{below min}} \left\lceil CPT_{n-i, 2} \right\rceil \\
& q_{\text{above}} \left\lfloor q_{\text{above}} + q_{\text{above min}} \right\rfloor \\
& i \left\lfloor i \right\rfloor + 1 \\
\end{align*}
\]

\[ q_{\text{above}} \left\lfloor \frac{q_{\text{above}}}{i} \right\rfloor \]

\[ q_2(L) = 67.893 \]
Calculation of the average cone tip resistance in Tumay and Fakhroo Method:

\[ q_{c1}(L) = 122.554 \]
\[ q_{c2}(L) = 67.893 \]

\[ q_t(L) = \begin{cases} \frac{q_{c1}(L) + q_{c2}(L)}{2} & \leq 150, \\ \frac{q_{c1}(L) + q_{c2}(L)}{2} & , 150 \end{cases} \left( \frac{\text{ton}}{\text{m}^2} \right) \]
\[ q_{t}(L) = 95.224 \left( \frac{\text{ton}}{\text{m}^2} \right) \]

Tip resistance:

\[ A := D^2 \quad A = 2.25 \text{ m}^2 \]

\[ Q_t(L) := A \cdot q_t(L) \quad Q_t(L) = 214.353 \text{ ton} \]

Side Friction (Tumay and Fakhroo Method):

Calculate Ultimate Side Friction: \( Q_s \)

\[ L_1 := 3.8 \text{ ft} \quad \text{Input the soil layer} \]
\[ L_2 := 11.2 \text{ ft} \]

\[ Q_s(L_1, L_2) := \begin{cases} n \left( \text{ceil} \left( \frac{L_1}{\text{exchange}} \right) \right) \\ m \left( \text{floor} \left( \frac{L_2}{\text{exchange}} \right) \right) \\ A_s = 4 \cdot D \cdot (L_2 - L_1) \\ \sigma_{sum} = 0 \\ \text{for } j = n \text{ to } m \\ \sigma_{sum} = \sigma_{sum} + CPT_j,3 \\ \sigma_s = \frac{\sigma_{sum}}{m - n + 1} \\ m = 0.5 + 9.5 \cdot e \\ \sigma_e = \begin{cases} \sigma_s & \text{if } m \cdot \sigma_s \leq 0.72, m \cdot \sigma_s \geq 0.72 \end{cases} \\ Q_s = \sigma_e \cdot A_s \left( \frac{\text{ton}}{\text{m}^2} \right) \\ \text{return } Q_s \end{cases} \]

\[ Q_s(L_1, L_2) = 14.242 \text{ ton} \]
Aoki and De Alencar Method (1975)

\[ L_1 = 2.8 \text{ m} \quad L_2 = 5.8 \text{ m} \quad L_3 = 27.9 \text{ m} \quad L_4 = 39.6 \text{ m} \quad L_5 = L \]

Layer 1:
\[ q_{e1,0}(L_1, L_1) = 7.532 \text{ ton} \quad q_{e1} = q_{e1,0}(L_1, L_1) \]

Layer 2:
\[ q_{e2}(L_1, L_2) = 5.951 \text{ ton} \quad q_{e2} = q_{e2}(L_1, L_2) \]

Layer 3:
\[ q_{e3}(L_2, L_3) = 9.846 \text{ ton} \quad q_{e3} = q_{e3}(L_2, L_3) \]

Layer 4:
\[ q_{e4}(L_3, L_4) = 23.086 \text{ ton} \quad q_{e4} = q_{e4}(L_3, L_4) \]

Layer 5:
\[ q_{e5}(L_4, L_5) = 21.667 \text{ ton} \quad q_{e5} = q_{e5}(L_4, L_5) \]

\[ q_s = q_{e1} + q_{e2} + q_{e3} + q_{e4} + q_{e5} \]

\[ Q_{	ext{Darendo}} = q_s + \frac{q_{e1,0}(L_1, L_1)}{3} \quad Q_{	ext{Davisaac}} = 169.499 \text{ ton} \]

Aoki and De Alencar Method (1975)

\[ \text{exchange} = 0.18014 \]

Ultimate Pile Capacity Based on CPT data Aoki & de Alencar (1975)

ORIGDF = 1

Input Data:

\[ 
\begin{array}{c}
\text{CPT} \\
\text{DATA input}
\end{array}
\]

\[ D = 1.8 \text{ m} \quad \text{Diameter or Edge Length of the Pile} \]

\[ L = 48.6 \text{ m} \quad \text{Length of the Pile} \]

Calculation of the average cone tip resistance using Aoki & de Alencar method:

\[ q_{e\text{ave,edge}}(L_1, L_2) = \left( \frac{L_1}{\text{exchange}} \right) \]

\[ a = 0 \]

\[ b = 0 \]

\[ q_{e} = 0 \]

\[ \text{while} \ CPT_{\text{edge}} \leq \frac{L_2}{a} \]

\[ q_{e} = q_{e} + CPT_{\text{edge}} \]

\[ i = i + 1 \]

\[ q_{e\text{ave}} = \frac{q_{e}}{i} \]

return \[ q_{e\text{ave}} \]
\[ q_{\text{ave, tip}}(D, L) = \frac{q_{\text{ave, side}}(D, L) + q_{\text{ave, side}}(L, L)}{2} \]

Influence zone is 5D above and 4D below the pile tip.

\[ q_{\text{ave, tip}}(D, L) = 112.72 \]

\[ F_p = 1.75 \]

\[ q(L) = \begin{cases} q_{\text{ave, tip}}(D, L) & \text{if } \frac{q_{\text{ave, tip}}(D, L)}{F_p} \leq 15.0, \frac{q_{\text{ave, side}}(L, L)}{F_p} \leq 15.0 \\ \tan \left( \frac{90}{\sqrt{A}} \right) & \text{otherwise} \end{cases} \]

\[ q(L) = 644.2 \left( \frac{1}{L^2} \right) \text{kN} \text{m} \]

Tip resistance:

\[ A = D^2 \]
\[ A = 2.25 \text{m}^2 \]

\[ Q(L) = A \cdot q(L) \]
\[ Q(L) = 144.927 \text{ton} \]

Side friction (Aoki & de Alencar method):

Calculate Ultimate Side Friction, \( Q_s \)

\[ F_p = 3.1 \]

\[ f_s(L, 1, \sigma_q) = \begin{cases} q_{\text{ave, side}}(L, 1, 12) \frac{\sigma_q}{F_p} & \text{if } 1.2, q_{\text{ave, side}}(L, 12) \frac{\sigma_q}{F_p} \leq 1.2, q_{\text{ave, side}}(L, 12) \frac{\sigma_q}{F_p} \leq 1.2 \\ \tan \left( \frac{90}{\sqrt{A}} \right) & \text{otherwise} \end{cases} \]

\[ Q_s(L, 1, \sigma_q) = f_s(L, 1, \sigma_q) \cdot 4 \cdot (2 - L) \]

\begin{table}
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>( \alpha_q ) (%)</th>
<th>Soil Type</th>
<th>( \alpha_q ) (%)</th>
<th>Soil Type</th>
<th>( \alpha_q ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>1.4</td>
<td>Sandy silt</td>
<td>2.2</td>
<td>Sandy clay</td>
<td>2.4</td>
</tr>
<tr>
<td>Silty sand</td>
<td>2.0</td>
<td>Sandy silt with clay</td>
<td>2.8</td>
<td>Sandy clay with silt</td>
<td>2.8</td>
</tr>
<tr>
<td>Silty sand with clay</td>
<td>2.4</td>
<td>Silty clay</td>
<td>3.0</td>
<td>Silt clay with sand</td>
<td>3.0</td>
</tr>
<tr>
<td>Clayey sand with silt</td>
<td>2.8</td>
<td>Clayey silt</td>
<td>3.0</td>
<td>Silty clay</td>
<td>4.0</td>
</tr>
<tr>
<td>Clayey sand</td>
<td>3.0</td>
<td>Clayey silt</td>
<td>3.4</td>
<td>Clay</td>
<td>6.0</td>
</tr>
</tbody>
</table>
\end{table}

\[ L_1 = 2D \quad L_2 = 5.0D \quad L_3 = 27.9D \quad L_4 = 36.9D \quad L_5 = L \]

\[ \sigma_q = 1.7\% \]

Layer 1:

\[ Q_{a1} = Q_s(L, 1, \sigma_q) \]
\[ Q_{a1} = 8.176 \text{ton} \]

\[ \alpha_q = 3.7\% \]

Layer 2:

\[ Q_{a2} = Q_s(L, 1, \sigma_q) \]
\[ Q_{a2} = 42.64 \text{ton} \]

\[ \alpha_q = 1.6\% \]

Layer 3:

\[ Q_{a3} = Q_s(L, 1, \sigma_q) \]
\[ Q_{a3} = 42.58 \text{ton} \]

\[ \alpha_q = 5\% \]

Layer 4:

\[ Q_{a4} = Q_s(L, 1, \sigma_q) \]
\[ Q_{a4} = 39.98 \text{ton} \]

\[ \alpha_q = 1.4\% \]

Layer 5:

\[ Q_{a5} = Q_s(L, 1, \sigma_q) \]
\[ Q_{a5} = 24.31 \text{ton} \]

\[ Q_s = Q_{a1} + Q_{a2} + Q_{a3} + Q_{a4} + Q_{a5} \]
\[ Q_s = 83.872 \text{ton} \]

\[ Q_{\text{Design}} = \frac{Q_s}{2} \]
\[ Q_{\text{Design}} = 41.936 \text{ton} \]
Philipponnat Method (1980)

\[
\text{exchange} = 0.164042\text{ft}
\]

Ultimate Pile Capacity Based on CPT data Philipponnat Method (1980)

\[\text{ORIGIN} = 1\]

Input Data:

- CPT := C:\...\CPT.xls

- \(D = 13\text{in}\)  \(\text{Diameter or Edge Length of the Pile}\)

- \(L = 48.5\text{ft}\)  \(\text{Length of the Pile}\)

Calculation of the average cone tip resistance using Philipponnat Method (1980):

\[
\begin{align*}
q_{\text{ave\_side}}(L_1, L_2) &= n \left\lfloor \frac{L_1}{\text{exchange}} \right\rfloor \\
i &= 0 \\
k &= 0 \\
q_c &= 0 \\
\text{while } CPT_{n+i, 1} \leq \frac{L_2}{n} \\
q_c &= q_c + CPT_{n+i, 2} \\
i &= i + 1 \\
q_{\text{ave}} &= \frac{q_c}{i} \\
\text{return } q_{\text{ave}}
\end{align*}
\]

\[q_{\text{ave\_tip1}}(D, L) = q_{\text{ave\_side}}(L - 3D, L)\]  \(\text{Average tip resistance within 3D above the pile tip}\)

\[q_{\text{ave\_tip2}}(D, L) = q_{\text{ave\_side}}(L + 3D, L)\]  \(\text{Average tip resistance within 3D below the pile tip}\)

\[q_{\text{ave\_tip}}(D, L) = \begin{cases} q_{\text{ave\_tip1}}(D, L) & \text{if } q_{\text{ave\_tip1}}(D, L) \leq q_{\text{ave\_tip2}}(D, L), \\
\frac{q_{\text{ave\_tip1}}(D, L) + q_{\text{ave\_tip2}}(D, L)}{2} & \text{otherwise.}
\end{cases}\]

\[q_{\text{ave\_tip1}}(D, L) = 118.409\]

\[q_{\text{ave\_tip2}}(D, L) = 147.483\]

\[q_{\text{ave\_tip}}(D, L) = 132.946\]
\( \alpha_0 = 0.4 \)

\[
\phi(L) = \frac{q_{\text{ave,tip}}(D, L) \cdot \alpha_0 \cdot \phi_{\text{c}}}{n^2} \cdot q(L) = 53.178 \frac{1}{n^2} \text{ton}
\]

Tip resistance:
\[
A = D^2 \quad A = 2.25 \text{ft}^2
\]

\[
Q(L) = A \cdot q(L) \quad Q(L) = 119.651 \text{ton}
\]

Side Friction (Philippouhat Method (1980)):

Calculate Ultimate Side Friction: \( Q_s \)

\[ L_1 = 6 \text{ft} \quad \text{Input the soil layer} \]

\[ L_2 = 36 \text{ft} \]

\[ F_s = 100 \]

\[
f_s[1,1,2,\alpha_s, F_s] = \left( q_{\text{ave, side}}(L_1, L_2) \cdot \frac{\alpha_s}{F_s} \right) + 1.27, q_{\text{ave, side}}(L_1, L_2) \cdot \frac{\alpha_s}{F_s} = 1.27 \]

\[
f_s[1,1,2,\alpha_s, F_s] = \frac{q_s[1,1,2,\alpha_s, F_s]}{A} - 1.27 \]

\[ Q_s[1,1,2,\alpha_s, F_s] = q_s[1,1,2,\alpha_s, F_s] \cdot 4D(L_2 - 1) \]

\[ Q_s[1,1,2,\alpha_s, F_s] = 1 \text{ton} \]

\( \alpha_s \) is equal to 1.25 for prestressed concrete driven piles

\[
\begin{align*}
L_1 & = 4 \text{ft} & L_2 & = 58 \text{ft} & L_3 & = 27.9 \text{ft} & L_4 & = 36.5 \text{ft} & L_5 & = L \\
\end{align*}
\]

Layer 1:
\[
\alpha_s = 1.25 \quad F_s = 150
\]

\[
Q_{s1} = q_s[0.61(3,1,\alpha_s, F_s)] \quad Q_{s1} = 14.227 \text{ton}
\]

Layer 2:
\[
\alpha_s = 1.25 \quad F_s = 60
\]

\[
Q_{s2} = q_s[1,1,2,\alpha_s, F_s] \quad Q_{s2} = 8.305 \text{ton}
\]

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Empirical factor ( F_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay and calcareous clay</td>
<td>50</td>
</tr>
<tr>
<td>Silt, sandy clay, and clayey sand</td>
<td>60</td>
</tr>
<tr>
<td>Loose sand</td>
<td>100</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>150</td>
</tr>
<tr>
<td>Dense sand and gravel</td>
<td>200</td>
</tr>
</tbody>
</table>
Layer 3: \( \omega_p = 1.25 \quad F_p = 150 \)

\[ Q_{S3} = Q_3 [L_2, L_3, \omega_p, F_p] \quad Q_{S3} = 77.032 \text{ ton} \]

Layer 4: \( \omega_p = 1.25 \quad F_p = 50 \)

\[ Q_{S4} = Q_4 [L_3, L_4, \omega_p, F_p] \quad Q_{S4} = 6.856 \text{ ton} \]

Layer 5: \( \omega_p = 1.25 \quad F_p = 150 \)

\[ Q_{S5} = Q_5 [L_4, L_5, \omega_p, F_p] \quad Q_{S5} = 50.648 \text{ ton} \]

\[ Q_S = Q_{S1} + Q_{S2} + Q_{S3} + Q_{S4} + Q_{S5} \quad Q_S = 156.369 \text{ ton} \]

\[ Q_{Deviation} = Q_S + \frac{Q_S(L)}{2} \quad Q_{Deviation} = 196.733 \text{ ton} \]

**LCPC (Bustamante and Gianceselli) Method (1982)**

exchange := 0.164042ft

Ultimate Pile Capacity Based on CPT data Bustamante and Gianceselli (1982)

**ORIGIN := 1**

Input Data:

CPT := \( \text{DATA input} \)

D := 18in

Diameter or Edge Length of the Pile

L := 48.5ft

Length of the Pile

Calculate the equivalent average cone tip resistance around the pile tip \( c_{eq}(CPT, D, L) \)
\[ q_{eq\_side}(L_1, L_2) := \begin{align*}
  n &\leftarrow \text{ceil}\left(\frac{L_1}{\text{exchange}}\right) \\
  i &\leftarrow 0 \\
  k &\leftarrow 0 \\
  q_c &\leftarrow 0 \\
  q_{eq} &\leftarrow 0 \\
  \text{while } CPT_{n+i,1} \leq \frac{L_2}{k} \\
    &\begin{align*}
      q_c &\leftarrow q_c + CPT_{n+i,2} \\
      i &\leftarrow i + 1 \\
      q_{ave} &\leftarrow \frac{q_c}{i} \\
    \end{align*} \\
  \text{for } m \in n..n+i-1 \\
  &\begin{align*}
    &\text{while } CPT_{m,2} \geq 0.7 \cdot q_{ave} \land CPT_{m,2} \leq 1.3 q_{ave} \\
    &\begin{align*}
      q_{eq} &\leftarrow q_{eq} + CPT_{m,2} \\
      k &\leftarrow k + 1 \\
      \text{break} \\
    \end{align*} \\
    q_{eq} &\leftarrow \frac{q_{eq}}{k} \\
  \end{align*} \\
\text{return } q_{eq} \\
\end{align*} \]

\[ q_{eq\_tip}(D, L) := q_{eq\_side}(L - 1.5D, L + 1.5D) \]

\[ q_{eq\_tip}(D, L) = 140.821 \]

Calculate unit tip bearing capacity \( q_t \) and Tip resistance \( Q_t \):

\[ \text{SoilType} = 2 \]

1 for clay and silt, 2 for sand and gravel, 3 for Chalk

\[ k_3(L) = \begin{cases} 
  1, & \text{if (SoilType = 1, 0.6, if (SoilType = 2, 0.375, 0.4)} \\
  1.06 \cdot \text{if (SoilType = 2, 0.375, 0.4)} \\
\end{cases} \]

\[ k_3(L) = 0.375 \]

\[ q_t(D, L) = q_{eq\_tip}(D, L) \cdot k_3(L) \cdot \frac{\text{ton}}{\text{ft}^2} \quad q_t(D, L) = 52.808 \cdot \frac{\text{ton}}{\text{ft}^2} \]

\[ A = D^2 \quad A = 2.25 \text{ ft}^2 \]

\[ Q_t(D, L) = A \cdot q_t(D, L) \quad Q_t(D, L) = 118.817 \text{ ton} \]
Side Friction (Bustamante and Giueselli)

Calculate $f_{max}(t):$

\[
\begin{bmatrix}
0 \\
5 \\
10 \\
20 \\
30 \\
40 \\
50 \\
60 \\
70 \\
80 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0 \\
25 \\
50 \\
100 \\
150 \\
200 \\
250 \\
300 \\
350 \\
400 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0 \\
10 \\
20 \\
30 \\
60 \\
90 \\
120 \\
150 \\
180 \\
210 \\
\end{bmatrix}
\]

\[
q_{\text{clay-silt}} = \begin{bmatrix}
0.239 & 0.375 & 0.5 & 0.5 & 0.739 \\
0.354 & 0.581 & 0.774 & 0.774 & 1 \\
0.374 & 0.765 & 1 & 1 & 1.337 \\
0.25 & 0.855 & 1.13 & 1.17 & 1.6 \\
0.375 & 0.87 & 1.174 & 1.261 & 1.739 \\
0.375 & 0.883 & 1.209 & 1.339 & 1.865 \\
0.25 & 0.913 & 1.23 & 1.374 & 1.922 \\
0.375 & 0.922 & 1.261 & 1.404 & 2 \\
0.375 & 0.937 & 1.291 & 1.439 & 2.043 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
1 \\
2 \\
3 \\
4 \\
5 \\
6 \\
7 \\
8 \\
9 \\
10 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0.385 \\
3.591 \\
1.13 \\
1.17 \\
1.6 \\
1.337 \\
1.13 \\
1.17 \\
1.261 \\
1.739 \\
1.209 \\
1.339 \\
1.865 \\
1.23 \\
1.374 \\
1.922 \\
1.261 \\
1.404 \\
2.043 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0.22 & 0.4 & 0.513 & 0.513 & 0.681 \\
0.369 & 0.534 & 0.832 & 0.832 & 1.16 \\
0.366 & 0.796 & 1.094 & 1.194 & 1.523 \\
0.377 & 0.89 & 1.215 & 1.361 & 1.937 \\
0.387 & 0.932 & 1.257 & 1.461 & 2.064 \\
0.398 & 0.937 & 1.282 & 1.482 & 2.188 \\
0.403 & 0.942 & 1.308 & 1.524 & 2.282 \\
0.408 & 0.953 & 1.339 & 1.555 & 2.334 \\
0.408 & 0.953 & 1.339 & 1.555 & 2.314 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\text{ton} \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0.14 & 0.21 & 0.307 & 0.307 \\
0.222 & 0.378 & 0.314 & 0.375 \\
0.246 & 0.492 & 0.626 & 0.809 \\
0.297 & 0.699 & 0.951 & 1.305 \\
0.367 & 0.802 & 1.126 & 1.577 \\
0.376 & 0.864 & 1.243 & 1.765 \\
0.386 & 0.9 & 1.305 & 1.991 \\
0.294 & 0.924 & 1.338 & 1.984 \\
0.404 & 0.934 & 1.399 & 2.057 \\
0.41 & 0.944 & 1.441 & 2.151 \\
\end{bmatrix}
\]
\[ L_1 = 0.16 \]
\[ L_2 = 36ft \]

\[ q_{eq,sub(L1,L2)} = 57.052 \]

\[ \text{ScTNo} = 2 \quad \text{1 for clay and silt, 2 for sand and gravel, 3 for chalk} \]

\[ \text{CurveNo} = 2 \]

\[
f_{\text{max}}(\text{ScTNo, CurveNo, L1, L2}) = \begin{cases} 
q_{\text{clay_silt, frac滟cay_silt} \frac{\text{CurveNo}}{q_{\text{eq, sub(L1, L2)}}}} & \text{if ScTNo} = 1 \\
q_{\text{sand_gravel, frac滟sand_gravel} \frac{\text{CurveNo}}{q_{\text{eq, silt(L1, L2)}}}} & \text{if ScTNo} = 2 \\
q_{\text{chalk, frac滟chalk} \frac{\text{CurveNo}}{q_{\text{eq, sub(L1, L2)}}}} & \text{otherwise} 
\end{cases}
\]

\[ f_{\text{max}}(\text{ScTNo, CurveNo, L1, L2}) = 0.657 \frac{kN}{m^2} \]

Calculate Ultimate Side Friction, Qs

\[ Q_s(L1, L2, \text{ScTNo, CurveNo}) = f_{\text{max}}(\text{ScTNo, CurveNo, L1, L2}) \cdot D \cdot (L2 - L1) \]

\[ Q_s(L1, L2, \text{ScTNo, CurveNo}) = 141.48 \text{ton} \]

\[ \text{SoilType} = 1 \text{ for cohesive soil, 2 for cohesionless soil, 3 for chalk} \]

<table>
<thead>
<tr>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
<th>L8</th>
<th>L9</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.9ft</td>
<td>26.5ft</td>
<td>27.3ft</td>
<td>27.5ft</td>
<td>30.5ft</td>
<td>30.7ft</td>
<td>40.3ft</td>
<td>117</td>
<td></td>
</tr>
</tbody>
</table>

Layer 1:
- \[ \text{SoilType} = 2 \quad \text{CurveNo} = 1 \]
- \[ Q_{s1} = Q_s(0.01ft, L1, \text{SoilType, CurveNo}) \]
- \[ Q_{s1} = 0.397 \text{ton} \]

Layer 2:
- \[ \text{SoilType} = 1 \quad \text{CurveNo} = 2 \]
- \[ Q_{s2} = Q_s(L1, L2, \text{SoilType, CurveNo}) \]
- \[ Q_{s2} = 3.193 \text{ton} \]

Layer 3:
- \[ \text{SoilType} = 2 \quad \text{CurveNo} = 3 \]
- \[ Q_{s3} = Q_s(L1, L2, \text{SoilType, CurveNo}) \]
- \[ Q_{s3} = 11.754 \text{ton} \]

Layer 4:
- \[ \text{SoilType} = 1 \quad \text{CurveNo} = 2 \]
- \[ Q_{s4} = Q_s(L1, L2, \text{SoilType, CurveNo}) \]
- \[ Q_{s4} = 5.952 \text{ton} \]

Layer 5:
- \[ \text{SoilType} = 2 \quad \text{CurveNo} = 2 \]
- \[ Q_{s5} = Q_s(L1, L2, \text{SoilType, CurveNo}) \]
- \[ Q_{s5} = 5.582 \text{ton} \]

Layer 6:
- \[ \text{SoilType} = 2 \quad \text{CurveNo} = 2 \]
- \[ Q_{s6} = Q_s(L1, L2, \text{SoilType, CurveNo}) \]
- \[ Q_{s6} = 52.706 \text{ton} \]
Layer 7: \( \text{SoilType} = 1 \quad \text{CurveNo} = 3 \)
\[ Q_87 = Q_{d}(L6, L7, \text{SoilType}, \text{CurveNo}) \quad Q_87 = 13.456 \text{ ton} \]

Layer 8: \( \text{SoilType} = 2 \quad \text{CurveNo} = 2 \)
\[ Q_88 = Q_{d}(L7, L8, \text{SoilType}, \text{CurveNo}) \quad Q_88 = 3.16 \text{ ton} \]

Layer 9: \( \text{SoilType} = 1 \quad \text{CurveNo} = 1 \)
\[ Q_89 = Q_{d}(L8, L9, \text{SoilType}, \text{CurveNo}) \quad Q_89 = 1.92 \text{ ton} \]

Layer 10: \( \text{SoilType} = 2 \quad \text{CurveNo} = 2 \)
\[ Q_{10} = Q_{d}(L9, L10, \text{SoilType}, \text{CurveNo}) \quad Q_{10} = 2.337 \text{ ton} \]

Layer 11: \( \text{SoilType} = 2 \quad \text{CurveNo} = 3 \)
\[ Q_{11} = Q_{d}(L10, L11, \text{SoilType}, \text{CurveNo}) \quad Q_{11} = 26.835 \text{ ton} \]

Layer 12: \( \text{SoilType} = 1 \quad \text{CurveNo} = 2 \)
\[ Q_{12} = Q_{d}(L11, L12, \text{SoilType}, \text{CurveNo}) \quad Q_{12} = 2.209 \text{ ton} \]

Layer 13: \( \text{SoilType} = 2 \quad \text{CurveNo} = 1 \)
\[ Q_{13} = Q_{d}(L12, L13, \text{SoilType}, \text{CurveNo}) \quad Q_{13} = 0.763 \text{ ton} \]

Layer 14: \( \text{SoilType} = 1 \quad \text{CurveNo} = 1 \)
\[ Q_{14} = Q_{d}(L13, L14, \text{SoilType}, \text{CurveNo}) \quad Q_{14} = 11.466 \text{ ton} \]

Layer 15: \( \text{SoilType} = 2 \quad \text{CurveNo} = 1 \)
\[ Q_{15} = Q_{d}(L14, L15, \text{SoilType}, \text{CurveNo}) \quad Q_{15} = 3.326 \text{ ton} \]

Layer 16: \( \text{SoilType} = 2 \quad \text{CurveNo} = 2 \)
\[ Q_{16} = Q_{d}(L15, L16, \text{SoilType}, \text{CurveNo}) \quad Q_{16} = 2.283 \text{ ton} \]

Layer 17: \( \text{SoilType} = 1 \quad \text{CurveNo} = 3 \)
\[ Q_{17} = Q_{d}(L16, L17, \text{SoilType}, \text{CurveNo}) \quad Q_{17} = 68.007 \text{ ton} \]

\[ Q_s = Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5} + Q_{s6} + Q_{s7} + Q_{s8} + Q_{s9} + Q_{s10} + Q_{s11} + Q_{s12} + Q_{s13} + Q_{s14} + Q_{s15} + Q_{s16} + Q_{s17} \]
\[ Q_s = 215.574 \text{ ton} \]

\[ Q_{\text{Davisson}} = Q_s + \frac{Q_{D(D,L)}}{3} \quad Q_{\text{Davisson}} = 255.18 \text{ ton} \]

\[
\text{exchange} = 0.164042a
\]


\[\text{ORIGIN} = 1\]

Input Data:

\[
\begin{align*}
\text{CPT} &= \text{C:\\CPT.xls} \\
D &= 18\text{ft} \\
L &= 48.5\text{ft} \\
\text{w} &= 5 \\
\text{w} &= \text{water table}
\end{align*}
\]

Diameter or Edge Length of the Pile

Length of the Pile

Calculation of the average cone tip resistance:

\[
\begin{align*}
q_{\text{ave\_side}}(L_1, L_2) &:= \begin{cases} 
\text{n} \leftarrow \text{ceil} \left( \frac{L_1}{\text{exchange}} \right) \\
i \leftarrow 0 \\
k \leftarrow 0 \\
q_c \leftarrow 0 \\
\text{while } CPT_{n+i,1} \leq \frac{L_2}{a} \\
q_c \leftarrow q_c + CPT_{n+i,2} \\
i \leftarrow i + 1 \\
q_{\text{ave}} \leftarrow \frac{q_c}{i} \\
\text{return } q_{\text{ave}}
\end{cases}
\end{align*}
\]

\[
\begin{align*}
q_{\text{ave\_tip}}(D, L) &= q_{\text{ave\_side}}(L - 8D, L) \\
q_{\text{ave\_tip2}}(D, L) &= q_{\text{ave\_side}}(L, L + 3D) \\
q_{\text{ave\_tip}}(D, L) &= \frac{q_{\text{ave\_tip}}(D, L) + q_{\text{ave\_tip2}}(D, L)}{2}
\end{align*}
\]

Average tip resistance within 8D above the pile tip

Average tip resistance within 3D below the pile tip

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\[ q_{\text{ave_tip}}(D, L) = 84.414 \]
\[ q_{\text{ave_tip}^2}(D, L) = 147.483 \]
\[ q_{\text{ave_tip}^3}(D, L) = 115.949 \]

\[ k_2 = 2.7 \]

\[ \sigma_{v0_{\text{tip}}} = \frac{95 \cdot h + 115 \left( \frac{L}{h} - h \right)}{2000} \]

\[ \sigma_{v0_{\text{tip}}} = 2.739 \]

\[ q_t(L) = 150 \left( \frac{q_{\text{ave_tip}^2}(D, L) - \sigma_{v0_{\text{tip}}}}{k_2} \right) \text{ ton/ft}^2 \]

\[ q_t(L) = 41.53 \frac{1}{\text{ton/ft}^2} \]

**Tip resistance:**

\[ A = D^2 \quad A = 2.25 \text{ ft}^2 \]

\[ Q_t(L) = A \cdot q_t(L) \]

\[ Q_t(L) = 94.342 \text{ ton} \]

**Side Friction:**

Calculate Ultimate Side Friction: \( Q_s \)

\[ L_1 = 0.01 \text{ ft} \]

\[ L_2 = 21.8 \text{ ft} \]

\[ \sigma_{v0} = 0.5 \]

\[ \sigma_{v0_{\text{ef}}} = 0.293 \]

\[ f_s(L_1, L_2, \sigma_{v0}, \sigma_{v0_{\text{ef}}}) = \max \left[ \frac{q_{\text{ave_side}}(L_1, L_2) - \sigma_{v0}}{11.8 + 14 \log \left( \frac{q_{\text{ave_side}}(L_1, L_2) - \sigma_{v0}}{\sigma_{v0_{\text{ef}}}} \right)} \right] \text{ ton/ft}^2 \]
\[ Q_s(\Omega, L, \sigma_{v0}, \sigma_{v0_{ef}}) = f_s(\Omega, L, \sigma_{v0}, \sigma_{v0_{ef}}) + \Omega (L - \Omega) \]

\[ Q_s(\Omega, L, \sigma_{v0}, \sigma_{v0_{ef}}) = 136.888 \text{ ton} \]

\[
\text{overburden}(\text{Depth}1, \text{Depth}2) = \begin{cases} 
\text{Depth} & \text{if} \left[ \frac{\text{Depth} \leq h}{2}, \frac{5.5 \text{ Depth}}{2000}, \frac{5.5 h + 11 \times (\text{Depth} - h)}{2000} \right] \\
\text{cut} & \text{if} \left[ \frac{\text{Depth} \leq h}{2}, \frac{5.5 \text{ Depth}}{2000}, \frac{5.5 h + (115 - 62.6) (\text{Depth} - h)}{2000} \right] 
\end{cases}
\]

\[\sigma_{v0}\text{ is equal to 1.25 for prestressed concrete driven piles}\]

<table>
<thead>
<tr>
<th>Layer</th>
<th>( L_0 = 0.01\text{ ft} )</th>
<th>( L_1 = 4\text{ ft} )</th>
<th>( L_2 = 5.8\text{ ft} )</th>
<th>( L_3 = 27.9\text{ ft} )</th>
<th>( L_4 = 36.5\text{ ft} )</th>
<th>( L_5 = 1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>( Q_{s1} = Q_s(\Omega, L, \text{overburden}(L_0, L_1), 1, \text{overburden}(L_0, L_1), 1, 2) )</td>
<td>( Q_{s1} = 28.728 \text{ ton} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 2</td>
<td>( Q_{s2} = Q_s(L_1, L_2, \text{overburden}(L_1, L_2), 1, 1, \text{overburden}(L_1, L_2), 1, 2) )</td>
<td>( Q_{s2} = 9.307 \text{ ton} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 3</td>
<td>( Q_{s3} = Q_s(L_2, L_3, \text{overburden}(L_2, L_3), 1, 1, \text{overburden}(L_2, L_3), 1, 2) )</td>
<td>( Q_{s3} = 159.12 \text{ ton} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 4</td>
<td>( Q_{s4} = Q_s(L_3, L_4, \text{overburden}(L_3, L_4), 1, 1, \text{overburden}(L_3, L_4), 1, 2) )</td>
<td>( Q_{s4} = 91.86 \text{ ton} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 5</td>
<td>( Q_{s5} = Q_s(L_4, L_5, \text{overburden}(L_4, L_5), 1, 1, \text{overburden}(L_4, L_5), 1, 2) )</td>
<td>( Q_{s5} = 86.41 \text{ ton} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ Q_s = Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5} \]

\[ Q_s = 292.741 \text{ ton} \]

\[ Q_{\text{Davisson}} = Q_s + \frac{Q_s(L)}{3} \]

\[ Q_{\text{Davisson}} = 324.188 \text{ ton} \]

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\[ \text{exchange} = 0.16402 \text{ft} \]

Ultimate Pile Capacity Based on CPT data MTD Method (1996)

**ORIGIN = 1**

Input Data:

\[ \text{CPT} := \quad \text{DATA input} \]

\[ \text{D} := 13 \text{m} \]

\[ L = 48.5 \text{ft} \]

Diameter or Edge Length of the Pile

Length of the Pile

Calculation of the average cone tip resistance:

\[
q_{\text{ave \_ side}}(L_1, L_2) := \begin{align*}
&n := \cos\left(\frac{L_1}{\text{exchange}}\right) \\
i &:= 0 \\
k &:= 0 \\
q_c &:= 0 \\
\text{while } &\text{CPT}_{n+i, 1} \leq \frac{L_2}{\text{ft}} \\
&q_c &:= q_c + \text{CPT}_{n+i, 2} \\
i &:= i + 1 \\
q_{\text{ave \_ side}} &:= \frac{q_c}{i} \\
\text{return } &q_{\text{ave \_ side}}
\end{align*}
\]

\[ q_{\text{ave \_ tip}}(D, L) = q_{\text{ave \_ side}}(L - 1.5D, L + 1.5D) \quad \text{Average tip resistance within 1.5D above and below the pile tip} \]

\[ q_{\text{ave \_ tip}}(D, L) = 140.82 \]

Soil type: 2

1 for cohesive soil, 2 for cohesionless soil

\[ k_{\text{sand}} = 0.445 \]

\[ k_{\text{clay}} = 1.3 \]

0.8 for drained loading, 1.3 for undrained loading
\[ q_t(I) = \begin{cases} \text{SoilType} = 1, k_{121} q_{ave\_tip}(D, L) & \text{ton} \\ k_{211} q_{ave\_tip}(D, L) & \text{ton} \end{cases} \]

Tip resistance:

\[ A = D^2 \quad A = 2.25 \text{ft}^2 \]

\[ Q_t(I) = A \cdot q_t(I) \]

\[ Q_t(I) = 141.32 \text{ton} \]

Side Friction:

Calculate Ultimate Side Friction: \( Q_s \)

\[ \text{SoilType} = 2 \quad 1 \text{ for cohesive soil, 2 for cohesionless soil} \]

\[ L_1 = 55 \text{ft} \quad \text{Input the soil layer} \]

\[ L_2 = 27.9 \text{ft} \]

\[ f_s = 0.8 \]

\[ W_T = 5 \quad \text{water table} \]

\[ \delta \rho = 11.31 \quad \frac{\text{ft}}{180} \]

\[ \tan(\delta \rho) = 0.2 \]

For Ts Calculation in Clay:

\[ q_{ave\_side}(L_1, L_2) = 69.712 \]

\[ \sigma_{\phi_0}(L_1, L_2) = \left[ \begin{array}{l} \text{Depth} \left\langle \frac{L_1 + L_2}{2} \text{ft} \right. \\
\sigma_{\phi_0} \left\langle \begin{cases} \text{Depth} \leq W_T, & \frac{95 \text{ Depth} + 95 \text{ W_T} + 115(\text{Depth} - \text{W_T})}{2000} \\
\sigma_{\phi_0} \left\rangle \begin{cases} \text{out} \left\langle \sigma_{\phi_0} \end{cases} \end{array} \right. \\
\sigma_{\phi_0}(L_1, L_2) = 0.919 \]

\[ \sigma_{\phi_0\_ef}(L_1, L_2) = \left[ \begin{array}{l} \text{Depth} \left\langle \frac{L_1 + L_2}{2} \text{ft} \right. \\
\sigma_{\phi_0\_ef} \left\langle \begin{cases} \text{Depth} \leq W_T, & \frac{95 \text{ Depth} + 95 \text{ W_T} + (115 - 62.4)(\text{Depth} - \text{W_T})}{2000} \\
\sigma_{\phi_0\_ef} \left\rangle \begin{cases} \text{out} \left\langle \sigma_{\phi_0\_ef} \end{array} \right. \\
\sigma_{\phi_0\_ef}(L_1, L_2) = 0.540 \]
\[ R_2(L_1, L_2) = \frac{n \left( \frac{L_1}{\text{exchange}} \right)}{R_2(L_1, L_2)} \]

\[ \text{while } CPT_{n+1,1} \leq \frac{L_2}{R_1} \]

\[ R_2 \leftarrow R_2 + \frac{CPT_{n+1,2}}{100} \]

\[ i \leftarrow i + 1 \]

\[ R_2 \leftarrow \frac{R_2}{i} \]

\[ \text{return } R_2 \]

\[ S(L_1, L_2) = \frac{10}{R_2(L_1, L_2)} \quad S(L_1, L_2) = 13.733 \]

\[ \text{YSR}(L_1, L_2) = 0.04427 \left( \frac{q_{\text{ave, sib}}(L_1, L_2)}{q_{\text{ave, ef}}(L_1, L_2)} \right)^{1.667} \quad \text{YSR}(L_1, L_2) = 142.175 \]

\[ K_2(L_1, L_2) = \left( 2.2 + 0.016 \frac{\text{YSR}(L_1, L_2) - 0.870 \log(S(L_1, L_2))}{\text{YSR}(L_1, L_2)} \right)^{0.42} \left( \frac{L - L_1 + L_2}{2} \leq \frac{L - L_1 + L_2}{2} \right) \]

\[ f_{s \text{- clay}}(L_1, L_2, \delta r) := \frac{K_2(L_1, L_2) \sigma_{\text{ave}} \tan(\theta r)}{n^2} \quad f_{s \text{- clay}}(L_1, L_2, \delta r) = 1.315 \frac{\tan(\theta r)}{n^2} \]

For \( f_s \) Calculation in Sand

\[ \sigma_{tr}(L_1, L_2) = 0.035 \frac{q_{\text{ave, sib}}(L_1, L_2)}{1.581} \left( \frac{\sigma_{\text{ef}}(L_1, L_2)}{1.081} \right)^{0.13} \left( \frac{L - L_1 + L_2}{2} \leq \frac{L - L_1 + L_2}{2} \right) \quad \sigma_{tr}(L_1, L_2) = 0.58 \]

\[ \sigma_{tr}(L_1, L_2) = \frac{q_{\text{ave, sib}}(L_1, L_2)}{1.081} \left( \frac{\sigma_{\text{ef}}(L_1, L_2)}{1.081} \right)^{0.5} \quad \sigma_{tr}(L_1, L_2) = 0.73 \]

\[ \frac{\Delta \sigma_{tr}(L_1, L_2)}{D} = \frac{4 \sigma_{tr}(L_1, L_2) \cdot 3 \times 10^{-5}}{m} \quad \Delta \sigma_{tr}(L_1, L_2) = 0.147 \]

\[ \sigma_{tr}(L_1, L_2) = \sigma_{tr}(L_1, L_2) + \Delta \sigma_{tr}(L_1, L_2) \quad \sigma_{tr}(L_1, L_2) = 0.73 \]

\[ f_{s \text{- sand}}(L_1, L_2, \delta r) := \frac{\sigma_{tr}(L_1, L_2) \tan(\theta r)}{n^2} \]

\[ Q_y(L_1, L_2, \text{SofType}, \delta r) := \text{if} \{ \text{SofType} = 1, f_{s \text{- clay}}(L_1, L_2, \delta r), f_{s \text{- sand}}(L_1, L_2, \delta r) \} \cdot 4 \cdot D \cdot (L_2 - L_1) \]

\[ Q_y(L_1, L_2, \text{SofType}, \delta r) = 19.233 \text{ ton} \]
\( \alpha_{gL} \) is equal to 1.25 for prestressed concrete driven piles.

\( \begin{align*}
L_0 &:= 0.01\text{ft} & L_1 &:= d_A & L_2 &:= 5.8\text{ft} & L_3 &:= 27.9\text{ft} & L_4 &:= 36.5\text{ft} & L_5 &:= L & L_6 &:= L
\end{align*} \)

Layer 1:
- \( \text{SoilType} = 2 \)
- \( \delta_f := 30 \frac{\pi}{180} \)
- \( Q_{s1} := Q_s(L_0, L_1, \text{SoilType}, \delta_f) \)
- \( Q_{s1} = 6.663 \text{ton} \)

Layer 2:
- \( \text{SoilType} = 1 \)
- \( \delta_f := 11.31 \frac{\pi}{180} \)
- \( Q_{s2} := Q_s(L_1, L_2, \text{SoilType}, \delta_f) \)
- \( Q_{s2} = 9.742 \text{ton} \)

Layer 3:
- \( \text{SoilType} = 2 \)
- \( \delta_f := 30 \frac{\pi}{180} \)
- \( Q_{s3} := Q_s(L_2, L_3, \text{SoilType}, \delta_f) \)
- \( Q_{s3} = 55.266 \text{ton} \)

Layer 4:
- \( \text{SoilType} = 1 \)
- \( \delta_f := 11.31 \frac{\pi}{180} \)
- \( Q_{s4} := Q_s(L_3, L_4, \text{SoilType}, \delta_f) \)
- \( Q_{s4} = 7.015 \text{ton} \)

Layer 5:
- \( \text{SoilType} = 2 \)
- \( \delta_f := 30 \frac{\pi}{180} \)
- \( Q_{s5} := Q_s(L_4, L_5, \text{SoilType}, \delta_f) \)
- \( Q_{s5} = 55.718 \text{ton} \)

Layer 6:
- \( \text{SoilType} = 1 \)
- \( \delta_f := 11.31 \frac{\pi}{180} \)
- \( Q_{s6} := Q_s(L_5, L_6, \text{SoilType}, \delta_f) \)
- \( Q_{s6} = 0 \text{ton} \)

Layer 7:
- \( \text{SoilType} = 2 \)
- \( \delta_f := 30 \frac{\pi}{180} \)
- \( Q_{s7} := Q_s(L_6, L, \text{SoilType}, \delta_f) \)
- \( Q_{s7} = 0 \text{ton} \)

\( Q_s = Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5} + Q_{s6} + Q_{s7} \)

\( Q_s = 135.005 \text{ton} \)

\( Q_{\text{Davison}} = \frac{Q_s(L)}{3} \)

\( Q_{\text{Davison}} = 182.049 \text{ton} \)
Eslami and Fellenius Method (1997)

\[ \text{exchange} = 0.164042 \phi \]

Ultimate Pile Capacity Based on CPT data Eslami and Fellenius (1996)

ORIGIN = 1

Input Data:

\[ \text{CPT} : \begin{cases} \text{Cl}\text{a}\text{,CPT_PP.xls} \\
\text{DATA input} \end{cases} \]

Diameter or Edge Length of the Pile

\[ D = 18\text{in} \]

Length of the Pile

\[ L = 48.5\text{ft} \]

Calculation of the average cone tip resistance using Eslami and Fellenius (1996)

\[
q_{\text{cave}}(L, L_2) := \begin{cases} 
\text{n} \leftarrow \text{cell} \left( \frac{L_1}{\text{exchange}} \right) \\
i \leftarrow 0 \\
k \leftarrow 0 \\
q_c \leftarrow 1 \\
\text{while} \ CPT_{n+i, 1} \leq \frac{L_2}{\Phi} \\
q_c \leftarrow q_c + \left( CPT_{n+i, 2} - CPT_{n+i, 1} \right) \\
i \leftarrow i + 1 \\
q_{\text{cave}} \leftarrow \sqrt{q_c} \\
\text{return } q_{\text{cave}} 
\end{cases}
\]

\[ q_{\text{cave tip1}}(D, L) := q_{\text{cave}}(L - 2D, L + 4D) \]

Influence zone is 2D above and 4D below the pile tip when the pile is installed through a dense soil into a weak soil

\[ q_{\text{cave tip2}}(D, L) := q_{\text{cave}}(L - 6D, L + 3.8D) \]

Influence zone is 6D above and 4D below the pile tip when the pile is installed through a weak soil into a dense soil

\[ C_1 = 1 \]

\[ q_{\text{f1}}(L) = q_{\text{cave tip1}}(D, L) \ C_1 \frac{\tan}{\Phi^2} \]

\[ q_{\text{f1}}(L) = 133.647 \frac{\tan}{\Phi^2} \]
\[ q_{2}(L) = q_{w_{1}} \times q_{L}(L) = \frac{1}{n^{2}} \]

Tip resistance:
\[ A = E^{2} \quad A = 2.25 \pi^{2} \]
\[ Q_{4}(L) = A \times q_{L}(L) \quad Q_{4}(L) = 360 \text{ ton} \]
\[ Q_{4}(L) = A \times q_{w}(L) \quad Q_{4}(L) = 179.407 \text{ ton} \]

Side Friction (Esani and Fellenius (1996)):
\[
q_{w, \text{side}}(L_1, L_2) = \begin{aligned}
&i = 0 \\
i = &1 \text{ if } L_i < \frac{L_2}{r} \\
q_{w} = &q_{w} + \left( q_{w, i+1, i} - q_{w, i+1, 0} \right) \\
i = &i + 1 \\
q_{w} = &q_{0} \\
&\text{return } q_{w}
\end{aligned}
\]

Calculate Ultimate Side Friction, Qs:
\[
Q_{s} = \begin{aligned}
&n = \text{ceil} \left( \frac{1}{\text{exchange}} \right) - 1 \\
Q_{s} = &0 \\
\text{for } i = 1 \ldots n \\
q_{e} = &\text{CPT}_{i, \text{2}} - \text{CPT}_{i, \text{4}} \\
f_{s} = &\text{CPT}_{i, \text{3}} \\
Q_{s} = &Q_{s} \text{ if } f_{s} = 0 \\
\text{otherwise} \\
Q_{s} = &Q_{s} + 0.00 \text{ qg-4D-exchange } \text{ if } q_{\text{g}} \leq 10 \text{ and } f_{s} \leq 0.21 \\
Q_{s} = &Q_{s} + 0.025 \text{ qg-4D-exchange } \text{ if } q_{\text{g}} \leq 10 \\
&0.28 \times \log_{10}(f_{s}) + 1.212 \\
&\wedge f_{s} > 0.21 \\
Q_{s} = &Q_{s} + 0.025 \text{ qg-4D-exchange } \text{ if } q_{\text{g}} > \max \left(10, 4, 10 \times 0.28 \times \log_{10}(f_{s}) + 1.212 \right) \\
&\wedge q_{\text{g}} \leq 10 \\
&0.45 \times \log_{10}(f_{s}) + 1.526 \\
Q_{s} = &Q_{s} + 0.01 \text{ qg-4D-exchange } \text{ if } q_{\text{g}} > \max \left(10, 4, 10 \times 0.45 \times \log_{10}(f_{s}) + 1.526 \right) \\
&\wedge q_{\text{g}} \leq 10 \\
&0.49 \times \log_{10}(f_{s}) + 1.3946 \\
Q_{s} = &Q_{s} + 0.004 \text{ qg-4D-exchange } \text{ if } q_{\text{g}} > \max \left(10, 4, 10 \times 0.49 \times \log_{10}(f_{s}) + 1.3946 \right) \\
&\wedge q_{\text{g}} \leq 10 \\
&0.49 \times \log_{10}(f_{s}) + 1.3946 \\
\text{return } Q_{s} \\
\end{aligned}
\]

\[ Q_{s} = 106.448 \text{ ton} \]

\[ Q_{\text{DenseSoil1}} = \frac{Q_{4}(L)}{3} \quad Q_{\text{DenseSoil1}} = 206.683 \text{ ton} \]

Influence zone is 2D above and 4D below the pile tip when the pile is installed through a dense soil into a weak soil.

\[ Q_{\text{DenseSoil2}} = \frac{Q_{4}(L)}{3} \quad Q_{\text{DenseSoil2}} = 166.271 \text{ ton} \]

Influence zone is 8D above and 4D below the pile tip when the pile is installed through a weak soil into a dense soil.
Powell et al. Method (2001)

\[ \text{exchange} = 0.164042 \text{ft} \]


\[ \text{ORIGIN} = 1 \]

Input Data:

\[
\begin{align*}
\text{CPT} := & \quad \text{DATA input} \\
D := & \quad 18 \text{ ft} \\
L := & \quad 48.5 \text{ ft} \\
\text{h} := & \quad 5 \quad \text{water table}
\end{align*}
\]

Diameter or Edge Length of the Pile

Length of the Pile

Calculation of the average cone tip resistance:

\[
q_{\text{ave, side}}(L_1, L_2) := \begin{cases} 
  n & \text{cell} \left( \frac{L_1}{\text{exchange}} \right) \\
  i & \text{if} \quad L_1 \leq \frac{L_2}{n} \\
  k & \text{if} \quad L_1 > \frac{L_2}{n} \\
  q_c & \text{if} \quad CPT_{n+1,1} \leq \frac{L_2}{n} \\
  q_c & \text{if} \quad CPT_{n+1,2} \\
  q_{\text{ave}} & \text{if} \quad i \leq \frac{L_2}{n} \\
  q_{\text{ave, side}} & \text{return} \quad q_{\text{ave}}
\end{cases}
\]

\[ q_{\text{ave, tip}}(D, L) = q_{\text{ave, side}}(L - 3D, L) \]  \quad \text{Average tip resistance within 3D above the pile tip}

\[ q_{\text{ave, tip}}(D, L) = q_{\text{ave, side}}(L, L + 3D) \]  \quad \text{Average tip resistance within 3D below the pile tip}

\[ q_{\text{ave, tip}}(D, L) = \frac{q_{\text{ave, tip}}(D, L) + q_{\text{ave, tip}}(D, L)}{2} \]
\[ q_{\text{ave, tip1}}(D, L) = 84.414 \]
\[ q_{\text{ave, tip2}}(D, L) = 147.483 \]
\[ q_{\text{ave, tip3}}(D, L) = 113.949 \]

\[ N_k = 1.5 \]

Cone factor: 15 for general clays, 10 for sensitive clays

\[ k_2 = \frac{N_k}{9} \]

\[ \sigma_{V0, \text{tip}} = \frac{9.5 \cdot h + 11.5 \left( \frac{L}{r} - h \right)}{2000} \]
\[ \sigma_{V0, \text{tip}} = 2.739 \]

\[ q_t(L) = \frac{\left( q_{\text{ave, tip1}}(D, L) - \sigma_{V0, \text{tip}} \right)}{k_2} \leq 150, \frac{\left( q_{\text{ave, tip2}}(D, L) - \sigma_{V0, \text{tip}} \right)}{k_2} \leq 150 \]

\[ q_t(L) = \frac{1}{\text{ton}} \cdot \frac{1}{\text{ft}^2} \]

\[ q_t(L) = 67.926 \cdot \frac{1}{\text{ton}} \cdot \frac{1}{\text{ft}^2} \]

Tip resistance:

\[ A = D^2 \quad A = 2.25 \text{ ft}^2 \]

\[ Q_t(L) = A \cdot q_t(L) \quad Q_t(L) = 132.833 \text{ ton} \]

Side Friction:

Calculate Ultimate Side Friction: \( Q_s \)

\[ L_1 = 0.01 \text{ ft} \] Input the soil layer

\[ L_2 = 21.8 \text{ ft} \]

\[ \sigma_{V0} = 0.5 \]

\[ \sigma_{V0, \text{ef}} = 0.293 \]

\[ f_2(L_1, L_2, \sigma_{V0}, \sigma_{V0, \text{ef}}) = \min \left[ \left( q_{\text{ave, side}}(L_1, L_2) - \sigma_{V0} \leq 0, \frac{\left( q_{\text{ave, side}}(L_1, L_2) - \sigma_{V0} \right)}{10.5 + 13.3 \log \left( \frac{q_{\text{ave, side}}(L_1, L_2) - \sigma_{V0}}{\sigma_{V0, \text{ef}}} \right)} \right)_{1.2} \right] \text{ton} \cdot \frac{1}{\text{ft}^2} \]
\[ Q_s = \left[ 1, L_2, \sigma_0, \sigma_{0-\text{ef}} \right] = f_s \left[ 1, L_2, \sigma_0, \sigma_{0-\text{ef}} \right] \cdot 4 \cdot \left[ L_2 - L_1 \right] \]

\[ Q_s = 156,888 \text{ ton} \]

\[ \sigma_{\text{ps}} \text{ is equal to } 1.25 \text{ for prestressed concrete driven piles} \]

\[ L_0 = 0.018 \quad L_1 = 4 \text{ft} \quad L_2 = 5.8 \text{ft} \quad L_3 = 27.9 \text{ft} \quad L_4 = 26.5 \text{ft} \quad L_5 = L \]

Layer 1:  \[ Q_{s1} = Q_s \left[ L_0, L_1, \text{overburden}(L_0, L_1)_{1,1}, \text{overburden}(L_0, L_1)_{1,2} \right] \]
\[ Q_{s1} = 28,728 \text{ ton} \]

Layer 2:  \[ Q_{s2} = Q_s \left[ L_1, L_2, \text{overburden}(L_1, L_2)_{1,1}, \text{overburden}(L_1, L_2)_{1,2} \right] \]
\[ Q_{s2} = 9,972 \text{ ton} \]

Layer 3:  \[ Q_{s3} = Q_s \left[ L_2, L_3, \text{overburden}(L_2, L_3)_{1,1}, \text{overburden}(L_2, L_3)_{1,2} \right] \]
\[ Q_{s3} = 159,12 \text{ ton} \]

Layer 4:  \[ Q_{s4} = Q_s \left[ L_3, L_4, \text{overburden}(L_3, L_4)_{1,1}, \text{overburden}(L_3, L_4)_{1,2} \right] \]
\[ Q_{s4} = 10,05 \text{ ton} \]

Layer 5:  \[ Q_{s5} = Q_s \left[ L_4, L_5, \text{overburden}(L_4, L_5)_{1,1}, \text{overburden}(L_4, L_5)_{1,2} \right] \]
\[ Q_{s5} = 36,4 \text{ ton} \]

\[ Q_s = Q_{s1} + Q_{s2} + Q_{s3} + Q_{s4} + Q_{s5} \]
\[ Q_s = 294,27 \text{ ton} \]

\[ Q_{\text{Dawson}} = \frac{Q_s + Q(\text{D})}{2} \]
\[ Q_{\text{Dawson}} = 345,214 \text{ ton} \]

**UWA-05 Method (2005)**

\[ \text{exchange} = 0.1640/\text{Q}_s \]
\[ \text{ORIGIN} = 1 \]

Ultimate Pile Capacity Based on CPT data UWA-05 method (2005)

Input Data:

**CPT:**

- DATA input
- Diameter or Edge Length of the Pile
- Length of the Pile
- water table

**D:** 18\text{in}

**L:** 48.5\text{ft}

**WT:** 5
Calculation of the average cone tip resistance using UWA-05 method (2005)

Average $q_c$ over a distance of $yD$ below the pile tip: $q_c \text{ ave}_{\text{below}}(CPT, D, L, y)$

$$q_{\text{ave}_{\text{below}}}(CPT, D, L, y) = \left\lfloor \frac{L}{\text{exchange}} \right\rfloor$$

$i \leftarrow 0$
$q_{\text{below} \_1} \leftarrow 0$
while $CPT_{n+i,1} \leq \frac{L + yD}{ft}$

$q_{\text{below} \_1} \leftarrow q_{\text{below} \_1} + CPT_{n+i,2}$
$i \leftarrow i + 1$

end while

$i \leftarrow i - 1$
$\text{counter} \leftarrow i$
$q_{\text{below} \_2} \leftarrow 0$
$q_{\text{below} \_\text{min}} \leftarrow CPT_{n+i,2}$
while $i \geq 0$

if $q_{\text{below} \_\text{min}} \leq CPT_{n+i,2}$

$q_{\text{below} \_2} \leftarrow q_{\text{below} \_2} + q_{\text{below} \_\text{min}}$
$i \leftarrow i - 1$

end if

otherwise

$q_{\text{below} \_\text{min}} \leftarrow CPT_{n+i,2}$
$q_{\text{below} \_2} \leftarrow q_{\text{below} \_2} + q_{\text{below} \_\text{min}}$
$i \leftarrow i - 1$

end if

$q_{\text{ave}_{\text{below}}} \leftarrow \frac{q_{\text{below} \_1} + q_{\text{below} \_2}}{2(\text{counter} + 1)}$

return $q_{\text{ave}_{\text{below}}}$

$q_{c1}(L) =$

$yy \leftarrow 0$
$q_{c1} \leftarrow q_{\text{ave}_{\text{below}}}(CPT, D, L, 0.7)$
for $y = 0.7, 0.71, \ldots 4$

if $q_{c1} \geq q_{\text{ave}_{\text{below}}}(CPT, D, L, y)$

$q_{c1} \leftarrow q_{\text{ave}_{\text{below}}}(CPT, D, L, y)$
$yy \leftarrow y$

end if

return $(yy)$

$q_{c1}(L) = \left( \begin{array}{c} 12.1 \ 2.38 \\ 2.75 \end{array} \right)$
Calculation of the average cone tip resistance in UWA-05 method:

\[ q_{21}(L) = 121.258 \]
\[ q_{22}(L) = 57.893 \]

\[ q(L) = 0.6 \cdot \frac{q_{11}(L) + q_{22}(L)}{2} \tan \frac{\theta}{\pi} \]
\[ q(L) = 56.745 \cdot \tan \frac{\theta}{\pi} \]
Tip resistance:

\[ A = D^2 \quad A = 2.25\text{ ft}^2 \]

\[ Q_T(L) = A \cdot q_L(L) \quad Q_T(L) = 127,677\text{ ton} \]

Side Friction (UWA-05 (2005)):

\[ \sigma_{\theta,ef}(\text{Depth}) = \begin{cases} \frac{95 \cdot \text{Depth}}{2000}, & \text{if Depth} \leq \text{WT} \\ \frac{95 \cdot \text{WT} + (115 - 62.4)(\text{Depth} - \text{WT})}{2000}, & \text{otherwise} \end{cases} \]

\[ \sigma_{\theta,ef}(\text{CPT}_{74,1}) = 0.421 \]

\[ \text{int}(\text{Depth}) = \begin{cases} \text{out} & \text{if ceil}(\text{Depth}) = \text{Depth} \leq 0.5 \\ \text{out} & \text{floor}(\text{Depth}) \text{ otherwise} \end{cases} \]

\[ q_c(\text{Depth}) = \text{CPT} \left( \frac{\text{Depth} \cdot \text{ft}}{\text{exch}} \right)^{1.2} \]

\[ q_c(\text{CPT}_{74,1}) = 61.17 \]

\[ G(\text{Depth}) = \begin{cases} q_c(\text{Depth}) & q_c(\text{Depth}) \neq 0, q_c(\text{Depth}) \\ 1.85 \cdot \frac{q_c(\text{Depth})}{1.081} \left( \frac{\sigma_{\theta,ef}(\text{Depth})}{1.081} \right)^{-0.7} & \text{otherwise} \end{cases} \]

Calculate Ultimate Side Friction: \( Q_s \)

\[ Q_s := \begin{cases} n & \text{ceil} \left( \frac{L}{\text{exch}} \right) - 1 \\ \text{Q}_s & = 0 \\ \text{for } i = 2 \ldots n \end{cases} \]

\[ q_c \leftarrow \text{CPT}_{i,2} \]

\[ LL \leftarrow \text{CPT}_{i,1} \]

\[ f_s \leftarrow 0.03 \cdot q_c \left( \max \left( \frac{L}{\text{ft}} - LL, 2 \right) \right)^{-0.5} + 4 \cdot G(\text{LL}) \cdot \frac{A_T}{D} \cdot \tan(\theta_f) \]

\[ Q_s \leftarrow Q_s + f_s \cdot 4 \cdot \text{D} \cdot \text{exchange} \]

return \( \frac{Q_s}{\text{ton}} \)

\[ Q_{\text{Davisson}} := Q_s + \frac{Q_T(L)}{3} \quad Q_{\text{Davisson}} = 142,454\text{ ton} \]

\[ \text{exchange} = 0.164042 \]


\[ \text{ORIGIN} = 1 \]

Input Data:

- CPT := [DATA input]
- \( D = 13 \text{in} \)
- \( L = 48.5 \text{ft} \)

Embedment of the Pile

Diameter or Edge Length of the Pile


\[
q_{\text{ave, side}}(L1, L2) = \begin{cases} 
\lfloor \text{cell} \left( \frac{L1}{\text{exchange}} \right) \rfloor \\
i = 0 \\
k = 0 \\
q_c = 0 \\
\text{while } CPT_{n+i,1} \leq \frac{L2}{\text{ft}} \\
q_c \leftarrow q_c + CPT_{n+i,2} \\
i \leftarrow i + 1 \\
q_{\text{ave}} \leftarrow \frac{q_c}{i} \\
\text{return } q_{\text{ave}} 
\end{cases}
\]

\[
f_{\text{ave, side}}(L1, L2) = \begin{cases} 
\lfloor \text{cell} \left( \frac{L1}{\text{exchange}} \right) \rfloor \\
i = 0 \\
k = 0 \\
f_0 = 0 \\
\text{while } CPT_{n+i,1} \leq \frac{L2}{\text{ft}} \\
f_2 \leftarrow f_2 + CPT_{n+i,2} \\
i \leftarrow i + 1 \\
f_{\text{ave}} \leftarrow \frac{f_2}{i} \\
\text{return } f_{\text{ave}} 
\end{cases}
\]
Average tip resistance within 4D above the pile tip

\[ q_{\text{ave tip}}(D, L) = q_{\text{ave side}}(L - 4D, L) \]

Average tip resistance within 4D below the pile tip

\[ q_{\text{ave tip}}(D, L) = q_{\text{ave side}}(L + 4D, L) \]

\[ q_{\text{ave tip}}(D, L) = \begin{cases} 
q_{\text{ave tip}}(D, L) \leq q_{\text{ave tip 2}}(D, L), & \frac{q_{\text{ave tip 1}}(D, L) + q_{\text{ave tip 2}}(D, L)}{2}, \\
q_{\text{ave tip 2}}(D, L) & \end{cases} \]

\[ q_{\text{ave tip 1}}(D, L) = 110.233 \]

\[ q_{\text{ave tip 2}}(D, L) = 141.022 \]

\[ q_{\text{ave tip 3}}(D, L) = 125.651 \]

\[ f_{\text{ave tip}}(D, L) = f_{\text{ave side}}(L - 4D, L + 4D) \]

\[ f_{\text{ave tip}}(D, L) = 0.897 \]

\[ \frac{f_{\text{ave tip}}(D, L)}{f_{\text{ave tip}}(D, L)} = 7.141 \times 10^{-3} \]

\[ \omega = \begin{cases} 
q_{\text{ave tip}}(D, L) \geq 2 \times 10.4427 & \leq \frac{f_{\text{ave tip}}(D, L)}{q_{\text{ave tip 1}}(D, L)} \leq 0.014, 0.71 \quad \left( \frac{q_{\text{ave tip 1}}(D, L)}{10.4427} \right)^{-0.25} \quad 1.07 \left( \frac{q_{\text{ave tip 1}}(D, L)}{10.4427} \right)^{-0.25} \\
\end{cases} \]

\[ q_t(L) = \omega \cdot q_{\text{ave tip}}(D, L) \quad \frac{\text{ton}}{\text{ft}^2} \]

\[ q_t(L) = 47.324 \quad \frac{\text{ton}}{\text{ft}^2} \]

Tip resistance:

\[ A = D^2 \quad A = 2.25 \text{ ft}^2 \]

\[ Q_t(L) = A \cdot q_t(L) \quad Q_t(L) = 107.762 \text{ ton} \]

Side Friction (Zhou et al. Method):

\[ \beta(n) = \begin{cases} 
CPT_{n, 3} \leq 0, 0, \max \left[ \begin{array}{c} CPT_{n, 2} \geq 2 \times 10.4427 \wedge \frac{CPT_{n, 3}}{CPT_{n, 2}} \leq 0.014, 0.23 \quad \left( \frac{CPT_{n, 3}}{10.4427} \right)^{-0.45} \quad 0.22 \left( \frac{CPT_{n, 3}}{10.4427} \right)^{-0.55} \end{array} \right] \end{cases} \]

Calculate Ultimate Side Friction \( Q_s \)

\[ Q_s := \begin{cases} 
n \leftarrow 2 \\
Q_s \leftarrow 0 \text{ ton} \\
A_s \leftarrow 4D \text{ exchange} \\
i \leftarrow 0 \\
\text{while } i \text{ exchange } \leq L \end{cases} \]

\[ Q_s := Q_s + \beta(n + i) \frac{\text{ton}}{\text{ft}^2} \cdot \frac{\text{CPT}_{n+1, 3} \cdot A_s}{n^2} \]

\[ i \leftarrow i + 1 \]

return \( Q_s \)

\[ Q_{\text{Devission}} = Q_s + Q_t(L) \quad Q_{\text{Devission}} = 223.356 \text{ ton} \]
Calculating LRFD Resistance Factors

LRFD Phi factors
(First Order Second Moment Approach)

Input Data:
\[ \gamma_D := 1.25 \quad \gamma_L := 1.75 \quad Q_D := 2 \quad Q_L := 1 \quad \frac{Q_D}{Q_L} = 2 \]

\[ \lambda_{QD} := 1.08 \quad \lambda_{QL} := 1.15 \quad \text{COV}_{QD} := 0.128 \quad \text{COV}_{QL} := 0.18 \]

\[ \beta_T := 2.5 \]

\[ n := 28 \]

\[ \begin{array}{|c|c|c|}
\hline
& 1 & 2 \\
\hline
1 & 195 & 139 \\
2 & 104 & 55 \\
3 & 151.5 & 125 \\
4 & 135 & 105 \\
5 & 113 & 118 \\
6 & 61 & 115 \\
7 & 140 & 105 \\
8 & 104 & 165 \\
9 & 103 & 90 \\
10 & 103 & 90 \\
11 & 111 & 82 \\
12 & 127 & 95 \\
13 & 108 & 70 \\
14 & 122 & 85 \\
15 & 143 & 80 \\
16 & 167 & 140 \\
\hline
\end{array} \]

\[ \lambda_R := \text{for } i \in 1..n \]
\[ \lambda_i := \frac{\text{DATA}_{i,2}}{\text{DATA}_{i,1}} \]
\[ \lambda_R := \text{mean}(\lambda) \]
return \( \lambda_R \)

\[ \lambda_R = 0.338 \]
\[
\Phi := \begin{cases}
\text{for } i \in 1..n \\
\lambda_i &\leftarrow \frac{\text{DATA}_{i,2}}{\text{DATA}_{i,1}} \\
\lambda_R &\leftarrow \text{mean}(\lambda) \\
\text{COV}_R &\leftarrow \sqrt{\frac{n}{n-1} \cdot \text{var}(\lambda)} \\
\end{cases}
\]

\[
\phi = \lambda_R \left( \frac{Q_D^2}{Q_L} + \gamma_L \right) \cdot \frac{\text{Q}_D^2 - \lambda_{Q_D}^2 \cdot \text{COV}_{QD}^2 + \lambda_{Q_L}^2 \cdot \text{COV}_{QL}^2}{1 + \frac{Q_D^2}{Q_L} - \lambda_{Q_D}^2 + 2 \frac{Q_D}{Q_L} \cdot \lambda_{Q_D} \cdot \lambda_{Q_L} + \lambda_{QL}^2}
\]

\[
\beta_T := \ln \left( 1 + \text{COV}_R \right)^2 \cdot \frac{\left( \frac{Q_D^2}{Q_L} - \lambda_{Q_D}^2 \cdot \text{COV}_{QD}^2 + \lambda_{Q_L}^2 \cdot \text{COV}_{QL}^2 \right)}{1 + \frac{Q_D^2}{Q_L} - \lambda_{Q_D}^2 + 2 \frac{Q_D}{Q_L} \cdot \lambda_{Q_D} \cdot \lambda_{Q_L} + \lambda_{QL}^2}
\]

\[
\Phi_{AR} = \frac{\Phi}{\lambda_R}
\]

\[
\text{COV}_R := \begin{cases}
\text{for } i \in 1..n \\
\lambda_i &\leftarrow \frac{\text{DATA}_{i,2}}{\text{DATA}_{i,1}} \\
\lambda_R &\leftarrow \text{mean}(\lambda) \\
\text{COV}_R &\leftarrow \sqrt{\frac{n}{n-1} \cdot \text{var}(\lambda)} \\
\end{cases}
\]

\[
\Phi = 0.452 \quad \text{COV}_R = 0.32 \quad \Phi_{AR} = 0.539
\]
Bootstrap Analysis

Schmertmann Method (Florida Soil)

\[
\text{ORIGIN} = 1
\]

\[
\text{Data} := \begin{array}{cccccccccccccccccccc}
& 1 & 2 & 3 & 4 & 5 & 6 & 7 & 8 & 9 & 10 & 11 & 12 & 13 & 14 & 15 & 16 \\
1 & 1.25 & 0.737 & 0.475 & 1.121 & 1.602 & 1.039 & 1.252 & 1.163 & 0.673 & 1.54 & 1.183 & 1.431 & 2.095 & 1.157 & 1.383 & 1.441 \\
2 & 1 & 0.2 & 0.4 & 0.6 & 0.8 & 1 & 1.2 & 1.4 & 1.6 & 1.8 & 2 & 2.2 & 2.4 & 2.6 & 2.8 & 3 \\
\end{array}
\]

\[
m := \text{rows(Data)} \quad m = 21 \quad \text{mean(Data)} = 1.327
\]

\[
\min(\text{Data}) = 0.475 \quad \text{stddev(Data)} \frac{\sqrt{m}}{\sqrt{m-1}} = 0.693
\]

\[
\max(\text{Data}) = 3.93
\]

\[
a := 0 \quad b := 4
\]

\[
n := 20
\]

\[
j = 1 \quad n + 1
\]

\[
h := \frac{b - a}{n}
\]

\[
\text{int}._j := a + (j - 1) \cdot h
\]

\[
f := \frac{\text{hist(int}._\text{Data})}{m \cdot h}
\]
Mean = \text{cei}([0, 1.0])
for i = 1 : 10000
    for j = 1 : m
        RD_{ij} \leftarrow \text{Index} \cdot (i - 1) + j
    end
end
Mean \leftarrow \text{mean}(RD)
return Mean

Stddev = \text{cei}([0, 1.0])
for i = 1 : 10000
    for j = 1 : m
        RD_{ij} \leftarrow \text{Index} \cdot (i - 1) + j
    end
end
Stddev \leftarrow \text{stddev}(RD) \cdot \frac{m}{\sqrt{m - 1}}
return Stddev

<table>
<thead>
<tr>
<th>i</th>
<th>Mean</th>
<th>Stddev</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.372</td>
<td>1.637</td>
</tr>
<tr>
<td>2</td>
<td>1.661</td>
<td>1.311</td>
</tr>
<tr>
<td>3</td>
<td>1.487</td>
<td>0.946</td>
</tr>
<tr>
<td>4</td>
<td>1.923</td>
<td>0.896</td>
</tr>
<tr>
<td>5</td>
<td>1.399</td>
<td>1.032</td>
</tr>
<tr>
<td>6</td>
<td>1.412</td>
<td>0.396</td>
</tr>
<tr>
<td>7</td>
<td>1.457</td>
<td>0.921</td>
</tr>
<tr>
<td>8</td>
<td>1.173</td>
<td>1.157</td>
</tr>
<tr>
<td>9</td>
<td>1.057</td>
<td>0.798</td>
</tr>
<tr>
<td>10</td>
<td>1.195</td>
<td>0.418</td>
</tr>
<tr>
<td>11</td>
<td>1.597</td>
<td>0.85</td>
</tr>
<tr>
<td>12</td>
<td>1.125</td>
<td>0.96</td>
</tr>
<tr>
<td>13</td>
<td>1.314</td>
<td>0.216</td>
</tr>
<tr>
<td>14</td>
<td>1.153</td>
<td>0.285</td>
</tr>
<tr>
<td>15</td>
<td>1.143</td>
<td>0.714</td>
</tr>
<tr>
<td>16</td>
<td>1.413</td>
<td>0.745</td>
</tr>
</tbody>
</table>
\[ \text{True mean} := \text{mean(Mean)} \quad \text{True mean} = 1.328 \quad \text{True mean} := \text{mean(Stdev)} \quad \text{True Stdev} = 0.631 \]

\[ \text{Stdev mean} := \text{stdev(Mean)} \quad \text{Stdev mean} = 0.143 \quad \text{Stdev stdev} := \text{stdev(Stdev)} \quad \text{Stdev stdev} = 0.263 \]

\[ m := \text{rows(Mean)} \]

\[ \text{Mean Analysis:} \]

\[ \text{min(Mean)} = 0.885 \]

\[ \text{max(Mean)} = 2.206 \]

\[ a := \text{min(Mean)} \]

\[ b := \text{max(Mean)} \]

\[ n := 30 \]

\[ j := 1..n + 1 \]

\[ h := \frac{b - a}{n} \]

\[ \text{int}_j := a + (j - 1) \cdot h \]

\[ f := \frac{\text{hist(int}_j, \text{Mean})}{m \cdot h} \]

\[ c := \text{max}(f) \quad c = 2.704 \]
Standard Deviation Analysis:

\[
\begin{align*}
\min(\text{Stddev}) &= 0.129 \\
\max(\text{Stddev}) &= 1.428 \\
a &= \min(\text{Stddev}) \\
b &= \max(\text{Stddev}) \\
n &= 30 \\
j &= 1..n + 1 \\
h &= \frac{b - a}{n} \\
\text{int.}j &= a + (j - 1) \cdot h \\
f &= \frac{\text{hist(int.}j, \text{Stddev})}{m \cdot h} \\
c &= \max(f) \quad c = 3.885
\end{align*}
\]

Mean = 0.631  
Stdev = 0.243
The Proposed Method (Florida Soil)

\[
\text{Origin} \equiv 1
\]

\[
\text{Data} := \begin{bmatrix}
\end{bmatrix}
\]

\[
\text{mean(Data)} = 1.079
\]

\[
\text{stdDev(Data)} \sqrt{\frac{m}{m-1}} - 0.288
\]

\[
\text{min(Data)} = 0.613
\]

\[
\text{max(Data)} = 1.701
\]

\[
a := 0.6
\]

\[
b := 1.8
\]

\[
n := 8
\]

\[
j := 1..n + 1
\]

\[
h := \frac{b - a}{n}
\]

\[
\text{int}_j = a + (j - 1) \cdot h
\]

\[
\text{Frequency} = \frac{\text{hist(int, Data)}}{m \cdot h}
\]

<table>
<thead>
<tr>
<th>1</th>
<th>0.921</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.971</td>
</tr>
<tr>
<td>3</td>
<td>0.72</td>
</tr>
<tr>
<td>4</td>
<td>0.94</td>
</tr>
<tr>
<td>5</td>
<td>1.182</td>
</tr>
<tr>
<td>6</td>
<td>0.869</td>
</tr>
<tr>
<td>7</td>
<td>1.01</td>
</tr>
<tr>
<td>8</td>
<td>0.791</td>
</tr>
<tr>
<td>9</td>
<td>0.68</td>
</tr>
<tr>
<td>10</td>
<td>1.277</td>
</tr>
<tr>
<td>11</td>
<td>1.281</td>
</tr>
<tr>
<td>12</td>
<td>1.055</td>
</tr>
<tr>
<td>13</td>
<td>1.584</td>
</tr>
<tr>
<td>14</td>
<td>0.954</td>
</tr>
<tr>
<td>15</td>
<td>1.195</td>
</tr>
<tr>
<td>16</td>
<td>1.066</td>
</tr>
</tbody>
</table>
\textbf{Mean} :=
\begin{align*}
\text{index} &\leftarrow \text{ceil}(\text{rand}(100000, 0, m)) \\
\text{for } i = 1 \text{ to } 100000 \\
\text{for } j = 1 \text{ to } m \\
\text{rd}_{j} &\leftarrow \text{data}_{\text{index}(i-1)+j} \\
\text{mean}_{i} &\leftarrow \text{mean(rd)} \\
\text{return mean}_{i}
\end{align*}

\textbf{Stddev} :=
\begin{align*}
\text{index} &\leftarrow \text{ceil}(\text{rand}(100000, 0, m)) \\
\text{for } i = 1 \text{ to } 100000 \\
\text{for } j = 1 \text{ to } m \\
\text{rd}_{j} &\leftarrow \text{data}_{\text{index}(i-1)+j} \\
\text{stddev}_{i} &\leftarrow \text{stddev} (\text{rd}) \cdot \sqrt{\frac{m}{m-1}} \\
\text{return stddev}_{i}
\end{align*}

\begin{tabular}{|c|c|}
\hline
\textbf{Mean} & \textbf{Stddev} \\
\hline
1 & 1.112 \\
2 & 1.212 \\
3 & 1.109 \\
4 & 1.249 \\
5 & 1.084 \\
6 & 1.151 \\
7 & 1.102 \\
8 & 1.069 \\
9 & 0.938 \\
10 & 1.044 \\
11 & 1.173 \\
12 & 1.089 \\
13 & 1.125 \\
14 & 1.024 \\
15 & 1.032 \\
16 & 1.14 \\
\hline
\end{tabular}
\[ \text{TrueMean} = \text{mean}(	ext{Mean}) \quad \text{TrueMean} = 1.079 \]

\[ \text{StdevMean} = \text{std}(	ext{Mean}) \quad \text{StdevMean} = 0.061 \]

\[ \text{TrueStdev} = \text{mean}(\text{Stdev}) \quad \text{mean}(\text{Stdev}) = 0.278 \]

\[ \text{StdevStdev} = \text{std}(	ext{Stdev}) \quad \text{StdevStdev} = 0.042 \]

\textbf{Mean Analysis:}

\[ \text{max}(\text{Mean}) = 0.822 \]
\[ \text{max}(\text{Mean}) = 1.369 \]
\( a := \text{min}(\text{Mean}) \)
\( b := \text{max}(\text{Mean}) \)
\( n = 30 \)
\( n := \text{rows}(\text{Mean}) \quad m = 1 \times 10^5 \)
\( j = 1 \ldots n + 1 \)
\( h = \frac{b - a}{n} \)
\( \text{int}_j := a + (j - 1) \cdot h \)

\[ \text{Frequency} := \frac{\text{hist}(\text{int}, \text{Mean})}{m \cdot h} \]
\( c := \text{max}(\text{Frequency}) \quad c = 6.328 \)
Standard Deviation Analysis:

\[
\min(\text{Stddev}) = 0.105 \\
\max(\text{Stddev}) = 0.435 \\
a := \min(\text{Stddev}) \\
b := \max(\text{Stddev}) \\
n := 30 \\
j := 1 \ldots n + 1 \\
h := \frac{b - a}{n} \\
\text{int}_j := a + (j - 1) \cdot h \\
f_j := \frac{\text{hist}(\text{int}_j, \text{Stddev})}{m \cdot h} \\
c := \max(f) \quad c = 9.481
\]

\[
\text{Mean} = 0.278 \\
\text{Stdev} = 0.042
\]
LIST OF REFERENCES


BIOGRAPHICAL SKETCH

Zhihong Hu was born in Kaifeng, Henan Province, China. He spent his childhood in that small, beautiful city and finished his primary school and middle school there. He moved to Zhengzhou with his parents and studied his high school. He was accepted by Civil and Architectural Engineering, Shanghai Jiaotong University in 1996 and spent 4 years in this university. He got his two bachelor’s degrees (civil engineering and applied electrical engineering) in September 2000. He found a job in a construction company and gained his first working experience there. After some time in the work, he realized that his knowledge was far from enough to deal with the real work problem. So he decided to go abroad to get advanced education. He was accepted by the Department of Civil and Coastal Engineering in the University of Florida and went to the US in January 2002. He had studied as well as being doing centrifuge research supervised by Dr. Michael McVay for two years. The study and research strengthened his background in geotechnical engineering and made him decide to devote the rest of his life in this field. He got his master’s degree in December 2003 and continued his Ph.D. in this school in the following 4 years. During these 4 years, he not only finished another FDOT project and his Ph.D. degree, but also met his wife, Meiyu, and got married in May 2006.