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Calibration of Resistance Factors Needed in the LRFD Design of Driven Piles

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ABSTRACT

This research project presents the calibration of resistance factors for the Load and Resistance Factor Design (LRFD) method of driven piles driven into Louisiana soils based on reliability theory. Fifty-three square Precast-Prestressed-Concrete (PPC) piles that were tested to failure were included in this investigation. The predictions of pile resistances were based on static analysis (α -method for clay and Nordlund method for sand), three direct CPT methods [Schmertmann method, De Ruiter and Beringen method, and Bustamante and Ganeselli (LCPC) method], and the average of the three CPT methods. Also, dynamic measurements with signal matching analysis of pile resistances using the Case Pile Wave Analysis Program (CAPWAP), which is based on the measured force and velocity signals obtained near the pile top during driving, were calibrated. The Davisson and modified Davisson interpretation methods were used to determine measured ultimate load carrying resistances from pile load tests. The predicted ultimate pile resistances obtained using the different prediction methods were compared with measured resistances determined from pile load tests. Statistical analyses were carried out to evaluate the capability of the prediction design methods to estimate measured ultimate pile resistance of driven piles. The results showed that the static method over-predicts pile resistance, while the dynamic measurement with signal matching analysis [CAPWAP-EOD (end of drive) and 14 days BOR (beginning of restrike)] under-predicts pile resistance. Among the three direct CPT methods, the De Ruiter and Beringen method was the most consistent prediction method with the lowest COV. Reliability based analyses using the First Order Second Moment (FOSM) method, the First Order Reliability Method (FORM), and the Monte Carlo (MC) simulation method were also conducted to calibrate the resistance factors (ϕ) for the investigated pile design methods. The resistance factors with the target reliability (β_T) of 2.33 for the different design methods were determined and compared with American Association of State Highway and Transportation Officials (AASHTO) recommendation values. In addition, the evaluation of different design methods was performed.

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IMPLEMENTATION STATEMENT

The Federal Highway Administration and AASHTO set a transition date of October 1, 2007 after which all new federal-funded bridges shall be designed using the LRFD design method. The LRFD Code is the first attempt by AASHTO to establish a National Bridge Code addressing geotechnical design for bridges. This fact calls for the establishment of a local calibration that takes into account the geology of Louisiana. This research project aims at calibrating resistance factors associated with different pile design methods used by engineers designing bridges for LADOTD. Reliability indexes and corresponding resistance factors for different pile design methods are recommended and ready for immediate implementation. Engineers designing bridges for Louisiana should use the recommended resistance factors in lieu of factors shown in Section 10 of the LRFD Code. As experience is gained in the application of LRFD to design, the role of past successful allowable stress design (ASD) practices will become less important, and the advantage of the LRFD design can be fully addressed by achieving consistent levels of reliability in the design of both superstructures and foundations. In addition, calibration efforts in this research are documented so that the calibration results become a heritage for future users of LADOTD and thereby enhance future LRFD research and development. One project will be selected to perform a cost/benefit analysis by comparing the design results obtained by using the related indexes and resistance factors obtained in this research and those recommended by the AASHTO LRFD Bridge Design Specifications (2007) and ASD Design.

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INTRODUCTION

The allowable stress design (ASD) method had been used in the design of bridges, which involves applying a factor of safety (FS). FS, defined as the fraction of the ultimate geotechnical pile resistance, that is traditionally known as pile capacity, over the allowable design load, accounts for uncertainties in the applied loads and soil resistance. The magnitude of FS has been empirically developed over time depending on the importance of the structure, the confidence levels of material properties, and the design methodology. However this approach suffers from the disadvantage that the uncertainties of the different sources of load and resistance are combined together in a single factor. Consequently, ASD may lead to a design with a different level of safety for similar structures. The LRFD method aims at separating uncertainties of load effect from uncertainties of resistance and then applying statistical theory to ensure the same level of structure safety.

The Bridge Design Specifications published by the American Association of Highway and Transportation Officials (AASHTO) has introduced the LRFD method to account for uncertainties associated with estimated loads and resistances [1], [2]. Since then, bridge superstructures have been designed using the LRFD method in most U.S. states. However, the LRFD method for design of bridge foundation is progressively gaining its prevalence; while the ASD method is still used for the bridge foundation design in practice. This can lead to inconsistent levels of reliability between superstructures and substructures. In an effort to maintain a consistent level of reliability, the Federal Highway Administration (FHWA) and AASHTO a mandated a date of October 1, 2007 after which all federal-funded new bridges including substructures shall be designed using the LRFD method. Accordingly, significant research efforts have been directed to implement the LRFD design methodology in bridge substructure and establish and to calibrate the proper resistance factors, which will be explained in greater detail in a later section, for local soil conditions in compliance with this mandate [3], [4], [5], [6], [7], [8], [9], [10].

The current AASHTO design specification recommends resistance factors, ϕ , for single driven piles in axial compression ranging from 0.10 to 0.65, depending on the design method [11]. However, the existing resistance factors are recommended based on a pile load test—a soil database that was collected from sites that do not necessary reflect Louisiana soils or design practice. For example, the driven pile database used in the existing AASHTO design specification is based on the data gathered by Florida DOT and FHWA [12], [13]. Therefore, the resistance factors recommended by the existing AASHTO Code need to be verified before being applied to local soil condition and design practice. Direct application of the

AASHTO resistance factors without calibration may result in over-conservative or unsafe design. When local experience and databases are available, the AASHTO recommends calibrating the resistance factor, ϕ , using reliability analyses to produce an overall reliability level that is consistent with local practice.

Several methods have been developed and used in geotechnical engineering practice to estimate ultimate axial bearing resistance of driven piles. This includes static pile load tests, statnamic pile load tests, traditional static analysis, dynamic analysis, and static analysis using the results of in-situ testing such as the Cone Penetration Test (CPT). Static analyses based on soil properties obtained from borings and laboratory tests have been the main method used in practice by LADOTD engineers. However, the application of CPT in predicting ultimate bearing resistance of piles has increased over the last two decades due to the similarity between the cone penetrometer and the pile in which the cone can be considered as a model pile (e.g., [14], [15], [16], and [17]). The CPT is a simple, fast, repeatable, and cost effective in-situ test that can provide continuous subsurface soundings with depth. The measured CPT data (tip resistance, q_c ; sleeve friction, f_s ; and porewater pressure, u) can be effectively utilized for many geotechnical engineering applications including the prediction of ultimate pile resistance. Compared with traditional static design, CPT design methods can provide more economical estimations of the ultimate pile resistance. In addition, the dynamic measurement with signal matching analysis using CAPWAP is also gaining popularity due to its simplicity and economical advantage. Although the dynamic measurement with signal matching analysis cannot be a substitute of pile design analysis, it is mainly used for pile drivability and usually helps in verifying the pile design resistance.

In this research project, design methods used by LADOTD were calibrated based on data collected in Louisiana. For each design method, target reliability values and corresponding resistance factors were developed depending on availability of test data.

OBJECTIVE

The objective of this research project is to calibrate resistance factors for different pile design methods needed in LRFD design of driven piles based on the Louisiana pile load test soil database, and LADOTD experience. The findings of this research effort will help Louisiana geotechnical engineers begin implementing the LRFD design methodology for the design of all driven piles in future Louisiana projects as mandated by AASHTO.

SCOPE

To achieve the objective of this study, reliability based analyses were performed on different design methods used by LADOTD for the estimation of axial load resistance of driven piles in soft Louisiana soils. A database of 53 precast-prestressed-concrete (PPC) piles of different lengths and sizes that were loaded to failure were investigated in this study. Statistical analyses were conducted to evaluate different pile design methods, including the static design method (α -method and Nordlund method), three different direct CPT design methods: [Schmertmann method, De Ruiter and Beringen method, and Bustamante and Ganeselli (LCPC) method], CAPWAP method. The target reliability (β_T) of 2.33 was selected. In addition, reliability analyses based on the FOSM, FORM, and MC simulation method were conducted to calibrate resistance factors (ϕ) for different design methods needed in the LRFD design of single piles.

BACKGROUND

Predication of Ultimate Pile Resistance

The ultimate axial resistance (Q_u) of a driven pile consists of the end bearing resistance (Q_b) and the skin frictional resistance (Q_s). The ultimate pile resistance can be calculated using the following equation:

$$Q_u = Q_b + Q_s = q_b \cdot A_b + \sum_{i=1}^n f_i A_{si} \quad (1)$$

where, q_b is the unit tip bearing resistance, A_b is the cross section area of the pile tip, f_i is the average unit skin friction of the soil layer i , A_{si} is the area of the pile shaft area interfacing with layer i , and n is the number of soil layers along the pile shaft. In clayey stratigraphies, the shaft frictional resistance dominates in many cases, while in sandy stratigraphies, the end bearing resistance can contribute to more than 50 percent of the ultimate pile resistance. Since Louisiana soils are mainly clay and silty clay deposits, the side friction (Q_s) is dominant in the total ultimate bearing resistance (Q_u) compared to the end bearing resistance (Q_b).

Static Methods

α -Tomlinson Method. The α -method is based on total stress analysis. For a soil with an angle of internal friction, $\phi = 0$, or in total stress analysis, the ultimate skin resistance per unit area of the pile can be calculated as follows [18]:

$$f = \alpha S_u \quad (2)$$

where, α is an empirical adhesion coefficient, and S_u is the undrained shear strength. In this research study, α values suggested by Tomlinson are used [18]. Figure 1 shows the α values versus the undrained shear strength (S_u).

The skin friction resistance (Q_s) is as follows:

$$Q_s = \int_0^L f C_d dz \quad (3)$$

where, L is length of the pile in contact with soil, and C_d is the effective perimeter of the pile.

The pile tip resistance (Q_b) is calculated as follows:

$$Q_b = A_b S_u N_c \quad (4)$$

where, A_b is the cross sectional area of the pile, and N_c of 9 is used in this study as recommended by FHWA [19].

Nordlund Method. In sand, the pile tip resistance (Q_b) can be calculated as:

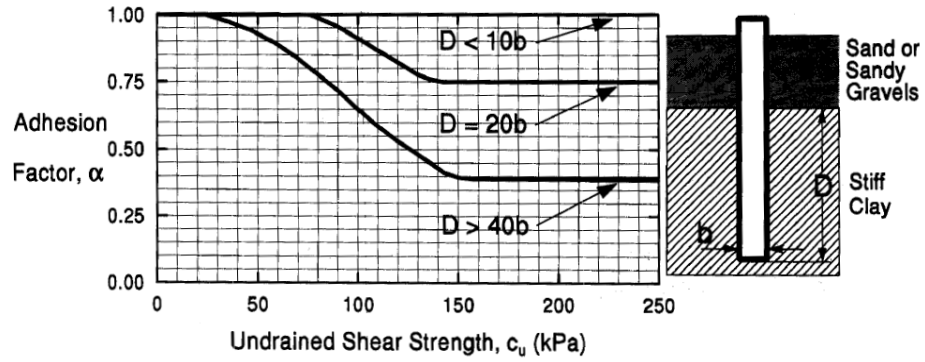
$$Q_b = A_b \cdot \bar{q} \cdot \alpha \cdot N'_q \quad (5)$$

where, \bar{q} is the effective vertical stress at tip level, α is a dimensionless correction factor, and N'_q is a bearing resistance factor varying with ϕ . In this research, the values proposed by Thurman are used for calculation [20]. Figure 2 presents the bearing resistance factor (N'_q) versus the friction angle (ϕ).

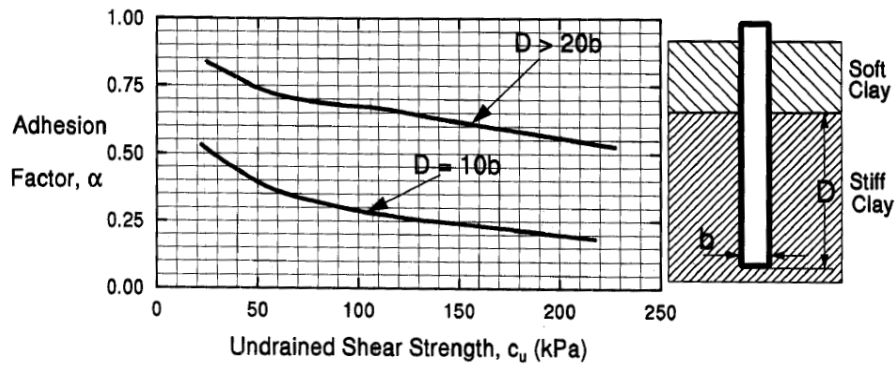
The skin friction resistance (Q_s) was evaluated using the equation proposed by Nordlund in this research as follows [21], [22]:

$$Q_s = \int_0^L K_\delta C_f \bar{P}_D \sin(\delta) C_d dz \quad (6)$$

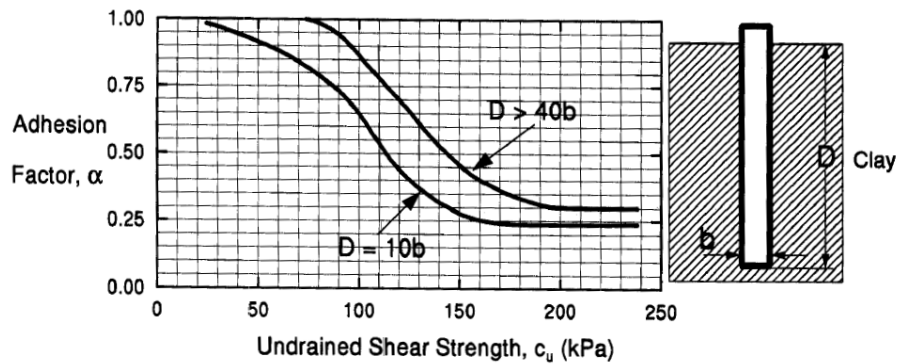
where, K_δ is a coefficient of lateral stress, C_f is a correction factor, \bar{P}_D is the effective overburden pressure, δ is the pile-soil friction angle, and C_d is the effective pile perimeter. Table 1 and Figure 3 present the coefficient of lateral stress (K_δ) and the correction factor (C_f), respectively.



(a) Sand layer over stiff clay layer



(b) Soft clay layer over stiff clay layer



(c) Clay layer only

Figure 1
 α factors for driven piles in clay [19]

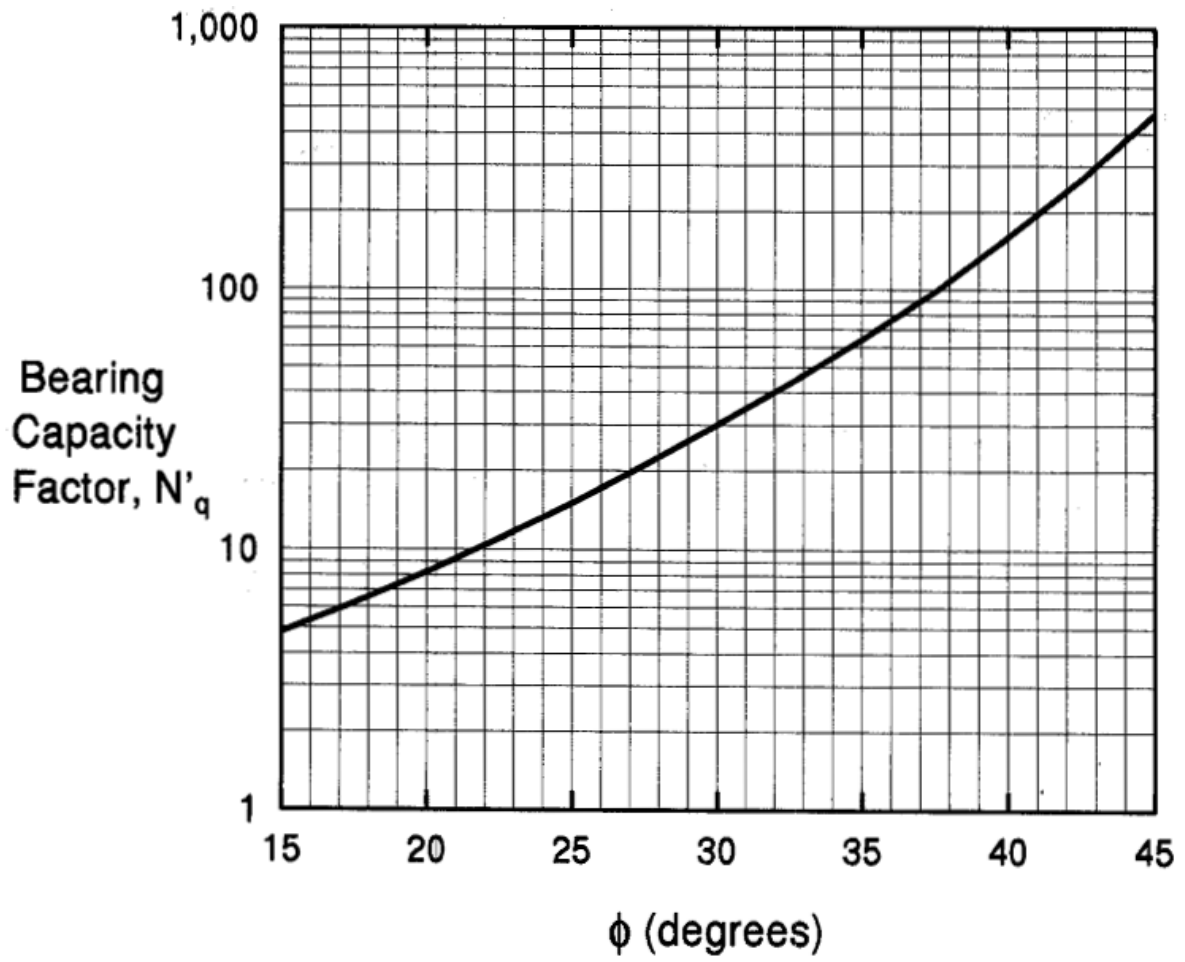


Figure 2
Chart for bearing capacity factor N'_q [19]

Table 1
 K_s for non-tapered piles [19]

ϕ	Volume of soil displaced per unit length (ft ³ /ft)								
	0.1	0.3	0.5	0.7	1.0	3.0	5.0	7.0	10.0
25	0.70	0.77	0.80	0.83	0.85	0.92	0.95	0.98	1.00
26	0.73	0.82	0.86	0.88	0.91	1.00	1.04	1.06	1.09
27	0.76	0.86	0.91	0.94	0.97	1.07	1.12	1.15	1.18
28	0.79	0.90	0.96	0.99	1.03	1.14	1.2	1.23	1.27
29	0.82	0.95	1.01	1.05	1.09	1.22	1.28	1.32	1.36
30	0.85	0.99	1.06	1.10	1.15	1.29	1.36	1.40	1.45
31	0.91	1.08	1.16	1.21	1.27	1.44	1.52	1.57	1.63
32	0.97	1.17	1.26	1.32	1.39	1.59	1.68	1.74	1.81
33	1.03	1.26	1.37	1.44	1.51	1.74	1.85	1.92	1.99
34	1.09	1.35	1.47	1.55	1.63	1.89	2.01	2.09	2.17
35	1.15	1.44	1.57	1.66	1.75	2.04	2.17	2.26	2.35
36	1.26	1.61	1.78	1.89	2.00	2.35	2.52	2.63	2.74
37	1.37	1.79	1.99	2.11	2.25	2.67	2.87	2.99	3.13
38	1.48	1.97	2.19	2.34	2.50	2.99	3.21	3.36	3.52
39	1.59	2.14	2.40	2.57	2.75	3.30	3.56	3.73	3.91
40	1.70	2.32	2.61	2.80	3.00	3.62	3.91	4.10	4.30

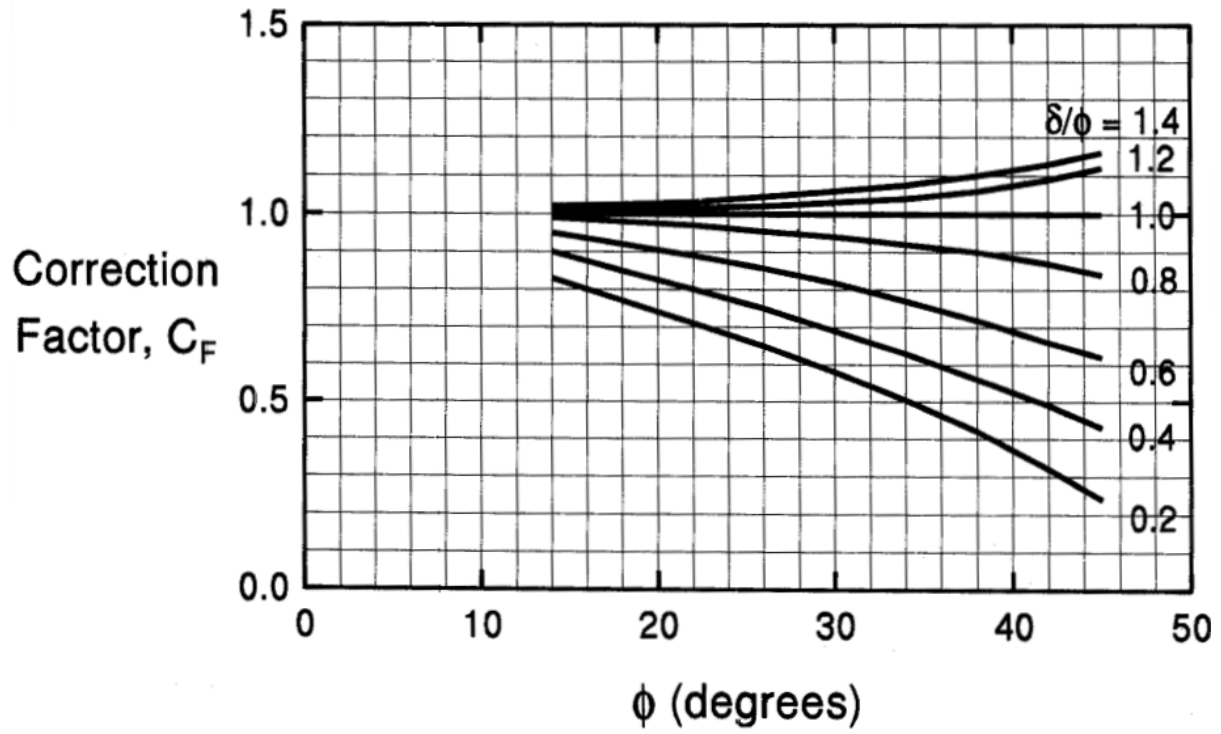


Figure 3
Correction factors (C_f) for coefficient of lateral stress (K_δ) [19]

Direct CPT Methods

There are two main approaches in the estimation of pile resistance using CPT data, the indirect method and the direct method. In the indirect method, CPT data (q_c and f_s) is used to estimate the soil strength parameters, such as the undrained shear strength (S_u) and the angle of internal friction (ϕ), to predict pile resistance. While in the direct method, the unit end bearing resistance (q_b) of the pile is evaluated from the q_c , and the unit skin friction (f) of the pile is evaluated from either f_s or q_c profiles. It is believed that the direct method is more suitable in engineering practices [17].

In the direct CPT methods, the pile resistance is predicted using a pile tip resistance (Q_b) and the skin friction resistance (Q_s), which can be expressed as the following equations:

$$Q_b = q_b A_b = (c_b \cdot q_{c,avg}) \cdot A_b \quad (7)$$

$$Q_s = \sum f_i \cdot A_{si} = \sum \alpha_c f_{si} A_{si} \quad (8)$$

or

$$Q_s = \sum f_i \cdot A_{si} = \sum (c_s \cdot q_{ci}) \cdot A_{si} \quad (9)$$

where q_b is the unit end bearing resistance, q_c is the cone tip resistance, $q_{c,avg}$ is the average cone tip resistance in the zone above and below the pile tip, f_i is the unit skin friction, f_{si} is the sleeve friction, c_b is the correlation coefficient of tip resistance, α_c is the reduction factor, c_s is the correlation coefficient of friction resistance, A_b is the pile tip area, and A_{si} is the pile surface area of each layer.

Schmertmann Method. Schmertmann proposed a direct CPT method based on model and full scale pile tests [14]. To estimate the pile tip resistance (Q_b), the average cone tip resistance ($q_{c,avg}$) is obtained in the zone ranging from 8D above to 0.7D-4D below the pile tip (see Figure 4). Schmertmann suggested a c_b of 1.0 for sand and 0.6 for clay. The unit skin friction is calculated from the sleeve friction (f_s) using an α_c value of 0.2 to 1.25 for clayey soil. A maximum f_s of 1.25 tsf is proposed.

De Ruiter and Beringen Method. The De Ruiter and Beringen method is known as the European method [15]. It is based on experience from offshore piles tested in the North Sea. In sand, unit tip resistance (q_b) is obtained similarly to the Schmertmann method [14]. The unit skin friction (f) for the compression piles is the minimum among f_s , $q_{c(side)}/300$, and 0.2 tsf. In clay, the unit tip resistance (q_b) is determined from the conventional bearing resistance theory as follows:

$$q_b = N_c S_u(\text{tip}) \quad (10)$$

$$S_u(\text{tip}) = \frac{q_c(\text{tip})}{N_k} \quad (11)$$

where, N_c is the bearing resistance factor, and N_k is the cone factor ranging from 15 to 20 depending on soil type and pile type. The unit skin friction (f) can be obtained from:

$$f = \beta S_u(\text{side}) \quad (12)$$

where β is the adhesion factor and $\beta = 1$ for normally consolidated (NC) clay and 0.5 for overconsolidated (OC) clay.

Bustamante and Gianceselli Method (LCPC Method). Bustamante and Gianceselli method is known as the French method or the LCPC (Laboratoire Central des Ponts et Chaussées) method [16]. In this method, both unit tip resistance (q_b) and unit skin friction (f) are calculated from cone tip resistance (q_c). Average cone tip resistance ($q_{c,avg}$) is obtained in the zone ranging 1.5 D above and below the pile tip. A correlation coefficient of the tip resistance (c_b) from 0.15 to 0.6 was proposed for different soil types and installation procedures based on the empirical correlation ($c_b = 0.6$ for piles driven into clay and silt and $c_b = 0.375$ for piles driven into sand and gravel).

The unit skin friction (f) is obtained from cone tip resistance (q_c) and the correlation coefficient of friction resistance (k_s) as follows:

$$f = \frac{q_{eq}(\text{side})}{k_s} \quad (13)$$

where, $q_{eq}(\text{side})$ is the equivalent cone tip resistance of the soil layer, and k_s is an empirical friction coefficient that varies from 30 to 150 depending on soil type, pile type, and installation procedure given in Table 2.

In this study, ultimate pile resistance was calculated using the Louisiana Pile Design by Cone Penetration Test (LPD-CPT) program developed by Louisiana Transportation Research Center. The program is capable of soil classification based on the probabilistic region estimation method proposed by Zhang and Tumay and provides the profile of the ultimate pile resistances with soil depth [24]. The ultimate pile resistances are predicted using the Schmertmann Method, De Ruiter and Beringen Method, LCPC Method, and the average of three CPT methods [14], [15], [16]. Figure 5 presents the general features of the LPD-CPT program. The program is available for free downloading at the LTRC Web site (www.ltrc.lsu.edu/downloads.html).

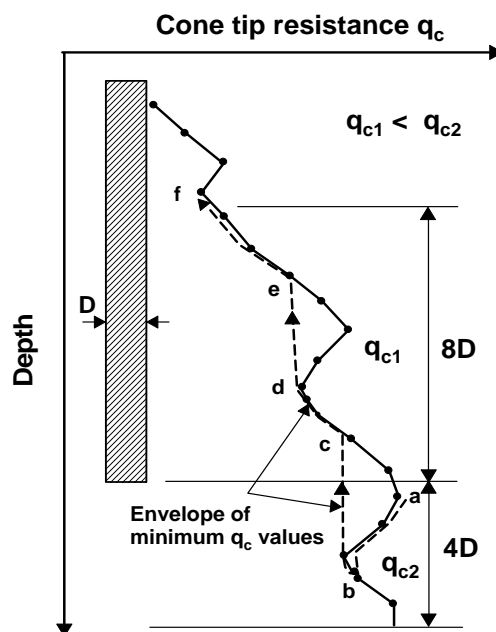
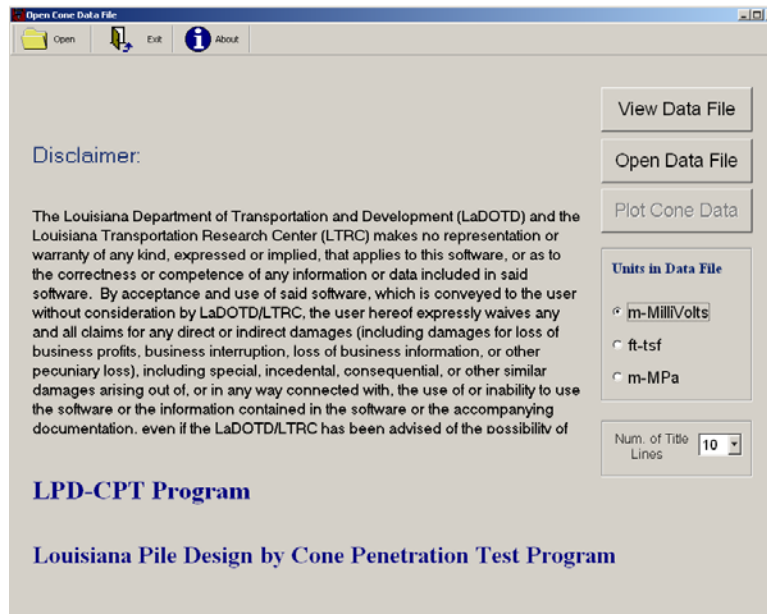


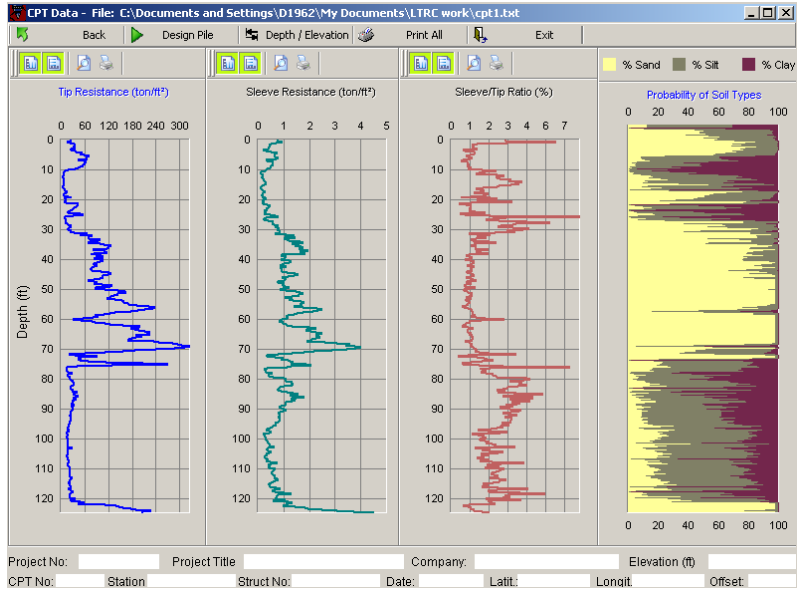
Figure 4
Average cone tip resistance ($q_{c,avg}$) in Schmertmann [14] method
(from Abu-Farsakh and Titi [23])

Table 2
Friction coefficient, k_s (from Bustamante and Gianeselli, [16])

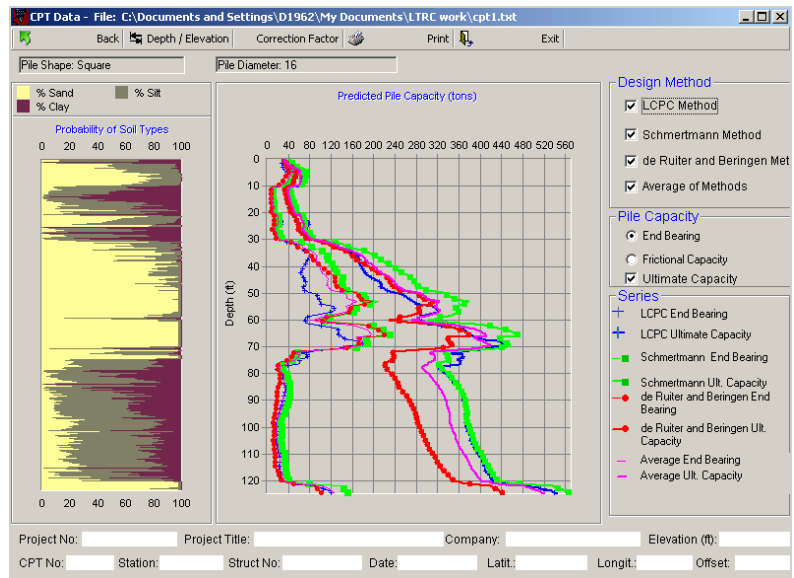
Nature of Soil	q_c (MPa)	k_s
Soft clay and mud	< 1	30
Soft chalk	≤ 5	100
Silt and loose sand	≤ 5	60
Moderately compact clay	1 to 5	40
Moderately compact sand and gravel	5 to 12	100
Compact to stiff clay and compact silt	> 5	60
Weathered to fragment chalk	> 5	60
Compact to very compact sand and gravel	> 12	150



(a) Data input screen



(b) CPT profiles and soil classification



(c) Soil classification and predicted pile resistance

Figure 5
General features of Louisiana Pile Design by Cone Penetration Test (LPD-CPT)
program (www.ltrc.lsu.edu/downloads.html)

Dynamic Measurement with Signal Matching Analysis (CAPWAP)

The evaluation of static capacity from pile driving is based on the concept that the driving operation includes failure in the pile-soil system in which a very fast load test is carried out under each blow. There are two main methods to estimate the ultimate capacity of driven piles on the basis of dynamic driving resistance. One method is using pile driving formulas (i.e., dynamic equations) and the second is wave-equation analysis. Dynamic equations predict total resistance based on work done by the pile during penetration. Wave equation methods are based on a numerical solution of the one-dimensional wave equation. Stress-wave propagation in a pile during driving can be described by the following one-dimensional wave equation modified by Paikowsky and Whitman to include frictional resistance along the pile [25]:

$$E_p \frac{\partial^2 u}{\partial x^2} - \frac{S_p}{A_p} f_s = \rho_p \frac{\partial^2 u}{\partial t^2} \quad (14)$$

where, E_p is the modulus of elasticity, ρ_p is the unit density of the pile material, $u(x,t)$ = longitudinal displacement of infinitesimal segment, f_s = frictional stress along the pile, A_p is the pile area, and S_p is the pile circumference.

The numerical solution uses mathematical models for the pile and the pile-soil system. The wave equation formulation is used in two general ways: pre-driving analysis and post-driving analysis. When the one-dimensional wave equation solution is used for pre-driving analysis, the driving system is also modeled. Post-driving analyses use the measured force signal (calculated from strain readings) and the measured velocity signal (integrated from acceleration readings) obtained near the pile top during driving. At that point the velocity signal is used as a boundary condition while varying the parameters describing soil resistance in order to match calculated and measured force signals. These parameters include the side and tip quake, side and tip damping, pile shaft resistance, and pile tip resistance. Additional parameters may be used to describe soil resistance and rebound ratio for unloading different from that of loading. Iterations are performed by changing soil-model variables for each pile in contact with soil until the best match between force signals is obtained. The results of these analyses are assumed to represent the actual distribution of the ultimate static resistance of the pile. The procedure was first suggested by Goble et al., using the computer program CAPWAP [26].

LRFD Calibration Using Reliability Theory

The uncertainties of load effect and resistance are attributed to various sources and their levels of uncertainties are wide in range. Generally, resistance has higher uncertainty compared to the load effect [3]. Figure 6 shows the probability density functions for the load effect and resistance. Failure can be defined as load exceeding resistance, and probability of failure is related to the shaded area where two probability density functions overlap. In traditional allowable stress design (ASD), the mean factor of safety is the fraction of the mean resistance over the mean load effect. The mean safety factor for the resistance with lower uncertainty (solid curve) and resistance with higher uncertainty (dotted curve) is unchanged. Whereas in LRFD, the larger variability in resistance effect (dotted curve) will lead to a larger probability of failure, which is represented as a larger overlapped area. Accordingly, it is very important to incorporate with the uncertainties in the foundation design by using reliability based theory.

The LRFD method intends to separate uncertainties of load effect from uncertainties of resistance and then apply the statistical theory to ensure the same level of structure safety. To achieve this level of safety, load and resistance factors should be calibrated. This process is performed using measured data to derive statistical parameters characteristic of design methods and determine the magnitude of load and resistance factors needed to obtain acceptable margins of safety. There are mainly two procedures for LRFD calibration: (1) LRFD calibration to achieve consistency with traditional safety factors in ASD but not with target reliability when data is not available and (2) LRFD calibration using reliability theory [27]. When the LRFD design code is calibrated, local experience should be included for several reasons. First, different states have different soil deposition and foundation designers have developed well-established design methods which implicitly incorporate past experience. Such experience can result in cost saving designs but is not included in the existing AASHTO LRFD code, which was developed based on limited databases from other geological origins or stress histories [11]. Under this situation, if the design is performed by the existing code, such experience will be lost, which will cause either over conservative or unsafe design. Instead, if the calibration is performed by including local experience, cost saving designs can be obtained. This calibration can be performed by simply fitting the ASD to LRFD to achieve at least the same cost efficiency. Second, load tests are usually conducted to verify the design loads to evaluate the actual response of the pile under loading and to establish local load test databases. If these local databases are included, the uncertainties are reduced and design is expected to be improved.

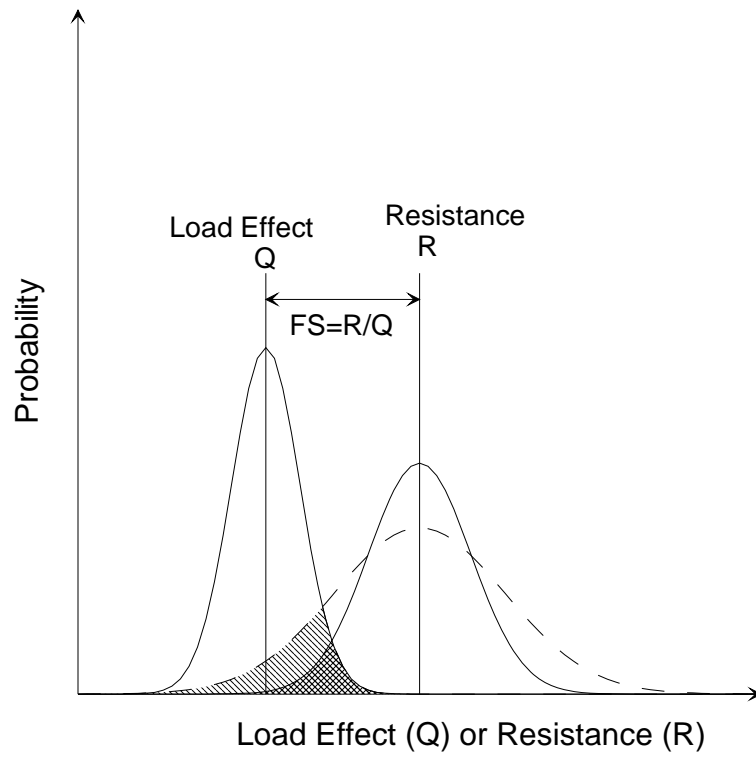


Figure 6
Probability density functions for load effect and resistance

LRFD Calibration via WSD

When there is not sufficient statistical data available for a statistical theory based calibration, calibration through fitting to the ASD method can be used [28]:

$$\phi = \frac{\gamma_D \frac{Q_D}{Q_L} + \gamma_L}{FS \left(\frac{Q_D}{Q_L} + 1 \right)} \quad (15)$$

where, ϕ is the resistance factor; Q_D and Q_L are the dead load and live load, respectively; γ_D and γ_L are the load factors for dead load and live load, respectively; and FS is the factor of safety.

Using equation (15), the resistance factors that need to be used in the LRFD method can be calculated to obtain a level of safety equal to that of the ASD method. This process provides the engineer with a visualization of uncertainty rather than the lumped single factor of safety in ASD. However, using the resistance factor obtained by fitting to the ASD method is not likely to improve the reliability of the design. Improved design can only be achieved when resistance factors are evaluated directly through statistical data based on reliability theory, rather than through calibration with existing factors of safety in ASD [29]. Thus, there is no difference in the design between the LRFD and ASD methods if the resistance factor is calibrated via ASD.

Ultimate pile resistances for the same site obtained by using different design methods show some variation depending on its reliability. Therefore, resistance factors (ϕ) associated with different design methods should reflect its accuracy in predicting the ultimate bearing resistance of piles. In this research, resistance factors (ϕ) were determined for the static design methods (α -Tomlinson method for clay and Nordlund method for sand), three direct CPT methods (Schmertmann method, De Ruiter and Beringen method, and LCPC method), the average of three CPT methods, and the dynamic measurement with signal matching analysis method (CAPWAP).

Statistical Characterization of the Data Collected

To perform an LRFD calibration, the performance limit state equations must first be determined. Two limit states that are usually checked in the design of piles are the ultimate limit state (ULS) or the strength limit state and the serviceability limit state (SLS). Both limit states designs are carried out to satisfy the following criteria [30]:

Ultimate limit state (ULS): Factored resistance \geq Factored load effects

Serviceability limit state (SLS): Deformation \leq Tolerable deformation to remain serviceable

It is usually considered that the design of deep foundations is controlled by the ultimate limit state. Therefore, in the following discussion, only the ultimate limit state is considered. The following basic equation is recommended to represent limit states design by AASHTO [11]:

$$\phi R_n \geq \sum \eta \cdot \gamma_i \cdot Q_i \quad (16)$$

where, ϕ is the Resistance factor, R_n is the Nominal resistance, and η is the Load modifier to account for effects of ductility, redundancy, and operational importance. The value of η usually is 1.00. Q_i is the Load effect, and γ_i is the Load factor.

Most driven piles develop both skin and toe resistances, but the percentage of skin or toe resistance to total resistance is not constant. Therefore, it is not possible to provide a fixed correlation between the three resistance factors (skin, toe, and total resistances). In this research, only the resistance for total resistance was calibrated. Thus, it should be noted that the same resistance factors for skin and end bearing are assumed, and the calibrated resistance factors are valid only for the ranges of pile dimensions (length and diameter) that employed in this study.

Considering the load combination of dead load and live load for AASHTO Strength I case, the performance limit equation is as follows [31]:

$$\phi R_n = \gamma_D Q_D + \gamma_L Q_L \quad (17)$$

The loads applied to piles are traditionally based on superstructure analysis, whereas actual load transfer to substructure, which is actually a pile-superstructure interaction problem, is poorly researched. Most researchers employ the load statistics and the load factors from

AASHTO LRFD specifications, which were originally recommended by Nowak, to make pile foundation design consistent with the bridge superstructure design [9]. For example, Zhang et al., Kim et al., and McVay et al., selected the statistical parameters of dead and live loads, which were used in the AASHTO LRFD specifications as follows [31], [32], [33]:

$$\begin{aligned}\gamma_L &= 1.75 & \lambda_L &= 1.15 & \text{COV}_L &= 0.18 \\ \gamma_D &= 1.25 & \lambda_D &= 1.08 & \text{COV}_D &= 0.13\end{aligned}$$

where, λ_D and λ_L are the load bias factors (mean ratio of measured over predicted value) for the dead load and live load, respectively. COV_D and COV_L are the coefficient of variation values for the dead load and live load, respectively.

The Q_D/Q_L is the dead load to live load ratio which varies depending on the span length [34]. In this research, a Q_D/Q_L of 3 is used for calibration.

The resistance statistics were calculated in terms of the bias factors. The bias factor is defined as the ratio of the measured pile resistance over the predicted pile resistance, i.e.:

$$\lambda_R = \frac{R_m}{R_p} \quad (18)$$

where, R_m is the measured resistance and R_p is the predicted (nominal resistance = R_n).

First Order Second Moment (FOSM) Method

Since FOSM is easy to use and valid for preliminary analyses, it was initially used in this study for calibration of resistance factors of driven piles. In the FOSM method, limit state function is linearized by expanding the Taylor series expansion about the mean value of the variable. Since only the mean and the variance are used in the expansion, it is called First (Mean) Order Second (variance) Moment. For the lognormal distribution of resistance and load statistics, Barker et al. derived the following relation for calculating the reliability index, β [35]:

$$\beta = \frac{\ln \left[\lambda_R \text{FS} \left(\frac{Q_D/Q_L + 1}{\lambda_D Q_D/Q_L + \lambda_L} \right) \sqrt{\frac{1 + \text{COV}_R^2 + \text{COV}_D^2 + \text{COV}_L^2}{1 + \text{COV}_R^2}} \right]}{\sqrt{\ln \left[(1 + \text{COV}_R^2) (1 + \text{COV}_D^2 + \text{COV}_L^2) \right]}} \quad (19)$$

For LRFD, this equation is modified by replacing overall factor of safety (FS) by partial factor of safety and then rearranging to express relation to resistance factor (ϕ) as follows:

$$\phi = \frac{\lambda_R \left(\gamma_D \frac{Q_D}{Q_L} + \gamma_L \right) \sqrt{\frac{1 + \text{COV}_D^2 + \text{COV}_L^2}{1 + \text{COV}_R^2}}{\left(\lambda_D \frac{Q_D}{Q_L} + \lambda_L \right) \exp \left(\beta_T \sqrt{\ln \left[(1 + \text{COV}_R^2) (1 + \text{COV}_D^2 + \text{COV}_L^2) \right]} \right)} \quad (20)$$

When statistical data is available, the calibration can be performed based on reliability theory. The design based on the resistance factors obtained from this process is expected to be better than ASD and fully achieved the advantages of the LRFD method. If both the resistances and the load are log-normally distributed and the performance limit state design equation is linear, the First Order Second Moment method can be used to obtain the reliability index.

Unfortunately, for deep foundations, under some conditions, neither the loads nor the resistance are normally or log-normally distributed, and sometimes the design equation may be nonlinear. Therefore, advance reliability theory based methods are needed. In this study, FORM and the Monte Carlo simulation were also used with FOSM for the LRFD calibration of pile design methods [36], [37].

First Order Reliability Method (FORM)

Hasofer and Lind proposed a modified reliability index that did not exhibit the invariance problem [36]. The “correction” is to evaluate the limit state function at a point known as the “design point” instead of the mean values. The design point is a point on the failure surface $g = 0$. Since the design point is generally not known in advance, an iteration technique must be used to solve the reliability index. Detailed procedure regarding FORM can be found in Nowak and Collins [37]. Only information on the means and the standard deviations of the

resistances and the loads are needed while detailed information on the type of distribution for each random variable are not needed.

FORM is an analytical scheme that is used to approximate probability integral when basic variables have strictly increasing continuous joint cumulative distribution function (CDF).

Steps for FORM using the Rackwitz-Fiessler method [38]:

1. Define limit state function, $g(x_1, x_2, x_3, \dots)$.

The limit state function for LRFD is developed as follows:

$$Q = Q_D + Q_L = \lambda_D Q_D + \lambda_L Q_L \quad (21)$$

$$\phi R = \gamma_D Q_D + \gamma_L Q_L \quad (22)$$

from (21) and (22)

$$g(R.L) = \left(\frac{\gamma_D Q_D + \gamma_L Q_L}{\phi} \right) \lambda_R - (\lambda_D Q_D + \lambda_L Q_L) = 0 \quad (23)$$

Now the specified live load to dead load ratio, (Q_L/Q_D) equation (23) can be rearranged as:

$$g(R.L) = \left(\frac{\gamma_D + \gamma_L * \kappa}{\phi} \right) \lambda_R - (\lambda_D + \lambda_L * \kappa) = 0 \quad (24)$$

where, $\kappa = \frac{Q_L}{Q_D}$

2. Assuming initial design point (x_1^*) , mean values are a reasonable choice for most cases. In this case, initial design values for dead load and live load (x_2 and x_3) assumed and that for resistance (x_1) is determined by equating the limit state function equal to zero. Also for lognormal variables equivalent normal parameters are then determined as:

$$\mu_x^e = x^* - \sigma_x^e \left[\Phi^{-1}(F_x(x)) \right] \quad (25)$$

$$\sigma^e_x = \frac{1}{f_x(x^*)} \phi \left[\frac{x^* - \mu^e_x}{\sigma^e_x} \right] = \frac{1}{f_x(x^*)} \phi \left[\Phi^{-1}(F_x(x^*)) \right] \quad (26)$$

where, ϕ and Φ denotes the mass probability density function (PDF) and the cumulative distribution function (CDF) for normal distribution, respectively.

3. Corresponding to the design point x^* , the reduced variable is found as:

$$z^*_i = \frac{x^* - \mu^e_{x_i}}{\sigma^e_{x_i}} \quad (27)$$

4. Partial derivatives of the limit state function is found at the design point and vector G is defined as:

$$G = \begin{Bmatrix} G1 \\ G2 \\ G3 \end{Bmatrix} \text{ where, } G_i = - \left. \frac{\partial g}{\partial Z_i} \right|_{\text{at design point}} = - \left. \frac{\partial g}{\partial X_i} \right|_{\text{at design point}} * (\sigma^e_{x_i}) \quad (28)$$

$$\beta = \frac{\{G\}^T \{z^*\}}{\sqrt{\{G\}^T \{G\}}} \text{ where, } \{z^*\} = \begin{Bmatrix} z^*_1 \\ z^*_2 \\ z^*_3 \end{Bmatrix} \quad (29)$$

$$\alpha = \frac{\{G\}}{\sqrt{\{G\}^T \{G\}}} \quad (30)$$

5. The new design point is determined in the reduced variable as:

$$z^*_i = \alpha_i \beta \quad (31)$$

$$x^*_i = \mu^e_{x_i} + z_i \sigma^e_{x_i} \quad (32)$$

Also at this step, the new design point for resistance (x_1) is determined by inserting new design values for loads (x_2 and x_3) into the g function. With new design points, steps from 1 to 5 are followed. The process is repeated until β and the design point converges. In this study, the excel sheet was used to get the FOSM solution with the “Goal seek” function for given load and resistance statistics. Iterations for FORM is done using the “SOLVER” tool.

Monte Carlo Simulation Method

For more complicated limit state functions, the application of the general statistical method for the calculation of the reliability index is either extremely difficult or impossible. Under this circumstance, the Monte Carlo simulation provides the only feasible way to determine the reliability index or the probability of failure. The procedures of the Monte Carlo simulation are described in the following paragraphs.

First, simulated values of the random variables in the limit state equation are generated. These values are then used to simulate values of the limit state function itself. Finally, the simulated values of the limit state function are plotted on normal probability paper. The probability of failure can be found by reading the probability value at the location where the plotted data curve intersects a vertical line passing through the origin. If there are too few data points and the plotted curve does not intersect the vertical axis, the plotted curve can be extrapolated.

The Monte Carlo method is a technique by which a random number generator is used to extrapolate CDF values for each random variable. Extrapolation of CDF makes estimating β possible; otherwise, a limited quantity of data would restrict the reliable estimate of β . Once the reliability index, β , is estimated, the probability of failure can be estimated by assuming the distribution of $g(x)$. The steps of the Monte Carlo simulation method are as follows;

1. Generate random numbers for each set of variables. Here there are three variables (resistance, dead load and live load bias factor), so three sets of random variable have to be generated independently for each one. The number of simulations required is found using the following equation:

$$N = \frac{1 - P_{\text{true}}}{V_p^2 * (P_{\text{true}})} \quad (33)$$

where, P_{true} is the lowest magnitude of probability that is to be determined using Monte Carlo, and V_p is the desired coefficient of variation of the simulation result. To estimate a probability as low as 10^{-2} and keep variance under 10 percent, the number of points to be generated in the Monte-Carlo simulation is 9900.

For each lognormal variable, the sample value x_i is estimated as:

$$x^*_i = \exp(\mu_{\ln x} + z_i \sigma_{\ln x}) \quad (34)$$

where, $\sigma^2_{\ln x} = \ln(V^2_x + 1)$ and $\mu_{\ln x} = \ln(\mu x) - \frac{1}{2} \sigma^2_{\ln x}$

In the previous expressions, μ_x and V_x are the arithmetic mean and variance of x , respectively, and $\mu_{\ln x}$ and $\sigma_{\ln x}$ are the equivalent lognormal mean and standard deviation [mean and standard deviation of $\ln(x)$] and $z_i = \text{NORMSINV}(\text{RAND}())$ is the random standard normal variable generated using the EXCEL function.

2. Define the limit state function. This is done in step 1 for FORM.
3. Find the number of cases where $g(x_i) \leq 0$. The probability of failure is then defined as:

$$Pf = \frac{\text{count}(g \leq 0)}{N} \quad (35)$$

and reliability index β is estimated as:

$$\beta = \Phi^{-1}(Pf) \quad (36)$$

METHODOLOGY

As discussed earlier, the objective of this research study is to calibrate resistance factors for different pile design methods needed in the LRFD design of driven piles based on the Louisiana database and experience. A total of 53 pile load tests, their corresponding borings and CPT tests were identified and collected from LADOTD files. The collected pile load test data, soil properties, and CPT profiles were compiled and analyzed. This section describes the methodology of collecting, compiling, and analyzing the pile load test reports.

Collecting of Pile Load Test Database

The LADOTD practice for pile load tests is mainly to validate and verify the pile capacity estimates based on a selected design method/procedure. If the tested pile fails to provide the required support for design loads, more piles will be added to increase the load carrying capacity and/or increase the lengths of the driven piles. The reports of all pile load tests conducted in Louisiana projects are available in the general files section at LADOTD headquarters in Baton Rouge. In this study, pile load test reports were obtained from the pavement and geotechnical design section at LADOTD.

The research team carefully reviewed and evaluated the collected pile load test reports from LADOTD for potential inclusion in the current research. The criterion for including a pile load test case in the analysis was based on the availability of proper documentation on pile data (installation and testing), pile failed during loading, adequate subsurface exploration and soil testing from borings, CPT soundings, and site location. If a pile load test report was found to be satisfactory, the report was then considered for inclusion in this study.

The pile load test database used for calibration was established by conducting an extensive search in LADOTD's project files. Only PPC piles that have been tested to failure and include adequate soil information were included in this study. The use of PPC piles is more economical in Louisiana where the driving of piles is not a problem. A total of 53 pile load tests met this criterion. Figure 7 presents a map of Louisiana with locations of the test piles that were investigated in this research. A summary of the characteristics of the investigated piles is presented in Table 3.

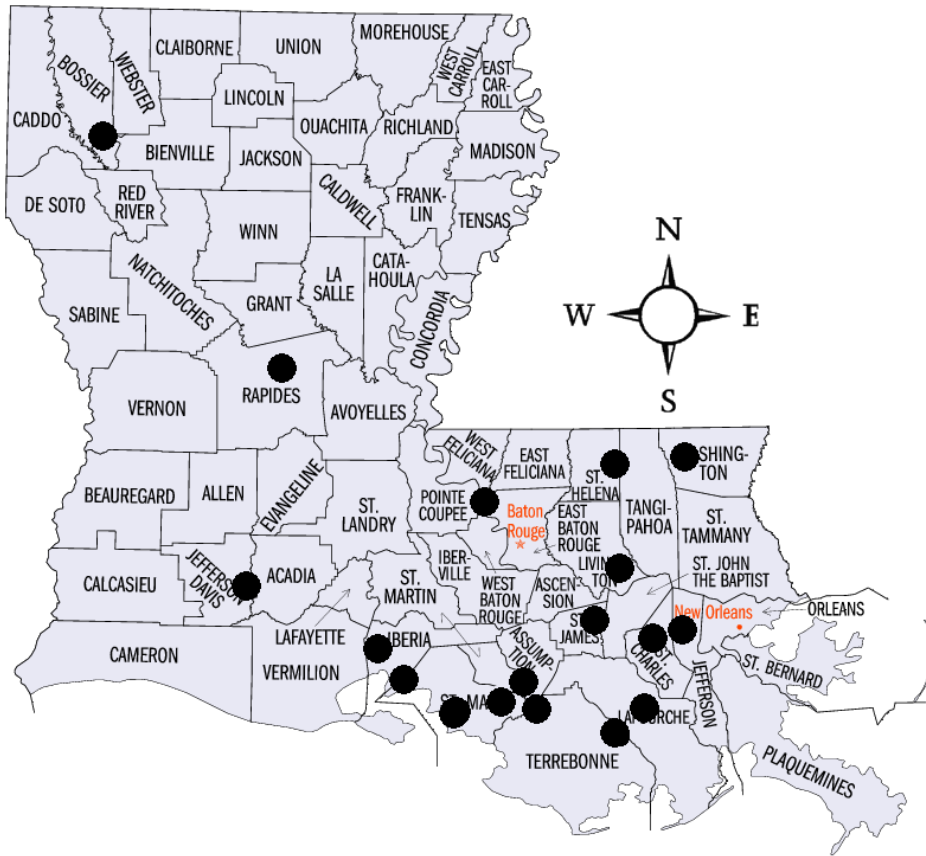


Figure 7
Approximate locations of the investigated piles

Compilation of Pile Load Test Reports

The data on the selected pile load test reports were compiled. The information and data regarding the project, soil stratification and properties, pile characteristics, load test data, CPT profiles, dynamic test data, etc. were processed and transferred from each load test report to tables, forms, and graphs. Some of the CPT soundings were obtained in digital format. The remainder of the CPT profiles were scanned using a high resolution scanner and then were converted to a digital format using the automated digitizing software UN-SCAN-IT. The following data and information were collected and compiled for each pile load test report.

The site data provide the necessary information to identify the location of the project. The site identification used herein is the Louisiana state project number. For example, the site ID 005-05-0065 is the state project number for St. Louis Canal Bridge.

Table 3
Summary of the characteristics of the investigated piles

Square PPC Pile Size	Pile Type		Predominant Soil Type		
	Friction	End-Bearing	Cohesive	Cohesionless	Limit of Information
14"	22	0	19	3	0
16"	5	0	3	0	2
18"	2	0	1	1	0
24"	9	1	7	3	0
30"	13	1	9	5	0
Total	51	2	39	12	2

The soil data consist of information on the soil boring location (station number), soil stratigraphy, unit weight, Atterberg limits, laboratory testing (shear strength, physical properties, etc.), and in-situ test results (e.g., SPT for cohesionless soil).

The foundation data consist of pile characteristics (pile ID, material type, cross-section, total length, and embedded length), installation data (location of the pile, date of driving, driving record, hammer type, etc.), and pile load test (date of loading, applied load with time, pile head movement, pile failure under testing, etc.).

The cone penetration soundings include test location (station number), date, cone tip resistance, and sleeve friction profiles with depth.

Ultimate Capacity of Piles from Load Test

The measured ultimate pile resistance (Q_u) was interpreted from the load-deformation curve using the Davisson method for piles with a size less than 24 inches and the modified Davisson method proposed by Kyfor et al. for piles exceeding a size of 24 inches [39], [40].

In the Davisson method, the ultimate bearing resistance is determined from the intersection between a load-settlement curve and a straight line with a slope of L/AE and an initial settlement intercept of $0.15 + D/120$ (in.) [(L: pile length, A: cross-sectional area of the pile, E: Young's modulus of the pile, and D: pile diameter (in.)). Figure 8 illustrates a sample load-deformation test analysis using the Davisson method. In addition to load test results, all other relevant information such as soil borings, pile driving logs, and CPT data were also collected. Figure 9 shows a typical summary of geotechnical data for a tested pile. The summary of geotechnical data for all projects investigated in this research is presented in the Appendix.

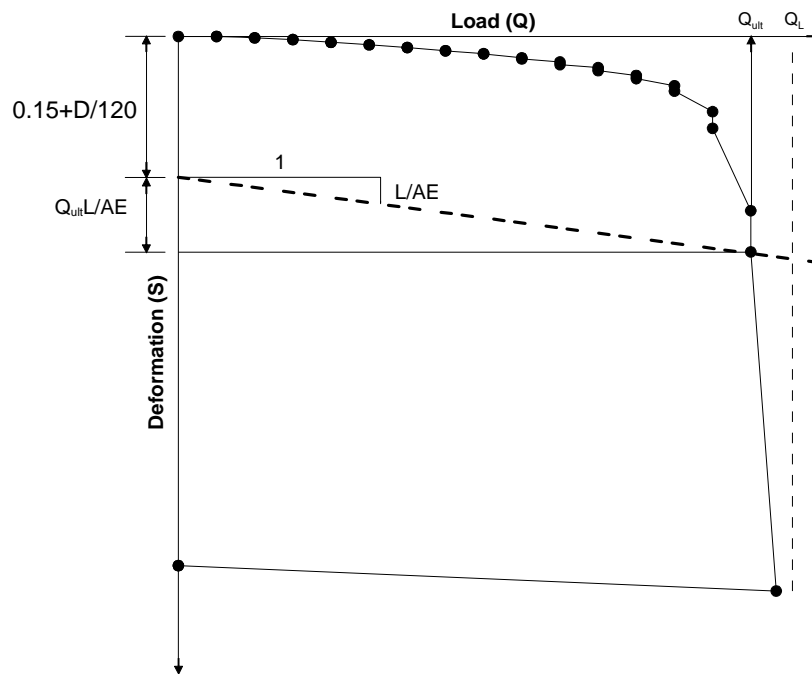


Figure 8
Estimation of ultimate bearing resistance using Davisson [39] method after Salgado [41]

Soil Identification and Classification

In addition to the soil classification determined based on soil borings and results of laboratory testing, the probabilistic region estimation CPT method was also used for soil classification and identification of site stratigraphy [24]. This method is similar to the classic soil classification methods since it is based on soil composition. It identifies three soil types:

clayey, silty, and sandy soils. The probabilistic region classification method provides a profile of the probability or the chance of having each soil type (clay, silt, and sand) with depth. In this study, the probability of soil types for each soil layer was used in connection with CPT pile prediction methods to calculate the appropriate correlation factors needed to estimate q_b and f from CPT data. A typical soil profile obtained by the probabilistic region estimation is shown in Figure 9.

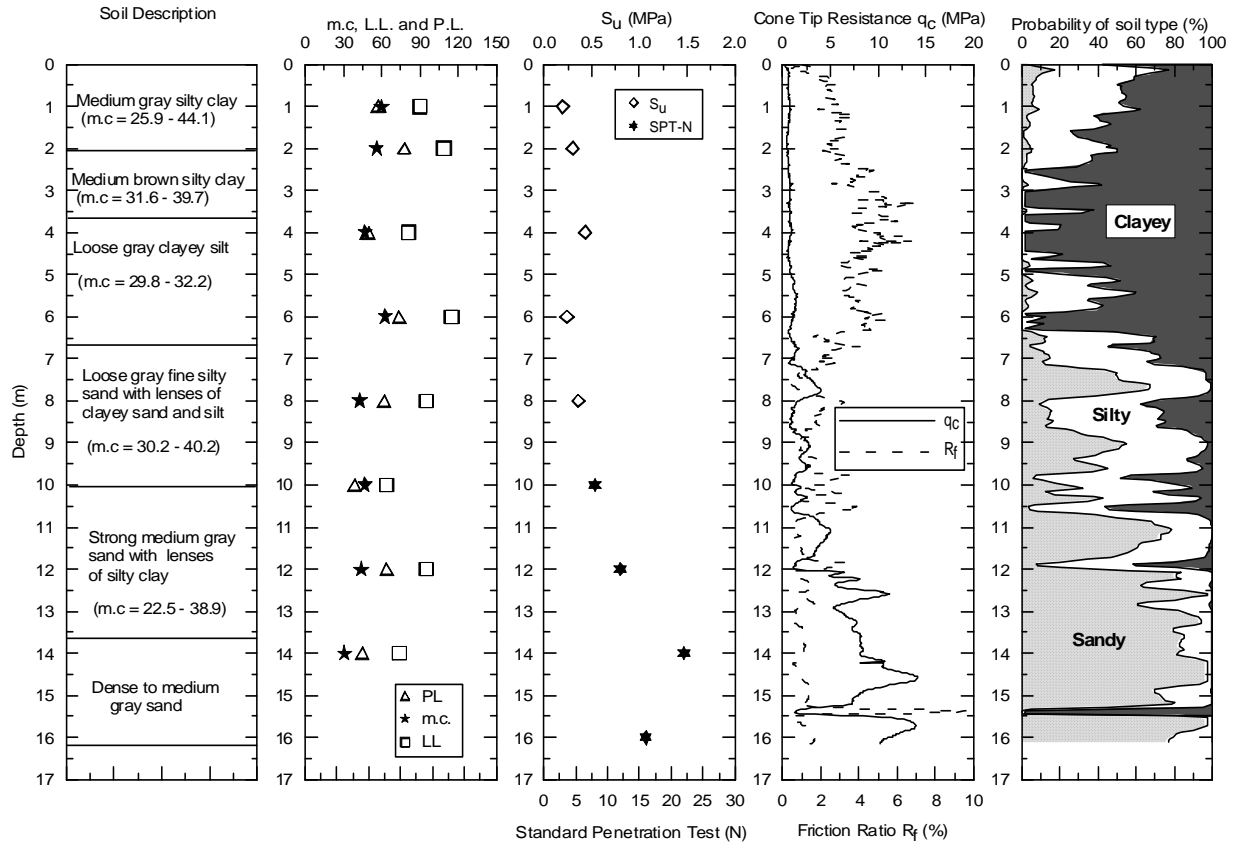


Figure 9
Typical summary of geotechnical data for a tested pile

DISCUSSION OF RESULTS

Predicted versus Measured Ultimate Pile Resistances

Based on the analysis of 53 driven piles, a statistical analysis was performed on the collected database to evaluate the accuracy and performance of the different pile capacity estimation methods. Table 4 presents the summary of the analysis performed on the investigated piles. The table shows the pile size, type, length, location of the pile, measured ultimate load resistance, and predicted load capacities of different methods. The mean and standard deviation of the predicted to measured pile capacity ratios (R_p/R_m), which are the inverses of the bias factor (λ), were calculated for the previously mentioned methods and are summarized in Table 5. Figures 10 through 16 present the comparison between the estimated pile capacities and the measured pile capacity using the Davisson method and modified Davisson method for the different pile design methods. Linear regression analysis was conducted on all methods to obtain the best fit line of the predicted/measured pile capacities (R_p/R_m) assuming zero intercept. The relationship R_p/R_m and the corresponding coefficient of determination (R^2) determined for all estimation methods are presented in Figures 10 through 16 and also summarized in Table 5.

The comparison of static calculated pile capacities versus the measured load tests compiled from all soil types are presented in Figure 10. The results indicated the ratio R_{fit}/R_m for the static calculation method is 1.0 with an R^2 of 0.84. The coefficient (slope) of 1.0 from the regression analysis implies that the relative bias of the pile capacity prediction based on the static calculation method does not vary with the measured capacities because the adhesion factor α already included the adjustment of biases inherent in the soil strengths. The mean and standard deviation of the R_p/R_m ratio for static method are 1.12 and 0.32, respectively, indicating an average of 12 percent overestimation using the static calculation method which includes Tomlinson's α -method in clay and Nordlund's method in sand.

Figures 11 through 14 depict the comparison between the CPT estimation methods and measured pile capacities using the Davisson interpretation method. It should be noted that the correlations resulted in R_{fit}/R_m ratios ranging from 0.94 to 1.20. The De Ruiter Beringen method had the lowest ratio of 0.94. All other methods had R_{fit}/R_m ratios higher than 1 indicating an increasing bias in pile capacity estimation with the increase in actual capacity. In other work, the CPT methods tend to overestimate the capacity for high capacity piles. The average ratios of R_p/R_m of the CPT methods range from 0.90 to 1.20. The De Ruiter-Beringen method has the smallest ratio implying an average of 10 percent underestimation of

pile capacity while the other CPT methods overestimate the capacity by 5 to 20 percent. On average, the LCPC method overestimates the ultimate pile capacity by 5 percent and the Schmertmann method overestimates the pile capacity by 20 percent. The comparison between the average of CPT methods (CPT-average method) and measured pile capacities are presented in Figure 14. The R_{fit}/R_m for the average of CPT methods is 1.08 with $R^2 = 0.82$. This is not surprising due to the fact that the CPT methods generally overestimate pile capacities for high capacity piles as discussed earlier. The mean and standard deviation of the ratio R_p/R_m for CPT-average method are 1.05 and 0.33, respectively. The use of the average CPT methods overestimates the pile capacity by 5 percent in average.

Figures 15 and 16 present the results of signal matching using CAPWAP, which show the average end of driving (EOD) resistance is about 35 percent of the average load test value indicating a setup factor of about 2.9. The 14-day beginning of restrike (BOR) data show the CAPWAP estimation of pile resistance still underestimated the measured value by 17 percent.

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils

	State Project Identification	003-07-0019	005-01-0056	005-01-0056	005-01-0056
		BNSS Overpass - Jennings	Southern pacific railroad overpass	Southern pacific railroad overpass	Southern pacific railroad overpass
Pile and Soil Identification	Pile ID	TP1, 24" Square PPC	TP1, 24" Square PPC	TP2, 14" Square PPC	TP3, 24" Square PPC
	Pile Length (ft.)	60	90	74	92
	Embedded Length (ft.)	54.2	85	63.7	87
	Pile Classification	Friction	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	—	246.0	94.6	324.5
	de Ruiter & Beringen	—	174.9	75.4	293.8
	LCPC	—	196.5	92.7	340.7
	Average	—	205.8	87.6	319.6
Static Analysis	α-method and Nordlund method	217.3	271	—	363.7
Dynamic measurement	CAPWAP (EOD)	75.0	—	—	—
	CAPWAP (14 days-BOR)	238.8	—	—	—
Load Test Interpretation Method	Davisson method	272	225	132	320

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	005-05-0065	008-01-0042	065-90-0024	065-90-0024
		St. Louis Canal Bridge	La 415 - La 983	Houma I.C.W.W. Bridges	Houma I.C.W.W. Bridges
Pile and Soil Identification	Pile ID	TP1, 16" Square PPC	TP1, 16" Square PPC	TP1, 14" Square PPC	TP2, 14" Square PPC
	Pile Length (ft.)	92	81.3	85	73.5
	Embedded Length (ft.)	79.2	50.7	80	70
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	—	—	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	—	—	174.4	90.0
	de Ruiter & Beringen	—	—	110.6	82.1
	LCPC	—	—	152.6	79.7
	Average	—	—	145.9	83.9
Static Analysis	α -method and Nordlund method	—	—	162.4	115.5
Dynamic measurement	CAPWAP (EOD)	20	10.6	—	—
	CAPWAP (14 days-BOR)	70	—	—	—
Load Test Interpretation Method	Davisson method	109	86	115	55

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

		065-90-0024	260-05-0020	260-05-0020	260-05-0020
State Project Identification		Houma I.C.W.W. Bridges	Tichfaw River Bridge and Approaches	Tichfaw River Bridge and Approaches	Tichfaw River Bridge and Approaches
Pile and Soil Identification	Pile ID	TP3, 14" Square PPC	TP1, 30" Square PPC	TP2, 30" Square PPC	TP3, 30" Square PPC
	Pile Length (ft.)	94.5	73	118	81
	Embedded Length (ft.)	80	59.3	61.2	73.9
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesionless	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	184.3	361.5	—	344.5
	de Ruiter & Beringen	72.2	325.6	—	288.7
	LCPC	94.7	301.9	—	325.5
	Average	117.1	329.7	—	319.6
Static Analysis	α-method and Nordlund method	139.9	441.9	578.9	396.9
Dynamic measurement	CAPWAP (EOD)	—	136.3	—	182.3
	CAPWAP (14 days-BOR)	—	517.1	—	379.6
Load Test Interpretation Method	Davisson method	120	478	542.5	464

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	262-06-0009 & 262-07-0012	424-05-0078	424-05-0078	424-04-0027
		Bridge #1 Tickfaw River	Bayou Boeuf Bridge Main Span	Bayou Boeuf Bridge Main Span	US 90 Interchange at John Darnell Rd.
Pile and Soil Identification	Pile ID	TP1, 24" Square PPC	TP1, 14" Square PPC	TP2, 14" Square PPC	TP2, 14" Square PPC
	Pile Length (ft.)	85	80	80	60
	Embedded Length (ft.)	84.9	77	75	38
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesionless	Cohesionless	Cohesionless	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	443.3	—	123.9	—
	de Ruiter & Beringen	371.8	—	118.8	—
	LCPC	536.0	—	149.1	—
	Average	450.3	—	130.6	—
Static Analysis	α-method and Nordlund method	—	185.6	147.3	104.1
Dynamic measurement	CAPWAP (EOD)	—	—	—	39
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	240	165	112.5	135

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	424-05-0078	424-05-0081	424-05-0081	424-06-0005
		Bayou Boeuf Bridge Main Span	Bayou Boeuf Bridge (West Approach)	Bayou Boeuf Bridge (West Approach)	Bayou Boeuf Bridge
Pile and Soil Identification	Pile ID	TP5, 14" Square PPC	TP1, 14" Square PPC	TP4, 16" Square PPC	TP1, 14" Square PPC
	Pile Length (ft.)	85	96	80	75
	Embedded Length (ft.)	80.5	89.5	69.9	67
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	71	134.9	130.4	85.9
	de Ruiter & Beringen	88.2	86.9	84.8	66.7
	LCPC	75.4	110.1	96.4	66.4
	Average	78.2	110.6	103.8	73.0
Static Analysis	α -method and Nordlund method	118.0	124.9	100.7	122.9
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days-BOR)	—	113.4	96	—
Load Test Interpretation Method	Davisson method	124	115	100	98

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

		424-06-0005	424-07-0021	450-15-0085	424-05-0087
	State Project Identification	Bayou Boeuf Bridge	Bayou L'ourse	I-10 Williams Boulevard Interchange	Morgan City - Gibson Highway
Pile and Soil Identification	Pile ID	TP2, 14" Square PPC	TP1, 30" Square PPC	TP3A, 14" Square PPC	TP1, 16" Square PPC
	Pile Length (ft.)	85	124	87	85
	Embedded Length (ft.)	71	117	75.5	69.5
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	94.9	369.9	—	115.9
	de Ruiter & Beringen	75.7	296.7	—	81.9
	LCPC	75.1	294.3	—	97.9
	Average	81.9	320.3	—	98.6
Static Analysis	α-method and Nordlund method	111.5	411.9	140.2	—
Dynamic measurement	CAPWAP (EOD)	—	123.3	—	—
	CAPWAP (14 days-BOR)	—	374.6	150.1	—
Load Test Interpretation Method	Davisson method	98	380	172	105

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	424-05-0087	424-05-0087	424-05-0087	424-05-0081
		Morgan City - Gibson Highway	Morgan City - Gibson Highway	Morgan City - Gibson Highway	Bayou Boeuf Bridge (West Approach)
Pile and Soil Identification	Pile ID	TP3, 30" Square PPC	TP5, 30" Square PPC	TP7, 16" Square PPC	TP3, 14" Square PPC
	Pile Length (ft.)	115	115	85	74
	Embedded Length (ft.)	104.1	113	77	64.5
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesionless	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	603.1	794.5	131.5	135.0
	de Ruiter & Beringen	532.2	603.8	95.6	108.1
	LCPC	612.1	776.2	111.5	131.6
	Average	582.5	724.8	112.9	124.9
Static Analysis	α -method and Nordlund method	526.6	454.7	133.1	119.7
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days - BOR)	—	—	—	66.5
Load Test Interpretation Method	Davisson method	482	565	111	165

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	424-06-0005	424-06-0005	014-02-0018	047-02-0022
		Bayou Boeuf Bridge	Bayou Boeuf Bridge	Union Pacific Railroad Overpass	Bogue Chitto Bridge
Pile and Soil Identification	Pile ID	TP3, 14" Square PPC	TP5, 14" Square PPC	TP1, 30" Square PPC	TP2, 30" Square PPC
	Pile Length (ft.)	78	85	107	92
	Embedded Length (ft.)	77.5	79	45.5	58
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesive	Cohesionless
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	99.8	94.4	—	468.5
	de Ruiter & Beringen	71.5	57.4	—	418.4
	LCPC	82.8	67.6	—	503.3
	Average	84.7	73.1	—	463.4
Static Analysis	α -method and Nordlund method	98.8	100.6	289.1	475.8
Dynamic measurement	CAPWAP (EOD)	—	—	162.5	120
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	100	90	455	390

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

		424-05-0087	064-06-0036	090-01-0015	090-01-0015
State Project Identification		Morgan City - Gibson Highway	Bayou Lafourche Bridge and Approaches	Lake Bistineau Spillway Bridge	Lake Bistineau Spillway Bridge
Pile and Soil Identification	Pile ID	TP4, 30" Square PPC	TP1, 24" Square PPC	TP1, 24" Square PPC	TP2, 24" Square PPC
	Pile Length (ft.)	115	59.1	56.5	65
	Embedded Length (ft.)	99.3	39	39.5	47
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesionless	Cohesionless	Cohesive	Cohesionless
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	664.6	74.4	263.4	686.9
	de Ruiter & Beringen	534.4	39.3	222.8	609.0
	LCPC	619.8	52.5	272.4	529.8
	Average	606.3	55.4	252.9	608.6
Static Analysis	α-method and Nordlund method	590.9	72.8	172.5	262.6
Dynamic measurement	CAPWAP (EOD)	—	17.5	149.9	60
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	570	77	149	120

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	424-05-0087	713-48-0083	713-48-0083	455-05-0036
		Morgan City - Gibson Highway	Bayou Milhomme Bridge & Approaches	Bayou Milhomme Bridge & Approaches	Sugarhouse Road
Pile and Soil Identification	Pile ID	TP2, 30" Square PPC	TP1, 24" Square PPC	TP2, 24" Square PPC	TP1, 14" Square PPC
	Pile Length (ft.)	115	72	96	75
	Embedded Length (ft.)	89.2	55	85	70
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesionless	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	546.3	—	—	130.9
	de Ruiters & Beringen	433.7	—	—	90.3
	LCPC	574.8	—	—	98.9
	Average	518.3	—	—	106.7
Static Analysis	α -method and Nordlund method	415.1	78.4	201.5	—
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	518	83.2	153	96

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

		455-05-0036	455-05-0036	434-01-0002	239-01-0080
State Project Identification		Sugarhouse Road	Sugarhouse Road	Mississippi River Bridge at Gramercy (West Approaches)	ICWW Bridge Approaches (Louisiana)
Pile and Soil Identification	Pile ID	TP2, 14" Square PPC	TP3, 14" Square PPC	TP3, 14" Square PPC	TP3, 14" Square PPC
	Pile Length (ft.)	75	65	105	60
	Embedded Length (ft.)	70	59	64	55
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesionless	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	111.6	121	201.8	130.5
	de Ruiter & Beringen	95.4	69	175.8	107.6
	LCPC	105	80.5	182.8	108.1
	Average	104	90.2	186.8	115.4
Static Analysis	α-method and Nordlund method	—	—	—	—
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	96.5	69	230	115.5

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	239-01-0080	855-14-0003	Alexandria	424-06-0005
		ICWW Bridge Approaches (Louisa)	Intercoastal Waterway Bridge (Prospect Ave.)	Pinville-Tioga Highway Railroad Overpass	Bayou Boeuf Bridge
Pile and Soil Identification	Pile ID	TP4, 30" Square PPC	TP1, 18" Square PPC	TP1, 14" Square PPC	TP4, 14" Square PPC
	Pile Length (ft.)	91.9			85
	Embedded Length (ft.)	84.3	95	31	79
	Pile Classification	Skin	Skin	Skin	Skin
	Predominant Soil*	Cohesive	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	411.2	107.8	132.2	124.3
	de Ruitter & Beringen	321.8	116.7	80.5	79.5
	LCPC	310	112.2	110.1	97.4
	Average	347.7	112.2	107.6	100.4
Static Analysis	α-method and Nordlund method	—	—	—	—
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	360.8	216	90	120

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4

Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	424-07-0009	424-07-0009	450-36-0002	283-03-0052 (New Orleans)
		Gibson-raceland highway	Gibson-raceland highway	Luling Bridge - US61	West Bank Expressway
Pile and Soil Identification	Pile ID	TP4A, 30" Square PPC	TP4, 30" Square PPC	TP8, 30" Square PPC	TP1, 18" Square PPC
	Pile Length (ft.)	125	125	120	
	Embedded Length (ft.)	124	120	112	120
	Pile Classification	—	Skin	End Bearing	Skin
	Predominant Soil*	—	Cohesive	Cohesive	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u	Q _u	Q _u	Q _u
	Schmertmann	739.2	740.2	794.7	128.6
	de Ruiter & Beringen	506	507	656.4	179.2
	LCPC	606.3	607.3	796.6	169.1
	Average	617.2	618.2	749.2	159
Static Analysis	α-method and Nordlund method	—	—	—	—
Dynamic measurement	CAPWAP (EOD)	—	—	—	—
	CAPWAP (14 days-BOR)	—	—	—	—
Load Test Interpretation Method	Davisson method	633	320	482	372

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 4
Results of the analysis conducted on square precast prestressed concrete piles driven into Louisiana soils (continued)

	State Project Identification	090-01-0015 Lake Bistineau Spillway Bridge & Approaches
Pile and Soil Identification	Pile ID	TP6, 24" Square PPC
	Pile Length (ft)	—
	Embedded Length (ft)	60
	Pile Classification	End Bearing
	Predominant Soil*	Cohesive
Methods of Predicting Pile Capacity by Cone Penetration Test	Predicted Ultimate Load (ton)	Q _u
	Schmertmann	347.7
	de Ruiter & Beringen	241.5
	LCPC	258.3
	Average	282.5
Static Analysis	α -method and Nordlund method	—
Dynamic measurement	CAPWAP (EOD)	—
	CAPWAP (14 days - BOR)	—
Load Test Interpretation Method	Davisson method	270.5

* Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); Q_u: Total ultimate capacity

Table 5
Evaluation summary of the different prediction methods

Pile Capacity Prediction Method	R_p/R_m^*		Best fit calculations	
	Mean	σ	R_{fit}/R_m^*	R^2
Static α -method	1.12	0.32	1.00	0.84
Schmertmann method	1.20	0.37	1.20	0.81
LCPC method	1.05	0.38	1.11	0.78
de-Ruiter & Beringen method	0.90	0.28	0.94	0.84
Average of CPT methods	1.05	0.33	1.08	0.82
CAPWAP–EOD method	0.35	0.23	0.32	0.69
CAPWAP–14 days BOR method	0.83	0.22	0.92	0.91

* Davisson method and modified Davisson method

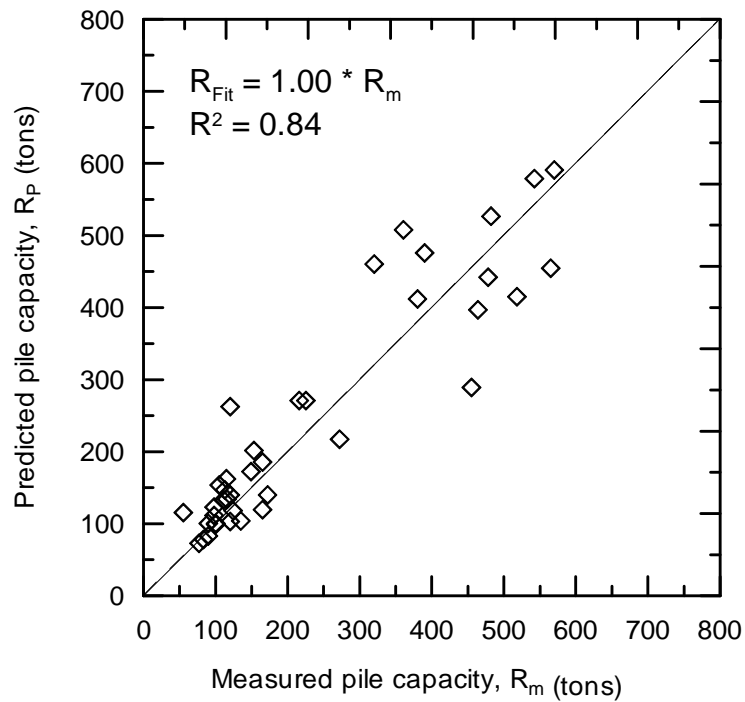


Figure 10
 R_m versus R_p (Static method)

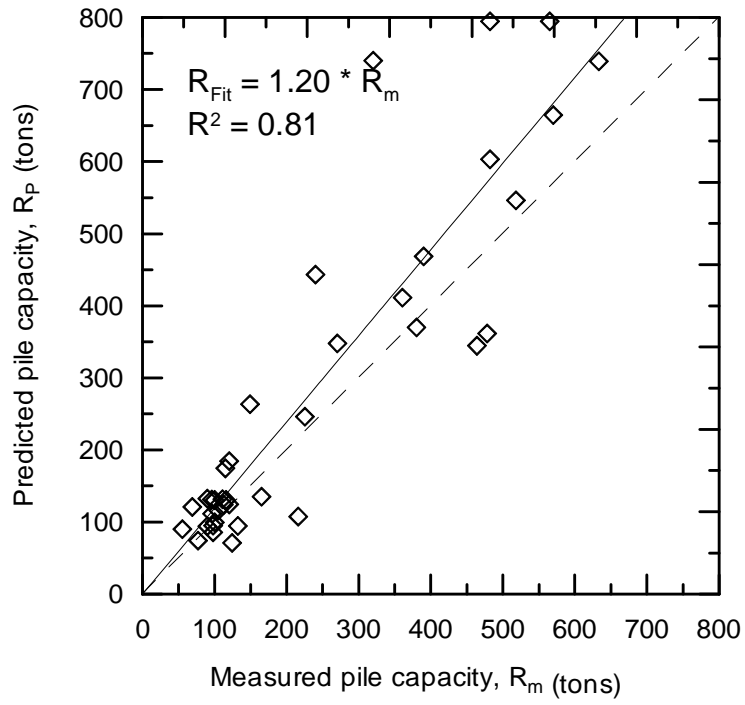


Figure 11

R_m versus R_p (Schmertmann method)

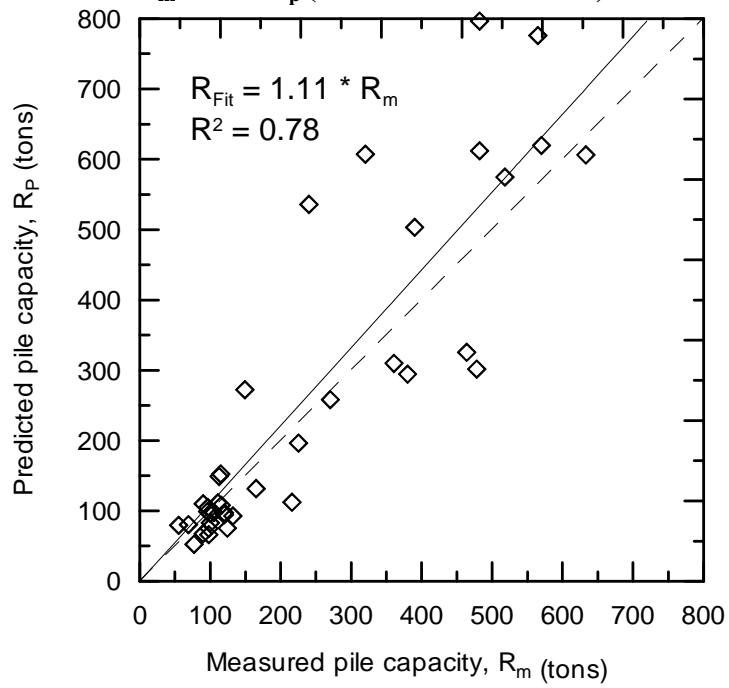


Figure 12

R_m versus R_p (LCPC method)

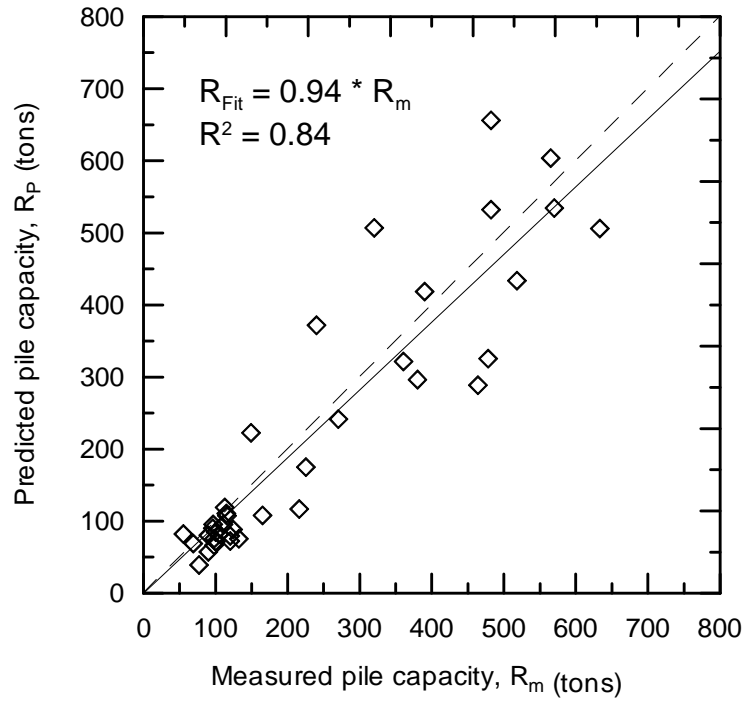


Figure 13
 R_m versus R_p (De Ruiter method)

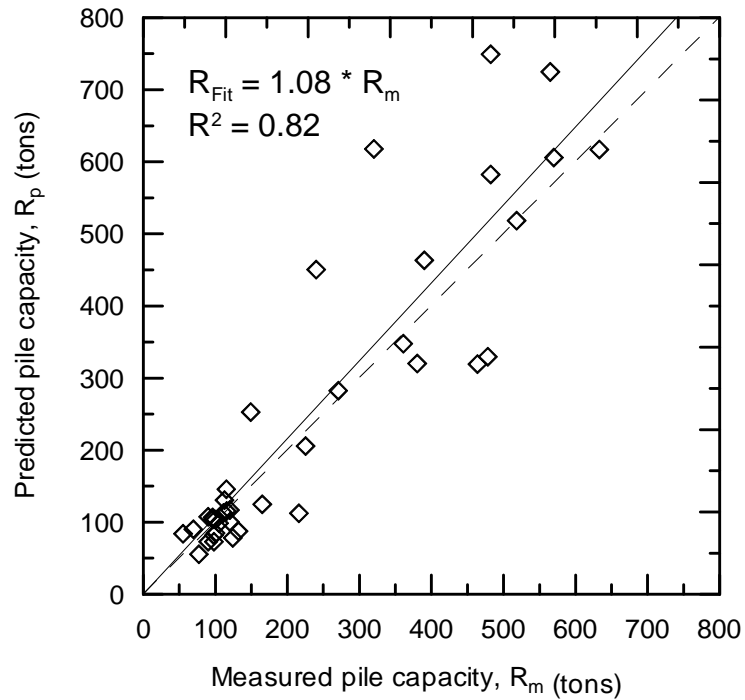


Figure 14
 R_m versus R_p (CPT - Average method)

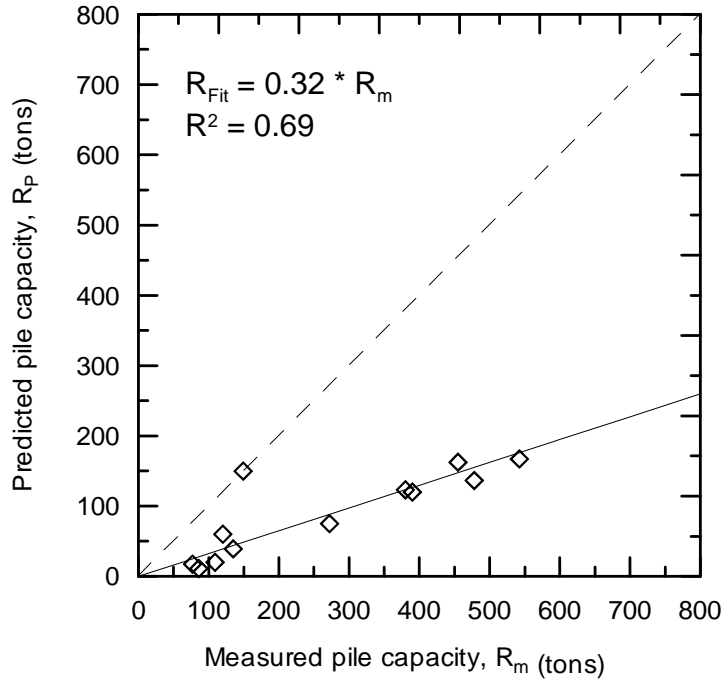


Figure 15
 R_m versus R_p (CAPWAP-EOD)

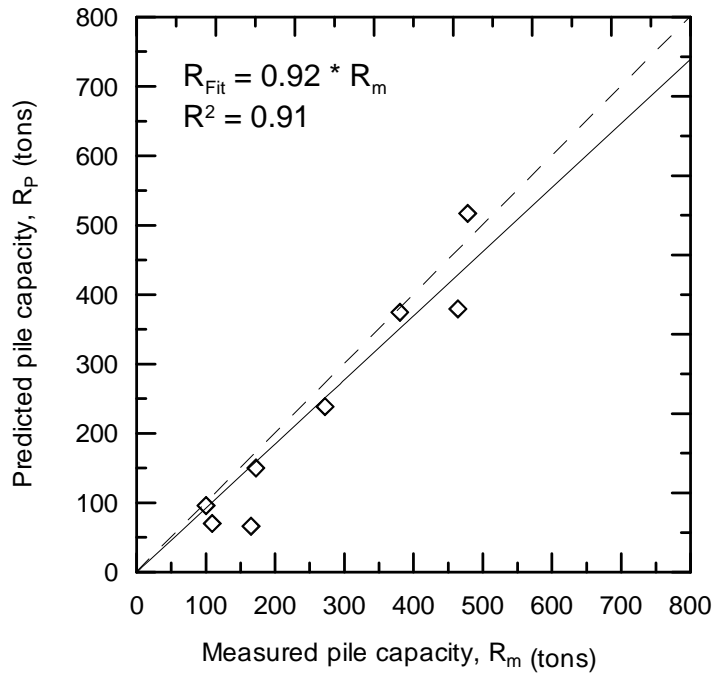


Figure 16
 R_m versus R_p (CAPWAP-14 days BOR)

LRFD Calibration

Figures 17 to 23 present the histogram and the normal and log-normal distribution of the measured to predicted pile resistance (R_m/R_p) of the static method, three CPT methods, the average of three CPT methods, and dynamic measurement with signal matching analysis (CAPWAP), respectively. Also Figures 24 to 30 illustrate the cumulative distribution function (CDF) of the bias values for each design method. As shown in the figures, generally the log-normal distribution matches the histogram and CDF better than the normal distribution. In addition, the resistance bias factor ($\lambda_R = R_m/R_p$) can range theoretically from 0 to infinity with an optimum value of one; therefore, the distribution of the resistance bias can be assumed to follow a log-normal distribution [42]. In this study, the log-normal distribution was used to evaluate the different methods based on prediction accuracy and to calibrate the resistance factors.

Reliability analyses were conducted and the resistance factors for all pile design methods were calibrated at $Q_D/Q_L = 3$. This ratio was selected since the reliability index (β) converges for Q_D/Q_L exceeding 3 [23]. Figures 31 through 37 present the resistance factors determined for various reliability indexes (β) for the different pile design methods. As shown in the figures, the resistance factors (ϕ) determined by the advanced methods (FORM and Monte Carlo simulation method) are relatively close and generally higher than the resistance factors (ϕ) obtained from FOSM.

A review of the literature indicates that the required target reliability indexes are between 2.33 and 3 for geotechnical applications. The resistance factors (ϕ) determined by FOSM, FORM, and the Monte Carlo simulation method for different pile design methods corresponding to reliability index of 2.33 are tabulated in Table 6. The resistance factors for the static method determined in this study are 0.56 (FOSM) and 0.63 (FORM and Monte Carlo simulation method), which is higher than the ϕ value recommended by AASHTO (e.g., $\phi = 0.34-0.45$ for static method, and $\phi = 0.50$ for Schmertmann method) [11]. It should be noted that this value is only valid for subsurface conditions similar to Louisiana soils that consist mainly of soft cohesive soils with some cohesionless inter-layering soils. For this condition, the driven pile resistance was determined using the α -Tomlinson method dominantly and the Nordlund method was employed for cohesionless inter-layering soils. Among the three direct CPT methods, the De Ruiter and Beringen method shows the highest resistance factor [$\phi = 0.66$ (FOSM), 0.74 (FORM), and 0.73 (Monte Carlo simulation method)], while the Schmertmann method shows the lowest resistance factor [$\phi = 0.44$ (FOSM), 0.48 (FORM), and 0.49 (Monte Carlo simulation)], which is lower than the

AASHTO value of 0.5. For the dynamic measurement with signal matching analysis, the resistance factors obtained for the CAPWAP (EOD) is 1.31 (FOSM) and 1.41 (FORM), which is higher than those of CAPWAP (14 days BOR) of 0.55 (FOSM), 0.61 (FORM) and 0.62 (Monte Carlo simulation method). This is mainly due to the pile setup effect. Although the CAPWAP (EOD) has a high resistance factor, it is not an economical and reliable approach because it significantly underestimates the resistance and has low efficiency factors of 0.36 (FOSM) and 0.39 (FORM). The dynamic measurement is mainly used for pile drivability rather than for design.

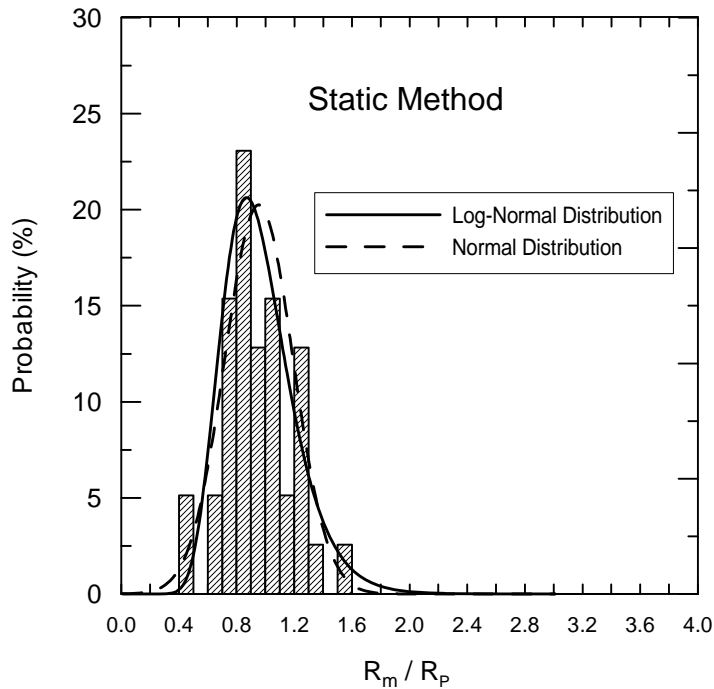


Figure 17
Histogram and PDF of resistance bias factors (Static method)

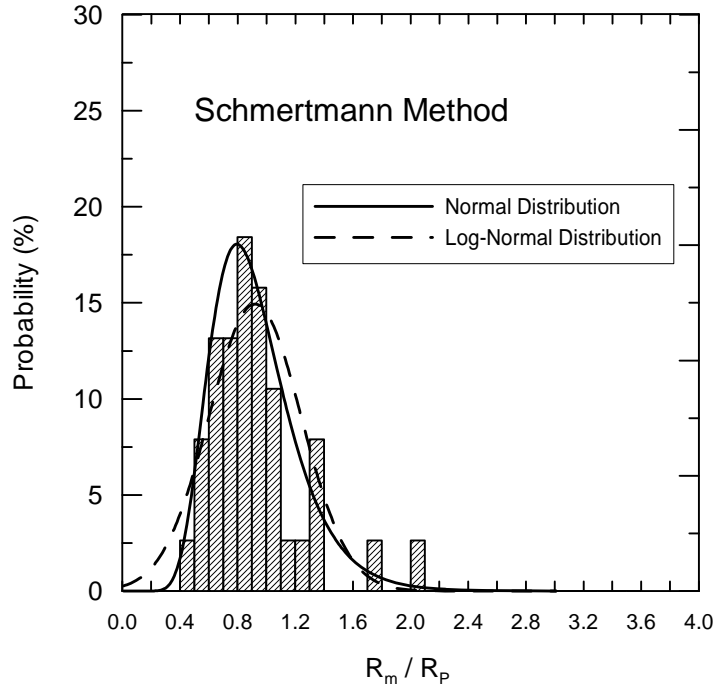


Figure 18
Histogram and PDF of resistance bias factors (Schmertmann method)

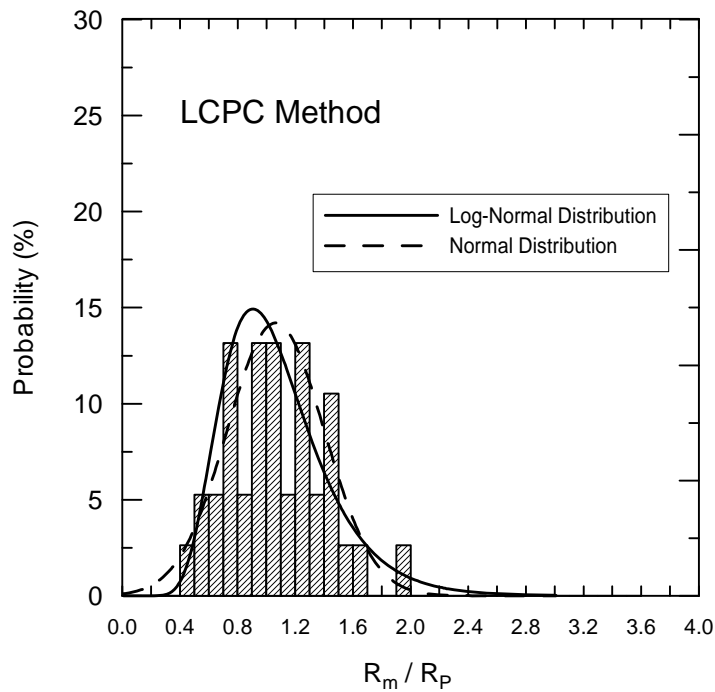


Figure 19
Histogram and PDF of resistance bias factors (LCPC method)

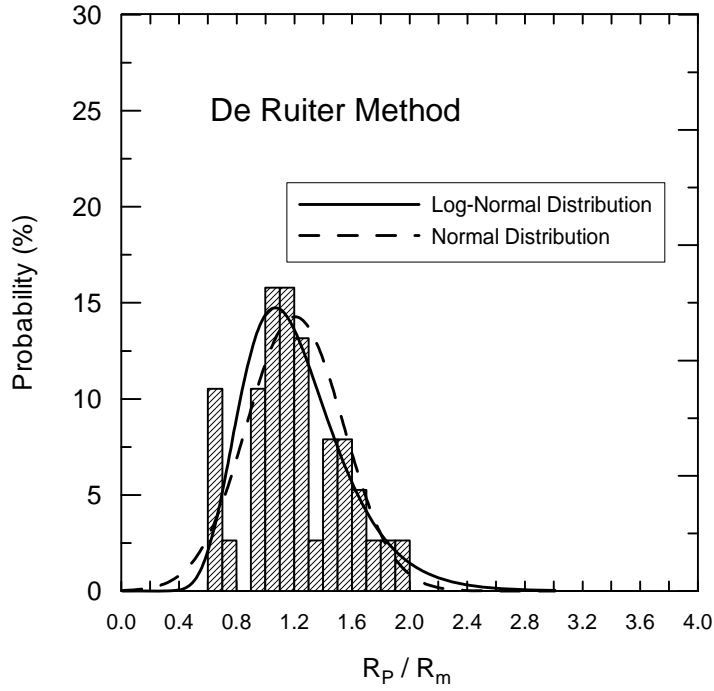


Figure 20
Histogram and PDF of resistance bias factors (De Ruiter method)

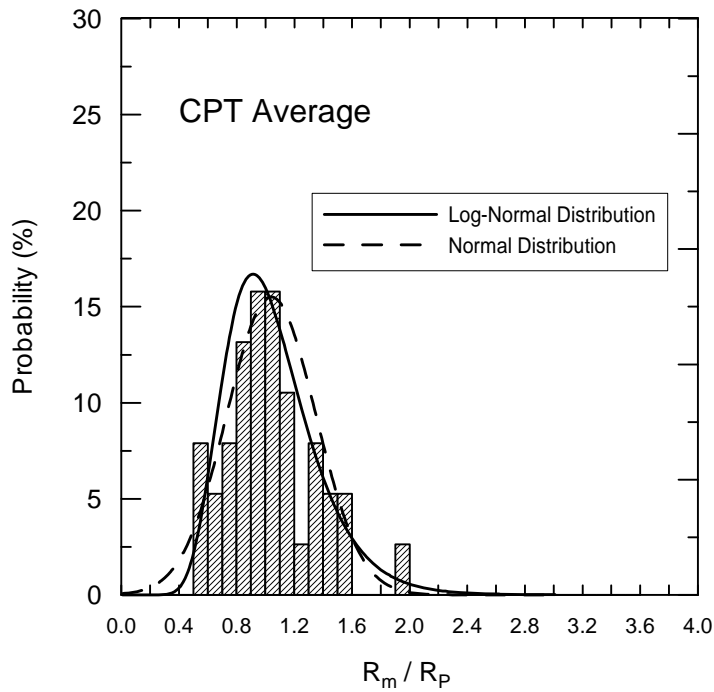


Figure 21
Histogram and PDF of resistance bias factors (CPT - Average method)

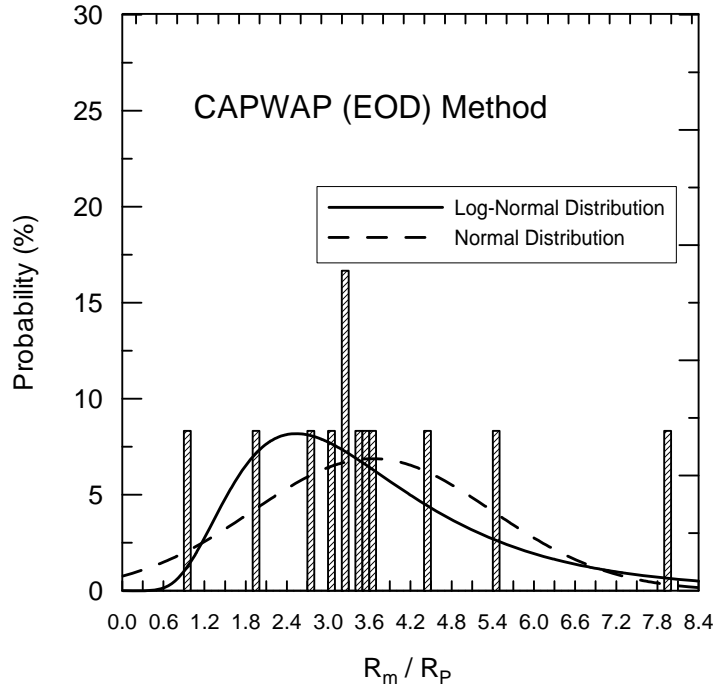


Figure 22
Histogram and PDF of resistance bias factors (CAPWAP-EOD)

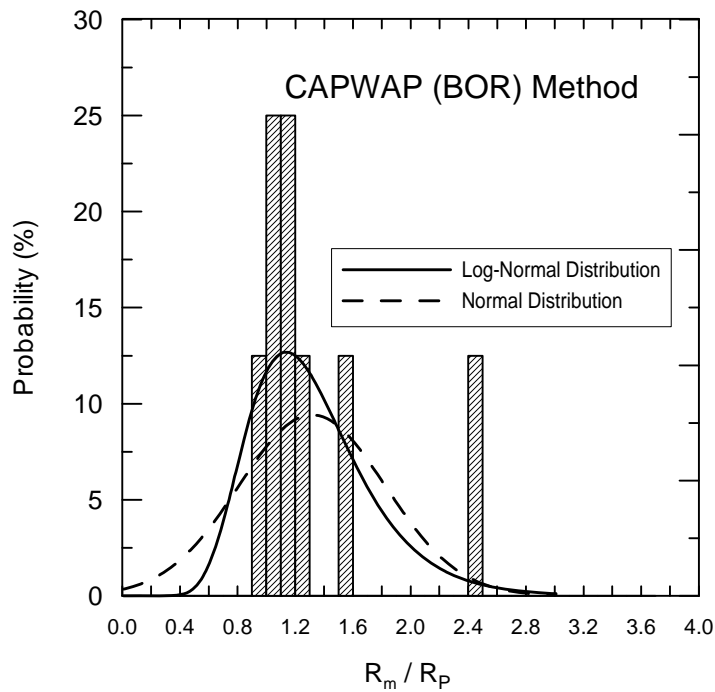


Figure 23
Histogram and PDF of resistance bias factors (CAPWAP-14 days BOR)

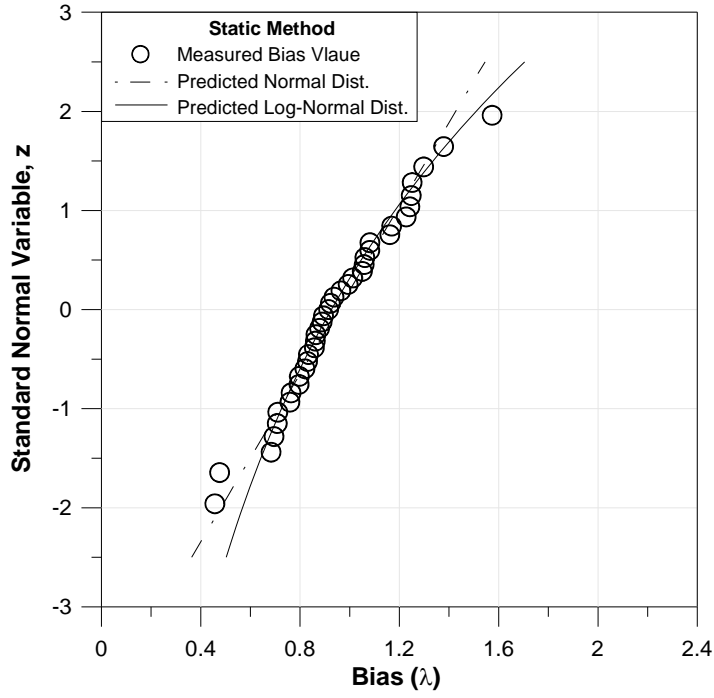


Figure 24
Cumulative distribution function (CDF) of bias values (Static method)

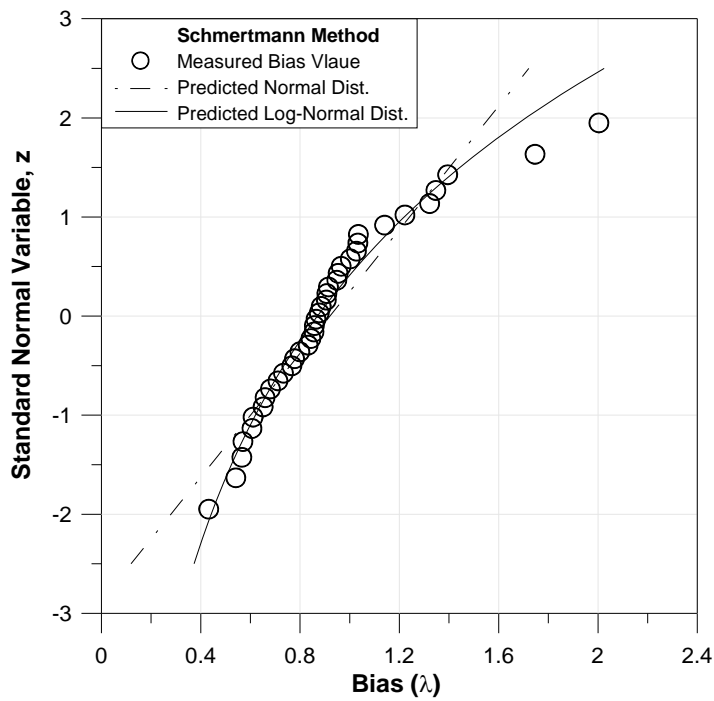


Figure 25
Cumulative distribution function (CDF) of bias values (Schmertmann method)

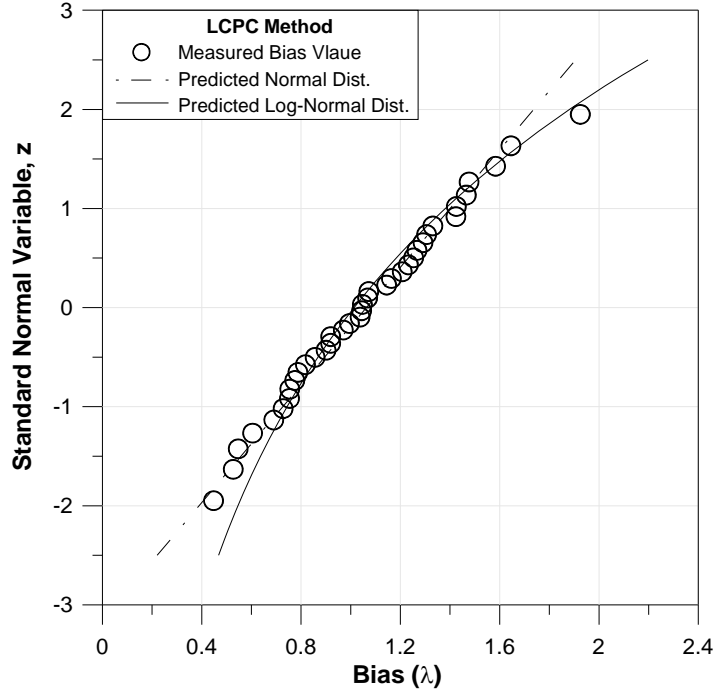


Figure 26
Cumulative distribution function (CDF) of bias values (LCPC method)

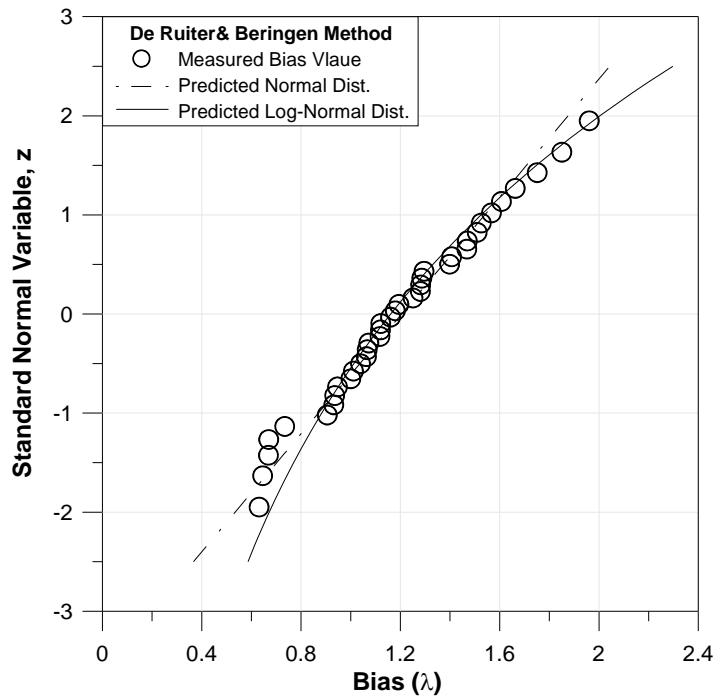


Figure 27
Cumulative distribution function (CDF) of bias values (De Ruiter method)

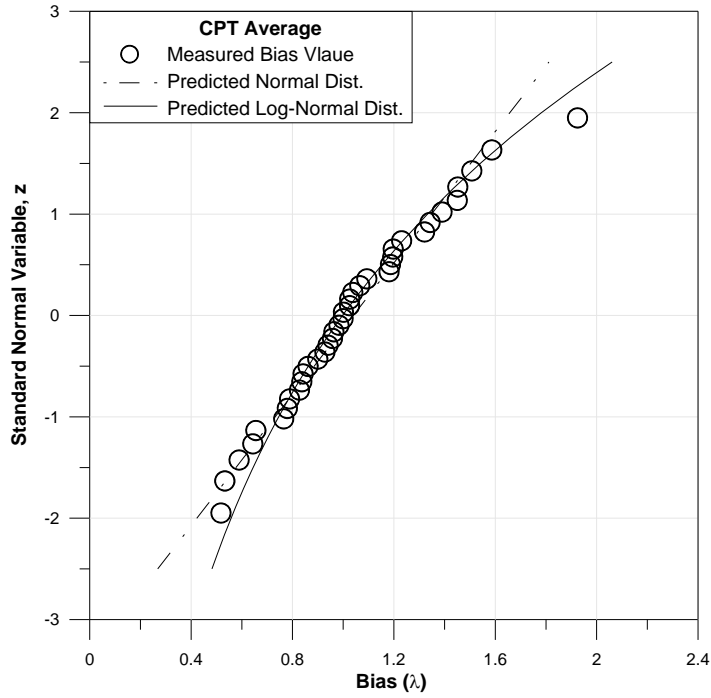


Figure 28
Cumulative distribution function (CDF) of bias values (CPT-Average method)

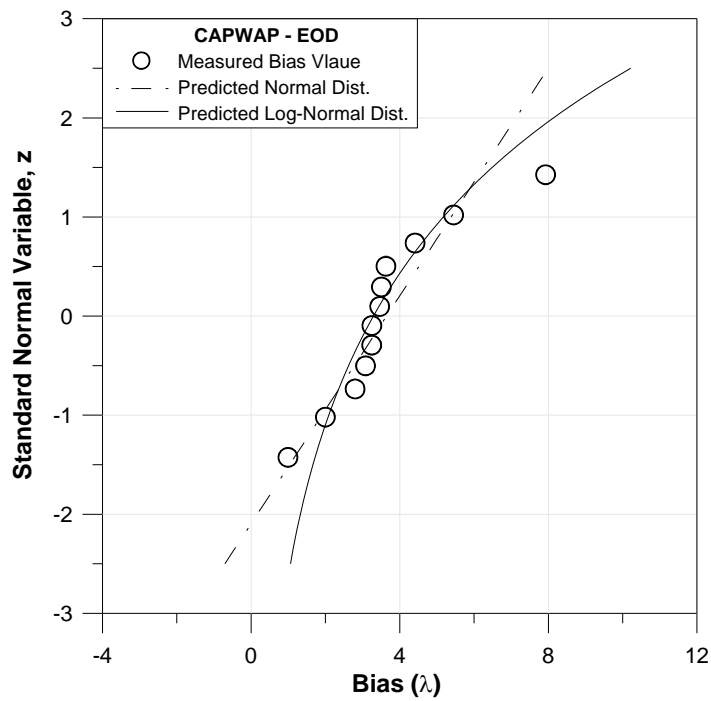


Figure 29
Cumulative distribution function (CDF) of bias values (CAPWAP-EOD)

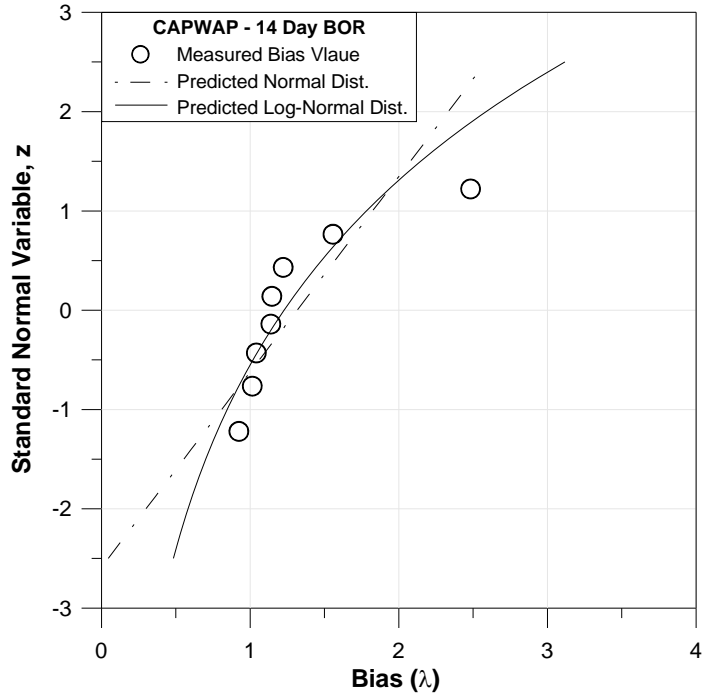


Figure 30
Cumulative distribution function (CDF) of bias values (CAPWAP-14 days BOR)

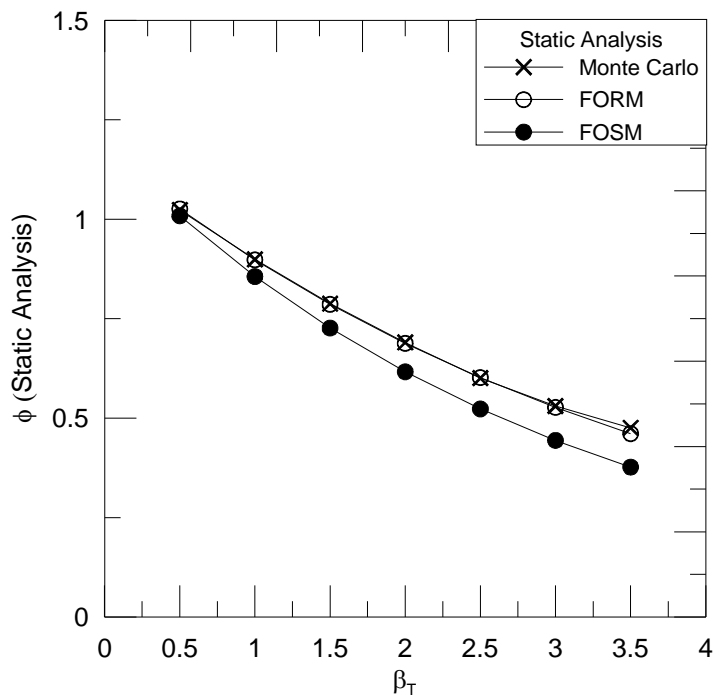


Figure 31
Resistance factors for different reliability indexes (Static method)

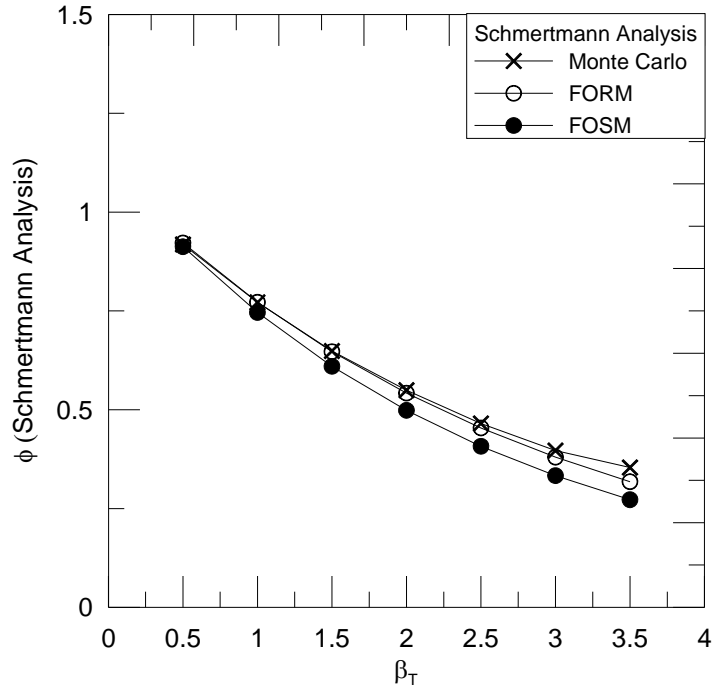


Figure 32

Resistance factors for different reliability indexes (Schmertmann method)

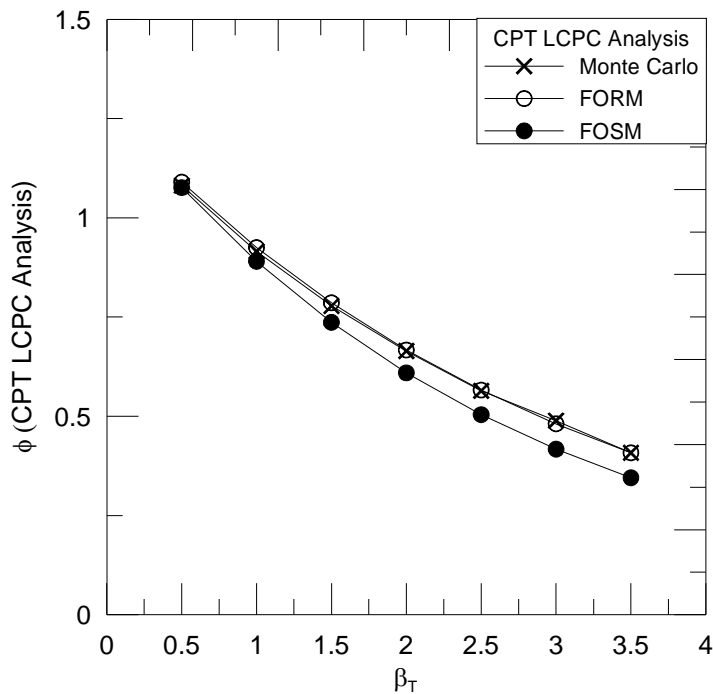


Figure 33

Figure 26 Resistance factors for different reliability indexes (LCPC method)

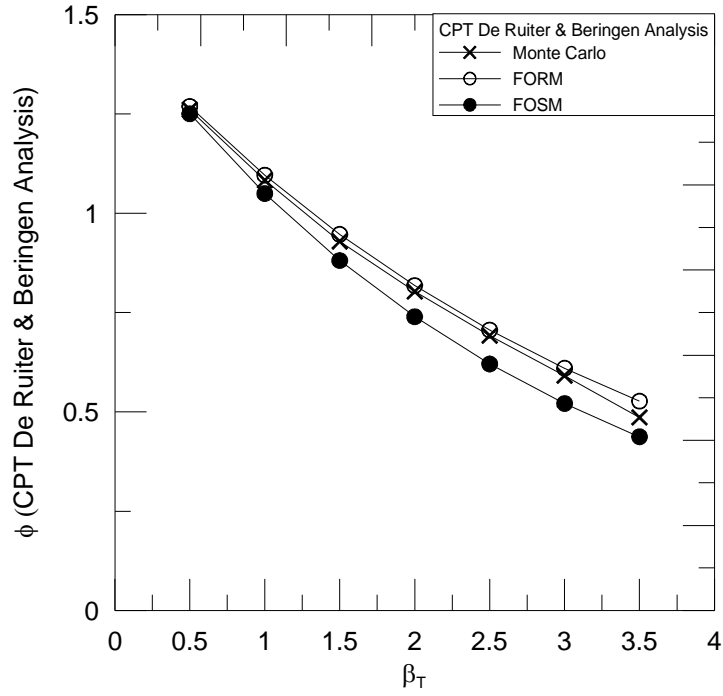


Figure 34
Resistance factors for different reliability indexes (De Ruiter method)

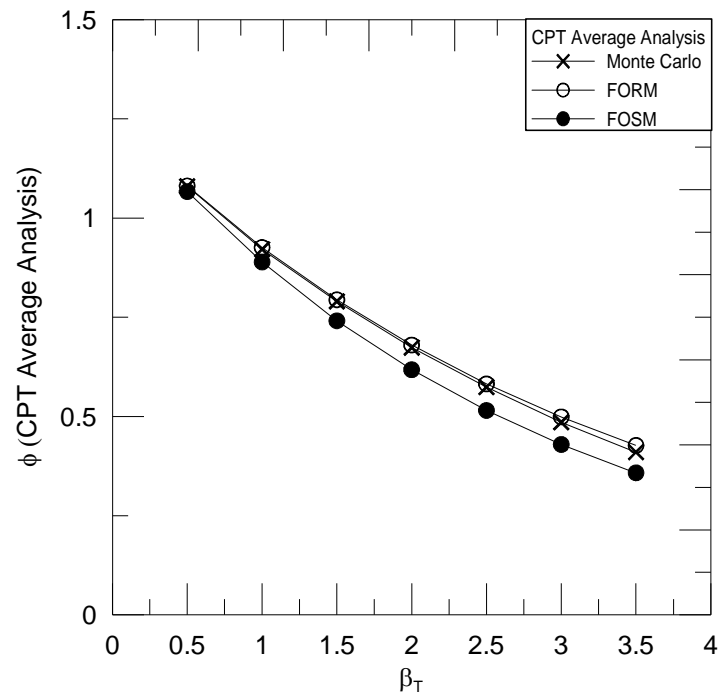


Figure 35
Resistance factors for different reliability indexes (CPT - Average method)

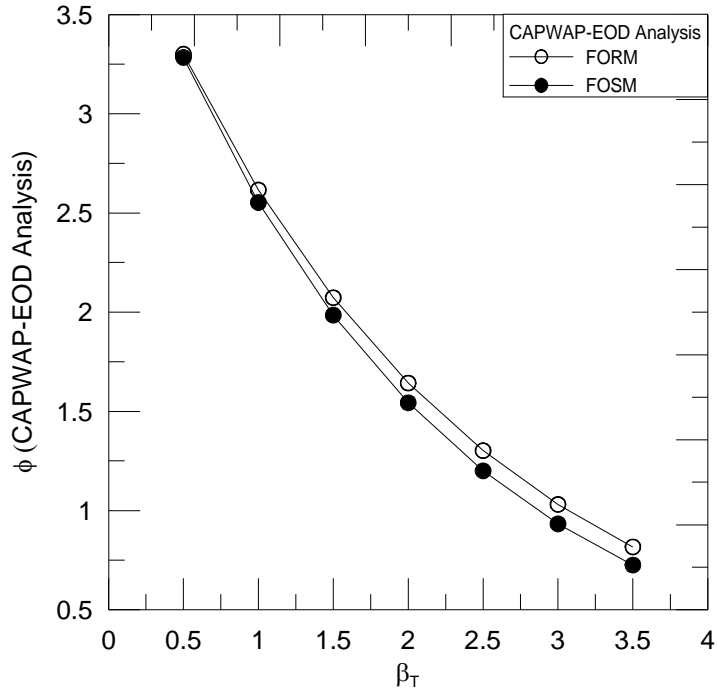


Figure 36
Resistance factors for different reliability indexes (CAPWAP-EOD)

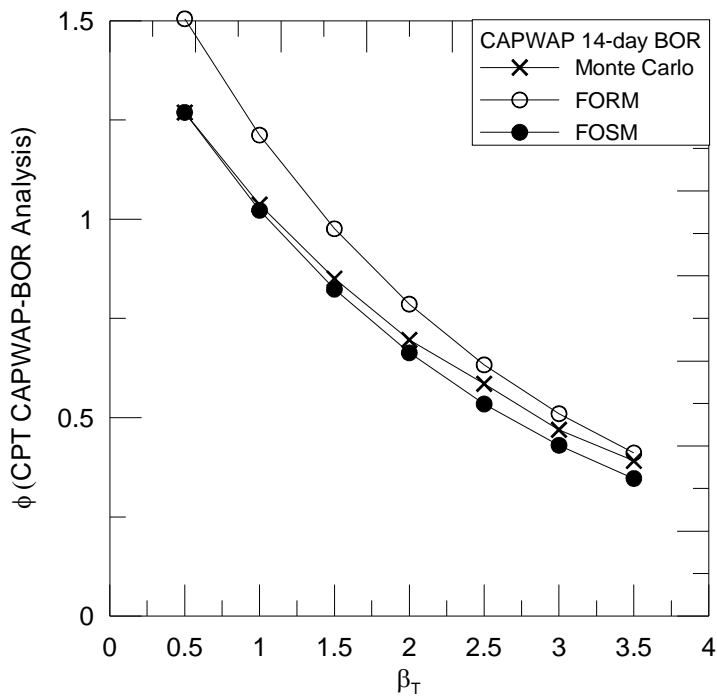


Figure 37
Resistance factors for different reliability indexes (CAPWAP-14 days BOR)

Table 6
Resistance Factors (ϕ) for Driven Piles ($\beta_T = 2.33$)

Design Method		Proposed Resistance Factor (ϕ) and Efficiency Factor (ϕ/λ) for Louisiana Soil						Resistance Factor, ϕ [AASHTO (11)]
		FOSM		FORM		Monte Carlo Simulation		
		ϕ	ϕ/λ	ϕ	ϕ/λ	ϕ	ϕ/λ	
Static method	α -Tomlinson method and Nordlund method	0.56	0.58	0.63	0.66	0.63	0.66	0.35-0.45
Direct CPT method	Schmertmann	0.44	0.47	0.48	0.52	0.49	0.53	0.50
	LCPC/LCP	0.54	0.51	0.60	0.56	0.59	0.56	—
	De Ruyter and Beringen	0.66	0.55	0.74	0.62	0.73	0.61	—
	CPT average	0.55	0.53	0.61	0.59	0.62	0.59	—
Dynamic measurement	CAPWAP (EOD)	1.31	0.36	1.41	0.39	—	—	—
	CAPWAP (14 days BOR)	0.55	0.44	0.61	0.52	0.62	0.47	0.65

CONCLUSIONS

This study presents a reliability based evaluation of different design methods for predicting the ultimate axial resistance of piles driven into Louisiana soils. Resistance factors (ϕ) of single driven piles needed to implement the LRFD design methodology were determined for each design method. A pile load test (a soil database of 53 square PPC piles of different sizes and lengths) that was tested to failure was collected from LADOTD archives and used to calibrate the resistance factors. For each pile load test, the measured ultimate pile resistance was estimated using the Davisson interpretation method and modified Davisson method for piles with a larger size. In addition, the load carrying resistance of each pile was predicted using the static method, three CPT methods (Schmertmann, De Ruiter-Beringen, and LCPC methods), the average of three CPT methods, and the dynamic CAPWAP (EOD and BOR) methods.

Statistical analyses comparing the predicted and measured pile resistances were conducted to evaluate the performance of the different pile design methods. The results of the statistical analyses showed that the static method (α -Tomlinson method and Nordlund method) over-predicted the pile resistance by 12 percent. Among the three direct CPT methods, the De Ruiter-Beringen method was the most consistent prediction method with the lowest COV. Both dynamic measurements with signal matching analysis methods (CAPWAP-EOD and 14 days BOR) showed under-predication of pile resistance with a setup factor of 2.9.

Reliability analyses based on FOSM, FORM, and the Monte Carlo simulation method were conducted to calibrate the resistance factors (ϕ) for the investigated pile design methods. These factors are needed to comply with the FHWA mandate in the LRFD design of single driven piles. The design input parameters were adopted from the AASHTO LRFD design specifications for bridge substructure. The resistance factors (ϕ) that corresponded to a dead load to live load ratio (Q_D/Q_L) of 3 as a function of target reliability index (β_T) were presented. Based on the results of reliability analyses for $\beta_T = 2.33$, the De Ruiter-Beringen method showed the highest resistance factor [$\phi_{\text{De-Ruiter}} = 0.66$ (FOSM), 0.74 (FORM), and 0.73 (Monte Carlo simulation method)], while the Schmertmann method showed the lowest resistance factor [$\phi_{\text{Schmertmann}} = 0.44$ (FOSM), 0.48 (FORM), and 0.49 (Monte Carlo simulation method)], which is lower than the AASHTO recommended value of 0.5. The resistance factors obtained for the CAPWAP (EOD) were 1.31 (FOSM), 1.41 (FORM), which is higher than CAPWAP (14 day BOR) resistance factors of 0.55 (FOSM), 0.61 (FORM), and 0.62 (Monte Carlo simulation method). This is mainly due to pile setup. Although the CAPWAP (EOD) has a high resistance factor, it is not an economical and

reliable approach because it significantly underestimates the resistance and has a low efficiency factor. However, the dynamic measurement is mainly used for pile drivability rather than for design.

RECOMMENDATIONS

1. LADOTD engineers need to start implementing the resistance factors (ϕ) recommended for the different pile design methods based on Louisiana pile load test—a soil database in the LRFD design of driven piles for all future state projects.
2. It is recommended to select a few projects to demonstrate the cost benefit study and comparison between the LRFD design and the traditional ASD design.
3. It is recommended to hold a workshop to train LADOTD engineers in the LRFD design of pile foundations.
4. It is recommended to continue collecting pile load test data from new projects, especially for those cases in which the end bearing and side frictional capacities can be separated for possible future re-calibration of resistance factors of different pile design methods every two years.
5. Since the Co-PI's are from the geotechnical design section at LADOTD, they are already in the process of implementing previously mentioned recommendation items 1, 2, and 3 based on the findings of this research project.
6. It should be noted that performing complete reliability analyses of a pile foundation requires the inclusions of all risk factors. Scour is a critical factor in the selection of pile tip elevations. The risk associated with scouring directly impacts the reliability of pile foundations. This is mainly due to expected changes on the in-situ stress state (overburden and stress history) of the subsurface soil that will affect the laboratory and in-situ test results. However, the scope of this study does not include the evaluation of scour and is recommended to be considered in the future.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO	American Association of Highway and Transportation Officials
ASD	Allowable Stress Design
BOR	Beginning of Restrike
CAPWAP	Case Pile Wave Analysis Program
EOD	End of Driving
CDF	Cumulative Distribution Function
COV	Coefficient of Variation
CPT	Cone Penetration Test
DOT	Department of Transportation
EOD	End of Driving
FHWA	Federal Highway Administration
FORM	First Order Reliability Method
FOSM	First Order Second Moment
FS	Factor of Safety
LADOTD	Louisiana Department of Transportation and Development
LCPC	Laboratoire Central Des Ponts et Chaussées
LDP-CPT	Louisiana Pile Design by Cone Penetration Test
LRFD	Load and Resistance Factor Design
LTRC	Louisiana Transportation Research Center
MC	Monte Carlo Method
NC	Normally Consolidated
OC	Overconsolidated
PDF	Probability Density Function
PPC	Precast-Prestressed Concrete
SLS	Serviceability Limit State
SPT	Standard Penetration Test
ULS	Ultimate Limit State or Strength Limit State

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APPENDIX

Summary of Geotechnical Data for the State Projects Investigated

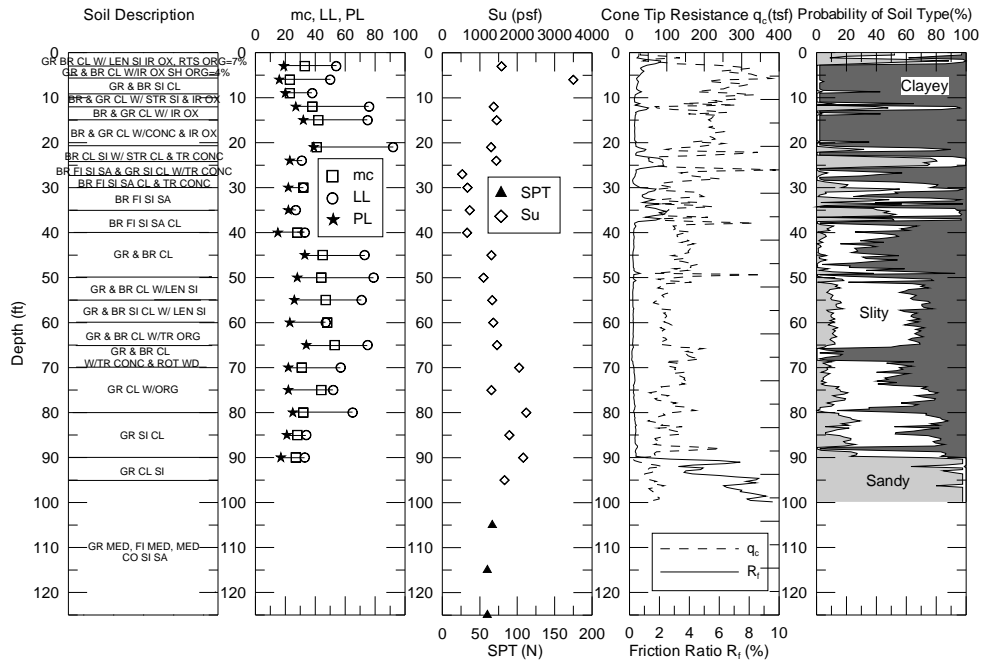


Figure 38
003-07-0019 TP#1

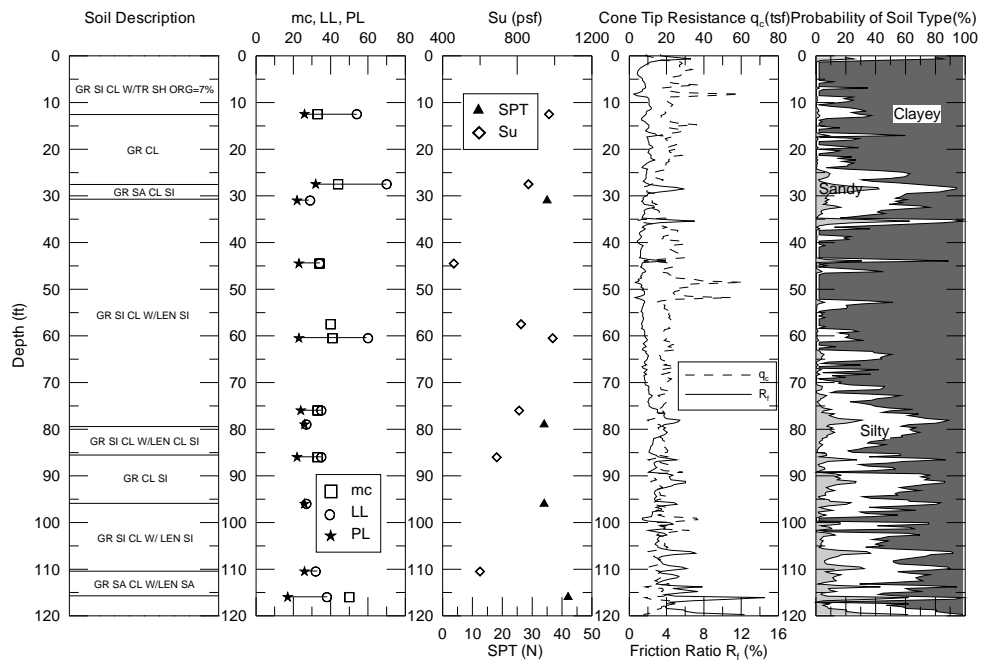


Figure 39
005-01-0056 TP#1

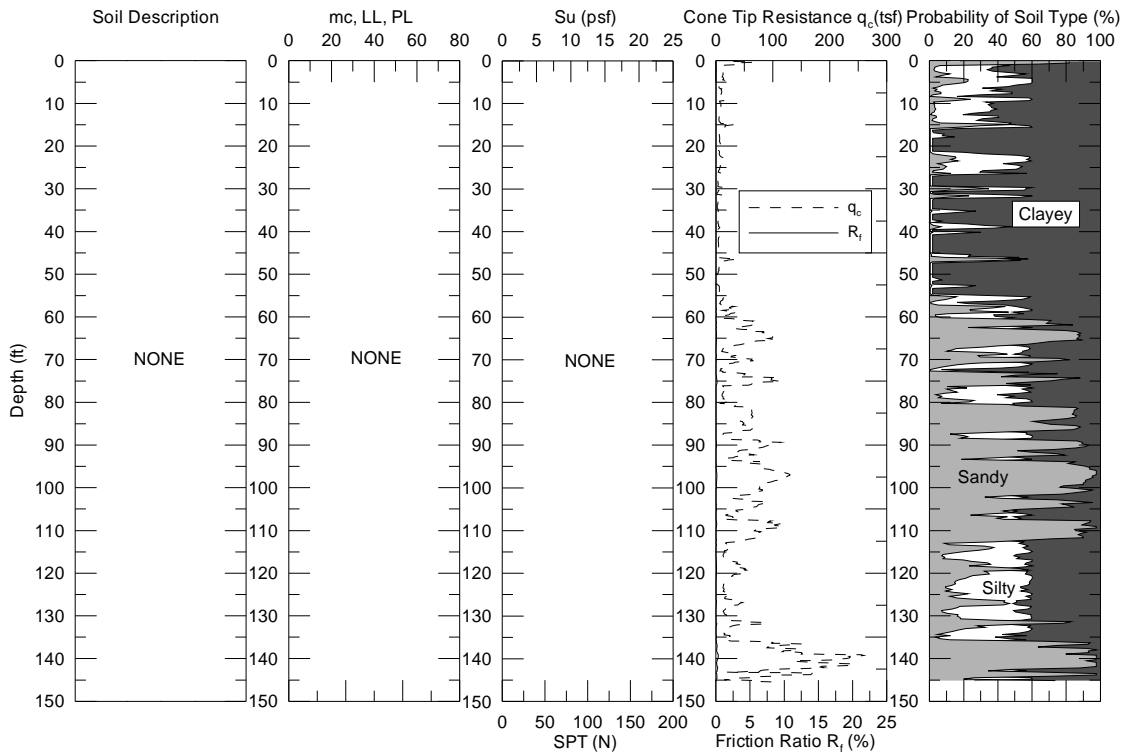


Figure 40
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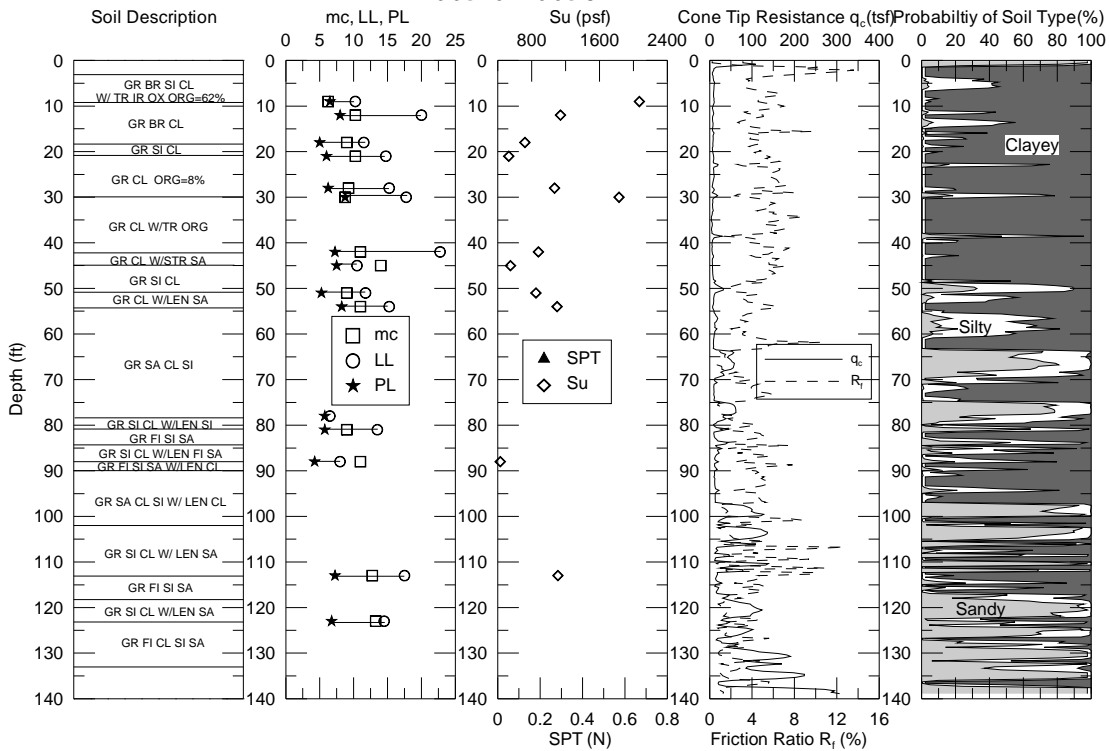


Figure 41
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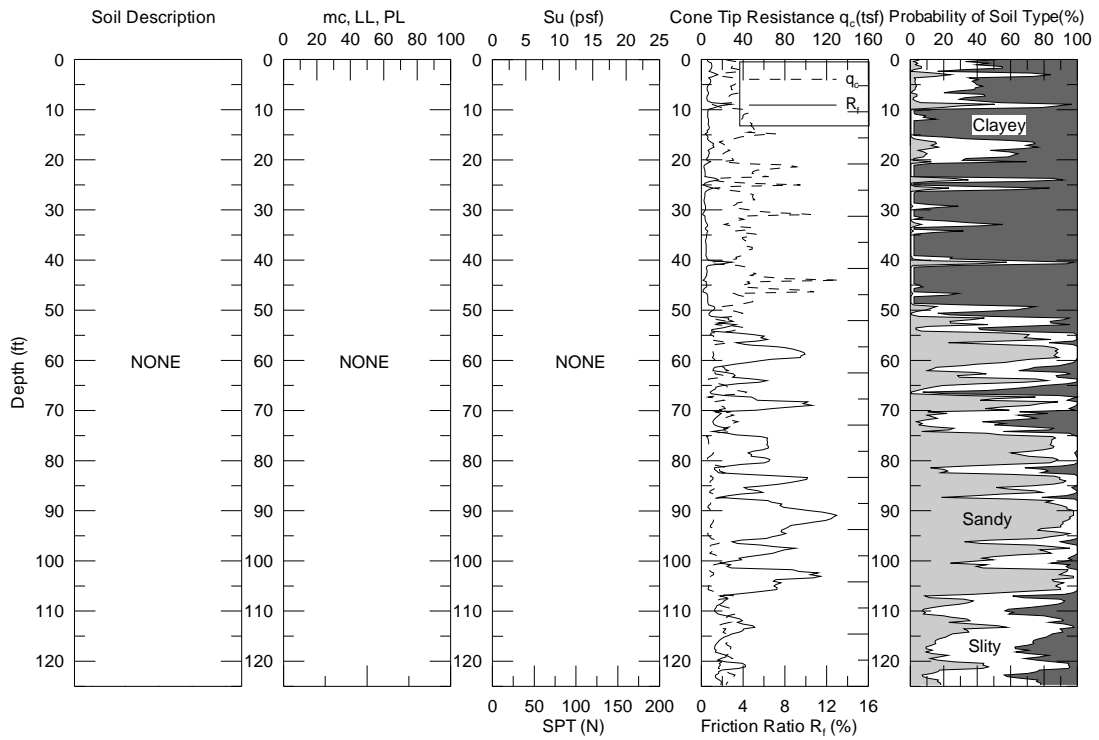


Figure 42
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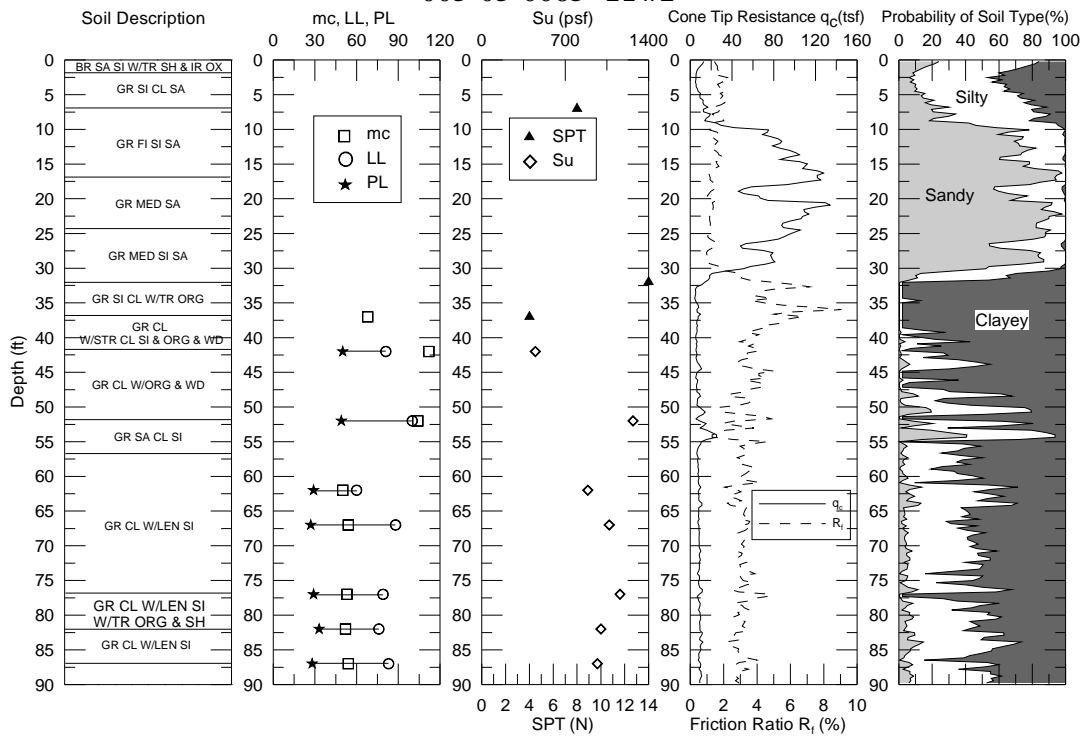


Figure 43
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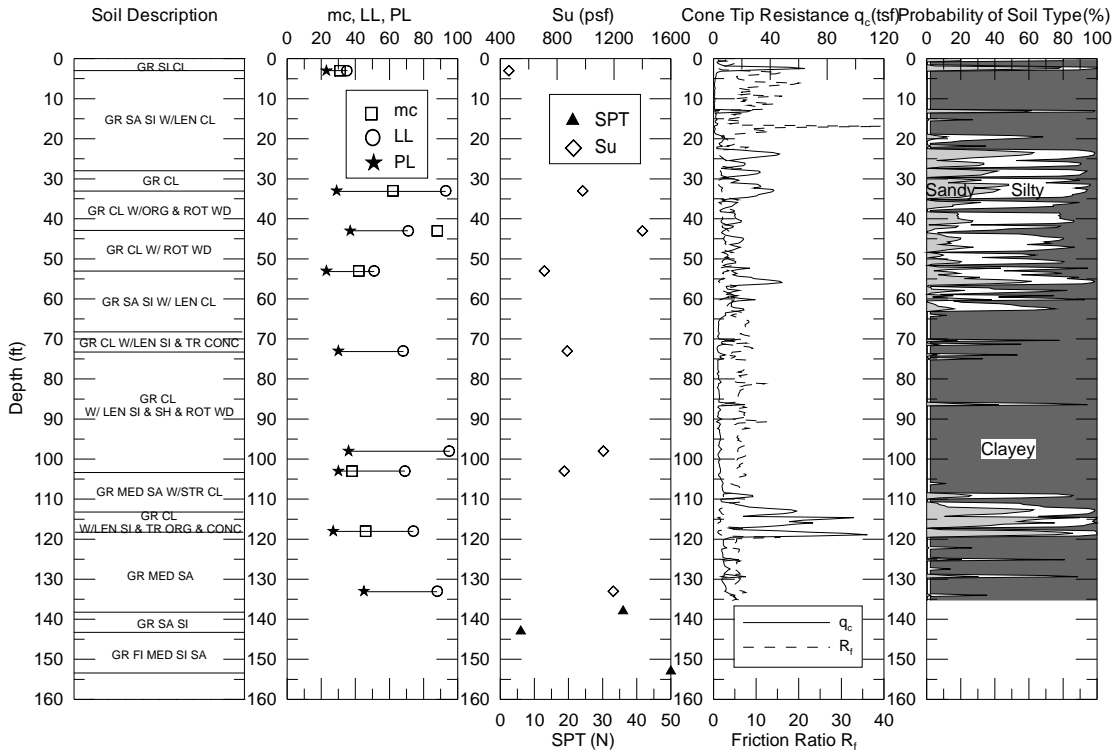


Figure 44
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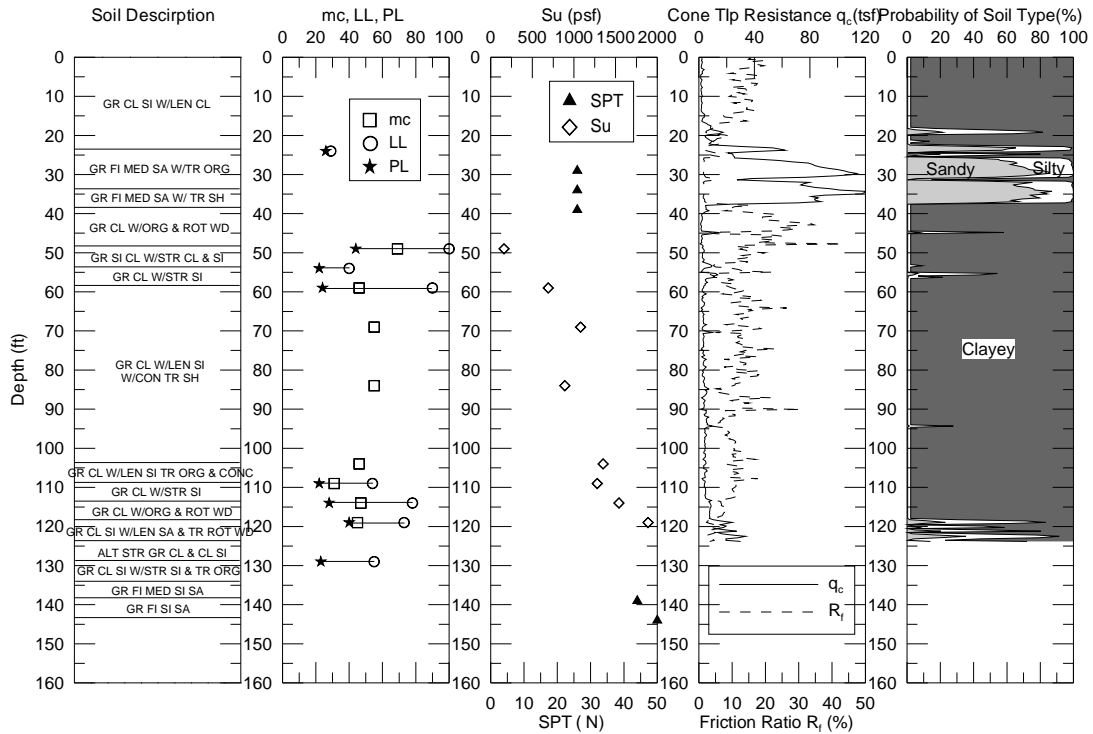


Figure 45
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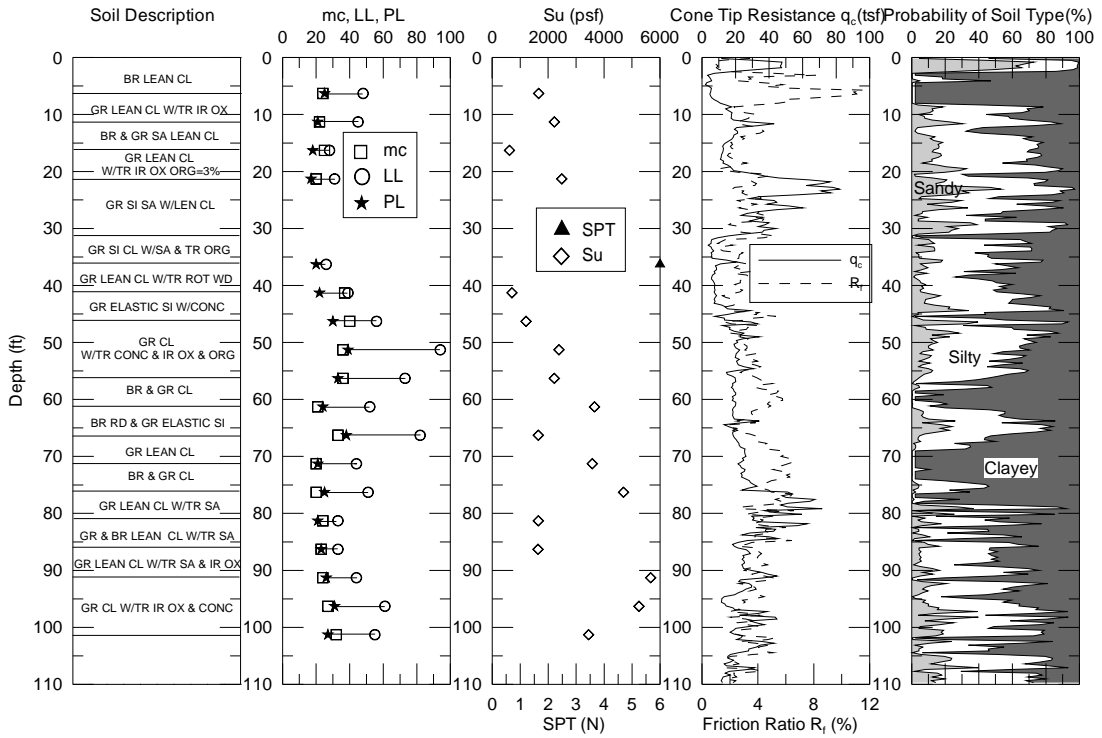


Figure 46
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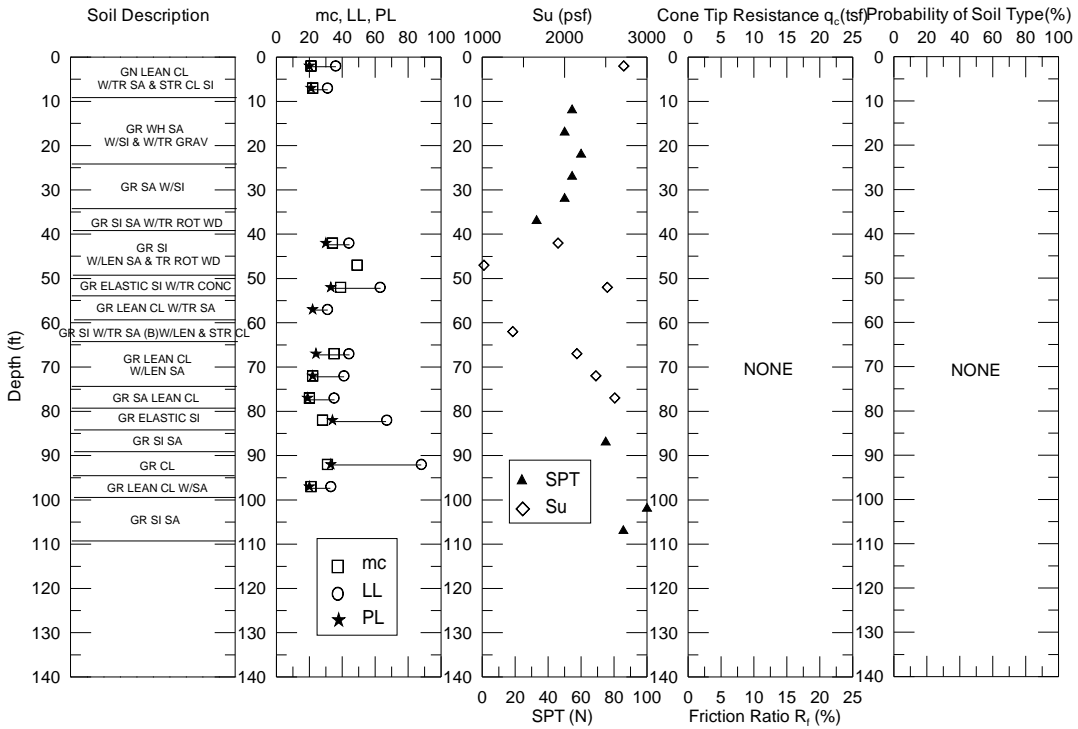


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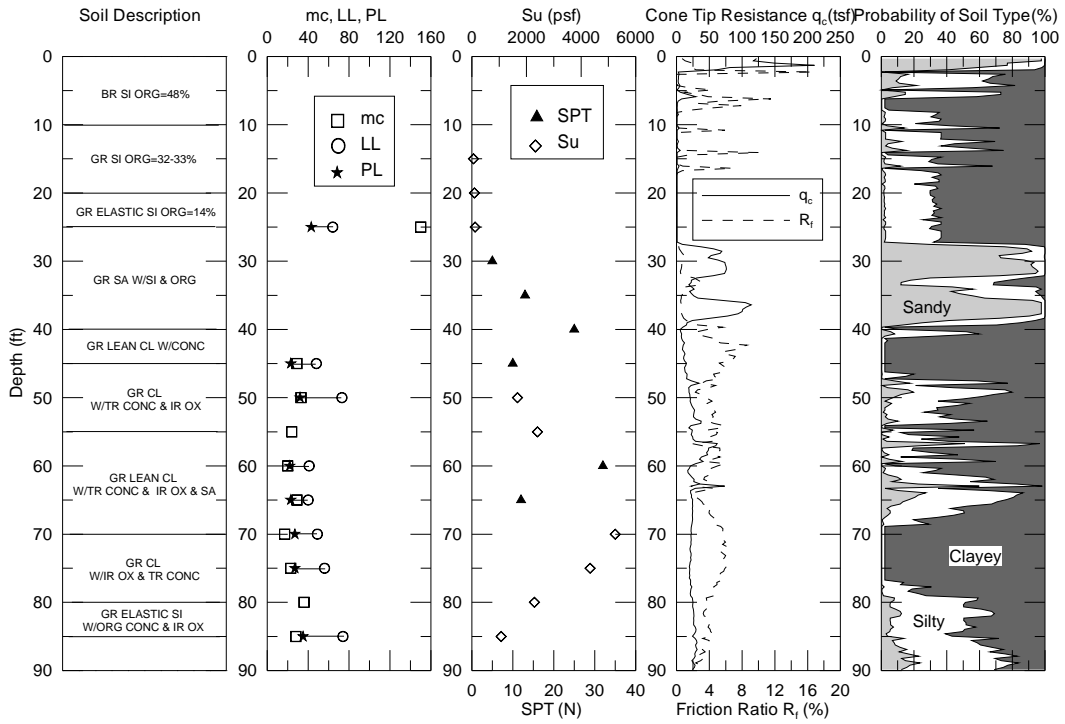


Figure 48
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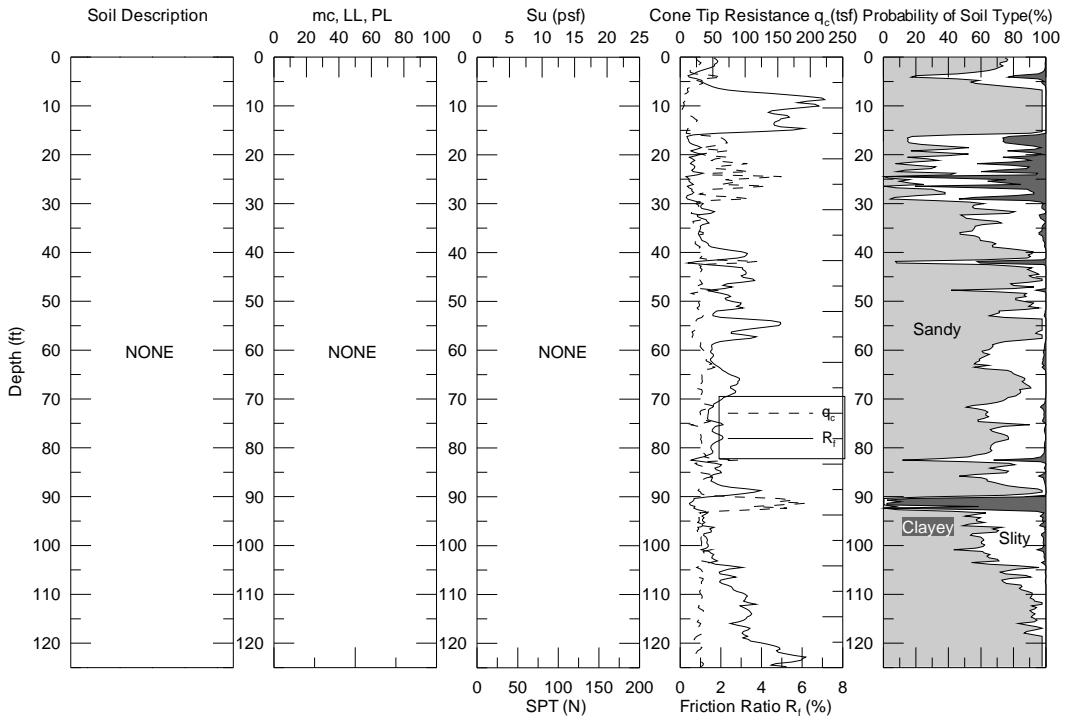


Figure 49
262-06-0009 & 262-07-0012 TP#1

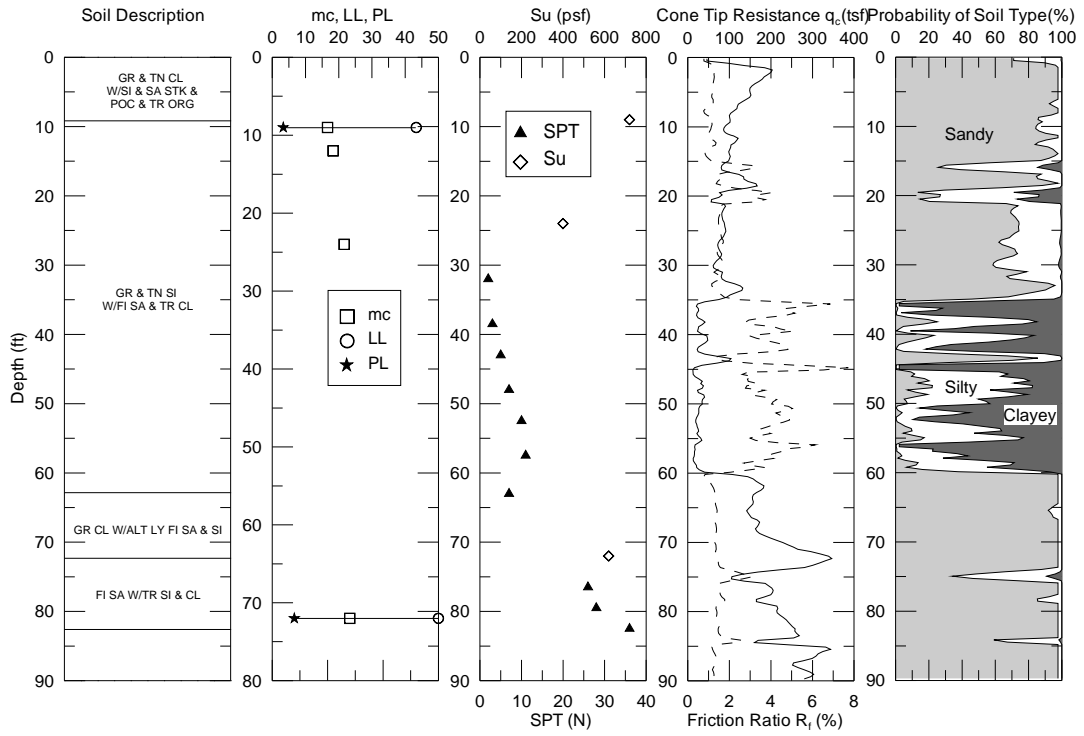


Figure 50
424-05-0078 TP#1

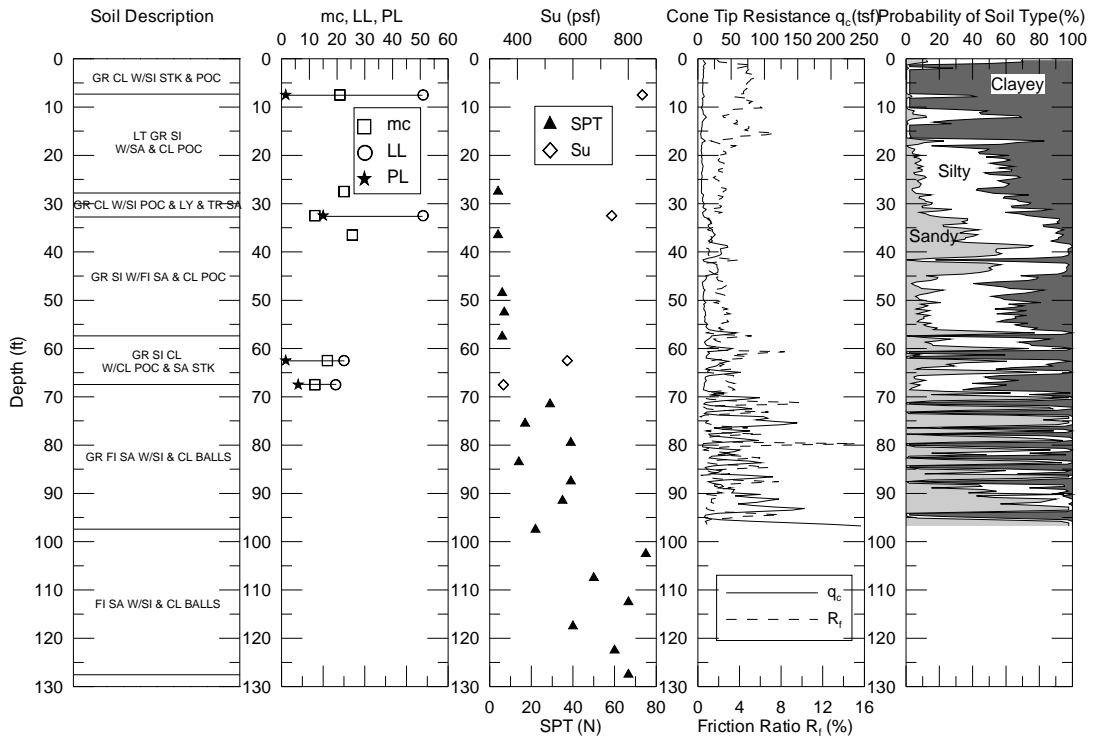


Figure 51
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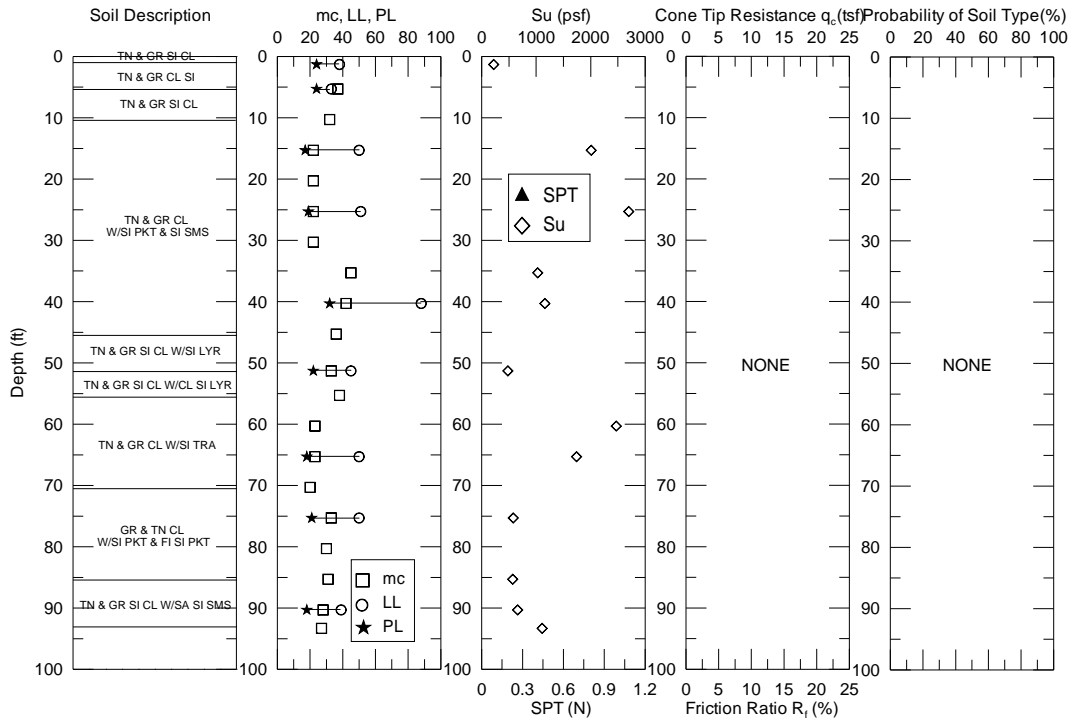


Figure 52
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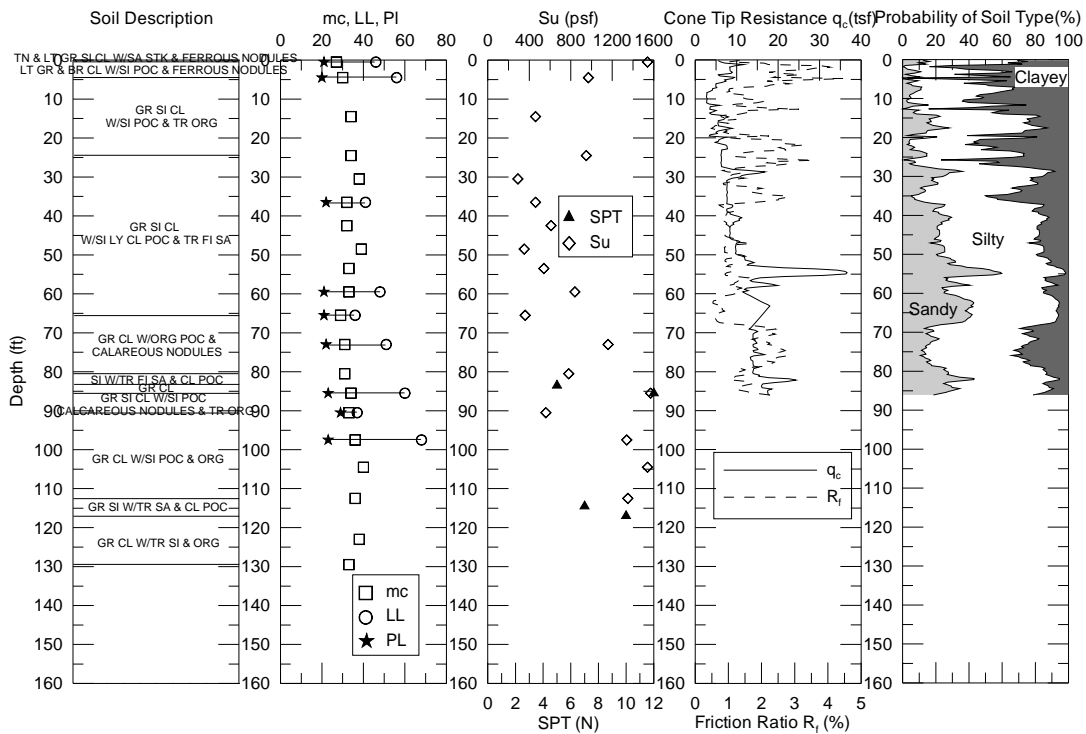


Figure 53
424-05-0078 TP#5

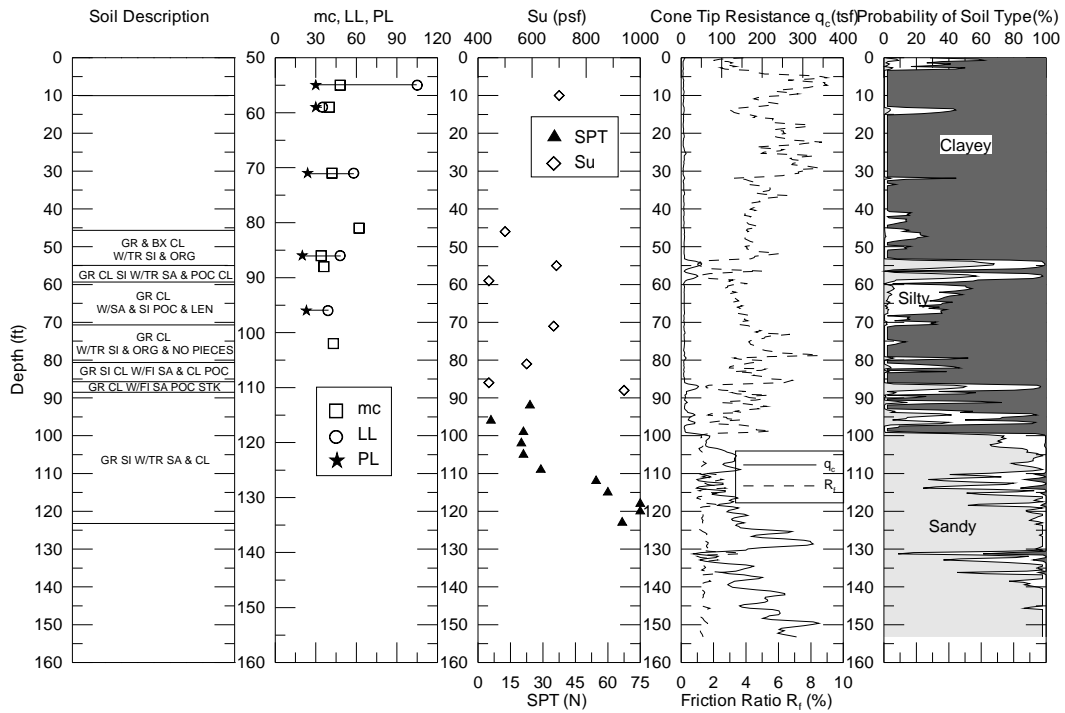


Figure 54
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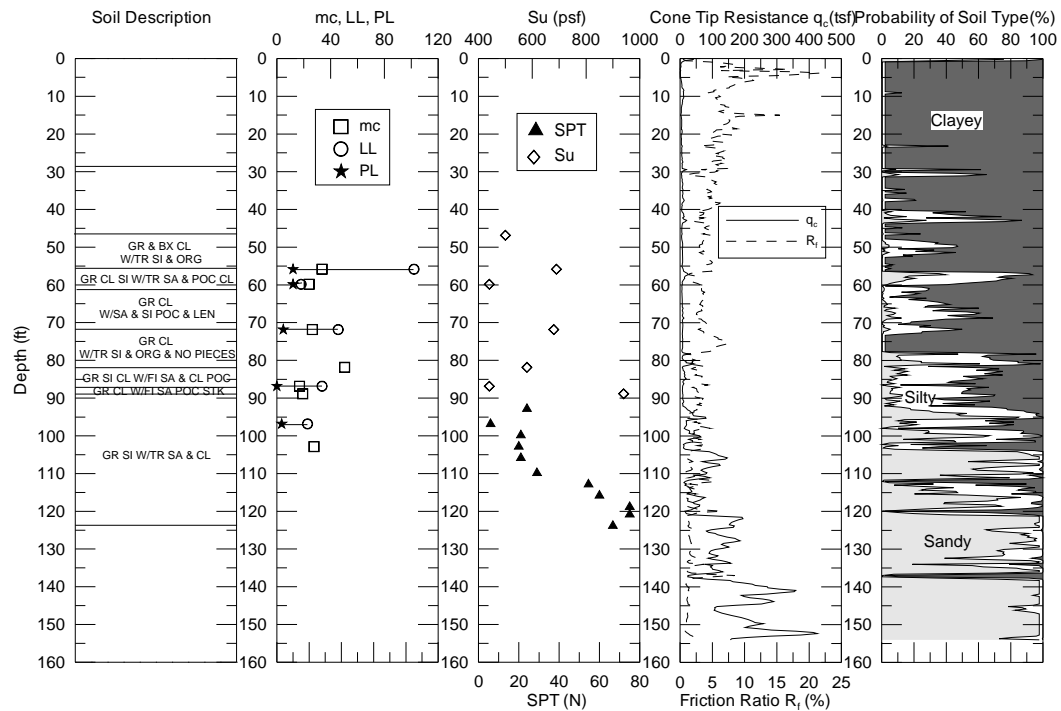


Figure 55
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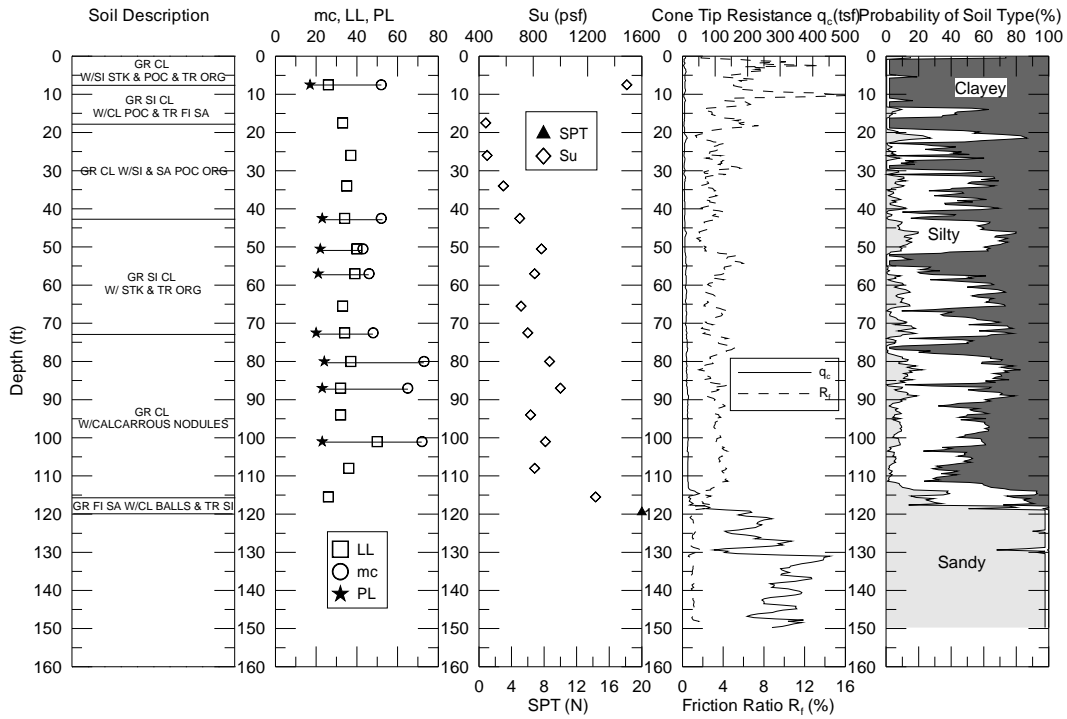


Figure 56
424-06-0005 TP#1

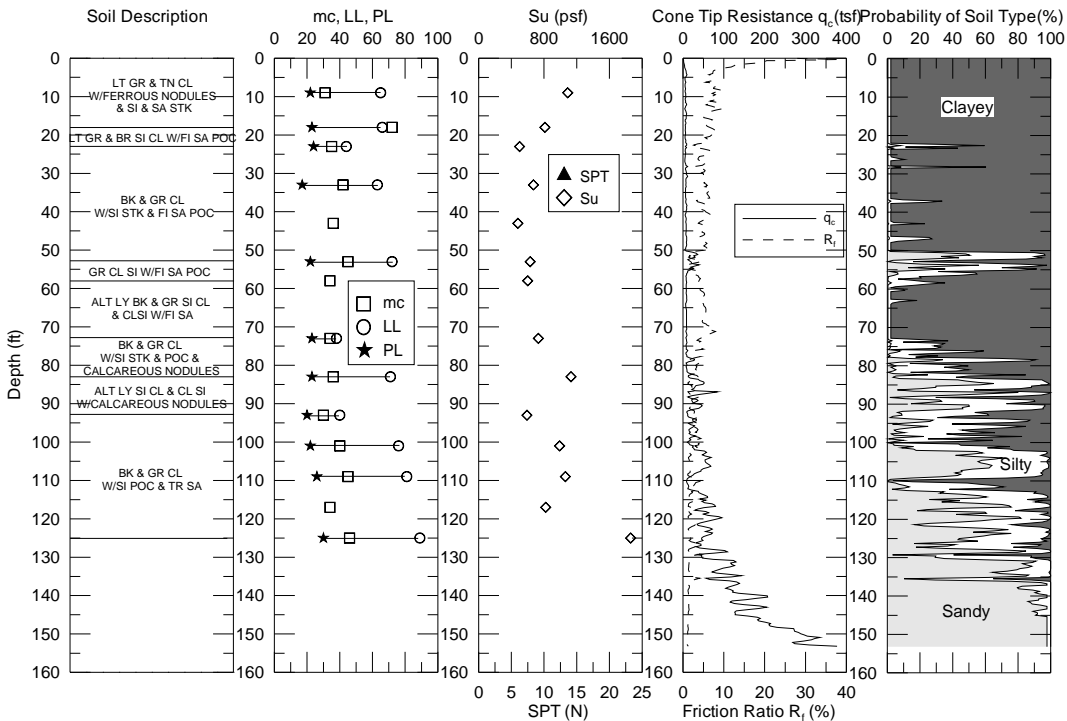


Figure 57
424-06-0005 TP#2

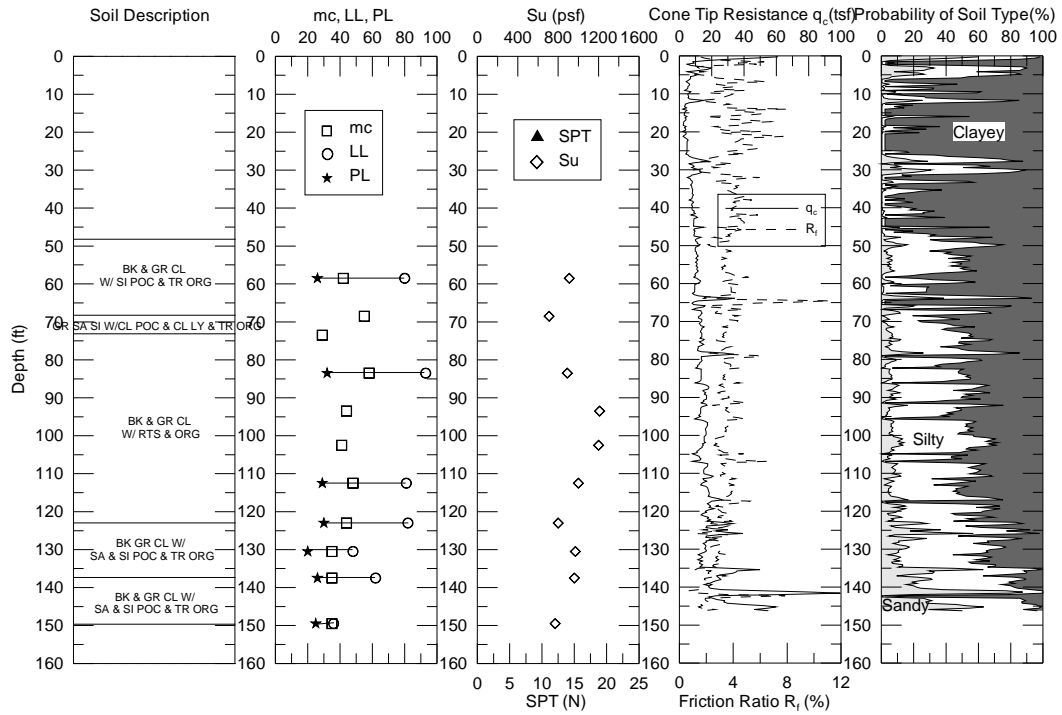


Figure 58
424-07-0021 TP#1

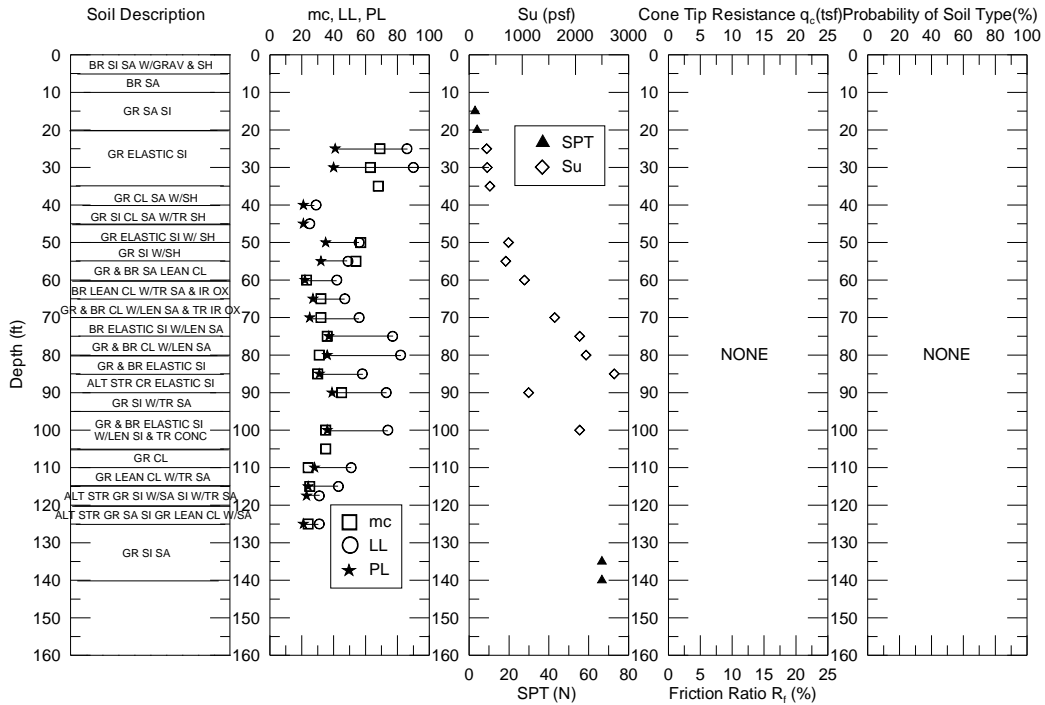


Figure 59
450-15-0085 TP#3A

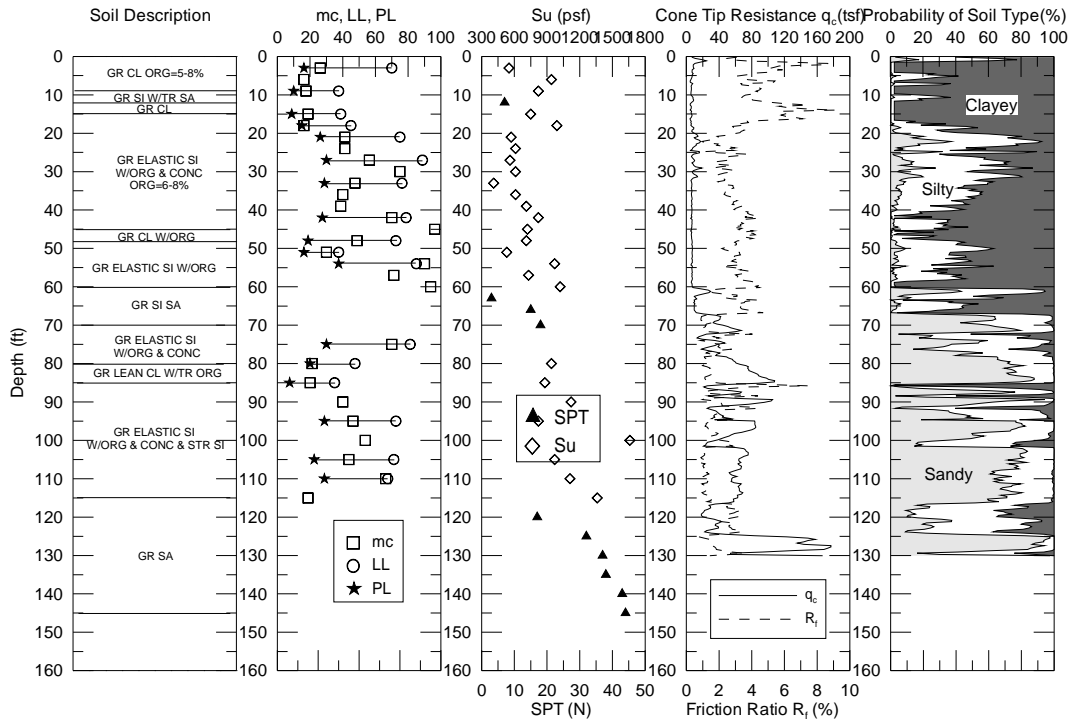


Figure 60
424-05-0087 TP#1

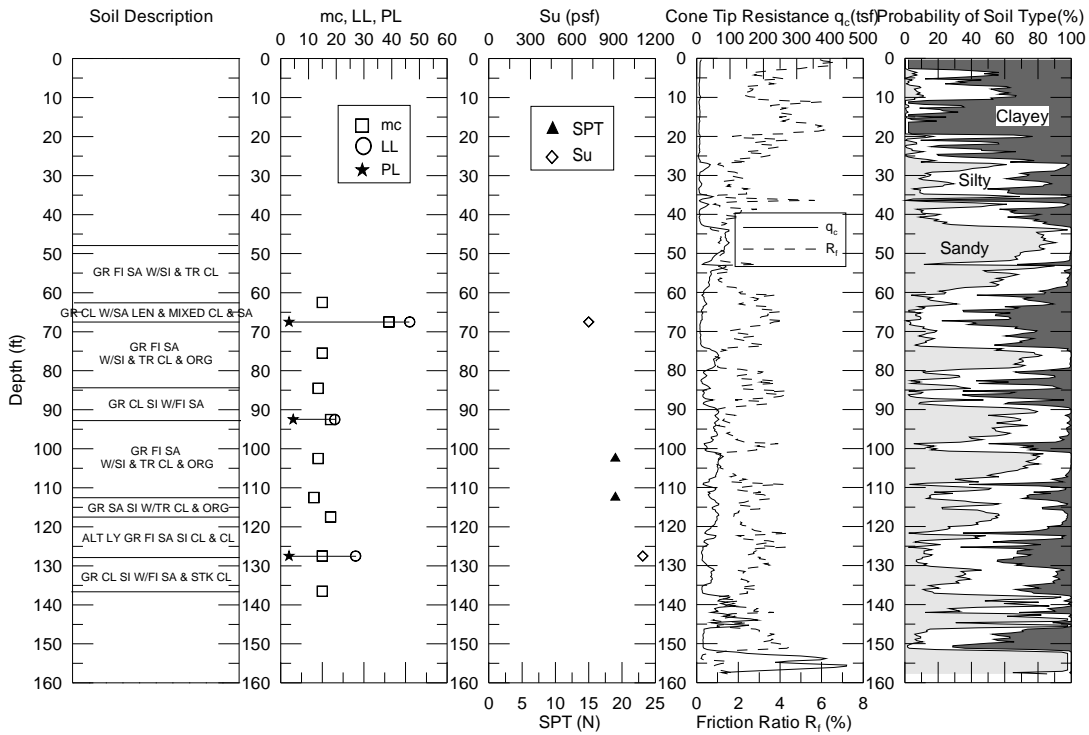


Figure 61
424-05-0087 TP#3

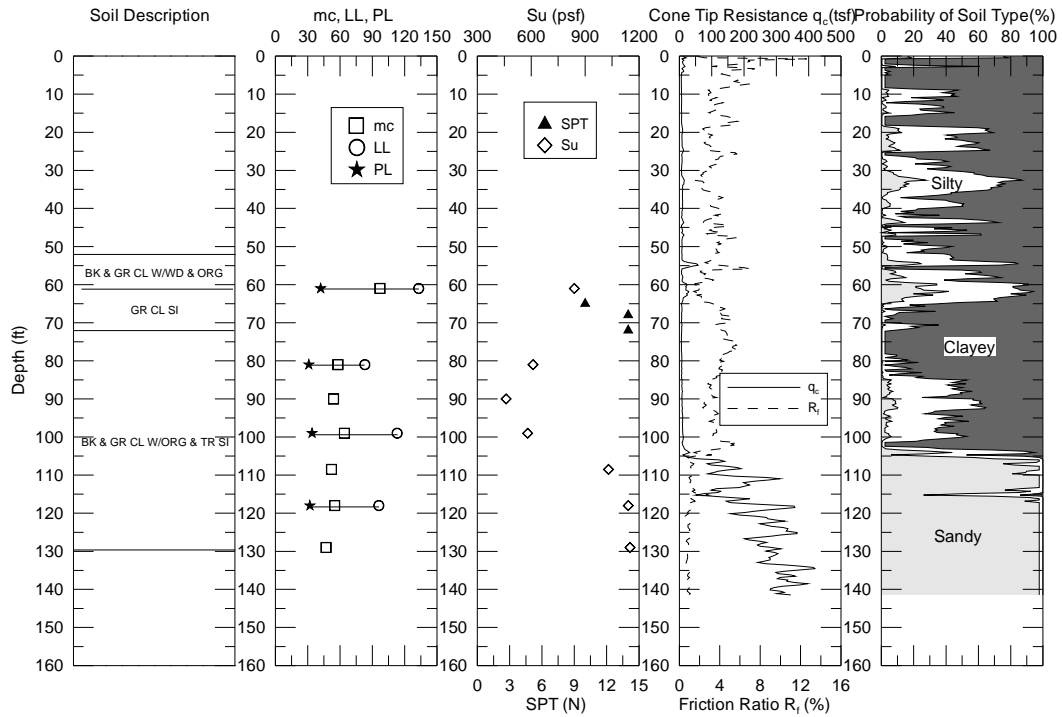


Figure 62
424-05-0087 TP#5

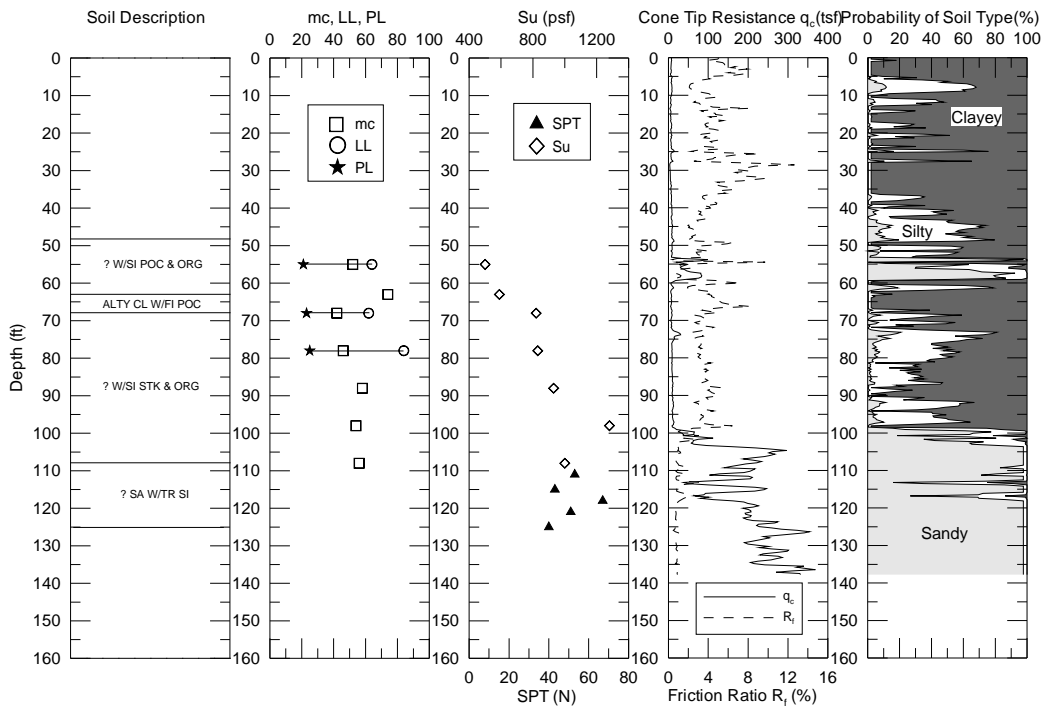


Figure 63
424-05-0087 TP#7

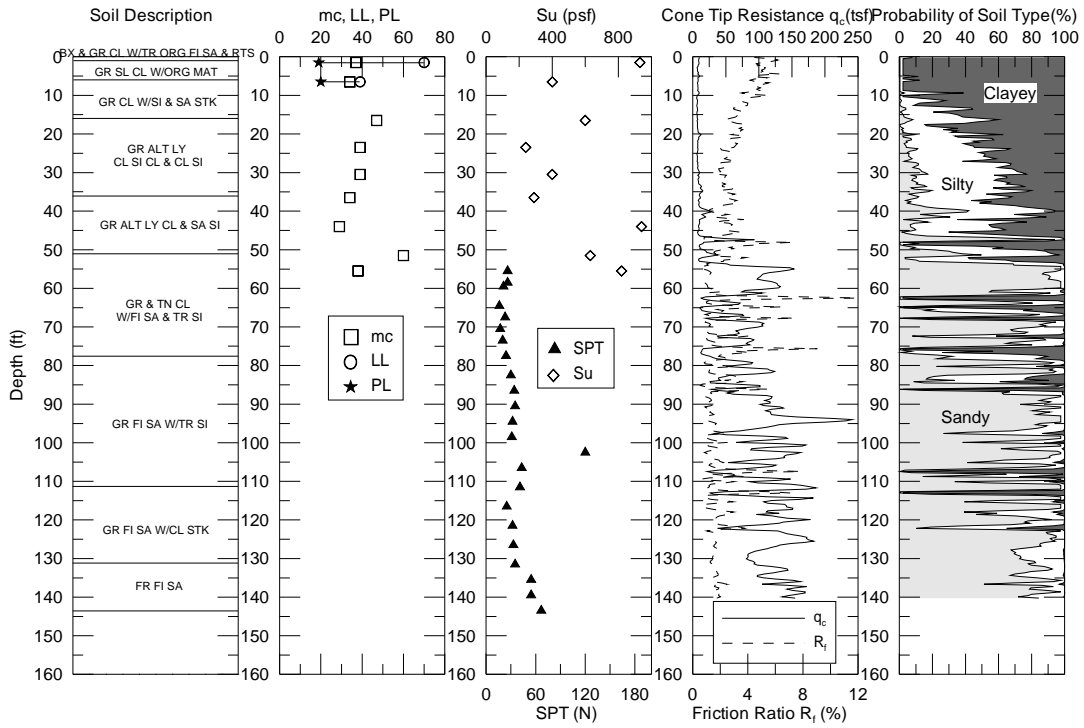


Figure 64
424-05-0081 TP#3

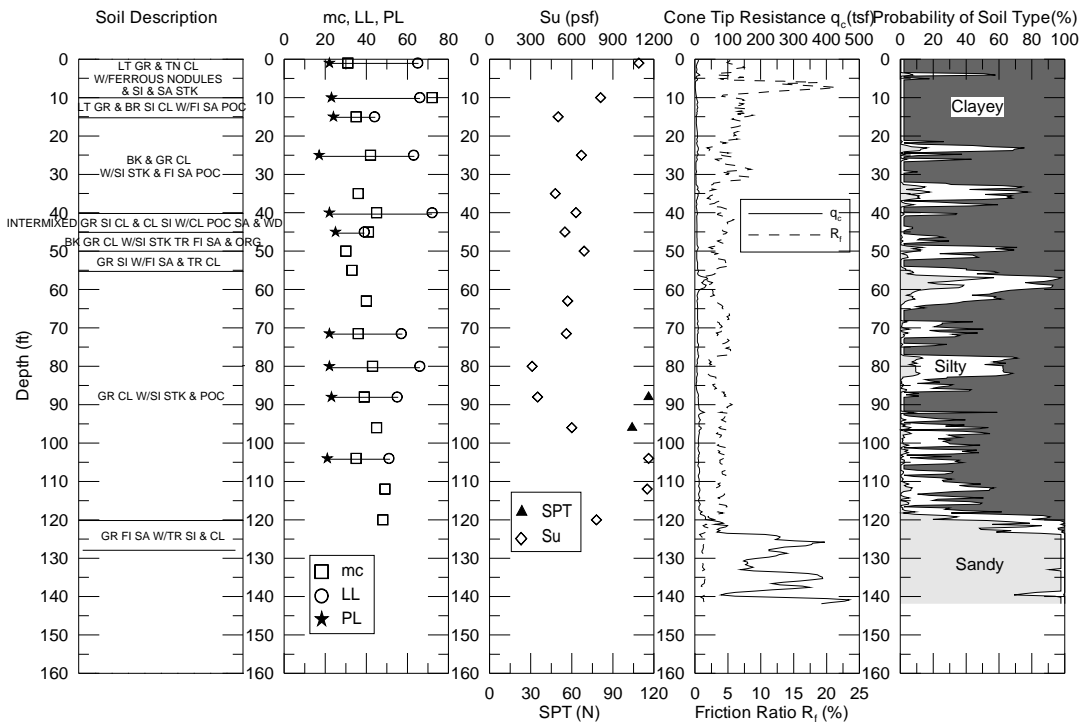


Figure 65
424-06-0005 TP#3

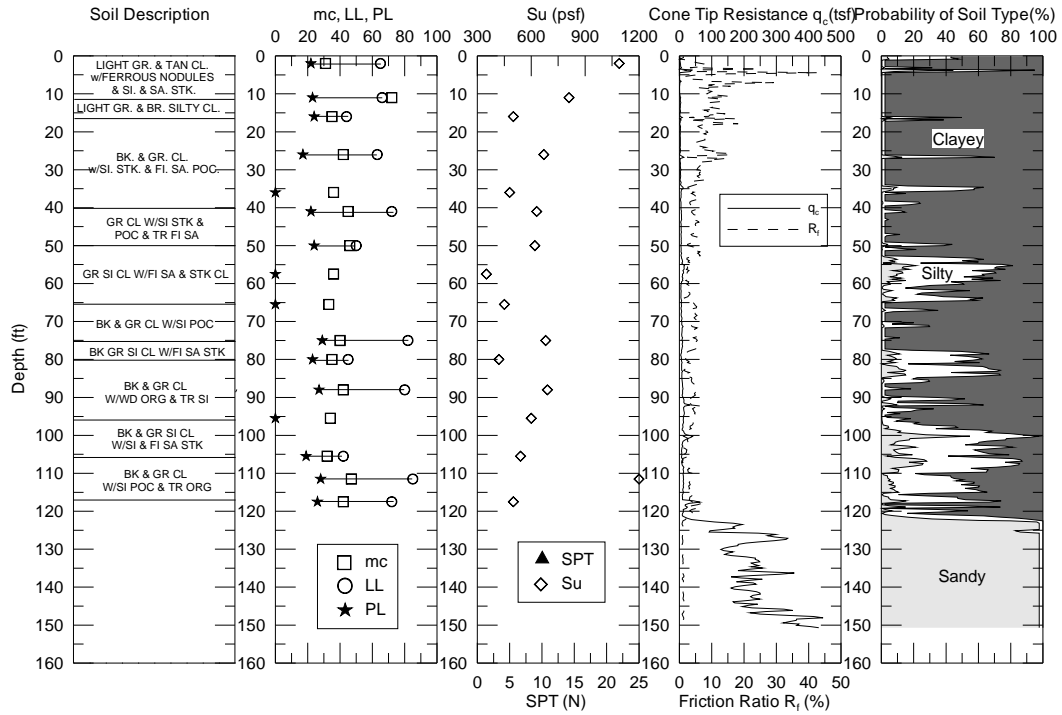


Figure 66
424-06-0005 TP#5

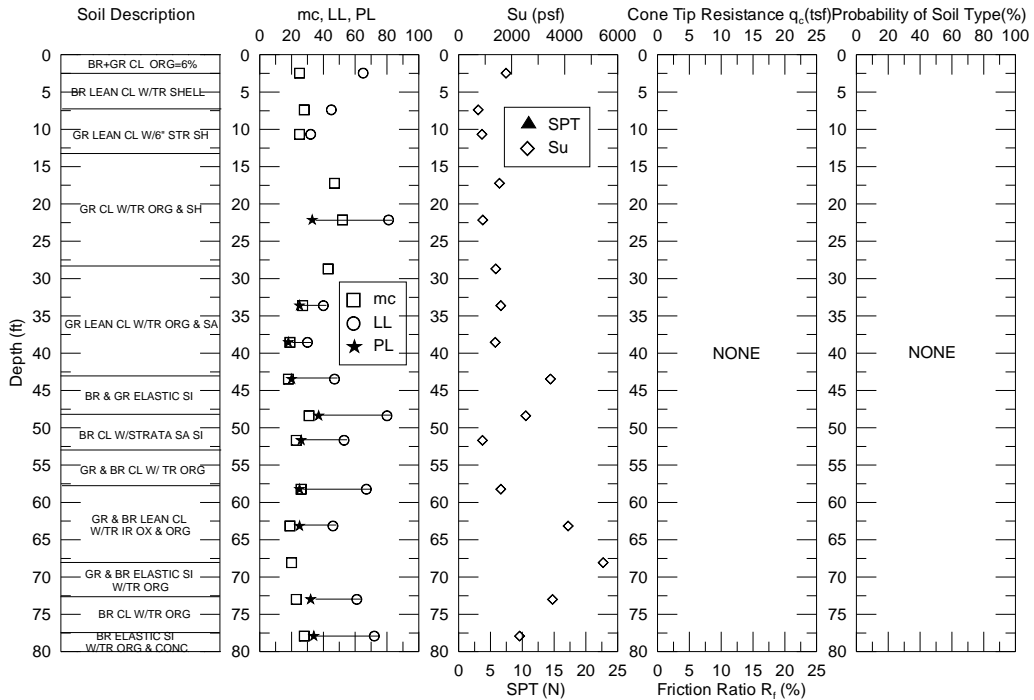


Figure 67
014-02-0018 TP#1

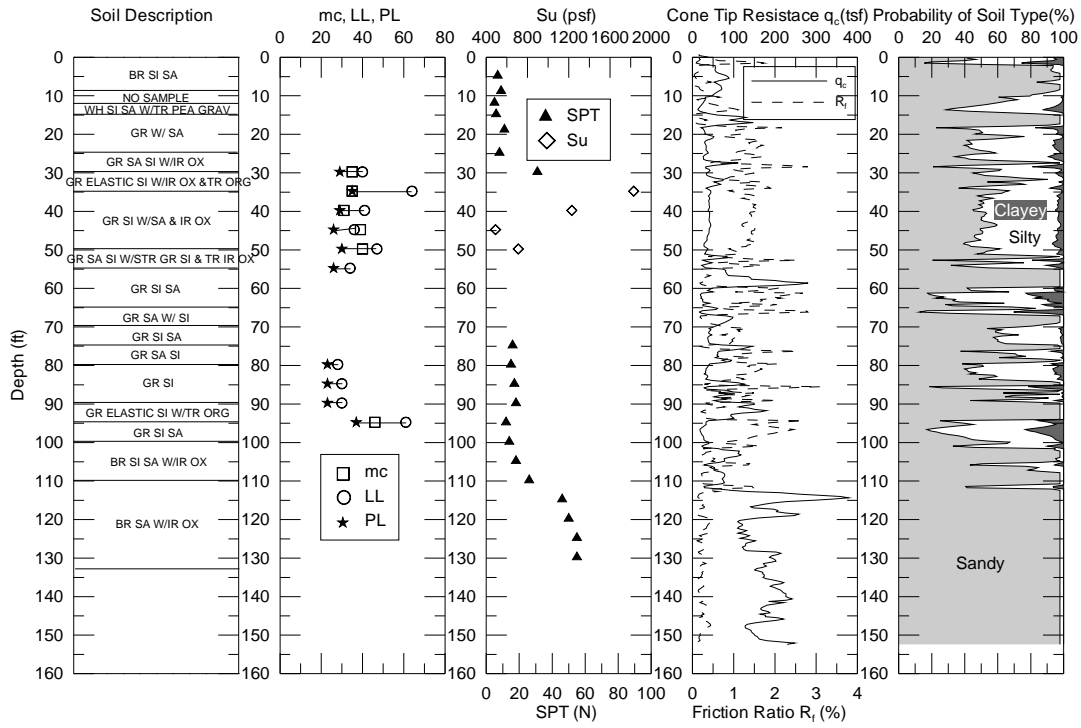


Figure 68
047-02-0022 TP#2

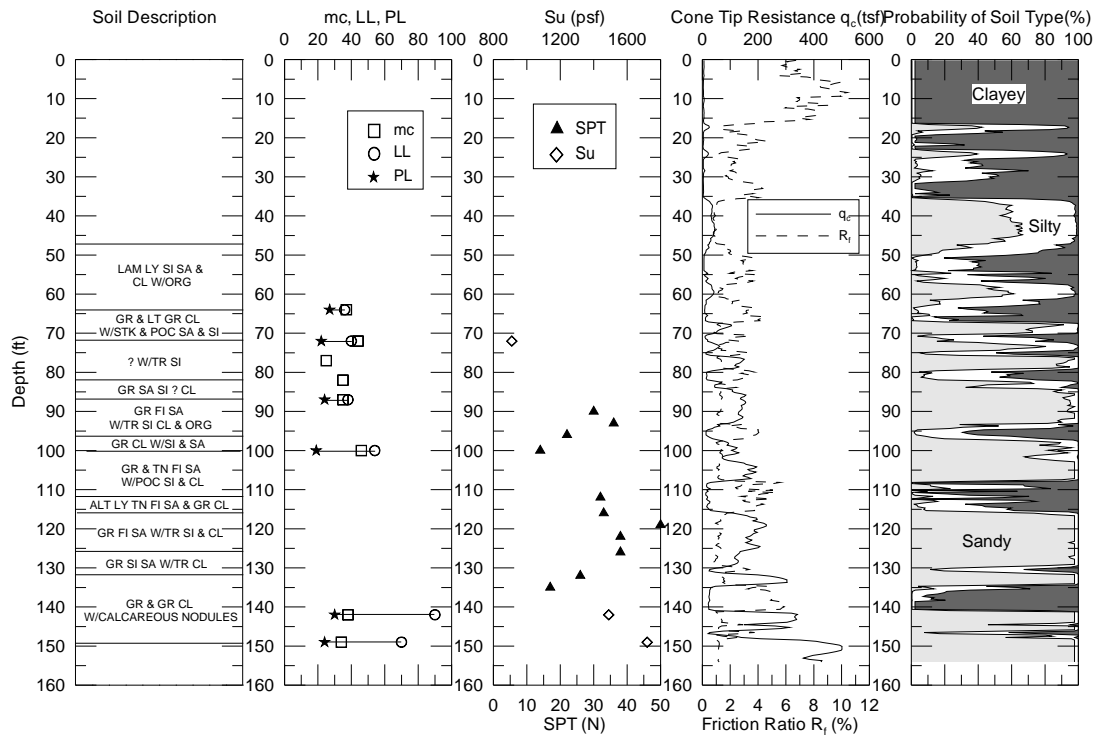


Figure 69
424-05-0087 TP#4

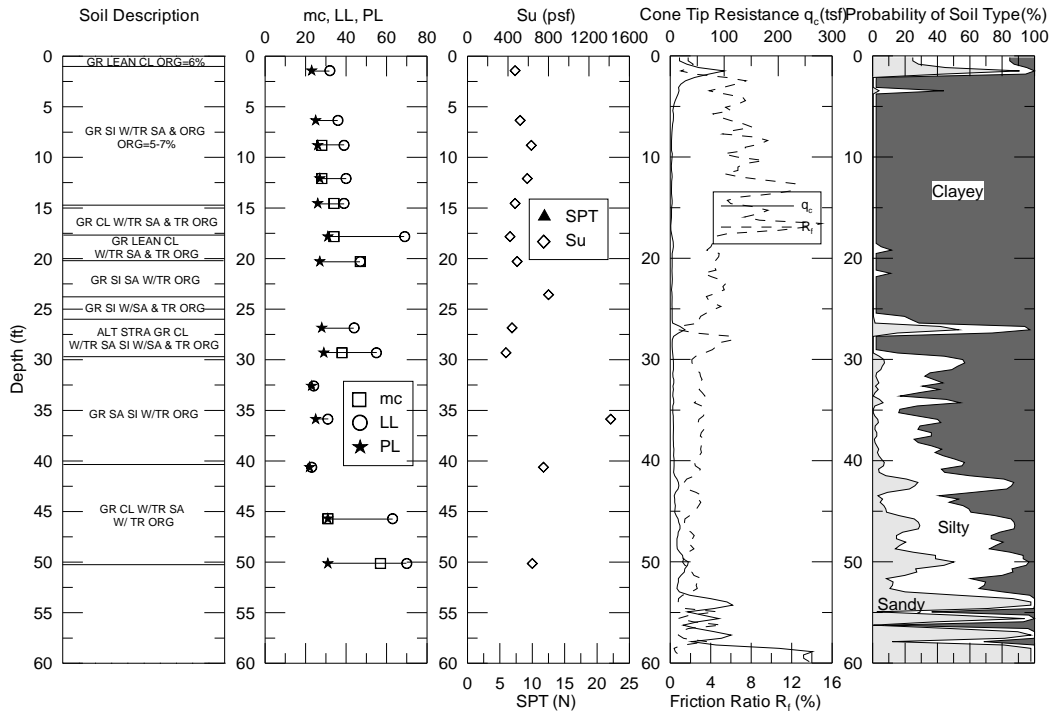


Figure 70
064-06-0036 TP#1

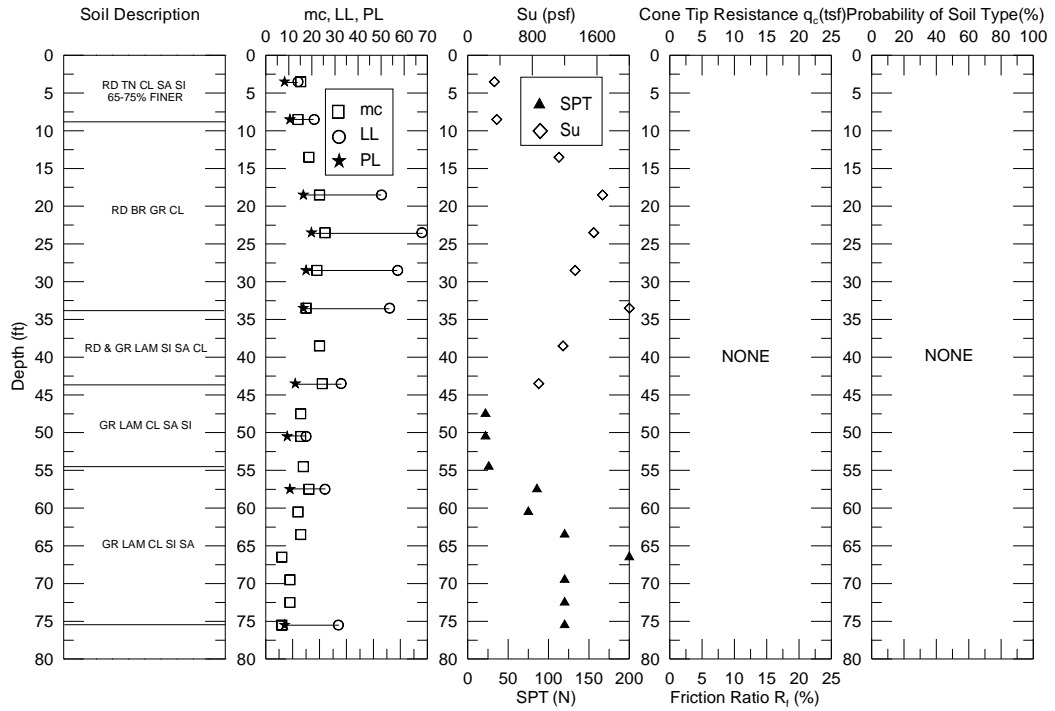


Figure 71
090-01-0015 TP#1

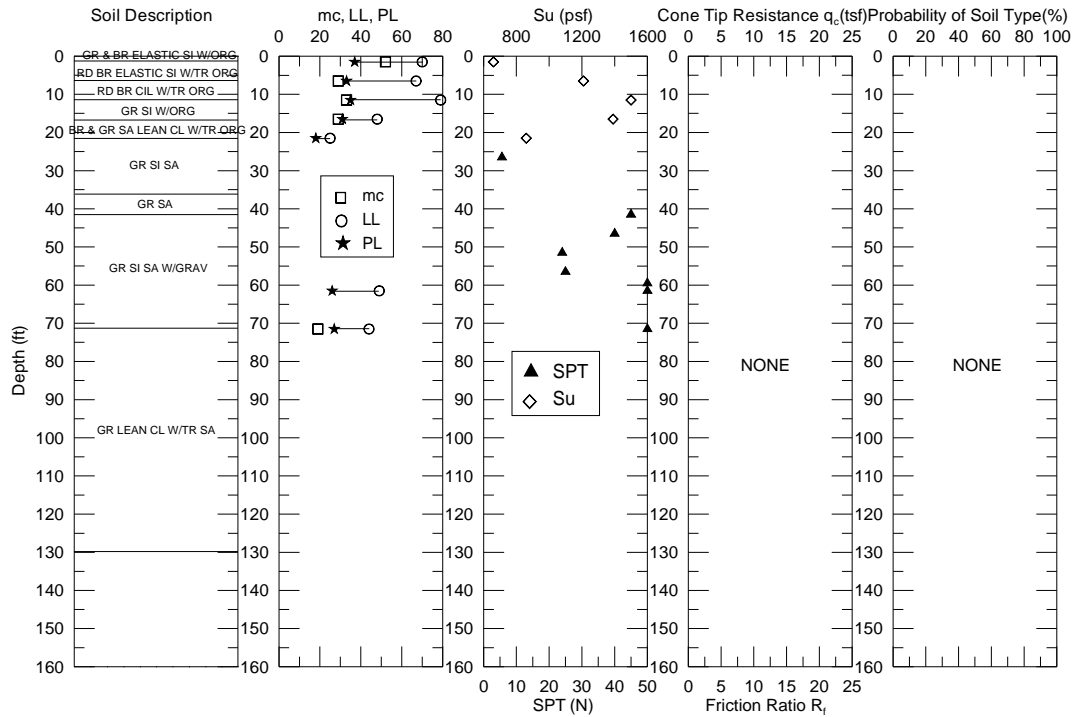


Figure 72
090-01-0015 TP#2

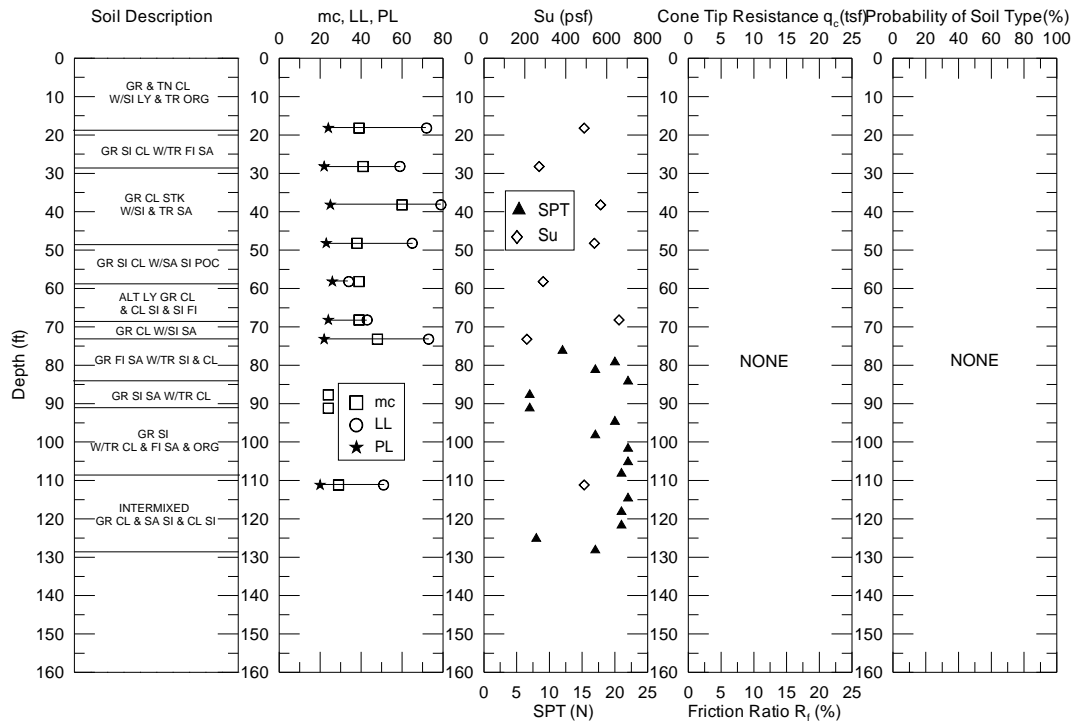


Figure 73
424-05-0087 TP#2

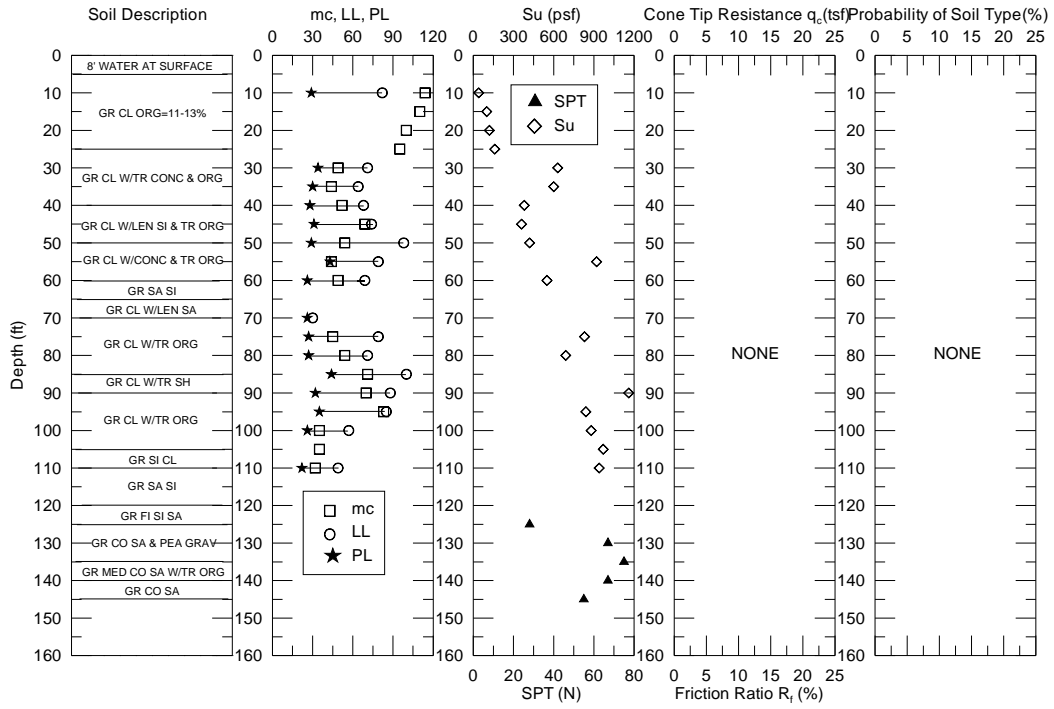


Figure 74
713-48-0083 TP#1

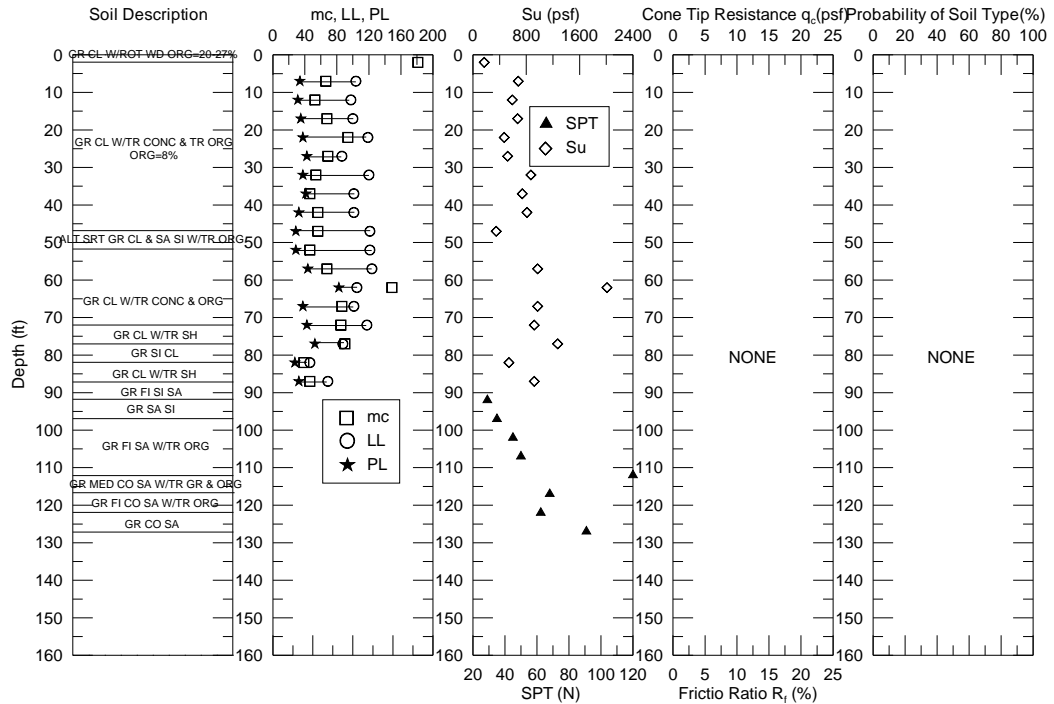


Figure 75
713-48-0083 TP#2

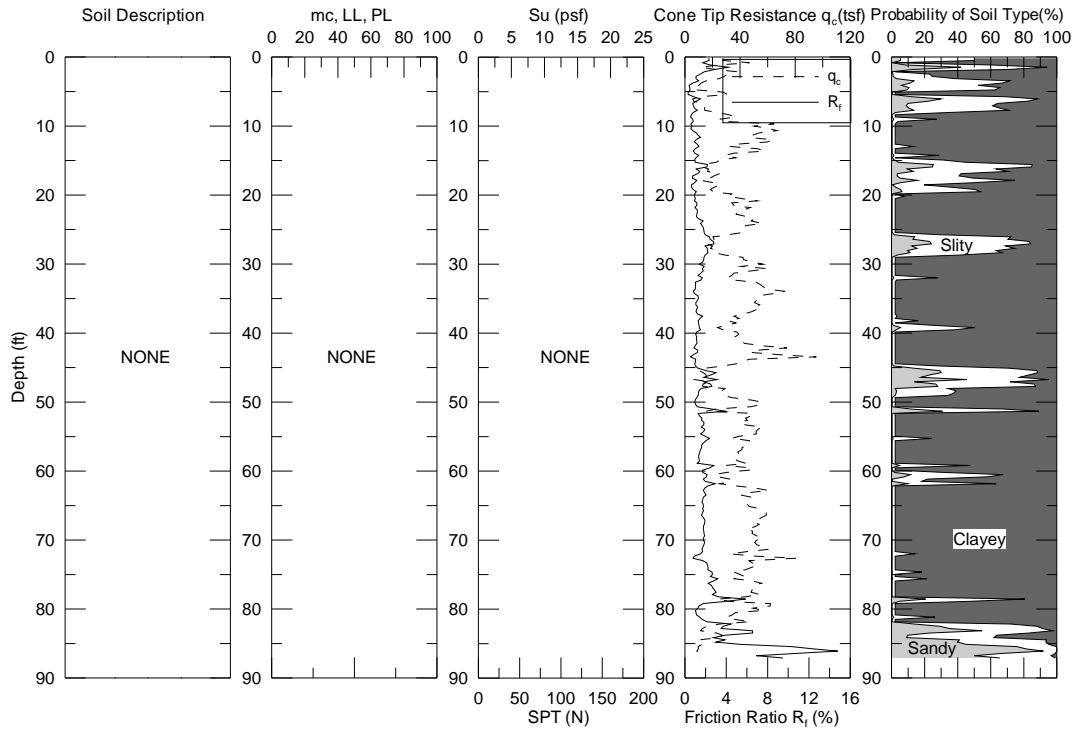


Figure 76
455-05-0036 TP#1

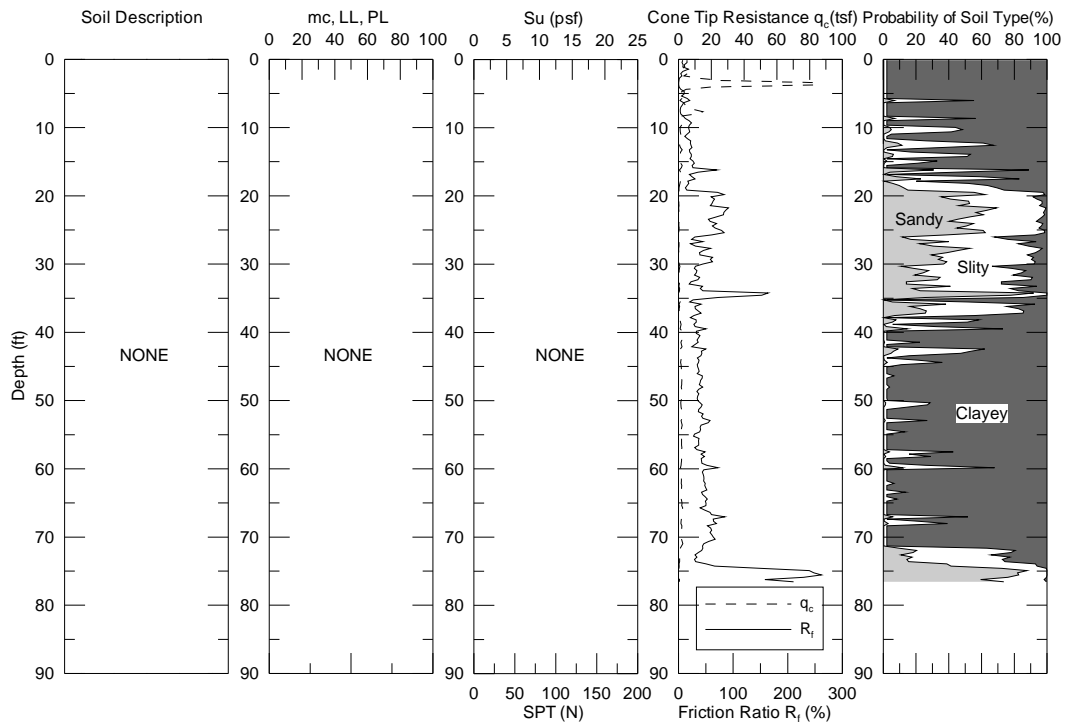


Figure 77
455-05-0036 TP#2

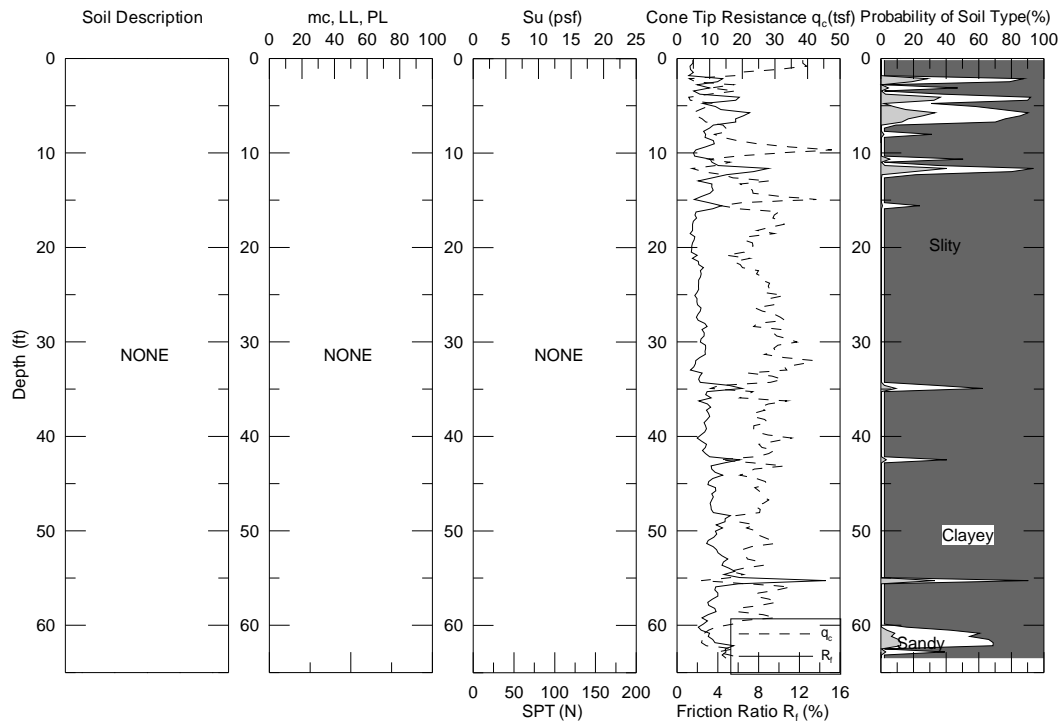


Figure 78
455-05-0036 TP#3

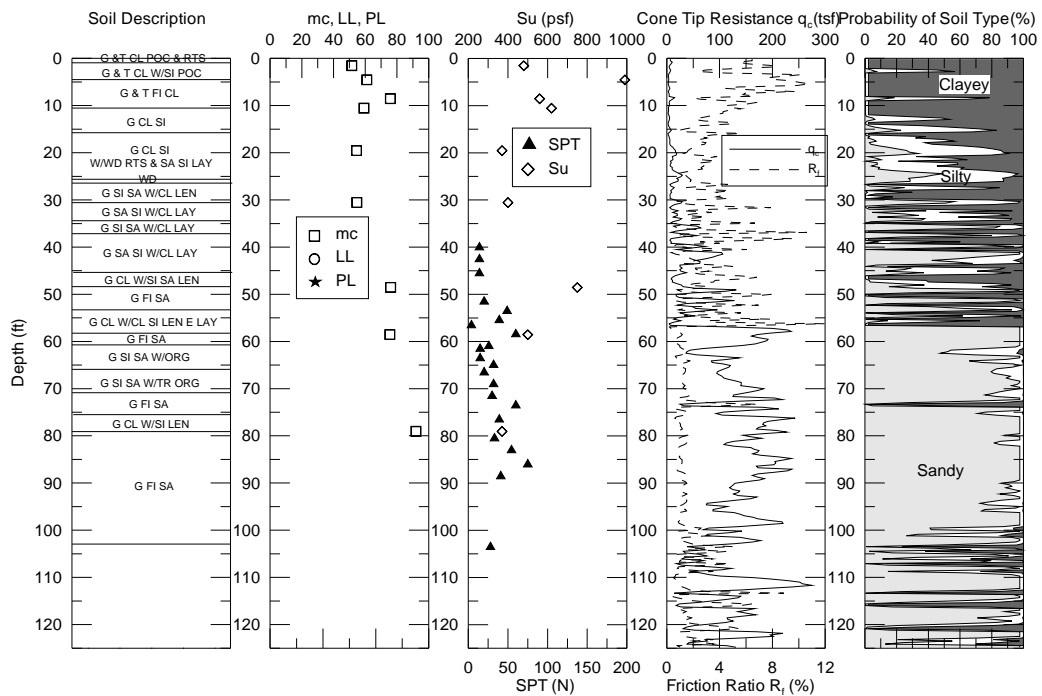


Figure 79
434-01-0002 TP#3

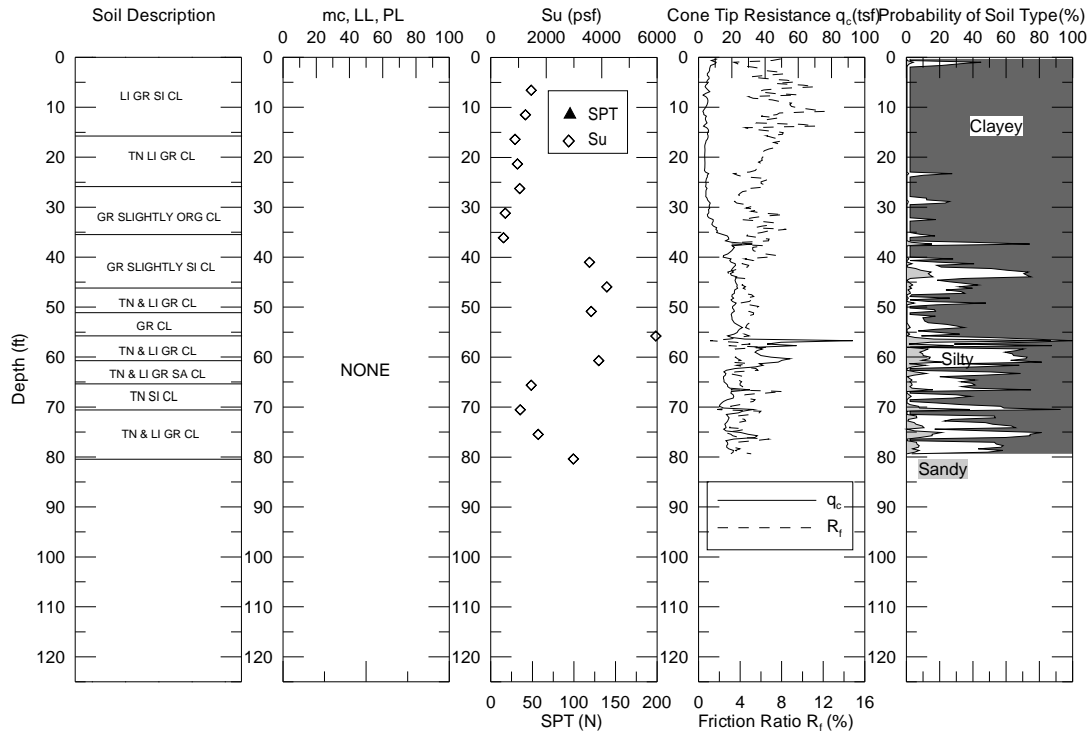


Figure 80
239-01-0080 TP#3

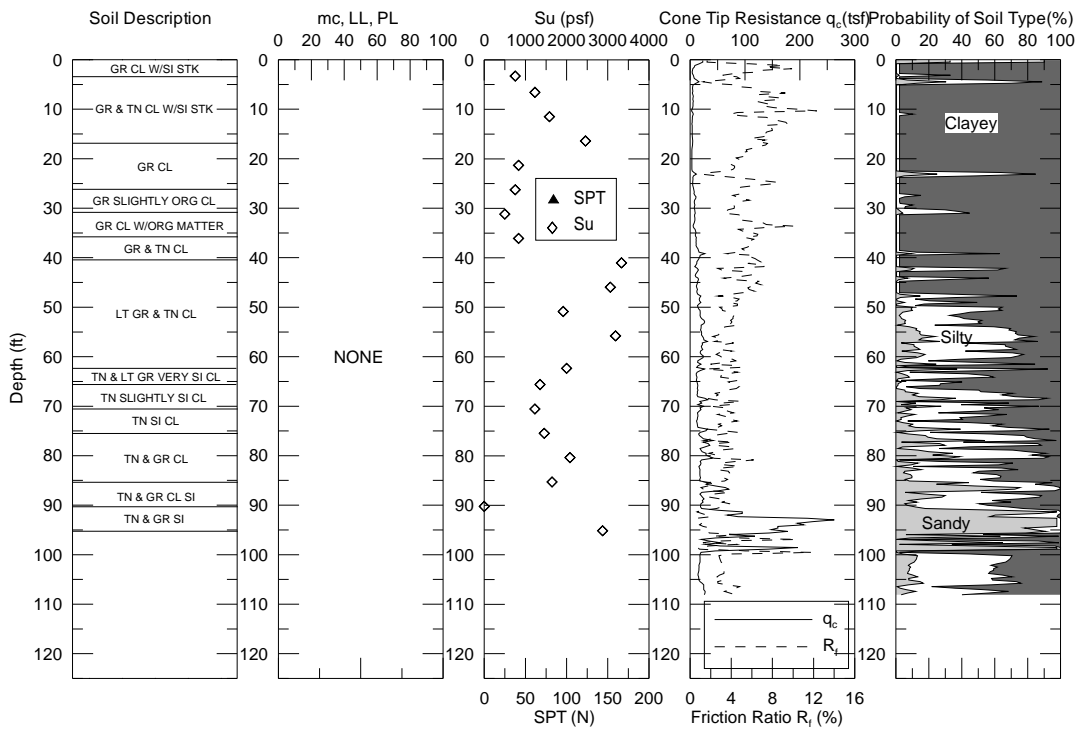


Figure 81
239-01-0080 TP#4

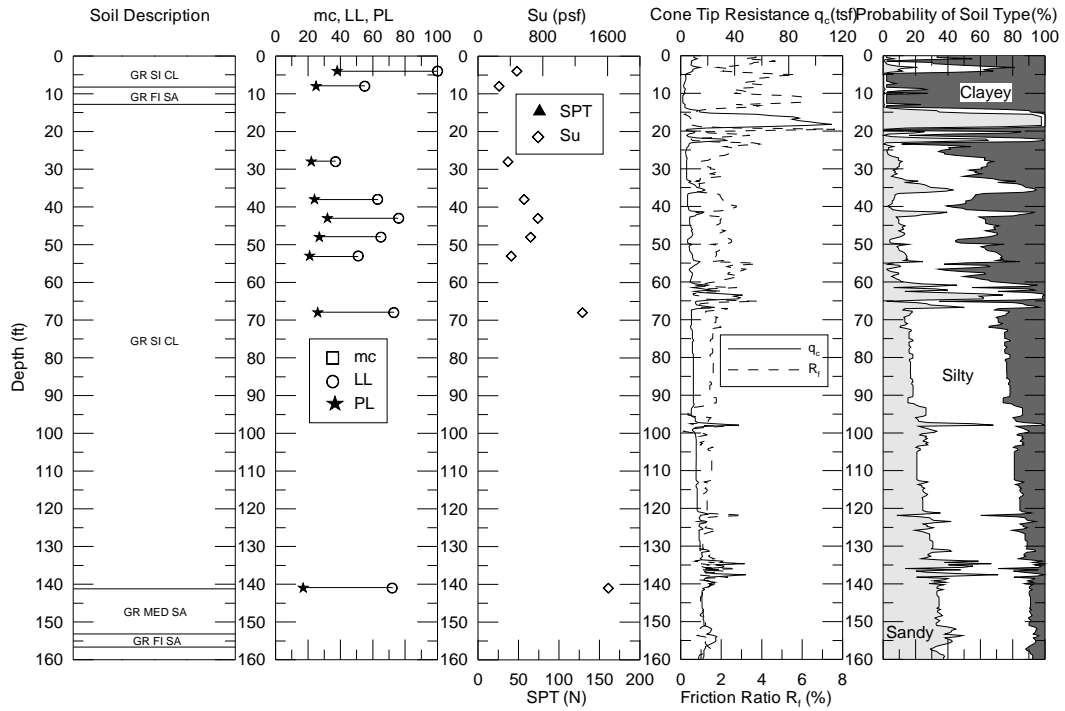


Figure 82
855-14-0036 (Houma) TP#1

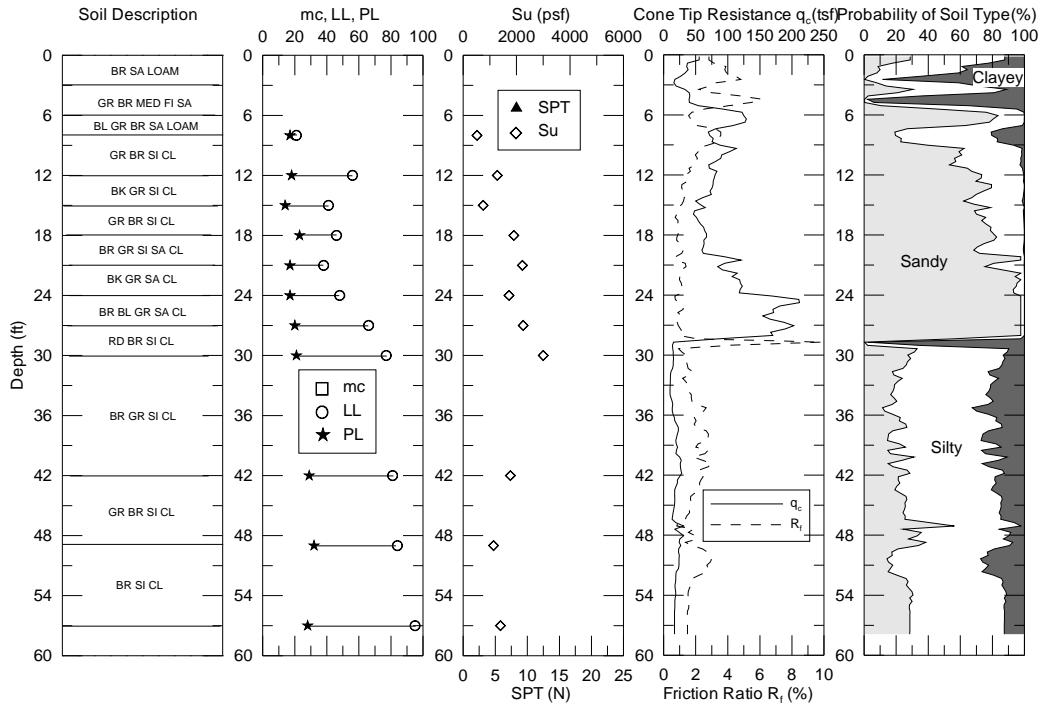


Figure 83
Alexandria TP#1

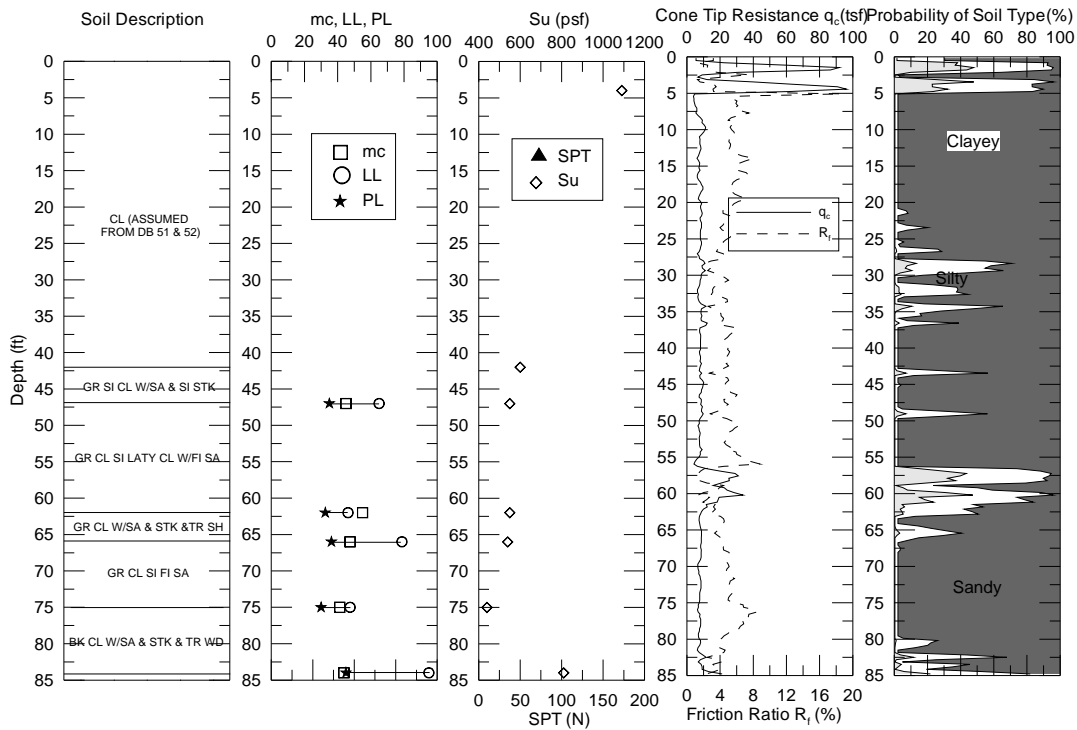


Figure 84
424-06-0005 TP#4

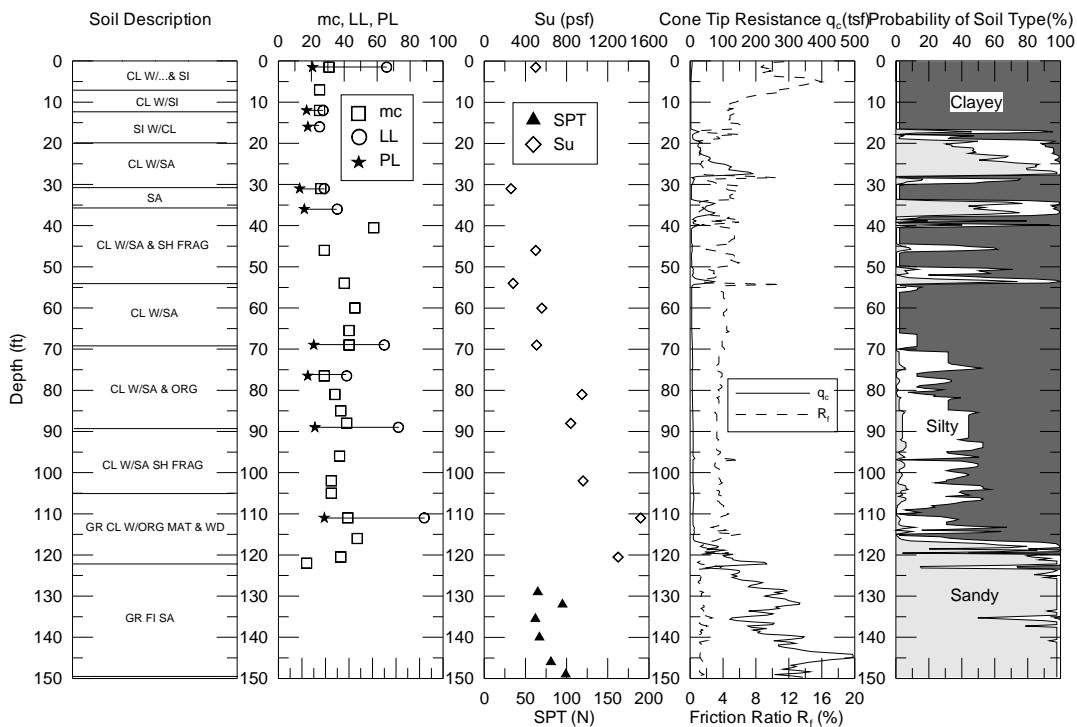


Figure 85
424-07-0009 TP#4 and 4A

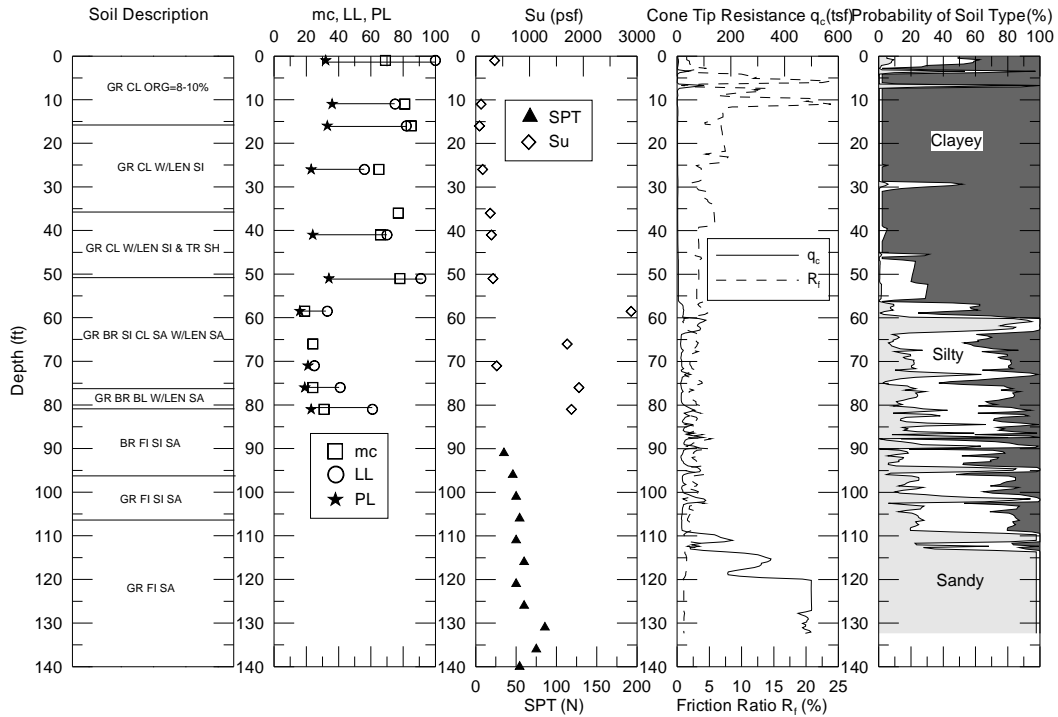


Figure 86
450-36-0002 TP#8

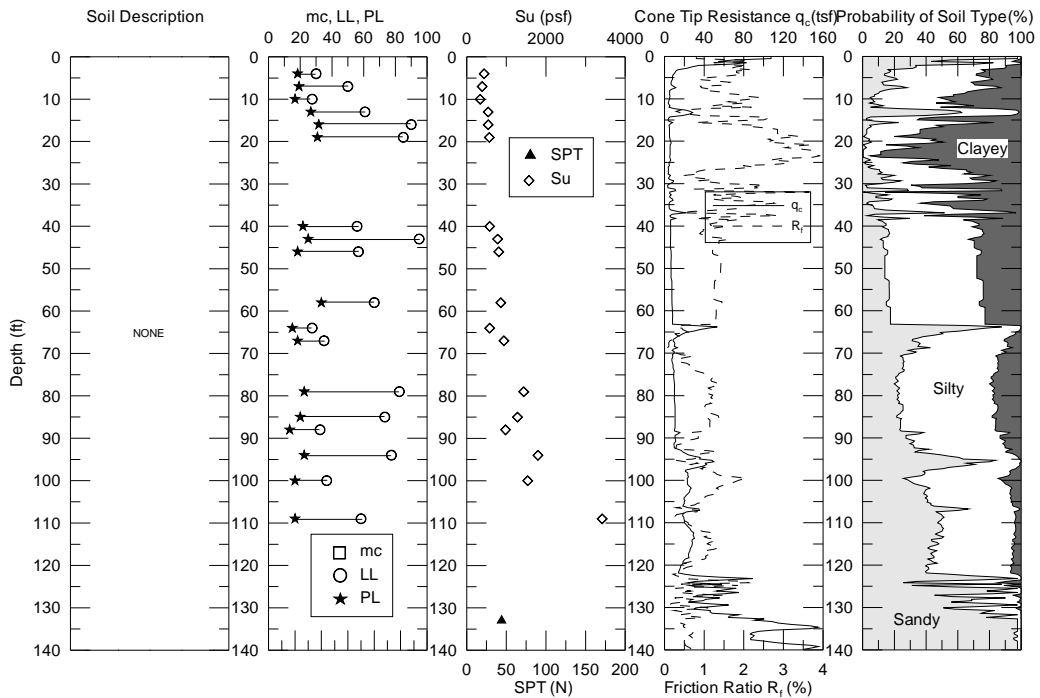


Figure 87
283-03-0052 (New Orleans) TP#1

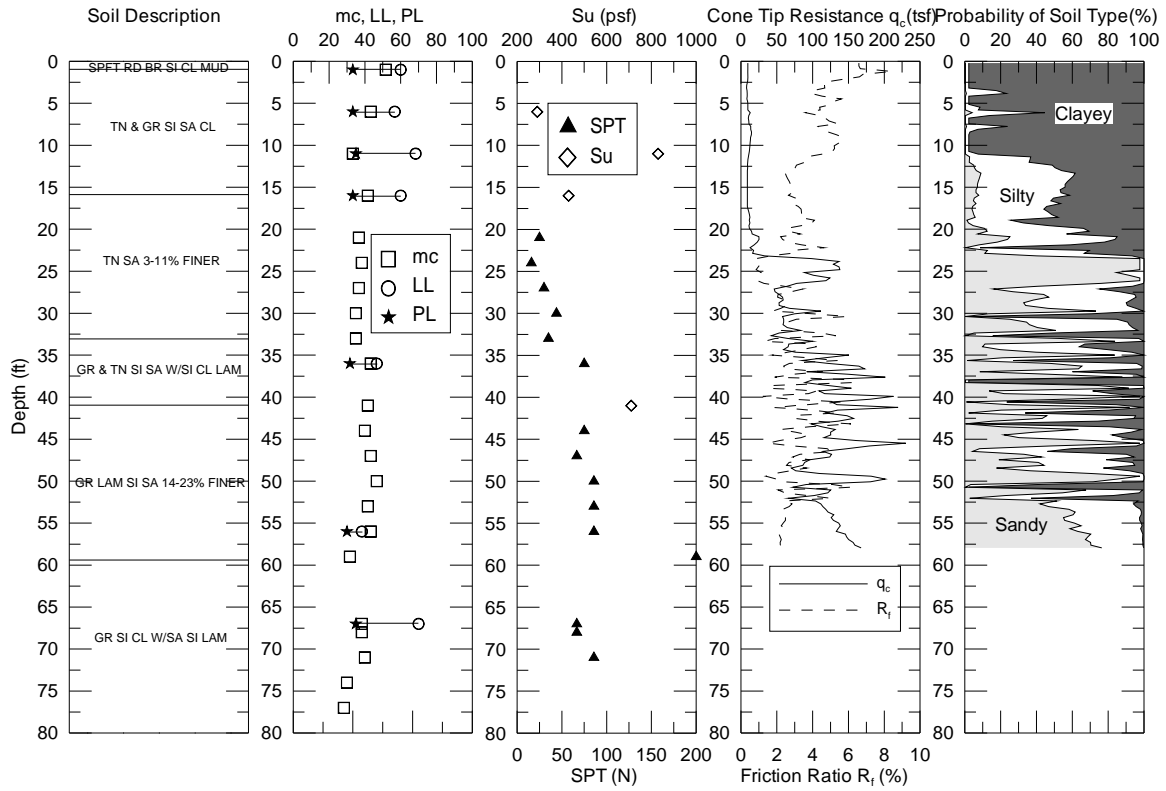


Figure 88
090-01-0015 TP#6