LRFD CALIBRATION OF **AXIALLY-LOADED CONCRETE PILES DRIVEN INTO SOFT SOILS**

Sungmin Yoon, Ph.D., P.E.
Research Associate
Louisiana Transportation Research Center
Louisiana State University
4101 Gourrier Avenue
Baton Rouge, LA 70808
Phone: 225-767-9194
E-mail: syoon@lsu.edu

Murad Y. Abu-Farsakh, Ph.D., P.E.
Assistant Professor, Research
Louisiana Transportation Research Center
Louisiana State University
4101 Gourrier Avenue
Baton Rouge, LA 70808
Phone: 225-767-9147
E-mail: cefars@lsu.edu

Ching Tsai, Ph.D., P.E.
Senior Geotechnical Engineer
Louisiana Department of Transportation and Development
1201 Capitol Access Road
Baton Rouge, LA 70804
E-mail: ChingTsai@dotd.la.gov

And

Zhongjie Zhang, Ph.D., P.E.
Pavement and Geotechnical Engineering Administrator
Louisiana Transportation Research Center
4101 Gourrier Avenue
Baton Rouge, LA 70808
E-mail: DocZhang@dotd.la.gov

Text =5,305 words
Figures = 6 × 250 = 1,500 words
Tables = 4 × 250 = 1,000 words
Total = 7,805 words

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ABSTRACT

This paper presents the evaluation of axial load resistance of piles driven into soft Louisiana soils based on reliability theory. Forty two 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) and three CPT direct methods (Schmertmann method, De Ruiter and Beringen method, and Bustamante and Gianeselli (LCPC) method). In addition, dynamic measurements with signal matching analysis of pile resistances using CAPWAP, which is based on the measured force and velocity signals obtained near the pile top during driving, were evaluated. The Davisson and modified Davisson interpretation method was used to determine the measured ultimate load carrying resistances from pile load tests. The predicted ultimate pile resistances obtained using the different prediction methods were compared with the measured resistances determined from pile load tests. Statistical analyses were carried out to evaluate the capability of the prediction design methods to estimate the measured ultimate pile resistance of driven piles. The results showed that the static method over-predicts the pile resistance, while the dynamic measurement with signal matching analysis (CAPWAP-EOD and 14 days BOR) under-predict the pile resistance. Among the three direct CPT methods, the De Ruiter and Beringen method is the most consistent prediction method with the lowest COV. Reliability based analyses, using First Order Second Moment (FOSM) method, were also conducted to calibrate the resistance factors (ϕ) for the investigated pile design methods. The resistance factors for different design method were determined and compared with AASHTO recommendation values. The calibration showed that De Ruiter and Beringen method has higher resistance factor ($\phi_{De\text{-}Ruiter} = 0.64$) than the other two CPT methods.

INTRODUCTION

The allowable stress design (ASD) method had been used in designing bridges, which involves applying a factor of safety (FS) to account for uncertainties in the applied loads and soil resistance. The magnitude of FS depends on the importance of the structure, the confidence level of the material properties, and design methodology. The Bridge Design Specifications published by the American Association of Highway and Transportation Officials or AASHTO (1, 2) has introduced the LRFD (Load and Resistance Factor Design) method to account for uncertainties associated with estimated loads and resistances. Since then the bridge superstructures have been designed using the LRFD method in most U.S. states. However LRFD method for the bridge foundation design is gaining its prevalence progressively, 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 consist level of reliability, the Federal Highway Administration and AASHTO set a mandate 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 to establish and calibrate the proper resistance factors for local soil conditions in compliance with this mandate (3, 4, 5, 6, 7, 8, 9, 10).

The current AASHTO (11) recommends resistance factors, ϕ, for single driven piles in axial compression range from 0.10 to 0.65, depending on the design method. However, the existing resistance factors are recommended based on pile database that was collected from sites that do not necessary reflect local soils or design practice. For example, the driven pile database used in the existing AASHTO code is based on the data gathered by the Florida DOT and the Federal Highway Administration (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 database are available, the AASHTO recommends calibrating the resistance factors, ϕ, using risk 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 the 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 utilizing the results of in-situ testing such as Cone Penetration Test (CPT). However, static analysis based on soil properties obtained from borings and laboratory tests have been mainly used in practice. However, the application of CPT in predicting the ultimate bearing resistance of pile has been 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 the traditional static design, the CPT design methods can provide more economical estimation of the ultimate pile resistance. In addition, the dynamic measurement with signal matching analysis (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 usually helps in verifying the pile design resistance.

OBJECTIVES AND SCOPE

This paper presents the reliability based analysis for the estimation of axial load resistance of driven piles in soft Louisiana soils. Forty two precast prestressed concrete piles with different lengths and sizes that were loaded to failure were investigated in this study. Statistical analyses were conducted to evaluate the different pile design methods, including the static design method (α-method and Nordlund method), three different direct CPT design methods: Schmertmann (14) method, De Ruiter and Beringen (15) method, and Bustamante and Gianeselli (LCPC) (16) method, and dynamic measurement with signal matching analysis (CAPWAP) method. In addition, reliability analyses based on first order second moment (FOSM) method were conducted to calibrate the resistance factors (ϕ) for the different design methods needed in the LRFD design of single piles.

PILE LOAD TEST DATABASE

The pile load test database used for the calibration was established by conducting an extensive research in the Louisiana Department of Transportation and Development (LADOTD)'s project library. Only precast prestressed concrete (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 driving of piles is not a problem. A total of forty two pile load tests met this criterion. A summary of the characteristics of the investigated piles is presented in Table 1. The measured ultimate pile resistance (Q_u) was interpreted from the load-settlement curve using the Davisson method (18) for piles with size less than 610mm and the modified Davisson method proposed by Kyfor et al. (19) for piles exceeding a size of 610mm. In the Davisson method, the ultimate bearing resistance is determined from the intersection between a load-settlement curve and a straight line with slope of L/AE and initial settlement of $0.004L_R + B/120$ (L: pile length, A: cross-sectional area of the pile, E: pile Young's modulus, L_R: reference length = 1m, and B: pile diameter). Figure 1 illustrates a sample load-settlement 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 collected. Figure 2 shows a typical summary of geotechnical data for a tested pile.

PREDICTION 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:

$$Q_u = Q_b + Q_s = q_b . A_b + \sum_{i=1}^{n} f_i A_{si} \tag{1}$$

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 usually dominates, while in sandy stratigraphies, the end-bearing resistance can contribute up to 50% 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) comparing to the end bearing resistance (Q_b).

Table 1. Summary of the Characteristic of the Investigated Piles

Square PPC Pile Size (mm)	Pile Type		Predominant Soil Type		
	Friction	End-Bearing	Cohesive	Cohesionless	Limit of Information
360	18	0	16	2	0
410	5	0	3	0	2
610	9	0	6	3	0
760	10	0	5	5	0
Total	42	0	30	10	2

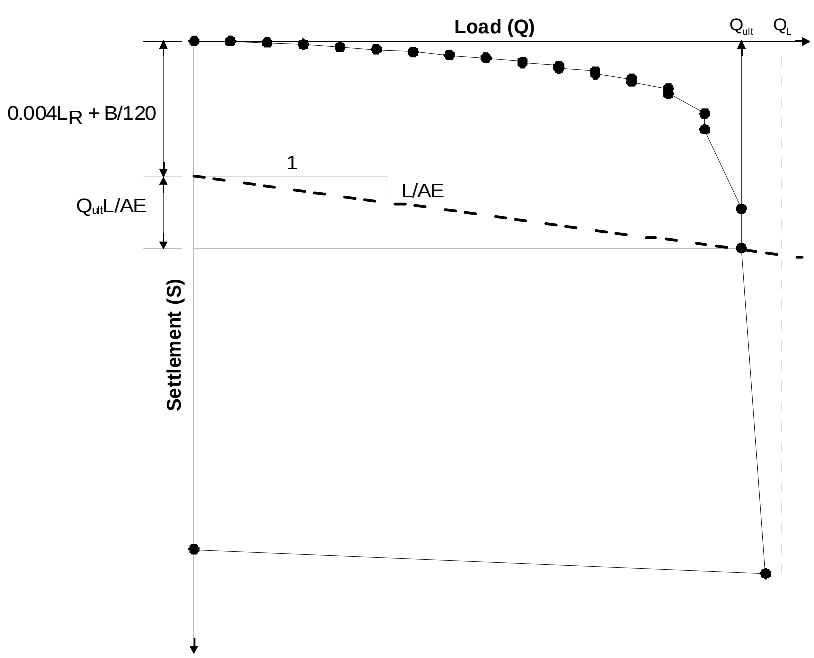


Figure 1 Estimation of ultimate bearing resistance using Davisson (18) method (after Salgado (20)).

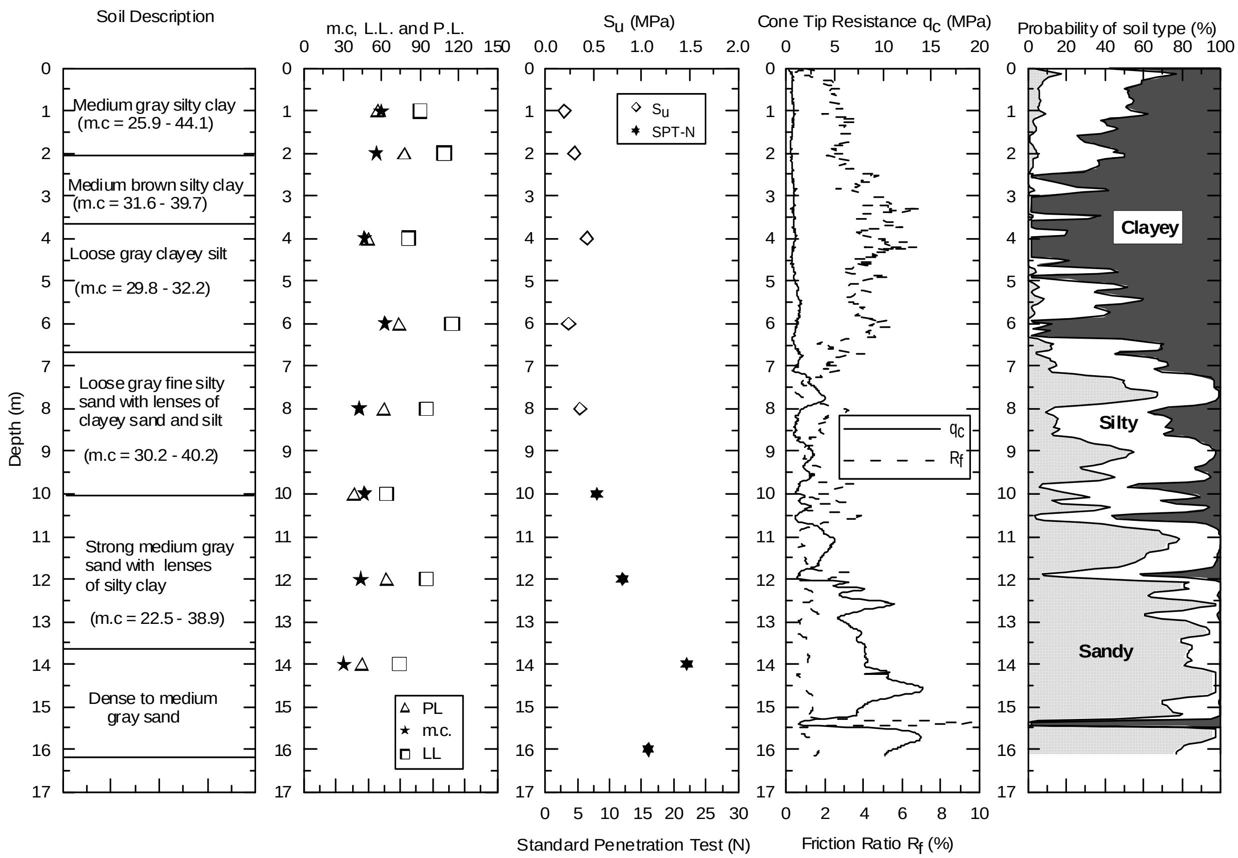


Figure 2 Typical summary of geotechnical data for a tested pile.

Static Methods

α-Tomlinson method

The α-method (21) is based on total stress analysis. For a soil with $\phi = 0$ or in total stress analysis, the ultimate skin resistance per unit area of pile can be calculated as follows:

$$f = \alpha\, S_u \qquad (2)$$

where α is an empirical adhesion coefficient and S_u is the undrained shear strength. In this research study, α values suggested by Tomlinson (21) are used.

The skin friction resistance (Q_s) is as follows:

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

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

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

$$Q_b = A_b \, S_u \, N_c \tag{4}$$

where A_b is cross sectional area of the pile and N_c of 9 is used in this study.

Nordlund method

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

$$Q_b = A_b.\overline{q}.\alpha.N'_q \tag{5}$$

where $\overline{q}$ 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 (22) are used for calculation.

The skin friction resistance (Q_s) was evaluated using the equation proposed by Nordlund (23, 24) in this research as follows:

$$Q_s = \int_0^L K_\delta \, C_f \, \overline{P}_D \sin(\delta).C_d \, dz \tag{6}$$

where K_δ is a coefficient of lateral stress, $\overline{P}_D$ is effective overburden pressure, δ is pile-soil friction angle, C_d is effective pile perimeter, and C_f is a correction factor.

Direct CPT methods

There are two main approaches in estimation of pile resistance using CPT data, which are 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 practice (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:

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

$$Q_s = f.A_s = \alpha_c f_s A_s \tag{8}$$

or

$$Q_s = f.A_s = (c_s.q_c).A_s \tag{9}$$

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 is the unit skin friction, f_s 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_s is the pile surface area.

Schmertmann method

Schmertmann (14) proposed a direct CPT method based on the model and full scale pile tests. 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 3). Schmertmann suggested 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 α_c value of 0.2 to 1.25 for clayey soil. A maximum f_s of 120 kPa is proposed.

De Ruiter and Beringen method

De Ruiter and Beringen (15) method is known as the European method. It is based on the experience from offshore piles tested in the North Sea. In sand, the unit tip resistance (q_b) is obtained from same way as Schmertmann (14) method. The unit skin friction (f) for the compression piles is the minimum among (f_s, $q_{c(side)}$/300, and 20 kPa). In clay, the unit tip resistance (q_b) is determined from the conventional bearing resistance theory as follows:

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

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

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 as:

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

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

Bustamante and Gianeselli method (LCPC/LCP method)

Bustamante and Gianeselli (16) method is known as the French method or the LCPC/LCP method. In this method, both unit tip resistance (q_b) and unit skin friction (f) are calculated from the cone tip resistance (q_c). The average cone tip resistance ($q_{c,avg}$) is obtained in the zone ranging 1.5 D above and below the pile tip. The correlation coefficient of the tip resistance (c_b) from 0.15 to 0.6 was proposed for different soil types and installation procedure based on the empirical correlation (c_b = 0.6 for piles driven into clay-silt and c_b = 0.375 for piles driven into sand-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:

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

where q_{eq}(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.

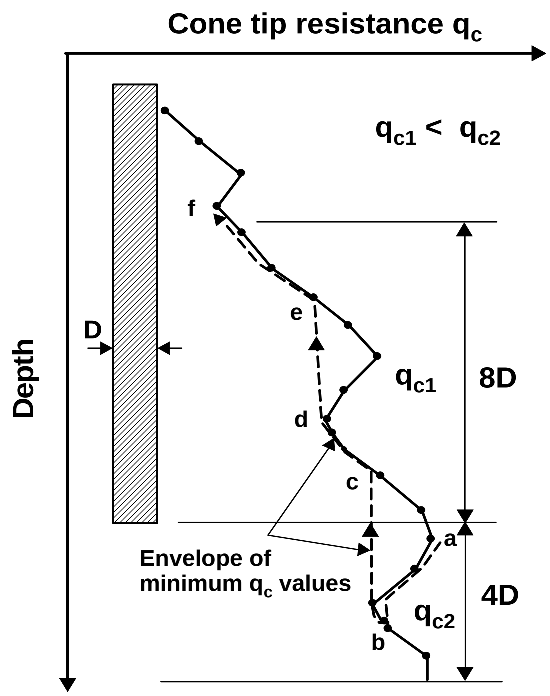


Figure 3 Average cone tip resistance ($q_{c,avg}$) in Schmertmann (14) method (from Abu-Farsakh and Titi, (25)).

Table 2 Friction Coefficient, k_s (Bustamante and Gianeselli, (16))

Nature of Soil	q_c (MPa)	ks
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

Dynamic Measurement with Signal Matching Analysis (CAPWAP)

Post-driving analyses utilize the measured force signal (calculated from strain readings) and the measured velocity signal (integrated from acceleration readings) obtained near the pile top during driving. The velocity signal is used as a boundary condition at that point while varying the parameters describing the soil resistance in order to match the calculated and measured force signals. These parameters include the side and tip quake, side and tip damping, the pile shaft resistance, and the 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 the soil-model variables for each pile in contact with the soil until the best match between the 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. (26), utilizing the computer program CAPWAP.

LRFD CALIBRATION USING RELIABILITY THEORY

The ultimate pile resistances for the same site obtained by using different methods show some variation related to its reliability. Consequently, the resistance factors (ϕ) associated with different methods should reflect its accuracy in predicting the ultimate bearing resistance of piles. In this research study, the resistance factors (ϕ) were determined for the static design methods (α-Tomlinson method for clay and Nordlund method for sand), three direct CPT methods (14, 15, 16) and 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 firstly be determined. The two limit states that are checked in the design of piles are the Ultimate Limit State (ULS) or Strength Limit State and the Serviceability Limit State (SLS). Both limit states designs are carried out to satisfy the following criteria (27):

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 strength limit state is considered. The following basic equation is recommended to represent limit states design by AASHTO (28):

$$\phi R_n \geq \sum \eta . \gamma_i . Q_i \tag{14}$$

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

Most of 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.

Consider the load combination of dead load and live load for AASHTO Strength I case, the performance limit equation is as follows (29):

$$\phi R_n = \gamma_{\mathrm{D}}\, \mathrm{Q}_{\mathrm{D}} + \gamma_{\mathrm{L}}\, Q_L \qquad (15)$$

where Q_D and Q_L are the dead load and live load, respectively, and γ_D and γ_L are the load factors for dead load and live load, respectively.

The loads applied to the piles are traditionally based on superstructure analysis whereas the 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 the AASHTO LRFD Specifications, which was originally recommended by Nowak (9), to make the pile foundation design consistent with the bridge superstructure design. For example, Zhang et al. (29), Kim et al. (30), and McVay et al. (31), selected the statistical parameters of dead and live loads, which used in the AASHTO LRFD Specifications as follows:

$$\gamma_{\mathrm{L}} = 1.75 \quad \lambda_{\mathrm{QL}} = 1.15 \quad \mathrm{COV}_{\mathrm{QL}} = 0.18$$
$$\gamma_{\mathrm{D}} = 1.25 \quad \lambda_{\mathrm{QD}} = 1.08 \quad \mathrm{COV}_{\mathrm{QD}} = 0.13$$

where γ_D and γ_L are the load factors for dead load and live load, respectively. λ_{QD} and λ_{QL} are the load bias factors for the dead load and live load, respectively. COV_{QD} and COV_{QL} 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 (32). In this research, Q_D/Q_L of 3 is used.

The resistance statistics was 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.

$$\lambda_{\mathrm{R}} = \frac{\mathrm{R}_{\mathrm{m}}}{\mathrm{R}_{\mathrm{p}}} \qquad (16)$$

where R_m=Measured resistance and R_p=Predicted (nominal resistance = R_n).

First Order Second Moment (FOSM) Method

Since FOSM is employed for existing AASHTO specification, it is used in this study for calibration of resistance factors of driven piles. In the First Order Second Moment method, limit state function is linearized by expanding the Taylor series expansion about the mean value of variable. Since only the mean and variance are used in the expansion, it is called First (Mean) order second (variance) Moment. For Lognormal distribution of resistance and load statistics, Barker et al. (33) suggested the following relation for calculating reliability index,

$$\beta = \frac{\ln\left[\lambda_R FS\left(\frac{Q_{DL}/Q_{LL}+1}{\lambda_{DL}\,Q_{DL}/Q_{LL}+\lambda_{LL}}\right)\sqrt{\frac{1+COV_R^2+COV_{DL}^2+COV_{LL}^2}{1+COV_R^2}}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{DL}^2+COV_{LL}^2\right)\right]}} \tag{17}$$

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

$$\phi = \frac{\lambda_R\left(\gamma_D\frac{Q_D}{Q_L}+\gamma_L\right)\sqrt{\frac{1+COV_{QD}^2+COV_{QL}^2}{1+COV_R^2}}}{\left(\lambda_{QD}\frac{Q_D}{Q_L}+\lambda_{QL}\right)\exp\left(\beta_T\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{QD}^2+COV_{QL}^2\right)\right]}\right)} \tag{18}$$

ANALYSIS AND EVALUATION OF RESULTS

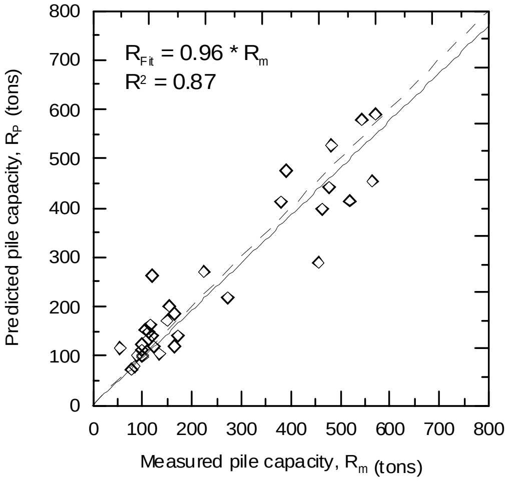
Predicted versus Measured Ultimate Pile Resistances

Based on the analysis of forty two driven piles, a statistical analysis was performed on the collected database to evaluate the different pile design methods, and to determine their corresponding resistance bias factors (λ_R), defined as the mean ratio of measured resistance over predicted resistance. The mean ratio, standard deviation (σ) and coefficient of variation (COV) of the measured to predicted pile resistance ratios (R_m/R_p) were calculated for the different pile resistance prediction methods and summarized in Table 3. Figures 4 (a) through 4 (f) present the comparison between the predicted and measured pile resistances for the different pile design methods. Regression analyses were conducted on all methods to obtain the best fit line of the predicted/measured pile resistances. The relationships R_{fit}/R_m and the corresponding R^2 for all prediction methods are presented in Figure 4, and are also summarized in Table 3. The results of predicted static resistances versus the measured load tests compiled from all soil types are presented in Figure 4 (a). The R_{fit}/R_m for static method is 0.96 with R^2 = 0.87, and the mean ratio of R_p/R_m is 1.11, indicating 11 percent over prediction the measured value using the α-Tomlinson's method in clay and Nordlund's method in sand. The COV for static method is 0.25, which is somewhat lower than the COV of static method in mixed soil (0.49) reported by Paikowsky (3). However, the mean resistance bias factors (λ_R) of 0.97 is very close to the one (0.96) reported by Paikowsky (3). This may be due to the different soil conditions of the investigated piles. Figures 4 (b) through 4 (d) depict the comparison between the three CPT prediction methods and the Davisson measured resistances. On average, the LCPC method overestimates the ultimate pile resistance by 4 %, and De Ruiter-Beringen method underestimates the pile resistance by 11%; while the average prediction from Schmertmann method overestimates the resistance by 17%. The De Ruiter-Beringer method is the most consistent method among the direct CPT methods with a coefficient of variation (COV) of 0.27

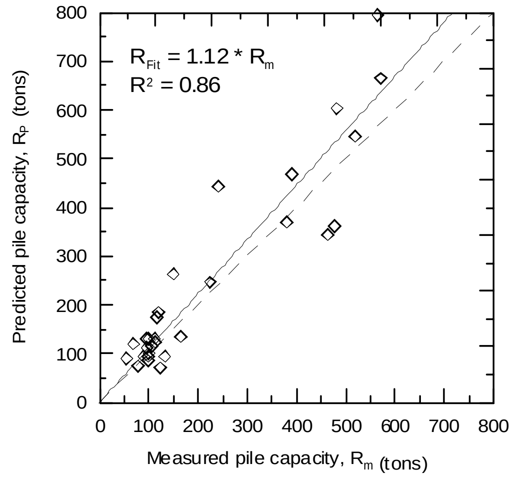
which is somewhat larger than the COV of the static analysis (0.25). Figures 4 (e) and 4 (f) present the results of signal matching using CAPWAP which shows the average end-of-driving (EOD) resistance is about 35 percent of the average load test value indicating a setup factor of 2.9. However, the data scatter is significantly higher than all other methods with a COV of 0.48. The 14-day beginning-of-restrike (BOR) data shows the CAPWAP estimated resistance still underestimates the measured value by 17%.

Table 3 Evaluation of different prediction methods

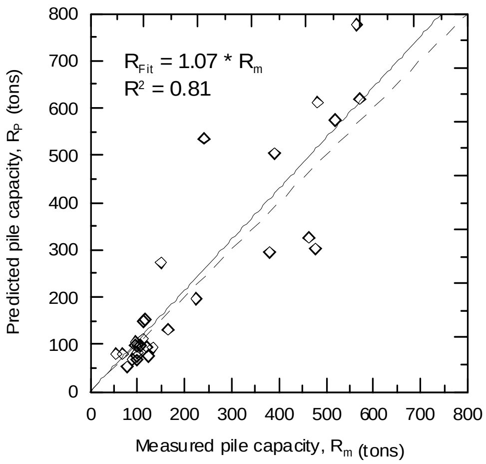
Pile Resistance Prediction Method	**No. of cases**	**Arithmetic calculations**				**Best fit calculations**	
		R_m/R_p			**R_p/R_m**		
		Mean	σ	**COV**	**Mean**	**R_{fit}/R_m**	**R^2**
Static method	33	0.97	0.24	0.25	1.11	0.96	0.87
Schmertmann method	29	0.93	0.28	0.30	1.17	1.12	0.86
LCPC method	29	1.07	0.32	0.30	1.04	1.07	0.81
De Ruiter& Beringen method	29	1.22	0.33	0.27	0.89	0.91	0.88
CAPWAP-EOD	12	3.65	1.74	0.48	0.35	0.32	0.69
CAPWAP-14 days BOR	8	1.32	0.51	0.39	0.83	0.92	0.91



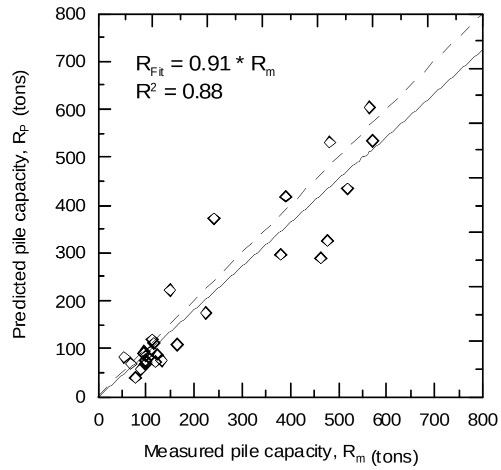
(a) Static analysis



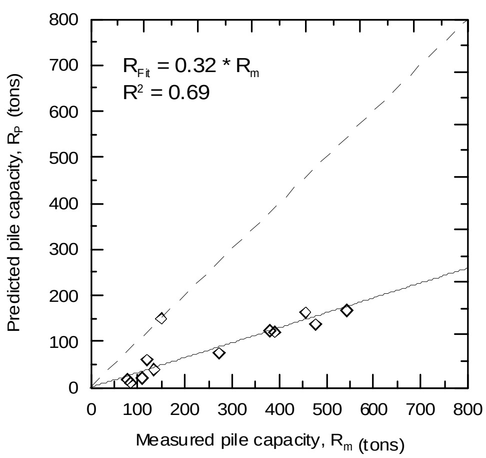
(b) Schmertmann method



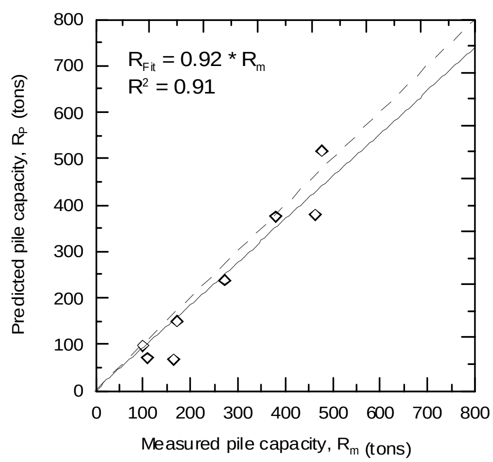
(c) LCPC method



(d) De Ruiter& Beringen method



(e) CAPWAP-EOD



(f) CAPWAP-14 days BOR

Figure 4 R_m versus R_p.

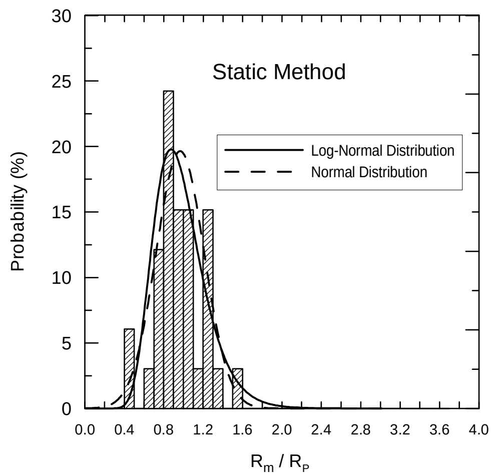
LRFD Calibration

Figures 5 (a) to 5 (f) show 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 and dynamic measurement with signal matching analysis (CAPWAP), respectively. As shown in the figures, generally the log-normal distribution matches better the histogram than the normal distribution, especially in lower tail region. 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 (34). In this study, the log-normal distribution was used to evaluate the different methods based on their prediction accuracy.

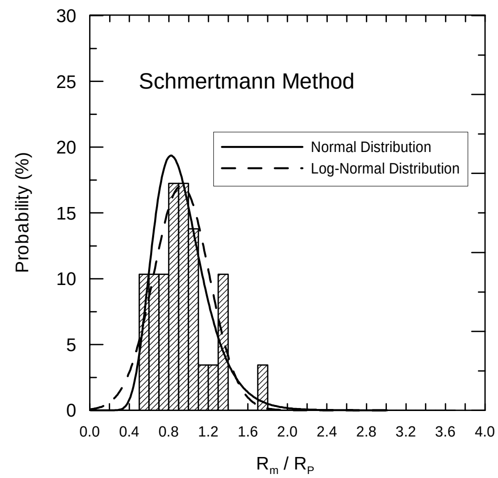
Reliability analyses were conducted and the resistance factors for all pile design methods were calibrated at Q_D/Q_L=3. This ratio was selected since the reliability index (β) converges for Q_D/Q_L exceeding 3 (25). Figures 6 (a) through 6 (f) present the resistance factors determined for various reliability indices (β) for the different pile design methods. A review of the literature indicates that the required reliability indices are between 2.33 and 3 for geotechnical applications. The resistance factors (ϕ) for different design method corresponding to reliability index of 2.33 are tabulated in Table 4. The resistance factor for the static method determined in this study is 0.56, which is slightly higher than the one recommended by AASHTO (11). It should be noted here 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 α-Tomlinson method dominantly with 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 (0.68); while the Schmertmann method shows the lowest resistance factor ($\phi_{Schmertmann}$ = 0.48), which is lower than the AASHTO value of 0.5. However, the LCPC method has a resistance factor equal to 0.56 (ϕ_{LCPC} = 0.56). For the dynamic measurement with signal matching analysis, the resistance factor obtained for the CAPWAP (EOD) is 1.31, which is higher than that of CPAWAP (14 days BOR) of 0.58. This is mainly due to the pile setup effect as was discussed in earlier section. Although the CAPWAP (EOD) has a high resistance factor, it is not economical and reliable approach because it significantly underestimates the resistance and has a high COV (0.48) and low efficiency factor of 0.36.

Table 4 Resistance Factors (ϕ) for Driven Piles (β_T = 2.33)

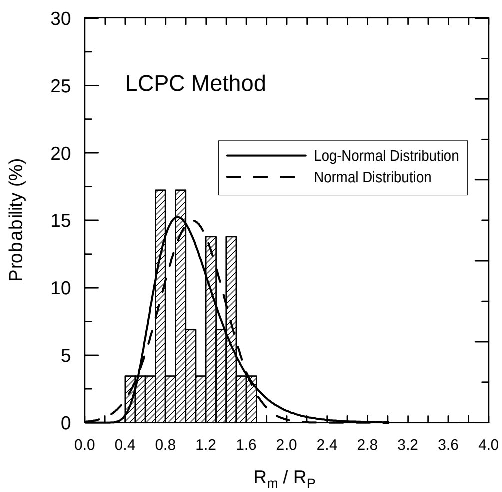
Design Method		**Resistance Factor, ϕ**		**Efficiency Factor (ϕ/λ)**
		Proposed for soft soil	**AASHTO (11)**	**Proposed for soft soil**
Static Method	α-Tomlinson method and Nordlund method	0.56	0.35 - 0.45	0.58
Direct CPT method	Schmertmann	0.48	0.5	0.52
	LCPC/LCP	0.56	NA	0.52
	De Ruiter and Beringen	0.68	NA	0.55
Dynamic measurement	CAPWAP (EOD)	1.31	NA	0.36
	CAPWAP (14 days BOR)	0.58	0.65	0.44



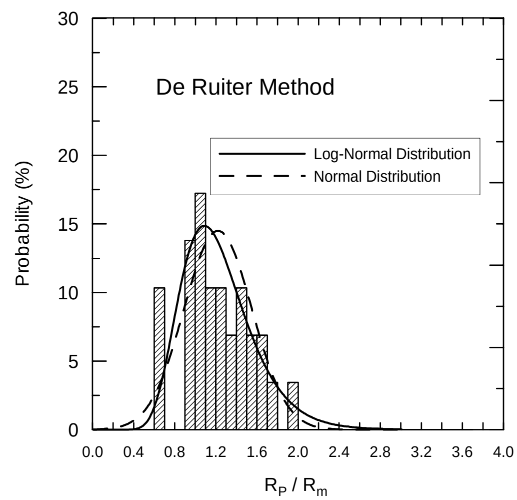
(a) Static analysis



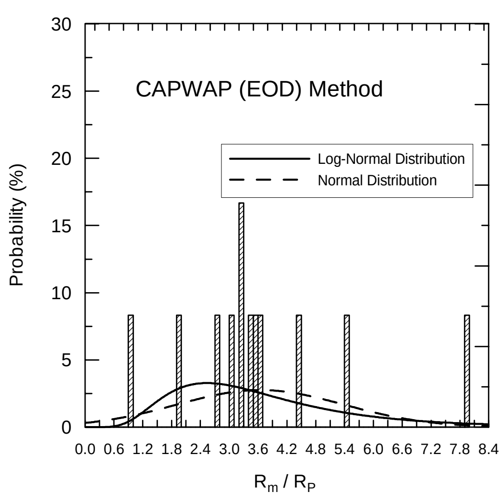
(b) Schmertmann method



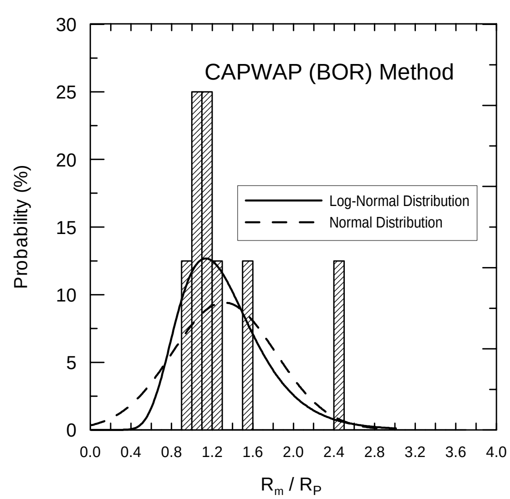
(c) LCPC method



(d) De Ruiter& Beringen method

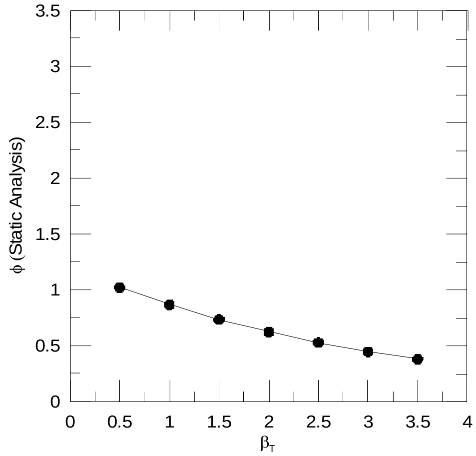


(e) CAPWAP-EOD

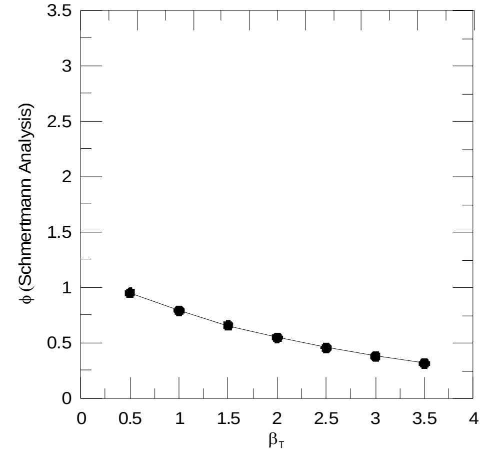


(f) CAPWAP-14 days BOR

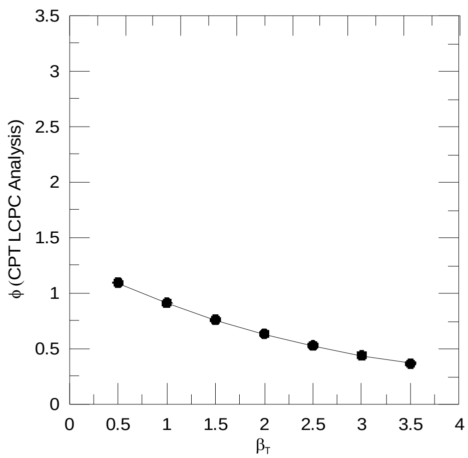
Figure 5 Histogram and probability density function of resistance bias factors.



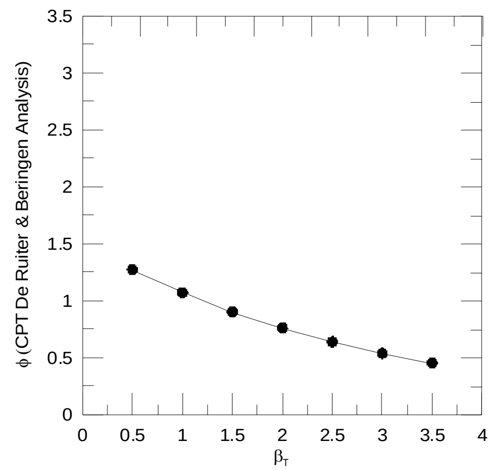
(a) Static analysis



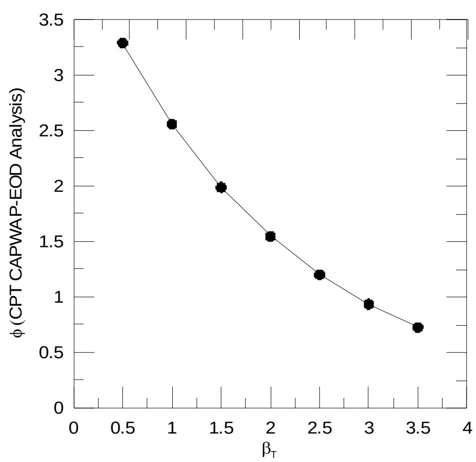
(b) Schmertmann method



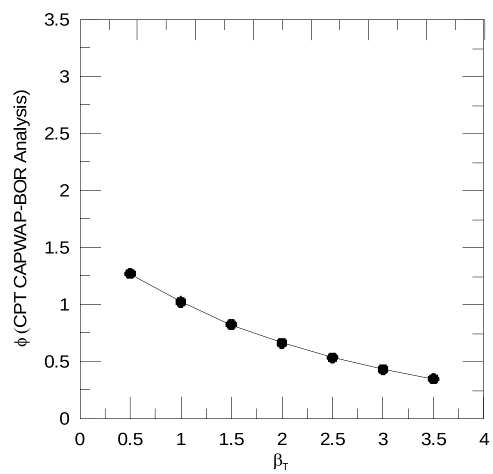
(c) LCPC method



(d) De Ruiter& Beringen method



(e) CAPWAP-EOD



(f) CAPWAP-14 days BOR

Figure 6 Resistance factors for different reliability indexes.

SUMMARY AND CONCLUSIONS

This paper presents a reliability based evaluation of different design methods for predicting the ultimate axial resistance of piles driven into soft Louisiana soils. The resistance factors (ϕ) of single driven piles, needed to implement the LRFD design methodology, were determined for each design method. A pile load test database of forty two square precast prestressed concrete piles of different sizes and lengths that were tested to failure were collected and used to calibrate the resistance factors. For each pile load test, the measured ultimate pile resistance was estimated using Davisson interpretation method and modified Davisson method for piles with 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), and the dynamic CAPWAP (EOD & 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- predicts the pile resistance by 11%. Among the three direct CPT methods, the De Ruiter-Beringen method is the most consistent prediction with the lowest COV. Both dynamic measurements with signal matching analysis methods (CAPWAP-EOD and 14 days BOR) show under-predication of pile resistance with setup factor of 2.9.

Reliability analyses based on first order second moment (FOSM) 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) correspond 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 $\beta_T = 2.33$, De Ruiter-Beringen method showed the highest resistance factor ($\phi_{De\text{-}Ruiter} = 0.68$); while Schmertmann method showed the lowest resistance factor ($\phi_{Schmertmann} = 0.48$), which is lower than AASHTO value of 0.5. The resistance factors for the static method and the LCPC/LCP method are both equal to 0.56. The resistance factor obtained for the CAPWAP (EOD) is 1.31, which is higher than CPAWAP (14 days BOR) resistance factor of 0.58. This is mainly due to pile setup. Although the CAPWAP (EOD) has a high resistance factor, it is not economical and reliable approach because it significantly underestimates the resistance and has a high COV and low efficiency factor.

ACKNOWLEDGEMENTS

This research project is funded by the Louisiana Transportation Research Center (LTRC Project No. 07-2GT) and Louisiana Department of Transportation and Development (State Project No. 736-99-1408). The authors gratefully acknowledge the support of Mark Morvant of LTRC.

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