# EVALUATION OF BEARING CAPACITY OF PILES FROM CONE PENETRATION TEST DATA

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# ABSTRACT

This study presents an evaluation of the performance of eight cone penetration test (CPT) methods in predicting the ultimate load carrying capacity of square precast prestressed concrete (PPC) piles driven into Louisiana soils. A search in the DOTD files was conducted to identify pile load test reports with cone penetration soundings adjacent to test piles. Sixty piles were identified, collected, and analyzed. The measured ultimate load carrying capacity for each pile was interpreted from the pile load test using Butler-Hoy method, which is the primary method used by DOTD. The following methods were used to predict the load carrying capacity of the collected piles using the CPT data: Schmertmann, Bustamante and Gianeselli (LCPC/LCP), de Ruiter and Beringen, Tumay and Fakhroo, Price and Wardle, Philipponnat, Aoki and De Alencar, and the penpile method. The ultimate load carrying capacity for each pile was also predicted using the static "-method, which is used by DOTD for pile design and analysis.

Prediction of pile capacity was performed on sixty piles, however, the statistical analyses and evaluation of the prediction methods were conducted based on the results of thirty five friction piles plunged (failed) during the pile load tests. End-bearing piles and piles that did not fail during the load tests were excluded from the statistical analyses.

An evaluation scheme was executed to evaluate the CPT methods based on their ability to predict the measured ultimate pile capacity. Four different criteria were selected to evaluate the ratio of the predicted to measured pile capacities. These criteria are: the best-fit line, the arithmetic mean and standard deviation, the cumulative probability, and the Log Normal distribution. Each criterion was used to rank the prediction methods based on its performance. The final rank of each method was obtained by averaging the ranks of the method from the four criteria. Based on this evaluation, the de Ruiter and Beringen and Bustamante and Gianeselli (LCPC/LCP) methods showed the best performance in predicting the load carrying capacity of square precast prestressed concrete (PPC) piles driven into Louisiana soils. The worst prediction method was the penpile, which is very conservative (underpredicted pile capacities).

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# IMPLEMENTATION STATEMENT

The results of this study demonstrated the capability of CPT methods in predicting the ultimate load carrying capacity of square PPC piles driven into Louisiana soils. de Ruiter and Beringen and Bustamante and Gianeselli (LCPC/LCP) methods showed the best performance in predicting the ultimate measured load carrying capacity of square PPC piles. It is strongly recommended that DOTD implement these two methods in design and analysis of square PPC piles. The Schmertmann method also showed good results and is recommended for implementation, since it is one of the most widely used CPT methods.

Cost-benefit analysis showed that the implementation would result in cost reduction in pile projects and timesaving without compromising the safety and performance of the pile supported structures. In fact, implementation of the CPT technology in pile design will reduce the level of uncertainties associated with traditional design methods.

In order to facilitate the implementation process, a computer program, Louisiana Pile Design by Cone Penetration Test (LPD-CPT), was developed for the design/analysis of square PPC driven piles from CPT data. The program, which is based on the MS-Windows platform, is easy to use and provides the profile of the pile load carrying capacity with depth.

Based on the results of the analyses, it is recommended that DOTD implement the cone penetration technology in different geotechnical applications within its practice. Regarding design and analysis of driven piles, the following steps are recommended:

1. Foster the confidence of DOTD design engineers in the CPT technology by adding the CPT to the list of the primary variables in subsurface exploration and use it in soil identification and classification and in site stratigraphy. Different soil classification methods can be used such as Zhang and Tumay, Robertson and Campanella, and Olsen and Mitchell.

2. Compare the test results from the traditional subsurface exploration methods and the results interpreted from the CPT methods. With time and experience, reduce the dependency level on the traditional subsurface exploration methods and increase dependency level on the CPT technology.

3. Use the CPT pile design methods in conjunction with the pile load tests and the static "-

method to predict the load carrying capacity of the square PPC piles. The following CPT methods are recommended: de Ruiter and Beringen method, Bustamante and Gianeselli (LCPC/LCP) method, and Schmertmann method. If a pile load test is conducted for the site, compare the results of the CPT methods with the measured ultimate pile load capacity. If the measured and predicted capacities are different, then make a correction to the predicted capacity in the amount of the difference between the measured and predicted capacity. Apply this correction to the other for the design of piles at this site.

4. Increase the role of the CPT design method and decrease the dependency on the static "-method.

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# INTRODUCTION

Among the different in situ tests, cone penetration test (CPT) is considered the most frequently used method for characterization of geomedia. The CPT is basically advancing a cylindrical rod with a cone tip into the soil and measuring the tip resistance and sleeve friction due to this intrusion. The resistance parameters are used to classify soil strata and to estimate strength and deformation characteristics of soils. Different devices added to cone penetrometers made it possible to apply this test for a wide range of geotechnical applications.

The CPT is a simple, quick, and economical test that provides reliable in situ continuous soundings of subsurface soil. Due to the soft nature of soil deposits in Louisiana, the CPT is considered a perfect tool for site characterization. Three CPT systems operate for the Louisiana Department of Transportation and Development (DOTD). These systems are Louisiana Electric Cone Penetration System (LECOPS), Research Vehicle for Geotechnical Insitu Testing and Support (REVEGITS), and Continuous Intrusion Miniature Cone Penetration Test system (CIMCPT). The CIMCPT system and REVEGITS are managed by the Louisiana Transportation Research Center (LTRC). Figure 1 depicts a photograph of the CIMCPT system and REVEGITS.

Deep foundations are usually used when the conditions of the upper soil layers are weak and unable to support the superstructural loads. Piles carry these superstructural loads deep in the ground. Therefore, the safety and stability of pile supported structures depend on the behavior of piles. Most soil deposits in southern Louisiana are soft in nature. In addition, the high percentage of wetlands, marshes, swamps, bayous, rivers, and lakes makes it necessary to consider deep foundations in the design of transportation infrastructure. Therefore, pile foundations are used by DOTD to support highway bridges and other transportation structures. The square precast prestressed concrete piles (PPC) are the most common piles currently used in DOTD projects.

Piles are expensive structural members, and pile projects are always costly. For example, DOTD spent about $19 million for driven piles in Louisiana in 1995 (DOTD Weighted Averages, 1996). Current DOTD practice of pile design is based on the static analysis ("-method) and some times in conjunction with the dynamic analysis using the Pile Driving Analyzer™. Soil properties are needed as input parameters for the static analysis. Therefore, it is necessary to conduct field and laboratory tests, which include soil boring, standard penetration test, unconfined compression test, soil classification, etc. Running these field and laboratory tests is

(a) Louisiana cone penetration test systems: the CIMCPT on the right and REGEVITS on the left

(b) The hydraulic push system of the REVEGITS

**Figure 1**
CPT systems managed by LTRC

expensive and time consuming. The cost of traditional soil boring and the associated laboratory tests ranges between $4,500 and $5,000, depending on the sampling depth and the laboratory tests involved.

Due to the uncertainties associated with pile design, load tests are usually conducted to verify the design loads and to evaluate the actual response of the pile under loading. Pile load tests are also expensive (the average cost of a pile load test in Louisiana is $15,000). Moreover, pile load tests are a verification tool for pile design and they cannot be a substitute for the engineering analysis of the pile behavior.

Cone penetration test can be utilized for a wide range of geotechnical engineering applications. Implementation of the CPT by DOTD is limited to identification of dense sand layers required to support the tip of the end-bearing piles. Moreover, DOTD uses the CPT to provide a supplemental subsurface information between soil borings. Unfortunately, these are very limited applications compared to the wide range of CPT applications. The CPT technology is fast, reliable, and cost effective especially when compared to the traditional site characterization method (borings and laboratory/field tests). The DOTD materials section CPT system can perform an average of six to eight tests per day. The estimated average cost per probe is $850. Compared to traditional borings, the CPT is faster and more economical. In subsurface exploration, the CPT can be effectively used to identify and classify soils and to evaluate the undrained shear strength. Implementation of the CPT can drastically decrease the number of soil borings and reduce the cost and time required for subsurface characterization. Therefore, implementation of the CPT technology by DOTD in different engineering applications should be seriously considered.

Due to the similarity between the cone and the pile, the prediction of pile capacity utilizing the cone data is considered among the earliest applications of the CPT. Cone penetration tests can provide valuable and continuous information regarding the *soil strength* with depth. Therefore, the in situ characteristics of the soil are available to the design engineers at a particular point. The pile design methods that utilize the CPT data proved to predict the pile capacity within an acceptable accuracy.

Generally, pile design depends on soil conditions, pile characteristics, and driving and installation conditions. Local experience usually played an important role in design/analysis of piles. Therefore, it is essential to take advantage of the DOTD experience in the CPT technology to identify suitable CPT design methods. Implementation of the CPT (in conjunction with the currently used method) in the analysis/design of piles will foster confidence in the CPT technology. With time and experience, the role of the CPT can be increased while the role of traditional subsurface exploration is reduced.

This report presents the research effort undertaken at LTRC to identify the most appropriate CPT methods for predicting the axial load carrying capacity of piles driven into Louisiana soils. To achieve this goal, state projects that have both pile load tests and CPT soundings were identified and collected from DOTD files. Pile load test reports were selected based on selection criteria, compiled onto sheets, and analyzed. The ultimate axial load carrying capacity for each pile was determined using the Butler-Hoy method, which is the primary load test interpretation method used by DOTD [*1*]. The CPT soundings close to the test pile location were identified and used to predict the ultimate pile capacity. Eight methods for predicting the ultimate pile capacity by CPT were selected. These methods are: Schmertmann, de Ruiter and Beringen, Bustamante and Gianeselli (LCPC/LCP), Tumay and Fakhroo, Aoki and De Alencar, Price and Wardle, Philipponnat, and the penpile method [*2*], [*3*], [*4*], [*5*], [*6*], [*7*], [*8*], [*9*]. The ultimate pile load carrying capacities predicted by the CPT methods were compared with the ultimate capacities obtained from pile load tests using Butler-Hoy method [*1*]. Statistical analyses were conducted to identify the most appropriate CPT method for predicting the ultimate capacity of the investigated piles.

In order to facilitate the implementation of the CPT capacity prediction methods, a Visual Basic MS-Windows program was developed and called Louisiana Pile Design by CPT(LPD-CPT). The program performs the analyses on the CPT soundings using the selected CPT method and provides the design engineers with pile ultimate capacity profile with depth.

In the current research, the existing data acquisition systems on the DOTD CPT systems are approaching obsolescence due primarily to the MS-DOS based applications required to operate the systems. Therefore, the data acquisition systems and software were updated to take advantage of the new available technologies and to provide DOTD personnel with better performance systems.

# OBJECTIVE

The goal of this research is to identify the most appropriate methods for estimating the ultimate axial load carrying capacity of piles from the cone penetration test data.

To achieve the objective of this research, the following tasks were executed:

i. Identification of the state projects that have both pile load test and cone penetration soundings close to the pile location. A total of 60 pile load test reports were collected from DOTD files based on this criterion.

ii. Comprehensive literature review to investigate and evaluate methods of estimating the load carrying capacity of piles using cone penetration test data.

iii. Identification of the most reliable CPT methods based on their ability to predict the load carrying capacity of square PPC piles driven into Louisiana soils.

iv. Implementation of these methods into MS-Windows based program, LPD-CPT, to facilitate their use by DOTD design engineers.

# SCOPE

This research effort was focused on the applicability of eight CPT methods to predict the ultimate axial compression load carrying capacity of piles from CPT data. These methods are described in detail in the *Background* section of this report. The predicted capacity was compared to the reference pile load capacity obtained from the pile load test using Butler-Hoy method [*1*].

The CPT methods were used to investigate the load carrying capacity of square precast prestressed concrete (PPC) piles of different sizes driven into Louisiana soils. Other pile types such as timber piles and steel pipes were not covered in the current analyses. Moreover, the analyses were conducted only on piles that were loaded to failure during the load test.

The CPT data used in this report are those acquired by the 10 and 15 $cm^2$ friction cone penetrometers. In these tests the total cone tip resistance ($q_c$) and sleeve friction ($f_s$) were recorded and no pore water pressures were measured. However, the selected CPT methods used in this investigation were developed based on the total cone tip resistance ($q_c$) and sleeve friction ($f_s$).

# BACKGROUND

## PILE FOUNDATIONS

Piles are relatively long and generally slender structural foundation members that transmit superstructure loads to deep soil layers. In geotechnical engineering, piles usually serve as foundations when soil conditions are not suitable for the use of shallow foundations. Moreover, piles have other applications in deep excavations and in slope stability. As presented in the literature, piles are classified according to:

a. the nature of load support (friction and end-bearing piles),
b. the displacement properties (full-displacement, partial-displacement, and non-displacement piles), and
c. the composition of piles (timber, concrete, steel, and composite piles).

The behavior of the pile depends on many different factors, including pile characteristics, soil conditions and properties, installation method, and loading conditions. The performance of piles affects the serviceability of the structure they support.

The prediction of pile load carrying capacity can be achieved using different methods such as pile load test, dynamic analysis, static analysis based on soil properties from laboratory tests, and static analysis utilizing the results of in situ tests such as cone penetration test.

In the design and analysis of piles, it is important to identify piles based on the nature of support provided by the surrounding soil, i.e. to classify piles as end-bearing piles and friction piles. While end-bearing piles transfer most of their loads to an end-bearing stratum, friction piles resist a significant portion of their loads via the skin friction developed along the surface of the piles. The behavior of friction piles mainly depends on the interaction between the surrounding soil and the pile shaft.

The ultimate axial load carrying capacity of the pile ($Q_u$) composed of the end-bearing capacity of the pile ($Q_t$) and the shaft friction capacity ($Q_s$). The general equation described in the literature is given by:

$$Q_u = Q_t + Q_s = q_t A_t + fA_s \tag{1}$$

where $q_t$ is the unit tip bearing capacity, $A_t$ is the area of the pile tip, $f$ is the unit skin friction, and $A_s$ is the area of the pile shaft. In sands, the end-bearing capacity ($Q_t$) dominates, while in soft clays the shaft friction capacity ($Q_s$) dominates. The design load carrying capacity ($Q_d$) of the pile can be calculated by:

$$Q_d = \frac{Q_u}{F.S.} \tag{2}$$

where $Q_u$ is the ultimate load carrying capacity and $F.S.$ is the factor of safety.

**CONE PENETRATION TEST**

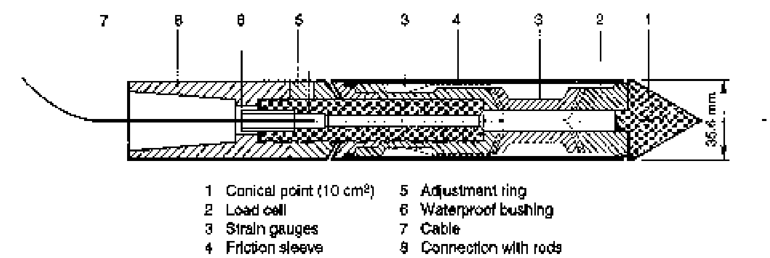
The cone penetration test has been recognized as one of the most widely used in situ tests. In the United States, cone penetration testing has gained rapid popularity in the past twenty years. The cone penetration test consists of advancing a cylindrical rod with a conical tip into the soil and measuring the forces required to push this rod. The friction cone penetrometer measures two forces during penetration. These forces are: the total tip resistance ($q_c$), which is the soil resistance to advance the cone tip and the sleeve friction ($f_s$), which is the sleeve friction developed between the soil and the sleeve of the cone penetrometer. The friction ratio ($R_f$) is defined as the ratio between the sleeve friction and tip resistance and is expressed in percent. A schematic of the electric cone penetrometer is depicted in figure 2. The resistance parameters are used to classify soil strata and to estimate strength and deformation characteristics of soils.

The cone penetration test data has been used to predict the ultimate axial pile load carrying capacity. Several methods are available in the literature to predict the axial pile capacity utilizing the CPT data. These methods can be classified into two well-known approaches:

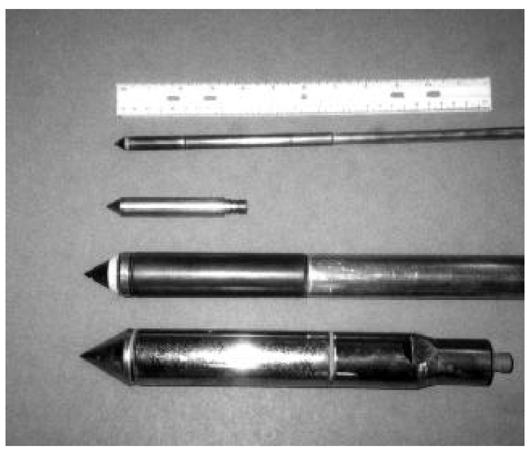
(1) Direct approach in which

- The unit tip bearing capacity of the pile ($q_t$) is evaluated from the cone tip resistance ($q_c$) profile.
- The unit skin friction of the pile ($f$) is evaluated from either the sleeve friction ($f_s$) profile or the cone tip resistance ($q_c$) profile.

(2) Indirect approach: in which the CPT data ($q_c$ and $f_s$) are first used to evaluate the soil strength parameters such as the undrained shear strength ($S_u$) and the angle of internal friction (N). These




(a) Schematic of the electric friction cone penetrometer



(b) The 1.27, 2, 10, and 15 $cm^2$ cone penetrometers used at LTRC

**Figure 2**
The electric cone penetrometer

parameters are then used to evaluate the unit tip bearing capacity of the pile ($q_t$) and the unit skin friction of the pile ($f$) using formulas derived based on semi-empirical/theoretical methods.

In the current research, only the direct methods of predicting the pile capacity from cone penetration test data are investigated.

## PREDICTION OF PILE CAPACITY BY CPT

In this report, the direct methods are described in detail. These methods are Schmertmann, de Ruiter and Beringen, Bustamante and Gianeselli (LCPC/LPC), Tumay and Fakhroo (cone-m), Aoki and De Alencar, Price and Wardle, Philipponnat, and the penpile method [*2*], [*3*], [*4*], [*5*], [*6*], [*7*], [*8*], [*9*]. The direct CPT methods evaluate the unit tip bearing capacity of the pile ($q_t$) from the measured cone tip resistance ($q_c$) by averaging the cone tip resistance over an assumed influence zone. The unit shaft resistance ($f$) is either evaluated from the measured sleeve friction ($f_s$) in some methods or from the measured cone tip resistance ($q_c$) in others.

### Schmertmann Method

Schmertmann  proposed the following relationship to predict the unit tip bearing capacity of the pile ($q_t$) from the cone tip resistance ($q_c$):

$$q_t = \frac{q_{c1} + q_{c2}}{2} \tag{3}$$

where $q_{c1}$ is the minimum of the average cone tip resistances of zones ranging from *0.7D to 4D* below the pile tip (where $D$ is the pile diameter) and $q_{c2}$ is the average of minimum cone tip resistances over a distance *8D* above the pile tip. To determine $q_{c1}$, the minimum path rule is used as illustrated in figure 3. The described zone (from *8D* above to *0.7D-4D* below the pile tip) represents the failure surface, which is approximated by a logarithmic spiral. Schmertmann suggested an upper limit of 150 TSF (15 MPa) for the unit tip bearing capacity ($q_t$).

According to Schmertmann's method, the unit skin friction of the pile ($f$) is given by:

$$f = \alpha_c f_s \tag{4}$$

where $\alpha_c$ is a reduction factor, which varies from 0.2 to 1.25 for clayey soil, and $f_s$ is the sleeve friction. Figure 4 depicts the variation of $\alpha_c$ with $f_s$ for different pile types in clay.

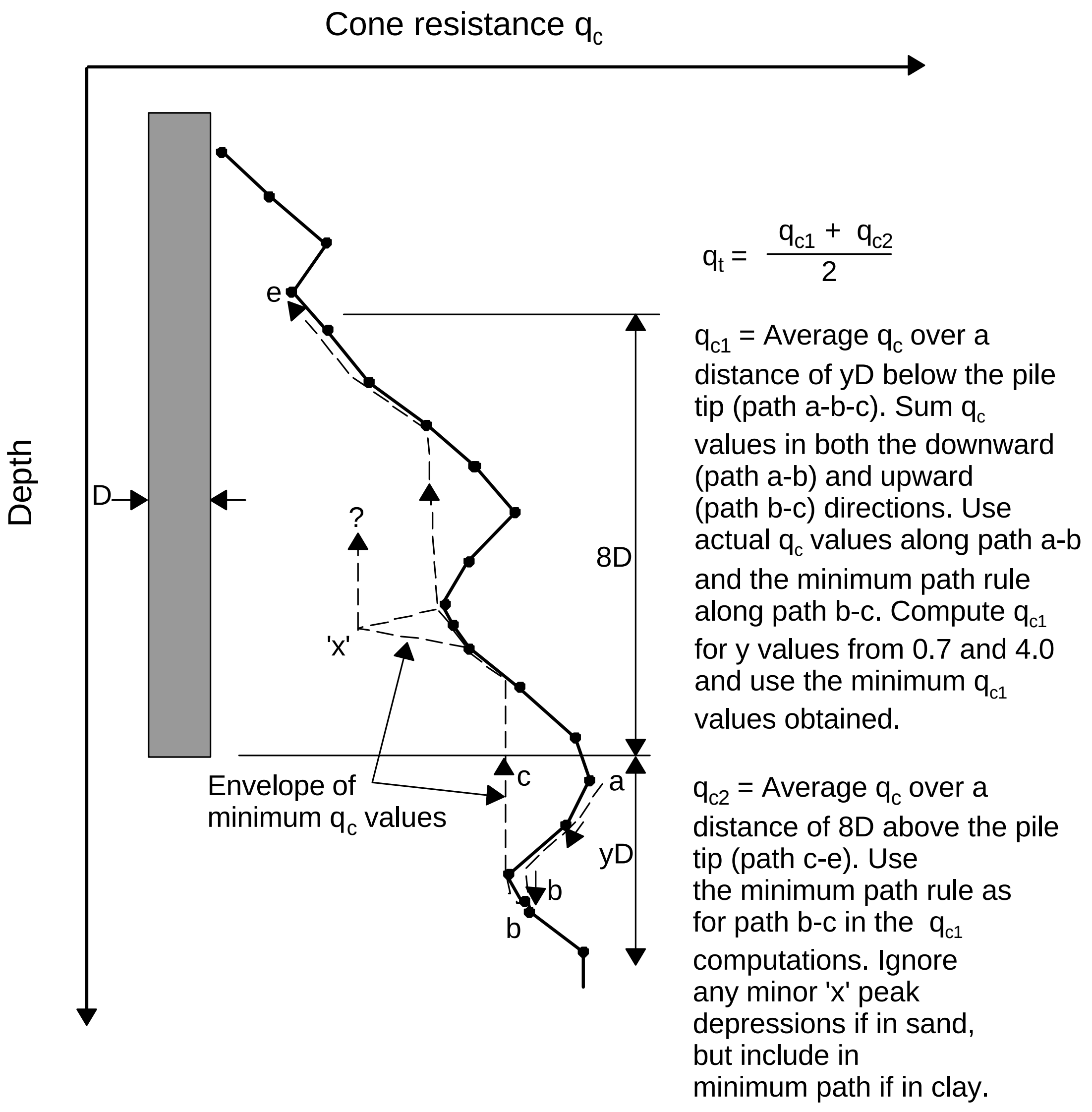



**Figure 3**

Calculation of the average cone tip resistance in Schmertmann method [2]

For piles in sand, the friction capacity ($Q_s$) is obtained by:

$$Q_s = a_s \left( \sum_{y=0}^{8D} \frac{y}{8D} f_s A_s + \sum_{y=8D}^{L} f_s A_s \right) \tag{5}$$

where $''_s$ is the correction factor for sand, which can be obtained from figure 5, *y* is the depth at which side resistance is calculated, and *L* is the pile length.

Schmertmann suggested a limit of 1.2 TSF (120 kPa) on *f*.

**de Ruiter and Beringen Method**

This method is proposed by de Ruiter and Beringen and is based on the experience gained in the North Sea [*3*]. This method is also known as the European method and uses different procedures for clay and sand.

In clay, the undrained shear strength ($S_u$) for each soil layer is first evaluated from the cone tip resistance ($q_c$). Then, the unit tip bearing capacity and the unit skin friction are computed by applying suitable multiplying factors. The unit tip bearing capacity is given by:

$$\begin{aligned} q_t &= N_c S_u(tip) \\ S_u(tip) &= \frac{q_c(tip)}{N_k} \end{aligned} \tag{6}$$

where $N_c$ is the bearing capacity factor and $N_c$=9 is considered by this method. $N_k$ is the cone factor that ranges from 15 to 20, depending on the local experience. $q_c(tip)$ is the average of cone tip resistances around the pile tip computed similar to Schmertmann method.

The unit skin friction is given by:

$$f = bS_u(side) \tag{7}$$

where $\$$ is the adhesion factor, $\$$=1 for normally consolidated (NC) clay, and $\$$ =0.5 for

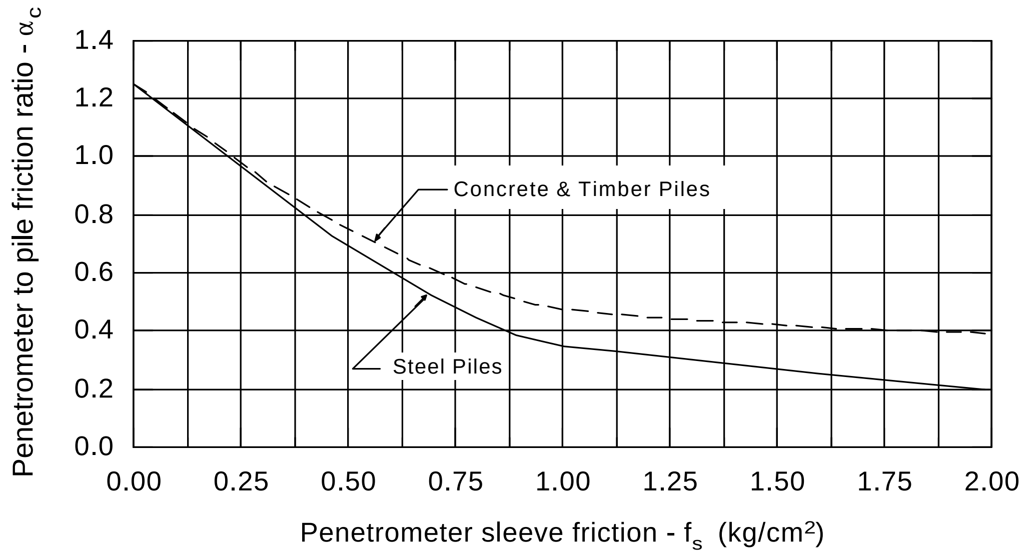



**Figure 4**

Penetration design curves for pile side friction in clay in Schmertmann method [2]

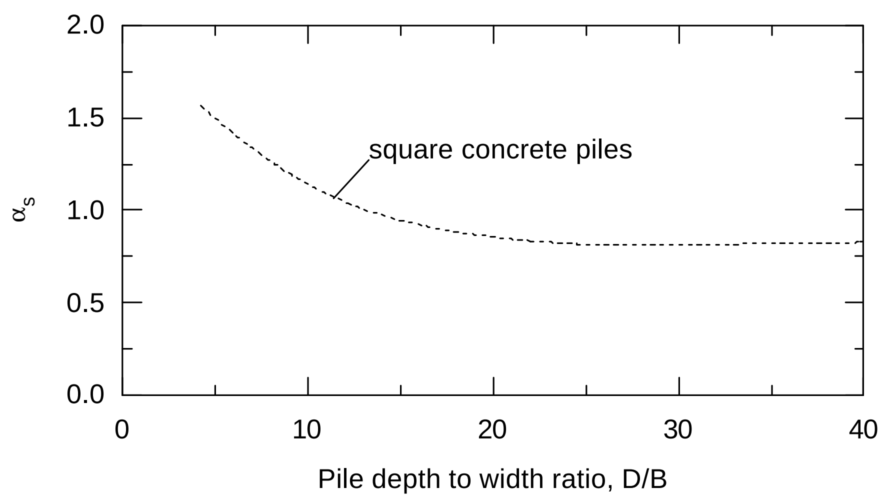



**Figure 5**

Penetrometer design curve for side pile friction in sand in Schmertmann method [2] overconsolidated (OC) clay. $S_u$(*side*), the undrained shear strength for each soil layer along the pile shaft, is determined by:

$$S_u(side) = \frac{q_c(side)}{N_k} \tag{8}$$

where $q_c$(*side*) is the average cone tip resistance along the soil layer.

In the current study, the cone factor $N_k$=20 and the adhesion factor $\$$ =0.5 were adopted in the analysis, since these values gave better predicted ultimate pile capacity for the investigated piles.

In sand, the unit tip bearing capacity of the pile ($q_t$) is calculated similar to Schmertmann method. The unit skin friction ($f$) for each soil layer along the pile shaft is given by:

$$f = \min \begin{cases} f_s \\ \dfrac{q_c(side)}{300} \text{ (compression)} \\ \dfrac{q_c(side)}{400} \text{ (tension)} \\ 1.2 \text{ TSF (120kPa)} \end{cases} \tag{9}$$

de Ruiter and Beringen imposed limits on $q_t$ and $f$ in which $q_t$# 150 TSF (15 MPa) and $f$# 1.2 TSF (120 kPa).

**Bustamante and Gianeselli Method (LCPC/LCP Method)**

Bustamante and Gianeselli proposed this method for the French Highway Department based on the analysis of 197 pile load tests with a variety of pile types and soil conditions [*4*]. It is also known as the French method and the LCPC/LCP method. In this method, both the unit tip bearing capacity ($q_t$) and the unit skin friction ($f$) of the pile are obtained from the cone tip resistance ($q_c$). The sleeve friction ($f_s$) is not used. The unit tip bearing capacity of the pile ($q_t$) is predicted from the following equation:

$$q_t = k_b q_{eq}(tip) \tag{10}$$

where $k_b$ is an empirical bearing capacity factor that varies from 0.15 to 0.60 depending on the soil type and pile installation procedure (table 1) and $q_{eq}(tip)$ is the equivalent average cone tip resistance around the pile tip, which is obtained as follows:

1. calculate the average tip resistance ($q_{ca}$) at the tip of the pile by averaging $q_c$ values over a zone ranging from 1.5$D$ below the pile tip to 1.5$D$ above the pile tip ($D$ is the pile diameter),
2. eliminate $q_c$ values in the zone that are higher than $1.3q_{ca}$ and those are lower than $0.7q_{ca}$ as shown in figure 6, and
3. calculate the equivalent average cone tip resistance ($q_{eq}(tip)$) by averaging the remaining cone tip resistance ($q_c$) values over the same zone (bordered by thick lines in figure 6).

The pile unit skin friction ($f$) in each soil layer is estimated from the equivalent cone tip resistance ($q_{eq}(side)$) of the soil layer, soil type, pile type, and installation procedure. The following procedure explains how to determine the unit skin friction ($f$):

A. based on the pile type, select the pile category from table 2 (for example, pile category is 9 for square PPC piles),

B. for each soil layer, select the appropriate curve number (tables 3 and 4) based on soil type, equivalent cone tip resistance along the soil layer ($q_{eq}(side)$), and pile category, use table 3 for clay and silt and table 4 for sand and gravel,

C. from figure 7, use the selected curve number and the equivalent cone tip resistance ($q_{eq}(side)$) to obtain the maximum unit skin friction ($f$), use figure 7a for clay and silt and figure 7b for sand and gravel.

**Table 1**

LCPC bearing capacity factor ($k_b$)

| Soil Type | Bored Piles | Driven Piles |
|---|---|---|
| Clay-Silt | 0.375 | 0.60 |
| Sand-Gravel | 0.15 | 0.375 |
| Chalk | 0.20 | 0.40 |

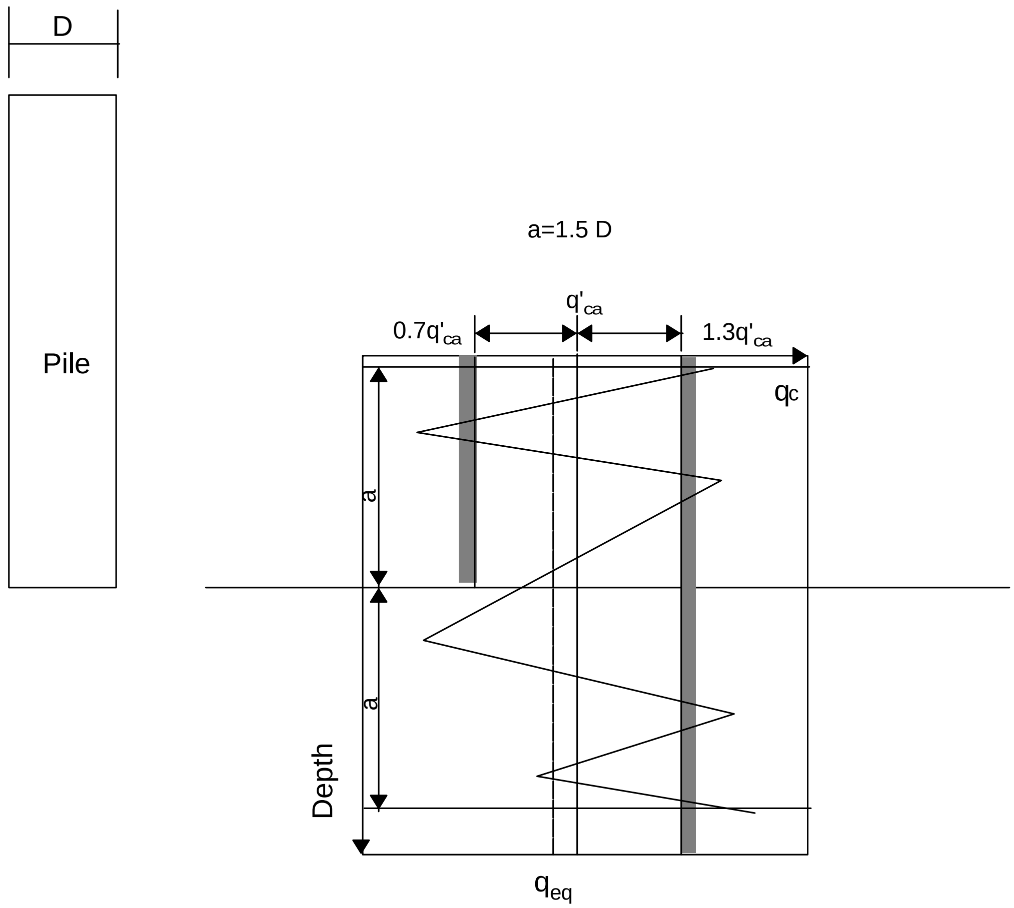



**Figure 6**

Calculation of the equivalent average tip resistance for LCPC method (after Bustamante and Gianeselli [4])

**Table 2**
Pile categories for the LCPC method

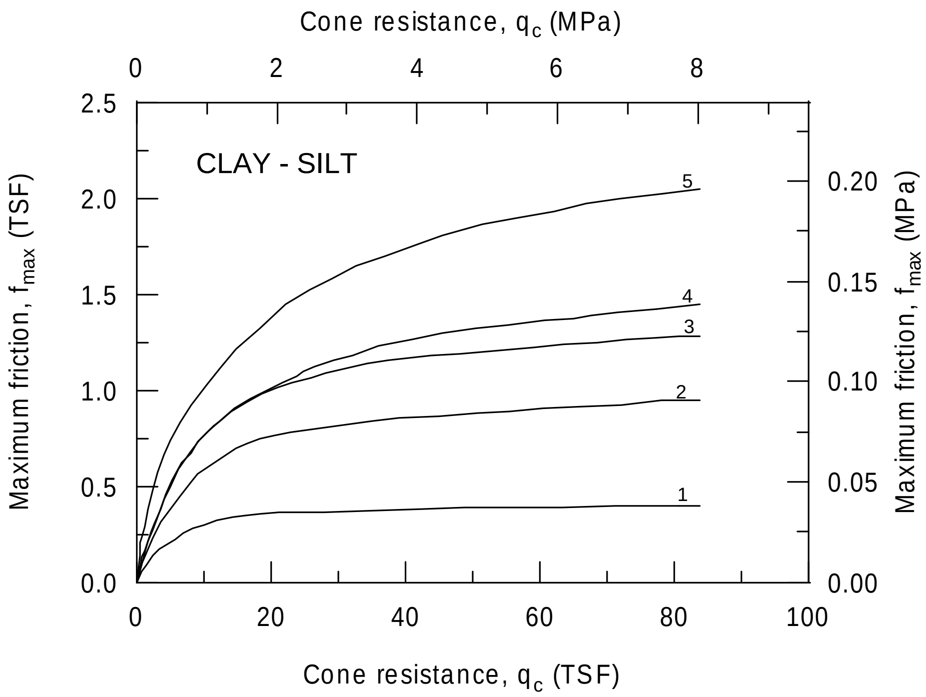
| | |
|---|---|
| 1. FS Drilled shaft with no drilling mud | Installed without supporting the soil with drilling mud. Applicable only for cohesive soils above the water table. |
| 2. FB Drilled shaft with drilling mud | Installed using mud to support the sides of the whole. Concrete is poured from the bottom up, displacing the mud. |
| 3. FT Drilled shaft with casing (FTU) | Drilled within the confinement of a steel casing. As the casting is retrieved, concrete is poured in the hole. |
| 4. FTC Drilled shaft, hollow auger (auger cast piles) | Installed using a hollow stem continuous auger having a length at least equal to the proposed pile length. The auger is extracted without turning while, simultaneously, concrete is injected through the auger stem. |
| 5. FPU Pier | Hand excavated foundations. The drilling method requires the presence of workers at the bottom of the excavation. The sides are supported with retaining elements or casing. |
| 6. FIG Micropile type1 (BIG) | Drilled pile with casting. Diameter less than 250 mm (10 inch). After the casting has been filled with concrete, the top of the casing is plugged. Pressure is applied inside the casting between the concrete and the plug. The casing is recovered by maintaining the pressure against the concrete. |
| 7. VMO Screwed-in piles | Not applicable for cohesionless or soils below water table. A screw type tool is placed in front of a corrugated pipe which is pushed and screwed in place. The rotation is reversed for pulling out the casting while concrete is poured. |
| 8. BE Driven piles, concrete coated | - pipe piles 150 mm (6 in.) To 500 mm (20 in.) External diameter<br>- H piles<br>- caissons made of 2, 3, or 4 sheet pile sections.<br>The pile is driven with an oversized protecting shoe. As driving proceeds, concrete is injected through a hose near the oversized shoe producing a coating around the pile. |
| 9. BBA Driven prefabricated piles | Reinforced or prestressed concrete piles installed by driving or vibrodriving. |
| 10. BM Steel driven piles | Piles made of steel only and driven in place.<br>- H piles<br>- Pipe piles<br>- any shape obtained by welding sheet-pile sections. |
| 11. BPR Prestressed tube pile | Made of hollow cylinder elements of lightly reinforced concrete assembled together by prestressing before driving. Each element is generally 1.5 to 3 m (4-9 ft) long and 0.7 to 0.9 m (2-3 ft) in diameter; the thickness is approximately 0.15 m (6 in.). The piles are driven open ended. |
| 12. BFR Driven pile, bottom concrete plug | Driving is achieved through the bottom concrete plug. The casting is pulled out while low slump concrete is compacted in it. |
| 13. BMO Driven pile, molded. | A plugged tube is driven until the final position is reached. The tube is filled with medium slump concrete to the top and the tube is extracted. |
| 14. VBA Concrete piles, pushed-in. | Pile is made of cylindrical concrete elements prefabricated or cast-in-place, 0.5 to 2.5 m (1.5 to 8 ft) long and 30 to 60 cm (1 to 2 ft) in diameter. The elements are pushed in by a hydraulic jack. |
| 15. VME Steel piles, pushed-in | Piles made of steel only are pushed in by a hydraulic jack.. |
| 16. FIP Micropile type II | Drilled pile < 250 mm ( 10 in.) In diameter. The reinforcing cage is placed in the hole and concrete placed from bottom up. |
| 17. BIP High pressure injected pile, large diameter | Diameter > 250 mm (10 in.). The injection system should be able to produce high pressures. |

**Table 3**
Input parameters for clay and silt for LCPC method

| CURVE # | $q_c$ (ksf) | PILE TYPE (see Table 2) | COMMENTS ON INSERTION PROCEDURE |
|---|---|---|---|
| 1 | < 14.6 | 1-17 | |
| | > 14.6 | 1,2 | - very probable values when using tools without teeth or with oversized blades and where a remoulded layer of material can be deposited along the sides of the drilled hole. Use these values also for deep holes below the water table where the hole must be cleaned several times. Use these values also for cases when the relaxation of the sides of the hole is allowed due to incidents slowing or stopping the pouring of concrete. For all the previous conditions, experience shows, however, that $q_s$ can be between curves 1 and 2; use an intermediate value of $q_s$ is such value is warranted by a load test. |
| 2 | > 25.1 | 4, 5, 8, 9, 10, 11, 13, 14, 15 | - for all steel piles , experience shows that, in plastic soils, $q_s$ is often as low as curve 1; therefore, use curve 1 when no previous load test is available. For all driven concrete piles use curve 3 in low plasticity soils with sand or sand and gravel layers or containing boulders and when $q_c$>52.2 ksf. |
| | > 25.1 | 7 | - use these values for soils where $q_c$<52.2 ksf and the rate of penetration is slow; otherwise use curve 1. Also for slow penetration, when $q_c$>93.9 ksf, use curve 3. |
| | | | - use curve 3 based on previous load test. |
| | > 25.1 | 6 | - use these values when careful method of drilling with an auger equipped with teeth and immediate concrete pouring is used. In the case of constant supervision with cleaning and grooving of the borehole walls followed by immediate concrete pouring, for soils of $q_c$>93.9 ksf, curve 3 can be used. |
| | > 25.1 | 1, 2 | |
| | > 25.1 | 3 | - for dry holes. It is recommended to vibrate the concrete after taking out the casing. In the case of work below the water table, where pumping is required and frequent movement of the casing is necessary, use curve 1 unless load test results are available. |
| 3 | > 25.1<br>< 41.8 | 12 | - usual conditions of execution as described in DTU 13.2 |
| 5 | > 14.8 | 16, 17 | - in the case of injection done selectively and repetitively at low flow rate it will be possible to use curve 5, if it is justified by previous load test. |

**Table 4**
Input parameters for sand and gravel for LCPC method

| CURVE # | $q_c$ (ksf) | PILE TYPE (see Table 2 ) | COMMENTS ON INSERTION PROCEDURE |
|---|---|---|---|
| 1 | < 73.1 | 2, 3, 4, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15 | |
| 2 | > 73.1 | 6, 7, 9, 10, 11, 12, 13, 14, 15 | - for fine sands. Since steel piles can lead to very small values of $q_s$ in such soils, use curve 1 unless higher values can be based on load test results. For concrete piles, use curve 2 for fine sands of $q_c$>156.6 ksf. |
| | >104.4 | 2, 3 | - only for fine sands and bored piles which are less than 30 m (100 ft) long. For piles longer than 30 m (100 ft) in fine sand, $q_s$ may vary between curves 1 and 2. Where no load test data is available, use curve 1. |
| | >104.4 | 4 | - reserved for sands exhibiting some cohesion. |
| 3 | > 156.6 | 6, 7, 9, 10, 11, 13, 14, 15, 17 | - for coarse gravelly sand or gravel only. For concrete piles, use curve 4 if it can be justified by a load test. |
| | > 156.6 | 2, 3 | - for coarse gravelly sand or gravel and bored piles less than 30 m (100 ft) long. |
| | | | - for gravel where $q_c$>83.5 ksf, use curve 4 |
| 4 | > 156.6 | 8, 12 | - for coarse gravelly sand and gravel only. |
| 5 | > 104.4 | 16, 17 | - use of values higher than curve 5 is acceptable if based on load test. |



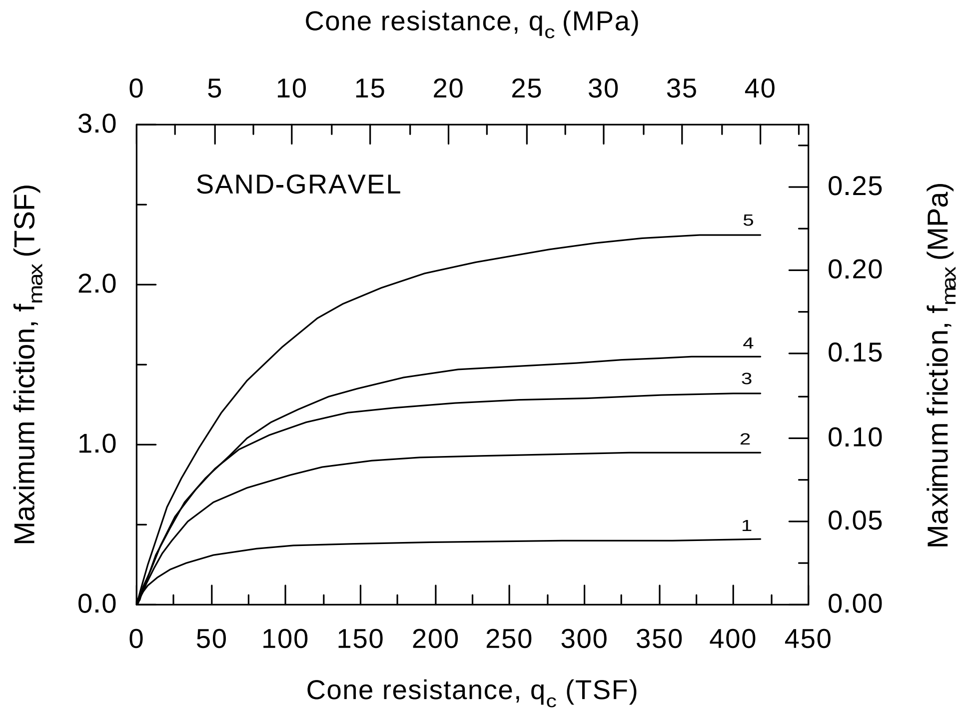




**Figure 7**

Maximum friction curves for LCPC method (after Briaud [10])

**Tumay and Fakhroo Method (Cone-m Method)**

Tumay and Fakhroo proposed this method to predict the ultimate pile capacity of piles in clayey soils [*5*]. The unit tip bearing capacity ($q_t$) is estimated using a procedure similar to Schmertmann's method as follows:

$$q_t = \frac{q_{c1} + q_{c2}}{4} + \frac{q_a}{2} \tag{11}$$

where $q_{c1}$ is the average of $q_c$ values 4*D* below the pile tip, $q_{c2}$ is the average of the minimum $q_c$ values 4*D* below the pile tip, and $q_a$ is the average of the minimum of $q_c$ values 8*D* above the pile tip. Tumay and Fakhroo suggested an upper limit of 150 TSF (15 MPa) for the unit pile tip bearing capacity ($q_t$).

The unit skin friction ($f$) is given by the following expression:

$$f = mf_{sa} \tag{12}$$

Tumay and Fakhroo suggested that $f$#0.72 TSF (72 kPa). The adhesion factor ($m$) is expressed as:

$$m = 0.5 + 9.5e^{-9 f_{sa}} \tag{13}$$

where $f_{sa}=F_t/L$ is the average local friction in TSF, and $F_t$ is the total cone penetration friction determined for pile penetration length ($L$).

**Aoki and De Alencar Method**

Aoki and De Alencar Velloso proposed the following method to estimate the ultimate load carrying capacity of the pile from CPT data [*6*]. The unit tip bearing capacity ($q_t$) is obtained from:

$$q_t = \frac{q_{ca}(tip)}{F_b} \tag{14}$$

where $q_{ca}(tip)$ is the average cone tip resistance around the pile tip, and $F_b$ is an empirical factor that depends on the pile type. The unit skin friction of the pile ($f$) is predicted by:

$$f = q_c(side)\frac{\alpha_s}{F_s} \tag{15}$$

where $q_c(side)$ is the average cone tip resistance for each soil layer along the pile shaft, $F_s$ is an empirical factor that depends on the pile type and $\alpha_s$ is an empirical factor that depends on the soil type. Factors $F_b$ and $F_s$ are given in table 5. The values of the empirical factor $\alpha_s$ are presented in table 6.

**Table 5**

Empirical factors $F_b$ and $F_s$

| **Pile Type** | $F_b$ | $F_s$ |
|---|---|---|
| Bored | 3.5 | 7.0 |
| Franki | 2.5 | 5.0 |
| Steel | 1.75 | 3.5 |
| Precast concrete | 1.75 | 3.5 |

**Table 6**

The empirical factor $\alpha_s$ values for different soil types

| **Soil Type** | $\alpha_s$ **(%)** | **Soil Type** | $\alpha_s$ **(%)** | **Soil Type** | $\alpha_s$ **(%)** |
|---|---|---|---|---|---|
| Sand | 1.4 | Sandy silt | 2.2 | Sandy clay | 2.4 |
| Silty sand | 2.0 | Sandy silt with clay | 2.8 | Sandy clay with silt | 2.8 |
| Silty sand with clay | 2.4 | Silt | 3.0 | Silt clay with sand | 3.0 |
| Clayey sand with silt | 2.8 | Clayey silt with sand | 3.0 | Silty clay | 4.0 |
| Clayey sand | 3.0 | Clayey silt | 3.4 | Clay | 6.0 |

In the current study, the following were used as reference values: for sand $\alpha_s$=1.4 percent, for silt $\alpha_s$=3.0percent, and for clay $\alpha_s$=6.0 percent. For soils consist of combination of sand, silt, and clay, $\alpha_s$ values were interpolated based on the probability percentages of sand, silt, and clay in that soil. For example if the probabilistic region estimation (refer to section *Soil Classification by CPT* in *Background*) of a soil gives 50 percent clay, 20 percent silt, and 30 percent sand then $\alpha_s = 0.50\times\alpha_s(\text{clay})+0.20\times\alpha_s(\text{silt})+0.30\times\alpha_s(\text{sand}) = 0.5\times6+0.2\times3+0.3\times1.4 = 4.02$ percent.

Upper limits were imposed on $q_t$ and $f$ as follows: $q_t$#150 TSF (15 MPa) and $f$#1.2 TSF (120 kPa).

**Price and Wardle Method**

Price and Wardle proposed the following relationship to evaluate the unit tip bearing capacity ($q_t$) of the pile from the cone tip resistance [7]:

$$q_t = k_b q_c \tag{16}$$

where $k_b$ is a factor depends on the pile type ($k_b$ = 0.35 for driven piles and 0.3 for jacked piles). The unit skin friction ($f$) is obtained from:

$$f = k_s f_s \tag{17}$$

where $k_s$ is a factor depends on the pile type ($k_s$=0.53 for driven piles, 0.62 for jacked piles, and 0.49 for bored piles). Price and Wardle proposed the values for these factors based on analysis conducted on pile load tests in stiff clay (London clay).

Upper limits were imposed on $q_t$ and $f$ as follows: $q_t$#150 TSF (15 MPa) and $f$#0.12 TSF (120 kPa).

**Philipponnat Method**

Philipponnat proposed the following expression to estimate the unit tip bearing capacity of the pile ($q_t$) from the cone tip resistance ($q_c$) [8]:

$$q_t = k_b q_{ca} \tag{18}$$

where $k_b$ is a factor that depends on the soil type as shown in table 7. The cone tip resistance ($q_{ca}$) is averaged as follows:

$$q_{ca} = \frac{q_{ca(A)} + q_{cb(B)}}{2} \tag{19}$$

where $q_{ca(A)}$ is the average cone tip resistance within $3B$ ($B$ is the pile width) above the pile tip and

$q_{cb(B)}$ is the average cone tip resistance within *3B* below the pile tip. Philipponnat recommended the removal of the extreme peaks (spikes) when the tip resistance profiles is irregular and imposed a condition in which $q_{ca(A)}$ # $q_{cb(B)}$.

The unit skin friction of the pile (*f*) is determined by:

$$f = \frac{a_s}{F_s} q_{cs} \qquad (20)$$

where $q_{cs}$ is the average cone tip resistance for each soil layer along the pile shaft, $F_s$ is a factor depends on the soil type as presented in table 8. The factor $''_s$ depends on the pile type where $''_s$ equals to 1.25 for precast concrete driven piles. Philipponnat suggested an upper limit for the skin friction ($f_{lim}$), for precast concrete driven piles $f_{lim}$# 1.2 $P_A$ ($P_A$ is the atmospheric pressure).

**Table 7**

Bearing capacity factor ($k_b$)

| Soil Type | $k_b$ |
|---|---|
| Gravel | 0.35 |
| Sand | 0.40 |
| Silt | 0.45 |
| Clay | 0.50 |

**Table 8**

Empirical factor $F_s$

| Soil Type | $F_s$ |
|---|---|
| Clay and calcareous clay | 50 |
| Silt, sandy clay, and clayey sand | 60 |
| Loose sand | 100 |
| Medium dense sand | 150 |
| Dense sand and gravel | 200 |

**Penpile Method**

The penpile method was proposed by Clisby et al. for the Mississippi Department of Transportation [9]. The unit tip bearing capacity of the pile ($q_t$) is determined from the following relationship:

$$q_t = \begin{cases} 0.25 q_c & \text{for pile tip in clay} \\ 0.125 q_c & \text{for pile tip in sand} \end{cases} \tag{21}$$

where $q_c$ is the average of three cone tip resistances close to the pile tip.

The unit skin friction of the pile shaft ($f$) is obtained from the following relationship:

$$f = \frac{f_s}{1.5 + 0.1 f_s} \tag{22}$$

where $f$ is expressed in psi (lb/in$^2$) and $f_s$ is the sleeve friction of the cone expressed in psi.

**DOTD STATIC METHOD (α-METHOD)**

DOTD uses α-method for the design and analysis of pile foundations. Static analysis is primarily governed by the laboratory test results conducted on the soil samples close to the pile location and by the standard penetration test (SPT) in cohesionless soils. The following soil characteristics and parameters are required to perform static analysis using α-method: (a) soil profile and thickness of each soil layer, (b) the shear strength parameters: cohesion ($C$) and angle of internal friction ($\phi$), and (c) unit weight. The angle of internal friction $\phi$ is obtained from the standard penetration test results or from laboratory tests.

For cohesive soil, the unit tip bearing capacity of the pile is evaluated from the following relationship:

$$q_t = CN_c + \sigma'_v \tag{23}$$

where $C$ is the cohesion of the soil layer, $N_c$ is the bearing capacity factor, and $\sigma_v'$ is the effective vertical stress. Figure 8 presents the variation of $N_c$ with the ratio $R$ ($R$=D/B, depth/pile diameter). The unit skin friction can be predicted by:

$$f = c_a \tag{24}$$

where $c_a$ is the limiting pile/soil adhesion for cohesive soil. The variation of $c_a$ with soil cohesion is shown in figure 9.

For cohesionless soil, the unit tip bearing capacity of the pile is predicted by:

$$q_t = \alpha N_{q'} \sigma'_v \tag{25}$$

where $\alpha$ is an empirical factor depends on the angle of internal friction $\phi$, pile width $B$, and pile depth $D$ (figure 10). $N_{q'}$ is the bearing capacity factor (figure 11), and $\sigma_v'$ is the effective vertical stress. For cohesionless soil, $q_t$ calculated from equation 25 should be less or equal to the maximum unit tip bearing capacity evaluated from figure 12.

The unit skin friction can be predicted by:

$$f = \sigma'_{avg} K \tan \varphi \tag{26}$$

where $\sigma_{avg}'$ is the average effective overburden pressure of the soil layer, $K$ is the coefficient of lateral stress ($K$=1.3 for PPC piles), and $\phi$ is the angle of internal friction. The unit skin friction ($f$) is evaluated from equation 26 should be reduced based on soil type by the friction limit factors presented in table 9.

**Table 9**

Friction limit factors for concrete piles

| **Soil Type** | **Friction Limit for Concrete Piles** |
|---|---|
| Clean sand | 1.00 |
| Silty sand | 0.75 |
| Clean silt | 0.60 |
| Sandy clay, clayey silt | 0.40 |

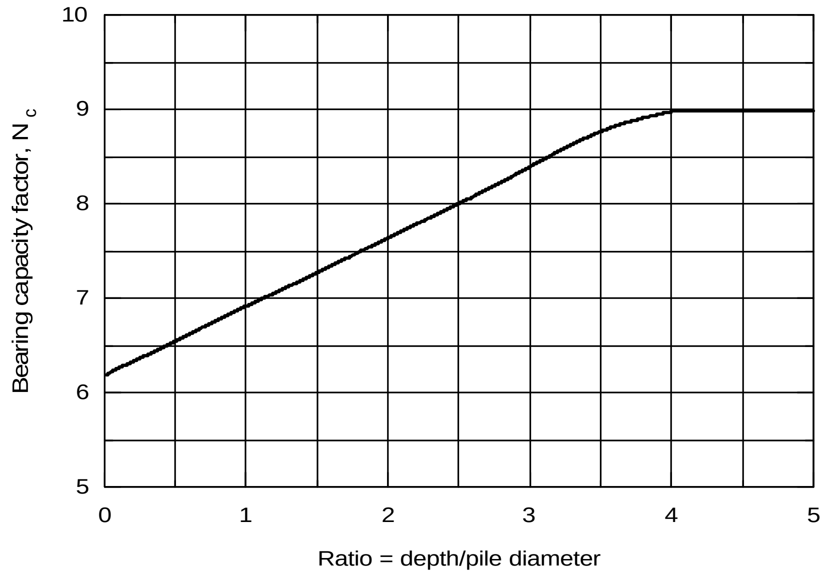



**Figure 8**

Bearing capacity factor $N_c$ for foundations in clay (after Skempton [11])

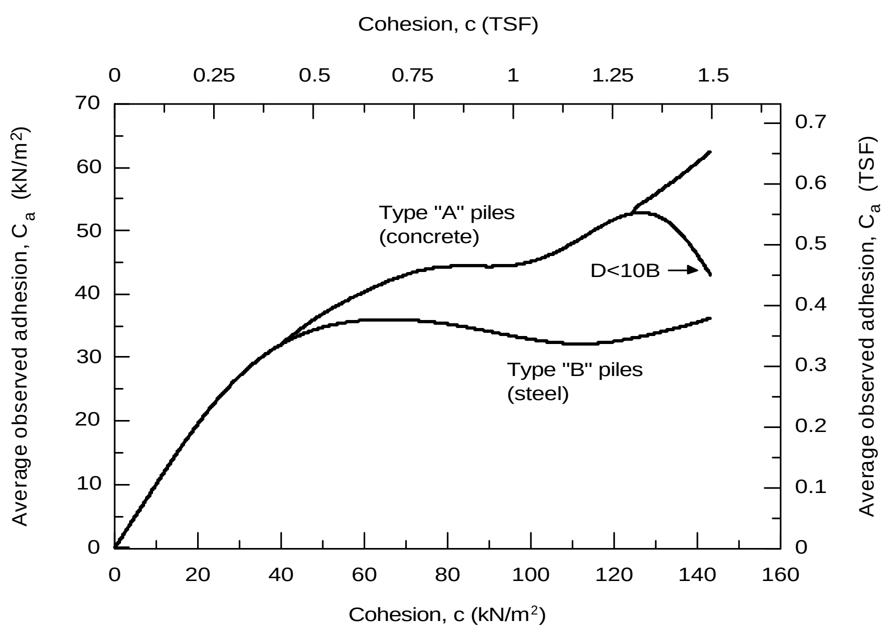



**Figure 9**

Limiting adhesion for piles in soft to stiff clays (after Tomlinson [12])

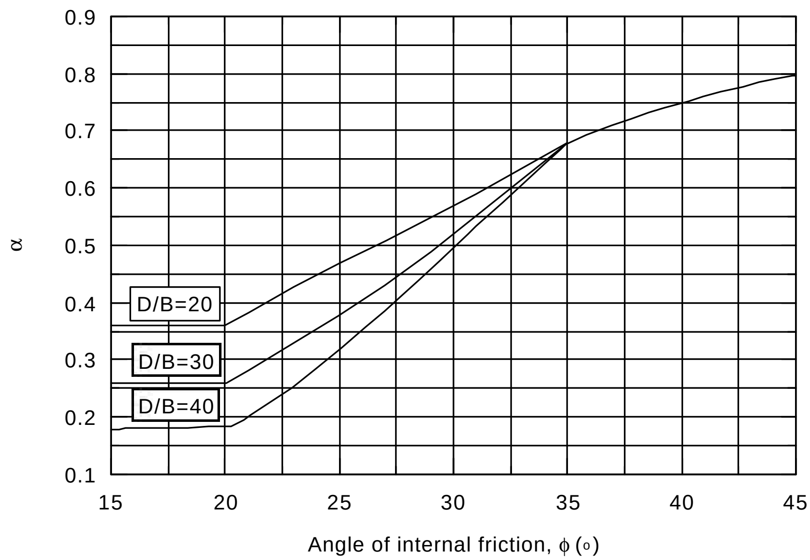



**Figure 10**

Relationship between α-coefficient and angle of internal friction for cohesionless soils (after Bowles [13])

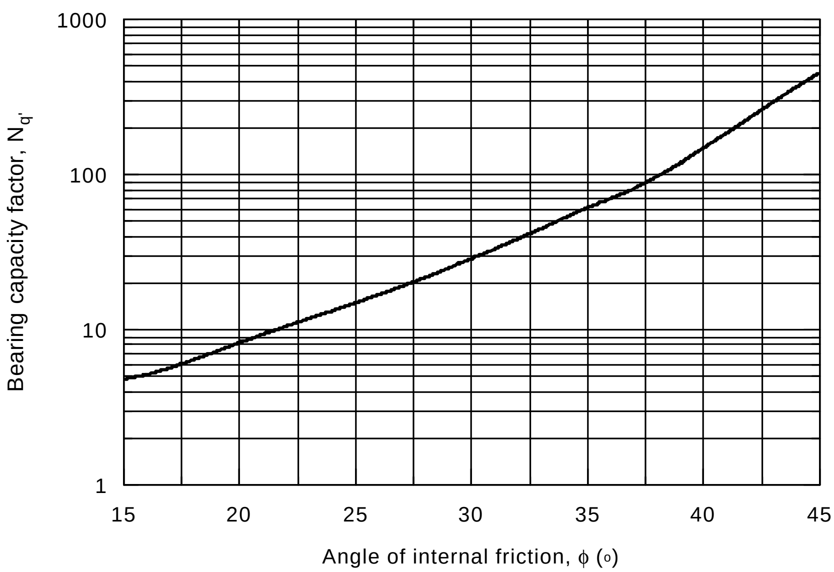



**Figure 11**

Estimating the bearing capacity factor $N_{q'}$ (after Bowles [13])

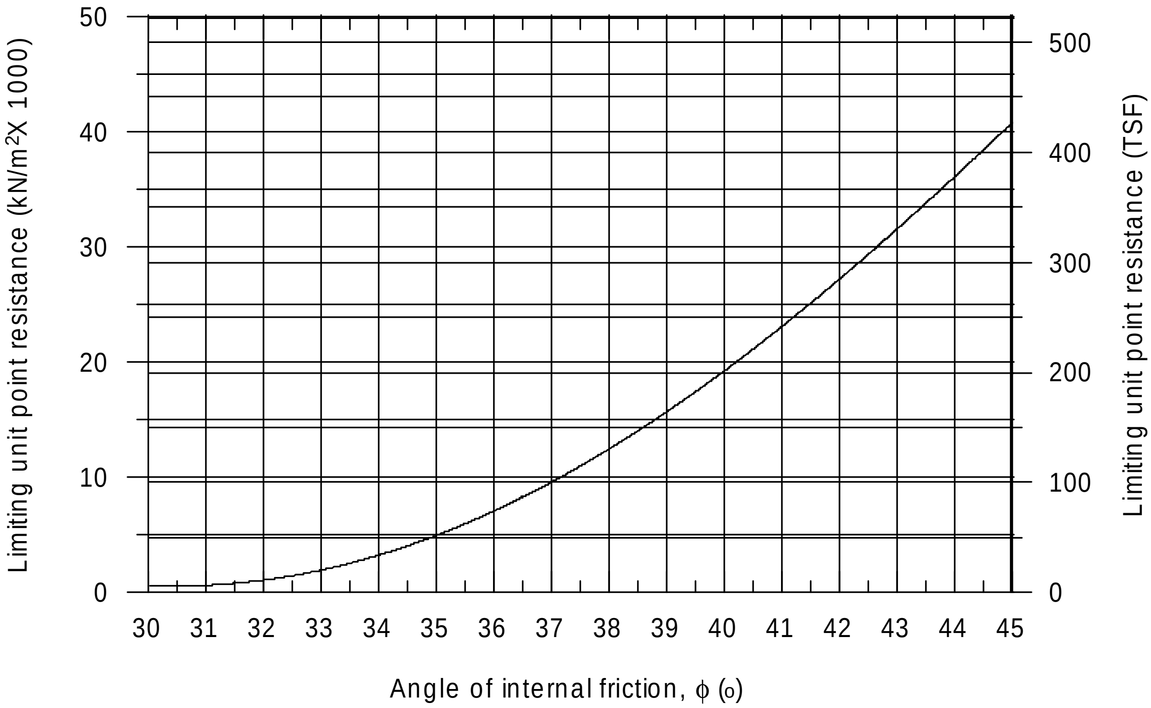
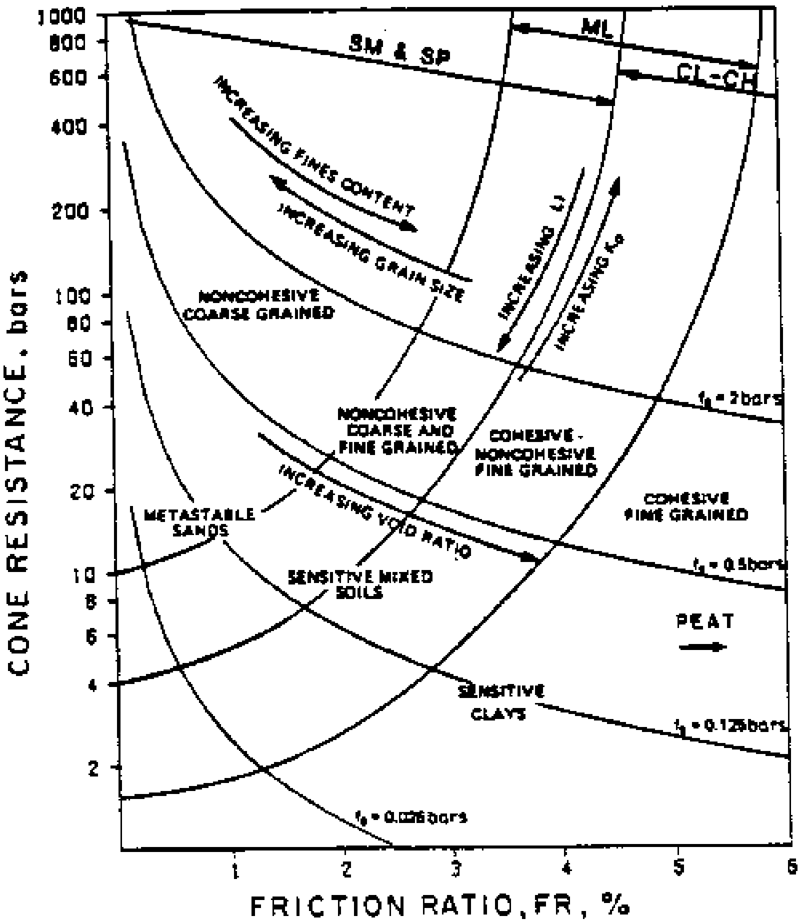



**Figure 12**

Relationship between the maximum unit pile point resistance and friction angle for cohesionless soils

## SOIL CLASSIFICATION BY CPT

Cone penetration test is a popular tool for in situ site characterization. Soil classification and identification of soil stratigraphy can be achieved by analyzing the CPT data. Clayey soils usually show low cone tip resistance, high sleeve friction and therefore high friction ratio, while sandy soils show high cone tip resistance, low sleeve friction, and low friction ratio. Soil classification methods by CPT employ the CPT to identify soil from classification charts. Soil classification charts by Douglas and Olsen and Robertson and Campanella are shown in figures 13 and 14, respectively [*14*], [*15*]. Zhang and Tumay proposed the probabilistic region estimation method for soil classification [*16*]. This method is similar to the classical soil classification methods where it is based on soil composition. The method identifies three soil types: clayey, silty, and sandy soils. The probabilistic region estimation determines the probability of each soil constituent (clay, silt, and sand) at certain depth. Typical soil profile obtained by the probabilistic region estimation is shown in figure 15.




1 bar = 100 kPa ≈ 1 kg/cm²

**Figure 13**
Soil classification chart for standard electric friction cone (after Douglas and Olsen [*14*])

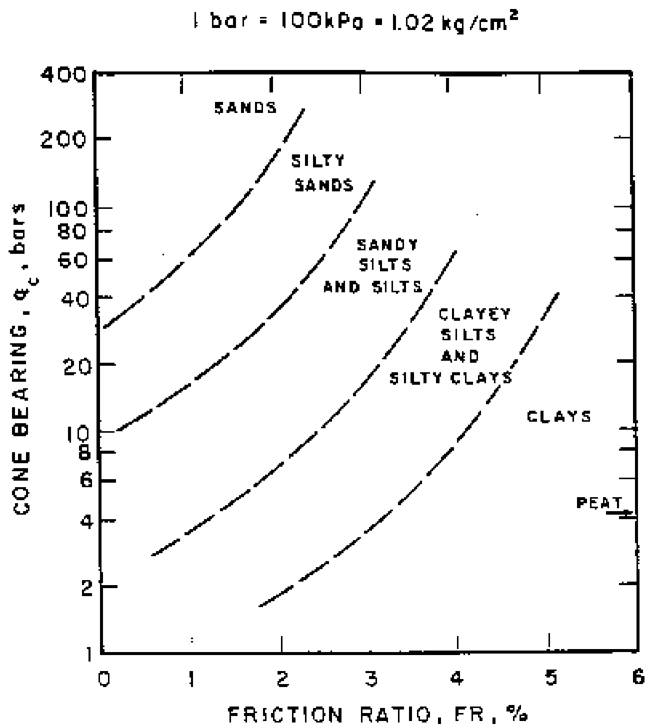



**Figure 14**
Simplified classification chart by Robertson and Campanella for standard electric friction cone [*15*]

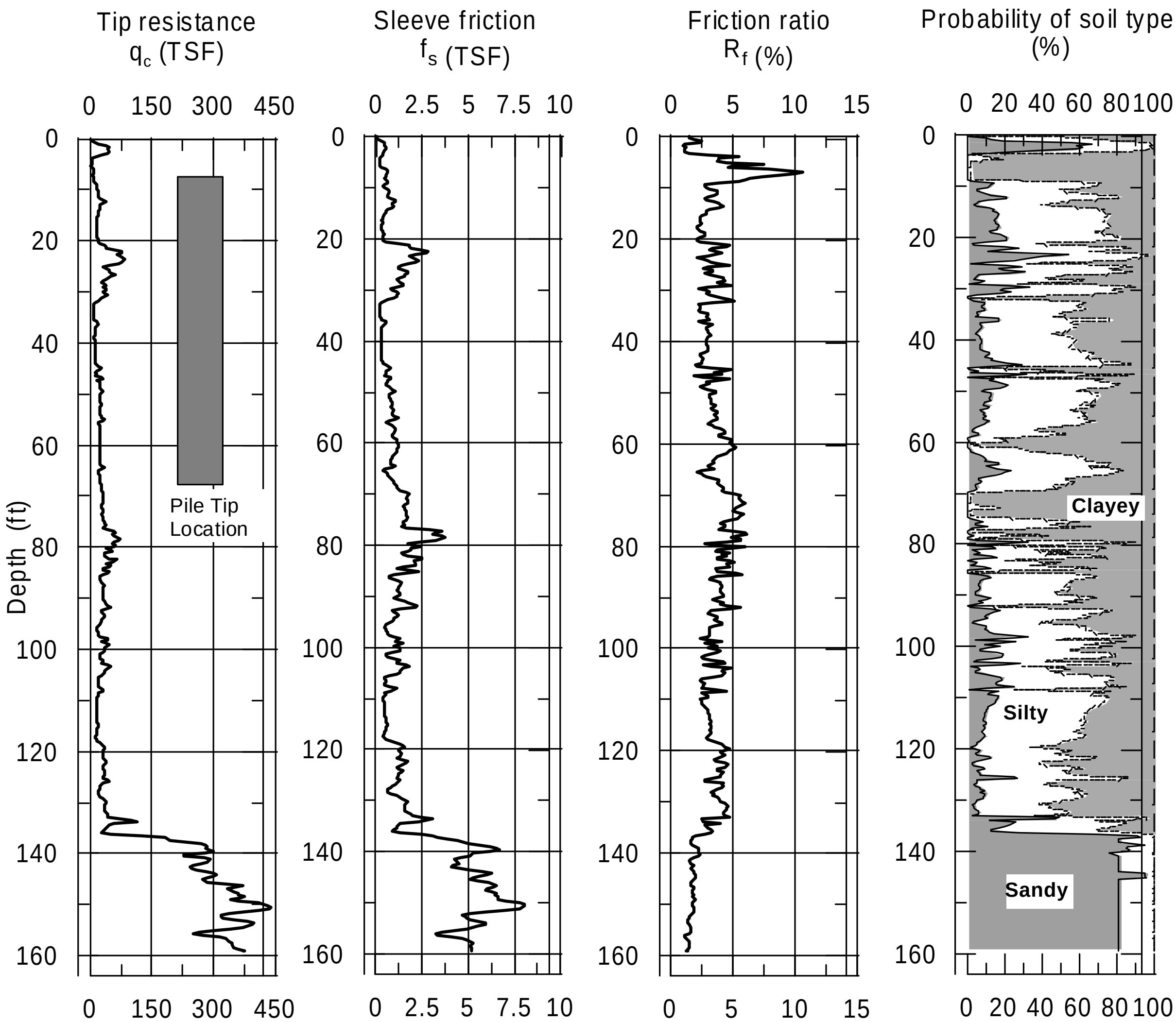



**Figure 15**
Soil classification using the probabilistic region method by Zhang and Tumay [16]

# METHODOLOGY

The primary objective of this study is to evaluate the ability of CPT methods to reliably predict the ultimate load carrying capacity of square PPC piles driven into Louisiana soils. To achieve this goal, 60 pile load tests and the CPT tests conducted close to the piles were identified and collected from DOTD files. Figure 16 depicts a map of the state of Louisiana with approximate locations of the test piles that were considered in this study. 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 and the corresponding CPT profiles.

## COLLECTION AND EVALUATION OF PILE LOAD TEST REPORTS

The practice of DOTD is to carry out pile load tests based on cost/benefit analysis for each project. The purpose of a pile load test is mainly to validate and verify the pile capacity predicted through design/analysis procedures. If, as a result of load testing, a pile fails to provide the required support for design loads, more piles are added to increase the capacity and/or the piles are driven deeper to hard soil layers.

Pile load test reports conducted in Louisiana state projects are available in the general files section at DOTD headquarters in Baton Rouge. These files are kept on microfilms as an integral part of the whole state project. In this study, pile load test reports were obtained from the pavement and geotechnical design section at DOTD.

Pile load test reports obtained from the DOTD were carefully reviewed in order to evaluate their suitability for inclusion in the current research. Special attention was directed to the availability of proper documentation on the pile data (installation and testing), CPT soundings, site location, and subsurface exploration and soil testing. If a pile load test report was found satisfactory, then the report was considered for inclusion in the study.

Pile load test reports provide information and data regarding the site, subsurface exploration, soil identification, testing, and deep foundation load test. Even though other pile types were good candidates for inclusion in the current research, only precast prestressed concrete piles were considered. These piles are the most commonly used in DOTD projects. The characteristics of the square precast prestressed concrete piles used by DOTD are presented in table 10.

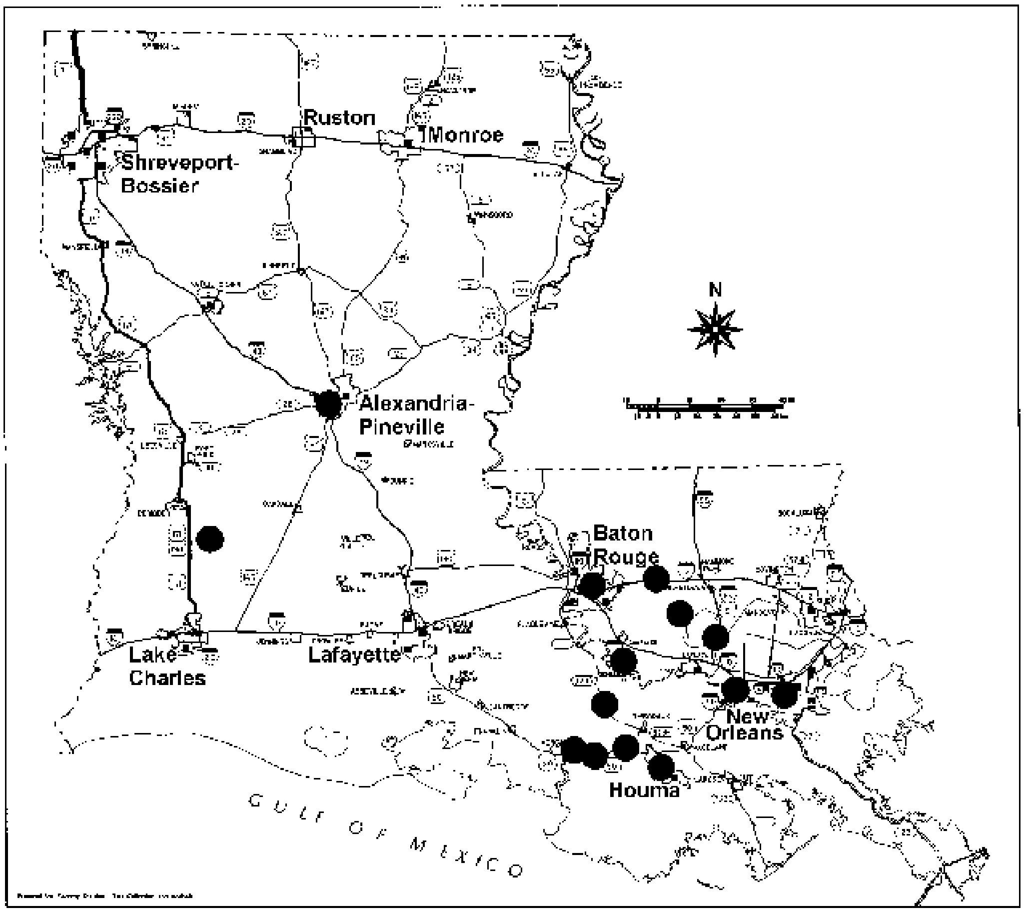



**Figure 16**
Louisiana state map with approximate locations of the analyzed piles

**Table 10**
Properties of square PPC piles used in DOTD projects

| Pile Size (in) | Area of Cross Section ($in^2$) | Void Diameter (in) | Weight per Linear Ft. (lb) |
|---|---|---|---|
| 14" Solid | 196 | None | 204 |
| 16" Solid | 256 | None | 267 |
| 18" Solid | 324 | None | 338 |
| 24" Hollow | 463 | 12" | 482 |
| 30" Hollow | 625 | 18.7" | 651 |

## COMPILATION AND ANALYSIS 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, 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 remaining 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, compiled, and analyzed for each pile load test report.

### Site Data

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 260-05-0020 is the state project number 260-05-0020 (Tickfaw River Bridge and Approaches on State Route LA-22).

### Soil Data

The soil data consist of information on the soil boring location (station number), soil stratigraphy, and laboratory testing (shear strength, physical properties, etc.). For each soil layer, the density, shear strength, and in situ test results (SPT available for cohesionless soil) were identified for input into the DOTD program PILE-SPT and the static analysis program PILELOAD. From soil stratification, the predominant soil type was qualitatively identified (cohesive or cohesionless). The importance of this identification will be addressed in the analysis section.

**Foundation Data**

Foundation data consist of pile characteristics (pile ID, material type, cross-section, total length, 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.).

**CPT Data**

The cone penetration soundings information includes test location (station number), date, cone tip resistance and sleeve friction profiles with depth. In most of the cases, the collected CPT soundings were not available as a digital data, therefore the CPT soundings were scanned and digitized using the UN-SCAN-IT program.

## ANALYSIS OF ULTIMATE CAPACITY OF PILES FROM LOAD TEST

The pile load tests were performed according to the standard loading procedure described in the ASTM D1143-81. The load was applied on the pile head in increments ranging from 10 to 15 percent of the design load and maintained for five minutes. The load was increased up to two to three times the design load or until pile failure. After loading, the piles were unloaded to obtain the shape of the rebound curve. Figure 17 depicts the load -settlement curve for a 30 inch square PPC pile driven at Tickfaw River Bridge.

Figure 17 illustrates how the ultimate pile load carrying capacity ($Q_u$) was interpreted from the load-settlement curve using Butler-Hoy method. This method was selected for the analysis because it is the primary load test interpretation method used by DOTD. In Butler-Hoy method, the ultimate pile load carrying capacity is determined using the following procedure:

(a) draw a line tangent to the initial portion (elastic compression of the pile) of the load-settlement curve,
(b) draw a line tangent to the plunging portion of the load-settlement curve with a slope of 1 inch/20 ton,

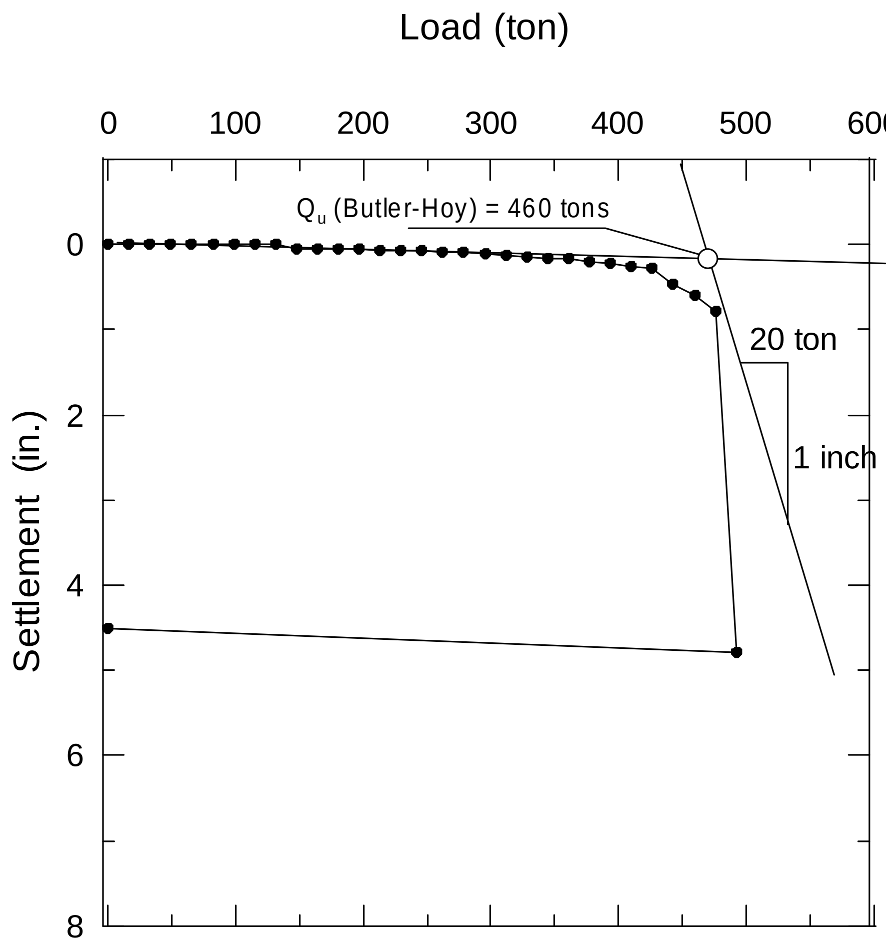



**Figure 17**
Load-settlement curve for 30 in square PPC pile (TP1) at Tickfaw River bridge

(c) locate the intersection of the two lines, which represents the ultimate capacity of the pile (as shown in figure 17).

## INTERPRETATION OF SOIL PROFILE FROM CPT

In this report, the probabilistic region estimation method was used for soil classification and identification of site stratigraphy [*16*]. This method was selected since it provides a profile of the probability of soil constituents with depth, while other methods show a sudden jump from soil layer to another. Moreover, this method is simple and provides output that can be easily understood. Figure 15 shows the soil profile next to pile TP1 at Tickfaw River Bridge as determined by the probabilistic region estimation method.

## ANALYSIS OF PILES USING THE CPT METHODS

The eight CPT methods used to estimate the ultimate load carrying capacity of the selected piles were discussed in the Background section. Prediction of the pile capacity using these methods is difficult to perform manually. A computer code was written in FORTRAN in which these methods were implemented. The program performs the analysis on the CPT sounding and then uses the eight CPT methods to predict the pile capacity.

The core of the FORTRAN program was then converted to a Visual Basic code in order to develop MS-Windows based application. The program called Louisiana Pile Design by CPT (LPD-CPT). Appendix A presents an illustration of using the program LPD-CPT to predict the pile capacity by different CPT methods.

## ANALYSIS OF PILE CAPACITY USING STATIC METHODS

The DOTD static analysis method ("-method) was applied to evaluate the load carrying capacity of the investigated piles. This is to provide a platform to compare the CPT methods with the method used by DOTD.

# ANALYSIS OF RESULTS

This section presents an evaluation of the ability of the investigated CPT methods to predict the ultimate load carrying capacity of square PPC piles driven into Louisiana soils. The performance of the different methods was evaluated based on criteria that include statistical analyses and evaluation of the predicted/measured pile capacity. Each method was given a rank based on its performance according to the selected criterion. The final rank of each method was obtained by adding the ranks of the all criteria. The analyses were conducted only on friction PPC piles that failed (plunged or showed large settlement) under load testing.

## CHARACTERIZATION OF THE INVESTIGATED PILES

Sixty square precast prestressed concrete piles were considered in the current study. However, the analyses were conducted on 35 friction piles that were failed during the pile load test. A summary of the characteristics of the investigated pile is presented in table 11. The piles are categorized based on the predominant soil type, load resistance mechanism, and failure under loading.

**Table 11**

Number of PPC piles investigated based on pile type, soil type, and load test

| Square PPC Pile Size | Pile Type | | Predominant Soil Type | | | Pile Failed Under Loading | |
|---|---|---|---|---|---|---|---|
| | Friction | End-bearing | Cohesive | Cohesionless | Both | Yes | No |
| 14" | 26 | 1 | 21 | 3 | 3 | 20 | 7 |
| 16" | 5 | 1 | 5 | 0 | 1 | 4 | 2 |
| 18" | 2 | 0 | 1 | 0 | 1 | 2 | 0 |
| 24" | 6 | 1 | 5 | 2 | 0 | 2 | 5 |
| 30" | 9 | 9 | 5 | 0 | 13 | 12 | 6 |
| Total | 48 | 12 | 37 | 5 | 18 | 40 | 20 |

## PREDICTED VERSUS MEASURED PILE CAPACITY

Among the sixty piles considered in this study, only thirty-five piles were used in the analyses. These piles were identified as friction piles loaded to failure. For each pile load test, the in situ CPT sounding adjacent to the pile was used to predict the ultimate axial load capacity of that pile using the CPT prediction methods. These methods are the Schmertmann, de Ruiter and Beringen, Bustamante and Gianeselli, Tumay and Fakhroo, Aoki and De Alencar, Price and Wardle, Philipponnat, and penpile method [2], [3], [4], [5], [6], [7], [8], [9]. The ultimate load capacity for each pile was also predicted from the soil properties (soil boring close to the pile) using the static "-method. The ultimate load carrying capacity predicted by these methods ($Q_p$) is compared to the measured pile capacity ($Q_m$) obtained from the pile load test using Butler-Hoy interpretation method [1]. Table 12 summarizes the results of the analyses conducted on the investigated piles. Among the data presented in table 12 are: the pile size, type, length, location of the load test, the measured ultimate load carrying capacity, and the predicted ultimate load carrying capacity.

The predicted ultimate load carrying capacity ($Q_p$) consists of pile tip capacity ($Q_t$) and pile shaft resistance ($Q_s$). The pile capacities $Q_t$, $Q_s$, and $Q_p$ predicted by the CPT methods and "-method are compared with $Q_m$ in figures 18-26. The results of 35 friction piles and five end-bearing piles are shown in these figures. Inspection of these figures shows that the ratio $Q_t/Q_p$ for the first 35 piles is relatively small, which is consistent with the previous classification of these piles as friction piles (pile capacity is derived mainly from the shaft resistance). On the other hand, the ratio $Q_t/Q_p$ for piles number 36 to 40 is relatively large and hence these piles are considered as end bearing piles (pile capacity is derived mainly from the pile tip capacity).

Figures 27-35 show the predicted ($Q_p$) versus the measured ($Q_m$) ultimate load carrying capacity for the friction and end-bearing piles. For each prediction method, regression analysis was conducted to obtain the best fit line for the predicted/measured pile capacities. The relationship $Q_{fit}/Q_m$ and the corresponding coefficient of determination ($R^2$) were determined for each CPT prediction method and for the static "-method.

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils

| | State Project Identification | 260-05-0020 Tickfaw River Bridge and Approaches on LA-22 | | | 260-05-0020 Tickfaw River Bridge and Approaches on LA-22 | | | 260-05-0020 Tickfaw River Bridge and Approaches on LA-22 | | | 424-05-0081 Bayou Boeuf Bridge West Approach | | | 424-05-0081 Bayou Boeuf Bridge West Approach | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 30" Square PPC | | | TP2, 30" Square PPC | | | TP3, 30" Square PPC | | | TP1, 14" Square PPC | | | TP2, 30" Square PPC | | |
| | Pile Length (ft) | 73 | | | 118 | | | 81 | | | 96 | | | 115 | | |
| | Embedded Length (ft) | 59.3 | | | 61 | | | 71 | | | 89.5 | | | 110 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive and Cohesionless | | | Cohesive | | | Cohesive | | | Cohesive and Cohesionless | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 263.6 | 115.5 | **379.1** | 540.7 | 108.0 | **648.7** | 357.4 | 251.5 | **608.9** | 128.0 | 11.1 | **139.1** | 409.3 | 125.5 | **534.8** |
| | de Ruiter & Beringen | 341.0 | 52.0 | **393.0** | 460.4 | 48.6 | **509.0** | 437.3 | 113.2 | **550.5** | 88.1 | 5.0 | **93.1** | 359.1 | 75.9 | **435.0** |
| | LCPC | 262.9 | 87.6 | **350.5** | 552.5 | 71.2 | **623.7** | 415.5 | 163.3 | **578.8** | 109.5 | 9.6 | **119.1** | 351.7 | 161.6 | **513.3** |
| | Tumay & Fakhroo | 234.4 | 120.4 | **354.8** | 403.0 | 133.3 | **536.3** | 380.5 | 255.3 | **635.8** | 140.1 | 12.2 | **152.3** | 363.9 | 152.2 | **516.1** |
| | Aoki & De Alencar | 233.8 | 88.5 | **322.3** | 485.4 | 92.4 | **577.8** | 348.1 | 165.0 | **513.1** | 59.9 | 12.3 | **72.2** | 242.6 | 124.2 | **366.8** |
| | Price & Wardle | 230.1 | 53.3 | **283.4** | 538.1 | 54.1 | **592.2** | 381.5 | 96.2 | **477.7** | 77.7 | 8.1 | **85.8** | 288.9 | 76.0 | **364.9** |
| | Philipponnat | 353.4 | 70.9 | **424.3** | 625.1 | 71.3 | **696.4** | 520.1 | 125.4 | **645.5** | 96.4 | 6.5 | **102.9** | 386.3 | 84.4 | **470.7** |
| | Penpile | 164.0 | 37.7 | **201.7** | 263.2 | 34.8 | **298.0** | 199.1 | 68.4 | **267.5** | 73.4 | 5.5 | **78.9** | 244.4 | 51.8 | **296.2** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **460.0** | - | - | **525.0** | - | - | **458.0** | - | - | **112.0** | - | - | **318.0** |
| Static Analysis | "-method (used by DOTD) | 302.1 | 46.4 | **348.5** | 367.1 | 38.5 | **405.6** | 215.1 | 30.1 | **245.2** | 68.3 | 9.3 | **77.6** | 319.3 | 105.8 | **425.1** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing); $Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification ' | 424-05-0081 Bayou Boeuf Bridge West Approach | | | 424-05-0081 Bayou Boeuf Bridge West Approach | | | 065-90-0024 Houma Intracoastal Waterway Bridge | | | 065-90-0024 Houma Intracoastal Waterway Bridge | | | 065-90-0024 Houma Intracoastal Waterway Bridge | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP3, 14" Square PPC | | | TP4, 16" Square PPC | | | TP1, 14" Square PPC | | | TP2, 14" Square PPC | | | TP3, 14" Square PPC | | |
| | Pile Length (ft) | 74 | | | 80 | | | 85 | | | 73.5 | | | 94.5 | | |
| | Embedded Length (ft) | 63.5 | | | 70 | | | 80 | | | 70 | | | 80 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive and Cohesionless | | | Cohesive | | | Cohesive and Cohesionless | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 106.3 | 24.7 | **131.0** | 120.9 | 18.3 | **139.2** | 162.8 | 14.4 | **177.2** | 98.7 | 3.5 | **102.2** | 126.8 | 12.3 | **139.1** |
| | de Ruiter & Beringen | 95.7 | 19.3 | **115.0** | 82.1 | 8.2 | **90.3** | 139.4 | 6.5 | **145.9** | 91.3 | 1.6 | **92.9** | 101.9 | 5.6 | **107.5** |
| | LCPC | 111.4 | 28.1 | **139.5** | 102.2 | 12.5 | **114.7** | 159.0 | 9.5 | **168.5** | 87.5 | 3.1 | **90.6** | 124.4 | 7.9 | **132.3** |
| | Tumay & Fakhroo | 118.0 | 27.9 | **145.9** | 120.2 | 18.4 | **138.6** | 151.6 | 14.7 | **166.3** | 114.3 | 3.8 | **118.1** | 152.9 | 12.9 | **165.8** |
| | Aoki & De Alencar | 74.2 | 46.1 | **120.3** | 56.1 | 11.3 | **67.4** | 115.0 | 9.4 | **124.4** | 63.7 | 3.3 | **67.0** | 80.7 | 7.9 | **88.6** |
| | Price &Wardle | 68.1 | 23.1 | **91.2** | 74.3 | 7.0 | **81.3** | 111.0 | 6.0 | **117.0** | 65.1 | 2.1 | **67.2** | 81.1 | 4.6 | **85.7** |
| | Philipponnat | 129.8 | 23.1 | **152.9** | 85.9 | 9.5 | **95.4** | 199.5 | 7.5 | **207.0** | 101.8 | 2.5 | **104.3** | 135.1 | 6.3 | **141.4** |
| | Penpile | 55.9 | 11.3 | **67.2** | 69.3 | 5.1 | **74.4** | 84.7 | 4.3 | **89.0** | 58.2 | 1.6 | **59.8** | 66.5 | 3.3 | **69.8** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **162.0** | - | - | **98.0** | - | - | **112.0** | - | - | **49.0** | - | - | **118.0** |
| Static Analysis | "-method (used by DOTD) | 83.1 | 12.3 | **95.4** | 62.1 | 4.6 | **66.7** | 144.8 | 6.2 | **151.0** | 105.2 | 5.9 | **111.1** | 145.5 | 5.5 | **151.0** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | State Project Identification | 855-04-0046 Houma Intra Coastal Waterway Bridge | | | 855-04-0046 Houma Intra Coastal Waterway Bridge | | | 855-04-0046 Houma Intra Coastal Waterway Bridge | | | 450-33-56 Baton Rouge | | | 424-05-0078 Bayou Boeuf Bridge Main Span | | |
| Pile and Soil Identification | Pile ID | TP4, 14" Square PPC | | | TP5, 16" Square PPC | | | TP6, 16" Square PPC | | | TP2A, 14" Square PPC | | | TP1, 14" Square PPC | | |
| | Pile Length (ft) | 92 | | | 72 | | | 110 | | | NA | | | 80 | | |
| | Embedded Length (ft) | 81 | | | 71.5 | | | 98 | | | 45 | | | 70 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | No | | | No | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 112.3 | 12.0 | **124.3** | 107.2 | 11.3 | **118.5** | 126.8 | 20.7 | **147.5** | 87.1 | 24.5 | **111.6** | 87.1 | 23.8 | **110.9** |
| | de Ruiter & Beringen | 106.6 | 5.4 | **112.0** | 62.8 | 5.1 | **67.9** | 118.1 | 9.3 | **127.4** | 96.0 | 11.0 | **107.0** | 93.8 | 14.3 | **108.1** |
| | LCPC | 127.5 | 7.6 | **135.1** | 90.4 | 7.7 | **98.1** | 140.6 | 13.4 | **154.0** | 71.4 | 15.9 | **87.3** | 93.6 | 23.2 | **116.8** |
| | Tumay & Fakhroo | 158.9 | 12.3 | **171.2** | 136.3 | 12.3 | **148.6** | 200.8 | 24.0 | **224.8** | 74.4 | 26.2 | **100.6** | 122.9 | 25.9 | **148.8** |
| | Aoki & De Alencar | 74.3 | 7.4 | **81.7** | 43.0 | 8.3 | **51.3** | 81.0 | 14.9 | **95.9** | 65.8 | 16.8 | **82.6** | 61.9 | 23.9 | **85.8** |
| | Price & Wardle | 71.7 | 4.5 | **76.2** | 62.4 | 4.3 | **66.7** | 70.5 | 8.8 | **79.3** | 75.9 | 9.9 | **85.8** | 51.1 | 14.3 | **65.4** |
| | Philipponnat | 134.3 | 6.1 | **140.4** | 65.0 | 6.1 | **71.1** | 122.4 | 11.5 | **133.9** | 99.5 | 13.4 | **112.9** | 114.6 | 18.5 | **133.1** |
| | Penpile | 61.1 | 3.2 | **64.3** | 60.6 | 3.1 | **63.7** | 71.0 | 5.6 | **76.6** | 57.4 | 6.6 | **64.0** | 49.6 | 8.2 | **57.8** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **116.0** | - | - | **106.0** | - | - | **168.0** | - | - | **146.0** | - | - | **161.0** |
| Static Analysis | "-method (used by DOTD) | 130.9 | 5.5 | **136.4** | 114.9 | 7.6 | **122.5** | 169.8 | 9.6 | **179.4** | 108.0 | 25.1 | **133.1** | 136.3 | 3.8 | **140.1** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | State Project Identification | 424-05-0078 Bayou Boeuf Bridge Main Span | | | 424-05-0078 Bayou Boeuf Bridge Main Span | | | 424-05-0087 Bayou Ramos Bridge | | | 424-05-0087 Bayou Ramos Bridge | | | 424-05-0087 Bayou Ramos Bridge | | |
| Pile and Soil Identification | Pile ID | TP2, 14" Square PPC | | | TP5, 14" Square PPC | | | TP1, 16" Square PPC | | | TP2, 30" Square PPC | | | TP3, 30" Square PPC | | |
| | Pile Length (ft) | 80 | | | 85 | | | 85 | | | 115 | | | 115 | | |
| | Embedded Length (ft) | 70 | | | 80 | | | 78 | | | 88 | | | 104 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | **$Q_u$** | $Q_s$ | $Q_t$ | **$Q_u$** | $Q_s$ | $Q_t$ | **$Q_u$** | $Q_s$ | $Q_t$ | **$Q_u$** | $Q_s$ | $Q_t$ | **$Q_u$** |
| | Schmertmann | 88.2 | 22.2 | **110.4** | 60.2 | 0.9 | **61.1** | 118.3 | 67.3 | **185.6** | 409.3 | 169.6 | **578.9** | 410.6 | 200.8 | **611.4** |
| | de Ruiter & Beringen | 96.6 | 14.0 | **110.6** | 65.4 | 0.4 | **65.8** | 110.1 | 51.2 | **161.3** | 382.8 | 132.4 | **515.2** | 719.9 | 839.6 | **609.8** |
| | LCPC | 93.8 | 23.8 | **117.6** | 90.1 | 0.7 | **90.8** | 133.0 | 41.9 | **174.9** | 396.8 | 179.5 | **576.3** | 447.3 | 183.4 | **630.7** |
| | Tumay & Fakhroo | 122.0 | 38.5 | **160.5** | 163.3 | 1.2 | **164.5** | 153.5 | 83.3 | **236.8** | 356.5 | 169.7 | **526.2** | 375.6 | 212.9 | **588.5** |
| | Aoki & De Alencar | 63.0 | 29.0 | **92.0** | 44.7 | 1.8 | **46.5** | 72.6 | 54.5 | **127.1** | 286.8 | 174.7 | **461.5** | 310.2 | 183.1 | **493.3** |
| | Price & Wardle | 52.0 | 14.7 | **66.7** | 30.3 | 0.5 | **30.8** | 69.3 | 32.6 | **101.9** | 277.6 | 108.1 | **385.7** | 281.6 | 111.7 | **393.3** |
| | Philipponnat | 119.9 | 19.1 | **139.0** | 70.9 | 0.7 | **71.6** | 125.6 | 39.5 | **165.1** | 531.0 | 132.2 | **663.2** | 648.2 | 136.1 | **784.3** |
| | Penpile | 50.3 | 10.6 | **60.9** | 33.0 | 0.3 | **33.3** | 66.6 | 15.6 | **82.2** | 210.6 | 58.3 | **268.9** | 231.4 | 62.1 | **293.5** |
| Load Test Interpretatio n Method | Butler-Hoy (used by DOTD) | **-** | **-** | **108.0** | **-** | **-** | **117.0** | **-** | **-** | **132.0** | **-** | **-** | **488.0** | **-** | **-** | **466.0** |
| Static Analysis | "-method (used by DOTD) | 130.4 | 9.3 | **139.7** | 132.2 | 9.3 | **141.5** | 114.3 | 2.9 | **117.2** | 301.6 | 42.6 | **344.2** | 380.9 | 42.6 | **423.5** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | State Project Identification | 424-05-0087 Bayou Ramos Bridge | | | 424-05-0087 Bayou Ramos Bridge | | | 424-05-0087 Bayou Ramos Bridge | | | 424-05-0087 Bayou Ramos Bridge | | | 424-06-0005 Bayou Boeuf Bridge East Approach | | |
| Pile and Soil Identification | Pile ID | TP4, 30" Square PPC | | | TP5, 30" Square PPC | | | TP6, 30" Square PPC | | | TP7, 16" Square PPC | | | TP1, 14" Square PPC | | |
| | Pile Length (ft) | 115 | | | 115 | | | 119 | | | 85 | | | 70 | | |
| | Embedded Length (ft) | 99.3 | | | 113 | | | 112 | | | 77 | | | 68 | | |
| | Pile Classification | End-bearing | | | End-bearing | | | End-bearing | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | No | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 394.0 | 289.9 | **683.9** | 384.7 | 460.1 | **844.8** | 384.4 | 287.4 | **671.8** | 127.5 | 14.0 | **141.5** | 84.9 | 16.2 | **101.1** |
| | de Ruiter & Beringen | 407.3 | 263.4 | **643.7** | 554.5 | 330.8 | **885.3** | 359.8 | 207.2 | **567.0** | 103.4 | 6.3 | **109.7** | 69.5 | 7.3 | **76.8** |
| | LCPC | 459.5 | 215.7 | **675.2** | 587.0 | 439.1 | **1026.1** | 394.6 | 317.4 | **712.0** | 123.8 | 9.9 | **133.7** | 87.7 | 10.5 | **98.2** |
| | Tumay & Fakhroo | 419.0 | 373.7 | **792.7** | 380.5 | 642.0 | **1022.5** | 431.8 | 524.9 | **956.7** | 140.8 | 14.1 | **154.9** | 117.6 | 16.5 | **134.1** |
| | Aoki & De Alencar | 306.6 | 384.7 | **691.3** | 380.3 | 598.6 | **978.9** | 270.5 | 457.4 | **727.9** | 68.1 | 12.2 | **80.3** | 47.6 | 9.9 | **57.5** |
| | Price & Wardle | 279.4 | 203.9 | **483.3** | 259.5 | 384.0 | **643.5** | 244.6 | 282.3 | **526.9** | 77.9 | 8.0 | **85.9** | 48.8 | 6.1 | **54.9** |
| | Philipponnat | 605.8 | 246.0 | **851.8** | 574.7 | 450.5 | **1025.2** | 435.0 | 335.1 | **770.1** | 118.0 | 7.9 | **125.9** | 72.0 | 8.2 | **80.2** |
| | Penpile | 213.0 | 76.0 | **289.0** | 228.5 | 158.0 | **386.5** | 205.6 | 121.0 | **326.6** | 72.7 | 4.7 | **77.4** | 47.8 | 4.4 | **52.2** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **552** | - | - | **538.0** | - | - | **447.0** | - | - | **108.0** | - | - | **97.0** |
| Static Analysis | "-method (used by DOTD) | 354.8 | 42.6 | **397.4** | 331.9 | 32.3 | **364.2** | 318.9 | 42.6 | **361.5** | 100.0 | 7.0 | **107.0** | 93.9 | 4.7 | **98.6** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 424-06-0005 Bayou Boeuf Bridge East Approach | | | 424-06-0005 Bayou Boeuf Bridge East Approach | | | 424-06-0005 Bayou Boeuf Bridge East Approach | | | 424-06-0005 Bayou Boeuf Bridge East Approach | | | 424-06-0005 Bayou Boeuf Bridge East Approach | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP2, 14" Square PPC | | | TP3, 14" Square PPC | | | TP4, 14" Square PPC | | | TP5, 14" Square PPC | | | TP6, 30" Square PPC | | |
| | Pile Length (ft) | 75 | | | 85 | | | 85 | | | 85 | | | 115 | | |
| | Embedded Length (ft) | 71 | | | 77.5 | | | 79 | | | 79 | | | 110 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | Yes | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 90.5 | 12.8 | **103.3** | 94.3 | 10.9 | **105.2** | 104.8 | 20.1 | **124.9** | 92.7 | 17.7 | **110.4** | 316.5 | 94.8 | **411.3** |
| | de Ruiter & Beringen | 65.6 | 5.8 | **71.4** | 85.7 | 4.9 | **90.6** | 122.3 | 9.0 | **131.3** | 95.1 | 7.9 | **103.0** | 318.4 | 42.6 | **361.0** |
| | LCPC | 86.8 | 8.5 | **95.3** | 103.1 | 8.7 | **111.8** | 121.5 | 12.8 | **134.3** | 110.4 | 12.3 | **122.7** | 340.5 | 60.5 | **401.0** |
| | Tumay & Fakhroo | 119.6 | 13.0 | **132.6** | 134.6 | 13.2 | **147.8** | 128.3 | 20.2 | **148.5** | 140.7 | 19.2 | **159.9** | 392.8 | 96.7 | **489.5** |
| | Aoki & De Alencar | 45.0 | 8.0 | **53.0** | 58.8 | 9.1 | **67.9** | 83.9 | 11.9 | **95.8** | 65.2 | 12.7 | **77.9** | 218.3 | 60.2 | **278.5** |
| | Price & Wardle | 51.9 | 4.9 | **56.8** | 53.2 | 5.6 | **58.8** | 60.8 | 7.3 | **68.1** | 51.0 | 7.3 | **58.3** | 189.7 | 37.2 | **226.9** |
| | Philipponnat | 68.0 | 6.7 | **74.7** | 88.9 | 8.0 | **96.9** | 126.7 | 9.4 | **136.1** | 98.5 | 9.5 | **108.0** | 329.9 | 47.6 | **377.5** |
| | Penpile | 51.0 | 3.6 | **54.6** | 52.9 | 3.5 | **56.4** | 59.4 | 5.3 | **64.7** | 51.6 | 5.1 | **56.7** | 180.7 | 27.5 | **208.2** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **94.0** | - | - | **97.0** | - | - | **117.0** | - | - | **87.0** | - | - | **316.0** |
| Static Analysis | "-method (used by DOTD) | 99.6 | 6.9 | **106.5** | 63.3 | 1.9 | **65.2** | 71.3 | 5.0 | **76.3** | 77.3 | 2.6 | **79.9** | 210.4 | 14.1 | **224.5** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$ +$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 262-06-09 Tickfaw River Bridge # 1 | | | 262-06-09 Tickfaw River Bridge # 1 | | | 424-07-09 Gibson-Chacahoula State Route LA 3052 | | | 424-07-09 Gibson-Chacahoula State Route LA 3052 | | | 424-07-09 Gibson-Chacahoula State Route LA 3052 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 24" Square PPC | | | TP2, 24" Square PPC | | | TP1, 30" Square PPC | | | TP3, 30" Square PPC | | | TP4, 30" Square PPC | | |
| | Pile Length (ft) | 85 | | | 105 | | | 120 | | | 125 | | | 125 | | |
| | Embedded Length (ft) | 84.9 | | | 105 | | | 116 | | | 122 | | | 124 | | |
| | Pile Classification | Friction | | | Friction | | | End-bearing | | | End-bearing | | | End-bearing | | |
| | Pile Tested to Failure | No | | | No | | | Yes | | | No | | | Yes | | |
| | Predominant Soil * | Cohesionless | | | Cohesionless | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 271.2 | 121.7 | **392.9** | 402.4 | 340.8 | **743.2** | 341.7 | 503.0 | **844.7** | 337.0 | 661.2 | **998.2** | 336.4 | 331.4 | **667.8** |
| | de Ruiter & Beringen | 323.8 | 99.4 | **423.2** | 431.0 | 308.1 | **739.1** | 330.8 | 366.3 | **697.1** | 543.5 | 476.1 | **1019.6** | 358.3 | 208.8 | **567.1** |
| | LCPC | 332.2 | 93.0 | **425.2** | 412.3 | 156.3 | **568.6** | 376.7 | 434.7 | **811.4** | 383.5 | 566.4 | **949.9** | 391.9 | 213.9 | **605.8** |
| | Tumay & Fakhroo | 229.5 | 121.7 | **351.2** | 288.4 | 346.9 | **635.3** | 474.3 | 487.5 | **961.8** | 448.3 | 661.9 | **1110.2** | 479.6 | 465.4 | **945.0** |
| | Aoki & De Alencar | 294.3 | 124.9 | **419.2** | 394.5 | 219.8 | **614.3** | 242.0 | 442.5 | **684.5** | 370.7 | 612.3 | **983.0** | 234.8 | 288.5 | **523.3** |
| | Price & Wardle | 166.8 | 83.1 | **249.9** | 263.3 | 132.8 | **396.1** | 199.0 | 302.5 | **501.5** | 198.0 | 404.2 | **602.2** | 194.5 | 160.0 | **354.5** |
| | Philipponnat | 519.4 | 98.6 | **618.0** | 687.3 | 150.1 | **837.4** | 405.0 | 356.4 | **761.4** | 576.4 | 477.2 | **1053.6** | 416.0 | 199.7 | **615.7** |
| | Penpile | 143.9 | 34.0 | **177.9** | 211.1 | 56.4 | **267.5** | 184.0 | 142.5 | **326.5** | 189.1 | 179.1 | **368.2** | 180.3 | 90.3 | **270.6** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **230.0** | - | - | **258.0** | - | - | **620.0** | - | - | **670.0** | - | - | **610.0** |
| Static Analysis | "-method (used by DOTD) | 416.8 | 27.3 | **444.1** | 557.3 | 84.9 | **642.2** | 392.4 | 42.6 | **435.0** | 352.6 | 627.4 | **980.0** | 366.5 | 627.4 | **993.9** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)

$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 424-07-08 Gibson to Raceland Highway | | | 424-07-08 Gibson to Raceland Highway | | | 424-07-08 Gibson to Raceland Highway | | | 450-36-02 Luling Bridge (North Approach)- US 61 | | | 858-01-0008 Rosepine-Beauregard Parish Line Bridges | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 30" Square PPC | | | TP2, 24" Square PPC | | | TP3, 30" Square PPC | | | TP8, 30" Square PPC | | | TP1, 14" Square PPC | | |
| | Pile Length (ft) | 137 | | | 129 | | | 137 | | | 120 | | | 80 | | |
| | Embedded Length (ft) | 132.7 | | | 115 | | | 127.4 | | | 112 | | | 80 | | |
| | Pile Classification | End-bearing | | | End-bearing | | | End-bearing | | | End-bearing | | | Friction | | |
| | Pile Tested to Failure | No | | | No | | | No | | | Yes | | | No | | |
| | Predominant Soil * | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesive and Cohesionless | | | Cohesionless | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 448.2 | 429.5 | **877.7** | 255.8 | 135.0 | **390.8** | 511.9 | 409.0 | **920.9** | 397.9 | 324.1 | **722.0** | 193.4 | 91.7 | **285.1** |
| | de Ruiter & Beringen | 459.6 | 313.8 | **773.4** | 158.3 | 78.7 | **237.0** | 452.7 | 342.6 | **795.3** | 513.3 | 232.3 | **745.6** | 202.1 | 80.6 | **282.7** |
| | LCPC | 483.5 | 312.8 | **796.3** | 214.0 | 97.0 | **311.0** | 480.8 | 373.3 | **854.1** | 493.3 | 502.4 | **995.7** | 187.8 | 47.1 | **234.9** |
| | Tumay & Fakhroo | 510.8 | 542.2 | **1053.0** | 342.2 | 155.2 | **497.4** | 523.5 | 481.4 | **1004.9** | 501.6 | 559.8 | **1061.4** | 135.8 | 97.9 | **233.7** |
| | Aoki & De Alencar | 312.9 | 392.6 | **705.5** | 108.1 | 112.2 | **220.3** | 381.3 | 493.3 | **874.6** | 338.6 | 640.0 | **978.6** | 162.8 | 62.4 | **225.2** |
| | Price & Wardle | 289.9 | 254.9 | **544.8** | 144.6 | 64.7 | **209.3** | 359.1 | 350.1 | **709.2** | 310.8 | 326.2 | **637.0** | 125.6 | 38.3 | **163.9** |
| | Philipponnat | 574.3 | 301.3 | **875.6** | 175.3 | 83.1 | **258.4** | 592.7 | 341.9 | **934.6** | 658.5 | 398.3 | **1056.8** | 282.7 | 44.6 | **327.3** |
| | Penpile | 238.8 | 111.8 | **350.6** | 140.5 | 40.3 | **180.8** | 259.0 | 140.1 | **399.1** | 228.8 | 182.1 | **410.9** | 96.4 | 16.0 | **112.4** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **688.0** | - | - | **340.0** | - | - | **658.0** | - | - | **460.0** | - | - | **119.0** |
| Static Analysis | "-method (used by DOTD) | 350.1 | 132.7 | **482.8** | 161.6 | 20.6 | **182.2** | 320.1 | 42.6 | **362.7** | 408.0 | 627.4 | **1035.4** | 246.4 | 9.3 | **255.7** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing) $Q_u$: Total ultimate capacity ($Q_s+Q_t$)

## Table–12

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 009-01-0059 Rocky Bayou Bridge | | | 452-90-0017 Wardline Road | | | 455-05-36 Sugarhouse Road | | | 455-05-36 Sugarhouse Road | | | 455-05-36 Sugarhouse Road | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 14" Square PPC | | | TP1, 14" Square PPC | | | TP1, 14" Square PPC | | | TP2, 14" Square PPC | | | TP3, 14"Square PPC | | |
| | Pile Length (ft) | 60 | | | 65 | | | 75 | | | 75 | | | 65 | | |
| | Embedded Length (ft) | 39 | | | 52 | | | 70 | | | 70 | | | 59 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | No | | | No | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 173.0 | 91.6 | **264.6** | 131.2 | 56.5 | **187.7** | 118.7 | 14.3 | **133.0** | 99.6 | 19.1 | **118.7** | 113.0 | 10.7 | **123.7** |
| | de Ruiter & Beringen | 180.6 | 41.2 | **221.8** | 125.3 | 39.8 | **165.1** | 88.7 | 6.4 | **95.1** | 90.4 | 8.6 | **99.0** | 67.2 | 4.8 | **72.0** |
| | LCPC | 172.3 | 70.6 | **242.9** | 120.8 | 79.9 | **200.7** | 97.5 | 11.1 | **108.6** | 94.2 | 16.8 | **111.0** | 78.7 | 7.5 | **86.2** |
| | Tumay & Fakhroo | 109.3 | 99.5 | **208.8** | 104.4 | 56.6 | **161.0** | 117.0 | 16.3 | **133.3** | 128.3 | 19.1 | **147.4** | 103.3 | 13.2 | **116.5** |
| | Aoki & De Alencar | 163.4 | 71.8 | **235.2** | 103.0 | 96.0 | **199.0** | 60.9 | 10.4 | **71.3** | 59.9 | 13.9 | **73.8** | 46.1 | 10.2 | **56.3** |
| | Price & Wardle | 161.9 | 43.3 | **205.2** | 112.0 | 70.9 | **182.9** | 90.7 | 6.1 | **96.8** | 75.1 | 9.1 | **84.2** | 96.5 | 5.3 | **101.8** |
| | Philipponnat | 213.7 | 55.9 | **269.6** | 160.6 | 56.0 | **216.6** | 92.0 | 8.2 | **100.2** | 103.3 | 13.0 | **116.3** | 69.7 | 6.9 | **76.6** |
| | Penpile | 87.1 | 28.6 | **115.7** | 73.1 | 29.8 | **102.9** | 74.0 | 4.5 | **78.5** | 62.4 | 6.9 | **69.3** | 73.6 | 3.3 | **76.9** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **129.0** | - | - | **164.0** | - | - | **104.0** | - | - | **83.0** | - | - | **55.0** |
| Static Analysis | "-method (used by DOTD) | 68.3 | 5.6 | **73.9** | 129.3 | 9.3 | **138.6** | 128.2 | 9.1 | **137.3** | 115.7 | 6.1 | **121.8** | 106.2 | 8.5 | **114.7** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 434-01-0002 Mississippi River Bridge | | | 434-01-0002 Mississippi River Bridge | | | 434-01-0002 Mississippi River Bridge | | | 452-01-25 Ruddock Frenier-Manchae | | | 452-01-25 Ruddock Frenier-Manchae | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 14" Square PPC | | | TP2, 14" Square PPC | | | TP3, 14" Square PPC | | | TP1, 24" Square PPC | | | TP2, 30" Square PPC | | |
| | Pile Length (ft) | 105 | | | 105 | | | 105 | | | NA | | | NA | | |
| | Embedded Length (ft) | 82 | | | 73 | | | 64 | | | 65 | | | 65 | | |
| | Pile Classification | Friction | | | Friction | | | End-bearing | | | Friction | | | Friction | | |
| | Pile Tested to Failure | No | | | No | | | Yes | | | No | | | No | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive and Cohesionless at the tip | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 114.9 | 33.7 | **148.6** | 117.1 | 60.9 | **178.0** | 121.8 | 95.2 | **217.0** | 135.8 | 59.3 | **195.1** | 167.0 | 95.4 | **262.4** |
| | de Ruiter & Beringen | 154.1 | 24.1 | **178.2** | 129.1 | 46.1 | **175.2** | 134.7 | 79.6 | **214.3** | 123.1 | 26.7 | **149.8** | 144.8 | 42.9 | **187.7** |
| | LCPC | 155.0 | 33.8 | **188.8** | 139.7 | 57.2 | **196.9** | 132.8 | 61.9 | **194.7** | 139.9 | 41.4 | **181.3** | 167.8 | 60.1 | **227.9** |
| | Tumay & Fakhroo | 152.9 | 39.7 | **192.6** | 145.9 | 66.2 | **212.1** | 122.7 | 101.5 | **224.2** | 169.1 | 60.1 | **229.2** | 206.4 | 95.4 | **301.8** |
| | Aoki & De Alencar | 103.3 | 39.7 | **143.0** | 93.6 | 65.7 | **159.3** | 105.7 | 86.3 | **192.0** | 84.0 | 40.9 | **124.9** | 98.3 | 63.8 | **162.1** |
| | Price & Wardle | 79.3 | 28.5 | **107.8** | 81.3 | 40.7 | **122.0** | 87.3 | 45.5 | **132.8** | 88.9 | 25.5 | **114.4** | 107.6 | 39.2 | **146.8** |
| | Philipponnat | 209.9 | 35.8 | **245.7** | 177.0 | 46.2 | **223.2** | 185.3 | 53.5 | **238.8** | 131.9 | 33.3 | **165.2** | 155.9 | 51.3 | **207.2** |
| | Penpile | 65.7 | 17.8 | **83.5** | 64.6 | 19.7 | **84.3** | 67.2 | 20.1 | **87.3** | 77.3 | 15.5 | **92.8** | 94.4 | 24.9 | **119.3** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **173.0** | - | - | **175.0** | - | - | **189.0** | - | - | **213.0** | - | - | **286.0** |
| Static Analysis | "-method (used by DOTD) | 124.2 | 9.3 | **133.5** | 173.4 | 2.5 | **175.9** | 122.5 | 9.3 | **131.8** | 228.1 | 49.8 | **277.9** | 285.1 | 77.8 | **362.9** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s$+$Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 452-01-25 Ruddock Frenier-Manchae | | | 452-01-25 Ruddock Frenier-Manchae | | | 005-01-0056 Southern Pacific Railroad Overpass | | | 005-01-0056 Southern Pacific Railroad Overpass | | | 005-01-0056 Southern Pacific Railroad Overpass | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP3, 24" Square PPC | | | TP4, 30" Square PPC | | | TP1, 24" Square PPC | | | TP2, 14" Square PPC | | | TP3, 24" Square PPC | | |
| | Pile Length (ft) | NA | | | NA | | | 90 | | | 74 | | | 92 | | |
| | Embedded Length (ft) | 90 | | | 80 | | | 85 | | | 64 | | | 87 | | |
| | Pile Classification | Friction | | | Friction | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | No | | | No | | | Yes | | | Yes | | | Yes | | |
| | Predominant Soil * | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ |
| | Schmertmann | 201.5 | 69.9 | **271.4** | 204.3 | 77.4 | **281.7** | 216.0 | 38.2 | **254.2** | 95.2 | 19.3 | **114.5** | 254.8 | 39.8 | **294.6** |
| | de Ruiter & Beringen | 231.7 | 41.6 | **273.3** | 224.4 | 45.8 | **270.2** | 145.1 | 17.2 | **162.3** | 87.3 | 13.4 | **100.7** | 214.7 | 17.9 | **232.6** |
| | LCPC | 214.8 | 78.1 | **292.9** | 225.6 | 49.7 | **275.3** | 183.7 | 27.7 | **211.4** | 101.1 | 26.1 | **127.2** | 241.4 | 36.6 | **278.0** |
| | Tumay & Fakhroo | 242.3 | 148.5 | **390.8** | 255.6 | 106.7 | **362.3** | 222.8 | 44.6 | **267.4** | 108.6 | 25.9 | **134.5** | 223.6 | 41.1 | **264.7** |
| | Aoki & De Alencar | 158.6 | 100.1 | **258.7** | 152.9 | 76.0 | **228.9** | 99.5 | 38.4 | **137.9** | 55.6 | 29.2 | **84.8** | 141.1 | 33.9 | **175.0** |
| | Price & Wardle | 137.1 | 48.4 | **185.5** | 124.1 | 39.1 | **163.2** | 134.4 | 20.5 | **154.9** | 60.8 | 13.3 | **74.1** | 166.6 | 21.0 | **187.6** |
| | Philipponnat | 244.5 | 64.5 | **309.0** | 238.4 | 44.8 | **283.2** | 150.4 | 28.0 | **178.4** | 108.9 | 17.4 | **126.3** | 250.5 | 23.7 | **274.2** |
| | Penpile | 117.8 | 30.9 | **148.7** | 113.9 | 21.7 | **135.6** | 124.7 | 13.2 | **137.9** | 54.3 | 8.5 | **62.8** | 146.0 | 16.8 | **162.8** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **297.0** | - | - | **495.0** | - | - | **204.0** | - | - | **129.0** | - | - | **309.0** |
| Static Analysis | "-method (used by DOTD) | 365.3 | 653.8 | **1019.1** | 370.9 | 17.7 | **388.6** | 223.4 | 12.4 | **235.8** | 106.5 | 154.6 | **261.1** | 272.8 | 39.4 | **312.2** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s+Q_t$)

**Table–12**

Results of the analyses conducted on square precast prestressed concrete piles driven into Louisiana soils, continued

| | State Project Identification | 283-03-52 New Orleans I | | | 283-09-52 New Orleans II | | | 23-01-12 Alexandria | | | 23-01-12 Alexandria | | | 855-14-13 Houma | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pile and Soil Identification | Pile ID | TP1, 18" Square PPC | | | TP6, 16" Square PPC | | | TP1, 14" Square PPC | | | TP3, 14" Square PPC | | | TP1, 18" Square PPC | | |
| | Pile Length (ft) | NA | | | NA | | | NA | | | NA | | | NA | | |
| | Embedded Length (ft) | 125 | | | 125 | | | 31 | | | 45 | | | 105 | | |
| | Pile Classification | Friction | | | End-bearing | | | Friction | | | Friction | | | Friction | | |
| | Pile Tested to Failure | Yes | | | No | | | Yes | | | No | | | Yes | | |
| | Predominant Soil * | Cohesive and cohesionless | | | Cohesive and cohesionless | | | Cohesionless | | | Cohesionless | | | Cohesive | | |
| Methods of Predicting Pile Capacity by Cone Penetration Test | **Predicted Ultimate Load (ton)** | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | Qs | Qt | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ | $Q_t$ | $\mathbf{Q_u}$ | $Q_s$ |
| | Schmertmann | 221.1 | 37.9 | **259.0** | 149.7 | 44.0 | **193.7** | 123.2 | 17.4 | **140.6** | 145.2 | 21.7 | **166.9** | 163.8 | 20.3 | **184.1** |
| | de Ruiter & Beringen | 208.9 | 24.5 | **233.4** | 114.3 | 28.2 | **142.5** | 92.0 | 12.8 | **104.8** | 116.7 | 12.8 | **129.5** | 148.0 | 9.1 | **157.1** |
| | LCPC | 217.2 | 22.4 | **239.6** | 119.8 | 34.9 | **154.7** | 95.8 | 10.6 | **106.4** | 117.1 | 12.6 | **129.7** | 171.5 | 13.9 | **185.4** |
| | Tumay & Fakhroo | 260.4 | 59.2 | **319.6** | 198.3 | 55.8 | **254.1** | 85.6 | 17.8 | **103.3** | 105.5 | 25.4 | **130.9** | 205.1 | 23.3 | **228.4** |
| | Aoki & De Alencar | 137.8 | 40.0 | **177.8** | 75.6 | 45.5 | **121.1** | 92.6 | 41.1 | **133.7** | 109.7 | 17.5 | **127.2** | 103.7 | 16.1 | **119.8** |
| | Price & Wardle | 130.0 | 22.0 | **152.0** | 87.5 | 30.4 | **117.9** | 100.2 | 17.5 | **117.7** | 114.0 | 10.1 | **124.1** | 93.6 | 10.3 | **103.9** |
| | Philipponnat | 256.4 | 27.6 | **284.0** | 139.2 | 37.6 | **176.8** | 143.8 | 7.4 | **151.2** | 175.5 | 12.3 | **187.8** | 166.6 | 13.5 | **180.1** |
| | Penpile | 121.2 | 10.0 | **131.2** | 83.8 | 14.4 | **98.2** | 53.7 | 4.4 | **58.1** | 66.5 | 5.5 | **72.0** | 90.6 | 6.0 | **96.6** |
| Load Test Interpretation Method | Butler-Hoy (used by DOTD) | - | - | **319.0** | - | - | **179.0** | - | - | **81.0** | - | - | **119.0** | - | - | **211.0** |
| Static Analysis | "-method (used by DOTD) | 307.7 | 167.7 | **475.4** | 208.6 | 14.8 | **223.4** | 59.3 | 10.8 | **70.1** | 88.8 | 7.0 | **95.8** | 210.1 | 16.3 | **226.4** |

*: Cohesive (mainly clayey and silty clay soils) and Cohesionless (mainly sandy soils); $Q_s$: Pile shaft capacity (friction); $Q_t$: Pile tip capacity (end-bearing)
$Q_u$: Total ultimate capacity ($Q_s+Q_t$)

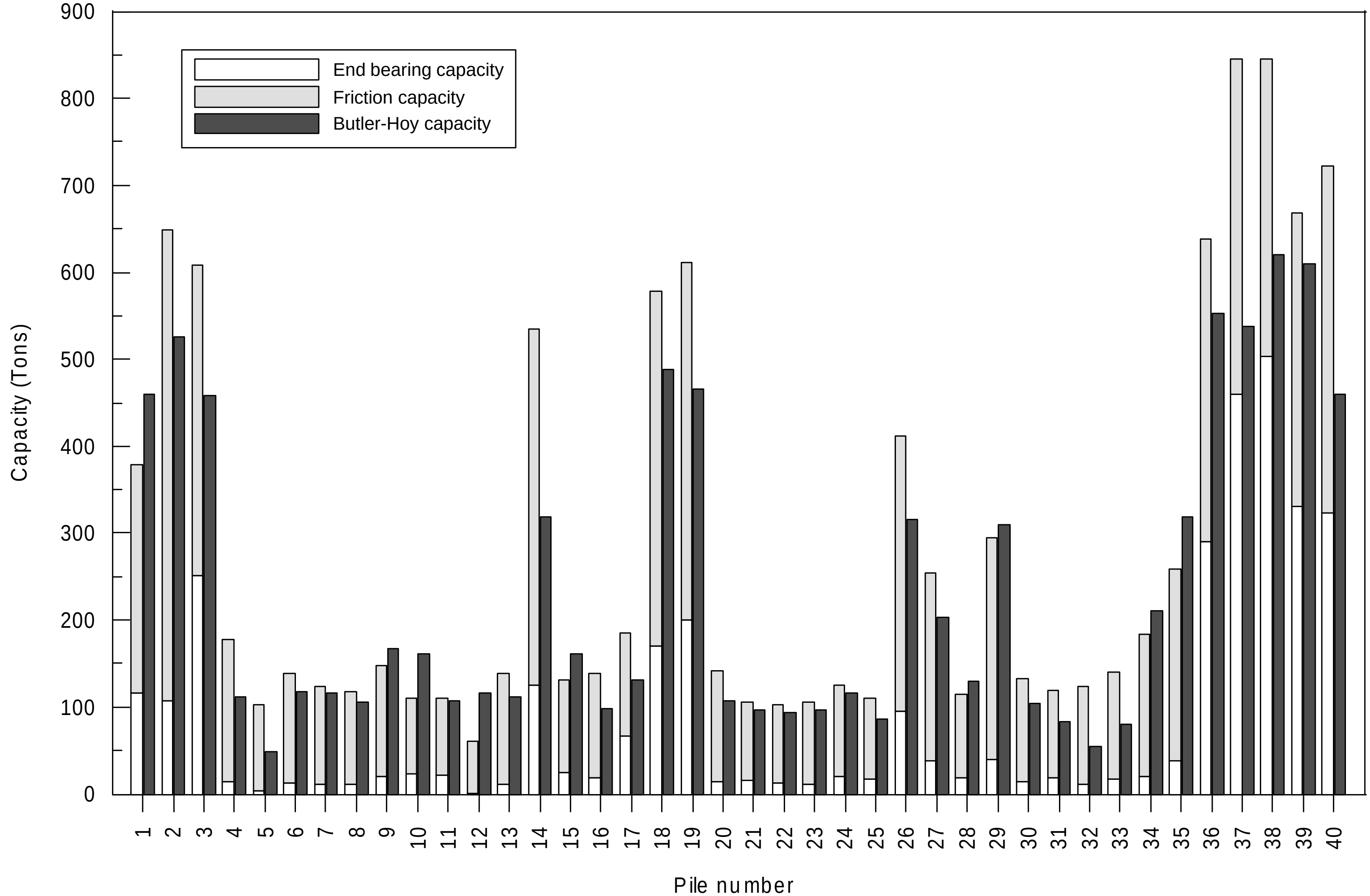



**Figure 18**

Comparison of measured and ultimate pile capacity predicted by Schmertmann method

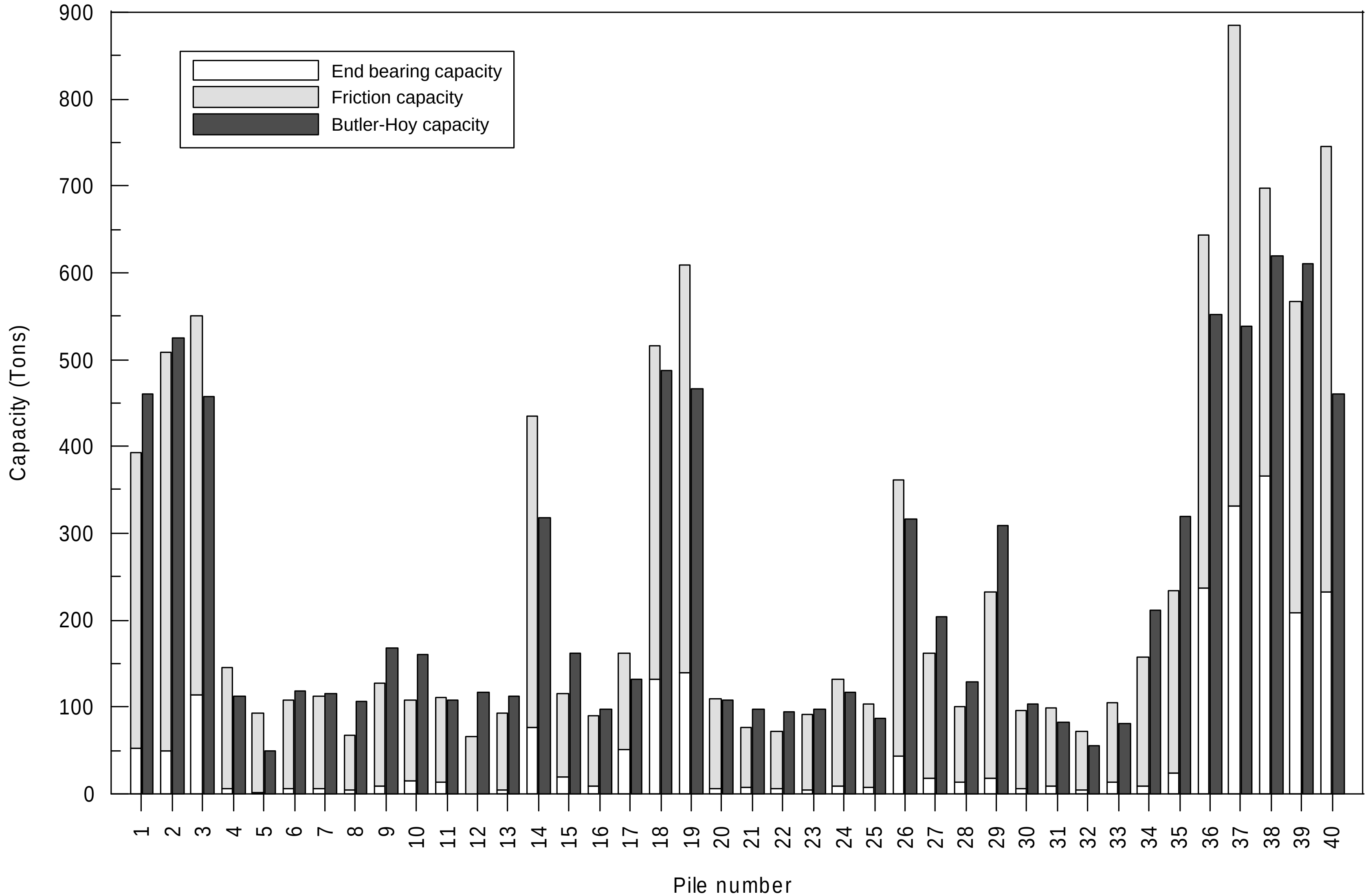



**Figure 19**

Comparison of measured and ultimate pile capacity predicted by de Ruiter and Beringen method

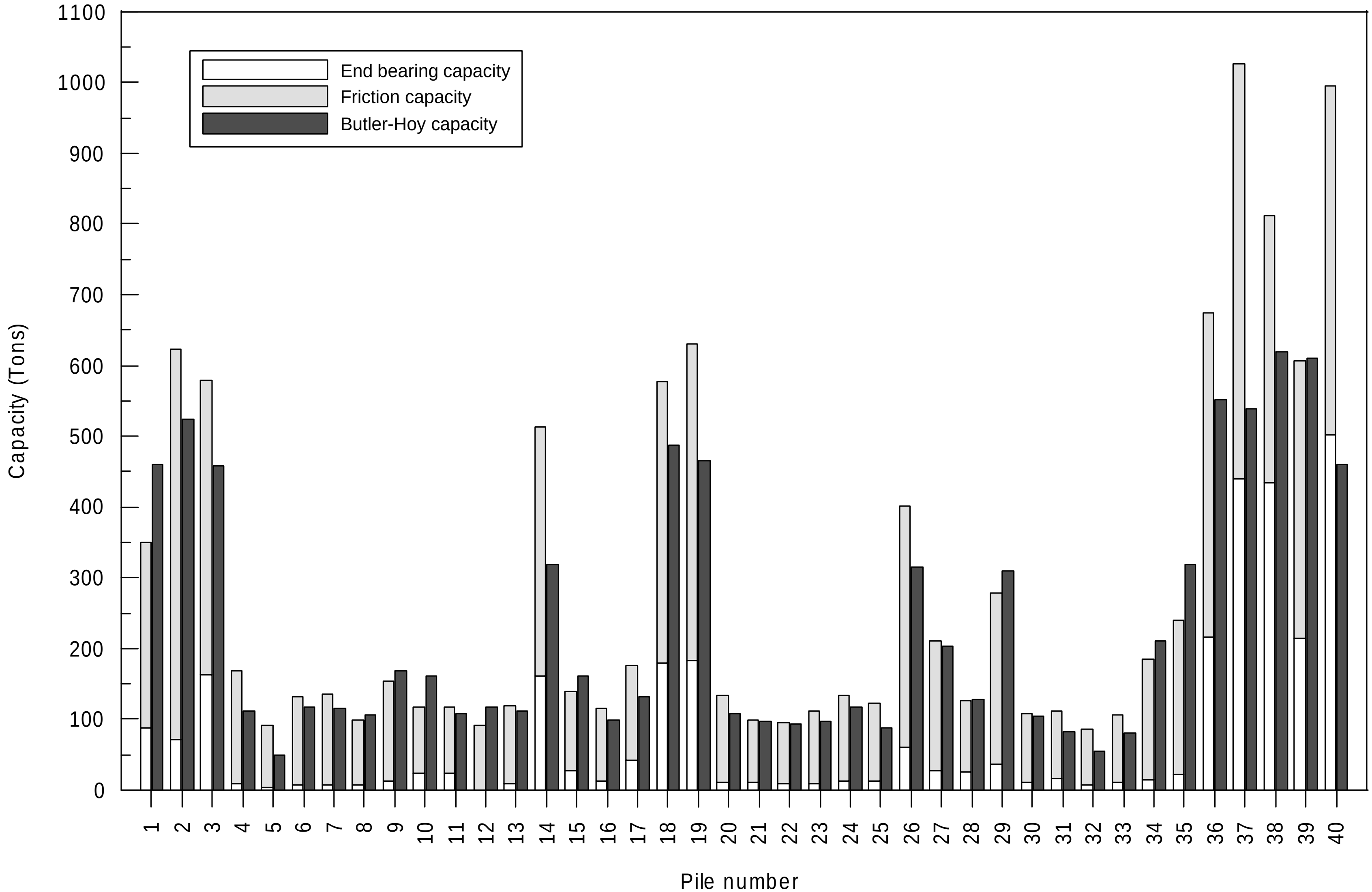



**Figure 20**

Comparison of measured and ultimate pile capacity predicted by LCPC method

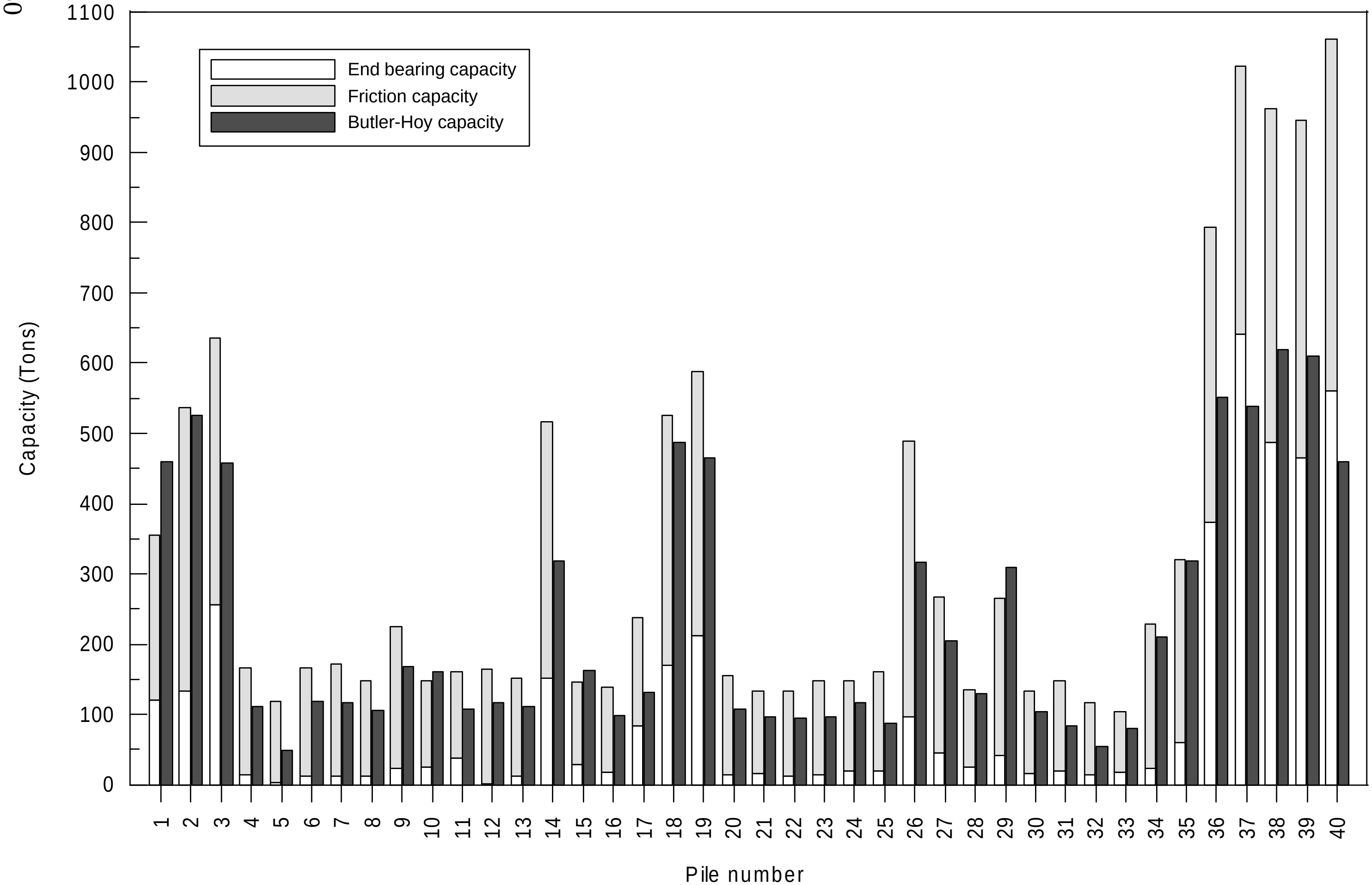



**Figure 21**

Comparison of measured and ultimate pile capacity predicted by Tumay and Fakhroo method

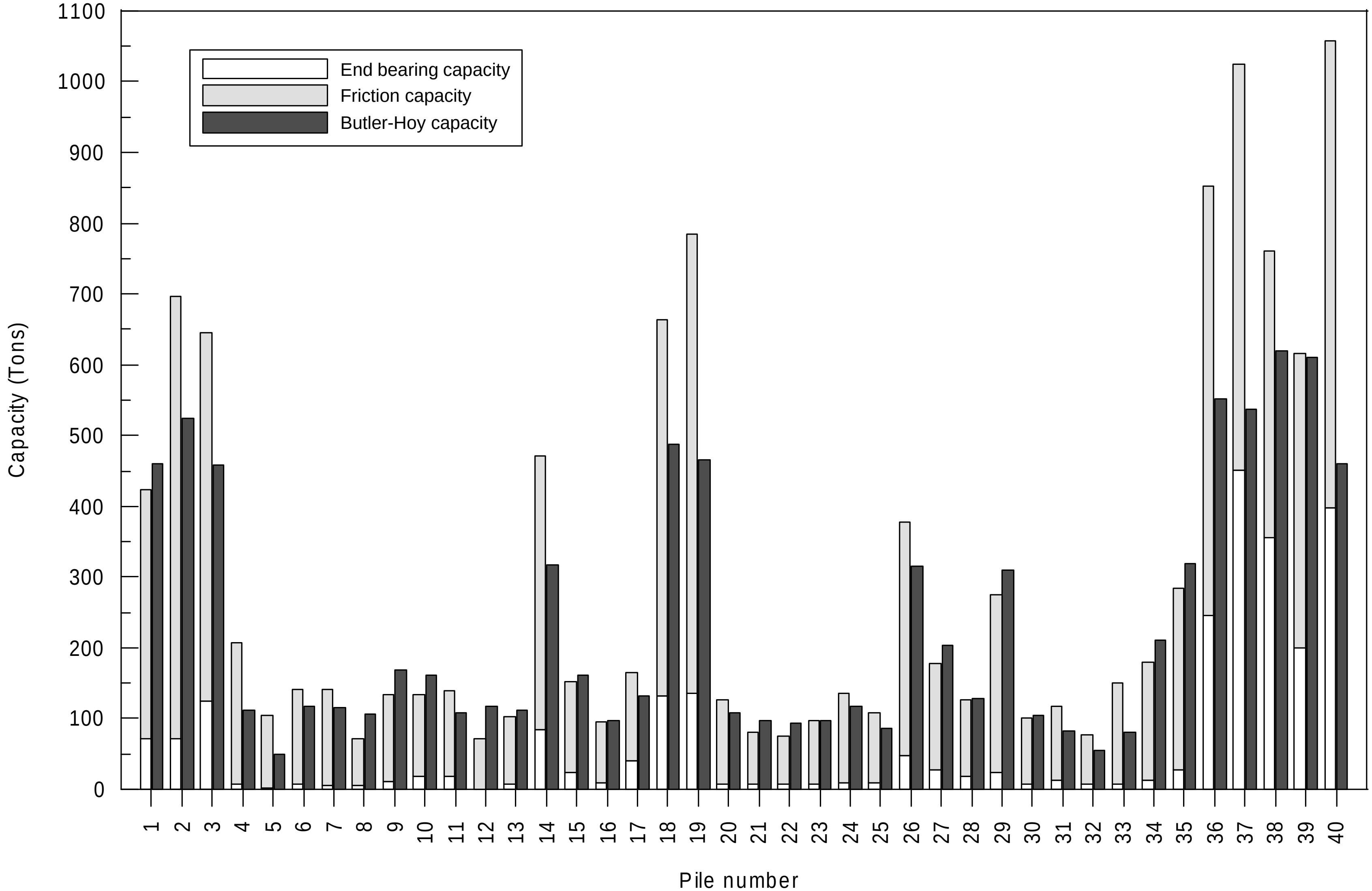



**Figure 22**

Comparison of measured and ultimate pile capacity predicted by Philipponnat method

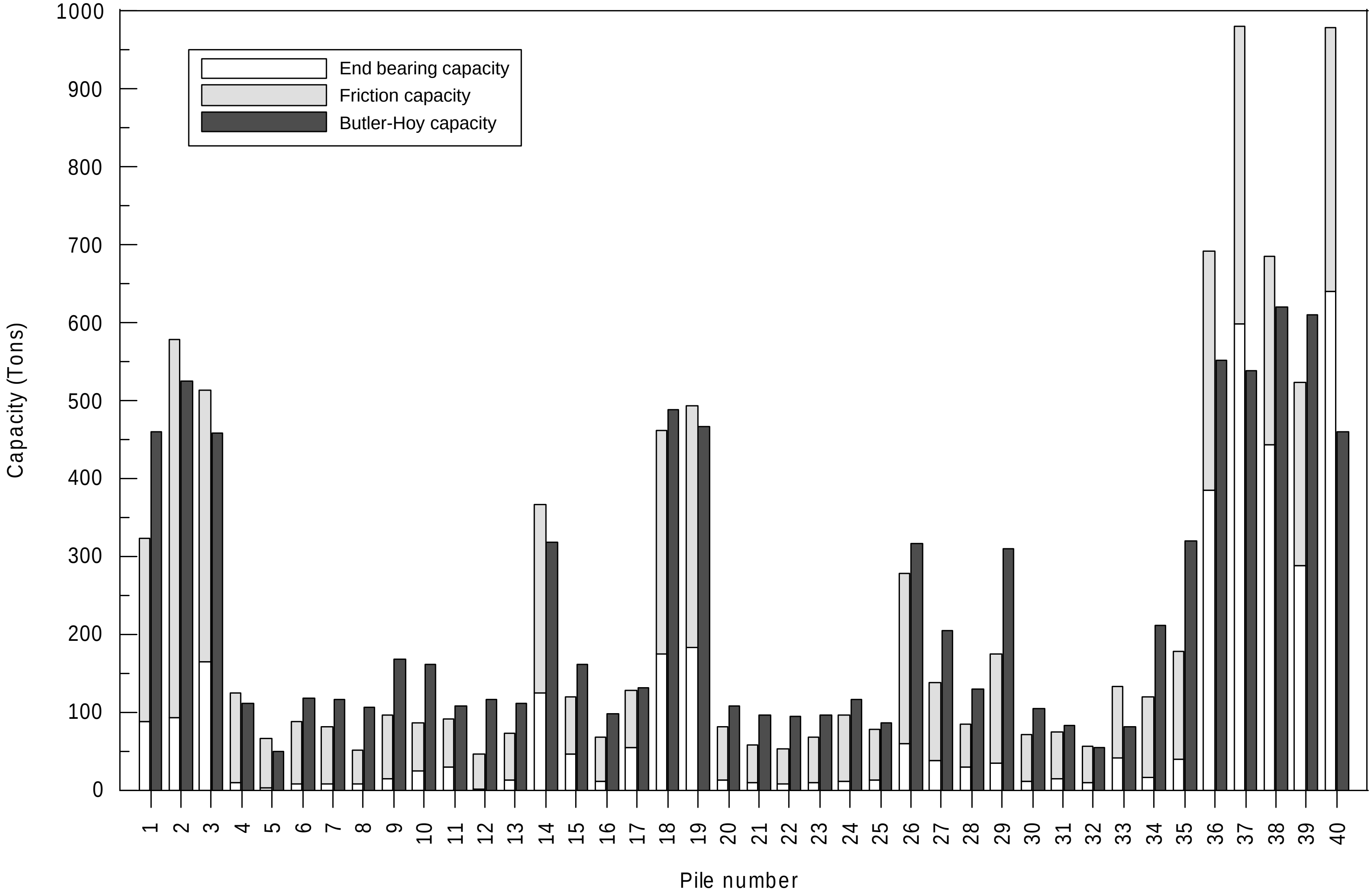



**Figure 23**

Comparison of measured and ultimate pile capacity predicted by Aoki and De Alencar method

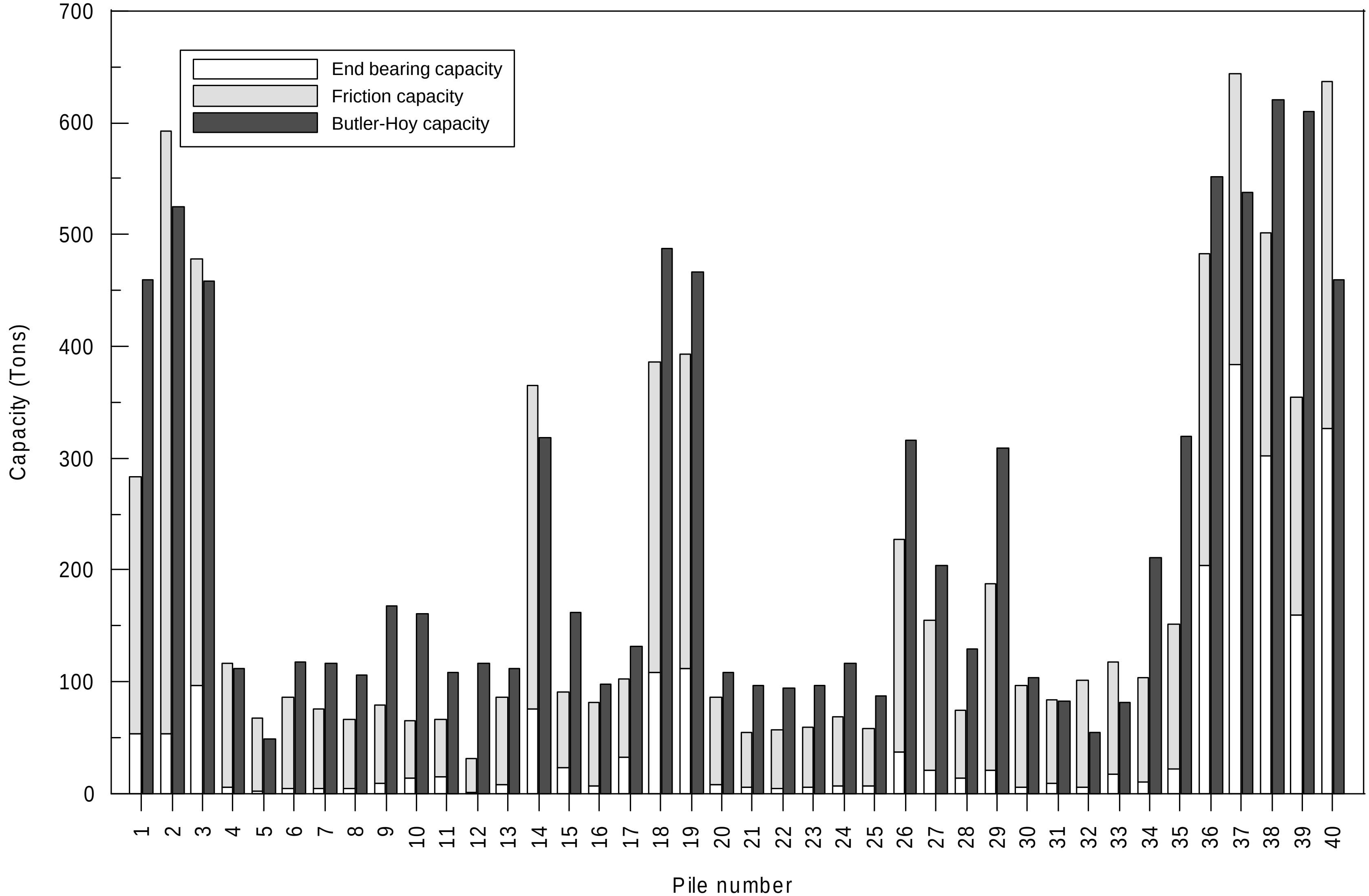



**Figure 24**

Comparison of measured and ultimate pile capacity predicted by Price and Wardle method

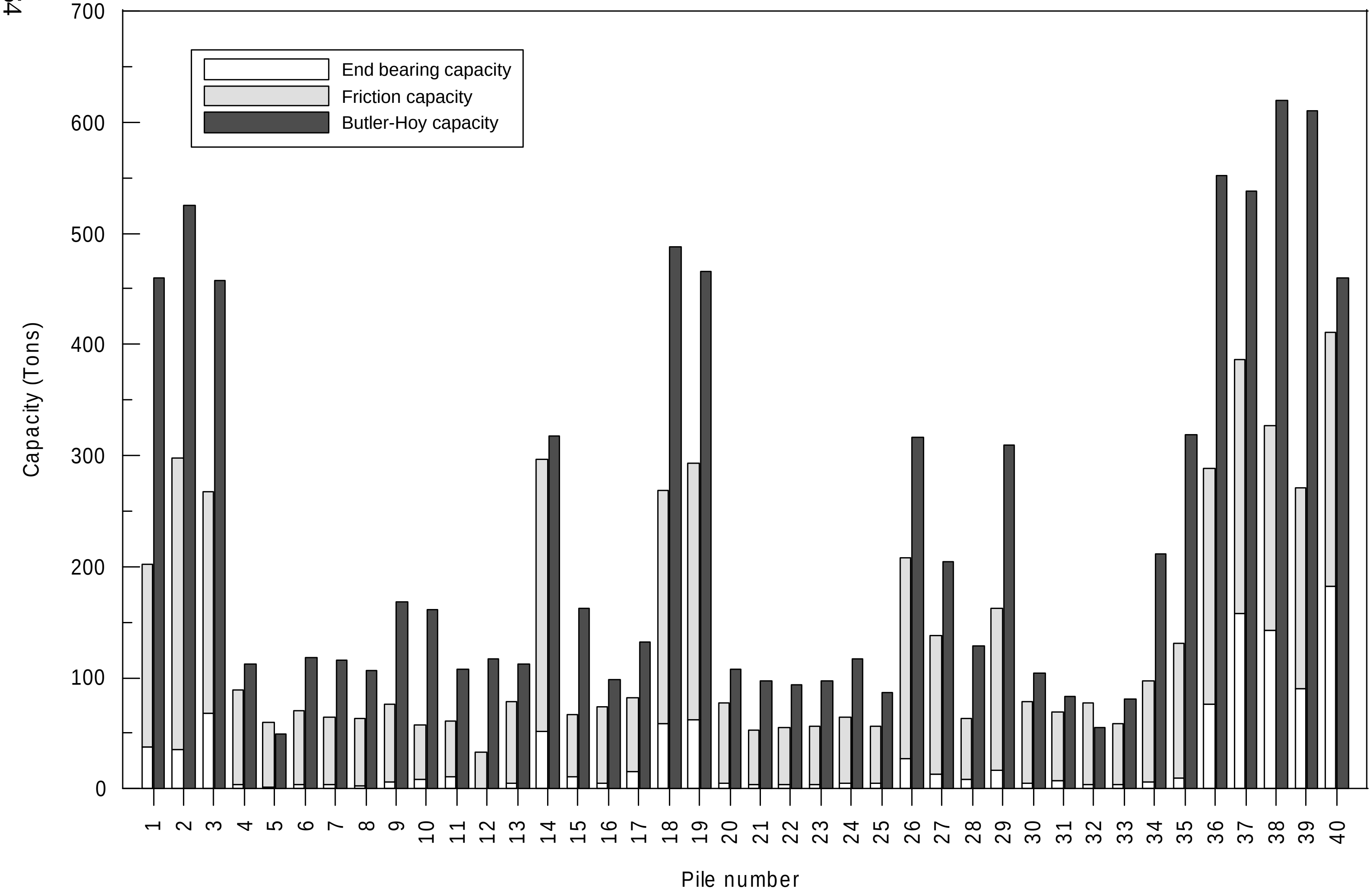



**Figure 25**

Comparison of measured and ultimate pile capacity predicted by Penpile method.

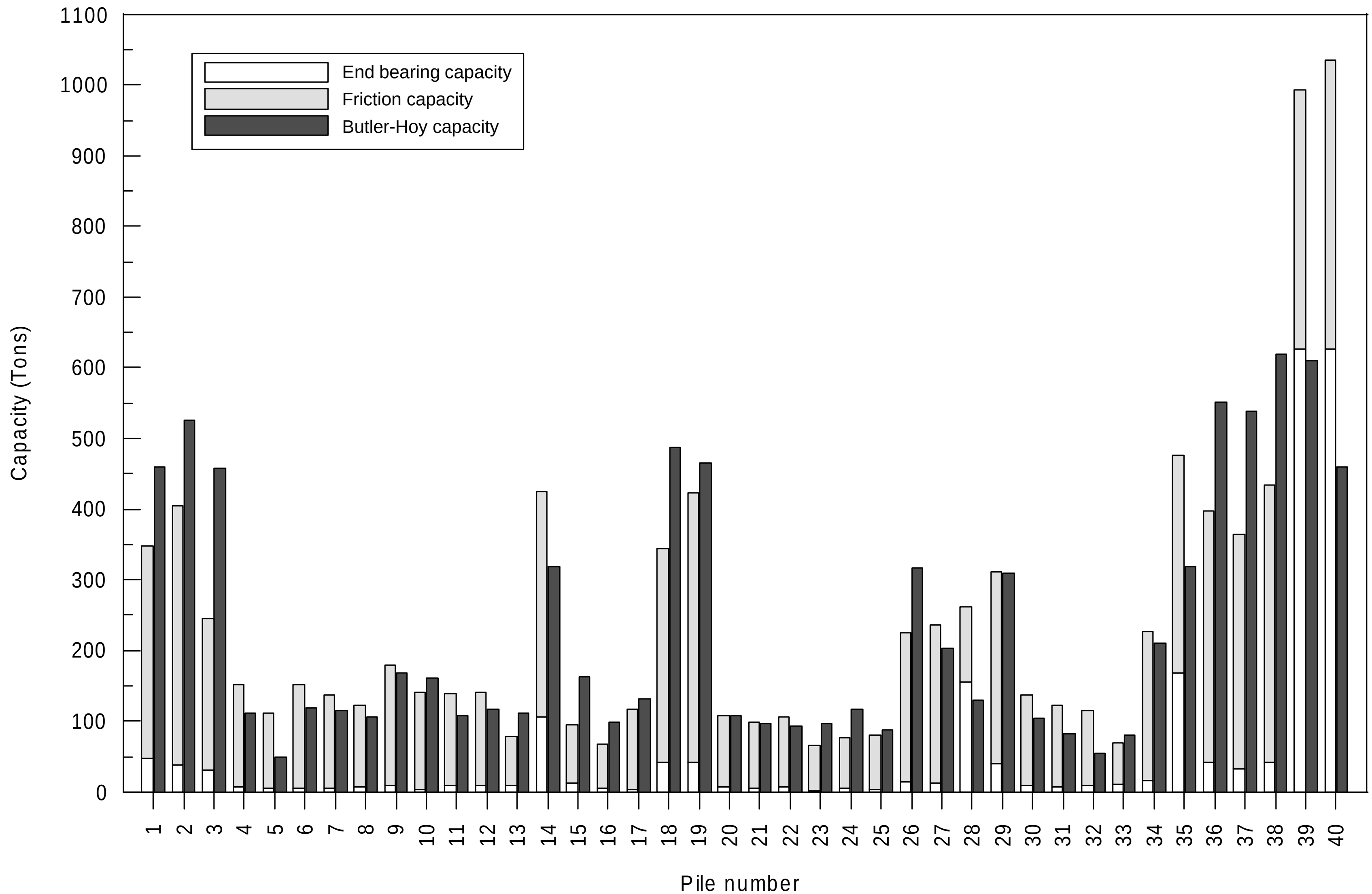
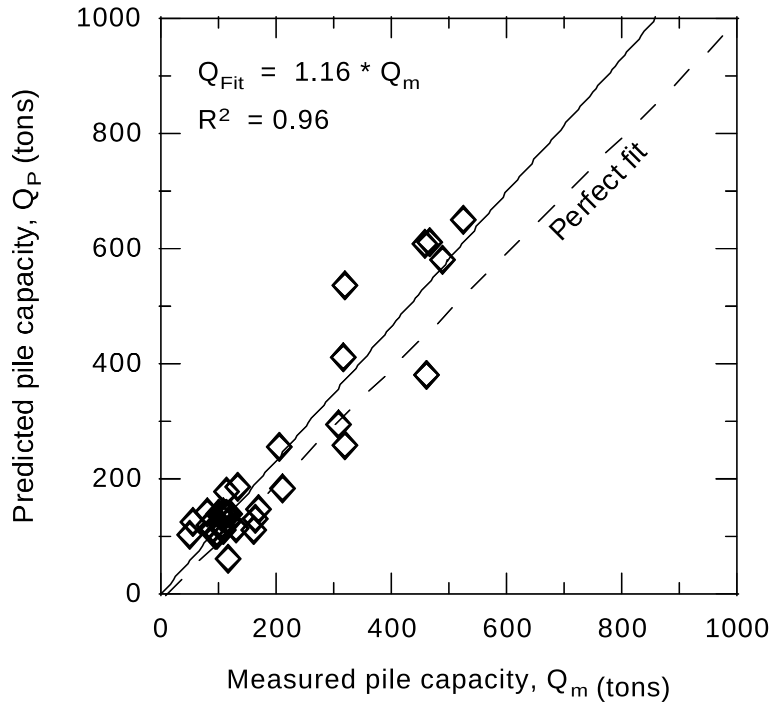
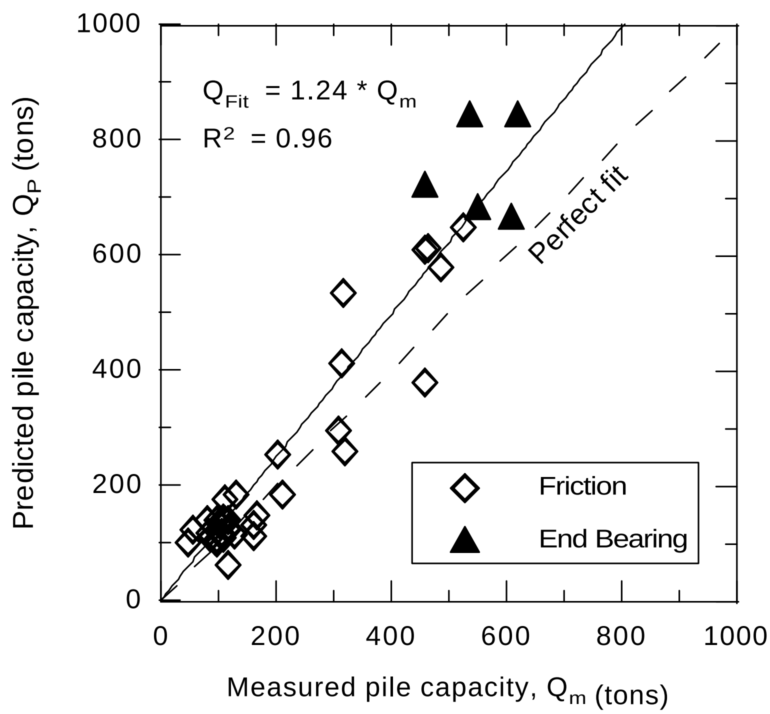



**Figure 26**

Comparison of measured and ultimate pile capacity predicted by $\alpha$-method



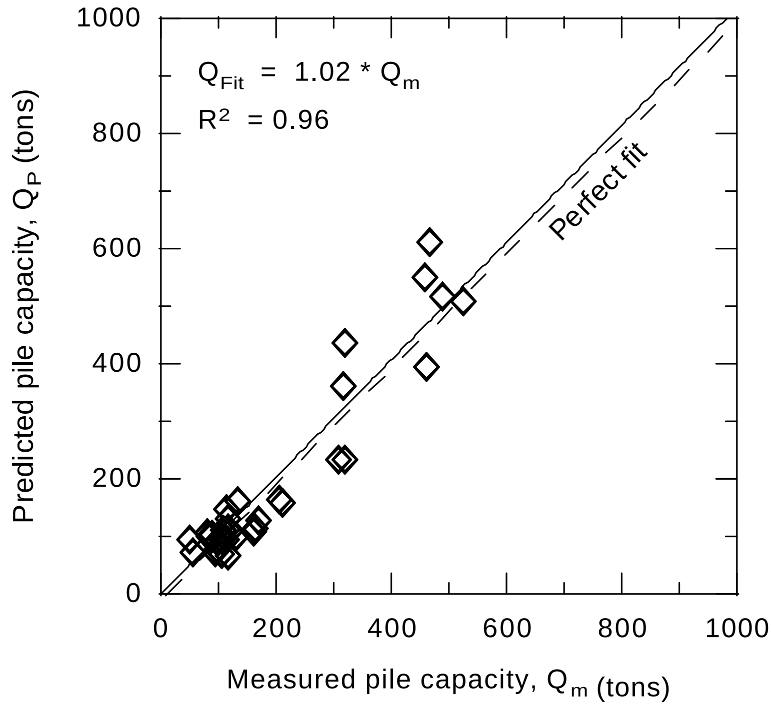

(a) Friction piles



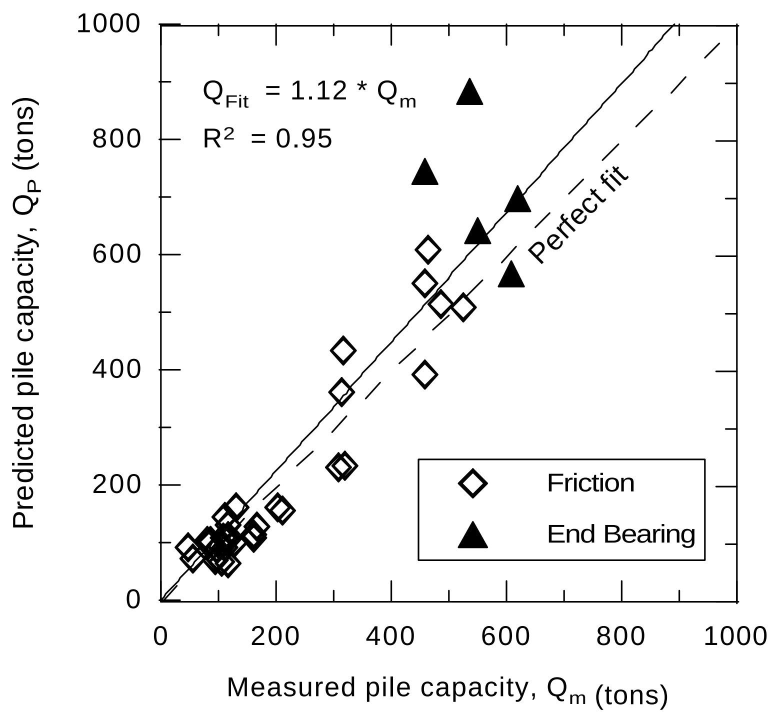

(b) All piles

**Figure 27**

Predicted (Schmertmann method) versus measured ultimate capacity



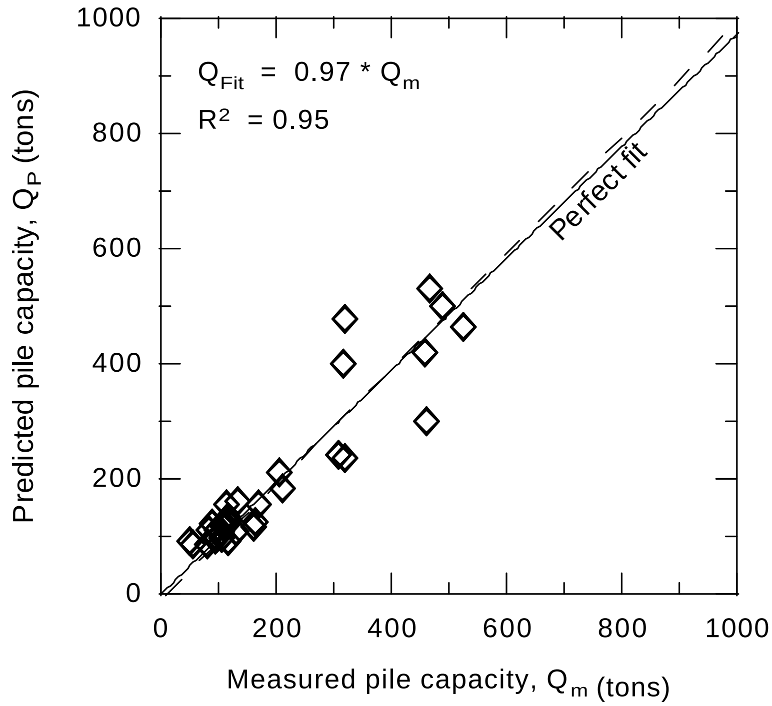

(a) Friction piles



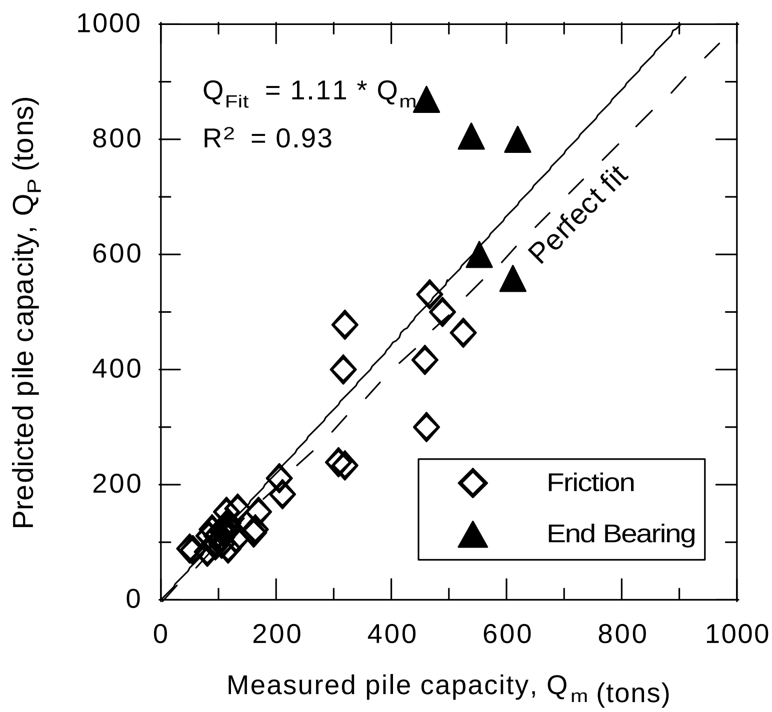

(b) All piles

**Figure 28**

Predicted (de Ruiter and Beringen method) versus measured ultimate capacity



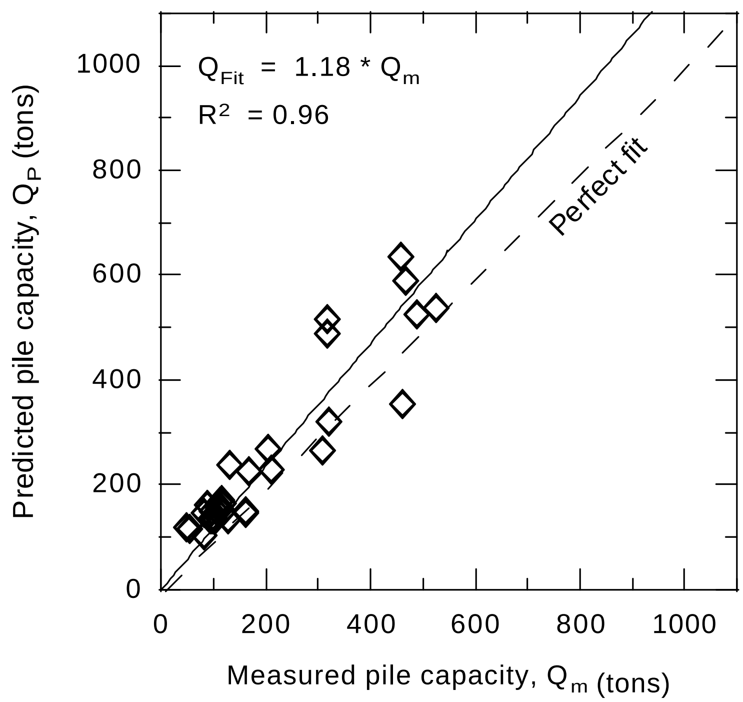

(a) Friction piles



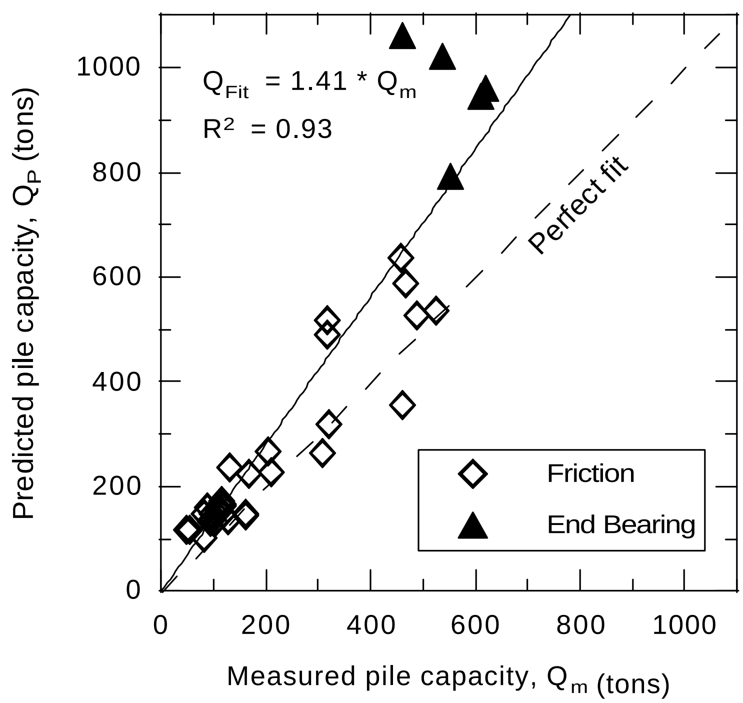

(b) All piles

**Figure 29**

Predicted (LCPC method) versus measured ultimate capacity



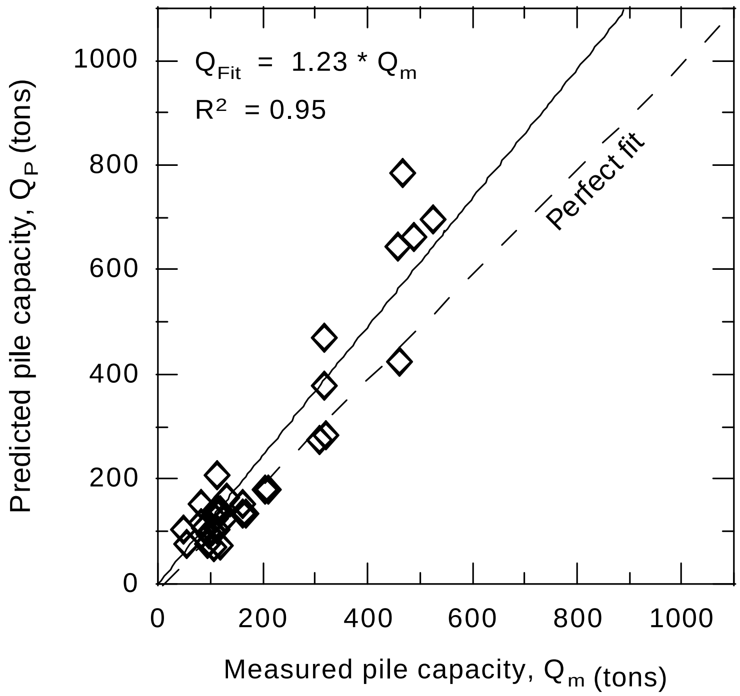

(a) Friction piles



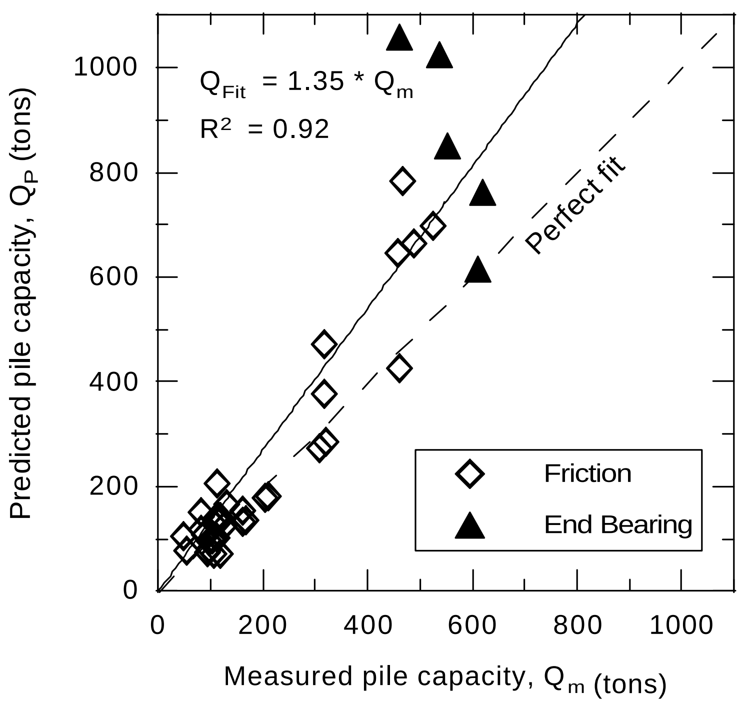

(b) All piles

**Figure 30**

Predicted (Tumay and Fakhroo method) versus measured ultimate capacity



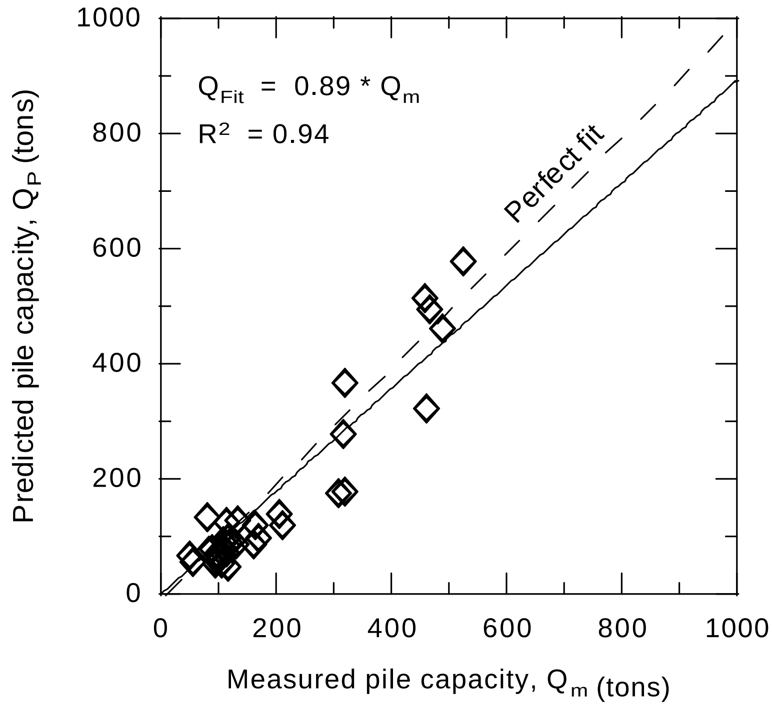

(a) Friction piles



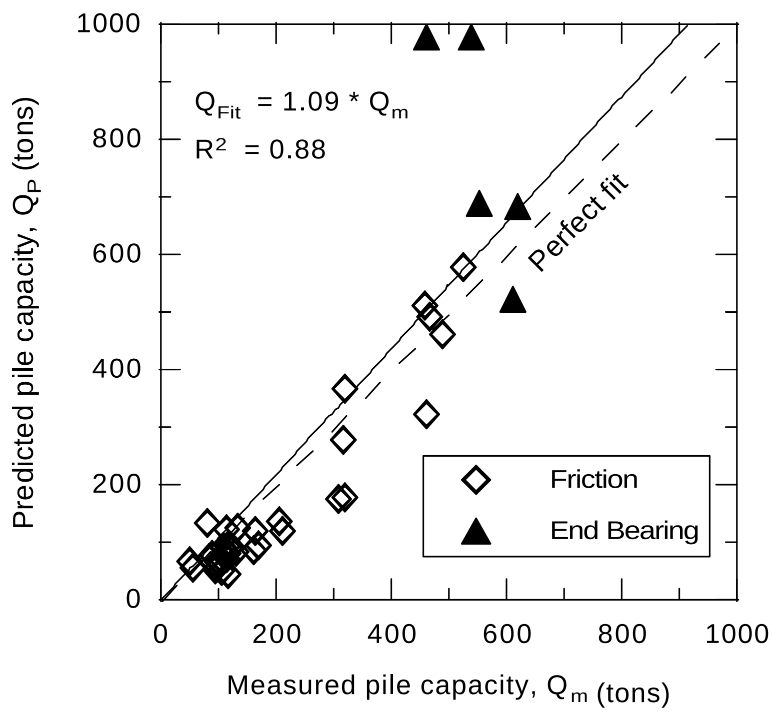

(b) All piles

**Figure 31**

Predicted (Philipponnat method) versus measured ultimate capacity



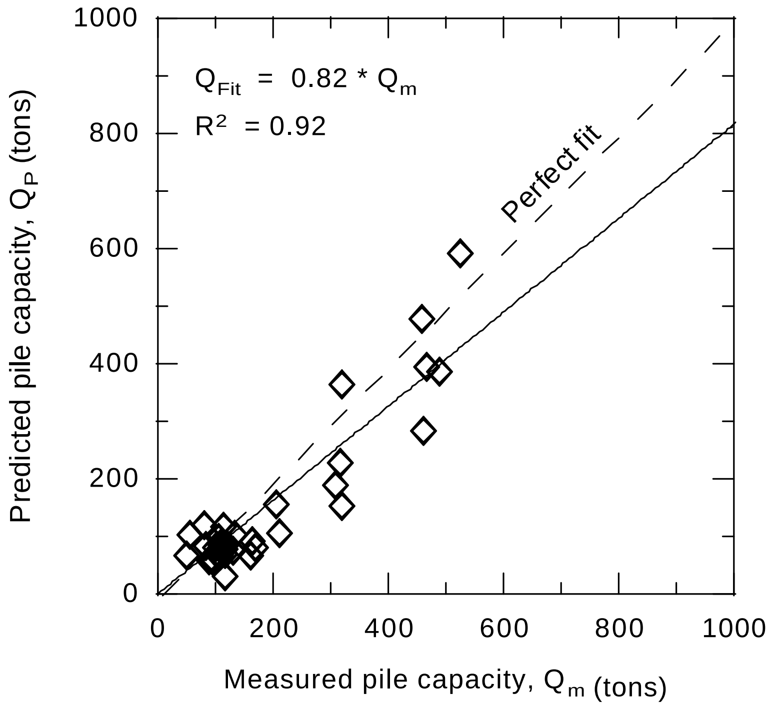

(a) Friction piles



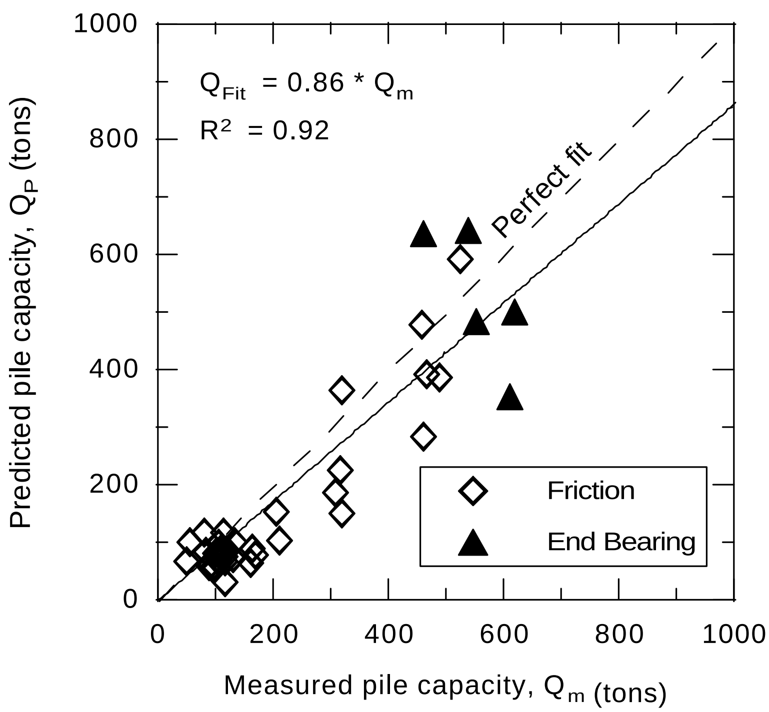

(b) All piles

**Figure 32**

Predicted (Aoki and De Alencar method) versus measured ultimate capacity



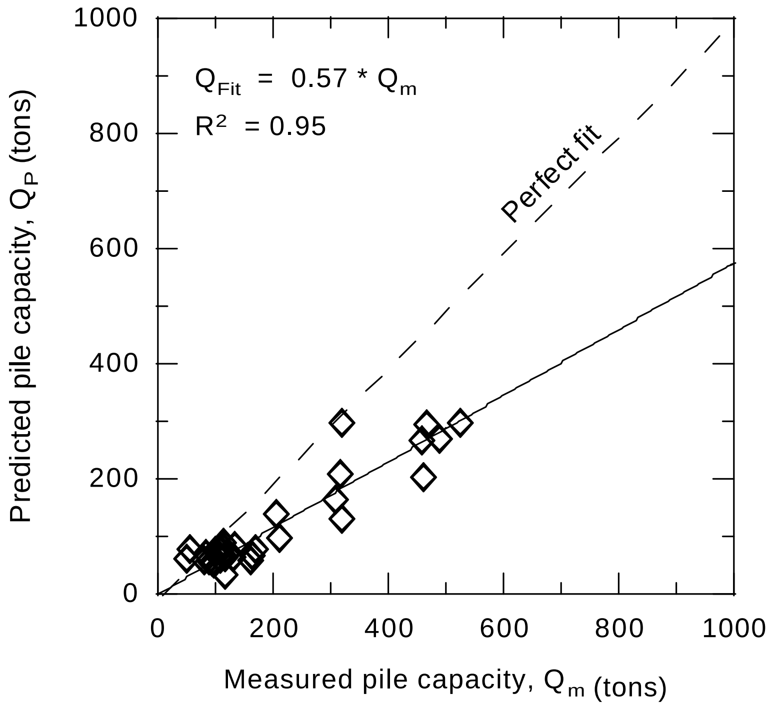

(a) Friction piles



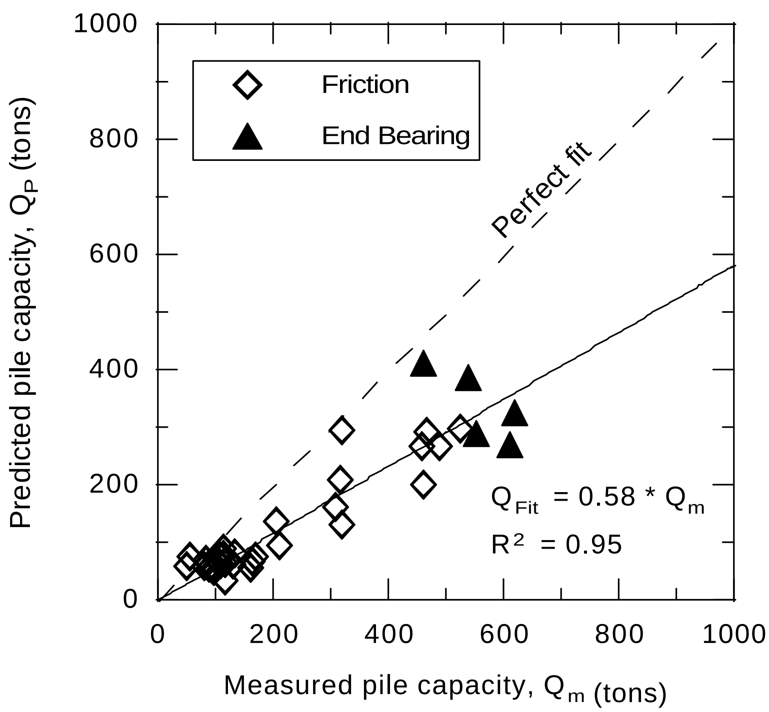

(b) All piles

**Figure 33**

Predicted (Price and Wardle method) versus measured ultimate capacity.



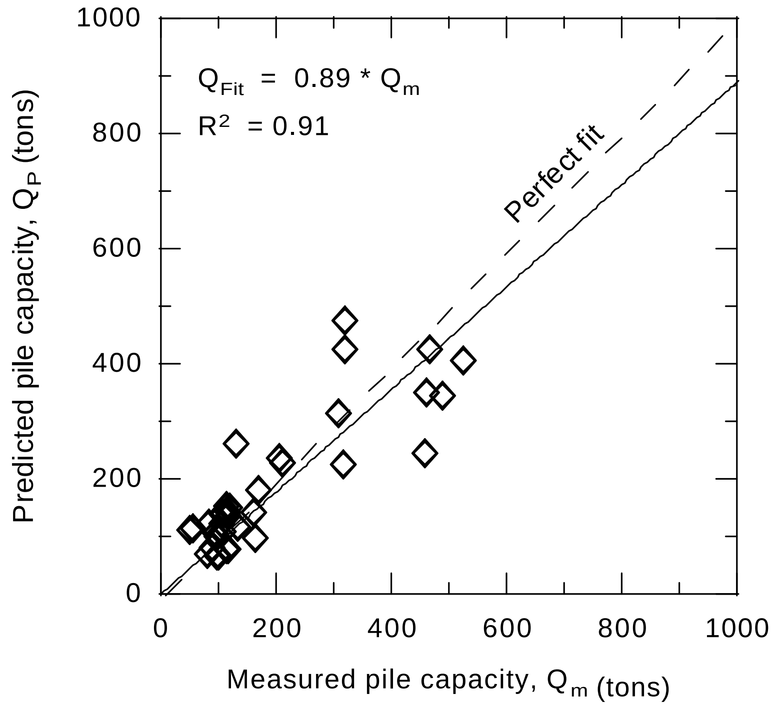

(a) Friction piles



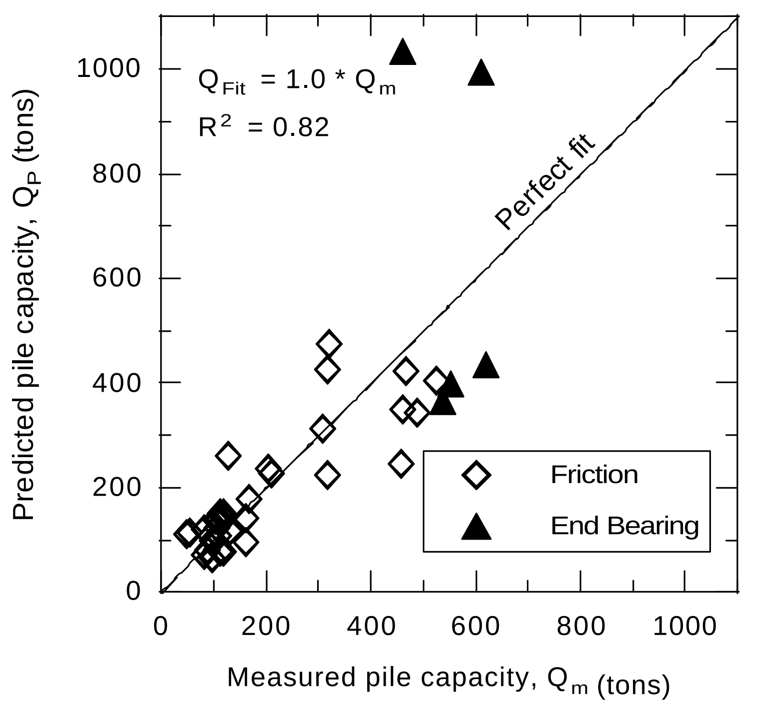

(b) All piles

**Figure 34**

Predicted (Penpile method) versus measured ultimate capacity




(a) Friction piles




(b) All piles

**Figure 35**

Predicted ($\alpha$-method) versus measured ultimate capacity

## STATISTICAL ANALYSES

Briaud and Tucker and Long et al. used statistical analyses to evaluate the performance of different methods to predict the ultimate load carrying capacity of piles [*17*], [*18*]. Statistical analyses provide the measures to rank the different methods based on their prediction accuracy. However, statistical analyses should not be the only criterion used to evaluate these methods. Comparisons of predicted and measured capacity (figures 27-35) should always be considered together with statistical evaluation [*17*].

Similar statistical analyses were conducted in this study to help in evaluating the performance of the CPT methods. The ratio of predicted to measured ultimate pile capacity ($Q_p/Q_m$) was the main variable considered in the analyses. This ratio ($Q_p/Q_m$) ranges from 0 to 4 with an optimum value of one. The CPT method underpredicts the measured capacity when $Q_p/Q_m<1$ and it overpredicts the measured capacity when $Q_p/Q_m>1$. The mean ( : ) and standard deviation ($F$) of $Q_p/Q_m$ are indicators of the accuracy and precision of the prediction method. An accurate and precise method gives :($Q_p/Q_m$)=1 and $F(Q_p/Q_m)=0$, respectively, which means that for each pile, the predicted pile capacity equals to the measured one. This case is *ideal*, however, in reality the method is better when :($Q_p/Q_m$) is closer to one and $F(Q_p/Q_m)$ is closer to 0.

Since 0#($Q_p/Q_m$)<4 with an optimum value of 1, Briaud and Tucker used the Log Normal distribution to evaluate the performance of pile capacity prediction methods. The Log Normal distribution is acceptable to represent the ratio of $Q_p/Q_m$, however, it is not symmetric around the mean, which means that the Log Normal distribution doesn't give an equal weight of underprediction and overprediction [*17*]. In order to use the Log Normal distribution, the mean (:$_{ln}$) and standard deviation ($F_{ln}$) are evaluated for the natural logarithm of $Q_p/Q_m$ as follows:

$$m_{\ln}\left(\frac{Q_p}{Q_m}\right) = \frac{1}{n}\sum_{i=1}^{n}\ln\left(\frac{Q_p}{Q_m}\right) \tag{27}$$

$$s_{\ln}\left(\frac{Q_p}{Q_m}\right) = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\ln\left(\frac{Q_p}{Q_m}\right)_i - m_{\ln}\right)^2} \tag{28}$$

The ratio $Q_p/Q_m$ and the natural logarithm of the ratio $ln(Q_p/Q_m)$ for each pile were calculated. Then, the mean ($\mu_{ln}$) and standard deviation ($\sigma_{ln}$), and coefficient of variation (COV) of $ln(Q_p/Q_m)$ for each method were determined.

The Log Normal distribution is defined as the distribution with the following density:

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma_{\ln} x} \exp\left( -\frac{1}{2} \left( \frac{\ln(x) - \mu_{\ln}}{\sigma_{\ln}} \right)^2 \right) \tag{29}$$

where $x = (Q_p/Q_m)$, $\mu_{ln}$ is the mean of $ln(Q_p/Q_m)$ and $\sigma_{ln}$ is the standard deviation of $ln(Q_p/Q_m)$. The distribution function of the Log Normal distribution is given by:

$$F(x) = \frac{1}{\sqrt{2\pi}\sigma_{\ln}} \int_0^x \frac{1}{u} \exp\left( \frac{-1}{2\sigma_{\ln}^2} (\ln(u) - \mu_{\ln})^2 \right) du \tag{30}$$

The Log Normal distribution was used to evaluate the different methods based on their prediction accuracy and precision. Figure 36 shows the Log Normal distribution for the different methods considered in this study. Evaluation of the different CPT prediction methods is presented later in this section.

Long et al. used the cumulative probability value to quantify the ability of different methods to predict the measured pile capacity [*18*]. The concept is to sort the ratio $Q_p/Q_m$ for each method in an ascending order. The smallest $Q_p/Q_m$ is given number $i$=1 and the largest is given $i$=$n$ where n is the number of piles considered in the analysis. The cumulative probability value for each $Q_p/Q_m$ is given by [*18*]:

$$CP_i = \frac{i}{n+1} \tag{31}$$

Together with the Log Normal distribution and the graphs of $Q_p/Q_m$, the cumulative probability concept was utilized to help in quantifying the performance of the investigated methods. The cumulative probability versus the ratio $Q_p/Q_m$ for the investigated methods are depicted in figures 37-45.

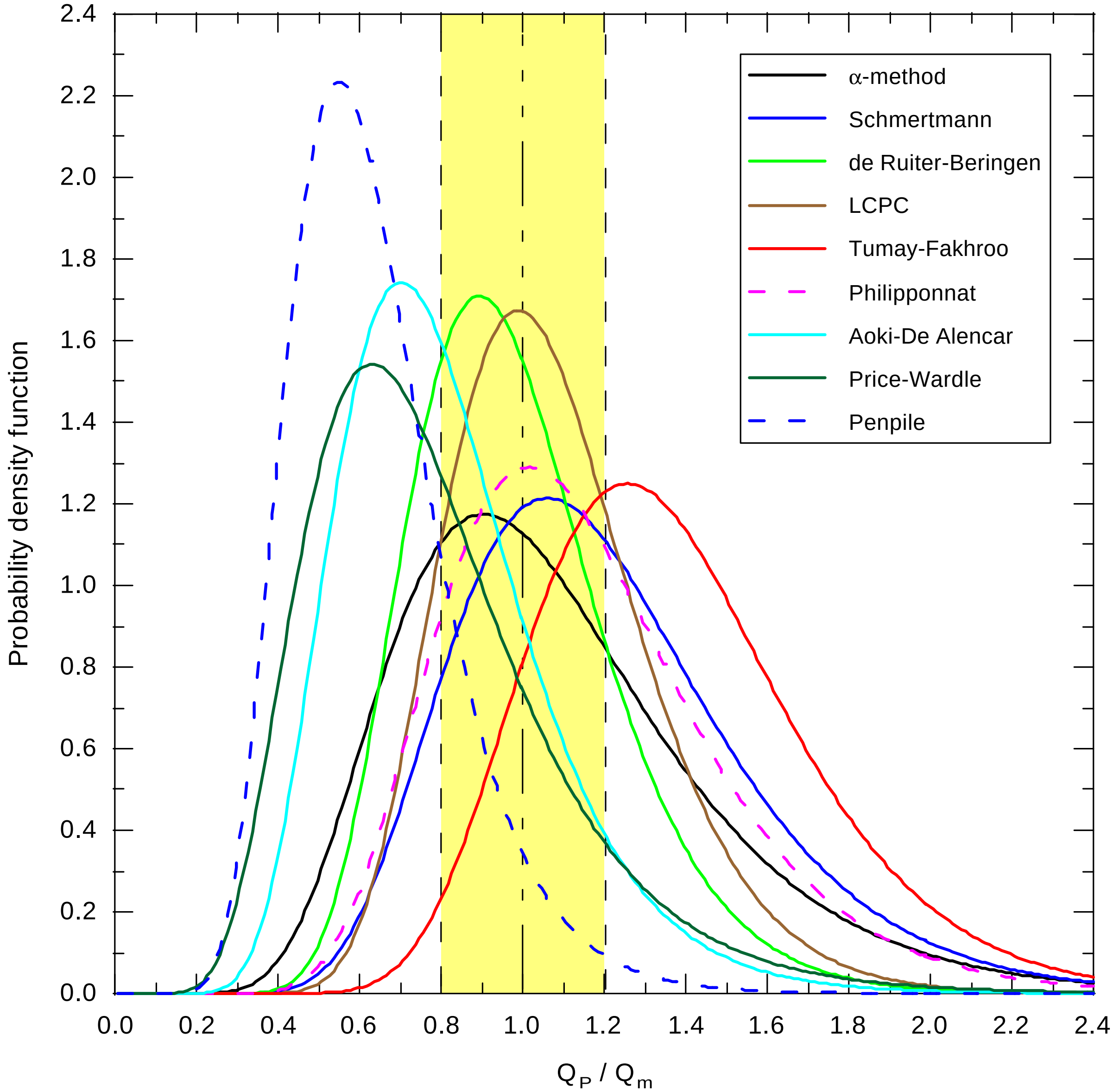



**Figure 36**

Probability distribution of $Q_P$ / $Q_m$ for all methods

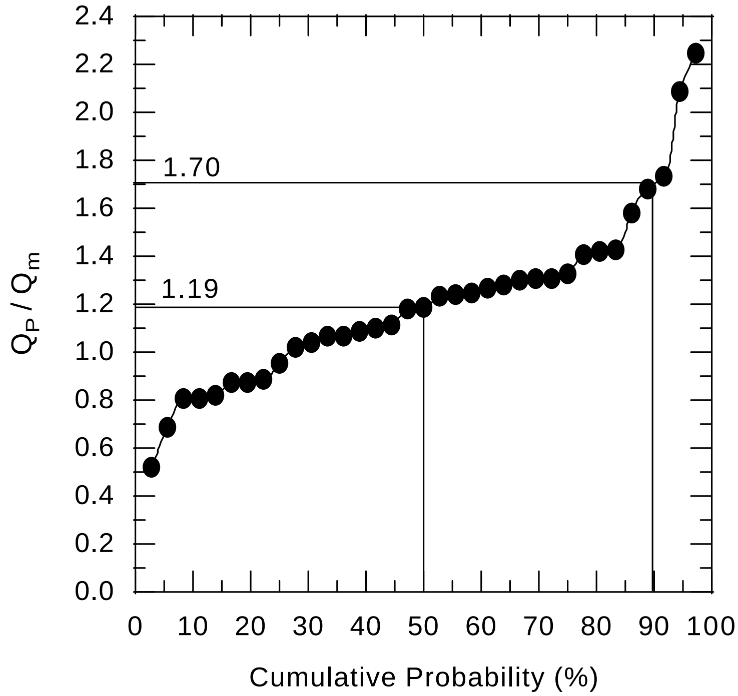



**Figure 37**
Cumulative probability plot for $Q_P$ / $Q_m$ -- Schmertmann method

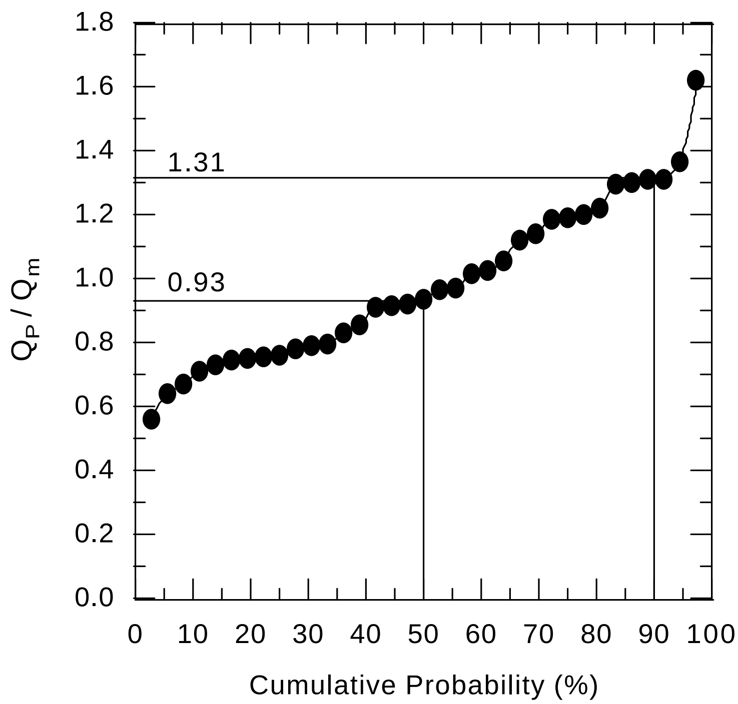



**Figure 38**
Cumulative probability plot for $Q_P$ / $Q_m$ -- de Ruiter and Beringen method

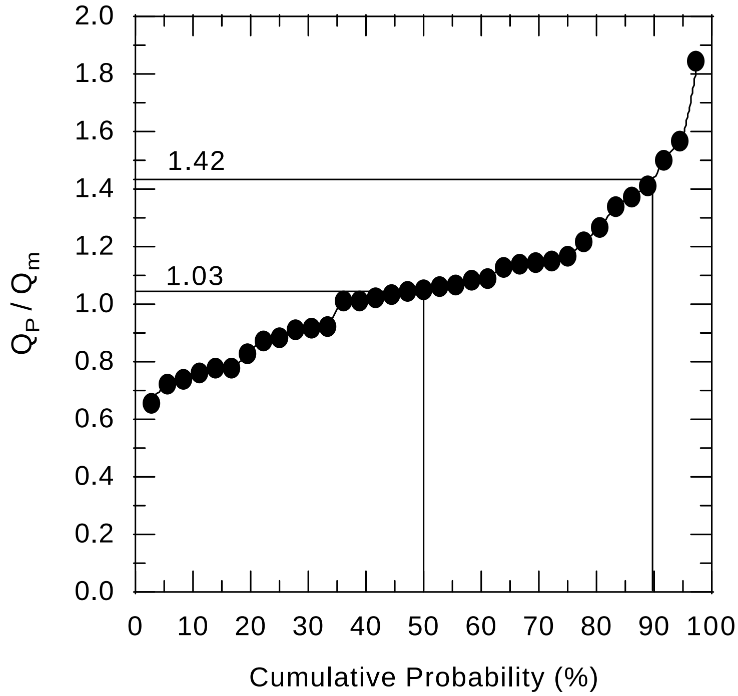



**Figure 39**
Cumulative probability plot for $Q_P$ / $Q_m$ **--** LCPC method

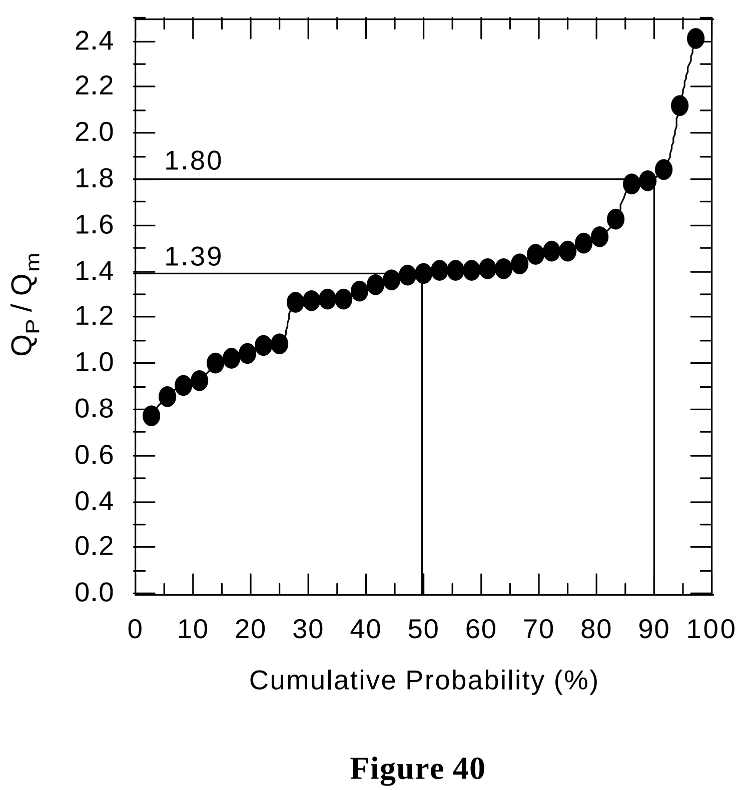



**Figure 40**
Cumulative probability plot for $Q_P$ / $Q_m$ **--** Tumay and Fakhroo method

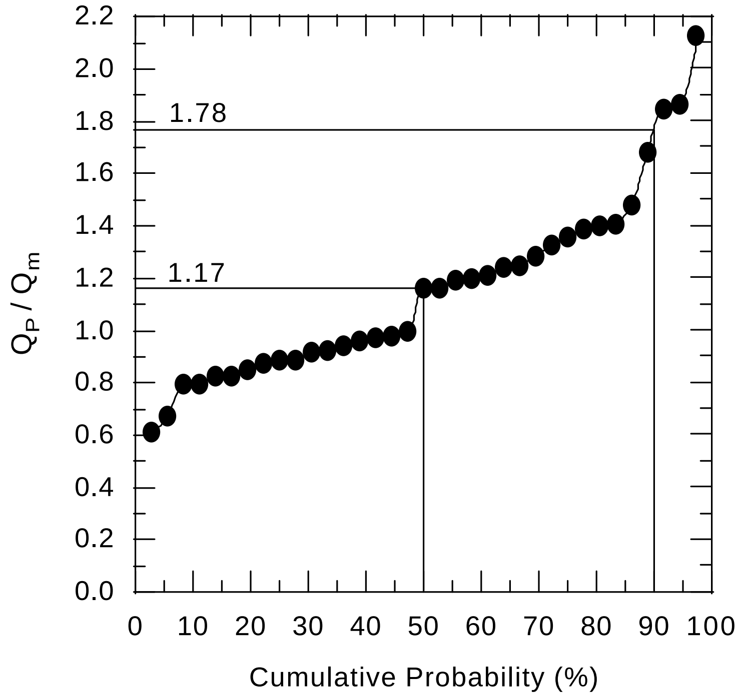



**Figure 41**
Cumulative probability plot for $Q_P$ / $Q_m$ -- Philiponnat method

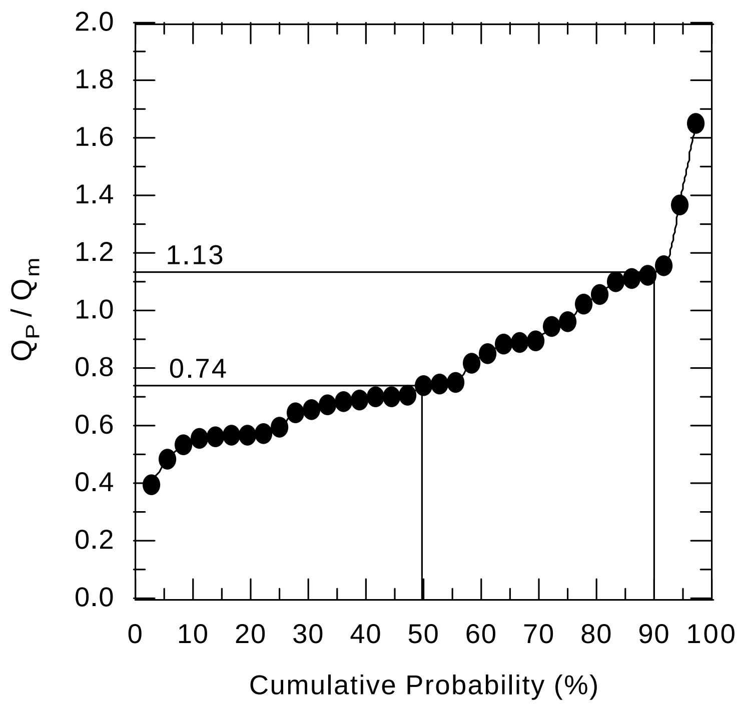



**Figure 42**
Cumulative probability plot for $Q_P$ / $Q_m$ -- Aoki and De Alencar method

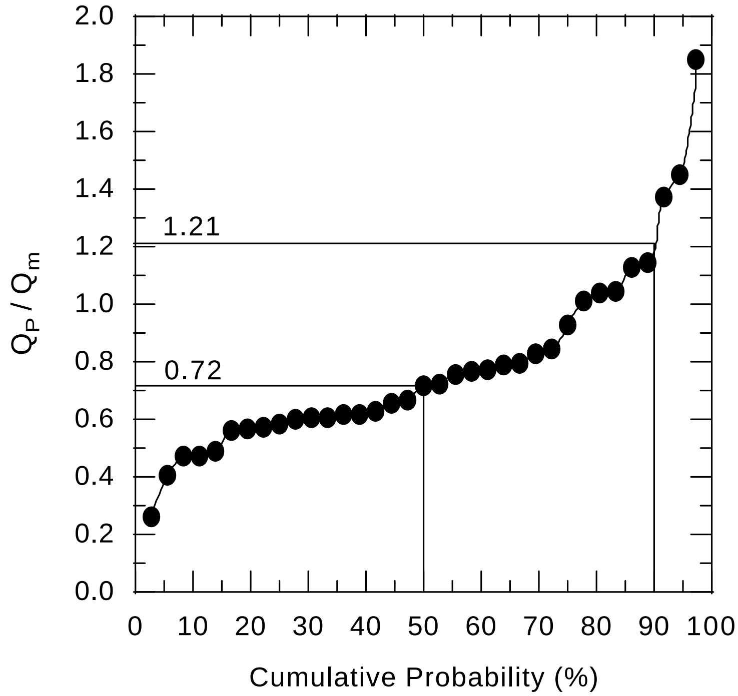



**Figure 43**

Cumulative probability plot for $Q_P$ / $Q_m$ -- Price and Wardle method

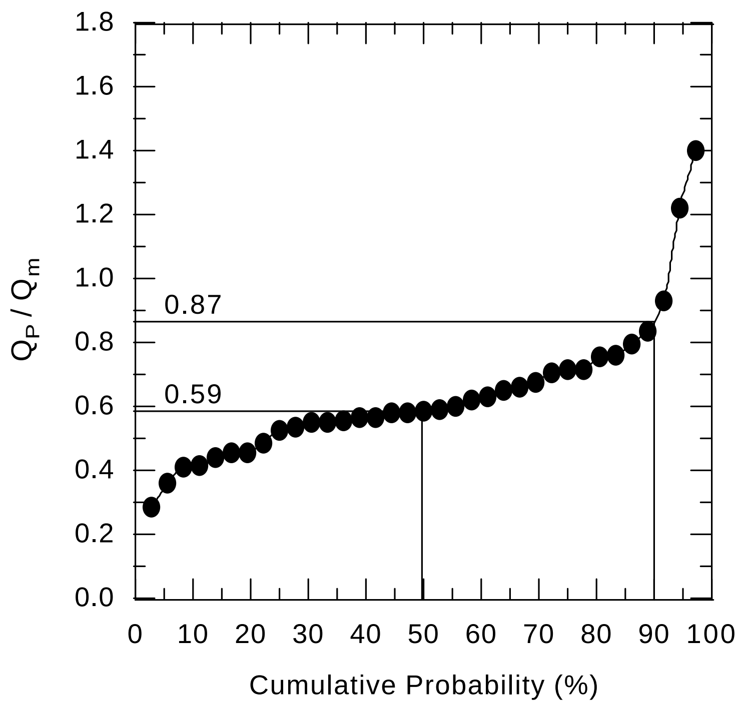



**Figure 44**

Cumulative probability plot for $Q_P$ / $Q_m$ -- Penpile method

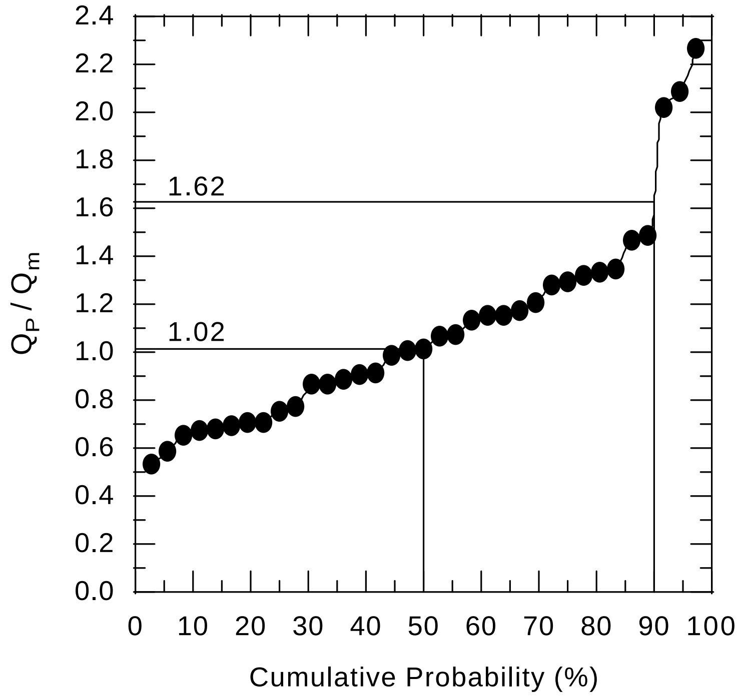



**Figure 45**
Cumulative probability plot for $Q_P$ / $Q_m$ -- $\alpha$-method

## EVALUATION OF THE CPT METHODS

Evaluating the performance of different pile capacity prediction methods is not an easy task and should not be done based on one criterion (e.g., statistical analyses). An attempt to evaluate the performance of the CPT methods based only on statistical analyses can give misleading conclusions and one must always consider the comparison plot of predicted versus measured ultimate capacity together with statistical analyses [*17*]. In this study, an evaluation scheme using four different criteria was considered in order to rank the performance of different CPT methods for predicting the ultimate axial capacity of piles. These criteria are: (1) the equation of the best fit line of predicted versus measured capacity ($Q_p/Q_m$) with the corresponding coefficient of determinations ($R^2$); (2) the arithmetic mean and standard deviation for $Q_p/Q_m$; (3) $Q_p/Q_m$ at 50 and 90 percent cumulative probability ($P_{50}$ and $P_{90}$); and (4) the 20 percent accuracy level obtained from the Log Normal distribution of $Q_p/Q_m$. A rank index (*RI*) was introduced in this study to quantify the overall performance of all methods. The rank index is the sum of the ranks from the different criteria, RI= R1+R2+R3+R4. The lower the rank index RI, the better the performance of the method. The performance of the prediction methods based on the four different criteria is discussed below.

Inspection of figures 27-35 ($Q_p/Q_m$ plots) shows that de Ruiter and Beringen method has best fit equation $Q_{fit} = 1.02Q_m$ with $R^2$=0.96. This method tends to overpredict the measured pile capacity by an average of 2 percent. Therefore, de Ruiter and Beringen method ranks number one according to this criterion and is given *R1=1* (*R1* is the rank based on this criterion). The LCPC method with $Q_{fit} = 0.97Q_m$ ($R^2$=0.95) tends to underpredict the measured capacity by 3 percent and therefore ranks number 2 (*R1=2*). According to this criterion, Schmertmann, Philipponnat, Tumay and Fakhroo methods tend to overpredict the measured ultimate pile capacity, while Aoki and De Alencar, Price and Wardle, penpile, and ″-method tend to underpredict the measured ultimate pile capacity. The penpile method showed the worse performance with $Q_{fit} = 0.57Q_m$ ($R^2$=0.95) and therefore was given *R1*=9.

In the second criterion, the arithmetic mean ( : ) and standard deviation (F) of the ratio $Q_p/Q_m$ values for each method were calculated. The best method is the one that gives a mean value closer to one with a lower standard deviation, which is the measure of scatter in the data around the mean. According to this criterion, de Ruiter and Beringen method with :($Q_p/Q_m$)=0.982 and F($Q_p/Q_m$)=0.25 ranks number one (*R2*=1) followed by the LCPC method (*R2*=2). Schmertmann, Philipponnat, Tumay and Fakhroo and "-method have :($Q_p/Q_m$)>1, which means that these methods on average are overpredicting the measured pile capacity. On the other hand, Aoki and

De Alencar, Price and Wardle, and the penpile method have $\mu(Q_p/Q_m)<1$, which means that these methods on average are underpredicting the measured pile capacity.

The cumulative probability curves (figures 37 to 45) were used to determine the 50 percent and 90 percent cumulative probability values ($P_{50}$ and $P_{90}$). The pile capacity prediction method with $P_{50}$ value closer to one and with lower $P_{50}$ - $P_{90}$ range is considered the best. Based on this criterion, the LCPC method with $P_{50}$=1.03 and $P_{90}$=1.42 ranks number one (*R3*=1) followed by de Ruiter and Beringen with *R3*=2. The penpile method has worst $P_{50}$ and $P_{90}$ values and therefore ranks number nine.

The Log Normal distribution provided the fourth criterion needed to rank the different methods based on their prediction performance. Using the Log Normal probability function, the probability of predicting the ultimate load carrying capacity at different accuracy levels was determined and plotted in figure 46. At a specified accuracy level, the higher the probability is the better the performance of the method. The 20 percent accuracy level means that the predicted pile capacity ($Q_p$) is within the range from 0.8 to 1.2 $Q_m$. The probability corresponding to 20 percent accuracy level is the likelihood that the predicted pile capacity will be within $0.8Q_m \leq Q_p \leq 1.2Q_m$. Based on the 20 percent accuracy level, Bustamante and Gianeselli (LCPC/LCP) method showed the highest probability value of 57.4 percent and therefore ranks number one (*R4*=1). de Ruiter and Beringen ranks number two (*R4*=2). The penpile method has the lowest probability value at this accuracy level and therefore ranks number nine (*R4*=9).

In order to evaluate the overall performance of the different prediction methods, all criteria were considered in a form of an index. The Rank Index (*RI*) is the algebraic sum of the ranks obtained using the four criteria. Considering de Ruiter and Beringen method, the *RI* equals to six as evaluated from *RI=R1+R2+R3+R4.* The Rank Index values for all other methods are presented in table 13. Inspection of table 13 demonstrates that de Ruiter and Beringen method ranks number one along with Bustamante and Gianeselli (LCPC/LCP) method. These two methods showed the best performance according to the evaluation criteria and therefore considered the best methods. The static $\beta$-method ranks number three. The penpile method showed the worst performance among all methods as it ranks number nine.

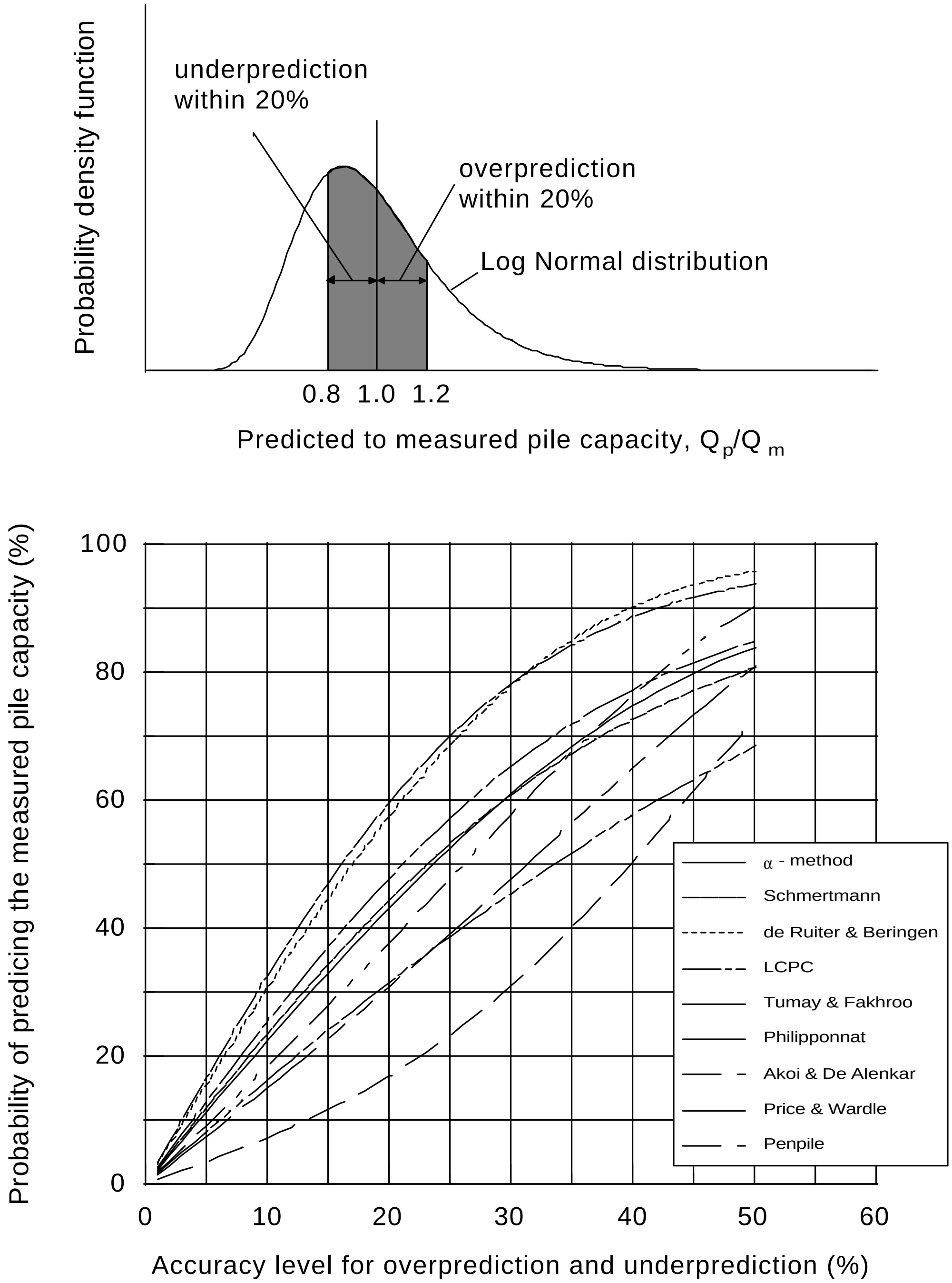



**Figure 46**

Comparison of the different prediction methods in terms of prediction accuarcy

**Table 13**

Evaluation of the performance of the different prediction methods considered in this study

| Pile Capacity Prediction Method | Best fit calculations | | | Arithmetic calculations $Q_p/Q_m$ | | | Cumulative Probability calculations | | | Log Normal distribution calculations | | Rank of the methods based on their performance | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $Q_{fit}/Q_m$ | $R^2$ | Rank R1 | Mean | Standard Deviation | Rank R2 | $Q_p/Q_m$ at $P_{50}$ | $Q_p/Q_m$ at $P_{90}$ | Rank R3 | 20% Accuracy level | Rank R4 | Rank Index RI | Rank |
| de-Ruiter& Beringen | 1.02 | 0.96 | 1 | 0.982 | 0.250 | 1 | 0.93 | 1.31 | 2 | 57.4 | 2 | 6 | 1 |
| LCPC | 0.97 | 0.95 | 2 | 1.072 | 0.260 | 2 | 1.03 | 1.42 | 1 | 59.5 | 1 | 6 | 1 |
| "-method | 0.89 | 0.91 | 4 | 1.089 | 0.414 | 3 | 1.02 | 1.62 | 3 | 43.1 | 5 | 15 | 3 |
| Philipponnat | 1.23 | 0.95 | 8 | 1.152 | 0.352 | 4 | 1.17 | 1.78 | 4 | 47.7 | 3 | 19 | 4 |
| Schmertmann | 1.16 | 0.96 | 5 | 1.207 | 0.361 | 6 | 1.19 | 1.7 | 5 | 44.2 | 4 | 20 | 5 |
| Aoki & De Alencar | 0.89 | 0.94 | 3 | 0.810 | 0.267 | 5 | 0.74 | 1.13 | 6 | 37.6 | 6 | 20 | 5 |
| Price & Wardle | 0.82 | 0.92 | 7 | 0.782 | 0.320 | 7 | 0.72 | 1.21 | 7 | 30.7 | 8 | 29 | 7 |
| Tumay & Fakhroo | 1.18 | 0.96 | 6 | 1.374 | 0.343 | 9 | 1.39 | 1.8 | 8 | 31.4 | 7 | 30 | 8 |
| Penpile | 0.57 | 0.95 | 9 | 0.633 | 0.218 | 8 | 0.59 | 0.87 | 9 | 16.8 | 9 | 35 | 9 |

Rank Index, RI=R1+R2+R3+R4

$R^2$: Coefficient of determination

$P_{50}$: Cumulative probability at 50%

$P_{90}$: Cumulative probability at 90%

## COST AND BENEFIT ANALYSIS

DOTD maintained a *traditional* practice in pile design and analysis, which is basically a static analysis using "-method. Properties of cohesive soils are obtained from laboratory tests on undisturbed samples from boreholes, while strength characteristics of cohesionless soils are evaluated from Standard Penetration Test (SPT). Conducting field and laboratory tests is expensive and time consuming. The average cost of a traditional soil boring and the corresponding laboratory and field tests, as reported by the DOTD Materials Section, is $50/ft when conducted by DOT and $60/ft when carried out by a consultant.

Due to the uncertainties associated with pile design, load tests are usually conducted to verify the design loads and to evaluate the actual response of the pile under loading. DOTD practice is to conduct pile load tests based on cost/benefit evaluation. In small bridge projects, it is often cost effective to increase the factor of safety (increase pile length) compared to conducting pile load tests. Pile load tests are also expensive. The cost of driving and loading a test pile in Louisiana ranges from $13,000 to $25,000.

Implementation of the CPT technology by DOTD has been limited to identification of dense sand layers required to support the tip of the end-bearing piles. The CPT technology is fast, reliable, and cost effective especially when compared to the traditional site characterization methods (borings and laboratory/field tests). The average cost of CPT soundings is $14/ft when the system is operated by DOTD Materials Section and $28/ft when the CPT is conducted by a consultant. Compared to the traditional borings, the CPT is faster and more economical.

In order to demonstrate the cost effectiveness of using the CPT in design and analysis of piles, the following are used to compare the cost of using CPT to the cost of using traditional methods:

1. Comparison of cost between the CPT and traditional exploration methods
2. Comparison of savings in pile length in projects with pile load tests

### CPT versus Traditional Subsurface Exploration Methods

This case compares the direct cost of conducting a CPT test versus a traditional soil boring with no pile load test being carried out at the project. This scenario usually happens in small-bridge projects where conducting pile load tests is not cost effective due to the small number of piles. Traditionally, one or two soil borings are usually taken for this kind of projects. In small-bridge projects, the CPT is assumed to replace the soil borings on one to one basis (i.e. one CPT replaces one soil boring). In this case, the average cost of conducting CPT is $14/ft, which is far less than $50/ft, the average cost of a traditional soil boring and the corresponding laboratory and field tests. The cost of a 100 ft CPT test is $1,400 while the cost of a 100 ft traditional soil boring and the corresponding tests is $5,000. Replacing the traditional soil boring by the CPT will result in a total saving of $3,600 or 72 percent for small-bridge projects.

The cost ratio of CPT and soil borings is 1 to 3.6 (i.e. the cost of CPT and soil borings are equal when 3.6 CPT tests replace one traditional soil boring). When the CPT is assumed to replace soil borings on two to one basis, then the cost of CPT will increase and the cost ratio of CPT and soil boring will be 1 to 1.8. This scenario shows that the CPT direct cost is still 44 percent less than the cost of traditional soil borings.  There will also be savings in the pile length since more CPT tests will result in more accurate pile design, which will reduce the cost of piling.

**Pile Length Savings in Projects with Pile Load Test**

The use CPT in design and analysis of piles will evidently lead to reduction in pile lengths without compromising the safety and the performance of the supported structure. First, the CPT can be used to quickly identify the weakest location at a project. This spot will be selected as the location of the pile load test, which will provide the design engineer with lowest possible load carrying capacity of the pile. The pile load test can also be used to modify the pile design using CPT methods the same way it is being used by DOTD to modify the design using the static method, i.e. applying the shift. It will also reduce the number of pile load tests conducted at a project, as an example about one third of the pile load tests considered in the analysis herein did not fail under load testing. Using the CPT, the weakest spot can be identified and selected for one pile load test for design verification.

Second, the CPT methods are proven herein to be more accurate in predicting the pile load carrying capacity of PPC driven pile in Louisiana soils compared to the currently used static method. An accurate design leads to less uncertainty and therefore less factor of safety. Finally, the CPT will reduce the number of  soil borings, which will result in monetary and timesaving.

# CONCLUSIONS

This study presented an evaluation of the performance of eight CPT methods in predicting the ultimate load carrying capacity of square precast prestressed concrete piles driven into Louisiana soils. Sixty pile load test reports, which have CPT soundings adjacent to the test pile, were collected from DOTD files. Prediction of pile capacity was performed on sixty piles; however, the statistical analyses and evaluation of the prediction methods were based on the results of thirty-five friction piles plunged (failed) during the pile load tests. End-bearing piles and piles that did not fail during the load tests were excluded from the statistical analyses.

The pile load test data were analyzed to obtain the measured ultimate load carrying capacity for each pile. Butler-Hoy method, the primary method used by DOTD, was used to determine the measured ultimate load carrying capacity for each test pile. The following CPT methods were used to predict the load carrying capacity of the collected piles using the CPT data: Schmertmann, Bustamante and Gianeselli (LCPC/LCP), de Ruiter and Beringen, Tumay and Fakhroo, Price and Wardle, Philipponnat, Aoki and De Alencar, and the penpile method. The ultimate load carrying capacity for each test pile was also predicted using the static "-method, which is used by DOTD engineers for pile design and analysis.

An evaluation scheme was executed to evaluate the CPT methods based on their ability to predict the measured ultimate pile capacity. Four different criteria were selected for the evaluation scheme: the line of best fit between the measured and predicted capacities, the arithmetic mean and standard deviation of the ratio of predicted to measured capacity ($Q_p/Q_m$), the cumulative probability of measured and predicted capacities, and the Log Normal distribution of the ratio of predicted to measured capacity. Each criterion was used to rank the prediction methods based on its performance. The final rank for each method was obtained by averaging the ranks of the method from the four criteria.

Based on the results of this study, de Ruiter and Beringen and Bustamante and Gianeselli (LCPC/LCP) methods showed the best capability in predicting the measured load carrying capacity of square PPC piles driven into Louisiana soils. These two CPT methods showed a better performance than the currently used "-method. Cost/benefit analysis showed that using the CPT methods for design/analysis of square PPC piles would cut the cost of initial design as well as the cost of piling.

The CPT methods that showed the best performance were implemented into a Visual Basic computer program to facilitate their use by DOTD design engineers. These methods are de Ruiter and Beringen, and LCPC/LCP. Schmertmann method was also implemented in the program since it is one of widely used CPT methods.

# RECOMMENDATIONS

The results of this study demonstrated the capability of CPT methods in predicting the ultimate load carrying capacity of square PPC piles driven into Louisiana soils. de Ruiter and Beringen and Bustamante and Gianeselli (LCPC/LCP) methods showed the best performance in predicting the ultimate measured load carrying capacity of square PPC piles. It is strongly recommended that DOTD implements these two methods in design and analysis of square PPC piles. Schmertmann method also showed good results and is recommended for implementation since it is one of the most widely used CPT methods.

Cost-benefit analysis showed that the implementation would result in cost reduction in pile projects and timesaving without compromising the safety and performance of the pile supported structures. In fact, implementation of the CPT technology in pile design will reduce the level of uncertainties associated with traditional design methods.

In order to facilitate the implementation process, a computer program, Louisiana Pile Design by Cone Penetration Test (LPD-CPT), was developed for design/analysis of square PPC driven piles from CPT data. The program, which is based on MS-Windows environment, is easy to use and provides the profile of the pile load carrying capacity with depth.

Based on the results of the analyses, it is recommended to implement the cone penetration technology in different geotechnical applications within the DOTD practice. Regarding design and analysis of driven piles, the following are recommended:

1. Foster the confidence of the DOTD design engineers in the CPT technology by adding the CPT to the list of the primary variables in subsurface exploration and use it in soil identification and classification, and in site stratigraphy. Different soil classification methods can be used such as Zhang and Tumay, Robertson and Campanella, and Olsen and Mitchell.

2. Compare the test results from the traditional subsurface exploration methods and the results interpreted from the CPT methods. With time and experience, reduce the dependency level on the traditional subsurface exploration methods and increase dependency level on the CPT technology.

3. Use the CPT pile design methods in conjunction with the pile load tests and the static "-method to predict the load carrying capacity of the square PPC piles. The following CPT methods are recommended: de Ruiter and Beringen method, Bustamante and Gianeselli (LCPC/LCP) method, and Schmertmann method. If a pile load test is conducted for the site, compare the results of the CPT methods with the measured ultimate pile load capacity. If the measured and predicted capacities are different, then make a correction to the predicted capacity in the amount of the difference between the measured and predicted capacity. Apply this correction to the other for the design of piles at this site.

4. Increase the role of the CPT design method and decrease the dependency on the static "-method.

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# APPENDIX

## Louisiana Pile Design by CPT

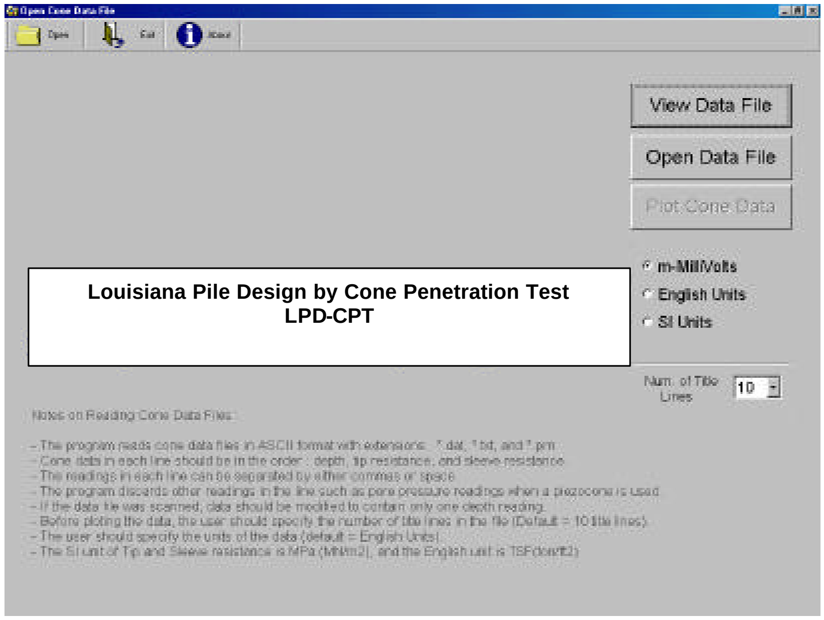


**Figure 47**
The main menu of the Computer Program Louisiana Pile Design by Cone Penetration Test (LPD-CPT) developed in the current study.

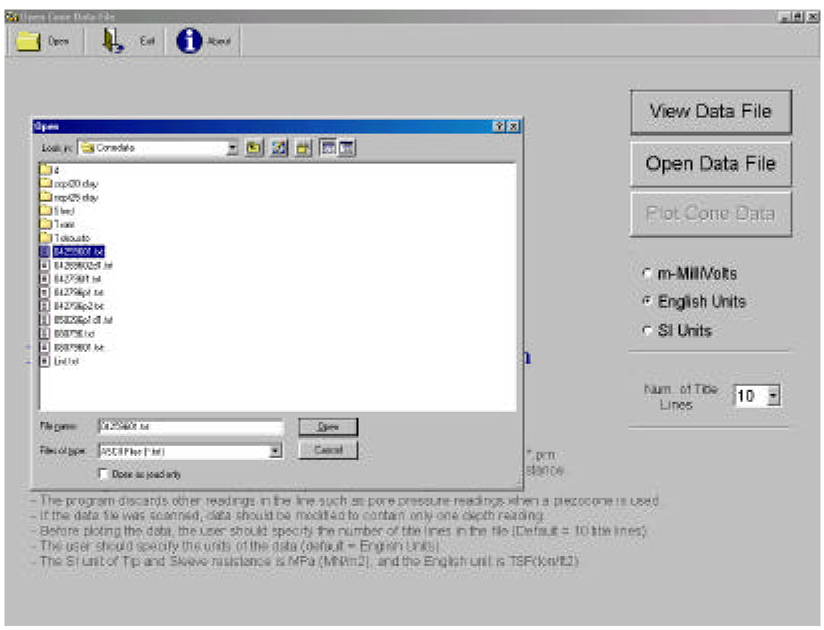


**Figure 48**
The data file menu of program LPD-CPT that allows the user to view, open, and then plot a CPT data.

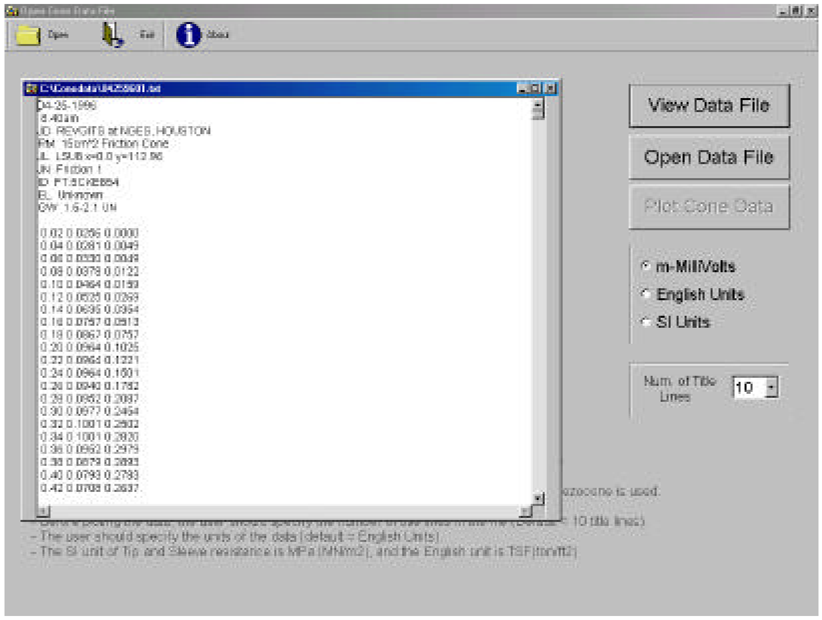


**Figure 49**
CPT data file viewed using the View Data File menu.

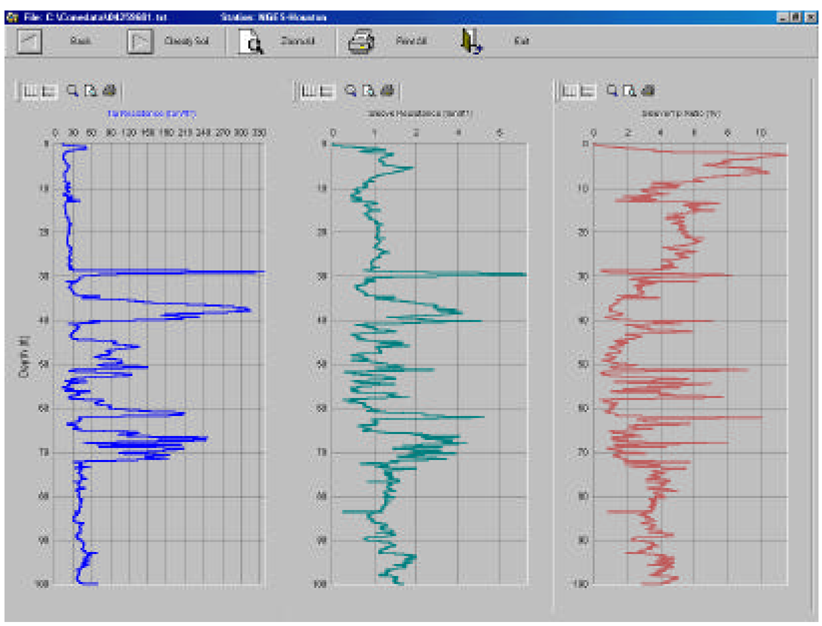

**Figure 50**
Plot of the CPT data file obtained using the Plot Cone Data menu.

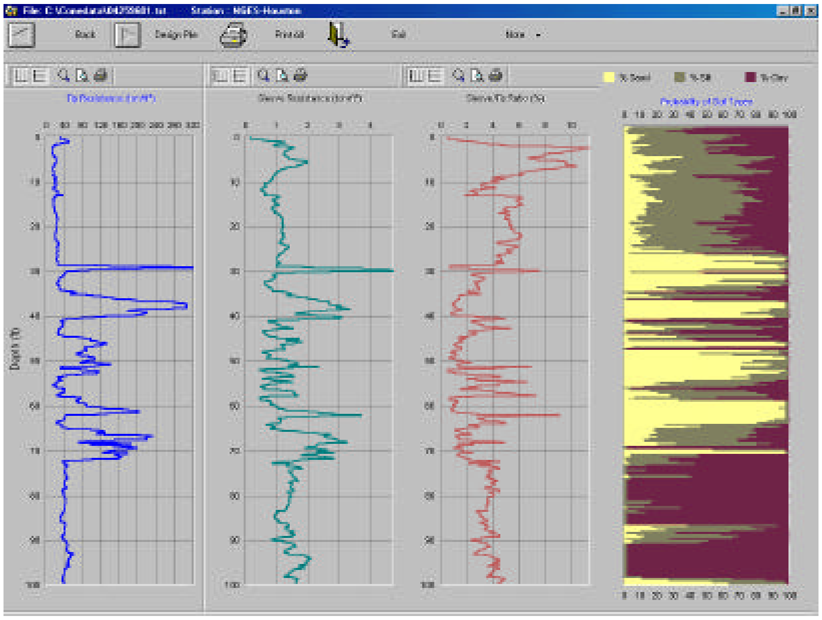

**Figure 51**
Probabilistic Soil classification obtained from CPT data.

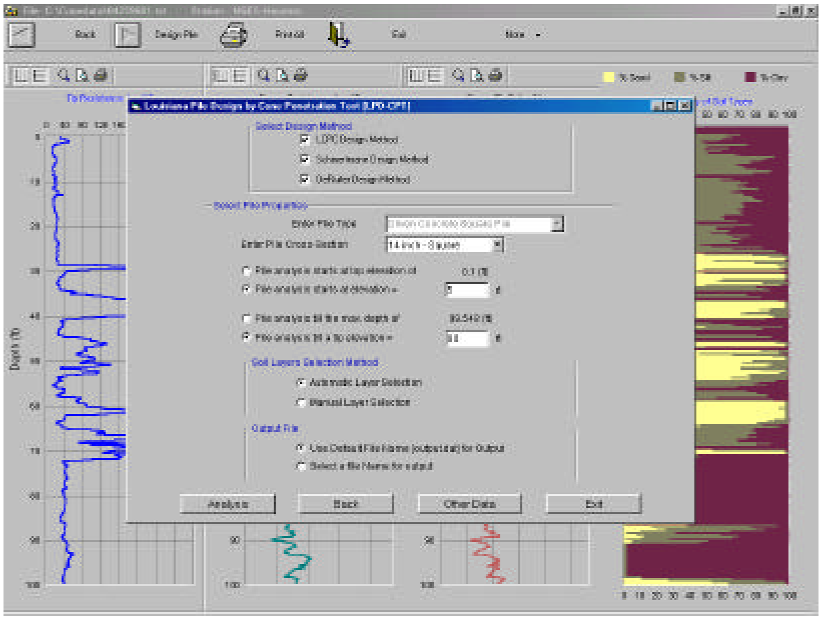

**Figure 52**
Pile Design Menu of the program LPD-CPT.

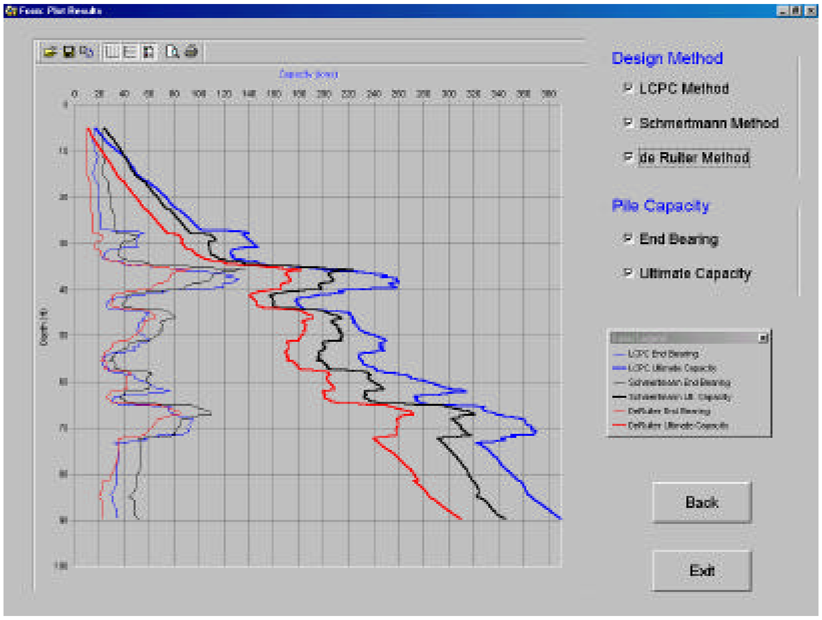


**Figure 53**
Variation of ultimate load carrying capacity of the
pile with depth using three different design methods.

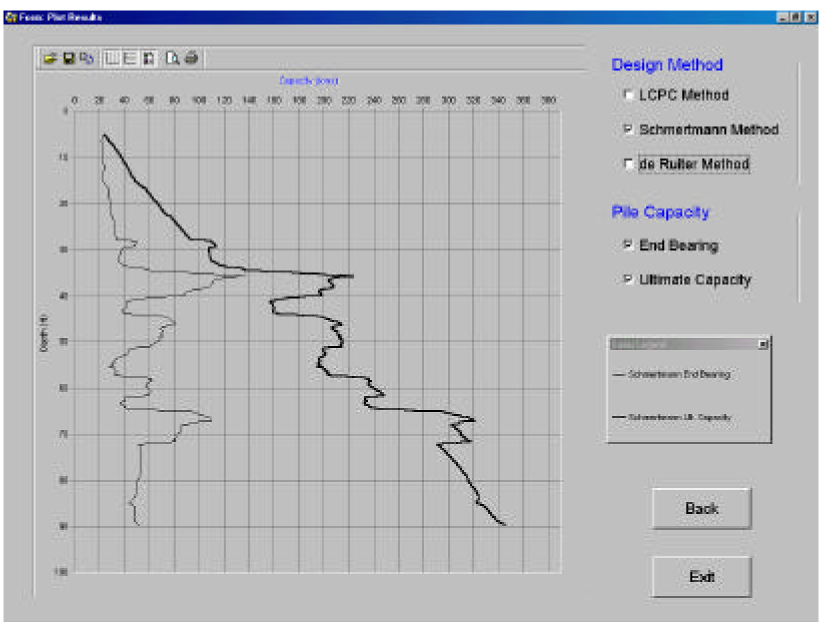


**Figure 54**
Variation of the ultimate load carrying capacity of the pile using Schmertmann method.