load (point resistance) ($Q_b$) and side friction load (shaft / skin friction) ($Q_s$). It is advisable to ignore any side friction capacity for helical piles especially with small diameter shafts similar to those assumed here (i.e. 1 3/4″ square shaft). Therefore, total pile axial capacity is:

$$Q_{Total} = Q_b = \left(q_p A_p\right) \ / \ FS_b$$

in which $q_p$ = the unit end bearing, $A_p$ = the pile end area, $FS_b$ = the factor of safety for the end bearing load.

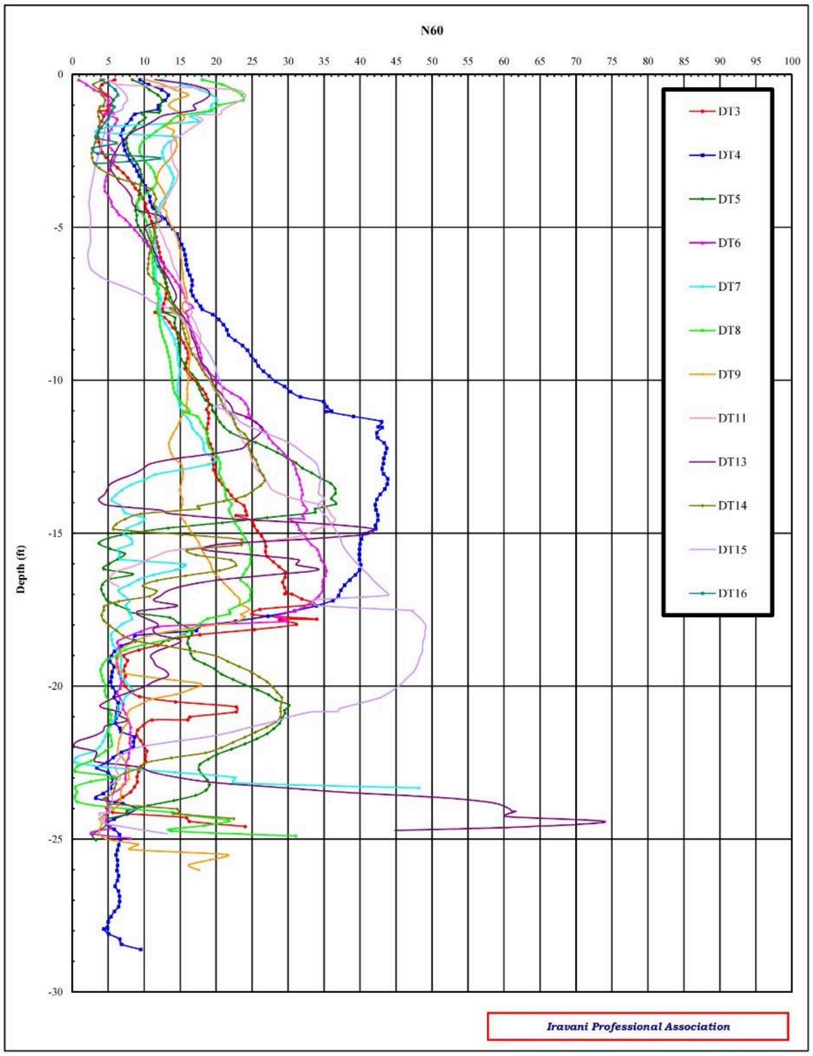


**Figure 16 – Variation of Equivalent SPT N60 from CPT with depth**

Following assumptions have been made:

- Helical piles will be driven to approximately 25 ft (or deeper) below existing grades.
- From bottom to top, Lead Section C150-0402 (with three helix), Extension Section C150-0186 (with two helix), and Extension Section C150-0185 (with one helix) will be used.
- If needed, use plain Extension C150-0184 to bring total length to required grades.
- Assume average undrained shear strength of 1,500 psf for lower fine grain soils and drained friction angle of 35 degrees for the upper coarse grain soils.
- Assume an estimated soil density of 110 pcf.

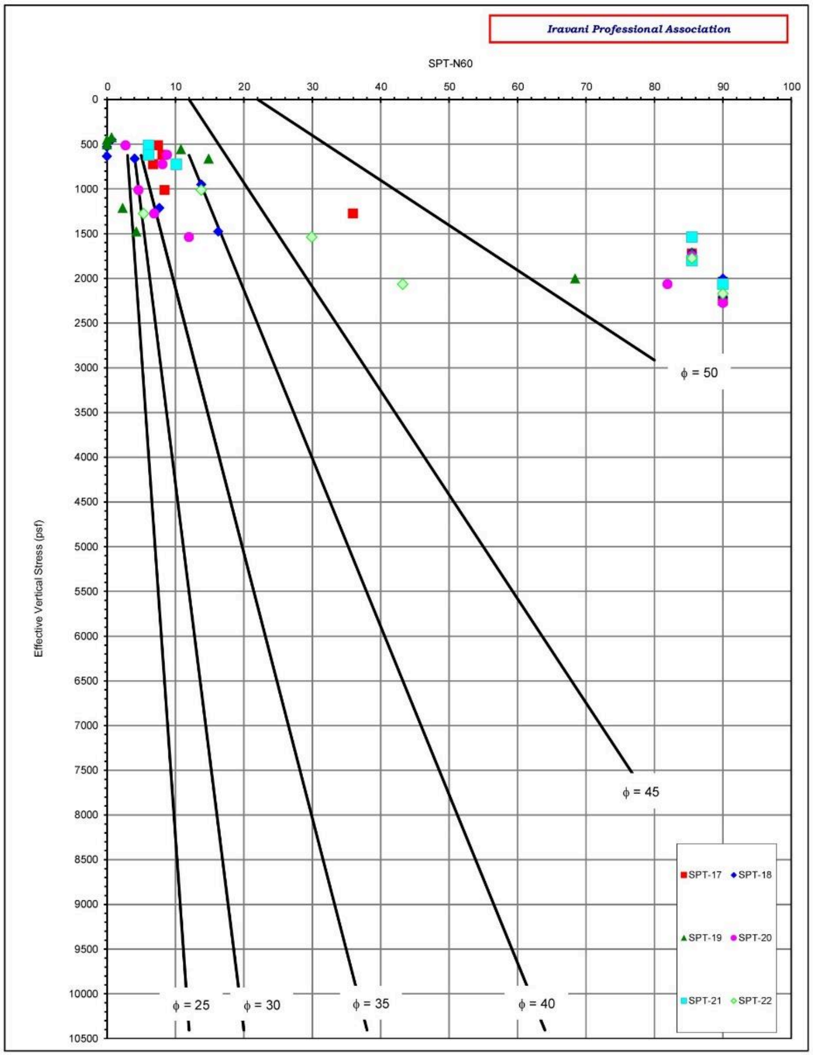


**Figure 17 – Estimation of peak drained friction angle from SPT N60**

It is projected that the helical piles have an estimated compressive capacity of approximately 30 tons (29.5 +/-). If Extension Section C150-0185 (with one helix) is not included in the assembly, total capacity in compression will be 26.5 tons. Helical piles shall be at least 3.5 ft apart (center to center) to minimize soil disturbance from adjacent helical piles and reduction in capacity in a pile group. Driving torque at refusal shall be 6,000 lbs-ft or more.

It is projected that the helical piles have an estimated capacity in tension of approximately 22 tons. If Extension Section C150-0185 (with one helix) is not included in the assembly, total capacity in tension will be 19.5 tons.

Lateral capacity of helical piles is estimated using L-pile (Version 2012). Reductions recommended by McVay (1995) were used to reflect the reduced lateral resistance of rows of piles behind the lead row. Evaluation showed that helical piles with 1 3/4″ square shaft have very low lateral capacity (less than 1 to 2 kips) but if they are battered, they could resist lateral loads. A 2V:1H batter was recommended which could provide at least 44 kips capacity in compression and 22 kips capacity against lateral loads for the proposed design.

Considering that the objective of this geotechnical investigation was optimizing the foundation capacity, it is noteworthy that some additional lateral capacity could be gained from the pile cap sides and grade beams. It might not be advisable to rely on such additional capacity because the friction at the interface of the pile cap / grade beam base and underlying soil may not be there in the long run due to soil settlement. Furthermore, full passive resistance might not be mobilized due to limited lateral deflection. That been said, assuming the ground water table at the surface, an effective soil density of 52 pcf, a strain-reduced Kp, and an equivalent fluid pressure of 80 pcf , for a 4 ft thick and 10 ft wide pile cap, an additional reserved lateral capacity of 6.4 kips might be available.

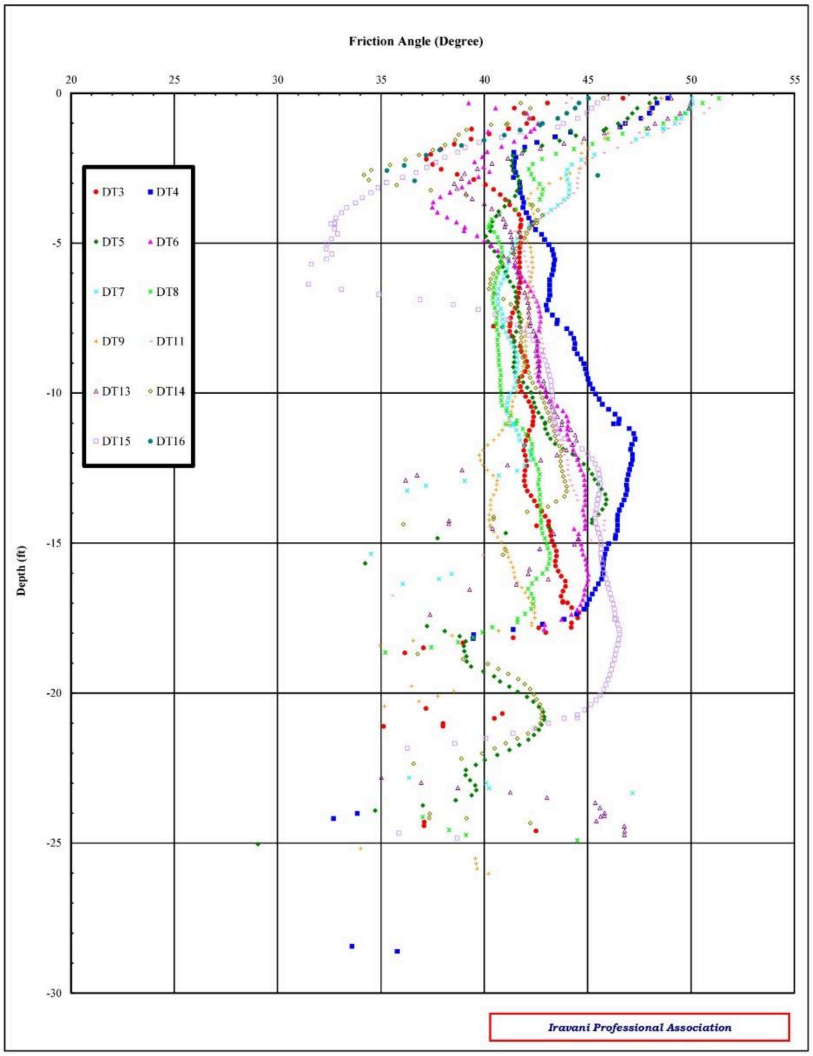


**Figure 18 – Estimation of friction angle from CPT**

## OTHER GEOTECHNICAL CONSIDERATIONS

The shallow subsurface soils in the area of CPT DT15 & DT16 and SPT-18 & SPT-19 were discovered to be very loose. While there was no good record about the design and construction of the building, further investigation showed there was a concentration of buried utilities in this area. Either there was leakage and erosion in this area during the life of the structure or the shallow surficial soils in this area (backfill) were not properly compacted during the original construction (due to congestion of utilities). It was recommended to perform some injection grouting to enhance soil properties in that area (i.e. near surrounding piles).

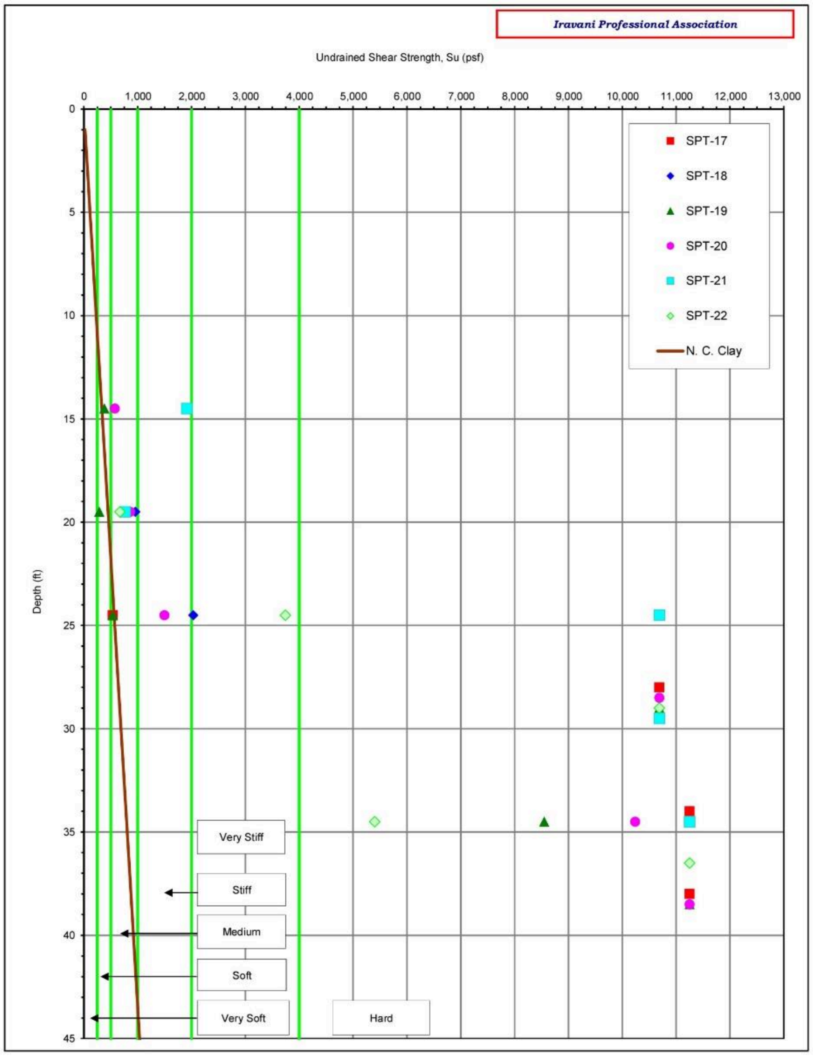


**Figure 19 – Estimation of undrained sheer strength from SPT N60**

Modulus of subgrade reaction (ks) for shallow sandy soils is approximately 60 kcf. During the foundation retrofitting, contractor needed to transfer some column loads to heavily concentrated collection of temporary shores. It was concluded that the subgrade of the ground floor slab was not strong enough. Limited pressure grouting to densify the subgrade soils and enhance the geotechnical engineering properties of the subgrade soils was recommended.

The foundation retrofitting / enhancement construction was carried out using a value engineering contract. The contractor optimized the design using alternative helical piles with larger helix, designed and manufactured by Magnum Piering, Inc. Helical piles with a 4.5″ shaft and four large dimeter (14 and 16 inches) helixes were used. While the space was confined and restrained, the small size equipment used provided adequate torque to install the helical piles to refusal (Figure 22). Final configuration of existing and new shear walls plus new helical piles, after optimization by contractor using value engineering, is shown the site plan (Figure 11).

## CONCLUSION

A geotechnical site investigation consisting of SPT, CPT, and limited laboratory index tests was performed with the objective of subsurface soil exploration, evaluation of geotechnical engineering properties of the subsurface soils and rock, analysis of the existing 14″ x 14″ driven precast concrete piles, and design of the new helical piles.

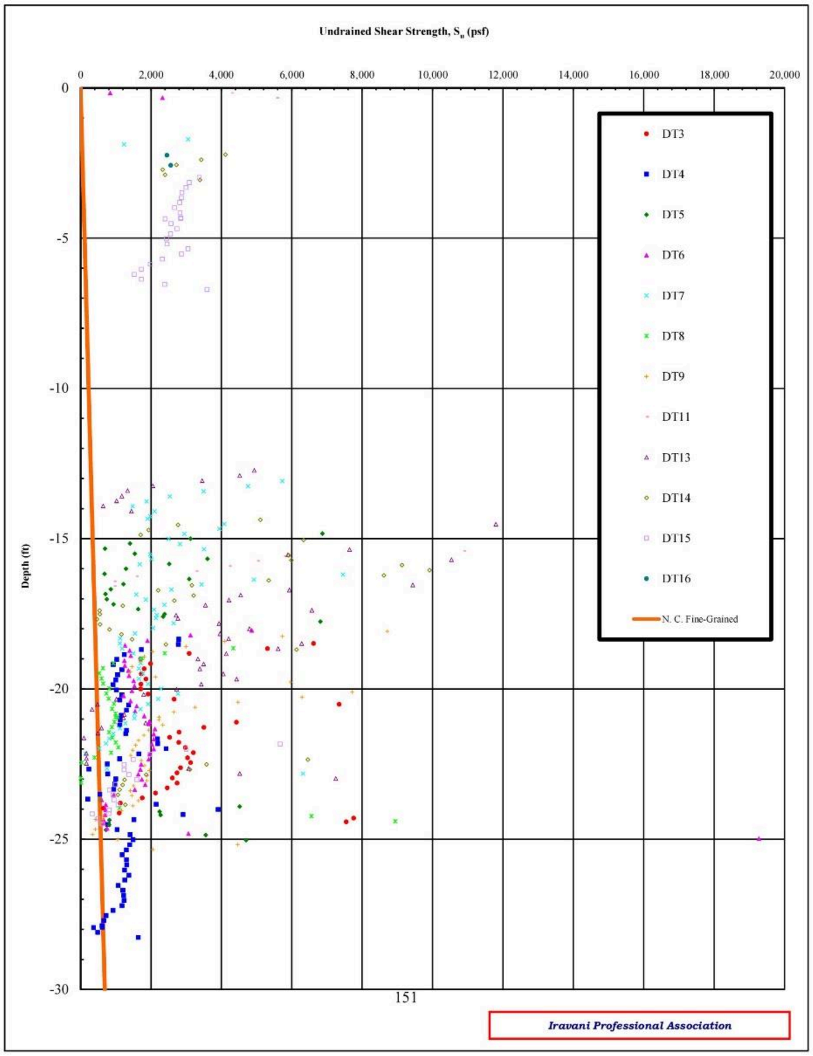


**Figure 20 – Estimation of undrained sheer strength from CPT**

Subsurface stratigraphy was characterized. From top to bottom, typically, it consisted of surficial coarse grain soils, fine grain soils, and weathered limestone / cemented fine grain soils / limestone. Detailed geotechnical engineering properties of the subsurface soils were evaluated using the in-situ tests in conjunction with empirical relationships.

Based on data obtained from CPT and SPT tests, it was reasonably concluded that original 14″ x 14″ precast concrete piles were driven to approximately 25 ft below ground surface (on average). It is noteworthy that the refusal depth of the new helical piles further confirmed this assessment. Except a handful, new helical piles encountered refusal at depths projected by CPT sounding and SPT tests.

The pile end bearing and cumulative pile side frictional capacity were estimated for the existing 14″ x 14″ driven precast concrete piles. It was found that the referenced piles had approximately 62 tons and 27 tons capacity in compression and tension, respectively. It was estimated that an individual 14″ x 14″ pile had approximately 15 tons lateral capacity. Range of lateral deflections for variety of loading conditions and the reduction in lateral capacity in variety of pile groups were also assessed and presented.

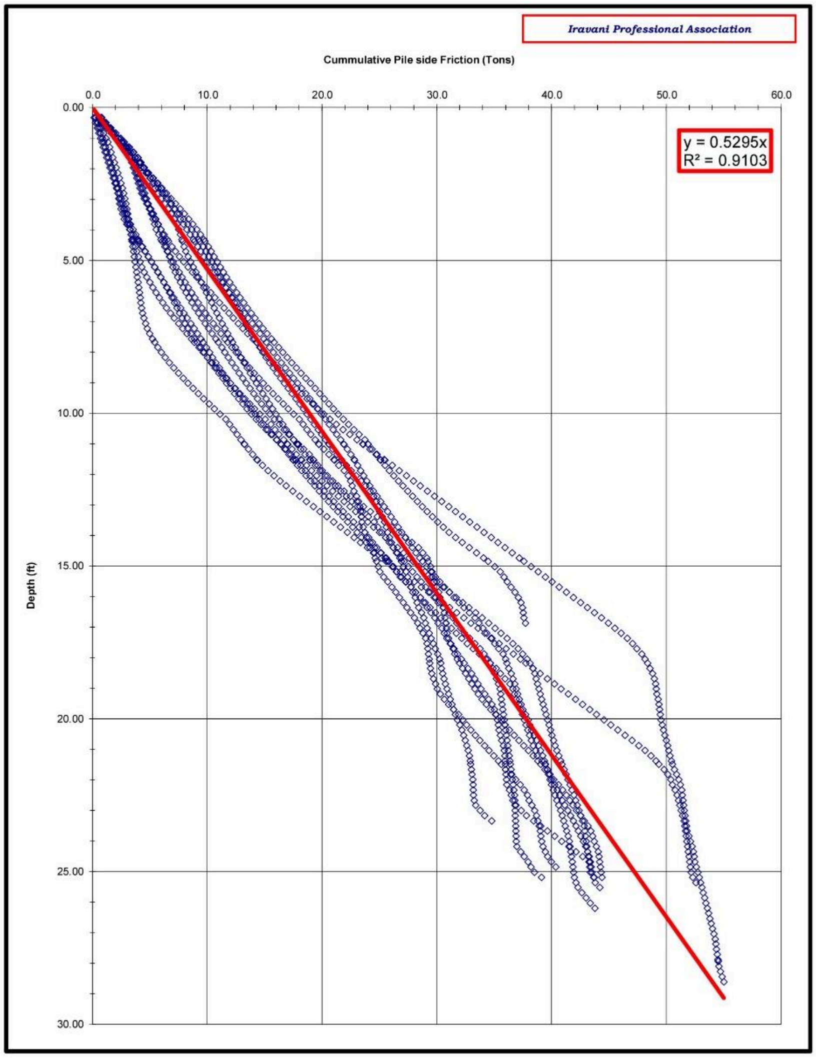


**Figure 21 – Cumulative pile side frictional capacity with length for 14″ x 14″ precast driven concrete piles**

The capacity of new helical piles were also estimated. It was projected that the referenced helical piles had approximately 26.5 to 29.5 tons and 19.5 to 22 tons capacity in compression and tension, respectively. It was found that if helical piles were not installed in battered configuration, their contribution to lateral support would be insignificant but a 2V:1H battered configuration would provide at least 22 kips lateral capacity at conditions present.

This geotechnical investigation not only addressed the original objectives but it detected a shallow weak soil zone in an area with heavy concentration of existing piles and utilities which would be central to retrofitting the structure with new shear walls. Data obtained was also used for improving the subgrade as foundation for temporary shoring and columns.

Bringing the structure to compliance with the current Florida Building code resulted in addition of five new shear walls and significant number of new helical piles.

**Figure 22 – Installation of helical pile**

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# Evaluation of Direct Pile-CPT Methods for Estimating the Ultimate Capacity of Driven Piles

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## ABSTRACT

The cone and piezocone penetration tests (CPT and CPTu) have been widely acknowledged as very useful in situ tests for subsurface investigation and soil characterization. The CPT is fast, robust, and economical test that can provide continuous soundings with depth. Due to similarity between pile and cone, the estimation of pile capacity from CPT data was among the earliest applications of the CPT. Accordingly, different direct pile-CPT methods have been developed based on this analogy between the cone and the pile. Analyses and evaluation were conducted on eighty driven friction piles of different sizes and lengths that were failed during pile load testing. The pile-CPT methods were used to estimate the load carrying capacities of the investigated piles ($Q_p$). The Davisson method was used to determine the measured load carrying capacities from pile load tests ($Q_m$). Four criteria were selected for evaluating the different pile-CPT methods: the best fit line for $Q_p$ versus $Q_m$, the arithmetic mean and standard deviation for the ratio $Q_p/Q_m$, the cumulative probability for $Q_p/Q_m$, and the histogram and log normal distribution for $Q_p/Q_m$. Results of the analyses have been used to evaluate the ability of different pile-CPT methods for estimating the pile capacity.

## INTRODUCTION

With the increase in traffic volume due to the rapid economic development, more and more bridges have been built across rivers and canals. Louisiana is not an exception to this role since it is developing of a rapid speed. The high percentage of wetlands, marshes, swamps, bayous, rivers, and lakes makes it necessary to construct pile supported structure in Louisiana. Most piles used for highway structures are driven precast prestressed concrete piles.

The ultimate capacity of driven piles, which can be defined as the sum of soil resistance along the pile's side and the pile's tip resistance, can be estimated using pile load tests, dynamic analyses, Statnamic load tests, and static analysis based on soil properties from laboratory tests or in-situ tests. A number of static analysis methods were developed over the years to predict the pile capacity from in-situ tests such as cone penetration test (CPT).

The direct Pile-CPT methods are based on analogy between cone and pile. Many methods have been developed in recent years which address different aspects of the pile-cone analogy. A description of Pile-CPT methods used in this research is available in literature such as (Titi and Abu-Farsakh 1999), (Bloomquist et al. 2007), (Niazi and Mayne 2013), and (Niazi 2014).

Different researchers have studied the ability of Pile-CPT methods for estimating the pile capacity. Briaud and Tucker (1988) evaluated the accuracy and precision of 6 different Pile-CPT methods using 98 pile load test database obtained from Mississippi State Highway Department. He stated that the accuracy of a Pile-CPT method is determined by means of $Q_p / Q_m$ being close to 1, and the precision of a method refers to the scatter around the mean, quantified by standard deviation of $Q_p / Q_m$. He used the log-normal distribution of $Q_p / Q_m$ for introducing a ranking index, RI, as follows:

$$RI = |\mu(a)| + \sigma(a) \quad (1)$$

where a is equal to $\ln(Q_p / Q_m)$ and $\mu$ and $\sigma$ are mean and standard deviation, respectively. The advantage of using log-normal distribution is that the overprediction leads to lower RI value compared to under-predicting methods. The better performance of a method was defined by the lower value for RI which showed that LCPC method (Bustamante and Gianeselli (1982)) was the best direct Pile-CPT method.

A research study by Abu-Farsakh and Titi (2004) sponsored by Louisiana Department of Transportation and Development (LA DOTD) investigated eight different Pile-CPT methods. Four evaluation criteria for evaluating the prediction methods was adopted. The overall performance of the Pile-CPT methods was evaluated based on summing up the ranking of methods for the four different criteria. Based on this analysis, LCPC (Bustamante and Gianeselli 1982) and De Ruiter (De Kuiter and Beringen 1979) methods showed the best performances. This criterion has been used by (Eslami et al. 2011), (Eslami et al. 2014), and (Moshfeghi and Eslami 2018) for evaluating different Pile-CPT methods. (Hu et al. 2012) used first-order second-moment (FOSM) resistance factor equation introduced by (Paikowsky 2002) with correction for coefficient of variation of load by (Styler 2006) in Load and Resistance factor Design (LRFD) equations for evaluating 14 different Pile-CPT prediction methods. Eq. (2) is obtained by substituting $\lambda_R = R_m / R_n$ in $\varphi = R_{design} / R_n$, where nominal resistance, $R_n$ equals to side resistance plus 1/3 of the tip resistance; measured resistance, $R_m$ is the failure load, and $R_{design}$, is the predicted capacity of the corresponding Pile-CPT method.

$$R_{design} = \left( \varphi / \lambda_R \right) R_m \quad (2)$$

The higher values for $(\varphi / \lambda_R)$ the better the estimation method is. The study on 21 piles in Florida and 28 from Louisiana showed that LCPC (Bustamante and Gianeselli 1982) and Philipponnat (Philipponnat 1980) methods yield higher $\varphi / \lambda_R$ values than the other Pile-CPT methods.

During the past two decades other CPT methods for estimating pile capacity have been developed. In order to take advantage of these new developments, it is necessary to evaluate the ability of these methods. In this research, 18 Pile-CPT methods were evaluated using a total of 80 pile load test cases with CPT tests performed within close proximity in Louisiana. Soil type is one of the parameters which most of the methods estimations are dependent on it. In this research, probabilistic (Zhang and Tumay 1999) and Robertson (2010) (Robertson 2010) soil classifications have been used for determining the soil type. Based on CPT data, probabilistic method determines the probability of soil behavior (clay, sand, and silt), while Robertson (2010) presents a chart dividing the soil behavior into 9 different soil types. Comparison between pile capacity values obtained using each of these two soil classification methods was discussed. For this purpose, statistical analysis was used to examine if there was a significant difference using any of these soil classification methods for each Pile-CPT method. The evaluation criteria used by Abu-Farsakh and Titi (2004) were used to evaluate these 18 Pile-CPT methods.

## RESEARCH METHODOLOGY

**Characteristics of the investigated piles:** Results from 80 PPC piles in Louisiana have been used to assess the ability of different Pile-CPT methods to predict the pile capacity. The information of the piles such as pile length, pile diameter, CPT tests, and static load tests have been collected from Louisiana Department of Transportation and Development (LA DOTD). The piles' lengths range from about 11 to 61 m (35–200 ft) and the diameters range from about 356 mm to 914 mm (14–36 inch). Also, boring data near to the pile locations has been used in DRIVEN software (using α and Nordlund methods for clays and sands, respectively) which shows that most of the pile capacity driven in Louisiana soil is due to side resistance. Only 4 piles had a tip resistance more than 50% of the total pile capacity. The proportion of pile capacity in clay layers to the total pile capacity (defined as clay contribution) has been used to characterize the dominant soil for the pile database. Based on this analysis, piles were driven into different sandy, clayey, and layered soils.

**Axial pile capacity from static load tests:** Quick Load Test procedure as described in ASTM D1143 (ASTM D 2013) were performed on different piles after 14 days of driving to obtain the load-settlement curve. The ultimate load capacity of the piles was determined based on the Davisson method (1972) . Davisson failure criterion defines pile capacity as the load causes the pile top deflection equal to the calculated elastic compression plus 3.8 mm (0.15 inch) plus 1/120 of the pile's width/diameter. For piles with diameters more than 610 mm (24 inch), based on Florida Department of Transportation FDOT specification 2010, section 455 the criterion is modified to calculated elastic compression plus 1/30 of the pile's width/diameter (Florida 2010).

**Estimating pile capacity, $Q_p$ for Pile-CPT methods:** In order to use CPT for calculating, adjacent CPT soundings have been used. For improving the quality of CPT results, the cone resistance should be corrected due to the pore pressure acting behind the cone shoulder (Campanella et al. 1981; Lunne et al. 1986), using the Eq. (3):

$$q_t = q_c + (1-a)u_2 \tag{3}$$

where a is the net area ratio for the cone (0.59 for CPT used in this research). For most of the cases (68 piles), no measured pore pressure, $u_2$ were available. A comparison between CPT and CPTu results conducted in different locations in Louisiana, led to obtain a corrected factor dependent on the measured cone resistance and its depth, as described below:

- If $q_c < 1\ MPa$: $q_t / q_c = \min(1 + 0.2 \times depth / 30, 1.2)$
- If $1 < q_c < 2.5\ MPa$ , $q_t / q_c = \min(1 + 0.15 \times (depth - 6) / 37, 1.15)$
- If $2.5 < q_c < 5\ MPa$ , $q_t / q_c = \min(1 + 0.1 \times (depth - 6) / 37, 1.1)$
- If $q_c > 5\ MPa$ , $q_t / q_c = 1$

Where depth is the measured depth (in meters) of the $q_c$ value.

Fourteen (14) different Pile-CPT methods including: LCPC (Bustamante and Gianeselli 1982), Schmertmann (Schmertmann 1978), De Ruiter (De Kuiter and Beringen 1979), Philipponnat (Philipponnat 1980), UF (Hu et al. 2012), probabilistic (Abu-Farsakh and Titi 2007), Aoki (Aoki and Velloso 1975), Penpile (Clisby et al. 1978), NGI (Karlsrud et al. 2005), ICP (Jardine et al. 2005), UWA (Lehane et al. 2005; Lehane et al. 2012), CPT2000 (Lehane et al. 2000), Fugro (Kolk and der Velde 1996; Van Dijk and Kolk 2011), and Purdue (Salgado et al. 2011) are dependent on the soil type. It means that in order to use CPT for calculating the pile