EVALUATION OF LOAD CARRYING CAPACITY OF PRECAST CONCRETE PILES BASED ON CPT

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EVALUATION OF LOAD CARRYING CAPACITY OF PRECAST CONCRETE PILES BASED ON CPT

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BY

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DEDICATION

I like to dedicate this work to my late mother who loved, raised and guided me in every single step of my life.

DECLARATION

It is hereby declared that, except where specific references are made, the work embodied in this thesis is the result of investigation carried out by the author under the supervision of Dr. Mehedi Ahmed Ansary, Professor, Department of Civil Engineering, BUET, Dhaka.

Neither this thesis nor any part of it is concurrently submitted to any other institution in candidature for any degree.

_____

Md. Mominur Rahman

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ABSTRACT

This thesis presents an evaluation of the performance of eight methods based on cone penetration test (CPT) for predicting the ultimate load carrying capacity of square precast RC concrete piles at a site in Siddhirganj. The following methods were used to predict the load carrying capacity of the 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 Penpile method. The ultimate load carrying capacity for each pile is also predicted using the traditional method based on SPT. The ultimate pile capacity obtained from CPT data is also compared graphically with the traditional method based on SPT data.

Prediction of pile capacity has been performed at seven locations within the site. However, Evaluation of the prediction methods was conducted using the statistical analysis based on the results of six friction piles and one end-bearing pile.

An evaluation scheme has been executed to evaluate the CPT methods based on their ability to predict the ultimate pile capacity. Only the criteria selected to evaluate the performance of the prediction methods is: the arithmetic mean and the standard deviation. Based on this evaluation, the Schmertmann method, de Ruiter and Beringen method and Philipponnat method show the best performance in predicting the load carrying capacity of square precast RC concrete piles. But Philipponnat method does not consider the consistency of cohesive soil (soft or hard) to determine the ultimate shaft friction capacity,Q_s of pile and assumes the same empirical factor for all types of clay. Aoki and De Alencar method exhibits moderate performance and Bustamante and Gianeselli (LCPC/LCP) method shows unsatisfactory performance to estimate the pile capacity. The worst prediction methods are Penpile method and Price and Wardle method, which are very conservative (underpredict the pile capacities) and the Tumay and Fakhroo method, which overpredicts the pile capacity excessively. The four CPT methods, which are de Ruiter and Beringen method, Philipponnat method, Schmertmann method and Aoki and De Alencar method show better performance than the currently used method based on SPT. The soil of this site is very erratic and the thickness of soil layers varies drastically throughout the site. SPT is not reliable for cohesive soil which is the predominant soil of the whole site.

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LIST OF ABBREVIATIONS

CPT	Cone Penetration Test
FS	Factor of Safety
Q_u	Ultimate Axial Load Carrying Capacity of Pile
Q_t	End-bearing Capacity of Pile
Q_s	Shaft Friction Capacity of Pile
A_t	Area of Pile Tip
A_s	*A*rea of Pile Shaft
q_t	*U*nit Tip Bearing Capacity
f_s	*U*nit Skin Friction Capacity
Q_d	Design Load Carrying Capacity
D	Diameter of Pile
q_c	Cone Tip Resistance
psi	Pound per square inch
mm	Millimeter
3D	3 Dimensional
l	Litres
hrs	Hour
kN	kilo Newton
R_f	Friction Ratio
SPT	Standard Penetration Test
S_u	Undrained Shear Strength
N_c	Bearing Capacity Factor
c	cohesion
σ'_{avg}	average effective overburden pressure
φ	angle of internal friction
K	coefficient of lateral stress
σ'_v	effective vertical stress
u	pore water pressure
c_a	limiting pile/soil adhesion
LTRC	Louisiana Transportation Research Center

CHAPTER ONE
INTRODUCTION

1.1 BACKGROUND

The prediction of axial pile capacity is a complex problem in geotechnical engineering. Traditional methods of data collection and subsequent analyses are frequently in error when compared to full-scale load tests of the piles. Cone penetration testing (CPT) provides a means by which continuous representative field data may be obtained. The CPT data are acquired by friction cone penetrometers and in these tests the total cone tip resistance and sleeve friction are recorded and pore water pressure is also measured.

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 at Siddhirganj, the CPT is considered a perfect tool for site characterization.

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. The square precast RC concrete piles are the most common piles used in Public Works Department.

Piles are expensive structural members, and pile projects are always costly. 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 expensive and time consuming.

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. 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 is limited to identification of dense sand layers required to support the tip of the end-bearing piles. 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 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 prove 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 this 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.

1.2 OBJECTIVE OF THE STUDY

The major objectives of this study are as follows:

1. To identify the most appropriate methods for estimating the ultimate axial load carrying capacity of precast RC concrete piles from the cone penetration test data.

2. To compare the ultimate pile capacity predicted by different methods utilizing the CPT data with the ultimate pile capacity calculated by the method using the SPT data.

1.3 OUTLINE OF THE THESIS

The thesis is composed of total 4 chapters:

Chapter 2 highlights the different methods of predicting the pile capacity utilizing the CPT data and the method of predicting the pile capacity using the SPT data.

Chapter 3 elaborates the SPT and CPT data collection and analysis in detail and presents the results of the analyses conducted on square reinforced concrete piles for Siddhirganj, Narayanganj soils to predict the pile capacity from the CPT and SPT data.. This research effort is focused on the applicability of eight CPT methods to predict the ultimate axial compression load carrying capacity of piles from CPT data. Comparison of ultimate pile capacity predicted by various methods of CPT with the one predicted by SPT data is also presented graphically.

Chapter 4 presents the conclusion and the recommendations regarding the findings and the future projects.

CHAPTER TWO
LITERATURE REVIEW

2.1 GENERAL

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:

$$Q_u = Q_t + Q_s = q_t A_t + f A_s \tag{2.1}$$

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:

$$Q_d = \frac{Q_u}{F.S.} \tag{2.2}$$

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

2.2 CONE PENETRATION TEST

The cone penetration test has been recognized as one of the most widely used in situ tests. In Bangladesh, cone penetration testing is not popular. 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 2.1. 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:

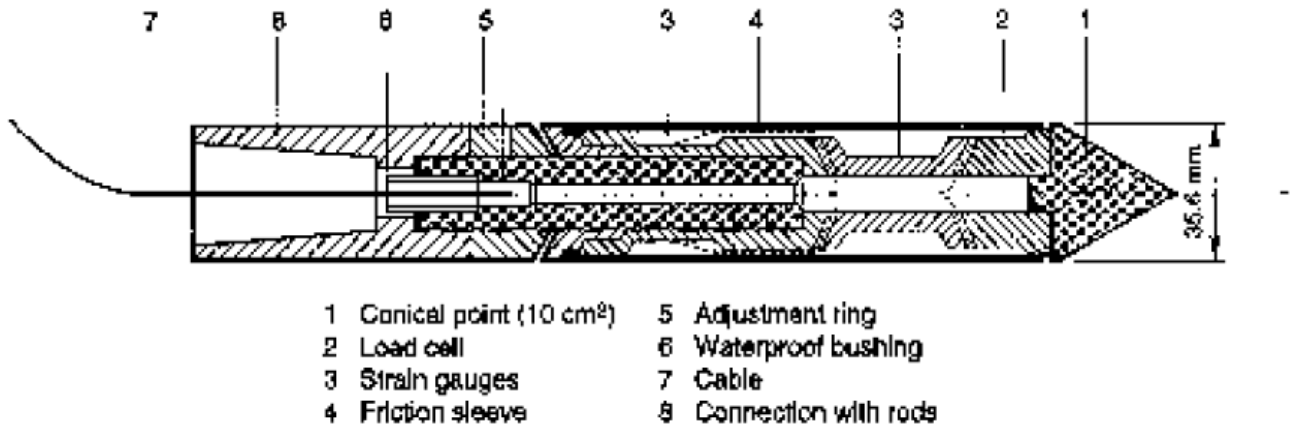
(1) Direct approach: in which

- 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.

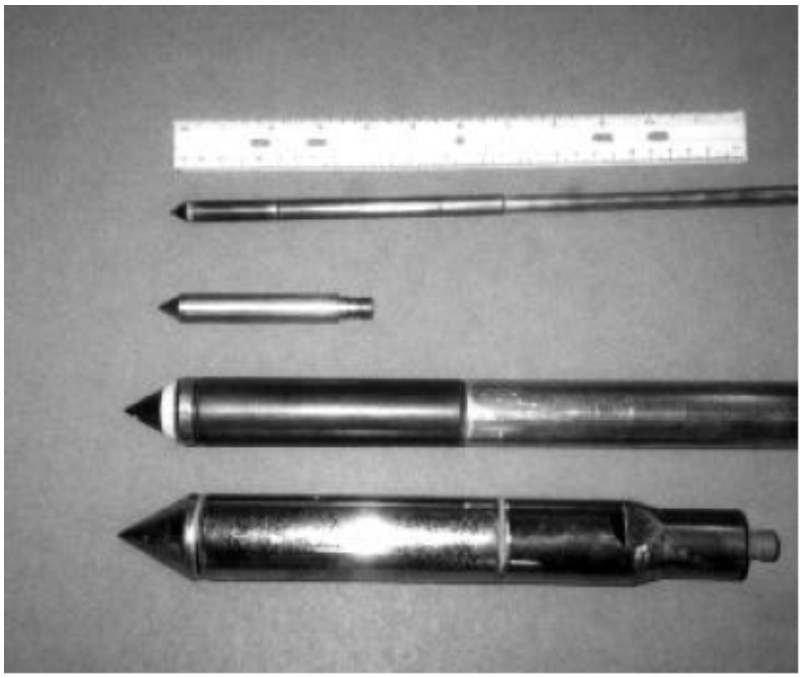
(2) Indirect approach: in which the CPT data (q_c and f_s) are first used to evaluate the soil strength parameters such as the undrained shear strength and the angle of internal friction.

These 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.



(a) Schematic of the electric friction cone penetrometer



(b) The 1.27, 2, 10, and 15 cm^2 cone penetrometers

Figure 2.1 The electric cone penetrometer (after LTRC)

2.3 PREDICTION OF PILE CAPACITY BY CPT

In this project, the direct methods are described in detail. These methods are Schmertmann (1978), de Ruiter and Beringen (1979), Bustamante and Gianeselli (LCPC/LPC) (1982), Tumay and Fakhroo (cone-m) (1982), Aoki and De Alencar (1975), Price and Wardle (1982), Philipponnat (1980), and the Penpile (1978) method. 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.

2.3.1 Schmertmann Method

Schmertmann (1978) proposed the following relationship to predict the unit tip bearing capacity of the pile (q_t) from the cone tip resistance (q_c):

$$q_t = \frac{q_{c1} + q_{c2}}{2} \tag{2.3}$$

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 2.2. 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 (1978) suggested an upper limit of 150 TSF (15 MPa) for the unit tip bearing capacity (q_t).

According to Schmertmann's method (1978), the unit skin friction of the pile (f) is given by:

$$f = \alpha_c f_s \tag{2.4}$$

Where α_c is a reduction factor, which varies from 0.2 to 1.25 for clayey soil, and f_s is the sleeve friction. Figure 2.3 depicts the variation of α_c with f_s for different pile types in clay.

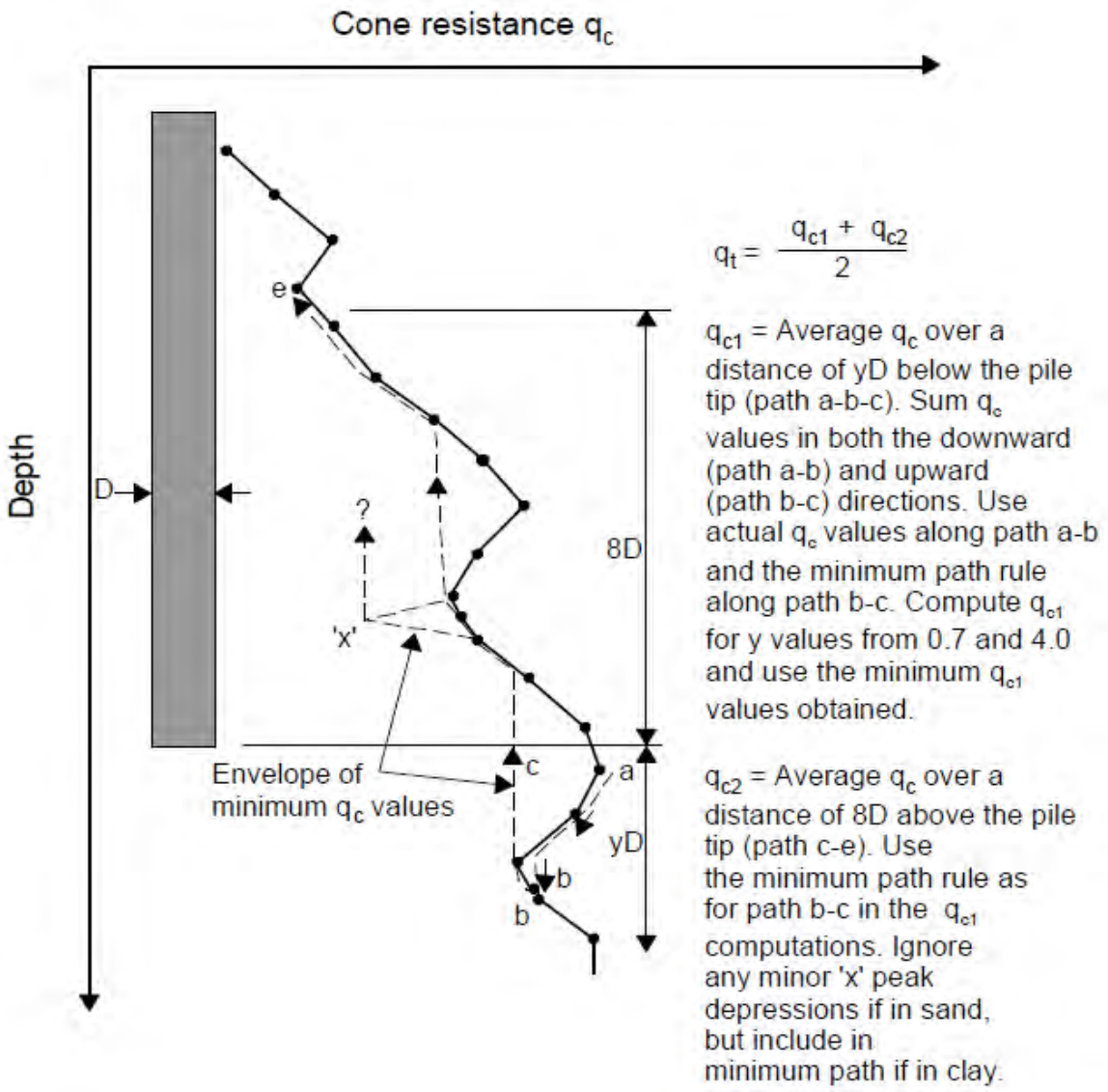


Figure 2.2 Calculation of the average cone tip resistance (after Schmertmann, 1978)

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

$$Q_s = \alpha_s \left(\sum_{y=0}^{\delta D} \frac{y}{\delta D} f_s A_s + \sum_{y=\delta D}^{L} f_s A_s \right) \tag{2.5}$$

where α_s is the correction factor for sand, which can be obtained from Figure 2.4, y is the depth at which side resistance is calculated, and L is the pile length.

Schmertmann (1978) suggested a limit of 1.2 TSF (120 kPa) on f.

2.3.2 de Ruiter and Beringen Method

This method is proposed by de Ruiter and Beringen (1979) and is based on the experience gained in the North Sea. 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:

$$q_t = N_c S_u(tip)$$
$$S_u(tip) = \frac{q_c(tip)}{N_k} \tag{2.6}$$

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 (1978).

The unit skin friction is given by:

$$f = \beta S_u(side) \tag{2.7}$$

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

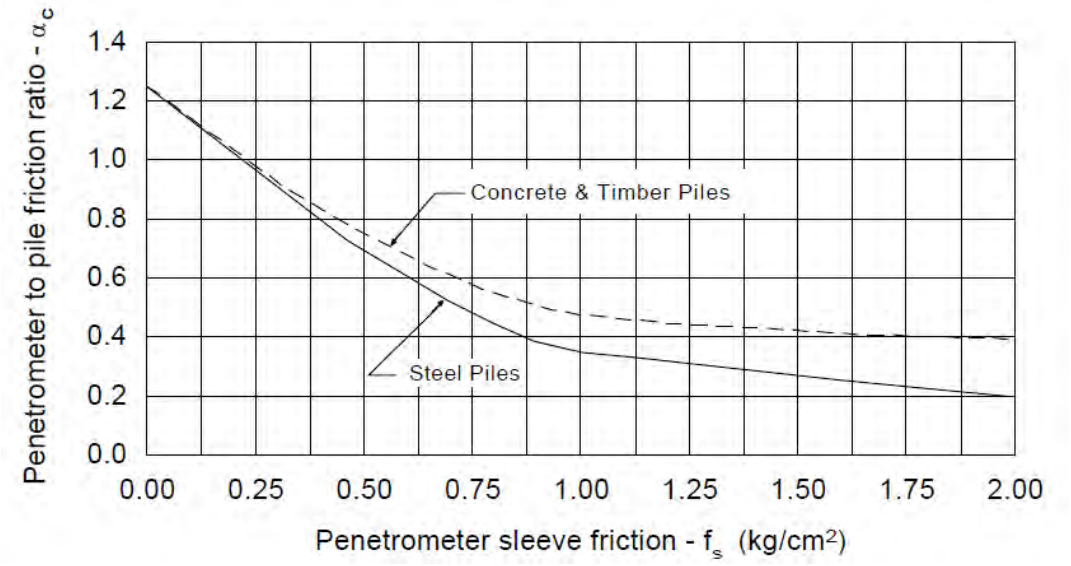


Figure 2.3 Penetration design curves for pile side friction in clay (after Schmertmann,1978)

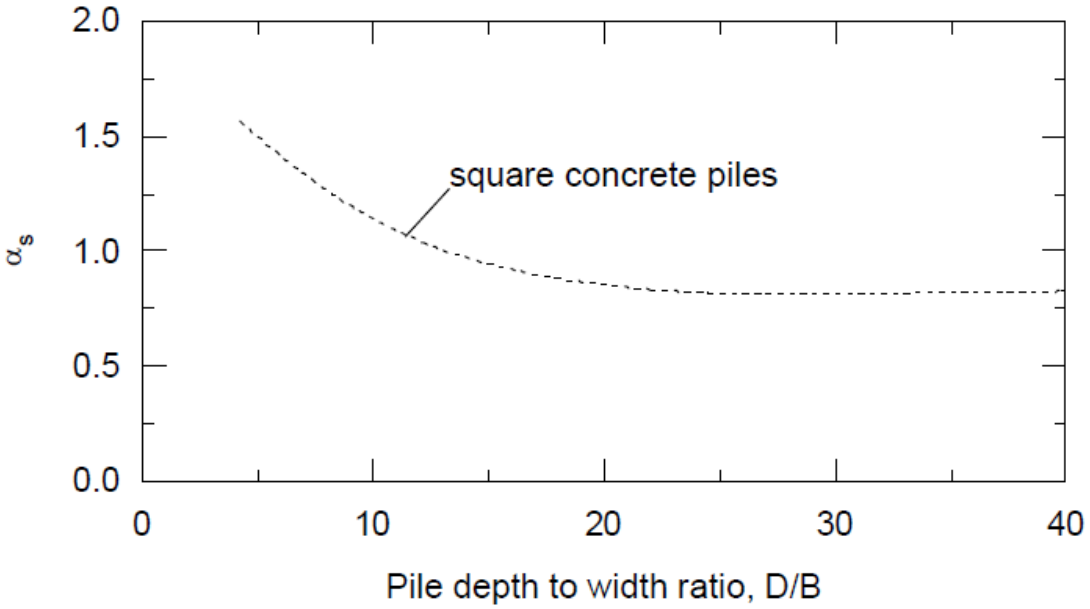


Figure 2.4 Penetrometer design curve for side pile friction in sand (after Schmertmann, 1978)

overconsolidated (OC) clay. $S_u(side)$, the undrained shear strength for each soil layer along the pile shaft, is determined by:

$$S_u(side) = \frac{q_c(side)}{N_k} \tag{2.8}$$

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

In sand, the unit tip bearing capacity of the pile (q_t) is calculated similar to Schmertmann method(1978). The unit skin friction (f) for each soil layer along the pile shaft is given by:

$$f = \min \begin{cases} f_s \\ \dfrac{q_c(side)}{300} \text{ (compression)} \\ \dfrac{q_c(side)}{400} \text{ (tension)} \\ 1.2 \text{ TSF (120kPa)} \end{cases} \tag{2.9}$$

de Ruiter and Beringen(1979) imposed limits on q_t and f in which q_t = 150 TSF (15 MPa) and f= 1.2 TSF (120 kPa).

2.3.3 Bustamante and Gianeselli Method (LCPC/LCP Method)

Bustamante and Gianeselli (1982) 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 . 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:

$$q_t = k_b q_{eq}(tip) \tag{2.10}$$

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 2.1) 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 1.5D below the pile tip to 1.5D 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 2.5, 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 2.5).

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 2.2 (for example, pile category is 9 for square PPC piles),
B. for each soil layer, select the appropriate curve number (Tables 2.3 and 2.4) based on soil type, equivalent cone tip resistance along the soil layer ($q_{eq}(side)$), and pile category, use Table 2.3 for clay and silt and Table 2.4 for sand and gravel,
C. from Figure 2.6, use the selected curve number and the equivalent cone tip resistance ($q_{eq}(side)$) to obtain the maximum unit skin friction (f), use Figure 2.6a for clay and silt and Figure 2.6b for sand and gravel.

Table 2.1 LCPC bearing capacity factor (k_b) (after Bustamante and Gianeselli,1982)

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

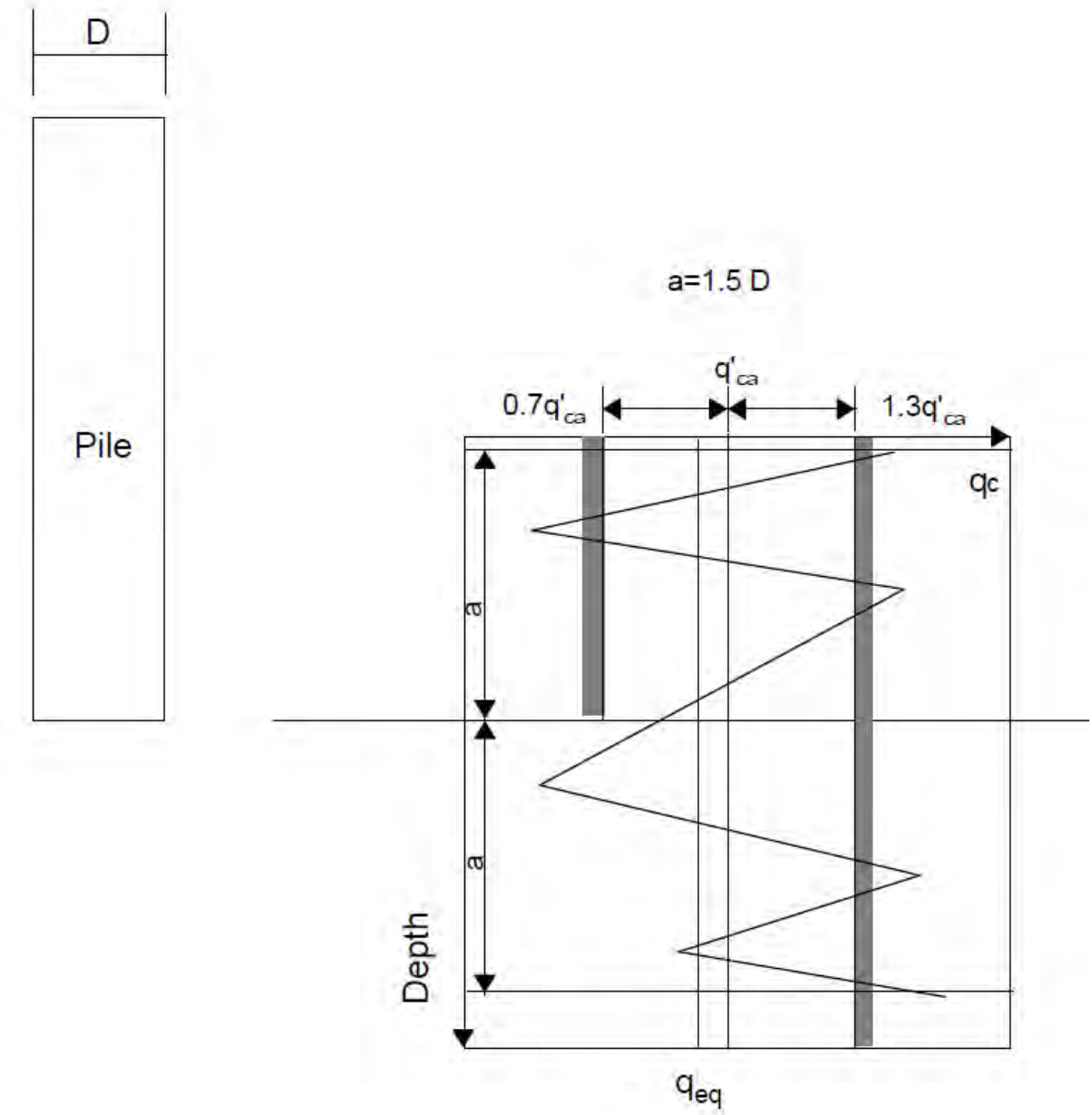


Figure 2.5 Calculation of the equivalent average tip resistance for LCPC method (after Bustamante and Gianeselli, 1982)

Table 2.2 Pile categories for the LCPC method (after Bustamante and Gianeselli, 1982)

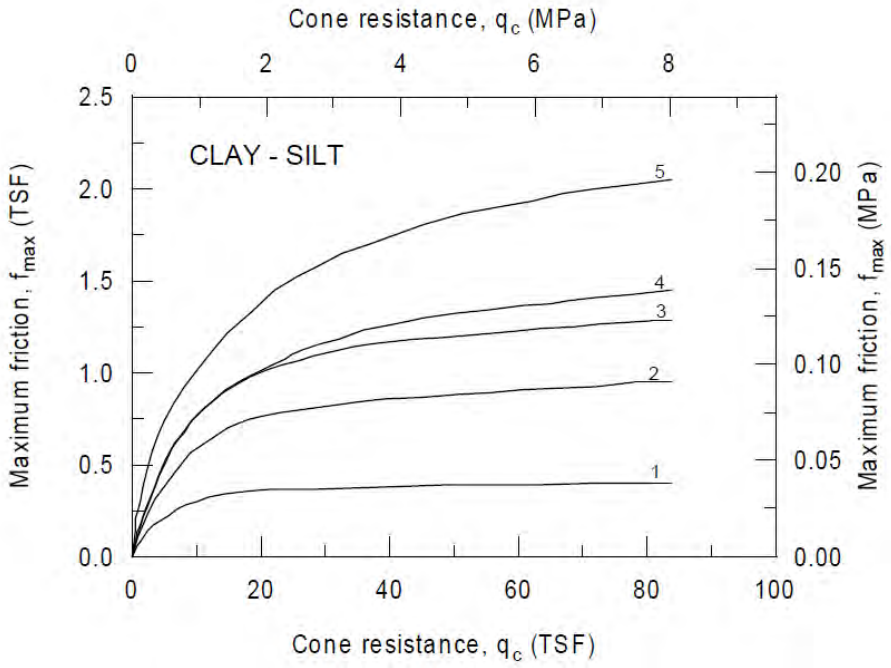
1. FS Drilled shaft with no drilling mud	Installed without supporting the soil with drilling mud. Applicable only for cohesive soils above the water table.
2. FB Drilled shaft with drilling mud	Installed using mud to support the sides of the whole. Concrete is poured from the bottom up, displacing the 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 - H piles - caissons made of 2, 3, or 4 sheet pile sections. 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. - H piles - Pipe piles - 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 2.3 Input parameters for clay and silt for LCPC method (after Bustamante and Gianeselli, 1982)

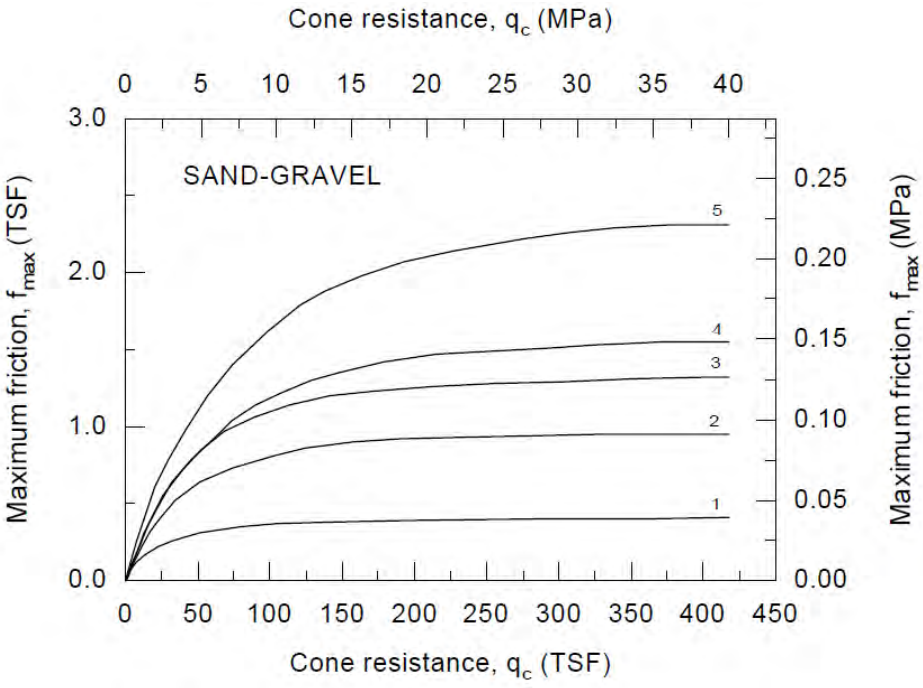
CURVE #	q_c (ksf)	PILE TYPE (see Table 2.2)	COMMENTS ON INSERTION PROCEDURE
1	< 14.6	1-17	
	> 14.6	1,2	- very probable values when using tools without teeth or with oversized blades and where a remoulded layer of material can be deposited along the sides of the drilled hole. Use these values also for 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 qs can be between curves 1 and 2; use an intermediate value of qs 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, qs 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 < 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 2.4 Input parameters for sand and gravel for LCPC method (after Bustamante and Gianeselli, 1982)

CURVE #	q_c (ksf)	PILE TYPE (see Table 2.2)	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.



(a) For clay and silt



(b) For sand and gravel

Figure 2.6 Maximum friction curves for LCPC method (after Briaud,1986) (a) for clay-silt and (b) for sand-gravel

2.3.4 Tumay and Fakhroo Method (Cone-m Method)

Tumay and Fakhroo (1982) proposed this method to predict the ultimate pile capacity of piles in clayey soils . The unit tip bearing capacity (q_t) is estimated using a procedure similar to Schmertmann's method(1978) as follows:

$$q_t = \frac{q_{c1} + q_{c2}}{4} + \frac{q_a}{2} \tag{2.11}$$

where q_{c1} is the average of q_c values 4D below the pile tip, q_{c2} is the average of the minimum q_c values 4D below the pile tip, and q_a is the average of the minimum of q_c values 8D 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:

$$f = mf_{sa} \tag{2.12}$$

Tumay and Fakhroo(1982) suggested that f=0.72 TSF (72 kPa). The adhesion factor (m) is expressed as:

$$m = 0.5 + 9.5e^{-9f_{sa}} \tag{2.13}$$

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).

2.3.5 Aoki and De Alencar Method

Aoki and De Alencar (1975) proposed the following method to estimate the ultimate load carrying capacity of the pile from CPT data .

The unit tip bearing capacity (q_t) is obtained from:

$$q_t = \frac{q_{ca}(tip)}{F_b} \tag{2.14}$$

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:

$$f = q_c(side)\frac{\alpha_s}{F_s} \tag{2.15}$$

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 α_s is an empirical factor that depends on the soil type. Factors F_b and F_s are given in Table 2.5. The values of the empirical factor α_s are presented in Table 2.6.

Table 2.5 Empirical factors F_b and F_s (after Aoki and De Alencar ,1975)

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 2.6 The empirical factor α_s values for different soil type (after Aoki and De Alencar,1975)

Soil Type	α_s(%)	Soil Type	α_s(%)	Soil Type	α_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 α_s =1.4 percent, for silt α_s =3.0percent, and for clay α_s =6.0 percent. For soils consist of combination of sand, silt, and clay, α_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 α_s = 0.50× α_s (clay)+0.20× α_s (silt)+0.30× α_s (sand) = 0.5×6+0.2×3+0.3×1.4 = 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).

2.3.6 Price and Wardle Method

Price and Wardle (1982) proposed the following relationship to evaluate the unit tip bearing capacity (q_t) of the pile from the cone tip resistance:

$$q_t = k_b q_c \tag{2.16}$$

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:

$$f = k_s f_s \tag{2.17}$$

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 (1982) 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).

2.3.7 Philipponnat Method

Philipponnat(1980) proposed the following expression to estimate the unit tip bearing capacity of the pile (q_t) from the cone tip resistance (q_c):

$$q_t = k_b q_{ca} \tag{2.18}$$

where k_b is a factor that depends on the soil type as shown in Table 2.7. The cone tip resistance (q_{ca}) is averaged as follows:

$$q_{ca} = \frac{q_{ca(A)} + q_{cb(B)}}{2} \tag{2.19}$$

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 (1980) recommended the removal of the extreme peaks (spikes) when the tip resistance profiles is irregular and imposed a condition in which *qca(A)* · *qcb(B)*.

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

$$f = \frac{\alpha_s}{F_s} q_{cs} \tag{2.20}$$

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 2.8. The factor α_s depends on the pile type where α_s equals to 1.25 for precast concrete driven piles. Philipponnat (1980) 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 2.7 Bearing capacity factor (k_b) **(**after Philipponnat,1980)

Soil Type	k_b
Gravel	0.35
Sand	0.40
Silt	0.45
Clay	0.50

Table 2.8 Empirical factor *Fs* **(**after Philipponnat,1980)

Soil Type	*Fs*
Clay and calcareous clay	50
Silt, sandy clay, and clayey sand	60
Loose sand	100
Medium dense sand	150
Dense sand and gravel	200

2.3.8 Penpile Method

The penpile method (1978) was proposed by Clisby et al. for the Mississippi Department of Transportation . The unit tip bearing capacity of the pile (q_t) is determined from the following relationship:

$$q_t = \begin{cases} 0.25q_c & \text{for pile tip in clay} \\ 0.125q_c & \text{for pile tip in sand} \end{cases} \tag{2.21}$$

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:

$$f = \frac{f_s}{1.5 + 0.1 f_s} \tag{2.22}$$

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

2.4 PREDICTION OF PILE CAPACITY BY SPT

The following soil characteristics and parameters are required to predict the pile capacity using this method: (a) soil profile and thickness of each soil layer, (b) the shear strength parameters: cohesion and angle of internal friction, and (c) unit weight. The angle of internal friction 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:

$$q_t = cN_c \tag{2.23}$$

where c is the cohesion of the soil layer, N_c is the bearing capacity factor.

Figure 2.7 presents the variation of N_c with the ratio R (R=D/B, depth/pile diameter). The unit skin friction can be predicted by:

$$f = c_a \tag{2.24}$$

where c_a is the limiting pile/soil adhesion for cohesive soil. The variation of c_a with soil cohesion is shown in Figure 2.8.

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

$$q_t = \alpha N_{q'} \sigma_v' \tag{2.25}$$

where α is an empirical factor depends on the angle of internal friction, pile width B, and pile depth D (Figure 2.9). Nq' is the bearing capacity factor (Figure 2.10), and σ'_v is the effective vertical stress. For cohesionless soil, q_t calculated from equation 2.25 should be less or equal to the maximum unit tip bearing capacity evaluated from Figure 2.11.

The unit skin friction can be predicted by:

$$f = \sigma'_{avg} K \tan\varphi \tag{2.26}$$

where σ'_{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 φ is the angle of internal friction. The unit skin friction f evaluated from equation 2.26 should be reduced based on soil type by the friction limit factors presented in Table 2.9.

Table 2.9 Friction limit factors for concrete piles (after Bowels, 1982)

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

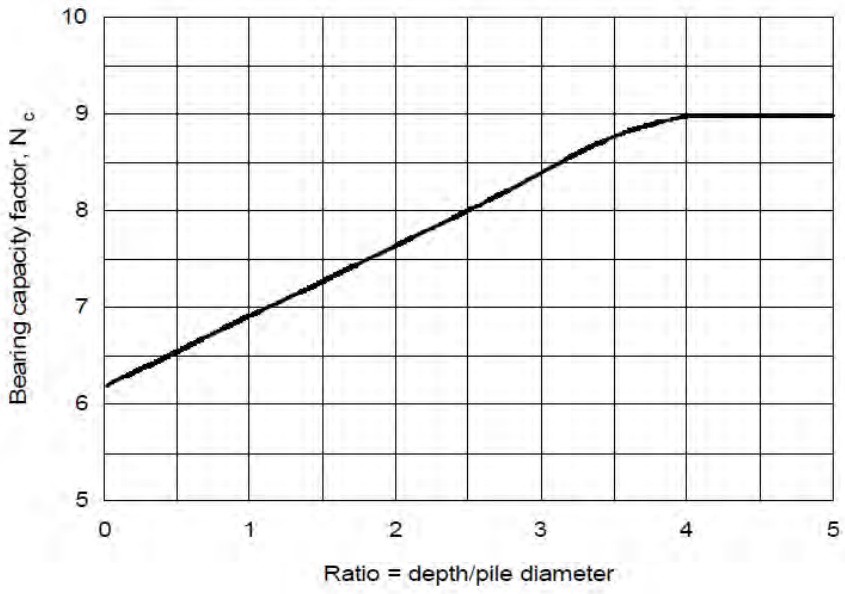


Figure 2.7 Bearing capacity factor N_c for foundations in clay (after Skempton,1951)

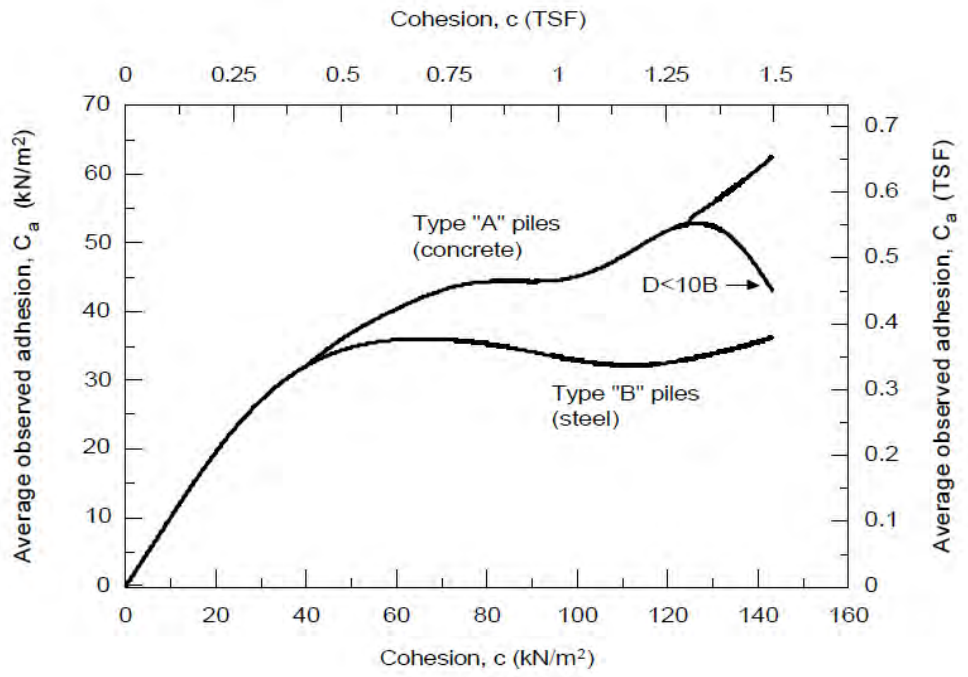


Figure 2.8 Limiting adhesion for piles in soft to stiff clays(after Tomlinson,1975)

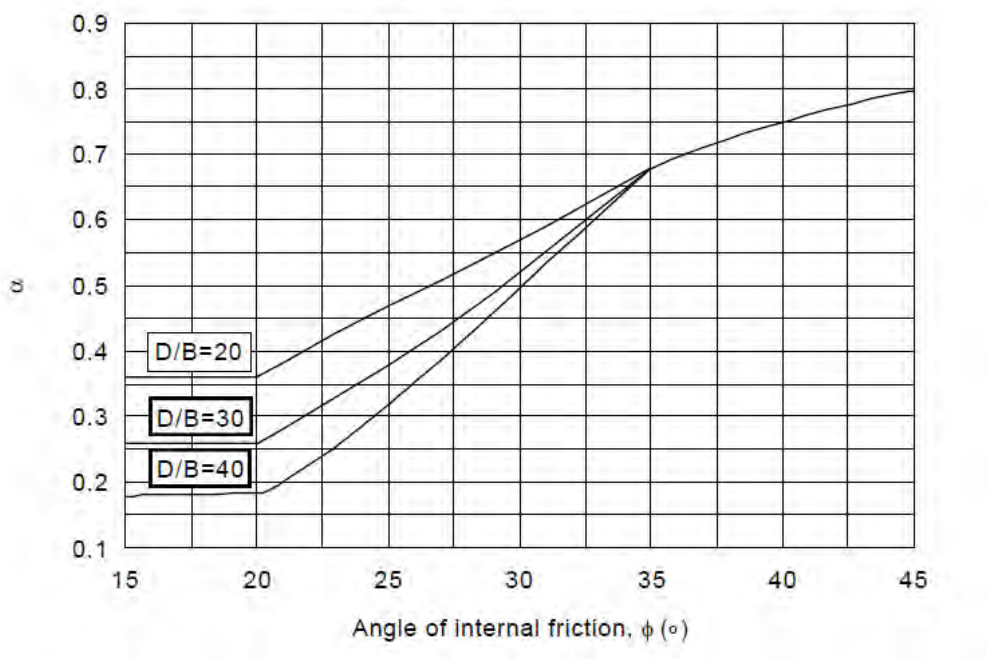


Figure 2.9 Relationship between α-coefficient and angle of internal Friction for cohesionless soils (after Bowles, 1982)

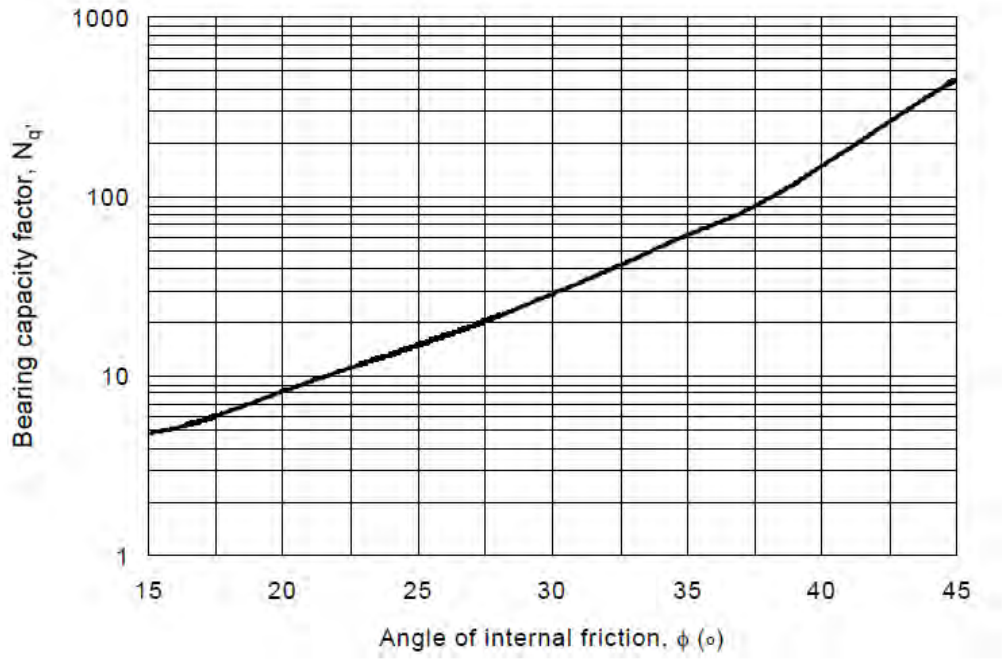


Figure 2.10 Estimating the bearing capacity factor Nq' (after Bowles, 1982)

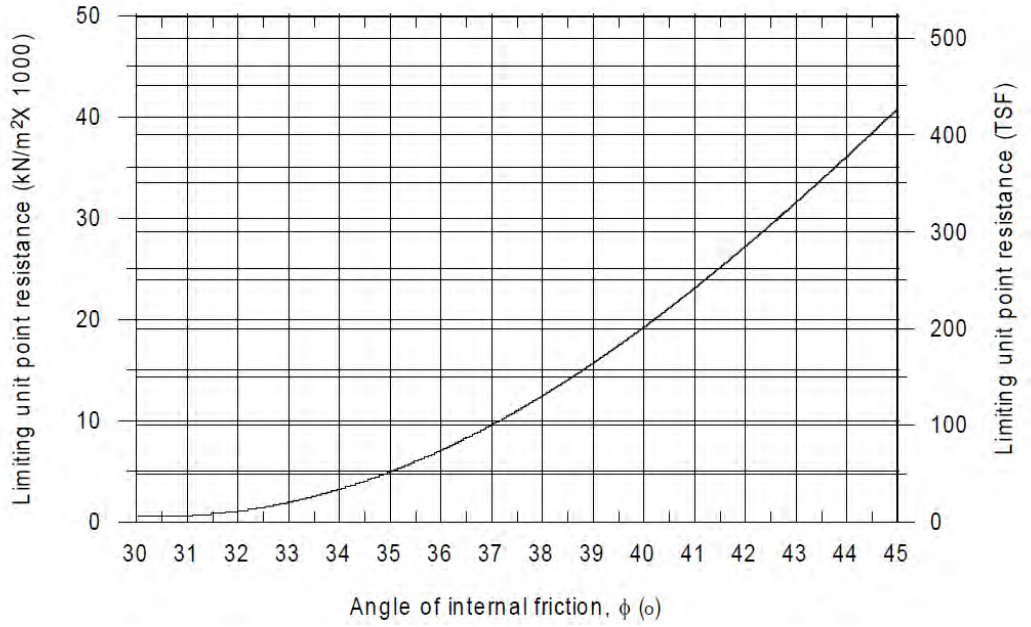


Figure 2.11 Relationship between the maximum unit pile point resistance and friction angle for cohesionless soils (after Bowles, 1982)

2.5 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 2.12 and 2.14. Zhang and Tumay proposed the probabilistic region estimation method for soil classification . 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 2.13.

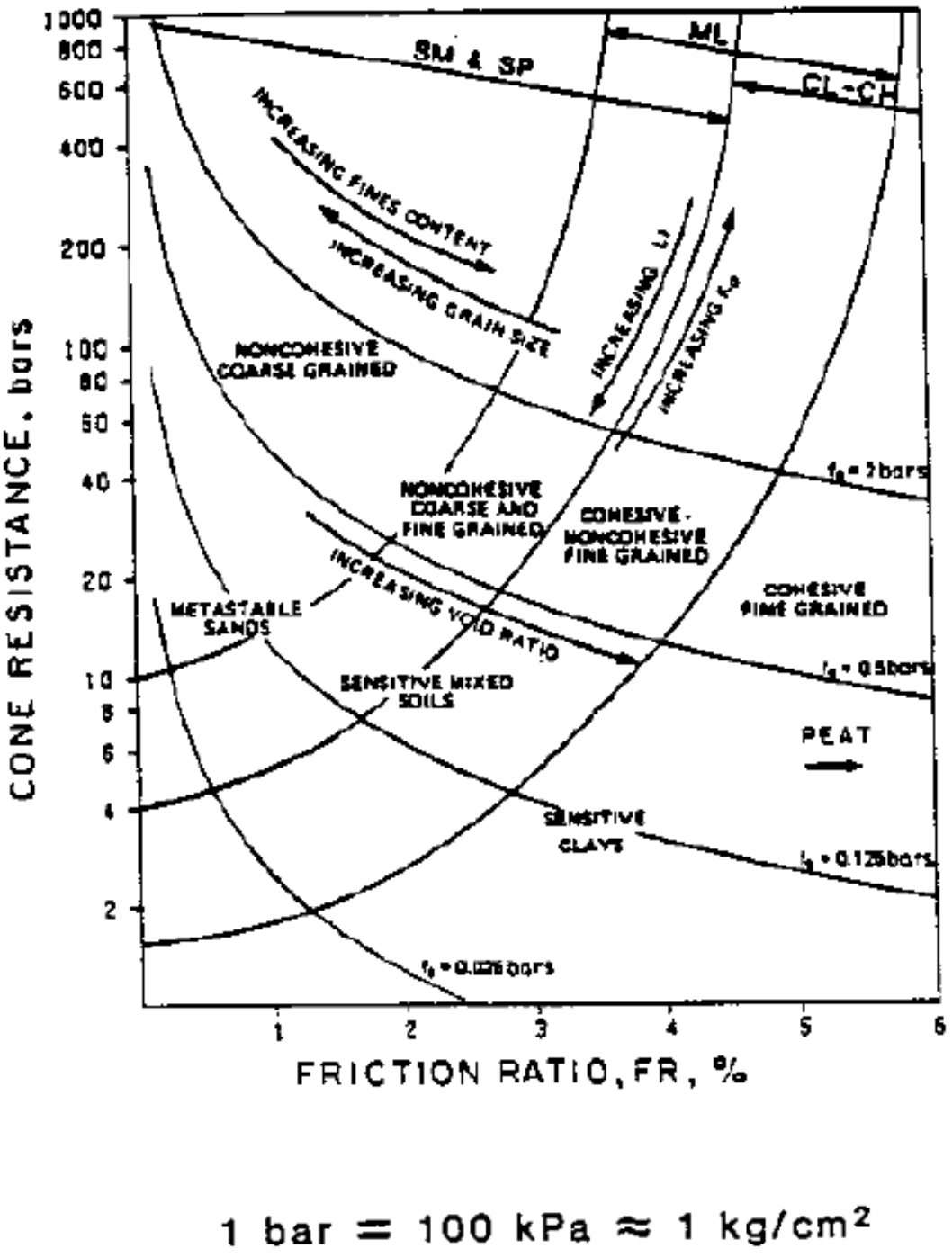


Figure 2.12 Soil classification chart for standard electric friction cone (after Douglas and Olsen, 1981)

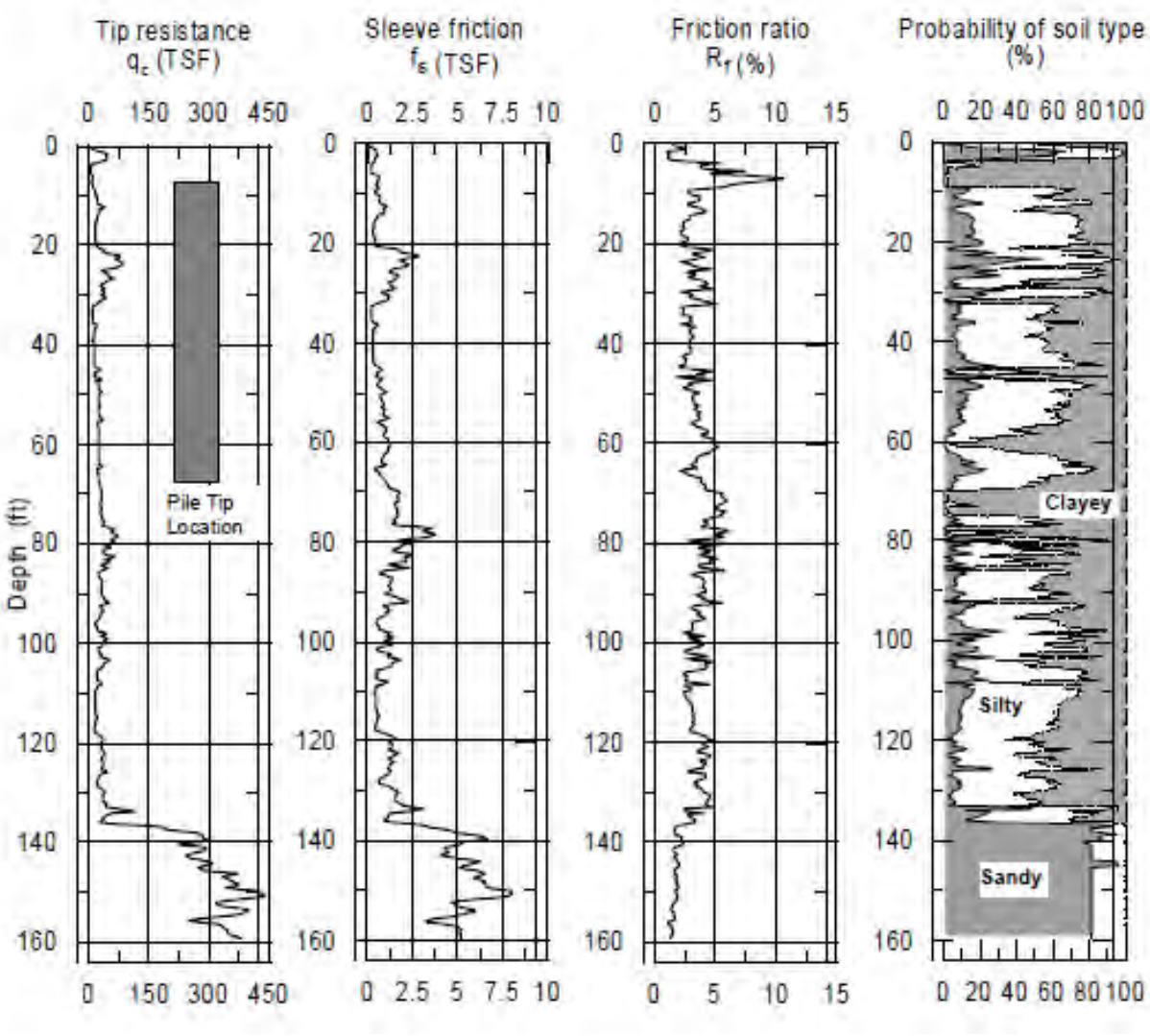


Figure 2.13 Soil classification using the probabilistic region method (after Zhang and Tumay, 1999)

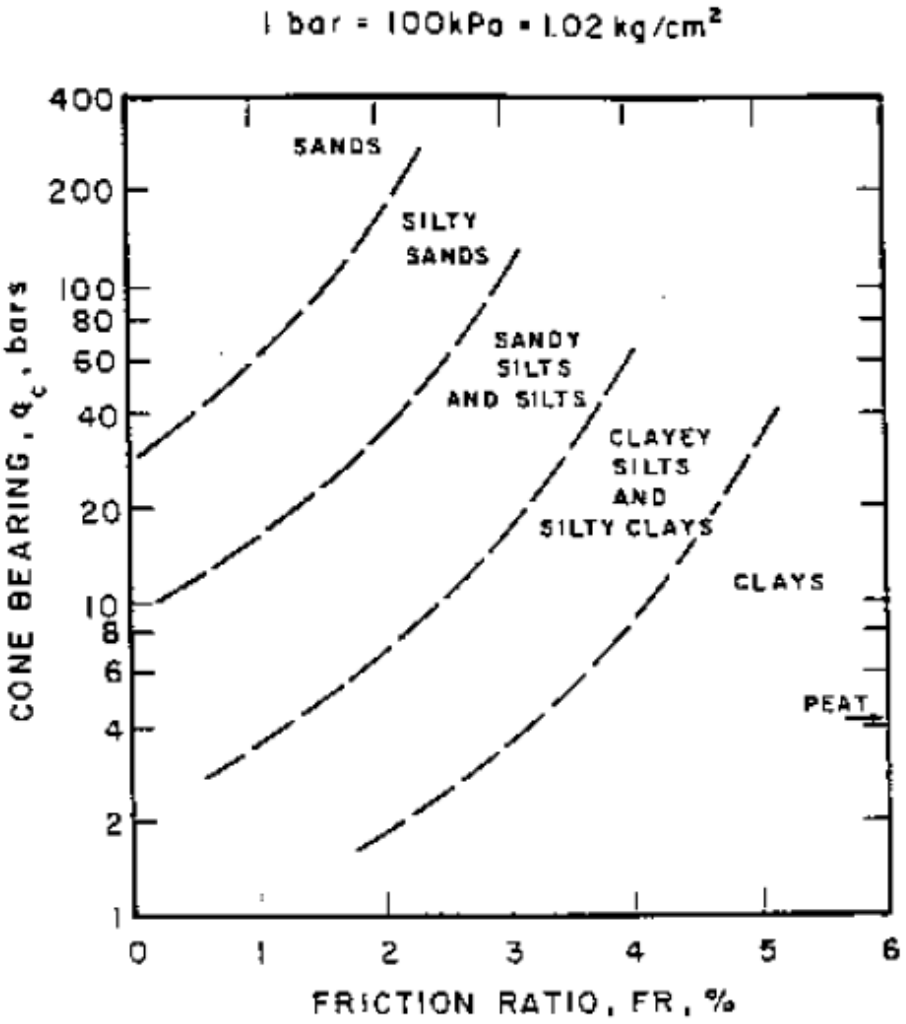


Figure 2.14 Simplified classification chart for standard electric friction cone (after Robertson and Campanella, 1984)

2.6 CONCLUDING REMARKS

In this chapter various methods for estimating pile capacity using CPT and SPT have been discussed. The ultimate axial load carrying capacity of the pile (Q_u) consists of the end-bearing capacity of the pile (Q_t) and the shaft friction capacity (Q_s). Classifying piles as end-bearing piles and friction piles is very important based on the nature of support provided by the surrounding soil in the case of pile design and analysis. In the case of the cone penetration test two forces are measured during penetration: the total tip resistance (q_c) and the sleeve friction (f_s). The friction ratio (R_f) is defined as the ratio between the sleeve friction and tip resistance and is expressed in percent. Eight methods of predicting the pile capacity utilizing the CPT data are described in detail. These methods are Schmertmann(1978), de Ruiter and Beringen(1979), Bustamante and Gianeselli (LCPC/LPC)(1982), Tumay and Fakhroo (cone-m)(1982), Aoki and De Alencar(1975), Price and Wardle(1982), Philipponnat(1980), and the Penpile (1978) method. 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. The following soil characteristics and parameters are required to predict the pile capacity using the method based on SPT: (a) soil profile and thickness of each soil layer, (b) the shear strength parameters: cohesion and angle of internal friction, and (c) unit weight. The angle of internal friction is obtained from the standard penetration test results or from laboratory tests.

CHAPTER THREE
DATA COLLECTION AND ANALYSIS

3.1 GENERAL

The study location is at a site in Siddhirganj. In this site seven CPT and twelve SPT have been carried out. In this chapter pile capacity has been estimated from CPT and SPT methods discussed in chapter 2.

3.2 GEOLOGY OF THE PROJECT

According to geological map of Bangladesh published in 1990 geological characterictics of Siddhirganj is described. Geology of this location is alluvial silt – light to medium grey, fine sandy to clayey silt. Commonly poorly stratified; average grain size decreases away from main channels. Chiefly deposited in flood basins and interstream areas. Unit includes small backswamp deposits and varying amounts of thin, interstratified sand, deposited during episodic or unusually large floods. Illite is the most abundant clay mineral. Most areas are flooded annually. Included in this unit are thin veneers of sand spread by episodic large floods over flood-plain silts. Historic pottery, artefacts, and charcoal (radiocarbon dated 500-6000 yrs. B.P.) found in upper 4 m.

3.3 SUBSOIL INVESTIGATION BASED ON SPT

Figure 3.1 represents the site plan showing the five sections of soil profiles, location of twelve bore holes for SPT values and location of seven CPT points. Figures 3.2-3.6 describe the five soil profile sections having twelve bore holes which exhibit SPT values with depth, visual soil classification and thickness of soil layers. RL of the site is not the same. RL is 5.7m for bore hole no. 02, 03,04, 05, 06, 07, 08, 09, 10, 11 and RL is 4.0m and 5.3m for bore hole no. 12 and 01 respectively.

The soil of the site is very erratic in nature. The predominant soil of the whole site is clay and the thickness of the clay layers varies drastically among the bore holes. The maximum and minimum thickness of clay layers is 26m and 12m respectively when total depth of all bore holes is 26m. For bore hole no. 05 and 06, soil of 26m depth is clay; for bore hole no. 3, 8, 9, 11and 12 the maximum 6m top soil is either fine sand or nonplastic silt when the soil of the remaining depth is clay; for bore hole no. 1, 7 and 10 the maximum 16m top soil is clay when the soil of the remaining depth is nonplastic silt and clay; for bore hole no. 4, the 8m top soil is fine sand, clay and nonplastic silt when the soil of the remaining depth is clay; for bore hole no. 2, the 2m top soil is nonplastic silt,then 18m depth is clay, then 2m depth is nonplastic silt and the remaining depth is clay.

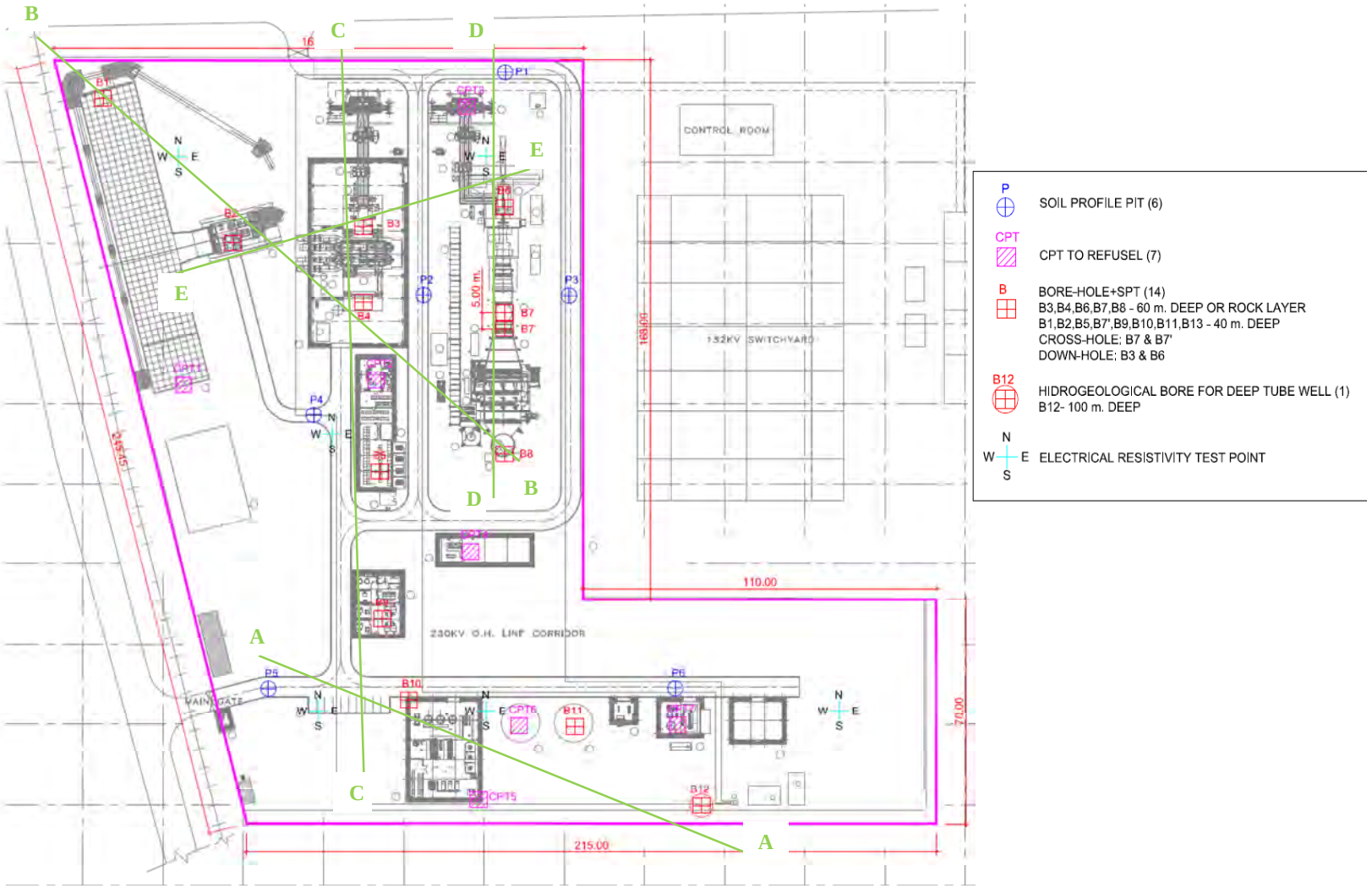


Figure 3.1 **Layout of bore holes and sections**

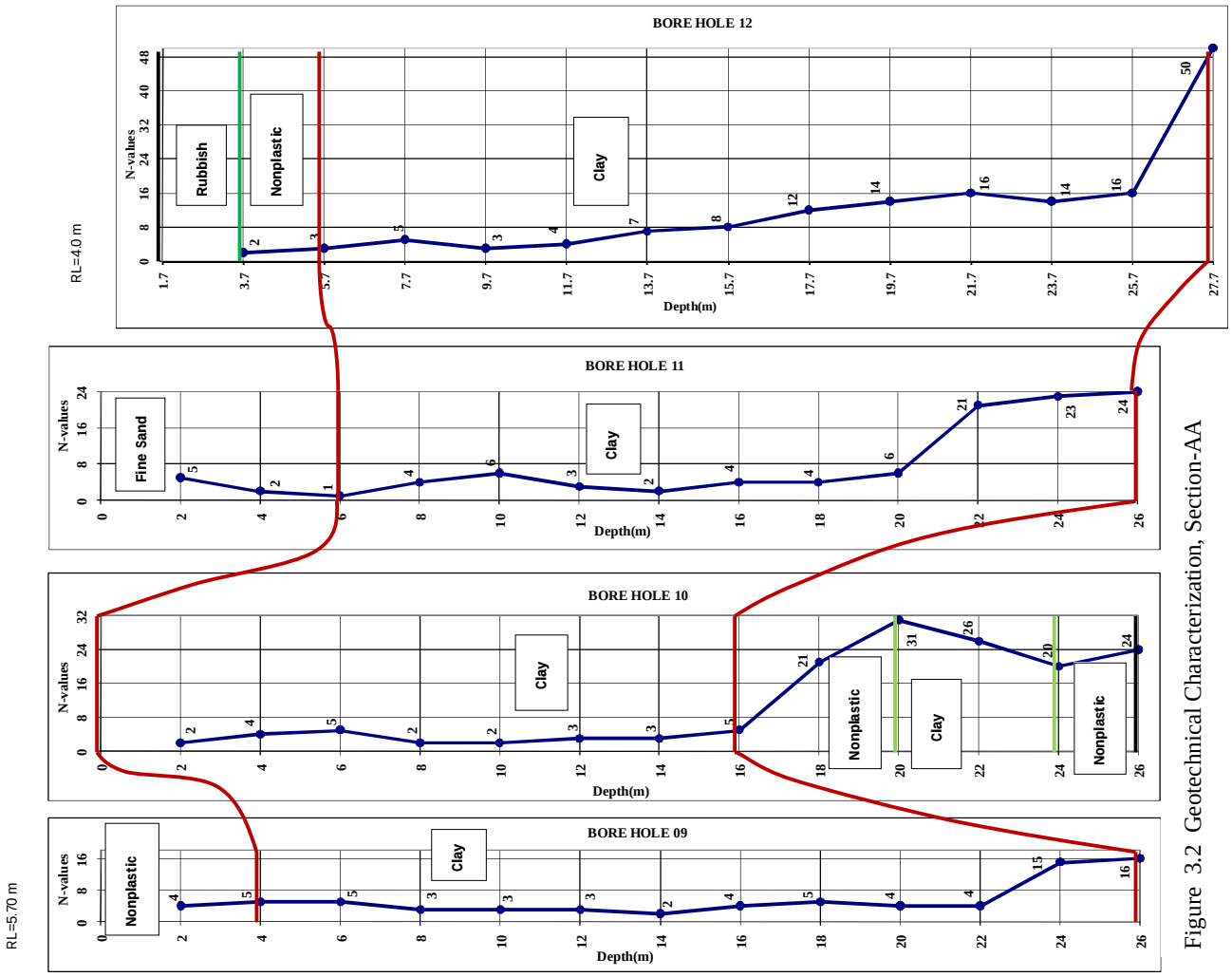


Figure 3.2 Geotechnical Characterization, Section-AA

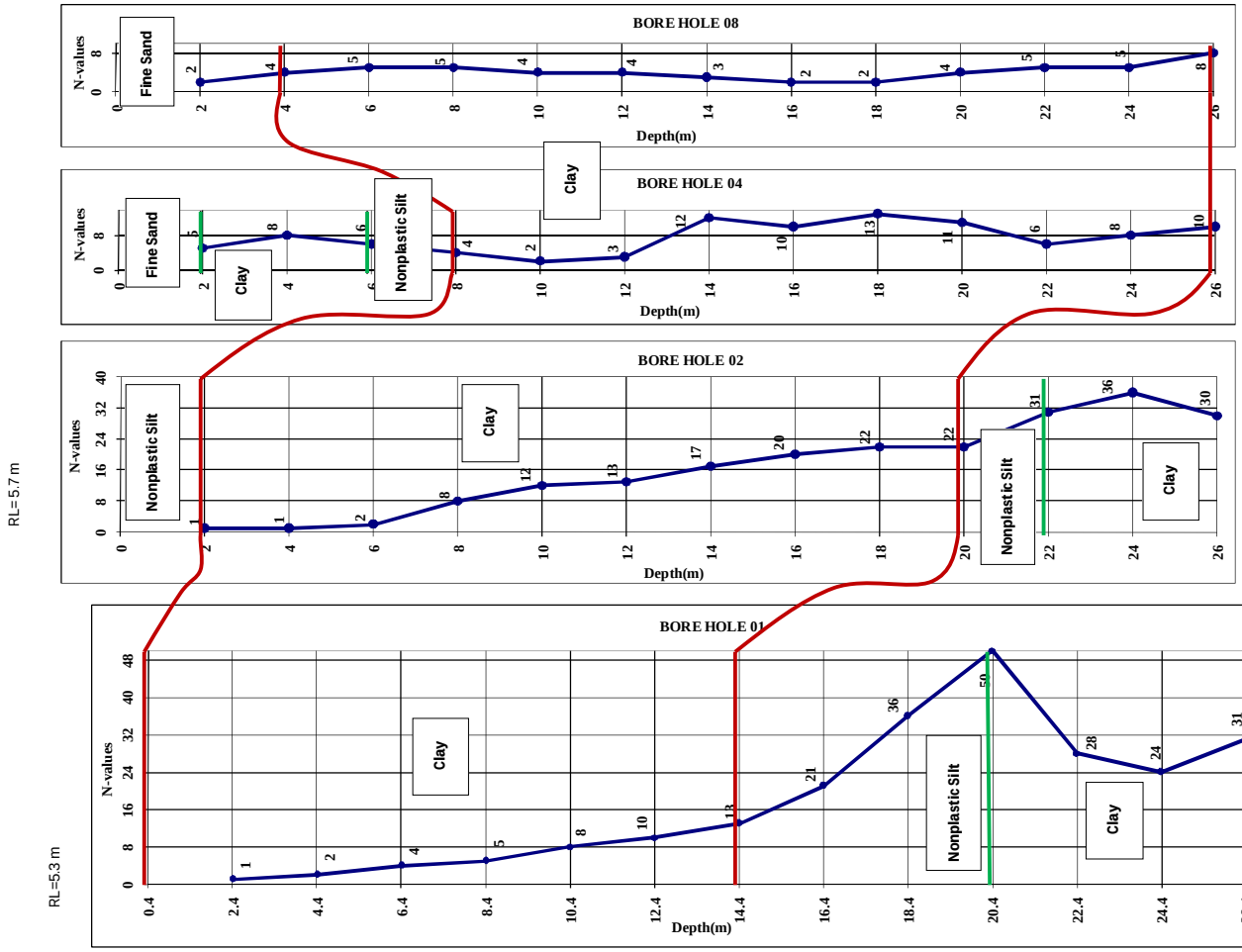


Figure 3.3 Geotechnical Characterization, Section-BB

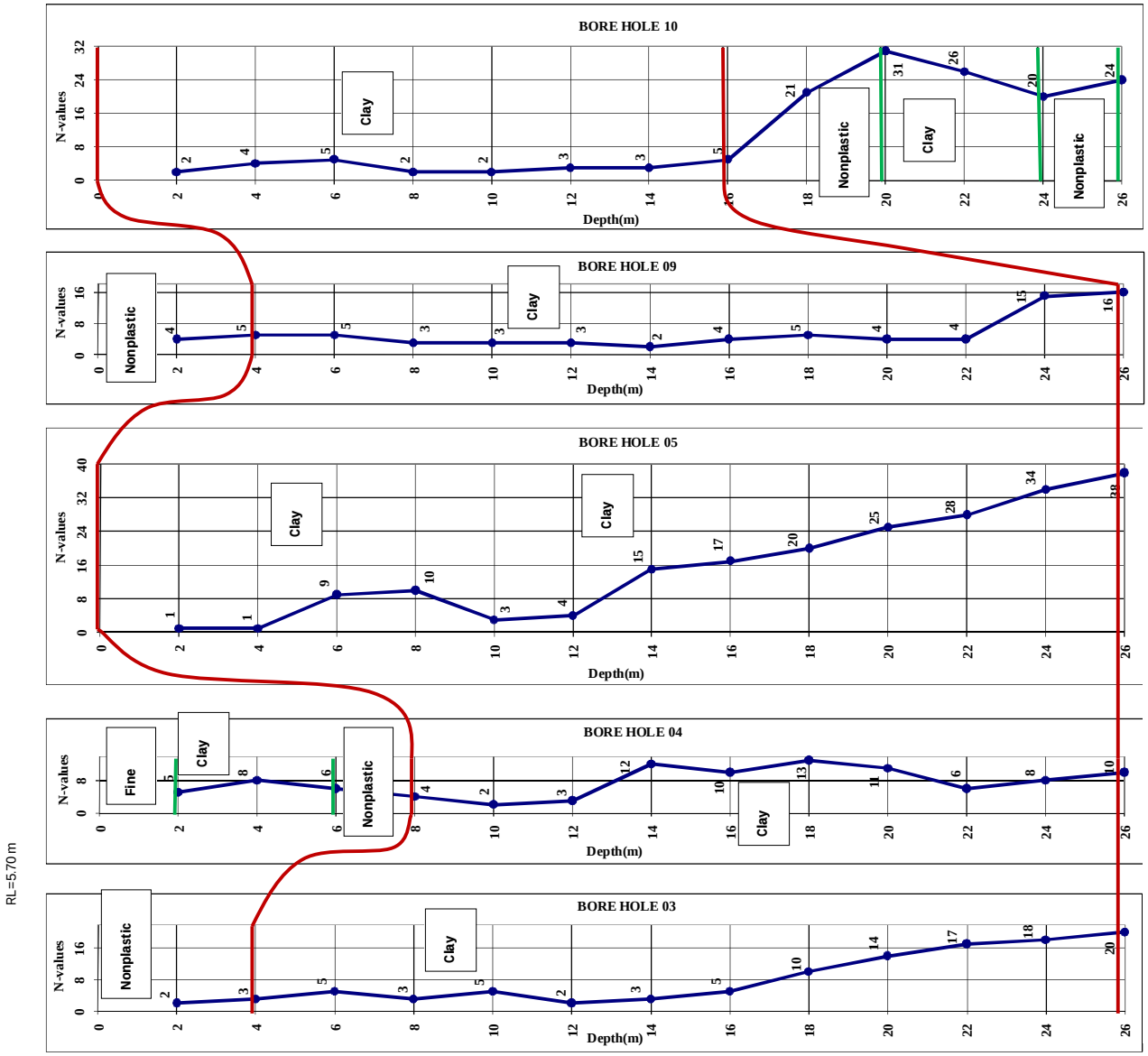


Figure 3.4 Geotechnical Characterization, Section-CC

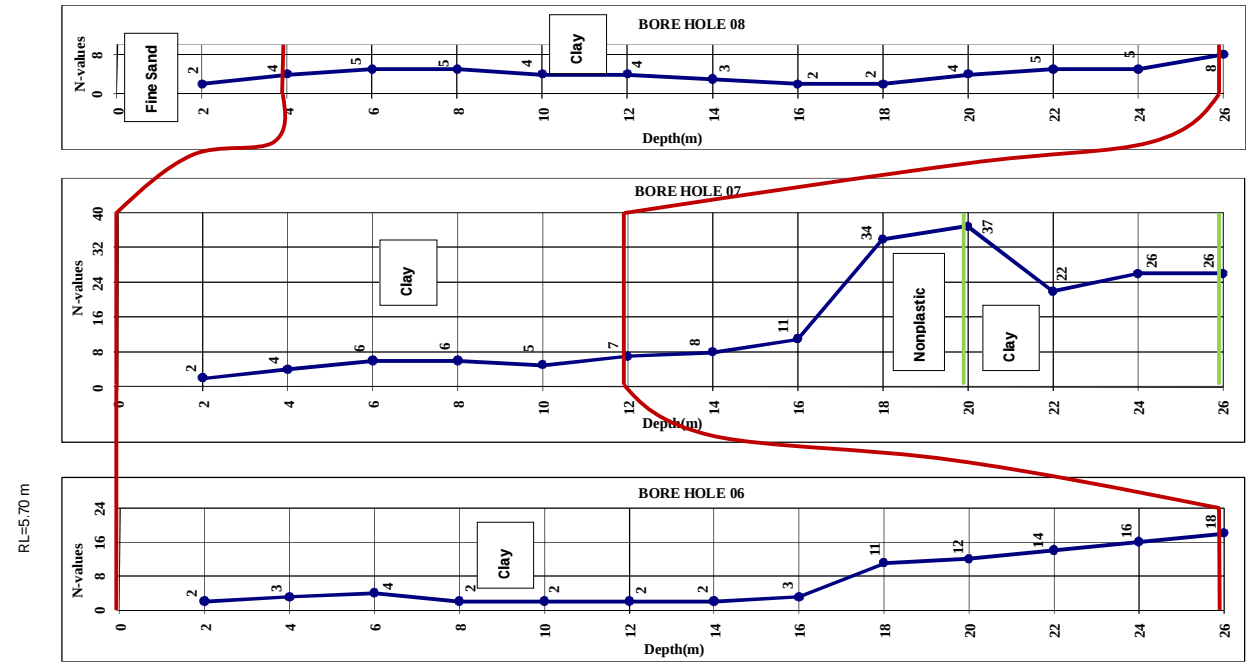


Figure 3.5 Geotechnical Characterization, Section-DD

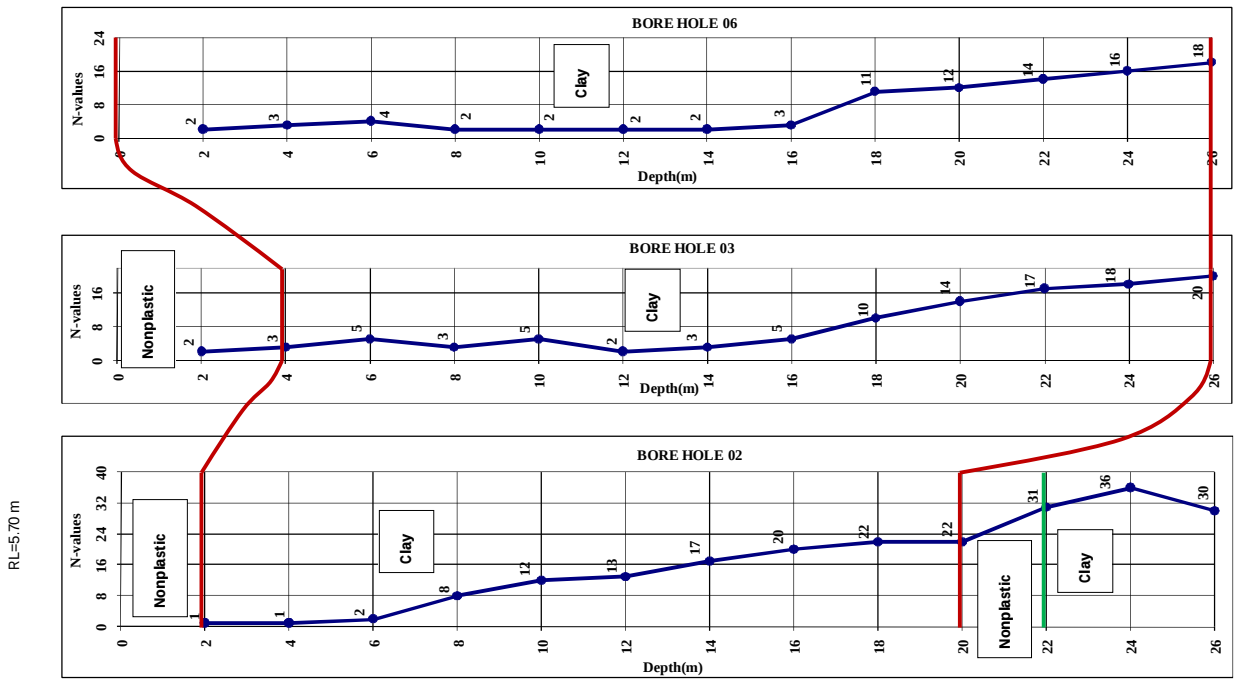


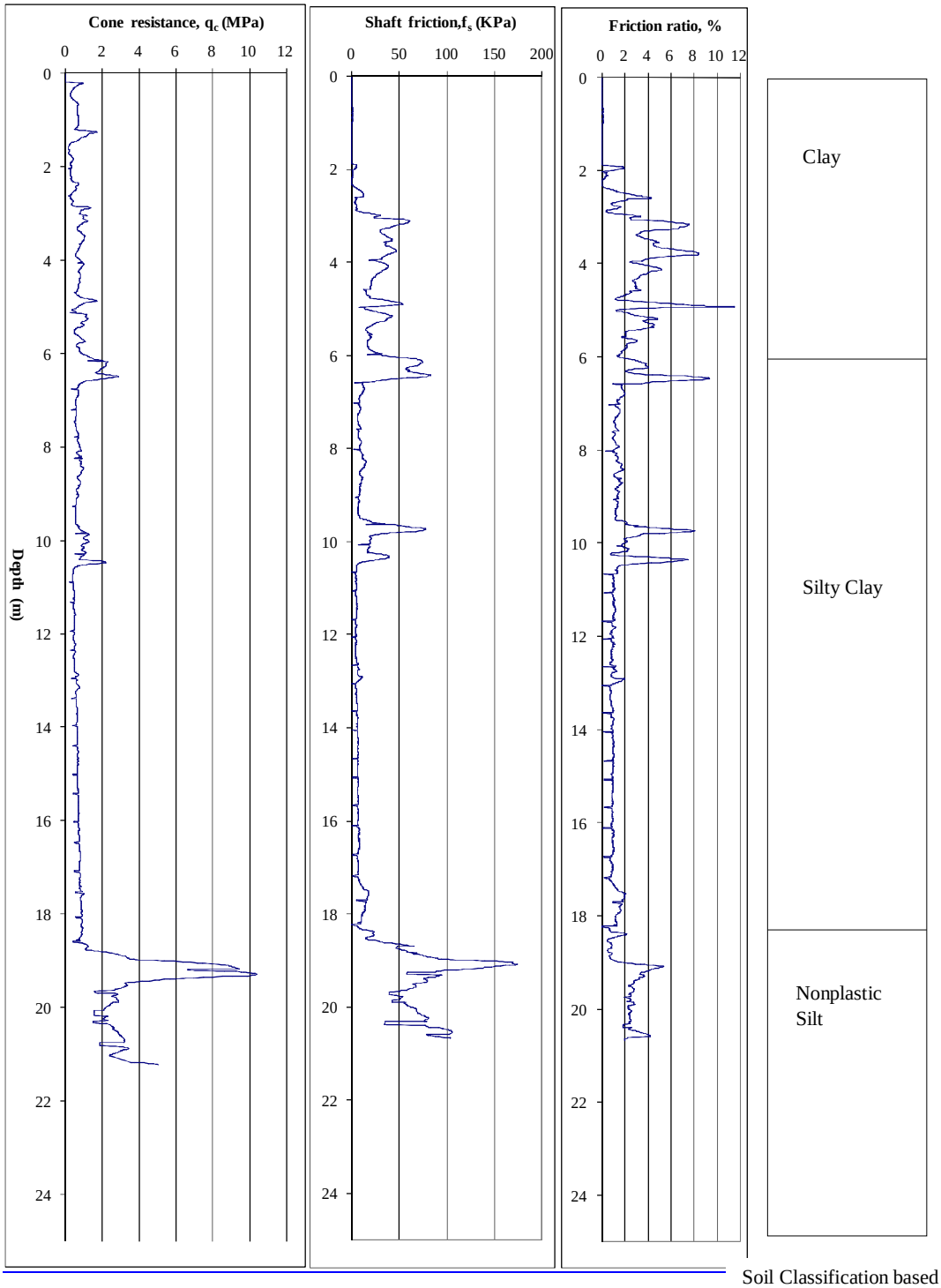
Figure 3.6 Geotechnical Characterization, Section-EE

3.4 SUBSOIL INVESTIGATION BASED ON CPT

Figure 3.7-3.13 presents cone tip resistance, shaft friction, friction ratio and soil classification for seven different locations. The total depth of the soil profile tested for CPT data is variable and the maximum total depth is 22m. From seven CPT points it is seen that the soil of the whole site is erratic and the predominant soil of the site is cohesive soil. For soil profiles as shown in Figures 3.7, 3.10 and 3.12, the plus and minus 18m top soil is cohesive soil (clay, silty clay) and the remaining soil of the profiles is cohesionless soil (nonplastic silt, silty sand). For soil profiles as shown in Figures 3.8, 3.9 and 3.13, the 2m top soil is fine sand and the remaining soil of the profiles is cohesive soil (clay, silty clay) except 1m nonplastic silt existing in the soil profile as shown in Figure 3.13. For soil profile as shown in Figure 3.11, the 2m top soil is fine sand, the bottom soil below 19.5m depth is nonplastic silt and the remaining 17.5m middle soil of the profile is cohesive soil (clay, silty clay).

3.5 INTERPRETATION OF SOIL PROFILE FROM CPT

In this report, soil classification chart by Robertson and Campanella as presented in Article 2.5 is used to identify the soil layers. This method is selected since it is simple and provides output that can be easily understood. Douglas and Olsen chart shows the soil classification change (diagonally) from SP to SM to ML to CL to CH as the cone tip resistance decreases and friction ratio increases. Douglas and Olsen (1981) method demonstrates that the CPT classification charts cannot provide an accurate prediction of soil type based on soil composition, but rather serve as a guide to soil behavior type. Figures 3.7-3.13 present CPT as well as soil classification after Robertson and Campanella.



Soil Classification based on CPT (after Robertson and Campanella)

Figure 3.7 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

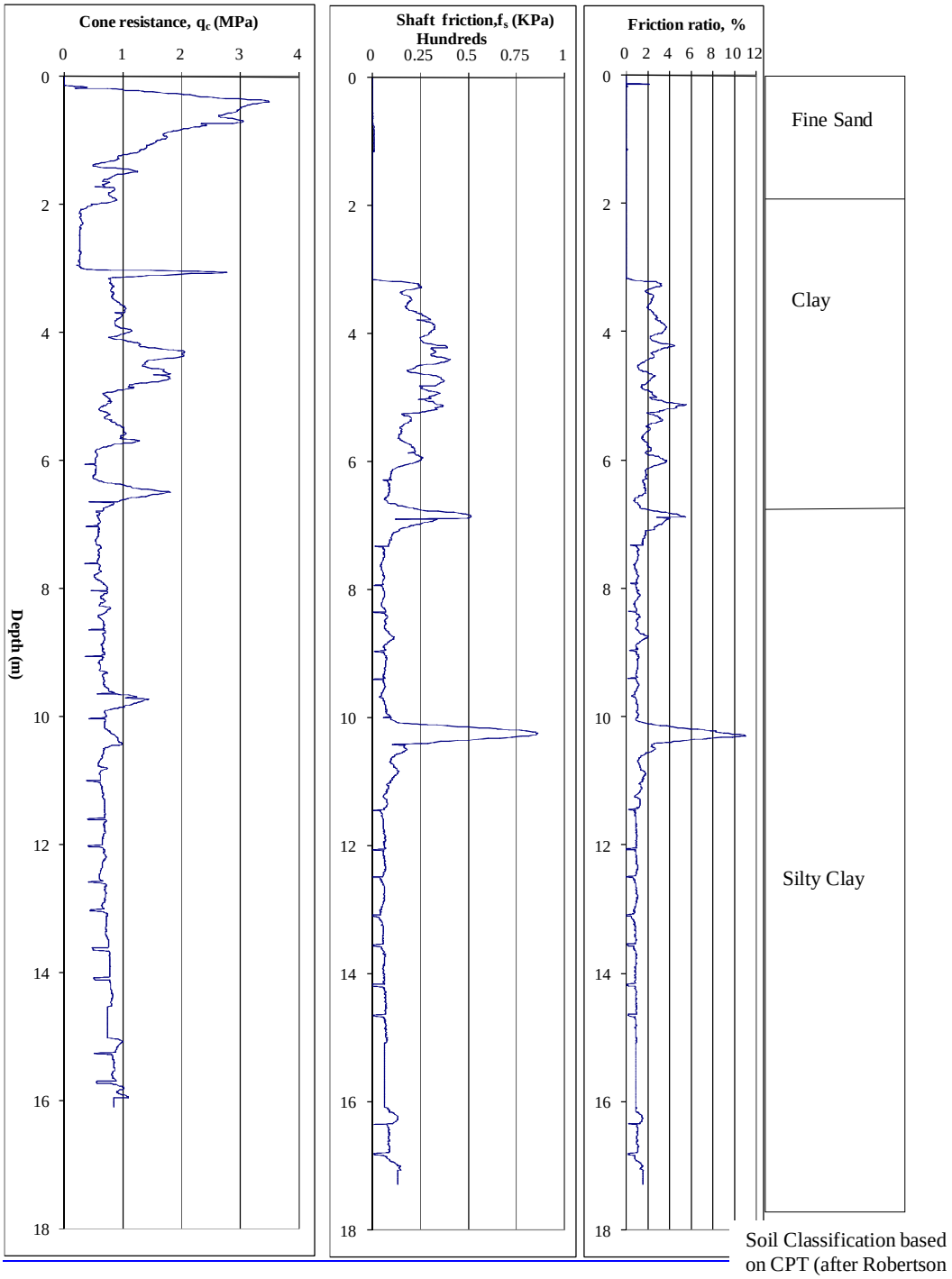


Figure 3.8 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

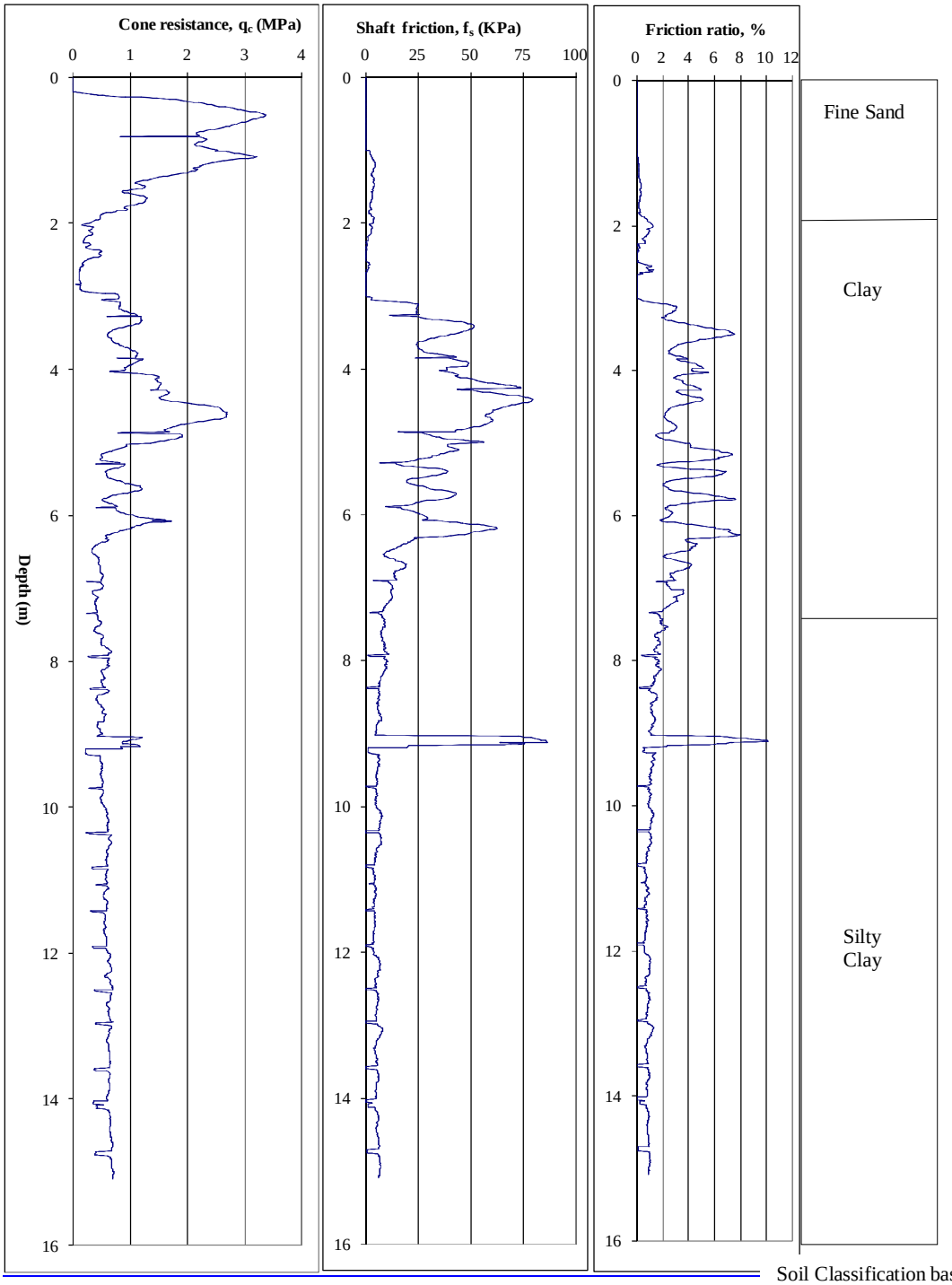


Figure 3.9 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

Soil Classification based on CPT (after Robertson and Campanella)

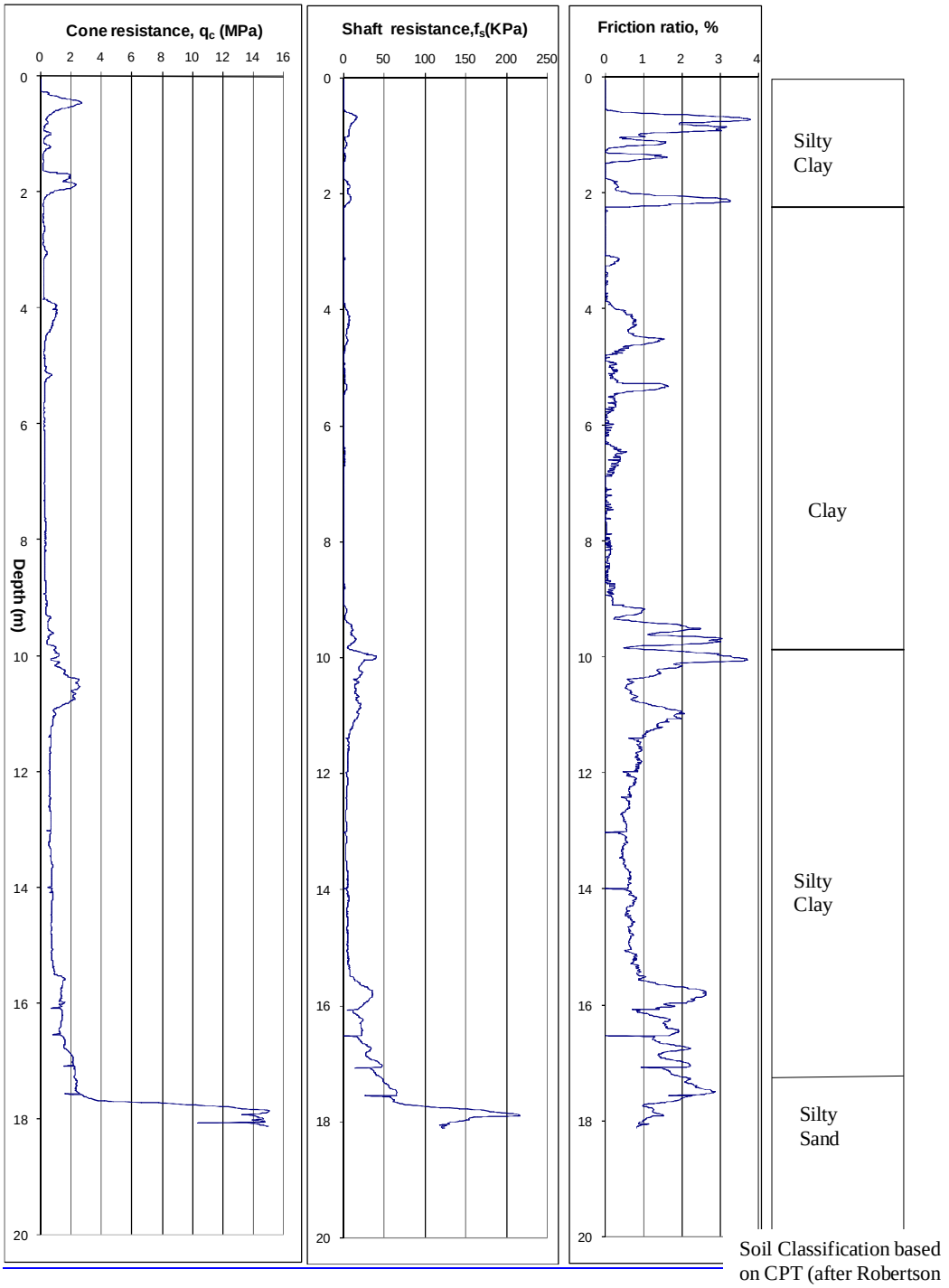


Figure 3.10 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

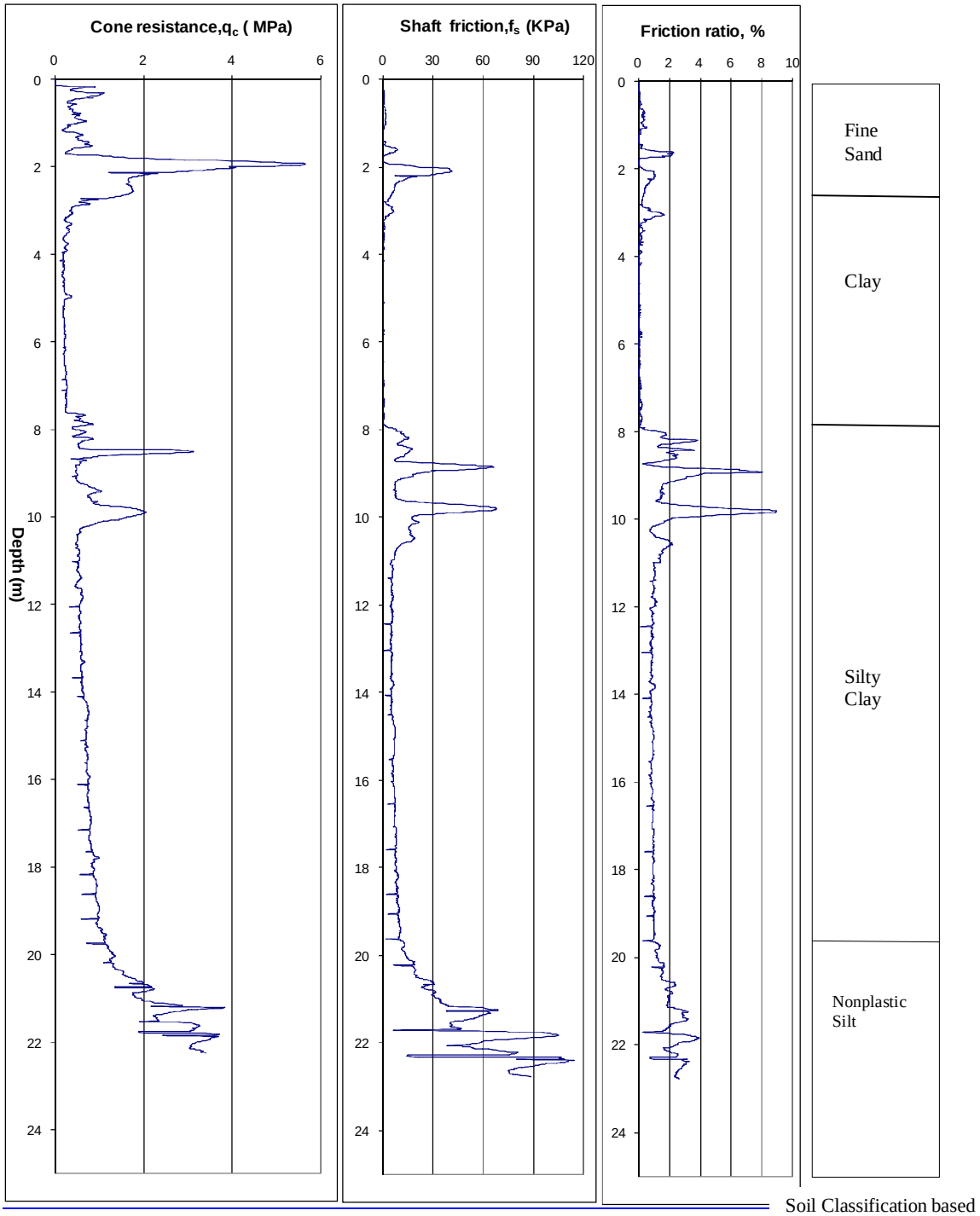


Figure 3.11 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

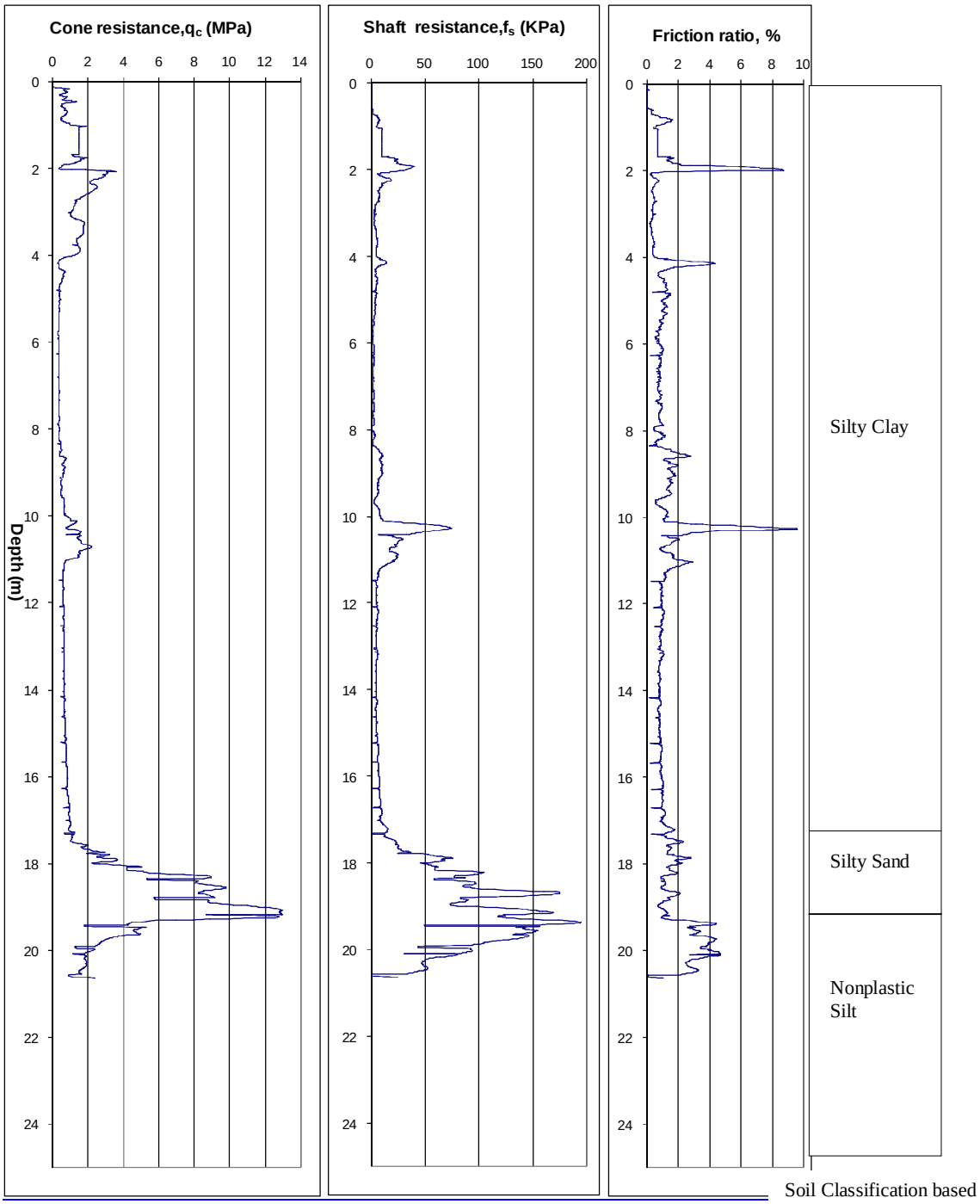


Figure 3.12 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

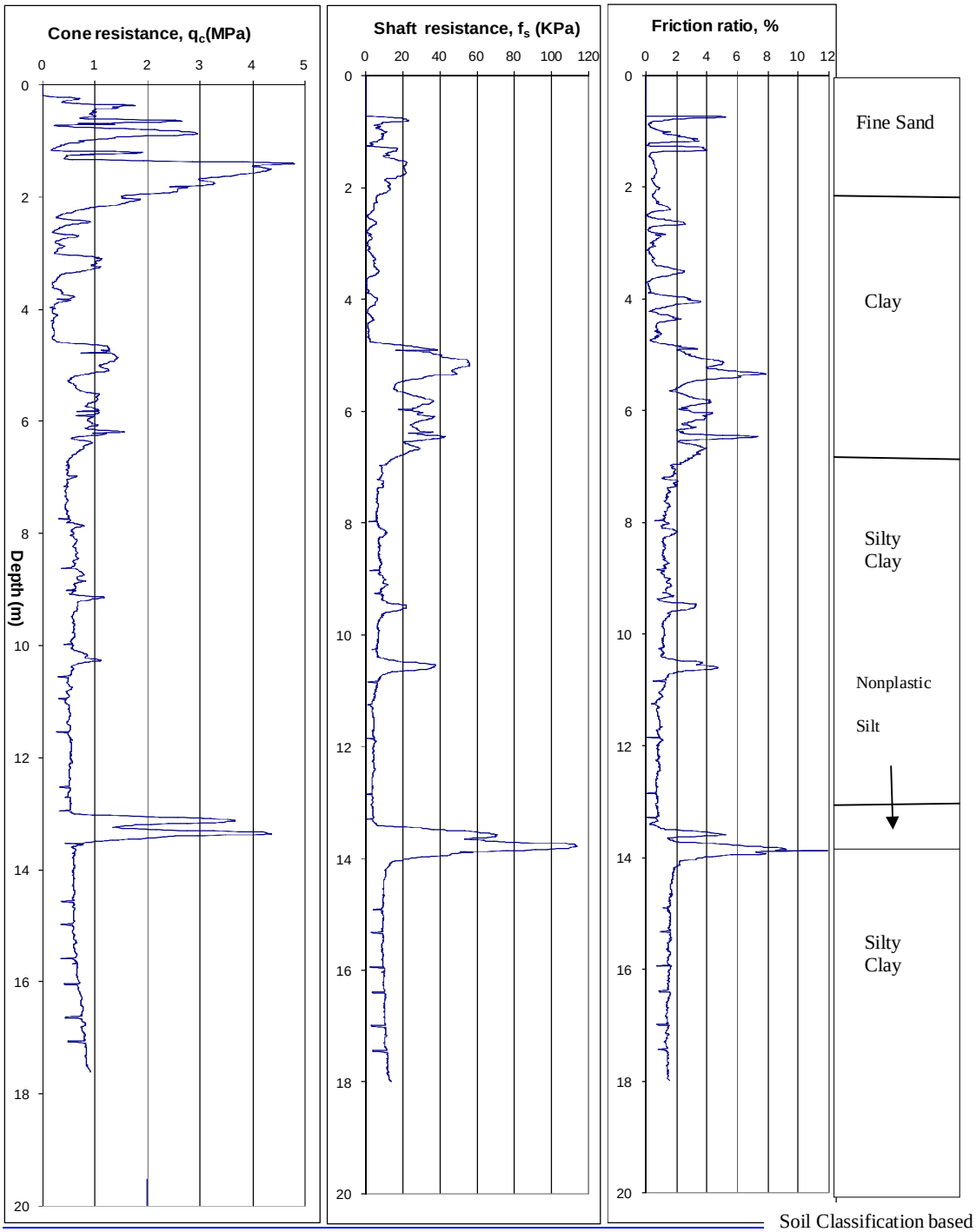


Figure 3.13 Cone Tip Resistance, Shaft Friction and Friction Ratio with Depth

3.6 CHARACTERIZATION OF THE INVESTIGATED PILES

Seven square precast RC concrete piles depending on depth and soil characteristics are considered in the current study. A summary of the characteristics of the investigated piles is presented in Table 3.1. The piles are categorized based on the predominant soil type, pile type and pile splicing. Appendix A shows the square precast RC concrete piles (spliced) before driving and during driving.

Table 3.1: Seven precast piles investigated based on pile type, soil type, and pile splicing

355mm Square Precast Pile Length	Pile Type		Predominant Soil Type			Splicing of Pile	
	Friction	End-bearing	Cohesive	Cohesionless	Both	Yes	No
21m	1	1	1	0	0	1	0
15m	1	0	1	0	0	0	1
14m	1	0	1	0	0	0	1
20.5m	0	1	0	0	1	1	0
22m	1	0	1	0	0	1	0
20m	1	0	1	0	0	1	0
17m	1	0	1	0	0	0	1

Note: 1 = Yes, 0=No

3.7 PREDICTED PILE CAPACITY USING SPT AND CPT DATA

Only seven piles depending on depth and soil characteristics are used in the analyses and most of the piles are identified as friction piles. To predict the ultimate axial load capacity of the piles, the methods used are the Schmertmann, de Ruiter and Beringen, Bustamante and Gianeselli, Tumay and Fakhroo, Aoki and De Alencar, Price and Wardle, Philipponnat, and Penpile method. The ultimate load capacity for each pile is also predicted from the soil properties (soil boring close to the pile) using the SPT data. The ultimate load carrying capacity predicted by these methods (Q_p) using the CPT data is compared to the pile capacity (Q_m) obtained from the SPT data. Table 3.2 summarizes the results of the analyses conducted on the investigated piles. Among the data presented in Table 3.2 are: the pile size, type, length, the predicted ultimate load carrying capacity from CPT, the predicted ultimate load carrying capacity from SPT, average ultimate pile capacity for each pile and standard deviation for each pile. Appendix B presents an illustration of predicting the pile capacity by different CPT methods and the SPT method.

The predicted ultimate load carrying capacity (Q_p) consists of pile tip capacity (Q_t) and pile shaft resistance (Q_s). Comparison between the pile capacities Q_t, Q_s, and Q_p predicted by the CPT methods and the pile capacities Q_t, Q_s, and Q_m predicted by the SPT method are shown in Figures 3.14-3.21. The results of five friction piles, one friction and end-bearing pile and one end-bearing pile are shown in these figures. Inspection of these figures shows that the ratio Q_t/Q_p for the 5 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 1 pile is relatively large and hence this pile is considered as end bearing pile (pile capacity is derived mainly from the pile tip capacity). The ratio Q_t/Q_p for 1 pile is almost one and hence this pile is considered as friction and end bearing pile (pile capacity is derived mainly from the shaft resistance and pile tip capacity). These figures also find out the method giving the maximum/minimum ultimate end bearing capacity, the maximum/minimum ultimate shaft friction capacity, the maximum /minimum total ultimate pile capacity for seven piles.

Table 3.2: Results of the analyses conducted on square reinforced concrete piles at a site in Siddhirganj.

Pile and Soil Identification	Pile ID	TP1, 355mm Square Precast RC Concrete Pile (Spliced)			TP2, 355mm Square Precast RC Concrete Pile			Increase or decrease of pile capacity compared with arithmetic mean (in %)	
	Pile Length(m)	21			15				
	Embedded Length(m)	21			15				
	Pile Classification	Friction and end bearing			Friction				
	Predominant Soil	Cohesive			Cohesive				
Methods of Predicting Pile Capacity by Cone Penetration Test (CPT)	Predicted Ultimate Load (KN)	Q_s	Q_t	**Q_u**	Q_s	Q_t	**Q_u**	TP1	TP2
	Schmertmann	453	450	**903**	240	94	**334**	2	-25
	de Ruiter & Beringen	478	450	**928**	394	56	**450**	5	1
	LCPC	905	221	**1126**	673	62	**735**	28	64
	Tumay & Fakhroo	1028	417	**1445**	722	93	**815**	64	82
	Aoki & De Alencar	257	303	**560**	261	54	**315**	-37	-30
	Price & Wardle	263	157	**420**	129	33	**162**	-52	-64
	Philipponnat	528	210	**738**	397	50	**447**	-16	0
	Penpile	280	56	**336**	142	23	**165**	-62	-63
Pile Capacity by SPT		847	630	**1477**	540	61	**601**	67	34
Arithmetic Mean		**881**			**447**				
Standard Deviation		**390**			**219**				

Q_s: Pile shaft capacity (friction), Q_t : Pile tip capacity (end-bearing), Q_u: Total ultimate capacity(Q_s+Q_t)

Table 3.2: Results of the analyses conducted on square reinforced concrete piles at a site in Siddhirganj, continued

Pile and Soil Identification	Pile ID	TP3, 355mm Square Precast RC Concrete Pile (Spliced)			TP4, 355mm Square Precast RC Concrete Pile (Spliced)			Increase or decrease of pile capacity compared with arithmetic mean (in %)	
	Pile Length(m)	14			20.5				
	Embedded Length(m)	14			20.5				
	Pile Classification	Friction			End bearing				
	Predominant Soil	Cohesive			Cohesive & Cohesionless				
Methods of Predicting Pile Capacity by Cone Penetration Test (CPT)	Predicted Ultimate Load (KN)	Q_s	Q_t	**Q_u**	Q_s	Q_t	**Q_u**	TP3	TP4
	Schmertmann	239	78	**317**	537	1628	**2165**	-18	36
	de Ruiter & Beringen	398	47	**445**	698	1628	**2326**	14	46
	LCPC	560	48	**608**	1244	699	**1943**	56	22
	Tumay & Fakhroo	647	75	**722**	480	1628	**2108**	86	32
	Aoki & De Alencar	188	45	**233**	615	930	**1545**	-40	-3
	Price & Wardle	137	27	**164**	335	570	**905**	-58	-43
	Philipponnat	338	40	**378**	771	746	**1517**	-3	-5
	Penpile	149	19	**168**	307	203	**510**	-57	-68
Pile Capacity by SPT		437	20	**457**	819	516	**1335**	18	-16
Arithmetic Mean		**388**			**1595**				
Standard Deviation		**181**			**575**				

Q_s: Pile shaft capacity (friction), Q_t : Pile tip capacity (end-bearing), Q_u: Total ultimate capacity(Q_s+Q_t)

Table 3.2: Results of the analyses conducted on square reinforced concrete piles at a site in Siddhirganj, continued

Pile and Soil Identification	Pile ID	TP5, 355mm Square Precast RC Concrete Pile			TP6, 355mm Square Precast RC Concrete Pile			TP7, 355mm Square Precast RC Concrete Pile			Increase or decrease of pile capacity compared with arithmetic mean (in %)		
	Pile Length(m)	22			20			17					
	Embedded Length(m)	22			20			17					
	Pile Classification	Friction			Friction			Friction					
	Predominant Soil	Cohesive			Cohesive			Cohesive					
Methods of Predicting Pile Capacity by Cone Penetration Test (CPT)	Predicted Ultimate Load (KN)	Q_s	Q_t	**Q_u**	Q_s	Q_t	**Q_u**	Q_s	Q_t	**Q_u**	TP5	TP6	TP7
	Schmertmann	333	321	**654**	497	434	**931**	318	98	**416**	-10	-5	-15
	de Ruiter & Beringen	435	321	**756**	663	434	**1097**	508	59	**567**	5	12	15
	LCPC	738	164	**902**	1239	272	**1511**	667	42	**709**	25	55	44
	Tumay & Fakhroo	909	310	**1219**	1045	406	**1451**	1036	90	**1126**	69	49	129
	Aoki & De Alencar	316	183	**499**	381	248	**629**	260	56	**316**	-31	-36	-36
	Price & Wardle	189	112	**301**	389	152	**541**	142	34	**176**	-58	-45	-64
	Philipponnat	470	148	**618**	673	278	**951**	380	44	**424**	-15	-3	-14
	Penpile	206	40	**246**	387	108	**495**	175	24	**199**	-66	-49	-59
Pile Capacity by SPT		749	554	**1303**	697	479	**1176**	461	20	**481**	80	20	-2
Arithmetic Mean		**722**			**976**			**490**					
Standard Deviation		**347**			**351**			**275**					

Q_s: Pile shaft capacity (friction), Q_t: Pile tip capacity (end-bearing), Q_u: Total ultimate capacity(Q_s+Q_t)

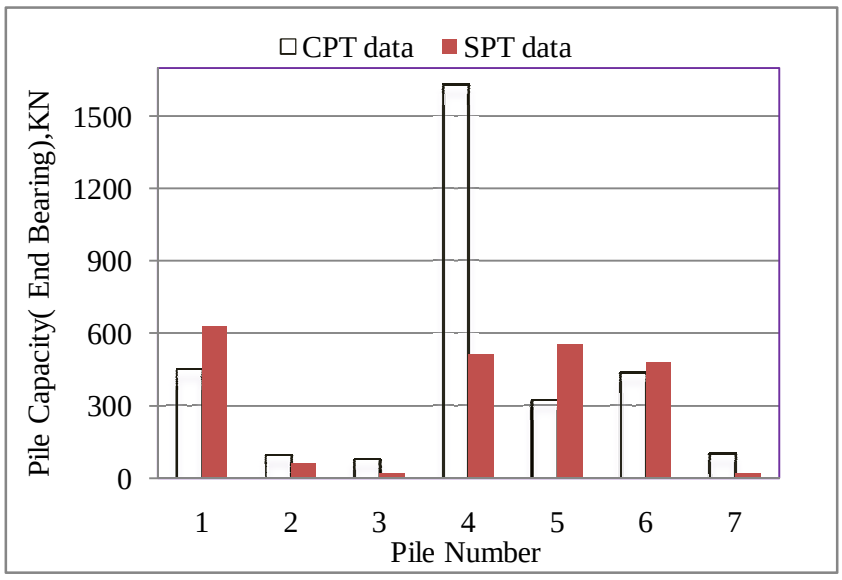


Figure 3.14a Comparison of ultimate end bearing capacity of pile predicted by Schmertmann method(1978) with the one predicted by SPT data

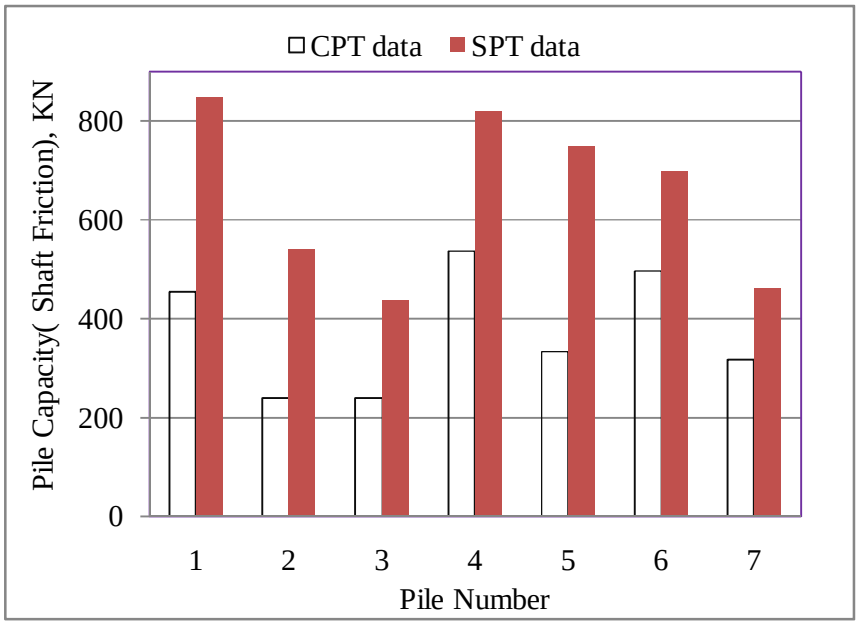


Figure 3.14b Comparison of ultimate shaft friction capacity of pile predicted by Schmertmann method(1978) with the one predicted by SPT data

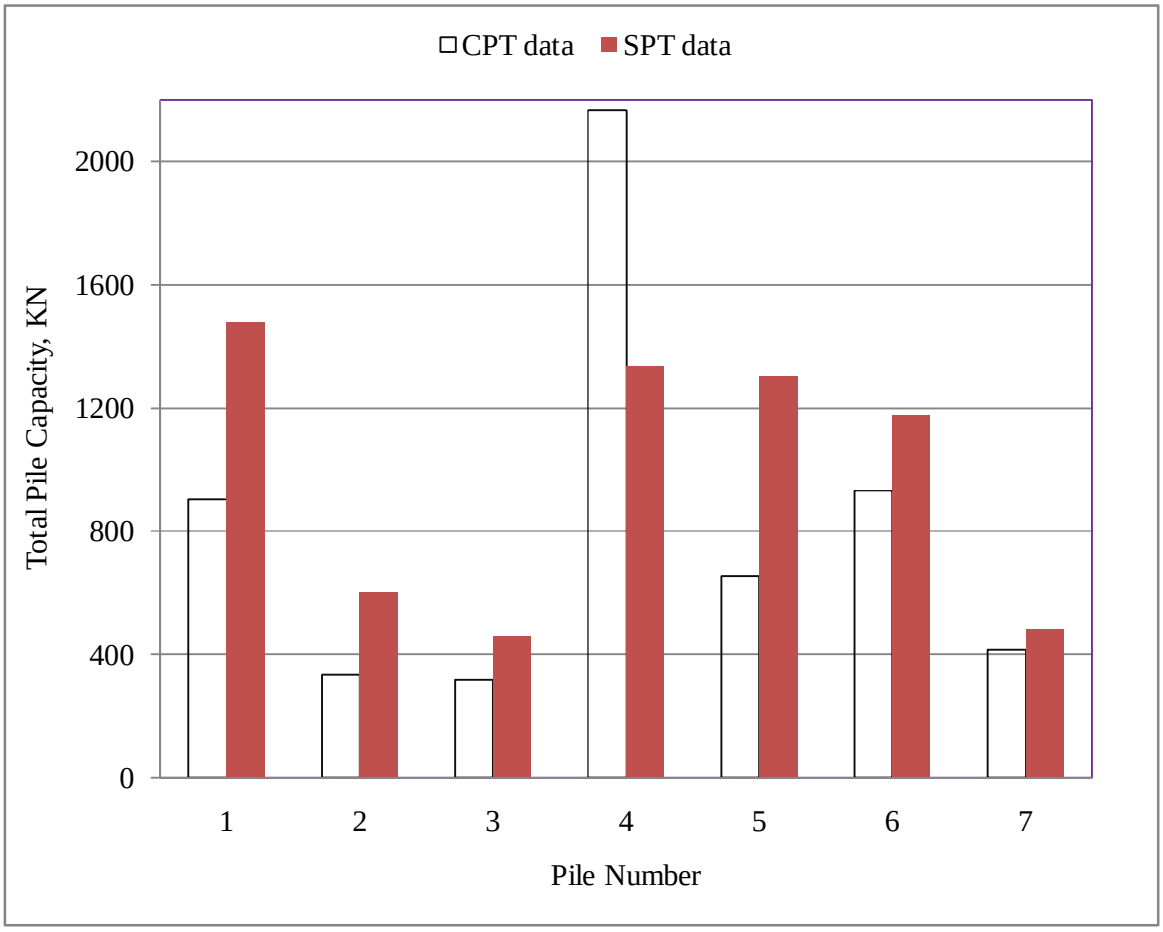


Figure 3.14c Comparison of ultimate pile capacity predicted by Schmertmann method(1978) with the one predicted by SPT data

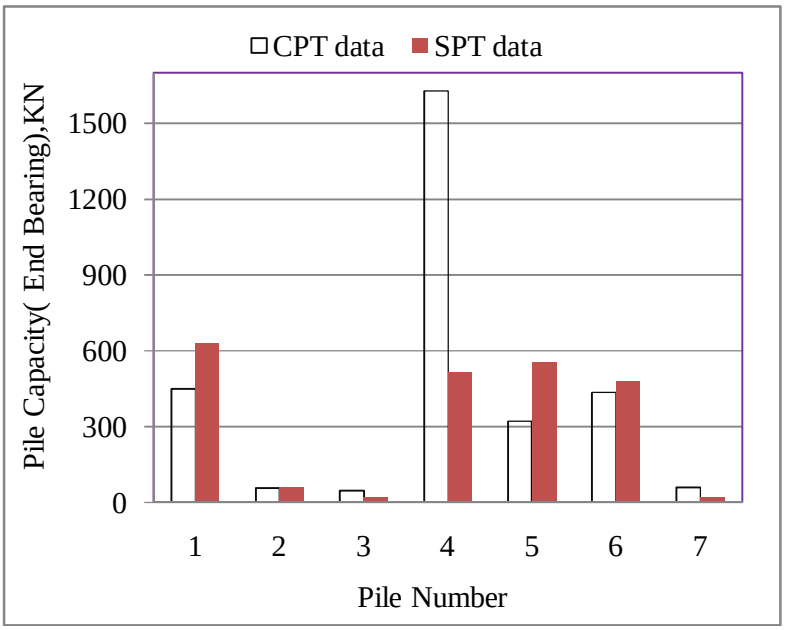


Figure 3.15a Comparison of ultimate end bearing capacity of pile predicted by de Ruiter and Beringen method (1979) with the one predicted by SPT data

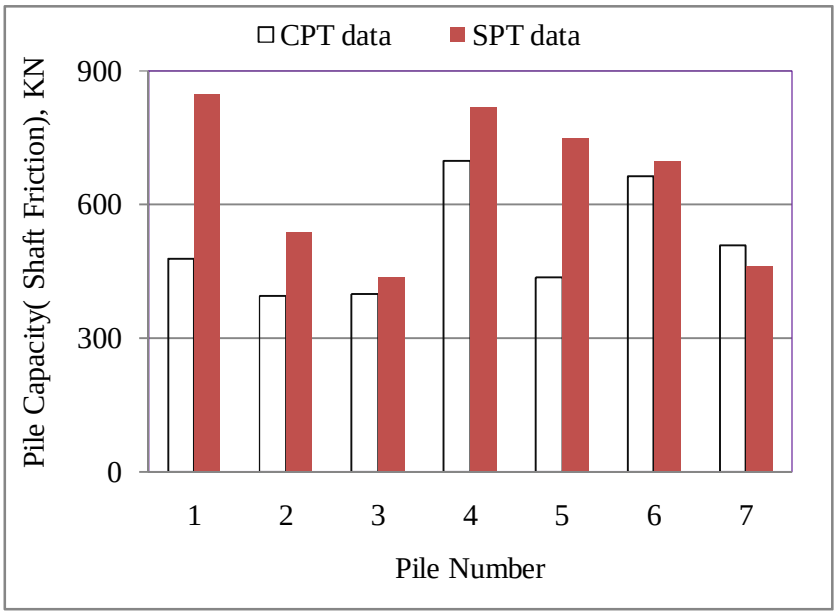


Figure 3.15b Comparison of ultimate shaft friction capacity of pile predicted by de Ruiter and Beringen method (1979) with the one predicted by SPT data

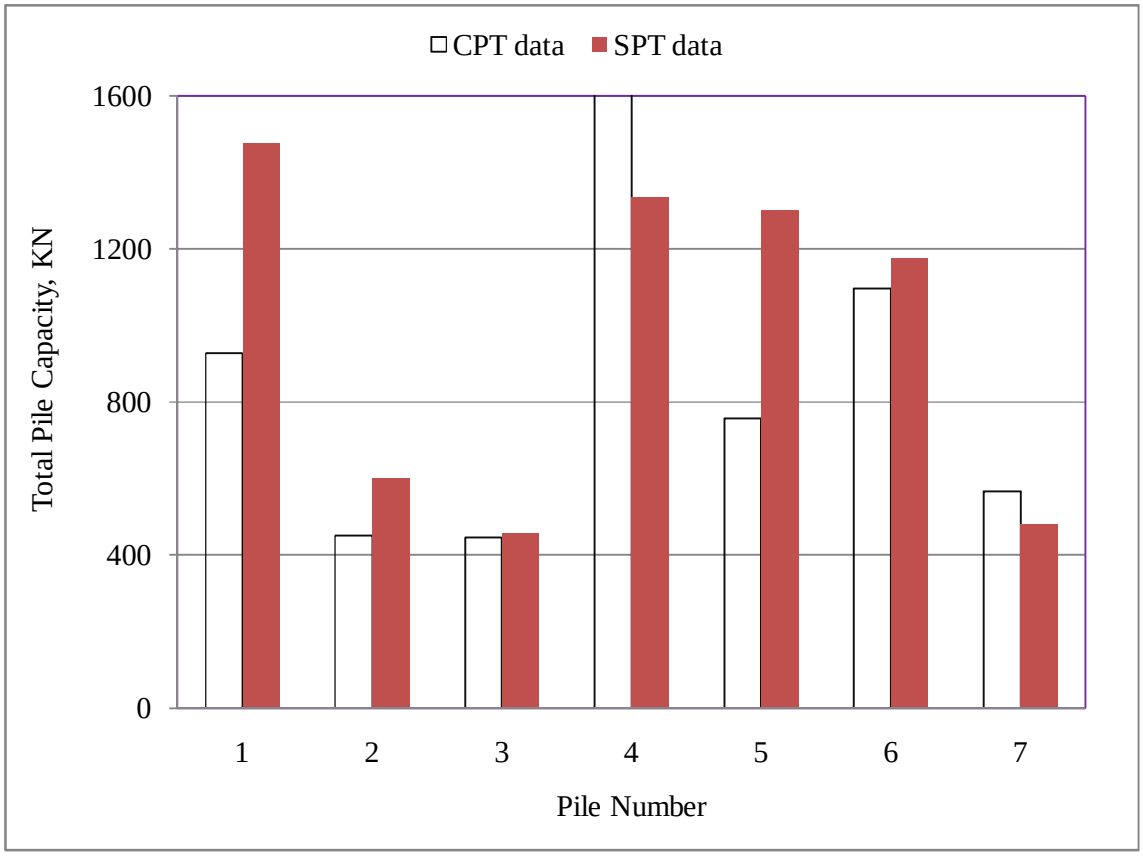


Figure 3.15c Comparison of ultimate pile capacity predicted by de Ruiter and Beringen method(1979) with the one predicted by SPT data

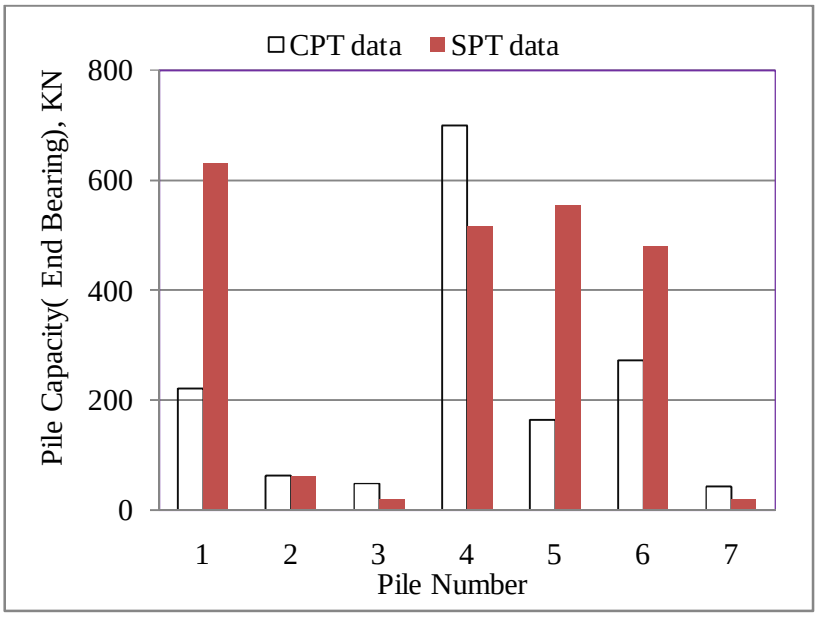


Figure 3.16a Comparison of ultimate end bearing capacity of pile predicted by LCPC method (1982) with the one predicted by SPT data

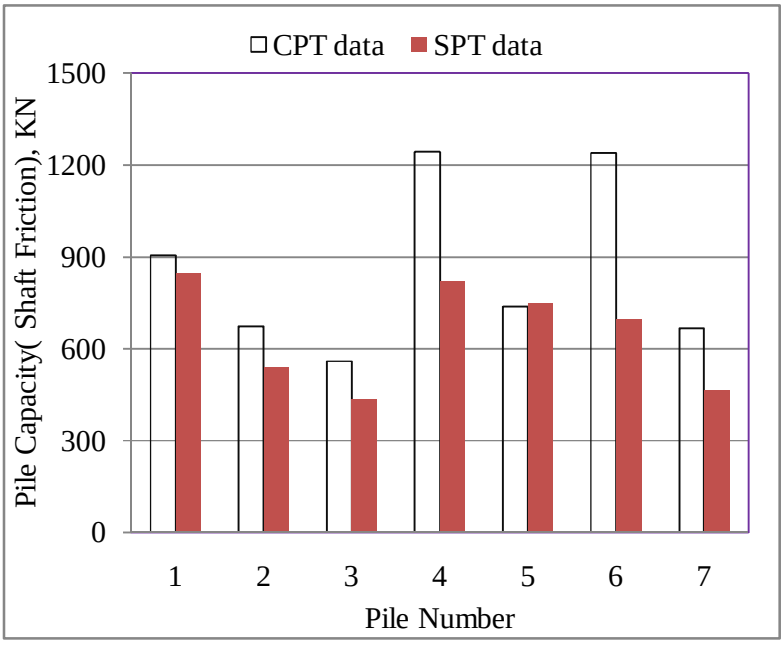


Figure 3.16b Comparison of ultimate shaft friction capacity of pile predicted by LCPC method (1982) with the one predicted by SPT data

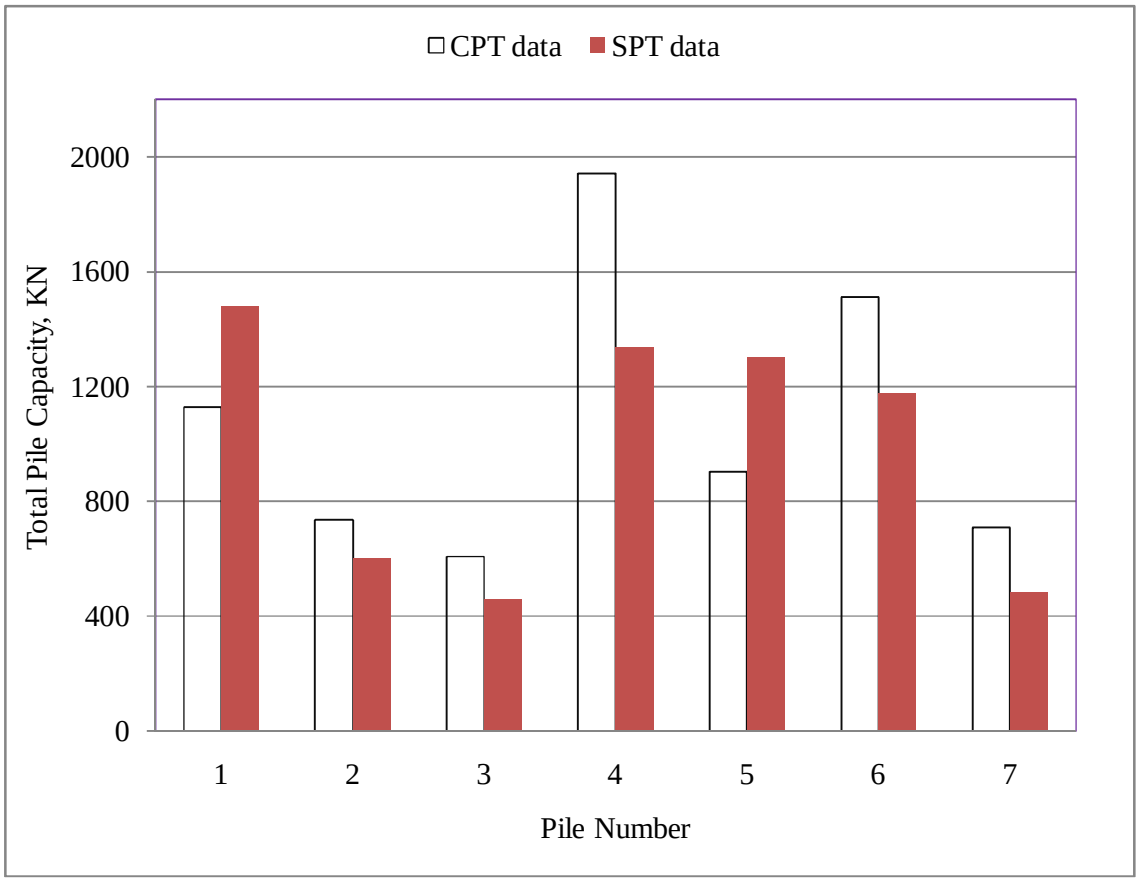


Figure 3.16c Comparison of ultimate pile capacity predicted by LCPC method (1982) with the one predicted by SPT data

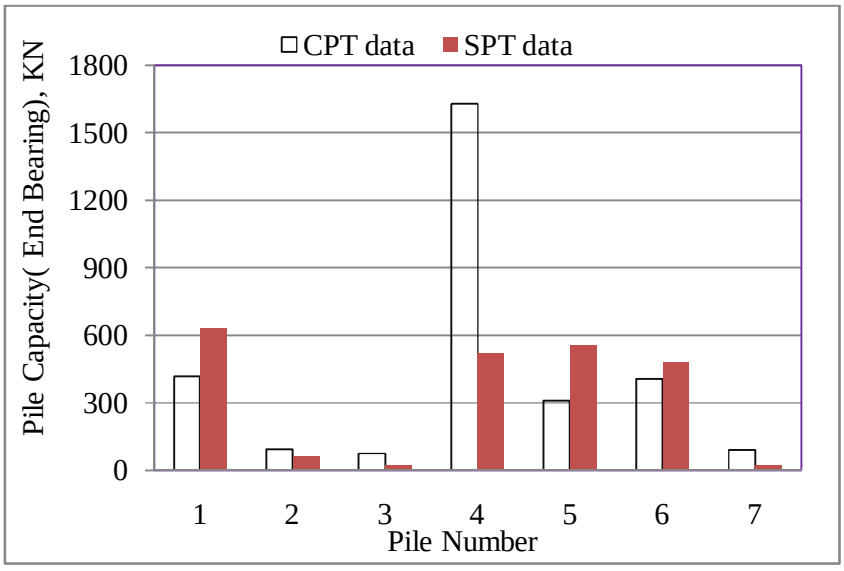


Figure 3.17a Comparison of ultimate end bearing capacity of pile predicted by Tumay and Fakhroo method (1982) with the one predicted by SPT data

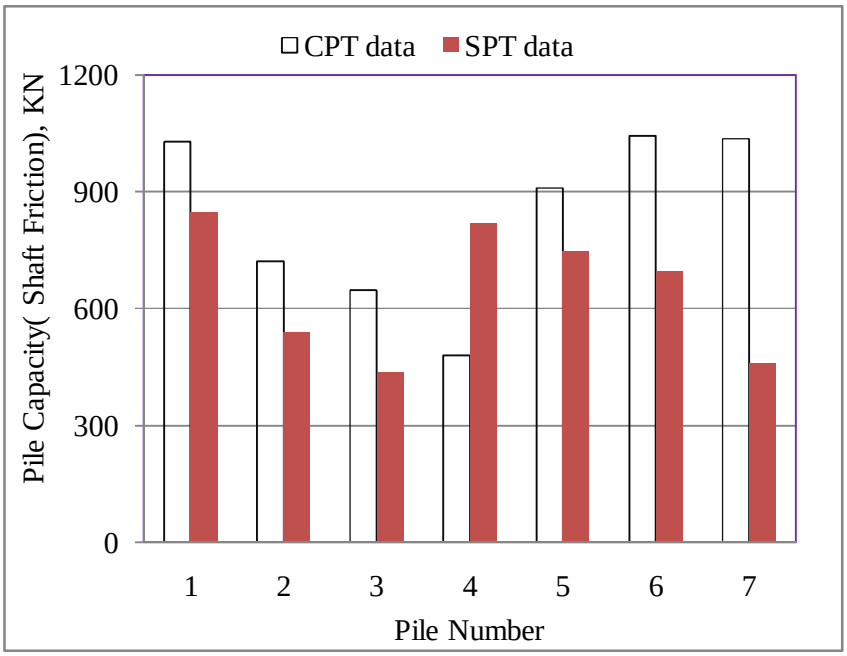


Figure 3.17b Comparison of ultimate shaft friction capacity of pile predicted by Tumay and Fakhroo method (1982) with the one predicted by SPT data

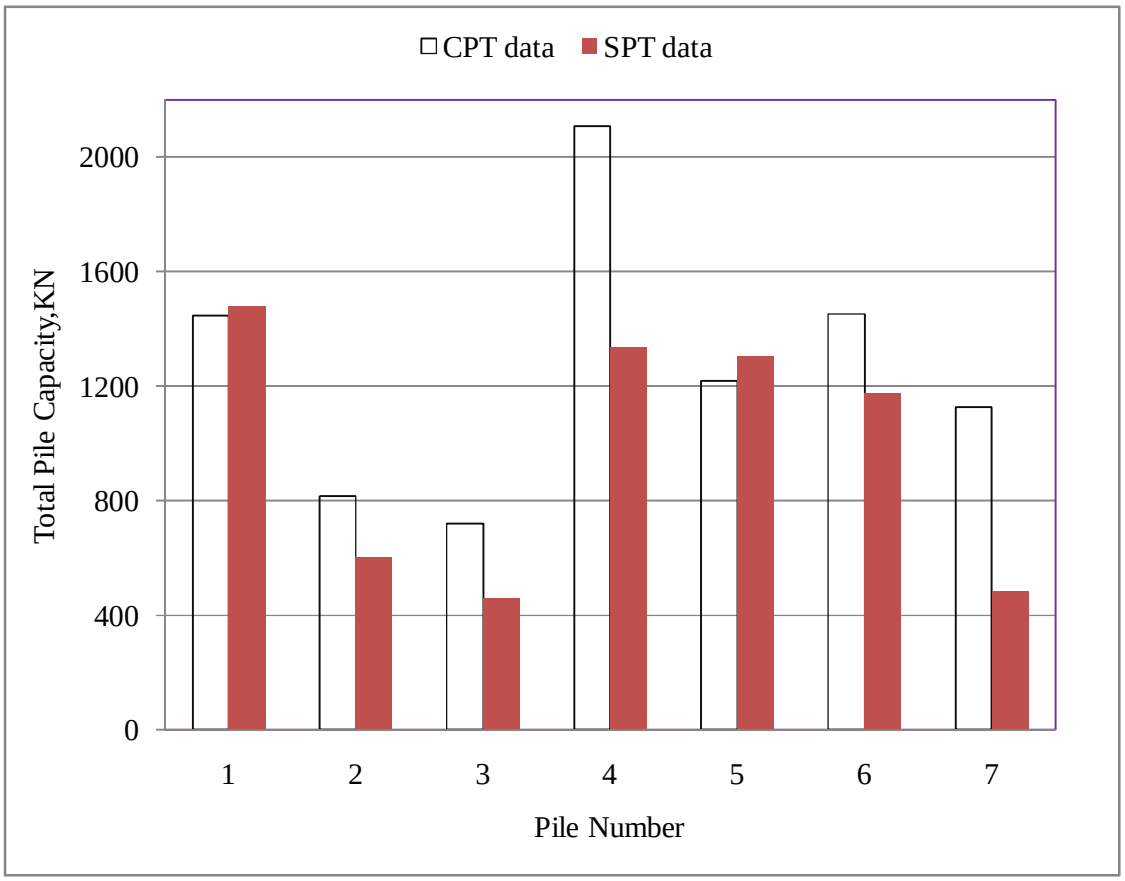


Figure 3.17c Comparison of ultimate pile capacity predicted by Tumay and Fakhroo method (1982) with the one predicted by SPT data

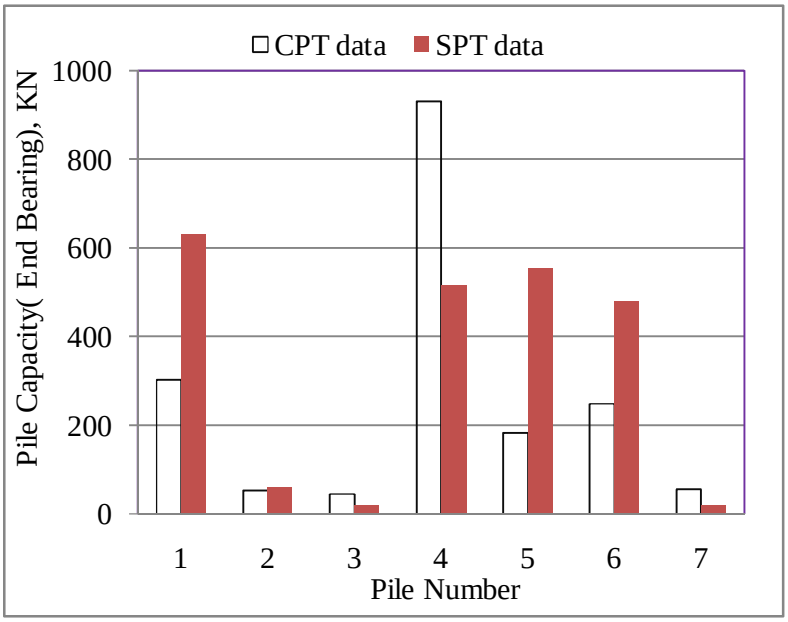


Figure 3.18a Comparison of ultimate end bearing capacity of pile predicted by Aoki and De Alencar method (1975) with the one predicted by SPT data

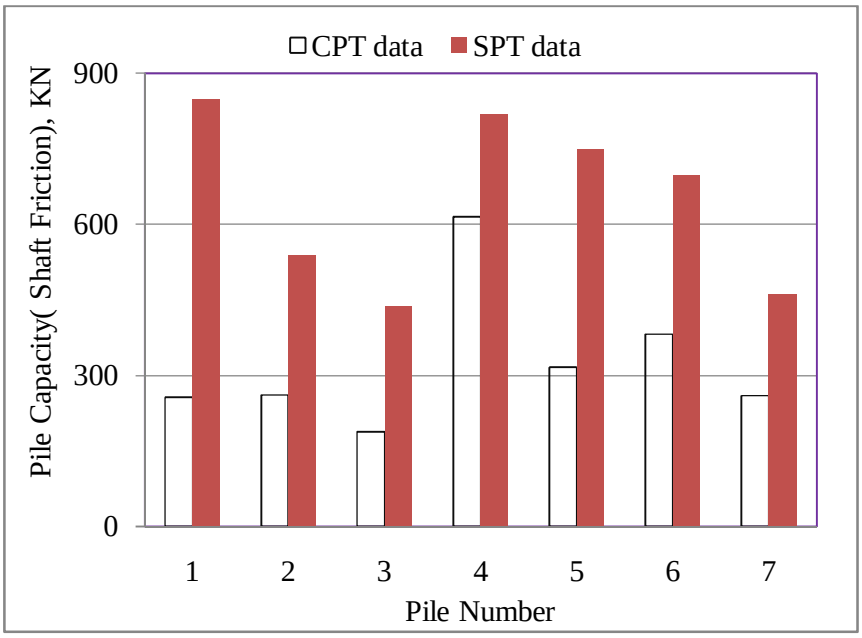


Figure 3.18b Comparison of ultimate shaft friction capacity of pile predicted by Aoki and De Alencar method (1975) with the one predicted by SPT data

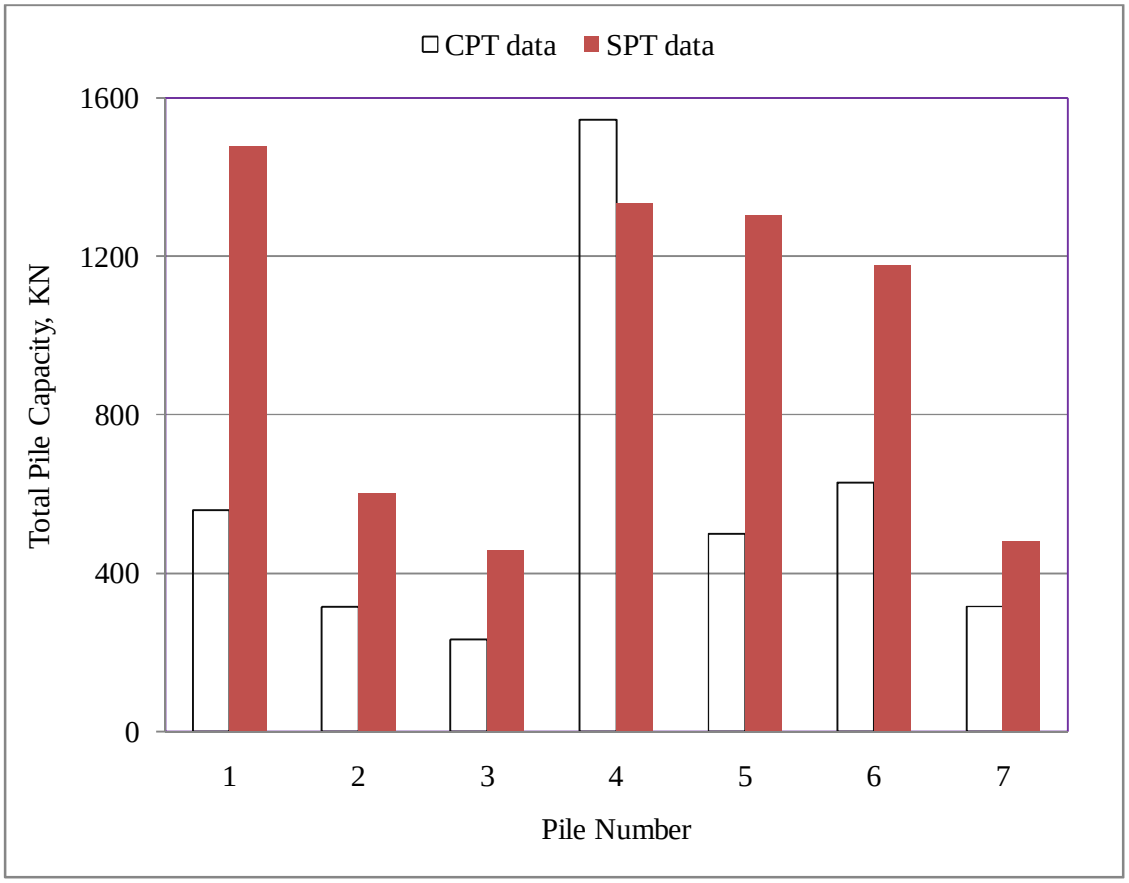


Figure 3.18c Comparison of ultimate pile capacity predicted by Aoki and De Alencar method (1975) with the one predicted by SPT data

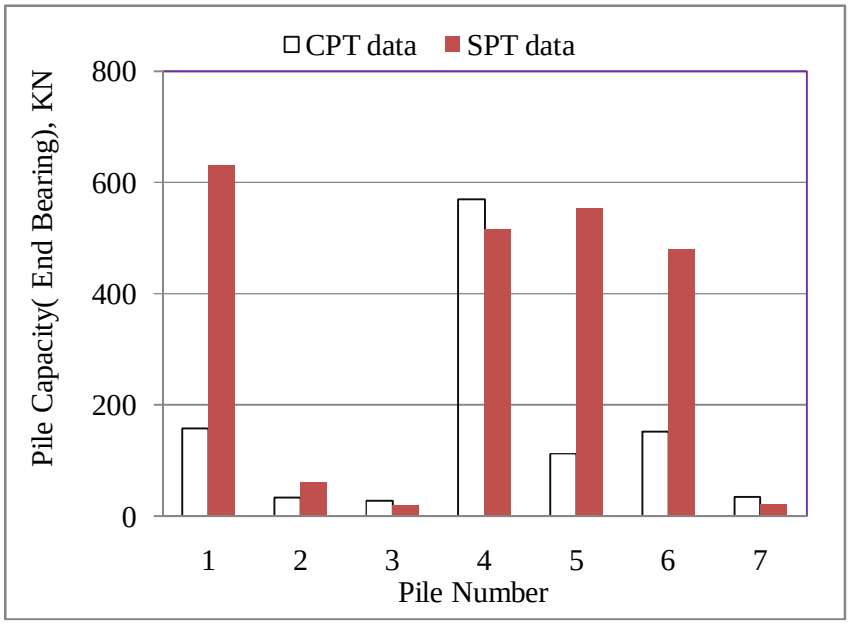


Figure 3.19a Comparison of ultimate end bearing capacity of pile predicted by Price and Wardle method (1982) with the one predicted by SPT data

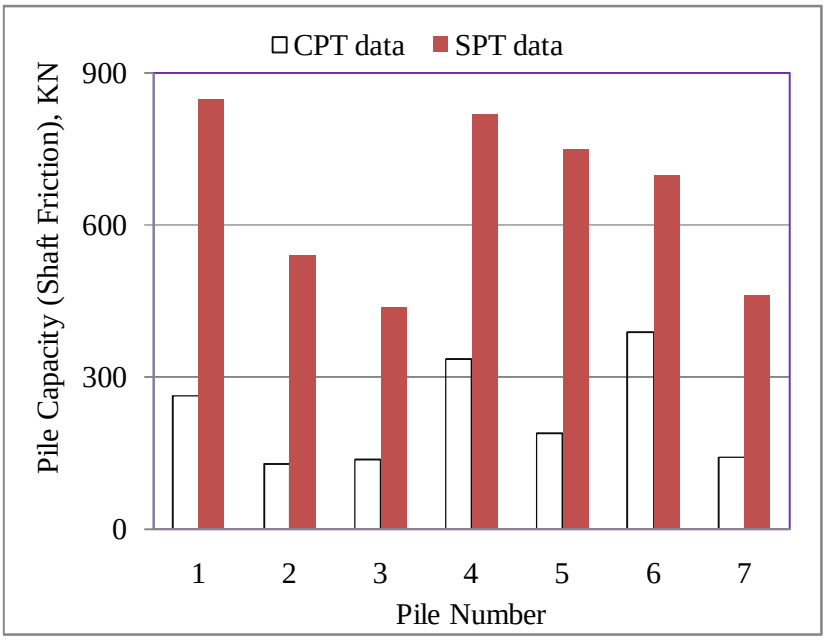


Figure 3.19b Comparison of ultimate shaft friction capacity of pile predicted by Price and Wardle method (1982) with the one predicted by SPT data

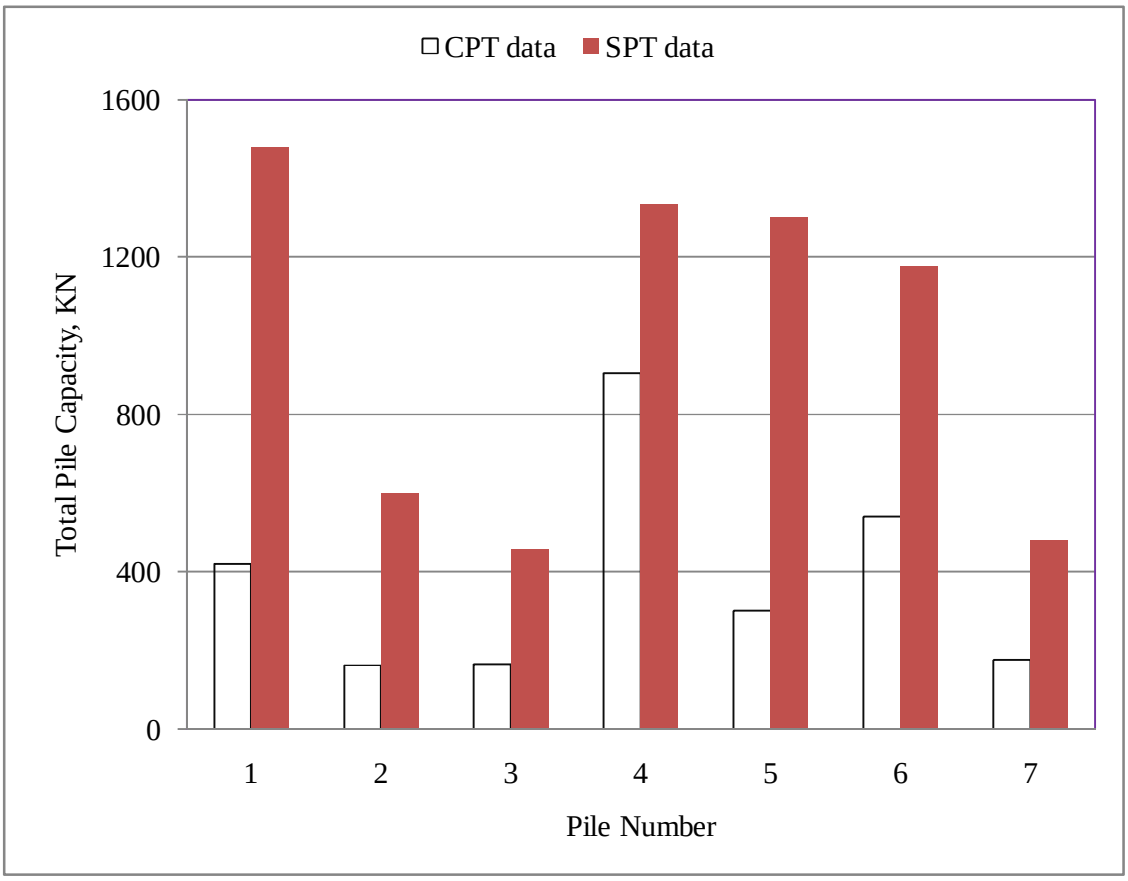


Figure 3.19c Comparison of ultimate pile capacity predicted by Price and Wardle method (1982) with the one predicted by SPT data

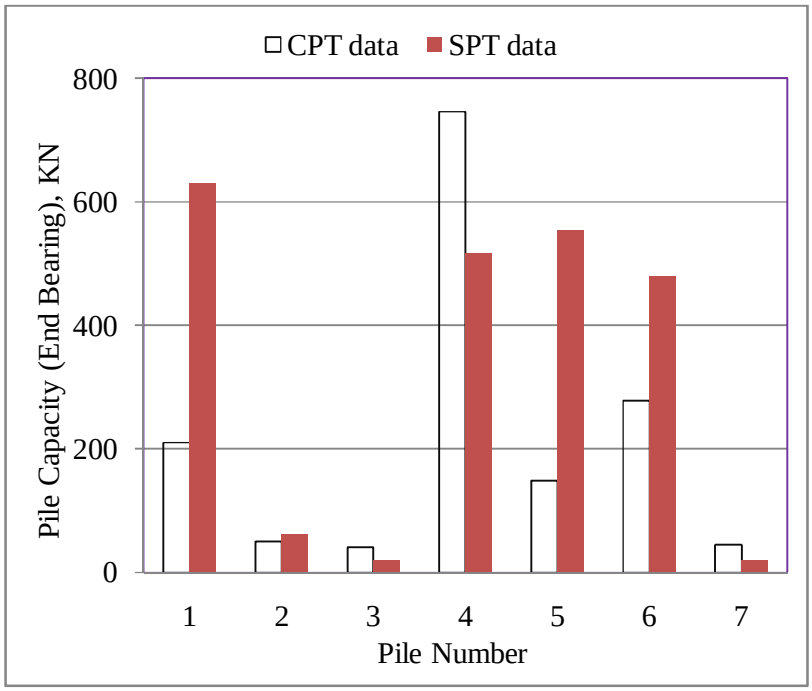


Figure 3.20a Comparison of ultimate end bearing capacity of pile predicted by Philipponnat method (1980) with the one predicted by SPT data

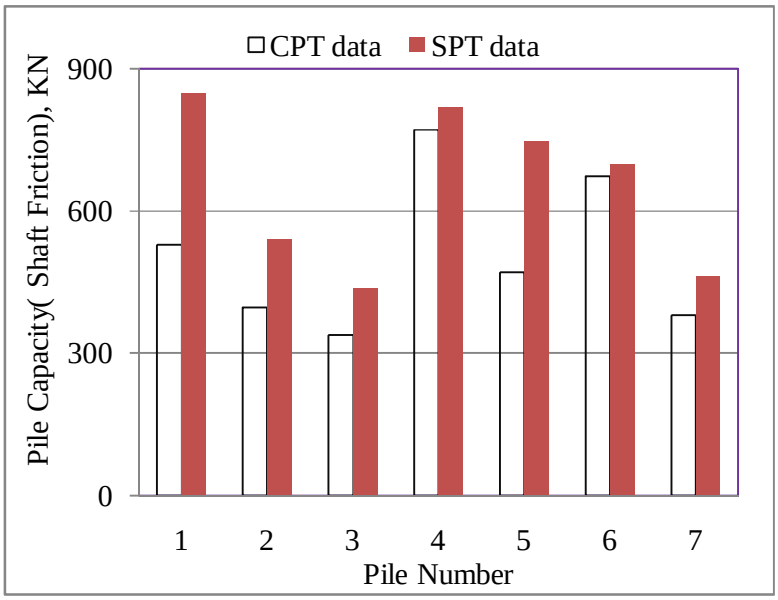


Figure 3.20b Comparison of ultimate shaft friction capacity of pile predicted by Philipponnat method (1980) with the one predicted by SPT data

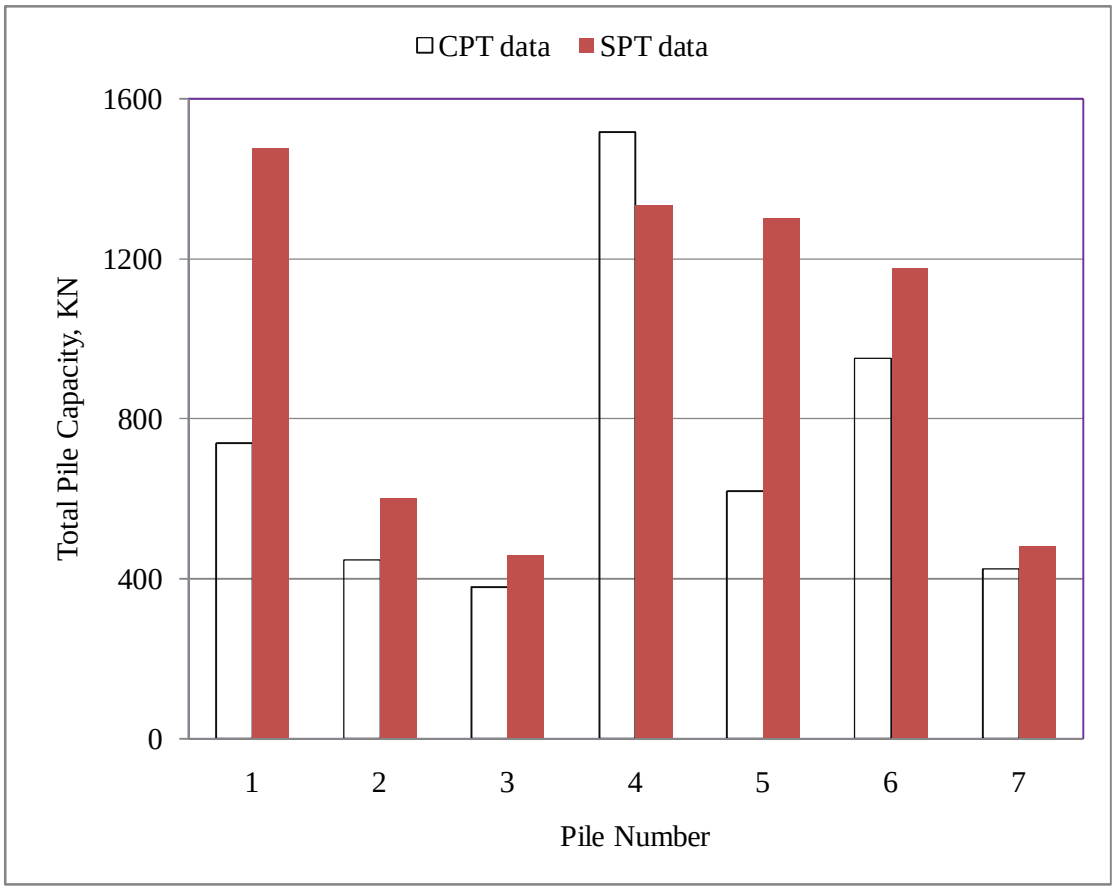


Figure 3.20c Comparison of ultimate pile capacity predicted by Philipponnat method (1980) with the one predicted by SPT data

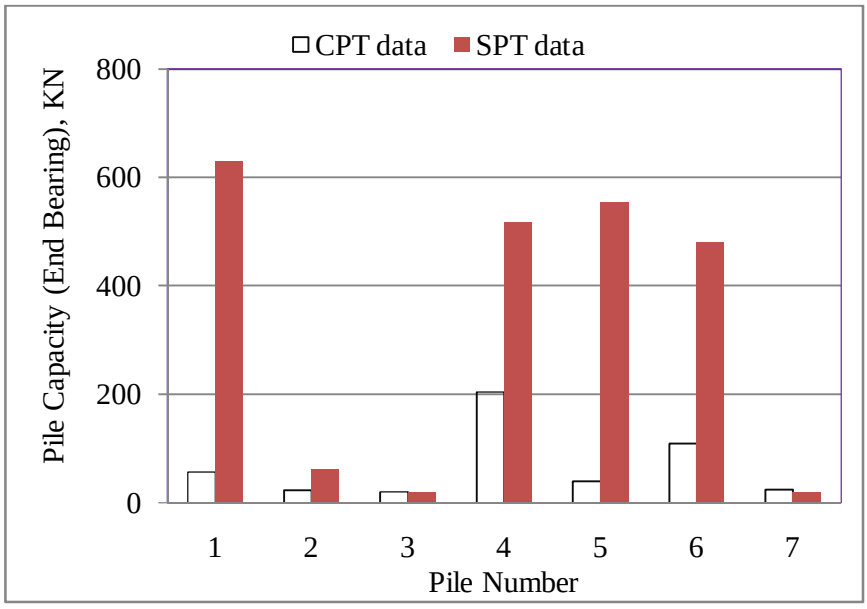


Figure 3.21a Comparison of ultimate end bearing capacity of pile predicted by Penpile method (1978) with the one predicted by SPT data

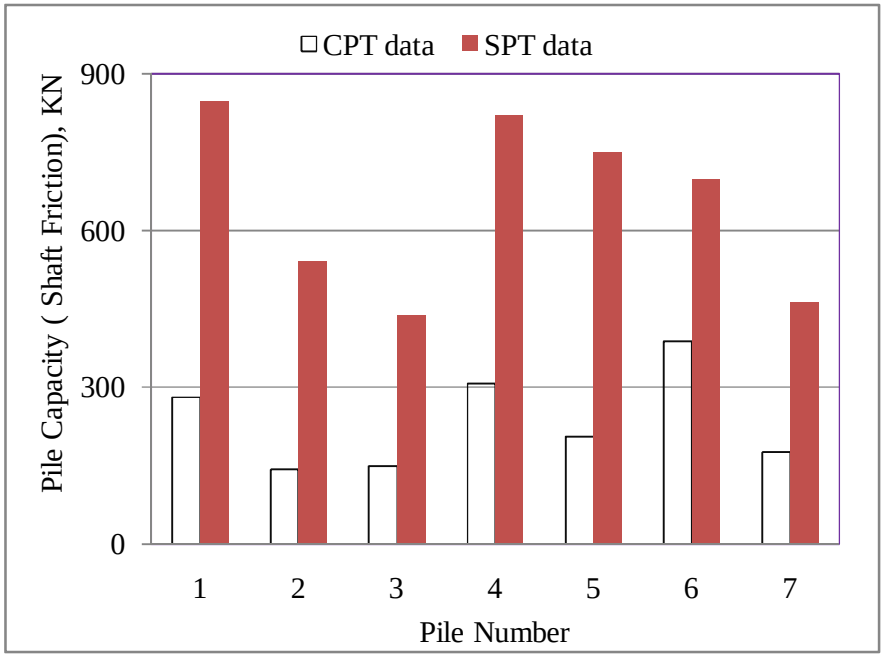


Figure 3.21b Comparison of ultimate shaft friction capacity of pile predicted by Penpile method (1978) with the one predicted by SPT data

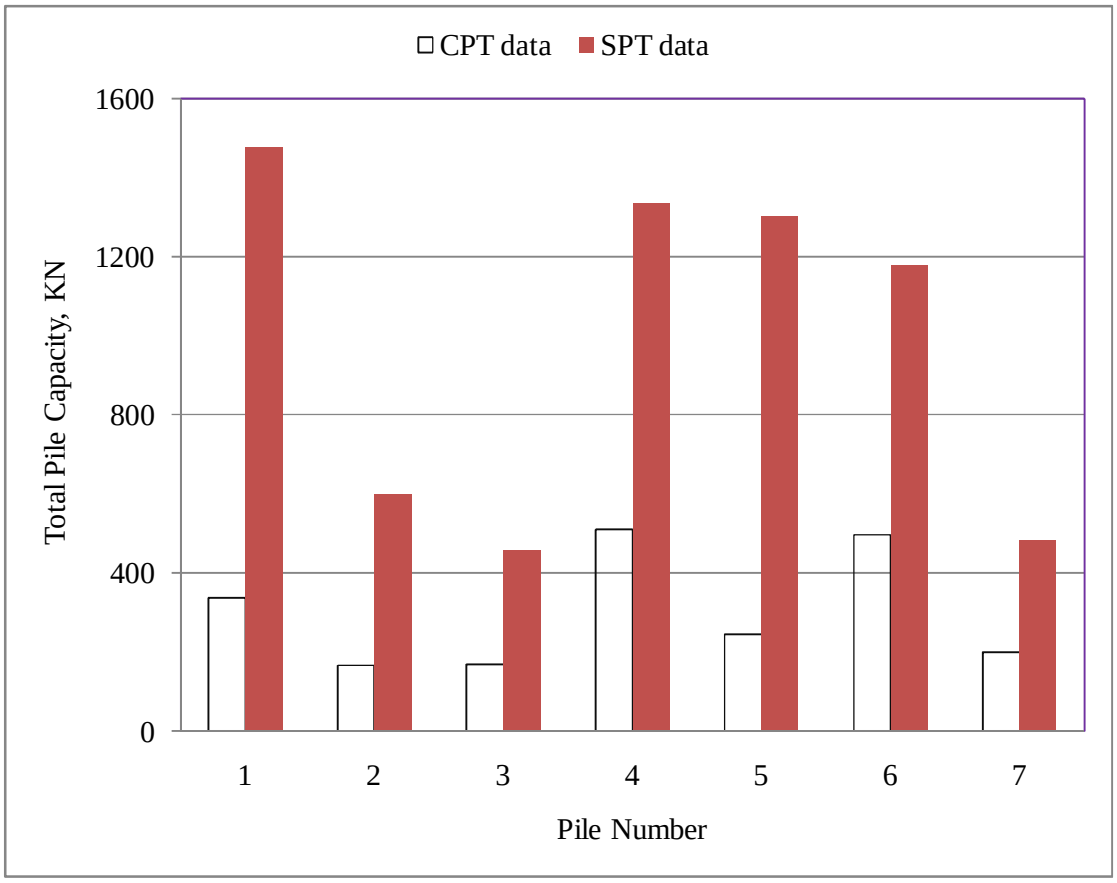


Figure 3.21c Comparison of ultimate pile capacity predicted by Penpile method (1978) with the one predicted by SPT data

3.8 APPLICABILITY OF THE VARIOUS METHODS USED FOR PREDICTING PILE CAPACITY

Evaluating the performance of different pile capacity prediction methods is not an easy task. In this study, an evaluation scheme using only analytical criteria is considered in order to observe the performance of different CPT methods for predicting the ultimate axial capacity of piles. The arithmetic mean and standard deviation of piles at seven locations(such as TP1, TP2, TP3, TP4, TP5, TP6 and TP7) for each method are calculated. The best applied method among nine methods is the one which is closer to the mean of nine methods.

Ultimate pile capacities of piles at six locations calculated by Schmertmann method and de Ruiter and Beringen method are closer to mean of nine methods than any other methods except Philipponnat method. All the values at six locations determined by these two methods are lying within plus and minus of one standard deviation.

Ultimate pile capacities for piles at five locations calculated by Schmertmann method tend to underpredict the ultimate pile capacity by not more than 25% in comparison with the mean of nine methods. This method uses sleeve friction,f_s to calculate ultimate friction capacity of pile and sleeve friction is lower in clayey soil which is predominant as shown in Figure 3.8. Ultimate pile capacities for piles at two locations calculated by this method tend to overpredict the ultimate pile capacity. According to CPT values as shown in Figure 3.10, this method exhibits 35% higher pile capacity than the mean of nine methods. In Figure 3.10 the bearing stratum of the pile is dense silty sand where high cone tip resistance exists. The ultimate end bearing capacity estimated by this method shows the largest value because of not applying any reduction factor to cone tip resistance like only Tumay and Fakhroo method. Besides this, ultimate shaft friction capacity by this method is lower, because cone shaft friction exhibits lower value in predominant clayey soil. Philipponnat method estimates better results for piles at all locations of this site fortunately, though it does not consider the consistency of cohesive soil (soft or hard) to determine the ultimate shaft friction capacity,Q_s of pile and assumes the same empirical factor for all types of clay.

de Ruiter and Beringen method also exhibits relatively better performance. Ultimate pile capacities of piles at six locations calculated by this method are closer to mean of nine methods. Ultimate pile capacity at CPT4 location is 46% higher than the mean of nine methods. In this method no reduction factor is imposed in sandy soil to calculate ultimate end-bearing capacity like Schmertmann method. This method assumes the larger empirical factor for soft clay and the lower empirical factor for stiff clay to calculate the ultimate shaft capacity of pile. Ultimate pile capacities for piles at seven locations calculated by de Ruiter

and Beringen method tend to overpredict the pile capacity. The maximum value has been observed at CPT4 location as shown in Figure 3.10.

The performance of Bustamante and Gianeselli method (LCPC/LCP method) is not satisfactory at this study. This method overpredicts the pile capacity by 24%-64% and the values at three locations are lying beyond one standard deviation. This method for the French Highway Department is based on the analysis of 197 pile load tests with a variety of pile types and soil conditions. It utilizes cone tip resistance,q_c instead of shaft friction,f_s to determine the ultimate shaft friction capacity,Q_s. To calculate shaft friction capacity it shows much larger value in clayey soil which is predominant at all seven CPT locations.

Tumay and Fakhroo method overpredicts the pile capacity excessively and pile capacities for piles at six locations are beyond one standard deviation. In this method no reduction factor is assumed for cone tip resistance,q_c to calculate the ultimate end bearing capacity. Same adhesion factor is used both for cohesive and cohesionless soil to calculate the ultimate shaft friction capacity of pile and this factor becomes very high when sleeve friction is very low.

Aoki and De Alencar method underpredicts the pile capacity and the results show the moderate performance. This method assumes the identical empirical factor for both clayey and sandy soil to find out the ultimate end bearing capacity of pile. It does not consider the consistency of cohesive soil (soft or hard) to determine the ultimate shaft friction capacity,Q_s of pile and assumes the same empirical factor for all types of clay.

Both Price and Wardle method and Penpile method tend to underpredict the pile capacity seriously and show the values beyond one standard deviation. Those are very conservative to calculate the pile capacity. Penpile method uses empirical factor 0.25 for clay and 0.125 for sand to calculate the end bearing capacity of pile when Price and Wardle method utilizes 0.35 for both types of soil. Both method uses sleeve friction,f_s to determine the shaft friction capacity.

The results of pile capacity have been estimated for different depths at seven locations using SPT values. Arithmetic mean and standard deviation at different depths for those seven locations have also been estimated. Ultimate pile capacities based on SPT values show the wide variation. Pile capacities at five locations overpredict or underpredict the pile capacity satisfactorily. The rest of the piles at two locations overpredict the pile capacity beyond one standard deviation. The soil of this site is very erratic and the thickness of soil layers varies drastically throughout the soil layer. SPT is not reliable for cohesive soil which is predominant for the whole site.

3.9 CONCLUDING REMARKS

In this chapter five soil profile sections having twelve bore holes describe SPT values with depth, visual soil classification and thickness of soil layers. Cone tip resistance, sleeve friction, friction ratio and soil classification at seven different locations are presented graphically. The results of the analyses based on SPT and CPT data are conducted on square reinforced concrete piles at a site in Siddhirganj. Applicability of eight CPT methods are focused to predict the ultimate axial compression load carrying capacity of piles. The static analysis using the SPT method is applied to evaluate the load carrying capacity for seven locations of the site. Ultimate pile capacity predicted by various methods of CPT are compared graphically with the one predicted by SPT data. The pile size, type, length, the predicted ultimate load carrying capacity from CPT, the predicted ultimate load carrying capacity from SPT, average ultimate pile capacity, and standard deviation are presented in tabular form. The arithmetic mean and standard deviation are calculated in order to observe the performance of different CPT methods for predicting the ultimate axial capacity of piles. The four CPT methods, which are de Ruiter and Beringen method, Philipponnat method, Schmertmann method and Aoki and De Alencar method show better performance than the currently used method based on SPT.

CHAPTER FOUR
CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

This study presents an evaluation of the performance of eight CPT methods in predicting the ultimate load carrying capacity of square precast RC concrete piles at a site in Sidhirganj. The following CPT methods are used to predict the load carrying capacity of the 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 is also predicted using the traditional method based on SPT, which is used by engineers for pile design and analysis. An evaluation scheme is executed to evaluate the CPT methods based on their ability to predict the ultimate pile capacity. The arithmetic mean and standard deviation of the piles are calculated for evaluation of different methods. Based on the results, the followings are the major finding of this study:

- Schmertmann method, de Ruiter and Beringen method and Philipponnat method show the best capability in predicting the load carrying capacity of square RC piles at a site in Siddhirganj. But Philipponnat method does not consider the consistency of cohesive soil (soft or hard) to determine the ultimate shaft friction capacity,Q_s of pile and assumes the same empirical factor for all types of clay.

- Aoki and De Alencar method shows moderate performance and Bustamante and Gianeselli (LCPC/LCP) method exhibits unsatisfactory performance to estimate the pile capacity for piles at this site.

- The worst methods of prediction are Penpile method and Price and Wardle method which are very conservative (underpredict the pile capacities) and the Tumay and Fakhroo method which overpredicts the pile capacities excessively.

- The four CPT methods, which are de Ruiter and Beringen method, Philipponnat method, Schmertmann method and Aoki and De Alencar method show better performance than the currently used method based on SPT. The soil of this site is very erratic and the thickness of soil layers varies drastically throughout the soil layer. SPT is not reliable for cohesive soil which is predominant for the whole site.

4.2 RECOMMENDATIONS FOR FURTHER STUDY

The results of this study demonstrates the capability of CPT methods in predicting the ultimate load carrying capacity of square RC piles at a site in Siddhirganj. Schmertmann method, de Ruiter and Beringen method and Philipponnat method show the best performance in predicting the ultimate load carrying capacity of square RC piles. But Philipponnat method does not consider the consistency of cohesive soil (soft or hard) to determine the ultimate shaft friction capacity, Q_s of pile and applies the same empirical factor to all types of clay. It is recommended to implement these three methods in design and analysis of square RC concrete piles.

In fact, implementation of the CPT technology in pile design will reduce the level of uncertainties associated with traditional design methods. Based on the results of the analyses, it is recommended to implement the cone penetration technology in different geotechnical applications. Regarding design and analysis of driven piles, the followings are recommended:

- Confidence of the design engineers in the CPT technology by adding the CPT to the list of the primary variables in subsurface exploration should be achieved and for soil identification and classification, and site stratigraphy it should be used. Different soil classification methods can be used such as Zhang and Tumay, Robertson and Campanella, and Olsen and Mitchell.

- The test results from the traditional subsurface exploration methods and the results interpreted from the CPT methods should be compared. With time and experience, the dependency level on the traditional subsurface exploration methods will be reduced and dependency level on the CPT technology will be increased.

- The CPT pile design methods in conjunction with the pile load tests and the method based on SPT to predict the load carrying capacity of the square RC concrete piles should be used. The following CPT methods are recommended: de Ruiter and Beringen method, Schmertmann method and Philipponnat method. If a pile load test is conducted for the site, the results of the CPT methods with the measured ultimate pile load capacity will be compared. If the measured and predicted capacities are different, then a correction to the predicted capacity in the amount of the difference between the measured and predicted capacity will be made. This correction for the design of other piles at this site should be applied.

- The role of the CPT design method will be increased and the dependency on the method based on SPT will be decreased.

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APPENDIX-A

SPLICED PRECAST R.C. CONCRETE PILES

Figure A1 Spliced Precast RC Concrete Piles before driving
(Source: PWD)

Figure A2 Spliced Precast RC Concrete Piles before casting and during driving (Source: PWD)

APPENDIX B

SAMPLE CALCULATION

$$Q_u = Q_s + Q_t$$

Where Q_u= total ultimate pile capacity

Q_s= ultimate shaft friction capacity

Q_t= ultimate end bearing capacity

Ultimate pile capacity of 355mm×355mm×21m spliced precast RC concrete pile is calculated from figure 3.7 as shown in chapter 3 using 8 different CPT methods.

1. Schmertmann method

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

q_{c1} = 5030 KPa

$$q_{c2} = \frac{\left(\frac{4.3+2.73}{2}\right)\times 1.3 + 2.73\times 0.82 + \left(\frac{2.73+1.24}{2}\right)\times 0.18 + 0.77\times 1.72}{4.02}\times 1000$$

=2110 KPa

Unit tip resistance, $q_t = \frac{5030+2110}{2}$

= 3570 KPa

Ultimate end bearing capacity, Q_t= 3570×0.355×0.355

=450 KN

Shaft friction is used to calculate ultimate friction capacity.

$$f_s = 0.8\times \frac{100+67.4}{2} \times 1.27 + 0.8\times 67.4\times 0.63 + 0.8\times \frac{67.4+6}{2} \times 0.7 +$$

$$1.16\times 5\times 7.9 + 1.05\times 18.2\times 0.88 + 1.14\times 8\times 2.99 + 0.78\times 46.3\times 0.6 + 1\times 21.1\times 3.23$$

$$= 319.4 \text{ KN/m}$$

Ultimate shaft friction capacity, $Q_s = 319.4\times 4\times 0.355 = 453.5$ KN

Ultimate pile capacity, $Q_u = 453.5 + 450$

$$= 903.5 \text{ KN}$$

2. de Ruiter and Beringen method

$q_{c1} = 5030$ KPa

$q_{c2} = 2110$ KPa

$$q_t = \frac{5030 + 2110}{2} = 3570 \text{ KPa}$$

Ultimate end bearing capacity, $Q_t = 3570\times 0.355\times 0.355$

$$= 450 \text{ KN}$$

Cone tip resistance,q_c is used to calculate ultimate friction capacity.

$$f_s = 11.72\times 1.3 + 9.1\times 0.82 + 6.62\times 0.18 + (770\times 1.72\times 0.5)/20 + (650\times 6.3)/20 + (0.5\times 900\times 0.9)/20 + (600\times 3.0)/20 + (0.5\times 770\times 5.8)/20$$

$$= 483.8 \text{ KN/m}$$

Ultimate shaft friction capacity, Q_s= 483.8×4×0.355=687 KN

Ultimate pile capacity, Q_u= 687+ 450

= 1137 KN

2. Bustamante and Gianeselli method (LCPC method)

$$q_{ca} = \frac{q_{c1} + q_{c2}}{2}$$

$$= \frac{5030 + 3305}{2}$$

= 4170 KPa

q_{eq}= 3900 KPa

Unit tip resistance, $q_t = k_b\, q_{eq}(tip)$

=0.45×3900

=1755 KPa

Ultimate end bearing capacity, Q_t= 1755×0.355×0.355

=221.2 KN

Cone tip resistance,q_c is used to calculate ultimate friction capacity.

Curve no. 1 or 2 based on pile category no. 9 and the value of q_c are used to obtaion friction capacity.

f_s= 30×1.3+ 28×0.82+ 21×0.18+ 40×1.72+ 25×6.3+ 40×0.9+ 24×3+ 40×5.8

=632 KN/m

Ultimate shaft friction capacity, Q_s= 632×4×0.355=897.5 KN

Ultimate pile capacity, Q_u= 897.5+ 221.2

= 1118.7 KN

4. Tumay and Fakhroo method(Cone-m method)

$$q_t = \frac{5030+4000}{4} + \frac{2110}{2}$$

$= 3310$ KPa

Ultimate end bearing capacity, $Q_t = 3310 \times 0.355 \times 0.355$

$= 417.5$ KN

Shaft friction is used to calculate ultimate friction capacity.

$$f_{sa} = \frac{F_t}{L}$$

$$= \frac{106.3+42.5+25.7+39.5+16+23.92+27.8+68.2}{18.2}$$

$= 19.23$ KPa

$= 0.2$ tsf

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

$= 2.07$

Ultimate shaft friction capacity, $Q_s = 2.07 \times 19.23 \times (4 \times 0.355 \times 18.2) = 1029$ KN

Ultimate pile capacity, $Q_u = 1029 + 417.5$

$= 1446.5$ KN

5. Aoki and De Alencar method

$q_{ca} = 3570$ KPa

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

$$q_t = \frac{3570}{1.75} = 2040 \text{ KPa}$$

Ultimate end bearing capacity, Q_t= 2040×0.355×0.355

=257.1 KN

Cone tip resistance,q_c is used to calculate ultimate friction capacity.

$$f_s = \frac{0.022}{3.5} \times (3.52 \times 1.3 + 2.73 \times 0.82 + 2 \times 0.18) \times 1000 + (0.04 \times 770 \times 1.72)/3.5 + \frac{0.04}{3.5} \times$$

(650×6.3+900×0.9+ 600×3)+ (770×5.8×0.06)/3.5

=213.4 KN/m

Ultimate shaft friction capacity, Q_s= 213.4×4×0.355=303.1 KN

Ultimate pile capacity, Q_u= 303.1+257.1

= 560.2 KN

6. Price and Wardle method

$q_t = k_b q_c$

=0.35×3570

=1249.5 KPa

Ultimate end bearing capacity, Q_t= 1249.5×0.355×0.355

=157.5 KN

Shaft friction is used to calculate ultimate friction capacity.

f_s= 0.53×(106.3+42.5+25.7+39.5+16+23.92+27.8+68.2)

= 185.5 KN/m

Ultimate shaft friction capacity, Q_s= 185.5×4×0.355=263.35 KN

Ultimate pile capacity, Q_u= 263.35+157.7

= 421.05 KN

7. Philipponnat method

$$q_{ca}= \frac{5030+3305}{2}$$

= 4170 KPa

$q_t=k_b q_{ca}$

=0.42×4170

= 1715.4 KPa

Ultimate end bearing capacity, Q_t= 1715.4×0.355×0.355

=220.7 KN

Cone tip resistance,q_c is used to calculate ultimate friction capacity.

$$f_s= \frac{1.25}{60} \times(3.52\times1.3+2.73\times0.82+2\times0.18)\times1000+ \frac{1.25}{50} \times(0.77\times1.72+0.65\times6.3$$

+0.9×0.9 +0.6×3+0.77×5.8)×1000

= 461.86 KN/m

Ultimate shaft friction capacity, Q_s= 461.86×4×0.355=655.8 KN

Ultimate pile capacity, Q_u= 655.8+262.8

= 876.5 KN

8. Penpile method

q_t= 0.125×3570

=446.3 KPa

Ultimate end bearing capacity, Q_t= 446.3×0.355×0.355

=56.2 KN

$$q_s = \frac{f_s}{1.5+0.1f_s}$$

$$f_s = \frac{106.3+42.5+25.7+39.5+16+23.92+27.8+68.2}{18.2}$$

= 19.23KPa

=2.79 psi

q_s= 1.57 psi

Ultimate shaft friction capacity, Q_s= (1.57×14×4×716.54)/1000=63 K=280.23KN

Ultimate pile capacity, Q_u= 280.23+56.2

= 336.4 KN

ULTIMATE PILE CAPACITY BASED ON SPT

For cohesionless soil, $q_t = \alpha N_{q'} \sigma_{v'}$

$$N_{avg} = \frac{32+26}{2} = 29$$

For N=29, $\varphi=34^0$

$\sigma_{v'}$= (18-9.8)×21= 172.2 KPa

Unit tip resistance, q_t= 0.62×50×172.2

=5338 KPa ≤ 5000 KPa

Ultimate end bearing capacity, Q_t= 5000×0.355×0.355

=630 KN

$q_{s1}= \sigma'_{avg} K \tan\varphi$

For N_{avg}=28, $\varphi=34^0$

q_{s1} =86×1.3tan34

=75.5 KPa

$q_{s2} = c_a l$

For N_{avg}=8, c=0.5tsf

q_{s2} = 35×8=280 KN/m

Ultimate shaft friction capacity, Q_s= (75.5×7×0.6+ 280)×4×0.355 =847.7KN

Ultimate pile capacity, Q_u= 847.7+630 = 1477.7 KN

APPENDIX-C

CPT EQUIPMENT

The components of CPT equipment are as follows:

- CPT Machine
- Hydraulic pump
- CPT cone
- CPT rods
- Soil anchors
- Reaction beams
- PC Interface monitor

Figure C1 Hydraulic Pump

Figure C2 CPT Machine

Figure C3 Short beams inside the machine

Figure C4 Long beams on top of the short ones

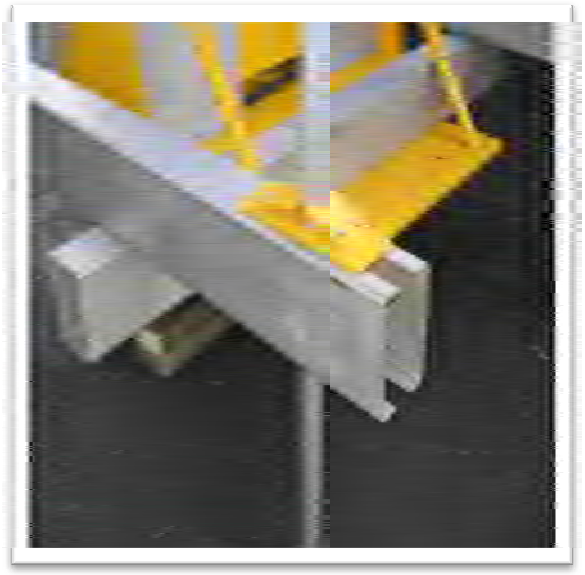

Figure C5 Automatic locks on the auger rods.

Figure C6 Depth Sensor Wheel

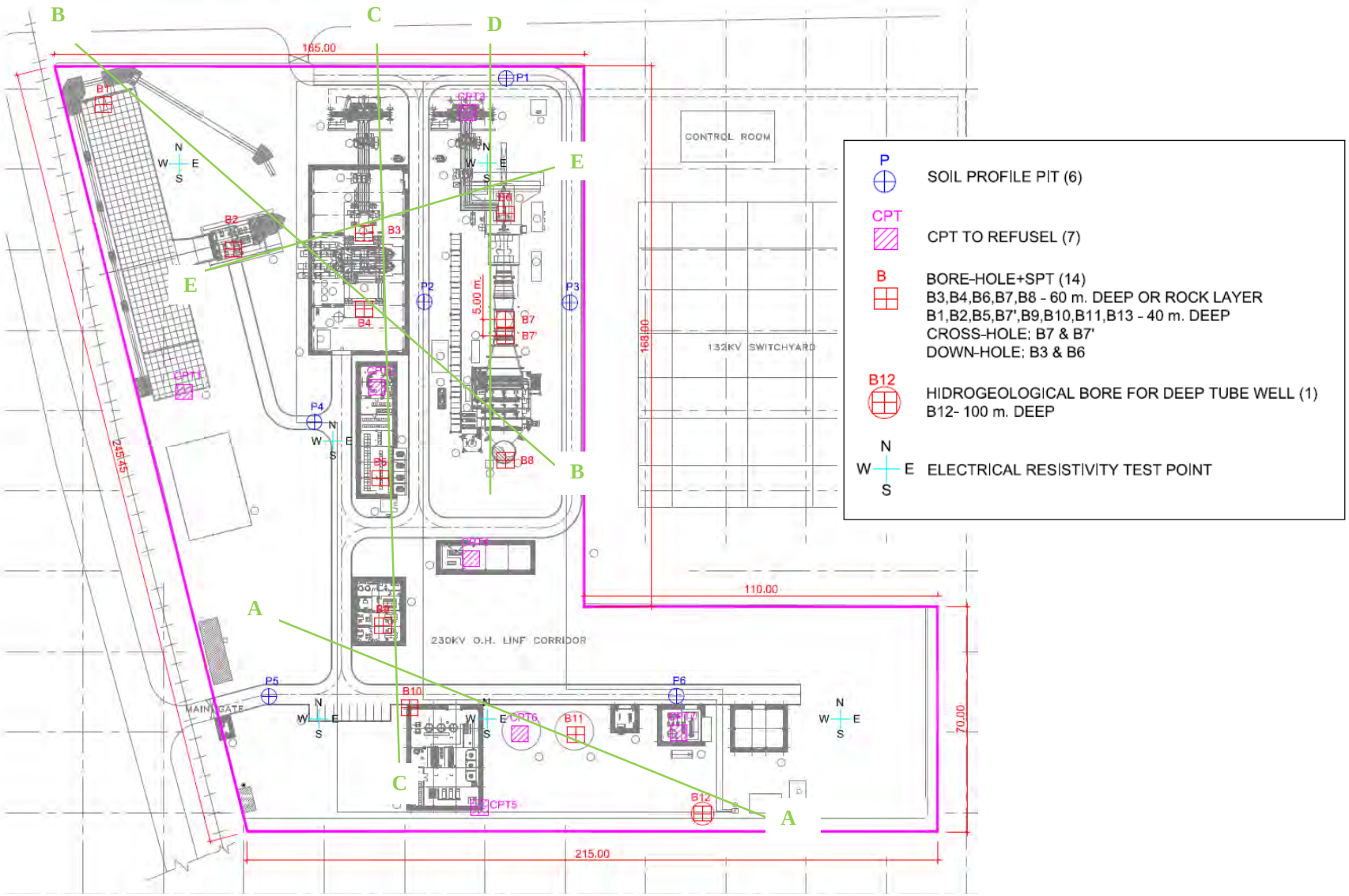

Figure A1: Layout of bore holes and sections

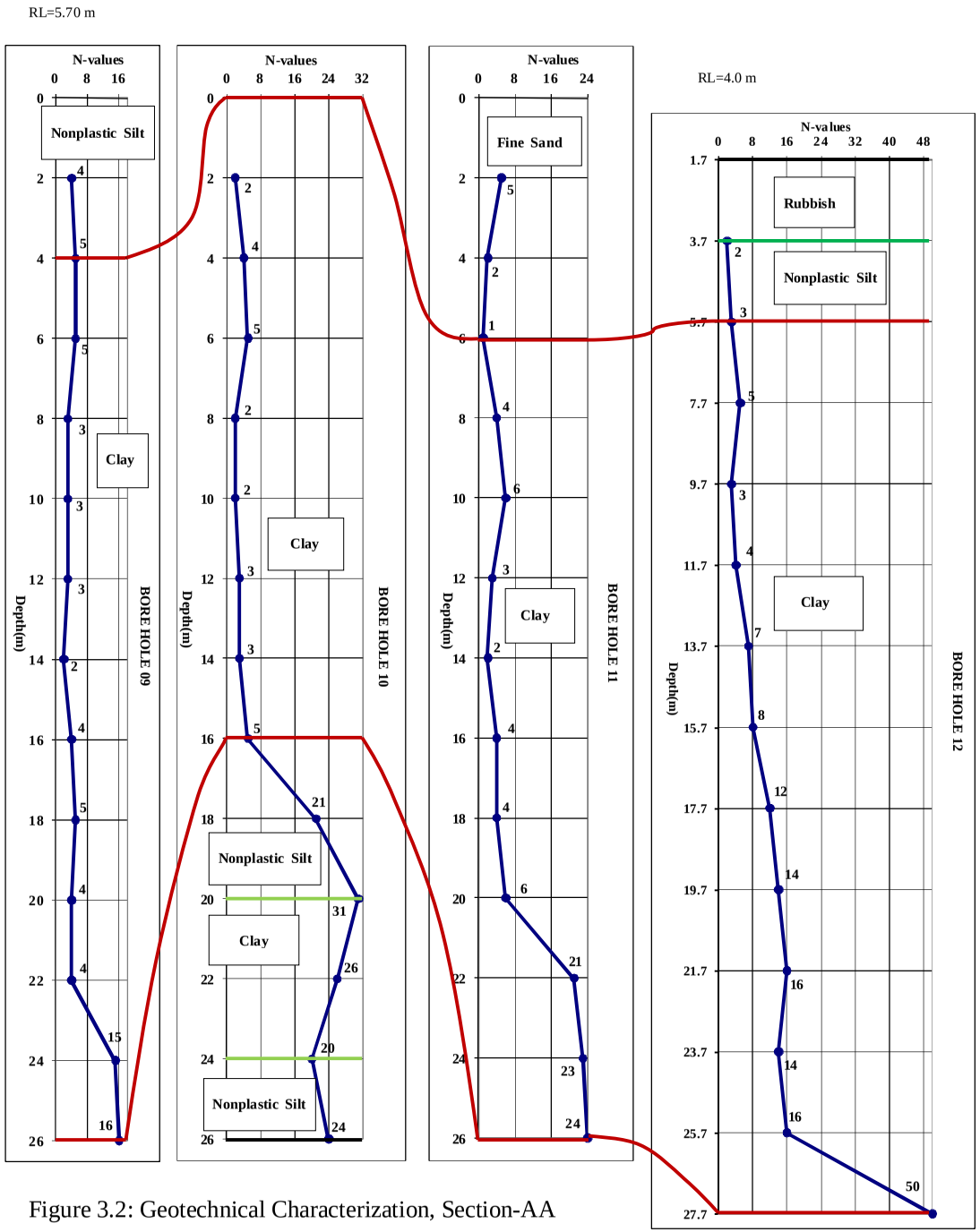

Figure 3.2: Geotechnical Characterization, Section-AA

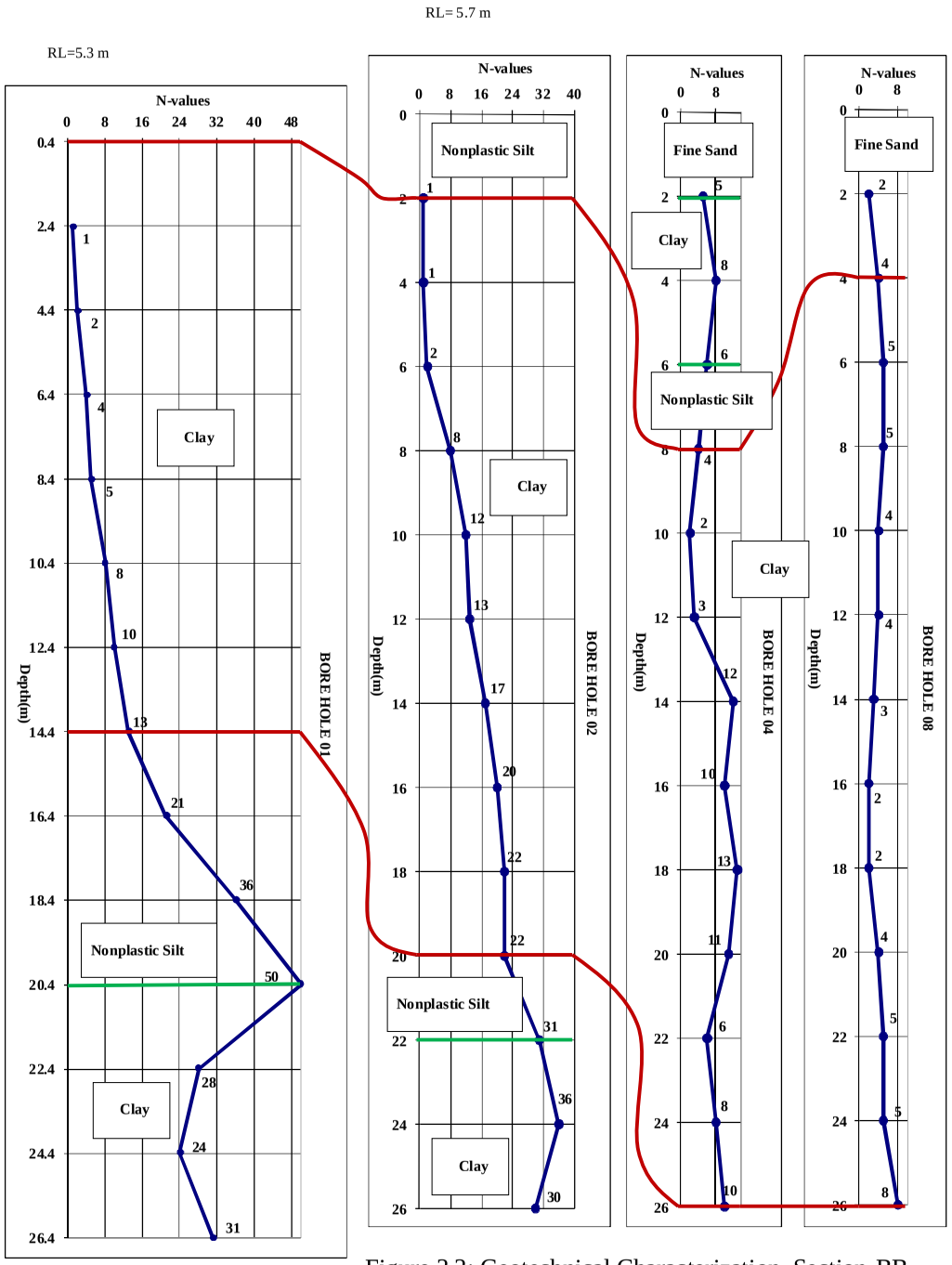

Figure 3.3: Geotechnical Characterization, Section-BB

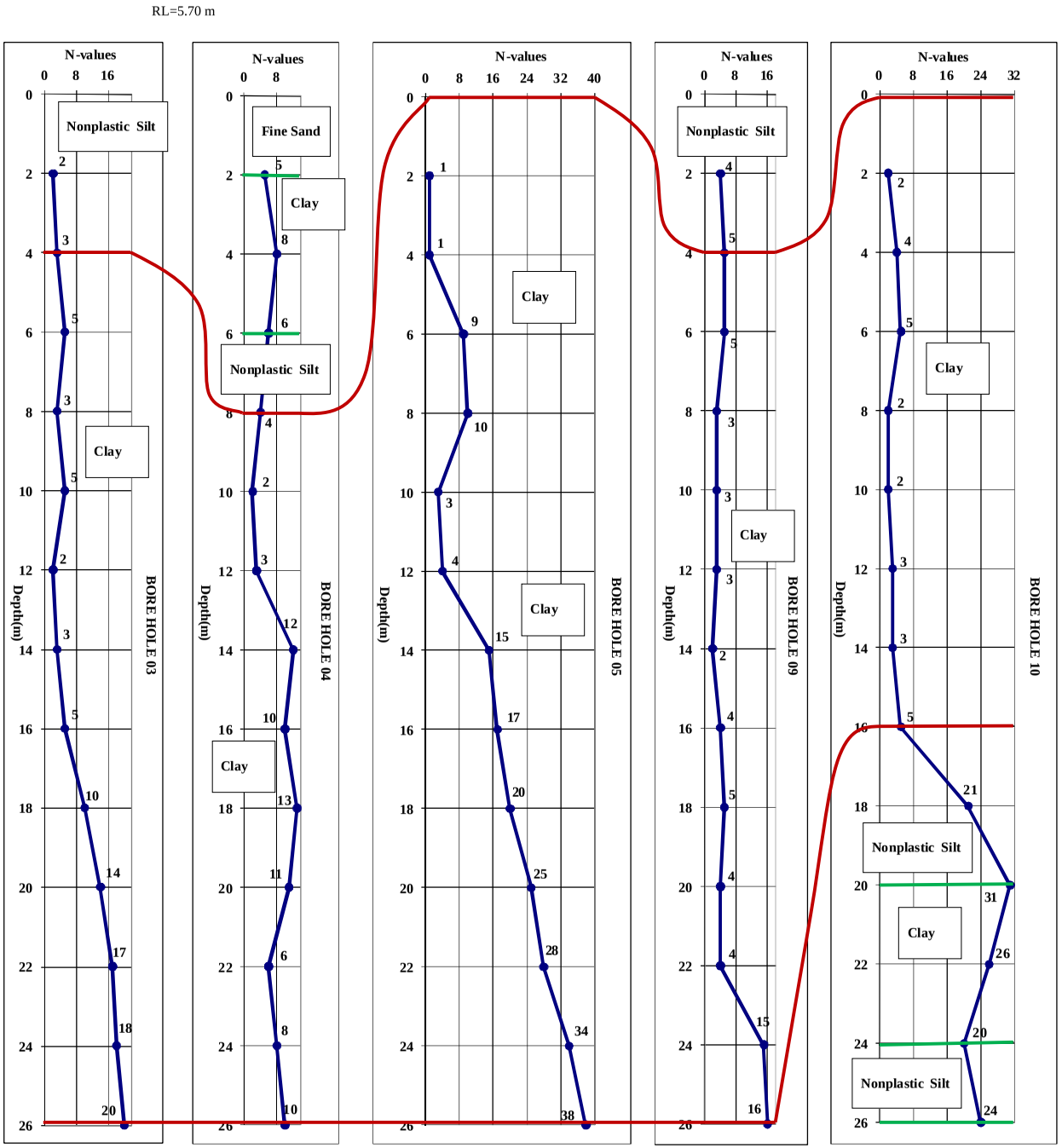

Figure 3.4: Geotechnical Characterization, Section-CC

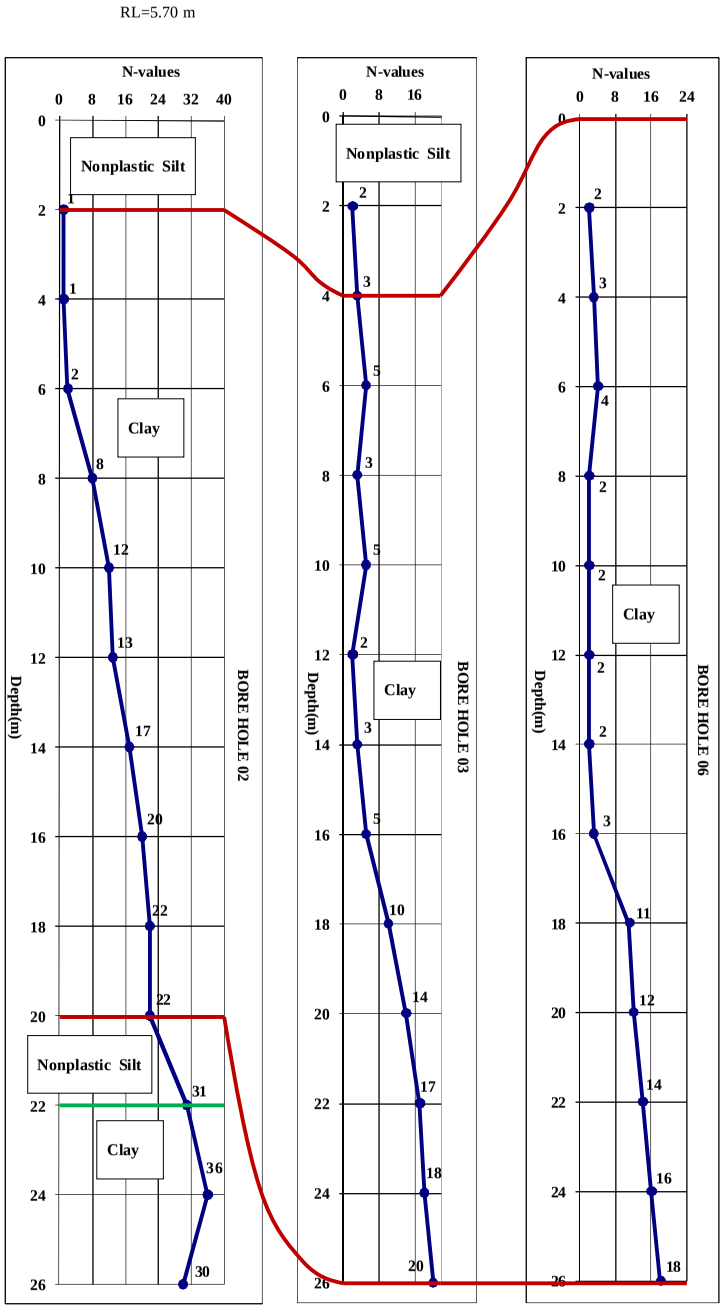

Figure 3.6: Geotechnical Characterization, Section-EE

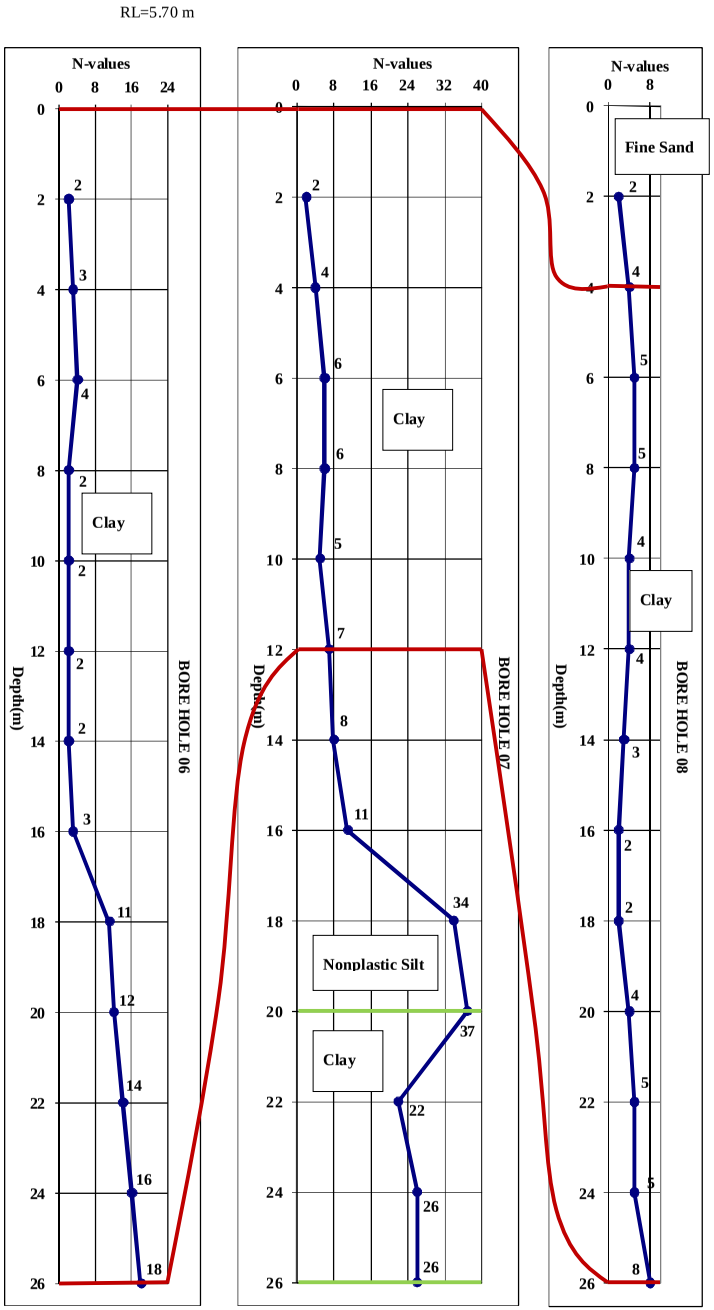

Figure 3.5: Geotechnical Characterization, Section-DD