COMPARISON OF AXIAL LOAD CAPACITY OF PILES FROM THEORETICAL METHODS AND STATIC LOAD TESTS

by

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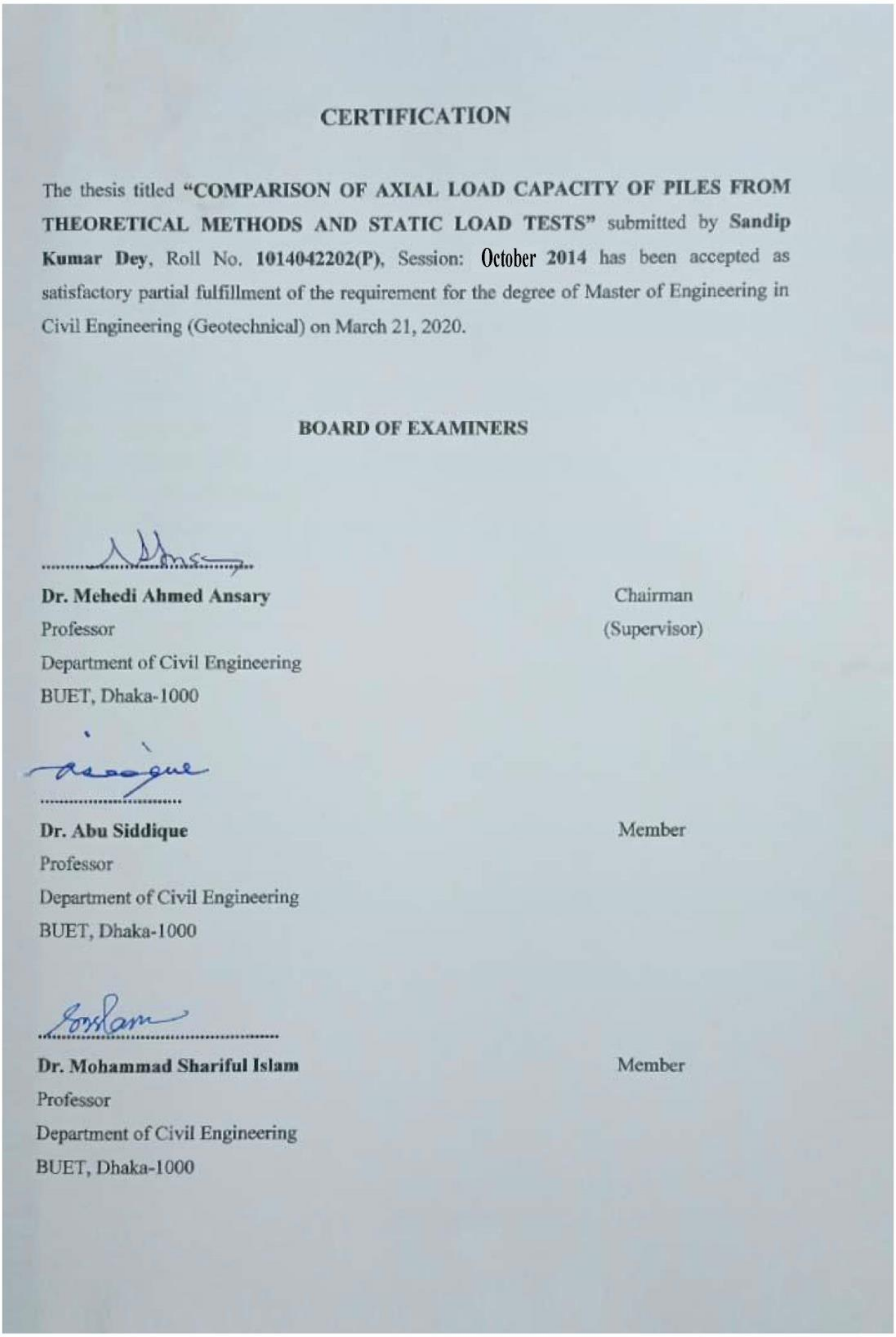

CERTIFICATION

The thesis titled **"COMPARISON OF AXIAL LOAD CAPACITY OF PILES FROM THEORETICAL METHODS AND STATIC LOAD TESTS"** submitted by **Sandip Kumar Dey**, Roll No. **1014042202(P)**, Session: **October 2014** has been accepted as satisfactory partial fulfillment of the requirement for the degree of Master of Engineering in Civil Engineering (Geotechnical) on March 21, 2020.

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ABSTRACT

This study has been carried out to compare some selected empirical and theoretical equations used globally. The comparisons have been made among the various equations concerning the static load test result, which is widely believed to be providing a reliable pile capacity. The study has been based on the data obtained from the sub-soil investigation reports and corresponding pile load test results collected from twenty-two projects all over the country. Among these projects, fewer than twelve projects fifteen precast piles have been tested and under another ten projects fifteen cast-in-situ piles have been tested. The tests have been performed between 1997 to 2018 and funded by Public Works Department (PWD), RAJUK, Roads & Highways Department, and Dhaka Mass Transit Company (MRT). Almost 70% pile load tests are carried out under the direct supervision of the Department of Civil Engineering, BUET, and the rest of the pile load test are carried out by Icon Engineering Services, Dhaka. In this study, five calculation methods, namely Meyerhof (1976), NAVFAC DM 7.2 (1984), AASHTO (1986), O'Neill & Reese (1988) and Decourt (1995) methods for cast-in-situ/ bored piles and Drilled shaft, and another five calculation methods, namely Meyerhof (1976), API RP 2A (1993), Tomlinson (1994), Norwegian Pile Guideline (2005), and Indian Standard (2010) methods for precast piles have been used. The static load test has been performed and analyzed by Davisson method, BNBC code (1993), and Indian Standard (2010).

It has been observed that the Tomlinson (1994), API (1993) and Meyerhof (1976) methods provide the most reliable and justified correlation between predicted and measured capacity for precast/ driven piles. On the other hand, for the cast in situ/ bored piles, AASHTO (1986) and O'Neill & Reese (1988) and NAVFAC DM 7.2 (1984) methods provide the most reliable and justified correlation between predicted and measured capacity.

It has also been observed that the methods for predicting the ultimate capacity of precast/ driven piles give relatively more reliable and justified result with minimum error compared with the cast in situ/ bored piles. In all the cases a considerable correlation between the static analysis of pile capacity and capacity of the pile from the static load test are found. This study has supported the idea to put a higher degree of confidence to use the statics formulae to find out the ultimate capacity of the piles. For precast piles, the correlation coefficients vary from 0.919 to 0.972 and for cast-in-situ bored piles the correlation coefficients vary from 0.518 to 0.794. No such reliable correlation can be established for cast-in-situ drilled shafts.

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NOTATIONS

A = Cross sectional area of pile
Ab = End bearing area of pile
As = Skin friction area (perimeter area) of pile
B,D = Diameter or width of pile
CTP = Cast-in-situ Test Pile
D_b = Diameter of pile at base
D_c = Critical depth of soil layer
Ep = Modulus of elasticity of pile material
Es = Modulus of elasticity of soil
FS = Factor of safety
H = Layer thickness
K = Coefficient of earth pressure
KS = Coefficient of horizontal earth pressure
K_o = Coefficient of earth pressure at rest
L = Length of pile
N = Standard penetration test value (SPT)
N_{160} = Corrected SPT value for overburden pressure
Nc, Nq, Nγ = Bearing capacity factors
OCR = Over consolidation ratio
PTP = Pre Cast Test Pile
Q_{allow} = Allowable load
Q_b = End bearing at the base or tip of the pile
Q_p = Load transferred to the soil at pile tip level
Q_s = Skin friction or shaft friction or side shear
Q_{ult} = Ultimate bearing/load carrying capacity
W = Weight of the pile
c = Apparent cohesion of soil
c_u = Undrained cohesion of soil
f_b = End bearing resistance on unit tip area of pile
f_s = Skin frictional resistance on unit surface area of pile
g = Gravitational acceleration

q_u = Unconfined compressive strength

s_u = Undrained shear strength; same as c_u

z = Depth

Δ_{z1} = Thickness of any (i_{th}) layer

α = Adhesion factor

β = Friction factor due to overburden

γ, γ_t = Unit weight of the soil

γ_w = Unit weight of water

μ = Poisson’s ratio of soil

σ_o = Initial effective stress at mid-point of a soil layer

σ_p = Increase in effective stress at mid-point of a soil layer due to increase in stress

σ_r = Reference stress (100 kPa) for computation of pile settlement

σ_v = The total vertical stress

σ'_v = Effective vertical stress

ϕ = Apparent angle of internal fiction

ϕ' = Effective/drained angle of internal fiction

δ = Soil Structure Interaction Angle

CHAPTER 1

INTRODUCTION

1.1 Background

Piles are structural members that transmit the superstructure loads to deep soil layers. They are preferred to be used as a foundation material when the shallow foundation is not practical to use it. Piles and pile foundations have been in use since prehistoric times. The Roman wooden piles are a classic example of this. Today piles can be made of wood, concrete, or steel.

Soft soil is very common in many parts of Bangladesh which is not suitable for the construction of a shallow foundation. Pile foundation provides the best possible solution to transfer the load to the deeper harder layers of soil. In Bangladesh, driven piles are used in large numbers because of their various advantages over bored piles; like the high quality of construction, idea of capacity during driving, etc. Recently large diameter cast in situ piles is also used in large numbers for bridge structures and high-rise buildings.

Estimating pile capacity accurately is a difficult job even for the experienced geotechnical engineer. There are many conventional methods for calculating pile capacity, but the selection of each method requires knowledge of soil properties as well as the limitation or applicability of any method in a regional boundary. Traditionally, pile capacity can be evaluated by using a bore log of the subsoil investigation report (Bowles 1997), and then, later it needs to be confirmed by the static load test. As per, static load testing of the driven pile and bored cast in situ pile is very time-consuming and expensive as well as needs constant supervision on operation processes. It is often very difficult to ensure the chances of accuracy and precision. Moreover, the test has several problems like transferring the load to the pile due to frictional errors. Besides, a manual data collection system introduces human error possibilities. In these circumstances, a suitable alternative to static load test or cross-checking options were necessary for foundation engineers. Pile capacity determination is a difficult thing. Several different designs practices here in Bangladesh and internationally exist, but seldom have they given the same computed capacity. Especially, determining the skin friction component is not an easy thing since the soil is not intact after the pile is driven or drilled and loses its intact engineering property (strength). So far, the precise determination of this value has not been possible. Thus, today design offices only believe a load test can only give a reliable capacity of the pile at the time of the test. After installation, the design values, i.e. the load-carrying capacities of piles are usually verified using different methods such as pile loading test and dynamic analysis.

Scientific approaches to pile design have advanced enormously in recent decades and yet, still the most fundamental aspect of pile design - that of estimating capacity –relies heavily upon empirical correlations.

Western researchers provided empirical methods based on extensive explorations and investigations for different types and conditions of soil. Meyerhof (1959) has arranged a speculative relationship between the corrected standard penetration test data and the ultimate axial capacity of driven pile. Also in 1976, he administered another formula for estimating the capacity of bored cast in situ piles based on the behavior of pile in granular soil. Whereas Vesic (1977) modified the bearing capacity factor that Meyerhof (1976) provided for end bearing of driven pile and bored pile in granular soil founded by the relationship between rigidity modulus and angle of internal friction of soil. The American Petroleum Institute (API) provides a static analysis procedure design developed for offshore construction. These projects almost exclusively use large diameter, open-end, steel pipe piles which are driven by impact hammer to final penetration (American Petroleum Institute 2003). Recently, large-diameter open-end pipe pile usage has increased significantly on transportation projects. This has heightened the need for more accurate nominal resistance estimates on these larger piles. So the design method proposed by API has more significance on large diameter steel piles rather than concrete piles. Tomlinson (1994) studied the behavior of driven piles in cohesive soil specially and established fascinating improvements of the adhesion or sometimes called the reduction factor previously provided by (Peck et al. 1974). As with any design method, the one should also confirm the appropriateness of selected coefficients in a given soil condition with local correlations between static resistance calculations and load test results. American navy in 1982 provided a guideline for offshore and onshore piles design named NAVFAC DM 7.2 and modified in 1984.Also, AASHTO time to time updated their code about pile capacity determination. O’Neill and Reese (1988) studied the behavior of piles in cohesive and cohesionless soil and established an acceptable theory for pile capacity determination later in 2005 AASHTO adopted this theory to their code for bored cast in situ piles.

The study focuses on some of the selected empirical (semi-empirical) and theoretical mathematical models used here in Bangladesh and internationally. To compare the various models, some of the thirty piles have been chosen from different projects all over Bangladesh including MRT (Mass Rapid Transit) project and Rajuk high rise building project. During the investigation, static load tests are performed to determine the pile capacity. The load tests are performed on single piles.

The study focuses only on the capacity of a single pile under compressive loading. Of course, seldom single piles are used; however, the capacity of group piles entirely depends on the capacity of a single pile within a group. It should be noted that the pile group capacity is not the intension of this study. A pile foundation is much more expensive than spread footings and is likely to be more expensive than mat foundation. Therefore, great care should be exercised in determining the soil properties at the site for the entire depth of possible pile penetration so that it can be accurately detern1ined whether a pile foundation is needed at all and, if so, the design can be optimized so that neither an excessive number nor excessive lengths are specified. This purpose can be achieved in two ways:

1. By taking adequate field and laboratory test programs which will help the designer to estimate the soil properties more accurately to design the foundation more economically.
2. By determining the ultimate carrying capacity of a pile by load test.

1.2 Objective of the Study

The Major objectives of the study are as follows:

i. To compare the ultimate capacity of piles determined by theoretical methods and from a static load, test results in selected areas.
ii. To compare among ultimate pile capacity using different semi-empirical methods for Cast-in-situ and Driven Piles.
iii. To establish a correlation between ultimate theoretical capacity [($\mathbf{Q_{theory}}$)] and capacity determined from static load test [($\mathbf{Q_{test}}$)].

1.3 Outline of the Study

In chapter one, the background and scope of the study have been discussed. The objectives of the study have also been stated.

In chapter two, the general concept of pile capacity, load capacity in compression, pile capacity by static formulae, and methods of determining ultimate pile capacity from pile load tests have been discussed. Also, statistical analysis procedure has been discussed here.

In chapter three, the methodology of the study and description of piles and project site with sub-soil conditions have been discussed.

In chapter four for every pile, load test the capacity of the pile from static formulae and pile load test procedure have been determined. The regression analysis has been conducted between the above two capacities. The analysis has been done in different dimensions. The relation between ultimate capacity from pile load test and static methods has also been drawn.

In chapter five, conclusions from this study and recommendations for further research have been made.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Pile foundation is used when the soil near the surface is not able to support foundation loads because of either low bearing capacity or the possibility of excessive settlement, so piles main function is to transfer foundations loads to deeper soil strata that are stronger and less compressible. Piles can be either precast driven or cast in place bored. Large diameter cast in place bored piles which diameter from 2ft to 15ft identify as drilled shafts according to AASHTO.

There are numerous equations available for evaluating the pile capacity for engineering professionals (Bowles, 1997). Pile capacity determination is a difficult thing. Several different designs practice here in Bangladesh and internationally exist. The methods include some simplifying assumptions empirical approaches regarding soil stratigraphy, soil pile structure interaction, and distribution of soil resistance along with the pile. Therefore they do not provide truly quantitative values directly useful in foundation design. Hence for proper judgment, it is necessary to verify the theoretical axial capacity with a load test. The axial capacity of piles can be determined by different approaches: static analysis, dynamic testing, static pile load test. So there is a scope to study pile capacity in different static methods and compare with static load test.

2.2 Pile Capacity

Generally, there are two alternative ways to determination of pile capacity i.e.:

i. Testing e.g. static load test and dynamic load test.
ii. Calculation e.g. static design equations based on laboratory and field investigations and pile driving formula.

Enough emphasis should be given to the accuracy in the estimation of pile capacity; this will lead us to not only to the safer structure but also economic savings. It should be noted that the term capacity in this thesis refers to the capacity of the bearing soil and it is not the structural strength of the pile itself. The ultimate axial capacity (Q_{ult}) of piles shall be determined by sum of the total side friction & total end bearing. Figure 2.1 represent the longitudinal section of a pile bear the skin friction and end bearing together. The following equation has use to compute the ultimate capacity of a pile.

$$Q_{ult} = Q_s + Q_p = f_s A_s + q_p A_p \qquad (2.1)$$

And Design load Capacity, In other words allowable bearings capacity is given as

$$Q_a = \frac{Q_{ult}}{F.S} \qquad (2.2)$$

Where,

Q_{ult} = Ultimate pile capacity.

Q_s = Shaft friction or side shear

Q_p = End bearing at the base or tip

A_s = Shaft friction area (perimeter area) of the pile = perimeter x length.

f_s = Unit shaft friction capacity

A_p = End bearing area of the pile = cross-sectional area of pile tip.

q_p = Unit pile tip resistance.

Q_a = Allowance pile capacity.

F_S = Factor of safety.

For a layered soil system containing ‘n’ number of layers, end bearings resistance can be calculated considering soil properties of the layer at which the pile rest and the skin friction resistance considerers all the penetrating layers calculated as:

$$Q_s = \sum_{i}^{n} \Delta z_i \times (\mathrm{perimeter})_i \times (f_s)_i \qquad (2.3)$$

Where, Δz_i, represent the thickness of any i^{th} layer, and (perimeter)is the perimeter of the pile in that layer. $f_s i$ is the unit skin friction at the i^{th} layer.

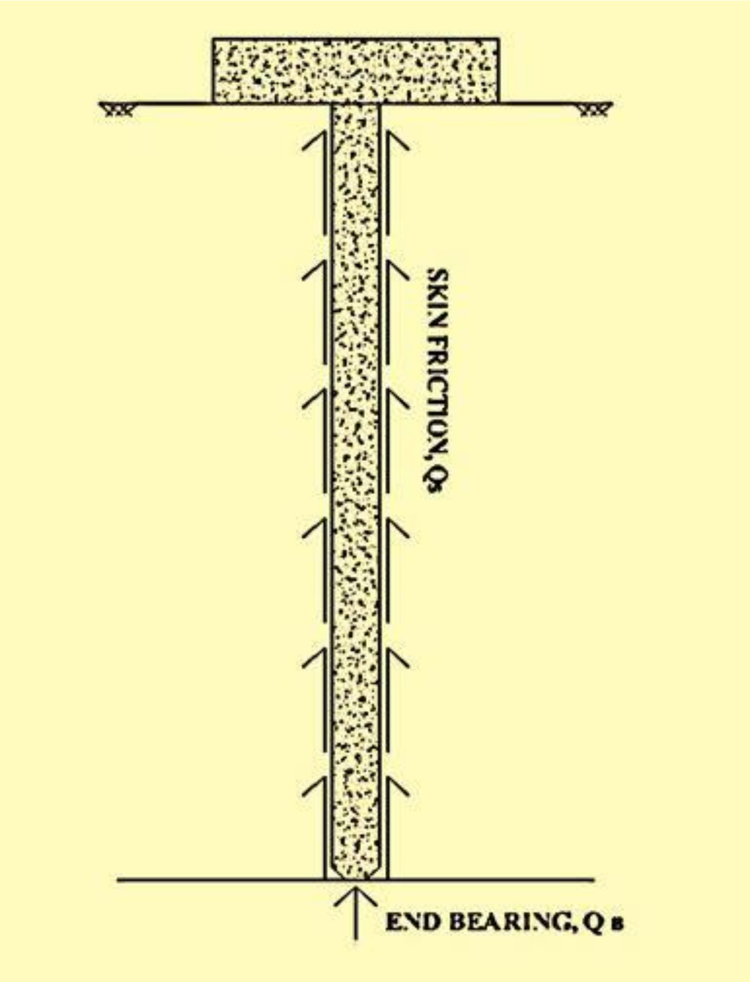

Figure 2.1: Longitudinal section of a typical pile (Das, 2002)

2.2.1 Axial Load Capacity of Driven Piles in Cohesive Soil

2.2.1.1 Meyerhof's (1976) Method

Side Friction

The average ultimate unit skin friction, f_s inhomogeneous saturated clay is usually expressed by $f_s = \alpha c_u$ in which α = the empirical adhesion coefficient for reduction of average undrained shear strength c_u of undisturbed clay within the embedded length of the pile. The coefficient α depends on the nature and strength of clay, dimensions, and method of installation of the pile, time effects, and other factors. The values of α vary within wide limits and they decrease rapidly with increasing shear strength. For driven piles, the values of α range on the average roughly from unity for soft clay to one-half or less for stiff clay, while for bored piles in stiff clay α is roughly one-half. These values of α, which represent a maximum side resistance, f_s of roughly 1 tsf (100 kN/m^2), Hence

$f_s = 0.5c_u \leq$ 1 tsf (2.4)

Ultimate total Side Resistance

Qs = fs As (2.5)

Where; Qs = Total Side Resistance

A_s = Surface area of the pile.

f_s = Unit Side Resistance

c_u = Undraincded shear strength of clay along the pile length

End Bearing

The ultimate unit end bearing in homogeneous cohesive soil may be expressed by

$$q_p = cN_c + p_0 N_q \leq q_m \tag{2.6}$$

In saturated homogeneous clay under undrained conditions, theory and observation have shown that the value of Nc below the critical depth varies with the sensitivity and deformation characteristics of the clay from about 5 for very sensitive brittle normally consolidated clay to about 10 for insensitive stiff over consolidated clay, although a value of 9 is frequently used for bearing capacity estimates of driven and bored piles. Figure 2.2 represent the nature of variation of the unit point end bearing resistance in homogenous soil.

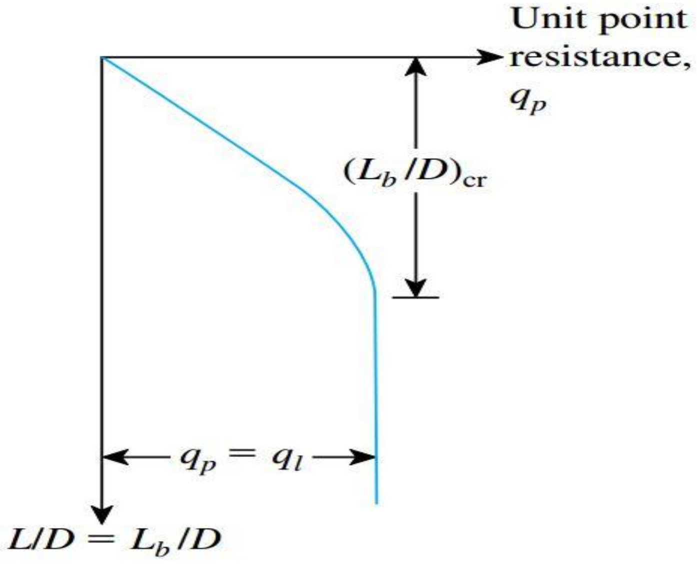

Figure 2.2: Nature of Variation of the unit point end bearing resistance in homogenous soil (Das, 2002)

Moreover, these values of Nc, are based on the initial undrained cohesion of the soil mass near the pile point, using carefully performed undrained triaxial compression tests on large samples of the clay. For undrained conditions of clay have no angle of friction, so N_q considerer equal to zero for the undrained condition of clay. Furthermore, any disturbance of the clay by pile installation mainly affects the initial point resistance, and subsequent consolidation of the clay will normally lead to a bearing capacity exceeding the undrained value at the end of the construction of the foundation. Empirical support for the net value of q_m is mainly limited so far to saturated clay.

Hence,

$$q_p = 9c_u \qquad (2.7)$$

Total end bearing capacity; $Q_P = 9c_u A_p$. (2.8)

Where

c_u = Undrained cohesion of soil surrounding the toe of the pile

A_p = Cross-Sectional area of pile toe.

$$\therefore Qu = Qp + Qs \qquad (2.9)$$

2.2.1.2 American Petroleum Institute (1993) Method

Side Friction

For cohesive soil, shaft resistance, QS can be determined from the following equation:

$$Q_s = \alpha c_u A_s \qquad (2.10)$$

Where,

α = Dimensionless adhesion factor (which is a function of vertical effective stress and undrained Shear strength of soil)

c_u = undrained shear strength of soil (The undrained shear strength, s_u, and undrained cohesion, c is assumed to be equal for calculations)

A_S = embedded surface area of the pile

The factor α can be computed based on effective stress of soil from equation (2.11) and (2.12):

$$\alpha = 0.5\psi^{-0.5}\ ;\ \text{when}\ \psi \leq 1.0 \tag{2.11}$$

$$\alpha = 0.5\psi^{-0.25}\ ;\ \text{when}\ \psi > 1.0 \tag{2.12}$$

Where,

$$\psi\ =\ \frac{\text{cu}}{\sigma' \text{V}} \tag{2.13}$$

c_u = undrained shear strength of soil (The undrained shear strength, s_u an undrained cohesion, c_u is assumed to be equal for calculations.)

σ'_V = effective vertical stress at mid-point of the layer = $\gamma' h$

An α value of 1.0 is recommended for unconsolidated clays. Reductions in resistance may be practical for very long piles where residual soil strength values are approached due to extended driving and subsequent soil displacement. For these cases, API (1993) recommends the use of engineering judgment.

End Bearing

The end bearing capacity in cohesive soil can be determined by the following expression:

$$Q_B = 9c_uA_B \tag{2.14}$$

Where,

c_u = undrained shear strength of soil (The undrained shear strength and undrained cohesion, c_u is assumed to be equal for calculations.)

A_B = cross-section area of pile tip

2.2.1.3 Tomlinson’s Method (1994)

Side Friction

This approach assumes that the shaft resistance is independent of the vertical effective stress and that the unit shaft resistance can be expressed in terms of an empirical adhesion factor times the undrained shear strength. The unit shaft resistance is equal to the adhesion which is the shear stress between the pile surface and the soil at failure. The total shaft resistance may be expressed in equation form as:

$$Q_s = \alpha c_uA_s \tag{2.15}$$

Where,
α = adhesion factor
c_u = undrained shear strength of soil (The undrained shear strength and undrained cohesion, c_u is assumed to be equal for calculations.)
A_S = embedded surface area of the pile

The adhesion factor, α depends on the nature and strength of the clay, pile dimension, method of pile installation, and time effects. The values of α vary within wide limits and decrease rapidly with increasing shear strength. It is recommended that Figure 2.3 generally be used for adhesion calculations unless one of the special soil stratigraphy cases identified in Figure 2.4 is present at a site. In cases where either Figures 2.3 or 2.4 could be used, the inexperienced user should select and use the smaller value obtained from either figure. In Figure 2.3, the pile adhesion, α, is expressed as a function of the undrained shear strength, c_u with consideration of both the pile type and the embedded pile length, D, to pile diameter, b, ratio. The embedded pile length used in Figure 2.3 should be the minimum value of the length from the ground surface to the bottom of the clay layer or the length from the ground surface to the pile toe. (Hannigan et. al., 2016)

Figure 2.4 presents the adhesion factor, α, versus the undrained shear strength of the soil as a function of unique soil stratigraphy and pile embedment. The adhesion factor from these soil stratigraphy cases should be used only for determining the adhesion in a stiff clay layer in that specific condition. For a soil profile consisting of clay layers of significantly different consistencies such as soft clays over stiff clays, adhesion factors should be determined for each clay layer. (Hannigan et. al., 2016). The top graph in Figure 2.4 may be used to select the adhesion factor when piles are driven through sand or sandy gravel layer and into an underlying stiff clay stratum. The middle graph in Figure 2.4 should be used to select the adhesion factor when piles are driven through a soft clay layer overlying a stiff clay layer. In this case, the soft clay is dragged into the underlying stiff clay stratum thereby reducing the adhesion factor of the underlying stiff clay soils. (Hannigan et. al., 2016)

Last, the bottom graph in Figure 2.4 may be used to select the adhesion factor for piles driven in stiff clays without any different overlying strata. In stiff clays, a gap often forms between the pile and the soil along the upper portion of the pile shaft. In this case, the shallower the pile penetration into a stiff clay stratum the greater the effect the gap has on the shaft resistance that develops. (Hannigan et. al., 2016)

End Bearing

The net ultimate end bearing capacity can be obtained from the following equation:

$$Q_B = N_c\ c_uA_B \qquad (2.16)$$

Where,

c_u = undrained shear strength of soil (The undrained shear strength and undrained cohesion, c_u is assumed to be equal for calculations.)

A_B = cross-section area of pile tip

N_c= bearing capacity factor

The term N_c is a dimensionless bearing capacity factor which depends on the pile diameter and the depth of embedment. The bearing capacity factor, N_c is usually taken as 9 for deep foundations. In the case of smaller piles in cohesive soils, the toe resistance contribution to the nominal resistance is a low percentage of the overall resistance and is therefore sometimes ignored. On larger piles, the movement required to mobilize the toe resistance is several times greater than that required to mobilize the shaft resistance. At the movement required to fully mobilize the toe resistance, the shaft resistance may have decreased to a residual value. These factors should be considered when performing nominal resistance assessments of various pile sections. (Hannigan et. al., 2016).

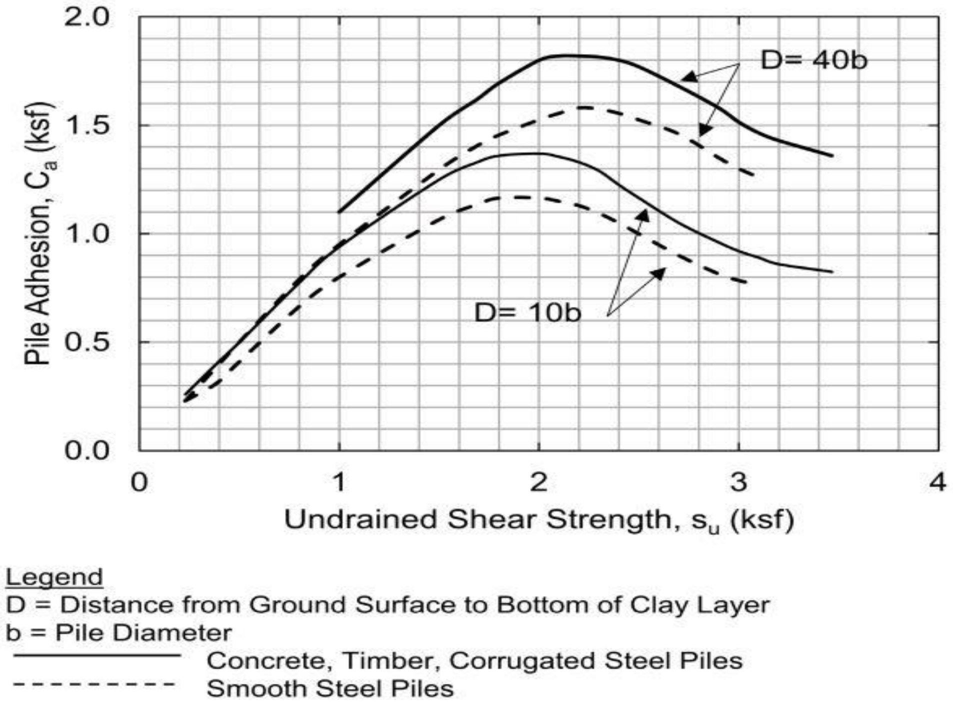

Figure 2.3: Adhesion values for piles in cohesive soil (Tomlinson 1994)

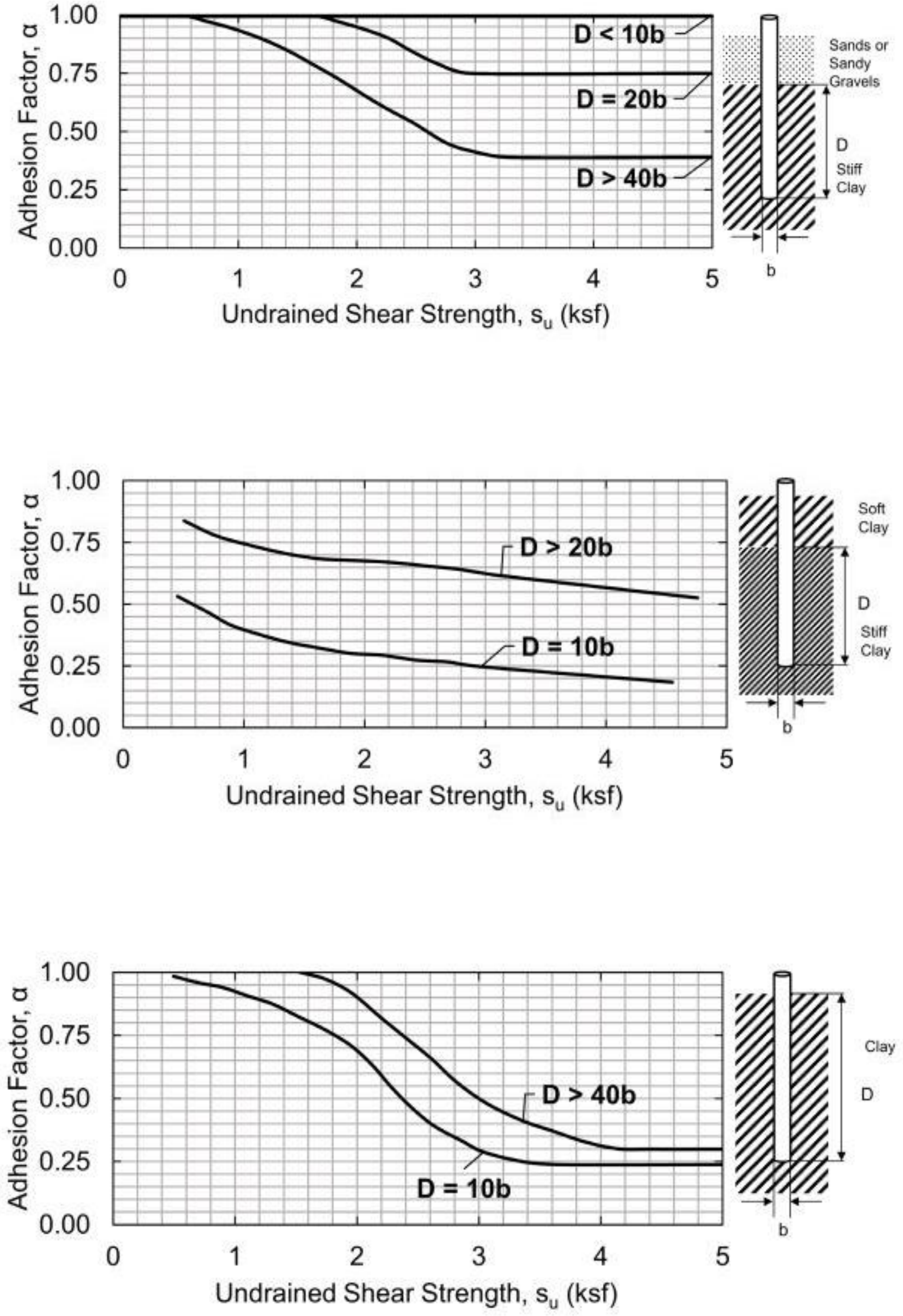

Figure 2.4: Adhesion factors for piles in clay (Tomlinson 1994)

2.2.1.4 Norwegian Pile Guideline (2005)

Side Friction

It is called a total stress approach used the estimation of the skin friction along the shaft of piles embedded in clay. Tomlinson (1957) initially proposed effective stress based general equation for the skin friction.

$f_s = \alpha c_u + P_0' K \tan \delta$ (2.17)

Later this equation is used simply as

$f_s = \alpha c_u$ (2.18)

Figure 2.5 represent that adhesion factor α decrease with the increasing of undrained shear strength suggested by API (1984), Peck et. al. (1974) and Tomlinson (1994).

Where

α = adhesion factor from Figure

c_u = Undrained shear strength for the point of interest

p_0' = average effective vertical stress

K = lateral earth pressure coefficient

δ = Soil Structure Interaction Angle

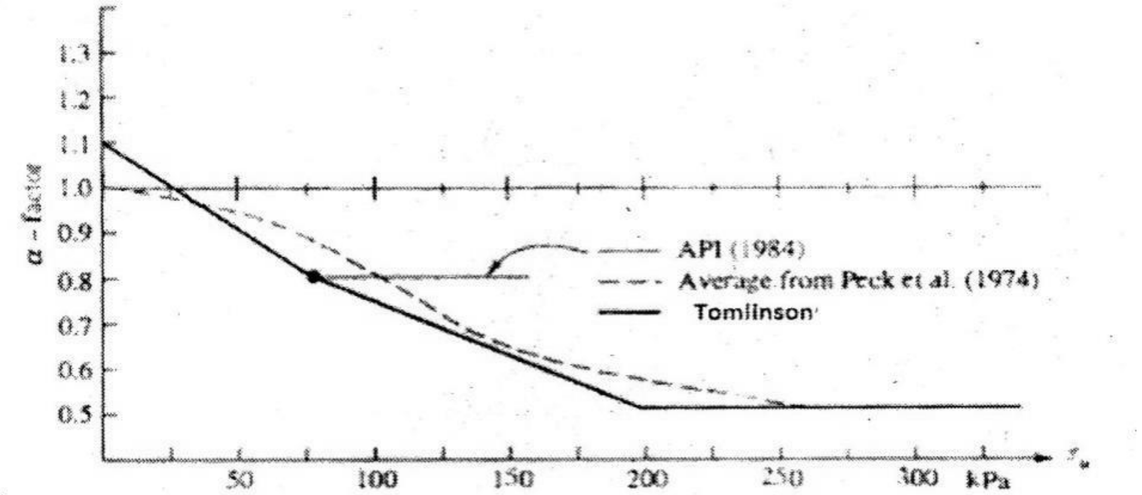

Figure 2.5: Relationship between the adhesion factor and undrained shear strength

According to the NPG guideline, the unit skin friction for cohesive soils along the pile shaft relates to the undrained strength through the following expression.

$f_s = \alpha c_u$ (2.19)

Total side resistance, $Q_s = f_s A_s$ (2.20)

Where

Q_s = Ultimate side resistance

α = an empirical factor

α is a function of length diameter ratio and ratio of the undrained shear strength & effective vertical stress. This adhesion factor α can be obtained from Figure 2.6 recommended by Gunnar et. al., (1991).

$$\text{Function} \left(\frac{L}{d}, \frac{c_u}{p_o} \right); \text{ see the figure}$$

L = pile length

d = width or diameter of pile

c_u = average undrained shear strength along the pile

p_o = average effective vertical stress along the pile

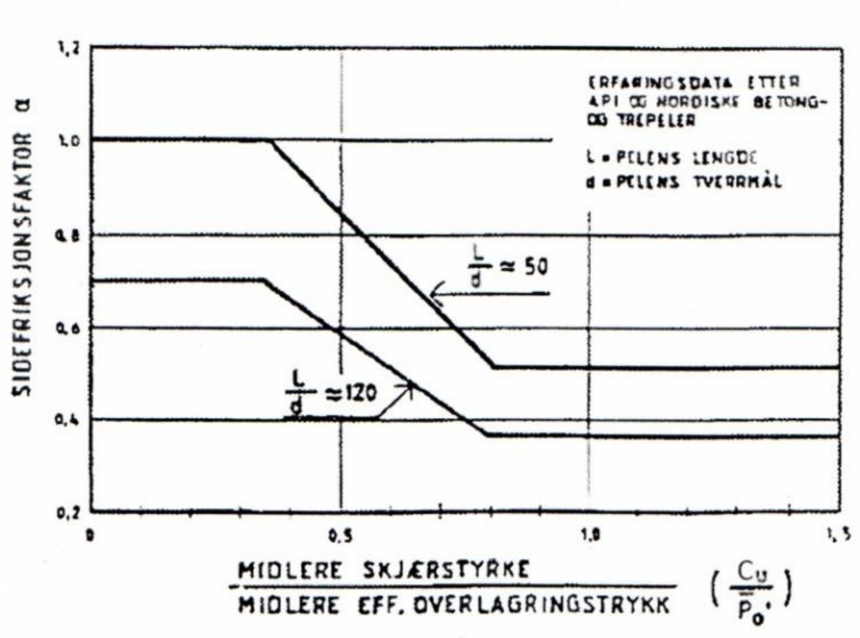

Figure 2.6: Shaft frictions according to prevailed design (after Gunnar et.al (1991))

End Bearing

The characteristics specific tip resistance (q_p)

$q_p = 9c_u$ (2.21)

Total end bearing, $Q_p = A_p q_p$ (2.22)

Where;

A_p = Cross sectional area of the pile tip.

c_u = undrained shear strength

2.2.1.5 Indian Standard (2010)

In cohesive soils, it is common to use a total stress analysis in which the load capacity is related to the undrained shear strength.

Side Friction

The skin friction of piles in cohesive soils, in kN, is given by the following formula.

$$Q_s = \sum_{i=1}^{n} \alpha_i \, c_i A_{si} \tag{2.23}$$

Where,

$\sum_{i=1}^{n}$ = Summation of layers 1 ton in which pile is installed and which contribute to positive skin friction

α_i = adhesion factor for the ***i*** th layer depending on the consistency of soil (The value of adhesion factor depends on the undrained shear strength of the clay and can be obtained from Figure 2.7)

c_i= average cohesion for the ith layer (kN/m^2)

A_{si} =surface area of the pile shaft in the ***i***th layer (in m^2) (Bureau of Indian Standards (BIS) 2010)

End Bearing

$$Q_B = A_p N_c c_p \tag{2.24}$$

Where,

A_p = cross-sectional area of pile tip (m^2)

N_c = bearing capacity factor (it is suggested to be taken as 9)

c_p = average cohesion at pile tip (kN/m^2) (Bureau of Indian Standards (BIS) 2010)

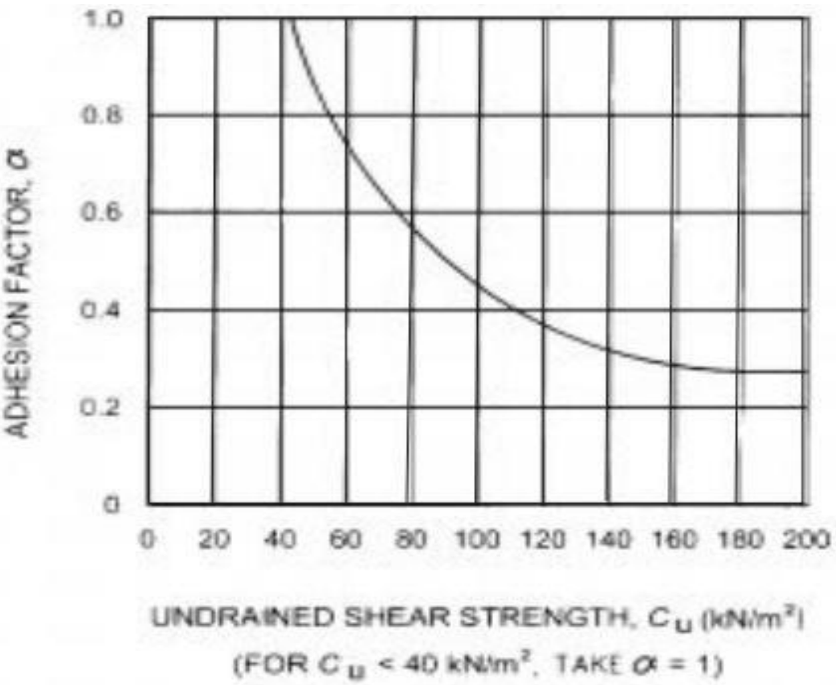

Figure 2.7: Variation of adhesion factor, α with undrained shear strength, c (Bureau of Indian Standards (BIS) 2010)

2.2.2 Axial Load Capacity of Driven Pile in Cohesionless Soil

2.2.2.1 Meyerhof's Method (1976)

Side Friction

The unit side friction can be calculated by equation (2.25). The value of unit side friction must be less than or equal to its limiting value, f_l

$$f_S = (K_s \tan\delta)\sigma'_v \leq f_l \tag{2.25}$$

Where,

f_S = unit side friction

K_s = lateral earth pressure coefficient (Ks can be obtained from Figure 2.9, which gives a relation between coefficient of earth pressure and angle of internal friction of soil)

δ = friction angle between pile material and soil = (2/3)ϕ ; (Here, ϕ is the angle of internal friction of soil. ϕ is obtained from Table: 2.1)

σ'v = effective vertical stress along the length of the pile = $\gamma' D_f \leq \gamma' D_c$ (the value of D_c is 15 to 20 times of pile diameter. (Meyerhof 1976)

f_l = limiting value of unit side friction = $\frac{N_{field}}{50}$ (in tsf), (here, N_{field} is the average SPT-N value, in blows per foot(blows per 0.3 m), within the embedded length of the pile). (Meyerhof, 1976)

For the determination of the angle of internal friction, ϕ from Table 2.1, the value of SPT-N must be corrected. The formula for this is given in equation (2.26). This formula is applied in case of overburden pressure.

$$N_{corr} = 0.77\log_{10} \frac{44.08}{\sigma'_v} \tag{2.26}$$

σ'_v =Effective stress in ksf

Hence, the side friction is as follows:

$$Q_s = f_s A_s \tag{2.27}$$

Where,

Q_s = total side friction

f_S = unit side friction

A_s = Shaft surface Area (ft^2)

End Bearing

According to Meyerhof, the unit end bearing capacity of a driven pile in cohesionless soil can be computed by equation (2.28). For piles in granular soil cohesion, c = 0.

$$q_b = N_q \sigma'_V \tag{2.28}$$

The variation of N_q with friction angle, ϕ is shown in Figure 2.8. And friction angle, ϕ is obtained from Table 2.1.

Where,

q_b = unit end bearing capacity

N_q = bearing capacity factor [For determining N_q Figure 2.8 is used and the parameter soil friction angle, ϕ used here is determined from Table 2.1]

σ'_v = effective vertical stress at the level of pile tip = $\gamma' D_f$

Where,

γ' = unit weight of soil (lb/ft^3)

D_f = Depth of pile (ft.)

However, q_b should not exceed the limiting end bearing value of q_{lim}, $q_b \leq q_{lim}$

Where,

$$q_{lim} = 0.5P_aN_q\tan\phi \qquad (2.29)$$

The pointed end bearing capacity of a pile generally increases with the depth of embedment of the pile and reaches a maximum value at a ratio of $(L_b/D)_{cr}$. For a homogenous soil, L = L_b, where L = actual embedment depth of the pile. Beyond the critical ratio, q_p remains constant ($q_p = q_{lim}$). Figure 2.2 represent the limiting unit friction after critical depth.

Here,

q_{lim} = limiting base resistance

P_a = atmospheric pressure (= 100 KN/m^2 or 2000 lb./ft^2)

N_q = bearing capacity factor (For determining N_q Figure 2.3 is used and the parameter soil friction angle, ϕ used here is determined from Table 2.1)

ϕ = angle of internal friction of soil (From Table 2.1)

The total end bearing is as follows:

$$Q_B = q_pA_p \qquad (2.30)$$

Where,

Q_B = total end bearing

q_p = unit end bearing

A_p = Area of end of the pile (ft^2)

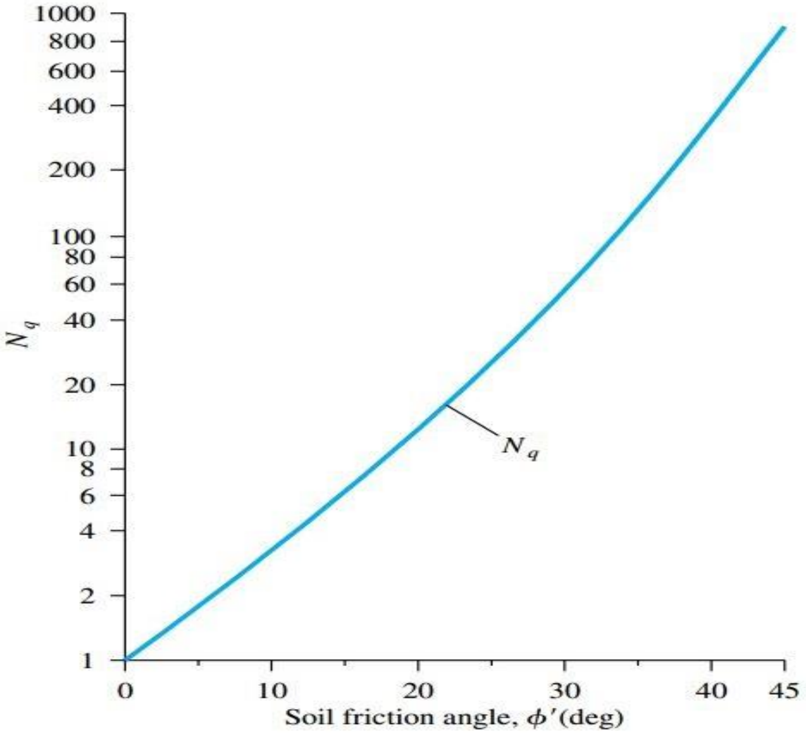

Figure 2.8: Variation of the bearing capacity factor Nq with soil friction angle φ. (Das 2002)

Table 2.1: Correlation between corrected SPT-N and Internal Friction Angle (ϕ) for Cohesionless soils (after Meyerhof 1956 ref by Hannigan et al. 2016)

State of packing	**SPT blow count N_{corr} (*blows/ft*)**	**Angle of internal friction, ϕ°**
Very loose	<4	<30
Loose	4-10	30-35
Medium Dense	10-30	35-40
Dense	30-50	40-45
Very dense	>50	>45

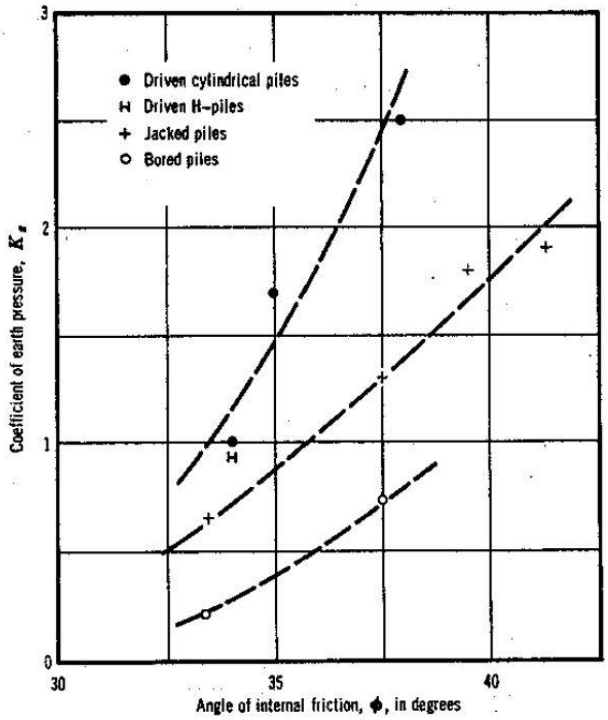

Figure 2.9: Relationship between the coefficient of earth pressure and angle of internal friction above critical depth. (Meyerhof, 1976)

2.2.2.2 American Petroleum Institute (1993) Method

The American Petroleum Institute (API) provides a static analysis procedure design developed for offshore construction. These projects almost exclusively use large diameter, open-end, steel pipe piles which are driven by impact hammer to final penetration (API 1993). Large diameter open end pipe piles can be either steel pipe piles or concrete cylinder piles. Recently, large-diameter open-end pipe pile usage has increased significantly on transportation projects. (Hannigan et. al., 2016)

Side Friction

When installing piles in cohesionless soils, the unit side friction can be determined by equation (2.31). (Hannigan et. al., 2016)

$$f_s = K_s \sigma'_v \tan\delta \qquad (2.31)$$

Where,

f_S = unit side friction

K_s = coefficient of lateral earth pressure for obtaining K_s value Figure 2.9 has been used.

σ'_v = vertical effective stress (ksf) at mid-point of layer = $\gamma' h$

δ = friction angle between the soil and the pile wall Obtained from Table 2.2

For obtaining the value of δ from Table 2.2, the density of soil particles is required. Which can be determined from Table 2.3. (Peck et. al., 1974)

API (1993) notes that assuming K_s = 0.8 for both tension and compression loading of unplugged, open-ended pipe pile is appropriate. Besides, for the plugged or closed-end case the assumption of K_s = 1.0 is recommended. (Hannigan et. al., 2016)

Hence, the side friction is as follows:

$$Q_s = f_s A_s \tag{2.32}$$

Where,

Q_s = total side friction

f_S = unit side friction

A_s = Shaft surface Area (ft^2)

So, the nominal shaft resistance = Sum of Shaft resistance from each layer.

Limiting values of unit toe resistances are applied for each type of cohesionless soil as shown in Table 2.2.

End Bearing

The unit end bearing for piles in cohesionless soils may be determined by the following relationship.

$$q_b = \sigma'_v N_q \tag{2.33}$$

Where,

σ'_v = Vertical effective stress at the end (ksf)

N_q = Dimensionless Bearing Capacity factor Obtained from Table 2.2

The total end bearing is as follows

$$Q_B = q_b A_b \tag{2.34}$$

A_b = Area of end of the pile (ft^2)

Table 2.2: Design Parameter Guidelines for Cohesionless Siliceous Soil.

(Hannigan et. al., 2016)

Density	**Soil**	**Soil-Pile friction angle, δ**	**Limiting unit shaft resistance (ksf)**	**N_q**	**Limiting unit toe resistance (ksf)**
Very Loose Loose Medium	Sand Sand-Silt* Silt	15	1	8	0
Loose Medium Dense	Sand Sand-Silt* Silt	20	1.4	12	60
Medium Dense	Sand Sand-Silt*	25	1.7	20	100
Dense Very Dense	Sand Sand-Silt*	30	2	40	200
Dense Very Dense	Gravel Sand	35	2.4	50	250

*In sand silt soils (soils with significant fractions of both sand and silt), the strength values generally increase with increasing sand fractions and decease with increasing silt fractions.

Table 2.3: Relationship between ϕ and standard penetration value for sands

(Peck et al. 1974)

SPT Penetration, N-Value (blows/ foot)	**Density of Sand**	**ϕ (degrees)**
<4	Very loose	<29
4-10	Loose	29-30
10-30	Medium	30-36
30-50	Dense	36-41
>50	Very dense	>41

2.2.2.3 Tomlinson’s Method (1994)

Side Friction

For coarse-grained soils, the side friction of the pile is given by the following formula. (Tomlinson 1994)

$$Q_s = 1/2 K_s \sigma'_v \tan \delta A_s \qquad (2.35)$$

Where,

K_s = coefficient of horizontal soil stress (depending on the installation method K_s/K_o values are given in Table 2.4. And typical values for K_o for a normally consolidated sand is given in Table 2.5) (Tomlinson, 1994)

σ'_v = effective overburden pressure along the pile shaft = $\gamma'h$

δ = angle of friction between pile and soil material (δ is obtained from the values given for various pile material to soil interface conditions given in Table 2.6. The required ϕ values are obtained from the relationship between SPT-N values and angle of internal friction of soil, which is given in Figure 2.10) (Peck et al. 1974)

A_s = Area of the shaft in contact with the soil

End Bearing

End bearing for cohesionless soils is given by the following relation:

$$Q_B = N_q \sigma'_v A_b \qquad (2.36)$$

Where,

A_b = Area of the base of the pile

σ'_v = effective overburden pressure at the pile base level

N_q = Bearing capacity factor

(The value of N_q is obtained from the relationship between the angle of internal friction of soil, ϕ and the penetration depth/width of the pile. The relationship developed by Berezantsev et al. (1961) is shown in figure 2.11. Vesic (1977) stated that these N_q values gave results which most nearly conform to the practical criteria of pile failure. The alternative is to use the Brinch Hansen Nq factors shown in figure 2.11. They should be multiplied by a shape factor of 1.3 to allow for the square or circular cross-section of pile base.) (Tomlinson and Woodward, 2008)

The value of the internal friction angle of soil is determined from Figure 2.10, which gives the relationship between SPT-N and ϕ. The most useful all-round test fort piling investigations is the standard penetration test. The blow counts (blows/0.3 m) have been correlated with the consistency and approximate unconfined compressive strength of soil by Terzaghi and Peck (1974), which is given in Table 2.7. (Tomlinson, 1994)

Table 2.4: Values of the coefficient of horizontal soil stress, Ks (Tomlinson, 1994)

Installation method	**K_0**
Driven piles, Large displacement	1.0-2.0
Driven piles, small displacement	0.75-1.25
Bored and cast-in-place piles	0.7-1.0
Jetted piles	0.5-0.7

Table 2.5: Typical values of coefficient of earth pressure at rest for normally consolidated sand (Tomlinson, 1994)

Relative density	**K_s/K_0**
Loose	0.5
Medium-dense	0.45
Dense	0.35

Table 2.6: Values of the angle of the pile to soil friction, δ for various interface conditions (Tomlinson, 1994)

Pile to soil interface condition	**Angle of pile to soil friction, δ**
Smooth (coated) steel to sand	0.5ϕ - 0.7ϕ
Rough (corrugated) steel to sand	0.7ϕ – 0.9ϕ
Precast concrete to sand	0.8ϕ – 1.0ϕ
Cast-in-place concrete to sand	1.0ϕ
Timber to sand	0.8ϕ – 0.9ϕ

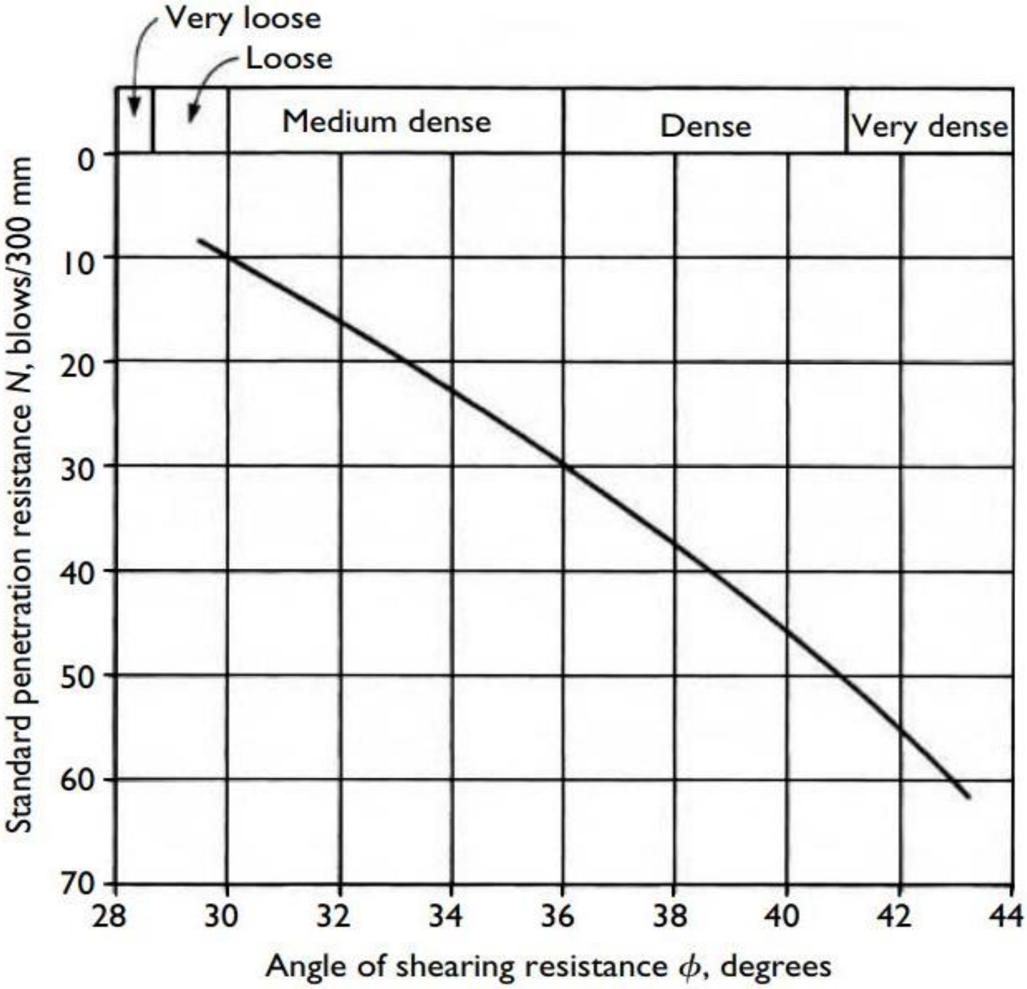

Figure 2.10: Relationship between standard penetration test N-values and angle of shearing resistance of soil, φ (after Peck et al. 1974 ref by Tomlinson, 1994)

Table 2.7: Relationship between consistency and unconfined compressive strength of with SPT-N (after Terzaghi and Peck (1974))

N-value (blows/300 mm)	**Consistency**	**Approx. unconfined compressive strength, (kN/m^2)**
<2	Very soft	>25
2-4	Soft	25-50
4-8	Medium	50-100
8-15	Stiff	100-200
15-30	Very stiff	200-400
>30	Hard	>400

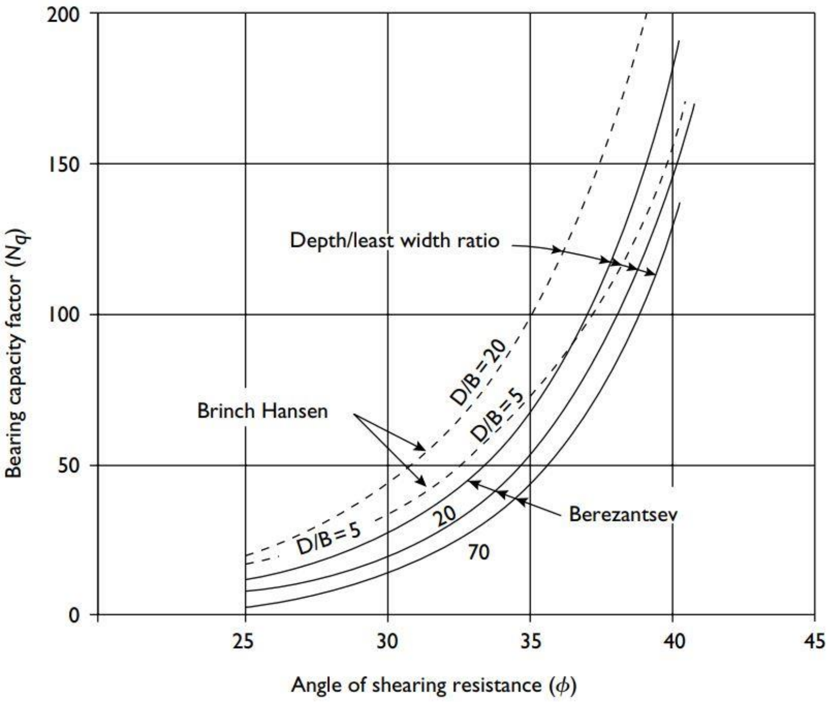

Figure 2.11: Bearing capacity factors of Berezantstev et al. (1961) and Brinch Hansen (1978)

2.2.2.4 Norwegian Pile Guideline (1991) Method

Side Friction

Recommend the drained bearing capacity of skin friction for the whole pile length based on the average characteristics specific side friction along with the pile as

$f_s=\beta\overline{P_0}$ (2.37)

Where,

$\beta = (0.4 \pm 0.1) \text{ x } \frac{L+20}{2L+20} O_{cr}^{0.5}$ (2.38)

P_o = Average effective vertical overburden pressure along pile.

O_{cr} = Over consolidation ratio (Average)

$O_{cr} = \frac{\sigma\prime p}{\sigma v}$ (2.39)

$\sigma' p = P_a \text{ x } 0.47 (N_{cor})^{0.7}$ (2.40)

P_a = Atmospheric

β = Empirical side friction factor

L = Pile length

σ'_v = Effective vertical stress

End Bearing

The characteristics specific tip resistance (q_p)

$q_p = N_q P_p'$ (2.41)

Bearing capacity factor N_q determined from Figure 2.12 referred by Peleveiledningen (1991).

Where;

N_q = Bearing capacity factor

P'_p = effective vertical overburden pressure at pile tip.

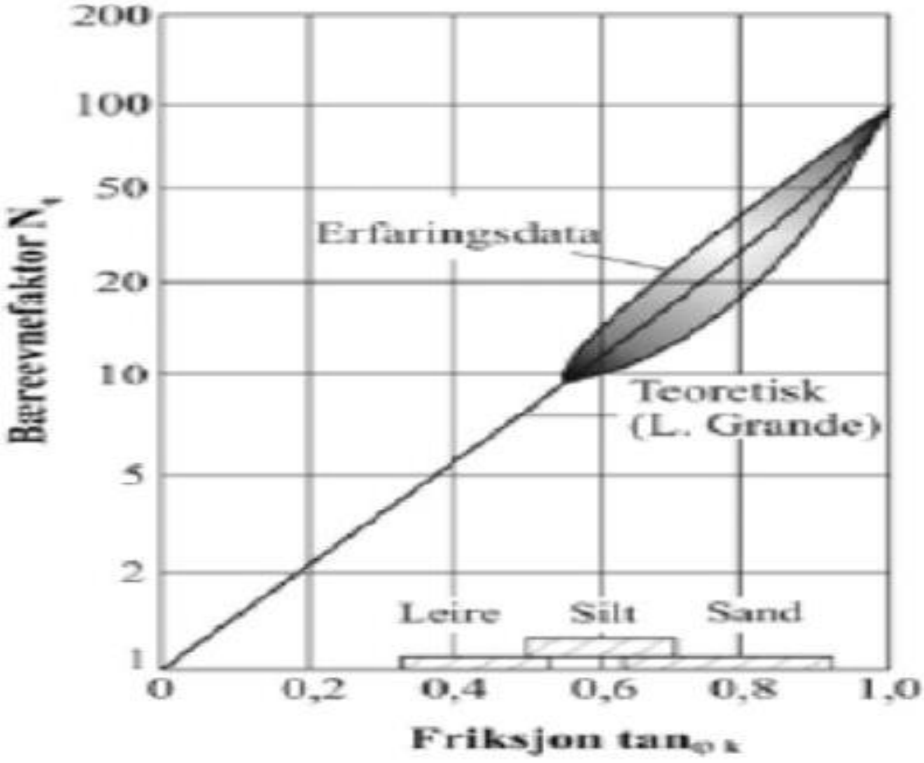

Figure 2.12: Bearing capacity factor in sand according to Peleveiledningen (1991)

2.2.2.5 Indian Standard (2010) Method

Side Friction

In this method, the side friction of driven pre-cast piles is given by the following formula.

$Q_S = \sum_{i=1}^{n} K_i \, \sigma'_{vi} \tan \delta_i \, A_{si}$ (2.42)

Where,

$\sum_{i=1}^{n}$ = summation for layers 1 to n in which pile is installed and which contribute to positive skin friction

K_i = coefficient of earth pressure applicable for the i^{th} layer (it depends on the nature of the soil strata, type of pile, a spacing of pile and it's a method of construction. For driven piles in loose to dense sand with ϕ varying between 30° and 40°, K_i values in the range of 1 to 2 may be used. The value of the angle of internal friction of soil, ϕ is obtained from Figure 2.13) (Kisan et al. 1981)

σ'_{vi} = effective overburden pressure for the i^{th} layer = $\gamma' h$ (in kN/m^2)

δ_i = angle of friction between pile and soil for the i^{th} layer (it is taken equal to the friction angle of soil, ϕ) (Kisan et al. 1981)

A_{si} = surface area of the pile shaft in the i^{th} layer (m^2)

End Bearing

End bearing for granular soils, in k_N is given by the following formula.

$$Q_B = A_b\left({}^1\!/_2\, D\gamma N_\gamma + \sigma'_v N_q\right) \tag{2.43}$$

Where,

A_b = cross-sectional area of the pile tip (m^2)

D = diameter of the pile shaft (m)

γ = effective unit weight of the soil at the pile tip

N_q = bearing capacity factor depending upon the angle of internal friction of soil at the pile tip (the values for N_q are determined from Figure 2.13. The value of angle of internal friction of soil, ϕ at pile tip is used to read the value of N_q and ϕ is determined from Figure 2.14 which gives the relationship between ϕ and SPT-N) (Bureau of Indian Standards (BIS) 2010)

N_γ = bearing capacity factor depending upon the angle of internal friction of soil, ϕ at pile tip (this factor can be taken from Table 2.8) (Kisan et al. 1981)

γ = effective unit weight of the soil at pile tip (kN/m^3)

σ'_v = effective overburden pressure at pile tip (kN/m^3)

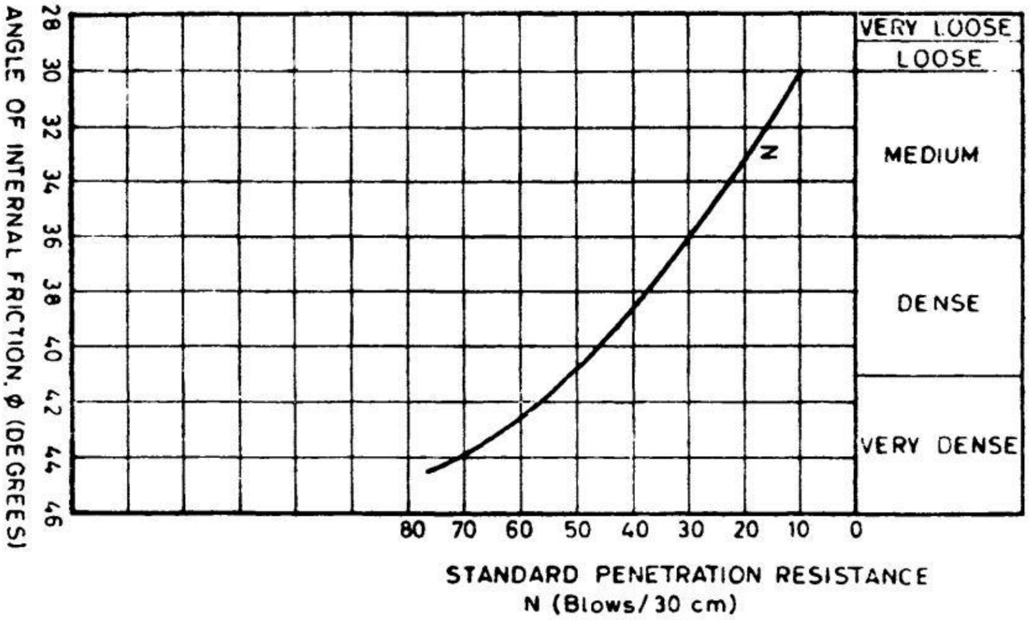

Figure 2.13: Relationship between the angle of internal friction of soil, ϕ and SPT-N (N_{field}) (Kisan et. al., 1981)

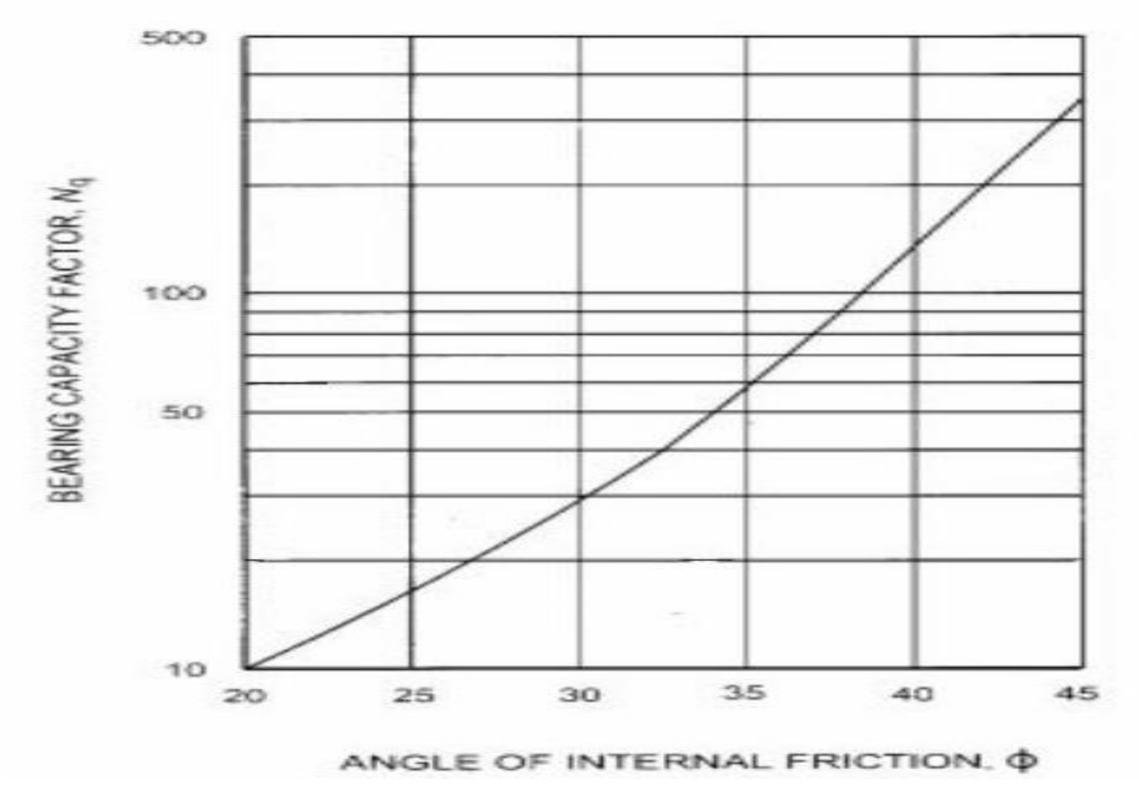

Figure 2.14: Relationship between bearing capacity factor N_q and angle of internal friction of soil, ϕ (Bureau of Indian Standards (BIS) 2010)

Table 2.8: Relationship between soil friction angle, ϕ and bearing capacity factor, Nγ (Kisan et al. 1981)

ϕ (Degrees	**Bearing capacity factor, Nγ**
0	0.00
5	0.45
10	1.22
15	2.65
20	5.39
25	10.88
30	22.40
35	48.03
40	109.41
45	271.76
50	762.89

2.2.3 Axial Load Capacity of Bored Pile and Drilled Shaft in Cohesive Soil

2.2.3.1 Meyerhof (1976) Method

Side Friction

The Meyerhof method is an empirical procedure based on load test data and allows the computation of capacity in sands and clays. Skin friction in clays is taken as zero when the base is resting on soil significantly stiffer than the soil around the stem. But Meyerhof has suggested a semi-empirical relationship for estimating skin friction in clays. By utilizing a value of 20 degrees for φ for the stiff to very stiff clays, the expressions reduce to

$$f_s = 0.36c_u \qquad (2.44)$$

Here,

f_s = Unit side resistance

c_u = Undrained shear strength

End Bearing

For calculating base resistance the same equation is used which was used for calculating base resistance for sand which is,

$$q_u = \frac{0.133.\bar{N}.D}{B} \leq q_1 \qquad (2.45)$$

Where

$\bar{N} = C_N * N$,

N = standard penetration resistance (blow/ft),

$C_N = 0.77 \log_{10} 20/p$ (for p ~ 0.25 tsf),

p = effective overburden stress at shaft tip (tsf),

D = depth drilled into granular bearing stratum (ft),

B = width or diameter of shaft (ft), and

Q_1 = limiting point resistance (tsf), N for clay.

but According to Meyerhof base resistance values for clays are taken as 9 times the undrained strength near the base.

2.2.3.2 NAVFAC DM 7.2 (1984) Method

Experience demonstrates that pile driving permanently alters the surface adhesion of clays having shear strength greater than 500 psf (Figure 2.15). In softer clays the remolded material consolidates with time, regaining adhesion approximately equal to original strength. Shear strength for point- bearing resistance is essentially unchanged by pile driving. For drilled piers, use C_A= 0.3 and f_s = 0.5 tsf from recommendation published in paper "soils and Geology, Procedures for foundation design of buildings and other structures", by lie departments of army and air force, for determining side friction. Ultimate resistance to pullout cannot exceed the total resistance of reduced adhesion acting over the pile surface or the effective weight of the soil mass which is available to react against pullout. The allowable sustained pullout load usually is limited by the tendency for the pile to move upward gradually while mobilizing an adhesion less than the failure value.

Bearing capacity factors in Figure 2.16 may be very conservative for evaluating piles driven into stiff but normal consolidated clays. Available data suggest that for piles driven into normally to slightly over consolidated clays, the side friction is about 0.25 to 0.4 times the effective overburden. For drilled piers, greater than 24 inches in diameter settlement rather than bearing capacity may control. A reduced end bearing resistance may result from entrapment of betonies slurry if used to be usually not stable in granular soils.

Side Friction

$f_s = C_A$ (2.46)

Where

f_s = unit skin friction

C_A = Adhesion factor

End Bearing

$q_b = c\ N_{cs}$ (2.47)

Where

q_b = unit end bearing capacity.

c = Su = Chesion or undrained shear strength of soil at the tip of the pile.

N_{cs} = Bearing capacity factor

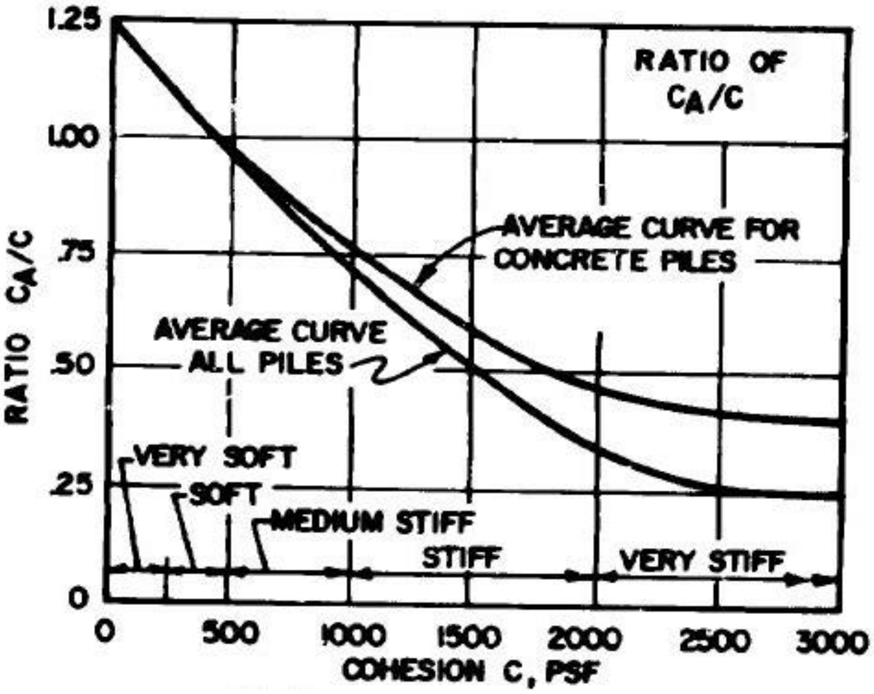

Figure 2.15: Relationship between Cohesion and the ratio of adhesion factor & Cohesion

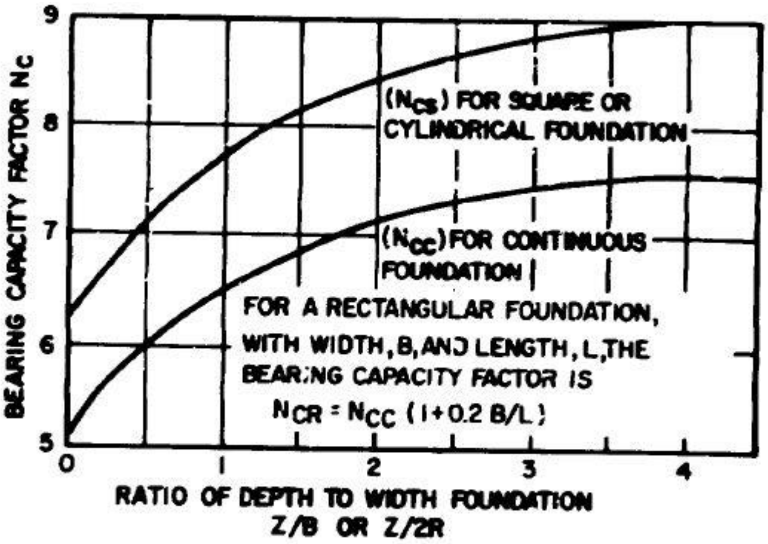

Figure 2.16: Bearing Capacity factor (Recommended by NAVFAC)

2.2.3.3 AASHTO (1986) Method

Side Friction

Unit shaft resistance (fs_z)

$$f_{sz} = \propto_z * c_u \tag{2.48}$$

Recommended Value of $\propto_z$ determined from Table 2.9

Where

f_{sz} = Ultimate load transfer in side resistance at depth Z.

c_u = Undrained shear strength at depth Z.

$\propto_z$ = empirical factor that can vary with depth Z.

Table 2.9: Recommended values of α for drilled shafts and bored piles in clay

Location along shafts	**Value of α**	**Limiting the value of load transfer, *f sz* (tsf)**
From ground surface to depth along with DS of 5 feet*	0	-
Bottom 1 diameter of the DS or 1 stem Diameter above the top at the bell (if skin friction is being used)	0	-
All other points along the sides of the DS	0.55	2.75

* The depth at 5ft may need adjustment if the drilled shaft is installed in expansive cloy, or if there is substantial ground line deflection from lateral loading.

End Bearing

Unit end bearing q_b

$q_b = N_c\, c_u$, Limiting value $q_b \leq 40$ ts*f*. (2.49)

Where:

$N_c = 6.0\ [1+0.2\ (L/B_b)]$; $N_c \leq 9$. (Limiting value of N_c) (2.50)

c_u = average undrained shear strength of the clay (the value is computed over a depth of one to two diameters below the base but judgment must be used if the shear strength varies strongly with depth)

L= Penetration of the shaft.

B_b = diameter of the base of the shaft. Special soft consistency clay, at the base the value of c_u (or N_c) may be reduced by about one-third ($^1/_3$) to account for local (high strain) bearing failure

When $B_b \geq 75$ inches.

Then; following expression be used to reduce q_b to q_{br}, where q_{br} is the net reduced ultimate end bearing stress:

$q_{br} = F_r.\ q_b$. (2.51)

Where;

$F_r = 2.5/[aB_b(in)+2.5b]$; $Fr \leq 1.0$ (2.52)

in which.

$a = 0.0071+0.0021\ (L/B_b)$;

$a \leq 0.015$. (2.53)

$b = 0.45\ [c_u]^{0.5}$;

$0.5 \leq b \leq 1.5$. (2.54)

2.2.3.4 O'Neill and Reese (1988) Method

Axial Capacity

Massive rock and cohesive materials occupy common properties. They seize low drainage qualities under normal loadings but drain more rapidly under large loads than cohesive soils. For these reasons, undrained shear strengths are used for rocks and IGMs. If the base of the pier lies in cohesive IGM or rock, the bearing capacity may be expressed as:-

Total axial capacity = Side resistance + End bearing

Side Resistance

Short-term undrained side resistance in cohesive soil layers is evaluated in terms of undrained shear strength.

$$R_{SN} = \pi B \Delta Z f_{SN} \tag{2.55}$$

Where:

R_{SN} = Nominal side resistance

B = shaft diameter

ΔZ = thickness of the soil layer over which resistance is calculated

S_u = average undrained shear strength over the depth interval

α = coefficient relating unit side resistance to undrained shear strength

f_{sn} = nominal unit side resistance

Evaluation of α is as follows:

- α = 0, between the ground surface & depth of 5 ft or to the depth of seasonal moisture change, whichever is greater
- α = 0.55 along the remaining portion of the shaft for $\frac{S_u}{P_a} \leq 1.5$
- α = 0.55 – $0.1(\frac{S_u}{P_a} - 1.5)$ along remaining portions of the shaft $1.5 \leq \frac{S_u}{P_a} \leq 2.5$
- P_a = atmospheric pressure in the same units as S_u (2116 psf or 14.7 psi)

End Bearing

Bearing capacity theory applied to the case of a deep foundation bearing on a cohesive soil, in terms of total stress analysis, yields the following approximate expression which is sufficient for design (O'Neill and Reese, 1988):

$q_{BN} = N_c s_u$ (2.56)

N_c Value determined from Table 2.10 for undrained shear strength recommended by O'neill & Reese (1988).

Where

N_c = bearing capacity factor

s_u = average undrained shear strength over the depth interval

Table 2.10: Values of N_c for different undrained shear strength according to O'neill and Reese (1988)

Undrained shear strength, s_u (lb./ft^2)	***N_c***
500	6.5
1000	8
2000	9

2.2.3.5 Decourt (1995) Method

Decourt investigated a lot of field load test data to established pile capacity from field SPT value in 1982.In 1995 some modifications have taken place in his empirical formula.

Side Friction

$f_s = \propto(2.8N_s+10)$ (2.57)

f_s = Unit Shaft Resistance

$\propto$ = adhesion factor = 1 for clay

N_s = average value of N_{field} around pile embedment depth

End Bearing

$q_p = K_p N_p$ (2.58)

q_p = Unit Base Resistance (MPa)

K_b = 0.08

N_b = Average of N_{field} around pile base

2.2.4 Axial Load Capacity of Bored Pile and Drilled Shaft in Cohesiveless Soil

2.2.4.1 Meyerhof (1976) Method

The Meyerhof method is an empirical procedure based on load test data and allows the computation of capacity in sands and clays.

Side Friction

The ultimate unit skin friction fs of drilled shafts in sands is computed using the equation

$$f_s = \frac{N}{100} \leq 0.5\ tsf \qquad (2.59)$$

Where,

N = the standard penetration blow count along the shaft.

This is half of the skin friction specified for driven piles in sands. For shafts in soils with no soil of exceptional stiffness below the base, the average cohesion is reduced by a factor of 0.15 to 0.6 and applied to the area of the shaft 5 ft beneath the ground surface to 5 ft above the base or top of the bell.

End Bearing

The ultimate base bearing pressure, qu, in tsf is calculated with the following equation:

$$f_s = \frac{N}{100} \leq 0.5\ tsf \qquad (2.60)$$

Where

$\bar{N} = C_N * N$,

N = standard penetration resistance (blow/ft),

$C_N = 0.77 \log_{10} 20/p$ (for p ~ 0.25 tsf),

p = effective overburden stress at shaft tip (tsf),

D = depth drilled into granular bearing stratum (ft),

B = width or diameter of shaft (ft), and

Q1= limiting point resistance (tsf), equal to 1.33N for sand.

According to Meyerhof, the ultimate base resistance for driven piles in sands is three times the value allowed for drilled shafts in similar materials.

2.2.4.2 NAVFAC DM 7.2 (1994) Method

Side Friction

$S = K.\sigma'_v \tan\delta \ Ap$ (2.61)

Where,

S= Skin friction of the pile

σ'_v = effective stress at the midpoint of the pile

K = lateral earth pressure coefficient

δ = pile skin friction angle

Table: 2.11 Pile skin friction angle (δ) for different materials [NAVFAC DM 7.2 (1994)]

Pile type	δ
Steel piles	20°
Timber piles	¾ϕ
Concrete Piles	¾ϕ

δ is the skin friction angle between pile material and surrounding sandy soils usually a smooth surface tends to have less skin friction compared to a rough surface. This value of δ obtained from Table 2.11. Lateral earth pressure coefficient found from 2.12.

Table 2.12: Lateral earth pressure coefficient (K) [NAVFAC DM 7.2 (1994)]

Pile type	**K (Piles under compression)**	**K (Piles Under tension)**
Driven H-Piles	0.5-1.0	0.3-0.5
Driven displacement Piles (Round Square shape)	1-1.5	0.6-1.0
Driven displacement tapered piles	1.5-2.0	1.0-1.3
Driven jetted piles	0.4-0.9	0.3-0.6
Bored piles (less than 24" diameter)	0.70	0.4

End Bearing

$q = \sigma'_t Nq$ (2.62)

q = End Bearing Capacity of the Pile (Unit Same as σ'_t)

σ'_t = effective stress at pile tip.

Nq = Bearing capacity factor.

Nq = obtained from Table 2.13.

Table 2.13: Friction angle (ϕ) vs Nq [NAVFAC DM 7.2 (1994)]

ϕ	26	28	30	31	32	33	34	35	36	37	38	39	40
Nq	10	15	21	24	29	35	42	50	62	77	86	120	145
Nq (Bored Pile)	5	8	10	11	14	17	21	25	30	38	43	60	72

If water jetting is used, ϕ should be limited to 28°. This is because water jets tend to loosen the soil. Hence, higher friction angle values are not warranted.

2.2.4.3 AASHTO (1986) Method

Side Friction

Unit shaft Resistance (fs)

$f_s = \beta\sigma' \quad \leq 2.0\text{tsf.}$ (2.63)

The limiting value of shaft resistance shown in equation (2.63) is not a theoretical limit but is the largest value that has been measured (Owens and Reese, 1982). The use of higher values should be justified by results from a load test.

From O'Neill and Reese, (1982)

β=1.5-0.135$Z^{0.5}$, 1.2 ≥ β ≥ 0.25. (2.64)

Z = depth below ground surface, ft.

σ'_Z = vertical effective stress in the soil at depth Z.

f_{sz}" remains constant below 85.7 feet.

End Bearing

Values of unit end bearing (q_b) are tabulated as a function of NSPT (uncorrected field values) in Table 2.14. However, the values in the table may have to be reduced for large diameter shafts, as shown by the following Equation:

Here; Reduction factor using for diameter greater than 50 inch

∴ Reduced ultimate base resistance:

$$q_{br} = \frac{50}{B_b} q_b. \quad (2.65)$$

B_b = the diameter of the base of the shaft (in inch)

Table 2.14: Recommended values of unit end bearing for cohesionless soil

Range of Value of NSPT (Uncorrected)	Value of q_b (Tsf)
0 to 75	0.6 N_{SPT}
Above 75	45

* Ultimate value or value at the settlement of 5 percent of base diameter.

2.2.4.4 O'Neill and Reese (1988) Method

Side Friction

The nominal side resistance of a drilled shaft in cohesionless soil can be expressed as the frictional resistance that develops over a cylindrical shear surface defined by the soil-shaft interface. The unit side resistance is directly proportional to the normal stress acting on the interface. Nominal side resistance is then given by:

$$R_{SN} = \pi B \Delta Z f_{SN} = \pi \, \mathrm{B} \Delta Z \, (\sigma'_v \, \mathrm{K} \tan \delta) \quad (2.66)$$

Where:

R_{SN} = Nominal side resistance

B = shaft diameter

ΔZ = thickness of the soil layer over which resistance is calculated

σ'_v = average vertical effective stress over the depth interval ΔZ

K = coefficient of horizontal soil stress

σ'_h = horizontal effective stress δ = effective stress angle of friction for the soil shaft interface (here we let δ is equal to frictional angle φ, as φ is the limiting value for δ. The value of δ cannot be greater than φ)

f_{SN} = unit side resistance

For convenience, the following term may be combined:

$$\beta = K\tan\delta \tag{2.67}$$

$$f_{SN} = \sigma'_v \beta \tag{2.68}$$

In which, β = side resistance coefficient

f_{SN} = nominal unit side resistance.

β is calculated solely as a function of depth below the ground surface, without explicit consideration of soil strength or the in-situ state of stress.

Here,

$$\varphi' = \delta = 27.5 + 9.2 \log [N] \tag{2.69}$$

$$K = (1 - \sin\varphi')\, OCR^{\sin\varphi'} \leq K_P \tag{2.70}$$

$$K_P = tan^2(45^0 + \frac{\varphi'}{2}) \tag{2.71}$$

$$\text{OCR} = {\sigma'_p}/{\sigma'_v} \tag{2.72}$$

$$\sigma'_p = pa\ 0.47\ (N_{60})^m \tag{2.73}$$

Where

δ = angle of friction for the soil-shaft interface

σ'_p = Effective vertical pre-consolidation stress

OCR = Over consolidated pressure

K_P= Passive earth pressure

m = 0.6 for clean quartzitic sands and m = 0.8 for silty sands to sandy silts

P_a= atmospheric pressure in the same units as σ'p (Value of P_a 2116 psf).

End Bearing

Direct empirical correlations between SPT N-values and mobilized base resistance determined from load tests recommended for design:

q_{BN} (tsf) = $0.60N_{60}$

(Shall not be greater than 30 tsf) (2.74)

In which,

q_{BN} = nominal unit base resistance and

$N_{field} = N_{60}$ = the average value between the base and two diameters beneath the base.

Total axial capacity= Side resistance in layers + End bearing

2.2.4.5 Decourt (1995) Method

Decourt investigated a lot of field load test data to established pile capacity from field SPT value in 1982. In 1995 some modifications have taken place in his empirical formula.

Side Friction

$f_s= \propto(2.8N_s+10)$ (2.75)

$Q_s = f_sA_s$ (2.76)

f_s= Unit Shaft Resistance (KPa)

$\propto$=Adhesion factor=0.5-0.6

N_s = average value of N_{field} around pile embedment depth.

End Bearing

$q_p = K_pN_p$ (2.77)

$Q_p = A_pq_b$ (2.78)

q_b = Unit Base Resistance (MPa)

K_b = Co-efficient

N_b = Average N_{field} around pile base

K_b = 0.325

2.3 Pile Capacity by Static Load Test

For projects involving pile foundations, it is usually necessary to confirm the actual ultimate compression capacity of the pile concerning the theoretical ultimate pile capacity. Often this is confirmed by performing a static load test on the test pile. The ultimate pile compression capacity can roughly be defined as the load for which rapid pile movement occurs under sustained or slight increase of the applied load or when the pile plunges. However, often distinct plunging ultimate load is not obtained during the test. Therefore, the pile ultimate capacity or failure load must be determined by some criterion using load-settlement data recorded in the test. Various researchers in the past suggested different methods for evaluation of pile ultimate capacity.

2.3.1 Methodology for Pile Load Test

Though methodology for pile load test is not within the scope of this study, this is discussed here briefly because the method of loading has a great impact on the carrying capacity of piles and it should be recorded in the report for the further conclusion if required in the future. Maintained loading static axial compression test was carried out on the test piles following the standard procedure outlined in ASTM D I 143-81 (1989) "Standard Test Method for Piles Under Static Axial Compressive Load". All the load tests were conducted with the application of load equal to two times the allowable load. The loads were applied in eight equal increments. Two strain dial gauges were placed each on either side of the pile to measure the vertical settlement of the collar firmly attached to the pile top concerning the reference beams. The reference beams were finely supported in the ground at enough distance away from both the pile and supports of the loading platform.

Aller applying the load on pile head through hydraulic jack, the settlements of the pile were recorded at 30 sec, 1, 2, 5, 10, 15, 30, 60 and 90 minutes intervals on strain gauges rested on reference beams and attached with the pile head. Each load increment was maintained until the rate of the settlement was not greater than 0.25 mm/hr. or until 2 (two) hours had c1apsed, whichever occurred first.

2.3.1.1 The Davisson Offset Limit Load

The ultimate load, as proposed by Davisson (1972), is the load corresponding to the movement that exceeds the elastic compression of the pile by a value of 0.15 inches (4 mm), plus soil quake, a factor equal to the diameter of the pile divided by 120. Soil quake is the deformation (or pile movement) required to mobilize the strength of the soil below the pile tip (NeSmith and Siegel, 2009). This method is probably the best known and widely used in North America and other regions worldwide because it provides the lowest estimate of axial compression capacity from the actual load-settlement curve without any requirement of extrapolation. The method is based on the assumption that capacity is reached at a certain small toe movement and tries to estimate that movement by compensating for the stiffness (length and diameter) of the pile. It is primarily intended for test results from driven piles tested according to quick methods. However, Davisson's method requires the pile to be loaded near failure to be applicable.

2.3.1.2 The Hansen 80-% Criterion (Fellenius, 2001)

J. Brinch Hansen in the year 1963, proposed a definition for pile capacity as the load that gives four times the movement of the pile head as obtained for 80% of that load. This '80%- criterion' can be estimated directly from the load movement curve but is more accurately determined in a plot of the square root of each movement value divided by its load value and plotted against the movement. Following simple relations can be derived for computing the capacity or ultimate resistance, Qu, according to the Hansen 80%-criterion for the Ultimate Load:

$$Q_u = \frac{1}{2\sqrt{C_1 C_2}} \qquad Q_u = \frac{1}{2\sqrt{0.0006 * 0.0335}} = 111.52\tau \tag{2.79}$$

Where Qu = capacity or ultimate load, C1 = slope of the straight line, C2 = y-intercept of the straight line.

2.3.1.3 Chin-Kondner Extrapolation

Chin (1970) proposed an application to piles of general work by Kondner (1963). Chin assumes that the relationship between load and settlement is hyperbolic. The method is similar to the Hansen method. To apply the Chin-Kondner method, divide each settlement with its corresponding load, and plot the resulting value against the settlement. After some initial variation, the plotted values will fall on a straight line. The inverse slope of this line is the Chin-Kondner Extrapolation of the ultimate load.

$$Q_u = \frac{1}{C_1} \qquad Q_u = \frac{1}{0.0082} = 121.95\tau \tag{2.80}$$

Where Q_u = applied load,
C_1 = slope of the straight line

Normally the correct straight line does not start to materialize until the test load has passed the Davisson Offset Limit. As an approximate rule, the Chin-Kondner Extrapolation load is about 20% to 40% greater than the Davisson limit. When this is not a case, it is advisable to take a closer look at all the test data. The Chin method appliesto both quick and slow tests, provided constant time increments are used.

2.3.1.4 Decourt Extrapolation (1999)

Decourt (1999) proposes a method in which the construction is similar to that used in the Chin-Kondner and Hansen methods. To apply the method, divide each load with its corresponding movement and plot the resulting value against the applied load. The Decourt extrapolation load limit is the value of the load at the intersection. The Decourt extrapolation load limit is equal to the ratio between the y-intercept and the slope of the line as given in the equation below.

$$Q_u = \frac{C_2}{C_1} \qquad Q_u = \frac{24.796}{0.2061} \quad Q_u = 120.31\tau \tag{2.81}$$

2.3.1.5 Indian Standard (2010) Method

Indian Standards (IS) code (1979) recommends that the ultimate capacity of the pile is smaller of the following two-

a) Load corresponding to a settlement equal to 10% of the pile diameter
b) Load corresponding to a settlement of 12 mm

The same code states that the allowable pile capacity is smaller of the following:

a) Two-thirds of the final load at which total settlement is 12 mm.
b) Half of the final load at which total settlement is equal to 10% of the pile diameter.

2.3.1.6 BNBC (2007) Method

The Bangladesh National Building Code (1993) recommends that the allowable load capacity of the pile shall not be more than one half of that test load which produces a permanent settlement (i.e. gross settlement less rebound) of not more than 0.00028 mm/kg of test load nor 20mm.

2.4 Current Status of Pile Load Test Results in Bangladesh

Generally maintained load static axial compression tests are carried out on the test piles following the Standard procedure outlined in ASTM D1l43 (ASTM1989). After pile-driving, at least a month is allowed before the compressive load capacity of the piles being carried out. The steps are as follows:

- Recording of load-time-settlement data during the progress of loading and unloading the test piles.
- Analyses of test data, presentation of test data in graphical forms, and interpretation of the test results to determine the ultimate and design (i.e., allowable) capacity of the test piles.

Several projects have been carried out in Bangladesh by PWD (Public Works Department, Bangladesh) to estimate the ultimate load capacity of large diameter cast-in-situ piles and pre-cast piles of small and large dimensions. Some projects have also been carried out on a prestressed pile of small dimensions. Load tests were performed on both test and service piles. The majority of the tests were carried out under the full-time supervision of BUET (Bangladesh University of Engineering and Technology) consultants. The results of pile load tests have been reported by several researchers (Abedin et al., 1998; Ansary et al., 1999; Sadek, 1998; Khan, 1997). Table 2.15 represent the summary of information of static load test in Bangladesh (after Ansary et al. 1999)

Abedin et al. (1998) reported that the small dimension concrete piles are a viable alternative to replace the wooden piles that are prone to deterioration in alternative wetting and drying conditions. He also stated that the static formula for pile capacity estimation in soft ground is in general conservative. He suggested further study to generalize the ultimate static capacity of piles in Bangladesh.

Ansary et al. (1999) summarized the pile load test performed by BUET in different sites of Bangladesh as consultants of PWD between 1996 to 1999. Table 2.4 presents a summary of their pile load test data collection. Sadek (1989) studied pile load tests on the bored pile at three different sites of Dhaka city and compared them with the existing theoretical results. The variables considered are critical depth, loosening effect of soil, and groundwater level. But due to a lack of sufficient data, Sadek could not draw any correlation between theoretical results and the actual results from the pile load tests.

Khan (1997) studied the behavior of small size prestressed piles. Pile load tests on prestressed piles were carried out at four sites of Dhaka City. Pile load test results were compared with predicted pile capacities of static and dynamic methods. The measured capacities of piles driven through Dhaka Clay and resting on Dhaka Clay can be predicted quite well with the lambda-method. On the other hand, the alpha-method is only good for predicting the skin friction of Dhaka Clay. Again the measured capacities of pile-driven through Dhaka clay but resting on medium dense sand can be predicted well with a combination of lamda and alpha-methods. Khan (1997) also observed that the ultimate capacity predicted by pile driving formulae such as Engineering news formula, Janbu formula and Hiley formula overestimate the ultimate pile capacity.

Table 2.15: Summary of information on static pile load test in Bangladesh (after Ansary et al. 1999)

Location	**Soil Type (SPT)**	**Structure Type**	**Pile characteristics**	**Range of Ultimate load capacity (Ton)**
Mouluvi Bazar	0-6m: Soft clay (2) 6-15m Find sand (15-45)	1 to 4-storied office building walls	Size: 175mm x 175mm (RCC Pre-cast) Length : 6-7 m Tested Piles: 18	3 to 30
Narail, Pabna & Sylhet	0-6m: Soft cilty clay (4) 6-12m: Organic Clay/loose sand (3-8) >14m: Medium sand trace silt (15)	3 to 6-storied office buildings	Size: 175mm x 175mm (RCC Pre-cast) Length : 7 m Tested Piles: 26	6 to 15
Dhaka & Rajshahi	0-6: Soft to medium stiff clayey silt (5) 6-9m: Loose to medium dense sandy silt (9) 9-13m: medium dense to dense silty fine sand (22) >13m: Stiff clayey silt(10)	10-storied office buildings	Size: 300mm x 300mm (RCC Pre-cast) Length : 8-11 m Tested Piles: 9	83 to 116
Dhaka	0-7m: Stiff clayey silt (3) 7-13m: Dense sandy silt (10-30) >13m: Very dense silty sand (45)	8-storied Dhaka Board Office	Size: 450mm (RCC Pre-bored) Length : 14 m Tested Piles: 4	135 to 170
Chittagong	0-4m: Very loose silt (4) 4-10m: Medium dense to dense silty sand (16)	4-storied Building, Mosque Complex	Size: 500mm (RCC Pre-bored) Length : 12 m Tested Piles: 3	104 to 122

Table 2.15: (Continued) Summary of information on static pile load test in Bangladesh (after Ansary et al. 1999)

Location	**Soil Type (SPT)**	**Structure Type**	**Pile characteristics**	**Range of Ultimate load capacity (Ton)**
Dhaka	0-8m: Soft silt (4) 8-16m: Medium silt (9) 16-30m: Dense fine sand (20) >30m: Dense fine sand (35)	Wall of Intellectual Museum	Size: 500mm (RCC Pre-bored) Length : 29-35m Tested Piles: 5	**
Dhaka	0-9m: Stiff red clay (8) >9m: Medium dense to dense silty sand (20)	18-storied Hospital Building	Size: 500mm (RCC Pre-bored) Length : 18 m Tested Piles: 4	156 to 212

** Load yields only elastic settlement

2.5 Statistical Analysis

For comparison of the prediction of the pile's bearing capacity estimation approaches and evaluation of their accuracy and efficiency, the Rank Index, RI was utilized. This index is calculated as follows:
RI=R1+R2+R3+R4+R5

Where R1 is the rank of the method based on the highest value of the coefficient of determination of Qp/Qm, R2 and R3 are the methods rank based on statistical analysis using the arithmetic mean and standard deviation, R4, and R5 is methods rank based on cumulative probability analysis. The lower the RI, the more precise would be the method. Analyses of residual error, the difference between observed and predicted values, can be used to evaluate method performance by characterizing, i.e., systematic under or over-prediction. In this approach, the Coefficient of Determination (COD) or modeling efficiency is employed to check the compatibility of predictions and measured values. COD is measured by equation 2.82.

$$\text{COD} = 1 - \frac{\sum_{i=1}^{n}(\mathrm{Qpi}-\mathrm{Qmi})2}{\sum_{i=1}^{n}(\mathrm{Qmi}-\mathrm{Qma})2} \qquad (2.82)$$

Where,
Qpi and *Qmi* are the predicted and measured values, and *Qmi* are the mean of the measured values, respectively, and *n* is the number of samples.

The COD provides a dimensionless statistic summary very similar to the coefficient of determination, R^2 from linear regression. It has been similarly interpreted as the proportional reduction in variation of observed values around the model expectation to variation around the observed mean value. Note *Qm* represents the "worst-case" regression line (slope = 0) indicating a lower bound of 0 for R^2, but Loehle pointed out that no such lower bound exists for COD. In the case of 100% accuracy in method predictions, the COD will be equal to one. The arithmetic average (μ) and standard deviation (σ) of the Qp/Qm values were calculated and utilized as a second-ranking criterion. The closer the arithmetic averages to one, the lower the methods prediction's error. Also, the closer the standard deviation to zero, the lower the scatter of the predictions.

The third approach employed to evaluate the accuracy of methods is the cumulative probability measure. According to the cumulative probability approach, the ratio of the predicted value (Qp) to the measured value (Qm) has been drawn versus cumulative probability. For a series of numerals, Qp/Qm has been set ascending and indexed with 1 ton. Then for each of the relative amounts, the cumulative probability factor has been calculated as follows:

$$P(\%) = \frac{i}{n+1} X100 \qquad (2.83)$$

Where

P is the cumulative probability factor, *i*is the index of the considered case, and n is the number of total cases. To determine the convergence or deviation tendency of the output of prediction, the following criteria have been referred. The value of Qp/Qm at the cumulative probability of 50% is a measurement of the tendency to overestimate or underestimate the pile capacity. The closer to a ratio of unity, the better the agreement. To estimate the average error the following equation can be used:

$$E_{ave} = (\frac{\mathrm{Qp}}{\mathrm{Qm}})_{\%50} - 1 \qquad (2.84)$$

The slope of the line through the data points is a measurement of the dispersion or standard deviation. The flatter the line, the better the general agreement. Fig. 2.17 illustrates the cumulative probability analysis in this research.

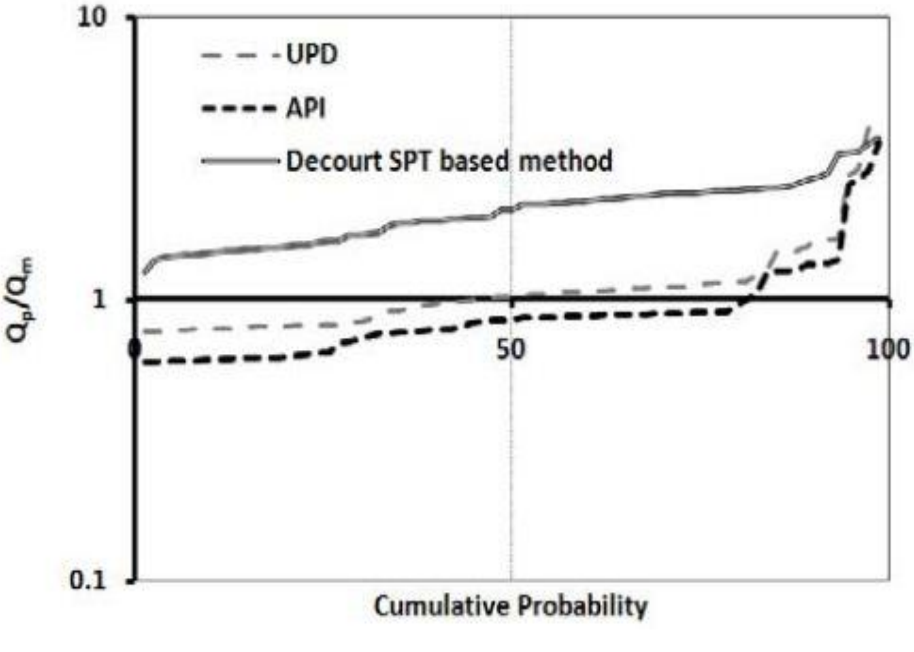

Figure 2.17: Sample Cumulative Probability

2.6 Summary

This chapter describes the methods to calculate the pile capacity. Two types of pile were discussed both precast and cast-in-situ type (Bored pile and Driled shaft). Later each type of pile was discussed based on the soil classification. To predict the capacity of each type of piles for cohesive and cohesionless soil five different methods was discussed. In addition to that, pile capacity using a static load test in various methods was discussed to ascertain the measured pile capacity. A summary of the present status of pile capacity in Bangladesh was mentioned. Finally, statistical methods for the analysis and compare the big data have been discussed. Based on the aforementioned literature the capacity of the pile will be determined in this study.

CHAPTER 3

DATA COLLECTION AND ANALYSIS

3.1 Introduction

The exact analysis of a pile theoretically is impossible because of the higher degree of indeterminacy and unpredictable behaviors. The pile may be analyzed theoretically in many ways considering the empirical relations and suggestions offered by numerous authors. This chapter deals mainly with the collection of pile data, development of soil model for the estimation of ultimate pile capacity, and discuss the methodology to establish correlations between ultimate pile capacity from static analysis and pile load tests, Figure 3.1 gives a flowchart, which explains different components of this study.

3.2 Collection of Data

Sub-soil investigation reports and corresponding pile load test results have been collected from twenty-two projects all over the country. Among these projects, from twelve projects fifteen precast piles have been tested and from ten projects fifteen cast-in-situ piles have been tested. The tests are performed between 1997 to 2018 and funded by the Public Works Department (PWD), Bangladesh, RAJUK, R&H Department, Bangladesh, and Dhaka Mass Transit Company (MRT). Almost 70% pile load tests are carried out under the direct supervision of the Department of Civil Engineering, BUET, and the rest of the pile load test carried out by Icon Engineering Services, Dhaka. The approximate geographical locations of the projects are shown in Fig. 3.2. Although most of the data are obtained from BUET, PWD, and Icon Engineering Services, Dhaka. The author felt the necessity of proper data archiving under a central national Organization such as BUET for future research purposes.

3.3 Idealization of Soil Data

Identical borehole locations and test piles have been identified. For the estimation of ultimate pile capacity in the static method, the total soil strata have been divided into some reasonable layers with specified soil properties. For the convenience of soil modeling, non-plastic silt is assumed as cohesionless soil and plastic silt assumed as cohesive soil. Field SPT value determined every one-meter interval. But for simplicity here average SPT value of every layer has to be used. The precast piles are indexed as PTP-1, 2, 3 etc. and the cast-in-situ piles are indexed as CTP-1, 2, 3 etc. The soil models together with other relevant information of the piles are presented in table 3.2 and 3.3 and Appendix A.

3.4 Analysis of Data

The principle approach used to calculate the pile's capacities to resist the compressive loads is the static or soil mechanics approach. During the past years, more research work is done to express a method based on the practical soil mechanics theory. For example, the calculation of skin friction on a pile shaft was based on a simple relationship between the effective overburden pressure, the drained angle of shearing resistance of the soil, and the coefficient of earth pressure at rest, but they realized through the results of the practical tests and researches that the coefficient of earth pressure must be modified by a factor takes into consideration the installation method of the pile.

In the same way, the calculation of the pile end bearing resistance was based on the undisturbed shearing resistance of the soil at the pile toe level, but they recognized the importance of the pile settlement at the working load and methods have been evolved to calculate this settlement, based on elastic theory and considering the transfer of load in shaft friction from the pile to the soil. A pile is subjected to a progressively increasing compressive load at a steady rate of application, the resulting load - settlement relationship plotted in Fig. 3.4. There is a straight-line relationship up to point A on the curve, this is means if the load released at any stage up to point ‘A’ the deformation or settlement of the pile head will return to its original condition. when the loading increased beyond point ‘A’ the relationship will have changed from linear to a nonlinear relationship, and there will be yielding at the pile-soil interface till reaching the maximum shaft friction 'point ‘B’. In case of load releasing at this stage, the pile head will have reached to point ‘C’. and the distance ‘OC’ will be the movement required to mobilize the maximum pile shaft resistance, usually, this distance is equal to 0.3% to 1% of the pile diameter. The pile base resistance requires more downward movement to full mobilization, point 'D', that movement is based on the pile diameter, and it is ranged between 10% to 20% of the pile diameter after point 'D' the pile will move downward without any increase in the load "failure point" in Figure 3.4.

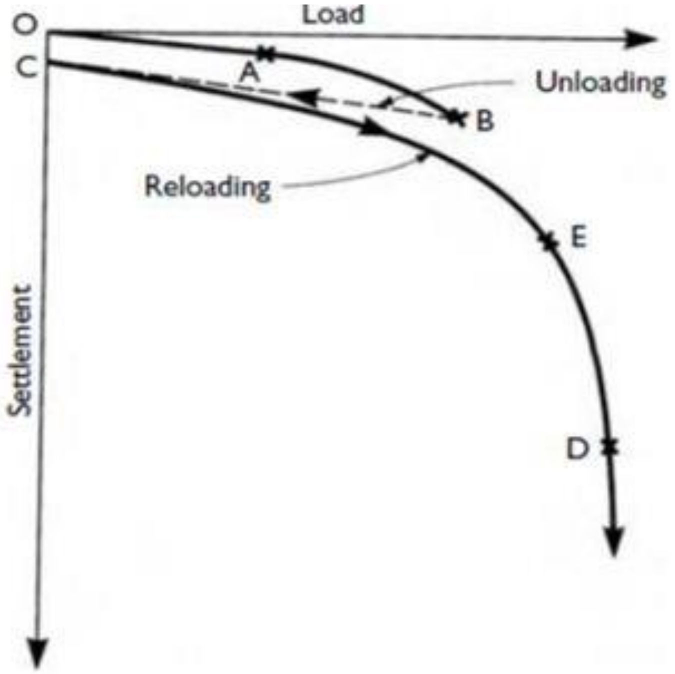

Figure 3.4: Load /settlement curve for the compressive load to failure on a pile

The exact calculation of the load-carrying capacity of a pile is a complex matter, which is based partly on theoretical concepts but mostly on empirical methods based on experience. The practice of calculating the ultimate load-carrying capacity of a pile based on the principles of soil mechanics differs greatly from the application of these principles to shallow spread foundations. The conditions, which govern the supporting capacity of the pile foundation, are quite different. No matter whether the pile is installed by driving with a hammer, by jetting, by vibration, by jacking, screwing or drilling, the soil in contact with the pile face from which the pile derives its support by skin friction and its resistance to lateral loads, is completely disturbed by the method of installation. Similarly, the soil beneath the toe of a pile is compressed or loosened to some extent which may affect significantly its end bearing resistance. Changes take place in the conditions at the pile-soil interface over periods of days, months, or years which materially affect the skin friction resistance of a pile.

3.4.1 Pile Load Capacity from Static Analysis

Static analysis methods estimate shaft and base resistances separately and differently. For shaft resistance, in cohesive as well as non-cohesive soils, considerable uncertainty and debate exist over the appropriate choice of the horizontal stress coefficient, Ks. Normally, bearing capacity theory is applied to estimate base resistance in non-cohesive soils. However, the theory involves a rather approximate ϕ-Nq relationship coupled with the difficulty of determining a reliable and representative in-situ value of the ϕ angle and the assumption of a proper shear failure surface around the pile tip. This creates doubts about relying on the bearing capacity theory in pile foundation design. Design guidelines based on static analysis often recommend using the critical depth concept. However, the critical depth is an idealization that has neither theoretical nor reliable experimental support and contradicts physical laws. For static analysis, few selected methods have been used to predict the ultimate capacity of driven and cast-in-situ bored piles. These methods are described elaborately in chapter two. For analysis purposes, the saturated and dry unit weight of soil obtained from the chart referred by Bowles (1977) is based on the SPT value of soil. For calculating the effective stress and total stress using the unit weight of every layer of soil individually and this value obtained from Table 3.1 and Figure 3.3. Different formulas have been used to compute the skin friction and end bearing of piles as per the early described methods. The Sum of the end bearing and skin friction of a single pile is the ultimate capacity of that pile. For a given soil condition different methods predict the different ultimate capacity of the same piles due to the postulation criteria of the methods. This has been discussed in chapter two and some of the points will be discussed in chapter four. Some of the calculation sheet attached in Appendix-D.

3.4.2 Pile Load Capacity from Pile Load Tests

Different criteria for obtaining ultimate pile load capacity from pile load test results are mentioned in chapter two. In this study, load settlement curves (sample curve is shown in Appendix-C) from pile load test results are used to find out the ultimate capacity of the pile by Davisson offset method, Indian Standard and BNBC Code. Davisson offset method is probably the best known and widely used in North America and other regions worldwide because it provides the lowest estimate of axial compression capacity from the actual load-settlement curve and for this versatile use of the Davisson offset method influenced the author to take as the ultimate capacity of piles for analysis. The other two methods have to use to show their validation. Generally, all three methods give almost nearly ultimate capacity, but the settlement of piles differs from one method to another. In Appendix-A load test results and corresponding settlement are presented. In chapter two we already discuss the load test methods and their theory. In some projects extrapolated load settlement curves for Test and Services piles are used for this purpose to determine the failure load. The validity of the extrapolated load settlement curve has been justified with some of the known load settlement curves. And it found the very little amount of error (+/-10%). Figure 3.5 is a photograph of a pile load test at site and Figure 3.6 represent schematic arrangement of the static load test

3.4.3 Data for Further Analysis

After obtaining ultimate capacities from both static and load test results, Table 4.2 is compiled for precast piles and Table 4.1 is compiled for cast-in-situ piles. From the twenty-two projects, fifteen results for precast pile and fifteen results for cast-in-situ piles are obtained. For all the pile load tests, settlement corresponding to the ultimate capacity of the piles obtained from load settlement curves by the early mentioned methods and are shown in Table 4.1 and 4.2.

3.5 Statistical Analysis

For comparison of the prediction of the pile’s bearing capacity estimation approaches and evaluation of their accuracy and efficiency, the Rank Index, RI was utilized. This index is calculated as follows:

RI=R1+R2+R3+R4+R5

Where

R1 is the rank of the method based on the highest value of the coefficient of determination of Qp/Qm, R2 and R3 are the methods rank based on statistical analysis using the arithmetic mean and standard deviation, R4, and R5 is methods rank based on cumulative probability analysis. The lower the RI, the more precise would be the method. Analyses of residual error, the difference between observed and predicted values, can be used to evaluate method performance by characterizing, i.e., systematic under or over-prediction. In this approach, the Coefficient of Determination (COD) or modeling efficiency is employed to check the compatibility of predictions and measured values. COD is measured by equation 3.1.

$$\text{COD} = 1 - \frac{\sum_{i=1}^{n}(\text{Qpi}-\text{Qmi})2}{\sum_{i=1}^{n}(\text{Qmi}-\text{Qma})2} \quad (3.1)$$

Where,

Qpi and *Qmi* are the predicted and measured values, and *Qmi* is the mean of the measured values, respectively, and *n* is the number of samples.

Table 3.2: Location, Size, Length and Soil Strata for Bored pile and Drilled Shaft

Index	**Project Name and year**	**Pile information**		**Soil Type**
		Length (m)	**Size (mm)**	
CTP-01	Education Board, Dhaka, 1998	14	ϕ-400	0-6.5 m stifft clay, N_{avg}=12,6.5-10.5 m medium dense sandy silt, Navg=28,10.5-26 m Dense sand, Navg=45
CTP-02	Education Board, Dhaka, 1999	14	ϕ-400	0-5 m soft clay, N_{avg}=3,5-9 m medium dense silty clay Navg=24,9-26 m Dense sand, Navg=45
CTP-03	JFICMASJID, Chittagong, 1998	12	ϕ-500	0-4 m medium stiff clay, N_{avg}=6,4-6 m loose sandy silt, Navg=9,6-9 m medium Dense silty sand, Navg=24,9-15 m very dense sand, Navg=45
CTP-04	JFICMASJID, Chittagong, 1998	12	ϕ-500	0-4.5 m soft silty clay, N_{avg}=5,4.5-8 m medium Stiff clayesilt, Navg=9,8-10 m loose silty sand, Navg=12,10-15 m medium dense sand, Navg=22
CTP-05 (Drilled Shaft-1)	Kalshi Flyover, Dhaka, 2018	30	ϕ-1000	0-6.75 m soft to stiff clay, Navg=2,6.75-18.75 m medium dense silty sand, Navg=15,18.75-26.25 m stiff clay, Navg=12,26.25-35 m Dense silty sand, Navg=48

Table 3.2: (Continued) Location, Size, Length and Soil Strata for Bored pile and Drilled Shaft

Index	**Project Name and year**	**Pile information**		**Soil Type**
		Length (m)	**Size (mm)**	
CTP-06 (Drilled Shaft-2)	Kalshi Flyover, Dhaka, 2018	34	ϕ-1000	0-8 m soft to stiff clay, Navg=6,8-18.5 m medium dense silty sand, Navg=17,18-26 m stiff clay, Navg=14,26-37m Dense silty sand, Navg=47
CTP-08 (Drilled Shaft-4)	Kumar Bridge, Keraniganj, Dhaka 2018	52.1	ϕ-1200	0-8.25m medium stiff clay, Navg=4,8.25-14.25 m loose non plastic silt. Navg=9,14.25-17.25 m soft fat clay, Navg=4,17.25-60 m medium dense silty sand, Navg=38
CTP-09 (Drilled Shaft-5)	MRT, Dhaka 2017	30	ϕ-1000	0-1.5 m soft clay Navg=1,1.5-7.6 m medium stiff clay, Navg=6,7.6-30 m, very dense sand Navg=45
CTP-10	NAM Village, Dhaka, 2000	14	ϕ-450	0-4 m medium stiff clayey silt, Navg=6,4-8.5 m mid dense sandy silt, Navg=15,8.5-16.5m medium Dense to dense sand, Navg=24
CTP-11	National Art Gallery, Dhaka, 1999	15	ϕ-510	0-7 m medium Stiff clay, Navg=6,7-11 m medium Dense silty sand, Navg=16,11-18.3 m dense sand, Navg=31
CTP-12(Drilled Shaft-6)	Postogola UP, Dhaka 2018	32.1	ϕ-1200	0-10 m soft clay, Navg=4,10-15 medium dense non plastic silt, Navg=33,15-38 m Dense sand, Navg=50
CTP-13	PG hospital, Dhaka, 1997	18.5	ϕ-500	0-6 m very soft clay, Navg=1,6-13.5 m medium Dense sandy silt, Navg=20,13.5-25 m very Dense sand Navg=42
CTP-14	PG Hospital, Dhaka, 1997	18.5	ϕ-500	0-4 m medium stiff clay, Navg=8,4-6 m stiff clay Navg=12,6-13 m medium dense sandy silt Navg=24,13-24.4 m very dense sand Navg=42
CTP-15 (Drilled Shaft-7)	Shibpur Bridge, Tanail, 2016	25	ϕ-1000	0-8.4 m loose non plastic silt Navg=4,8-26.5 m medium dense sand

Table 3.3: Location, Size, Length and Soil Strata for Pre cast pile

Index	**Project Name and year**	**Pile information**		**Soil Type**
		Length (m)	**Size (mm)**	
PTP-01	BPATC, Savar, 2000	12	300X300	0-3 m medium stiffclayesilt, N_{avg}=6,3-7.5 m stiff clayesilt, Navg=9,7.5-13 m medium Dense sand, Navg=16,13-18 m medium dense sand, Navg=24.
PTP-02	Court Building, Narail, 1998	7	175X175	0-4.5 m medium stiff clay Navg=5,4.5-9.5 m soft silty clay Navg=4,9.5-15 m Dark organic clay Navg=4
PTP-03	Uttara Apartment, Dhaka, 2014	30.5	400X400	0-3.75 m soft fat clay Navg=1,3.75-8 m loose silt Navg=3,8-15.75 m medium stiff clay Navg=4,15.75-28.5 m medium dense silty sand Navg=23,28.5-35 m dense silty sand Navg=49
PTP-04	Uttara Apartment, Dhaka, 2014	30.5	400X400	0-5.25 m soft fat clay Navg=2,5.25-9.75 loose silt Navg=9,9.75-15.75 medium stiff clay Navg=5,15.75-28.5 m medium dense silty sand Navg=17,28.5-35 m very dense silty sand.
PTP-05	Uttara Apartment, Dhaka, 2014	30.5	400X400	0-6.75 m soft fat clay Navg=3, 6.75-14.25 m loose silt Navg=8,14.25-20.25 medium stiff clay Navg=8,14.25-30 m medium dense silty sand Navg=20,30-37 m very dense silty sand Navg=42
PTP-06	Dist. Jail Building Moulovibazar, 1998	7.5	175X175	0-3 m soft clay Navg=3,3-6 m medium Dense sandy silt Navg=16,6-15.25 m medium dense to dense sand Navg=33

Table 3.3: (Continued) Location, Size, Length and Soil Strata for Pre cast pile

Index	Project Name and year	Pile information		Soil Type
		Length (m)	Size (mm)	
PTP-07	Dist. Jail Building Gopalganj, 2000	7.5	175X175	0-5.5 m soft dark silty clay Navg=3,5.5-8 m soft clayey silt Navg=3,8-10 loose fine sand Navg=7,10-14 medium dense stiff clay Navg=5
PTP-09	Divisional HQ, Sylhet, 2000	7	175X175	0-5 m soft to medium stiff clay Navg=4,5-9 m loose fine sand Navg=8,9-14 m Blackish medium stiff clay Navg=8
PTP-10	Imam Training Centre, Khulna, PTP-14, PTP-11	15.5	350X350	0-10.5 m soft dark organic clay Navg=5,10.5-30 m medium dense fine sand Navg=24.
PTP-11	Islamic Foundation, Dhaka, 2002	9.15	300X300	0-6.5m stiff clay Navg=14,6.5-10.5 m dense sand Navg=34,10.5-25 m very dense sand Navg=50
PTP-12	RDA, Bhaban, Rajshahi, 1997	10.6	300X300	0-6 m medium stiff clay, Navg=6,6-8 m loose sandy silt Navg=10,8-10 m medium dense silty fine sand Navg=12,10-12 m dense sandy silyNavg=22,12-14.6 m medium dense silty sand Navg=12.
PTP-13	Shishu Paribar, Munshiganj, 2000	12	350X350	0-5 m soft silty clay Navg=3,5-9 m medium stiff silty clay Navg=7,9-15 mid dense sand Navg=14,15-25 m medium dense silt Navg=13
PTP-14	Technical Training Centre, Patuakhali, 2002	7.5	300X300	0-6 m soft clay Navg=3,6-8 m very soft clay Navg=1,8-18 m medium dense sand Navg=18.
PTP-15	Technical Training Centre, Patuakhali, 2002	7.5	300X300	0-6 m medium stiff clay, Navg=4,6-8.5 m medium dense sandy silt Navg=12,8.5-18 m medium dense to dense sand Navg=22.

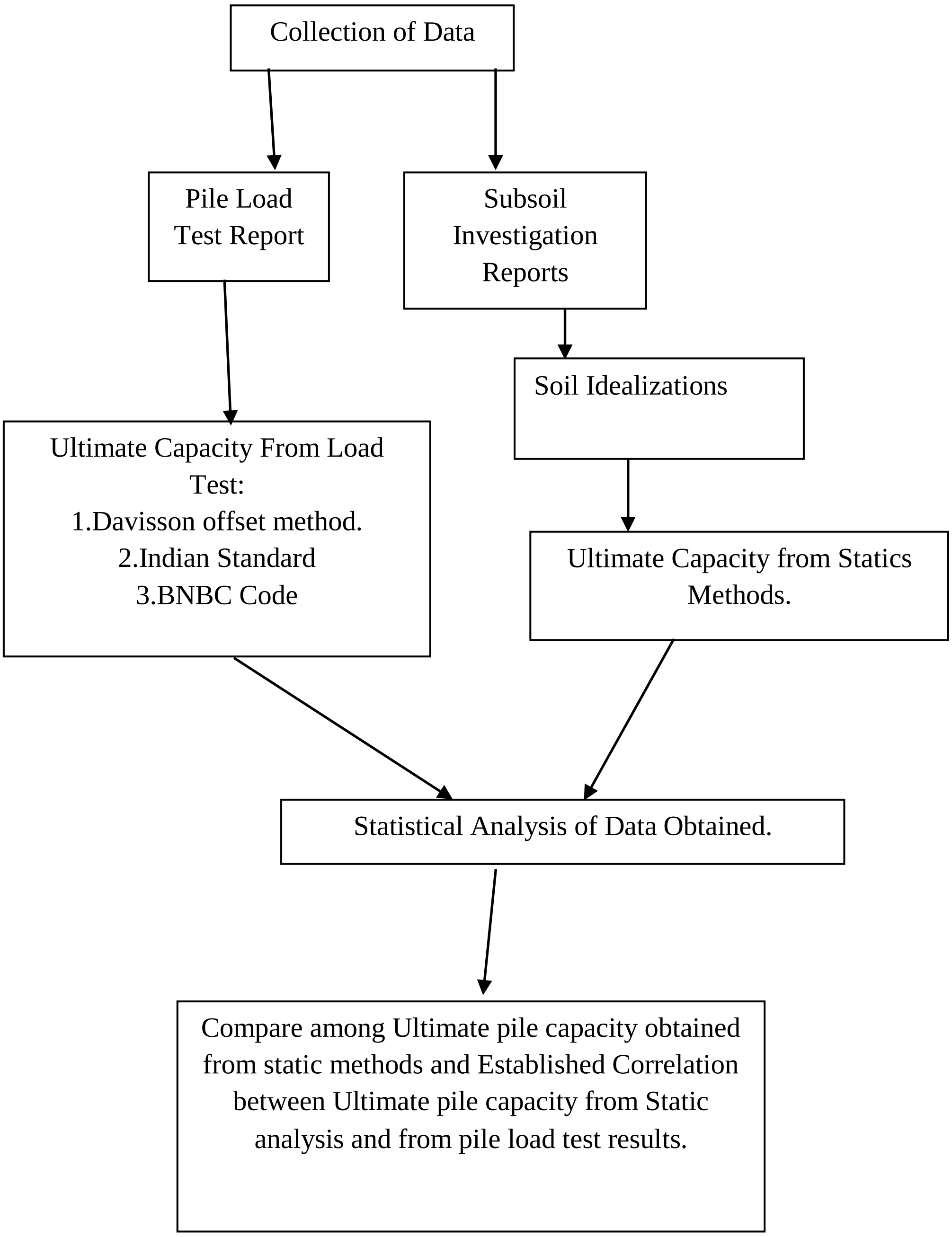

Figure 3.1: Flowchart of the Study

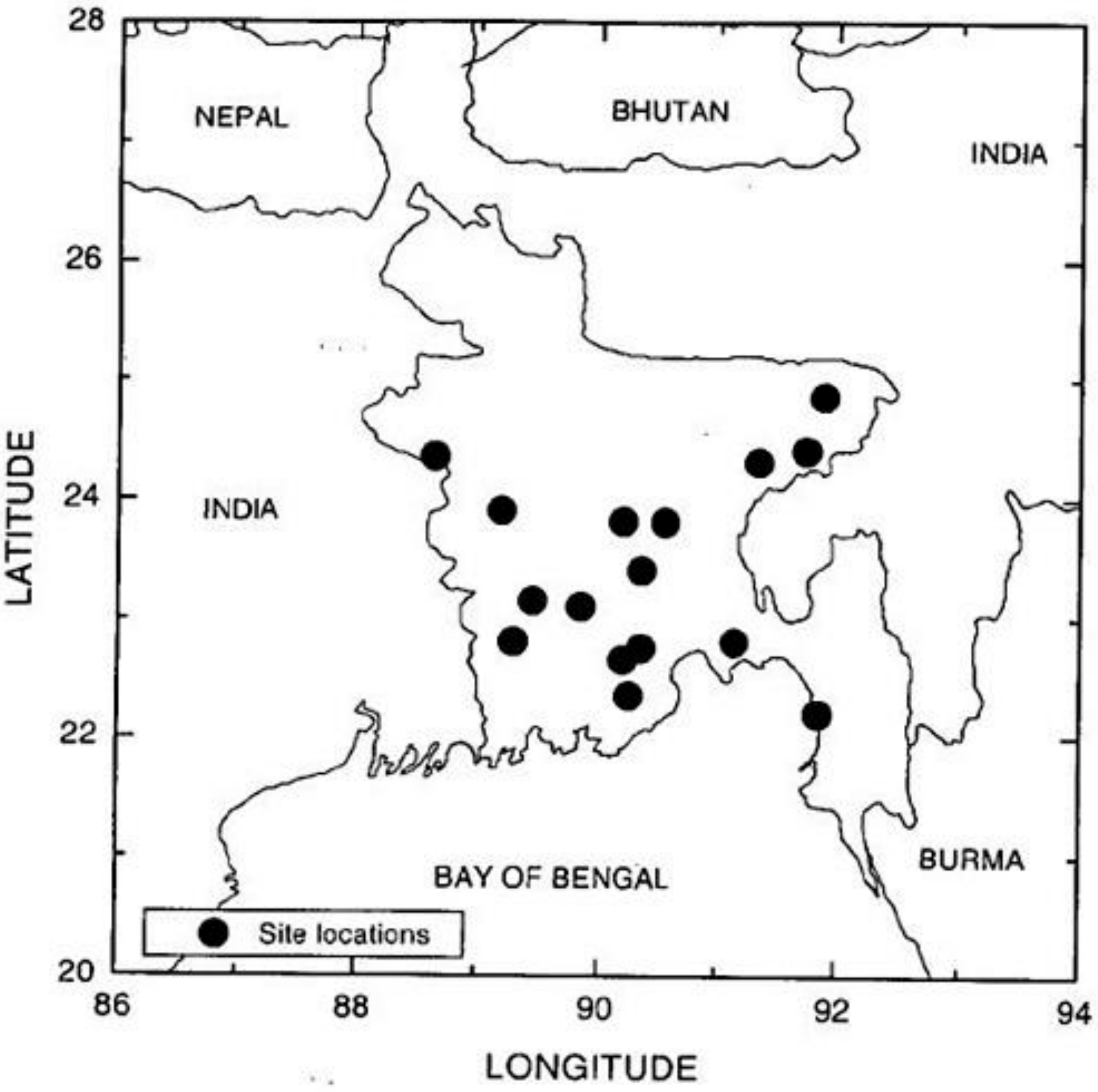

Figure 3.2: Geographical locations of pile load tests and soil borehole

Table 3.1: Empirical Values of Unconfined Compressive Strength, q_u and Consistency of Cohesive Soils Based on Uncorrected N-Value (after Bowles, 1977)

Consistency	**Very Soft**	**Soft**	**Medium**	**Stiff**	**Very Stiff**	**Hard**
q_u (ksf)	0-0.5	05-1.0	1.0-2.0	2.0-4.0	4.0-8.0	8.0+
SPT Value (N_{60})	0-2	2-4	4-8	8-16	16-32	32+
γ(saturated) lb/ft3	100-120	100-120	110-130	120-140	120-140	120-140
Undrained Cohesion, c_u (ksf)	0-0.25	0.25-0.5	0.5-1.0	1.0-2.0	2.0-4.0	4.0+

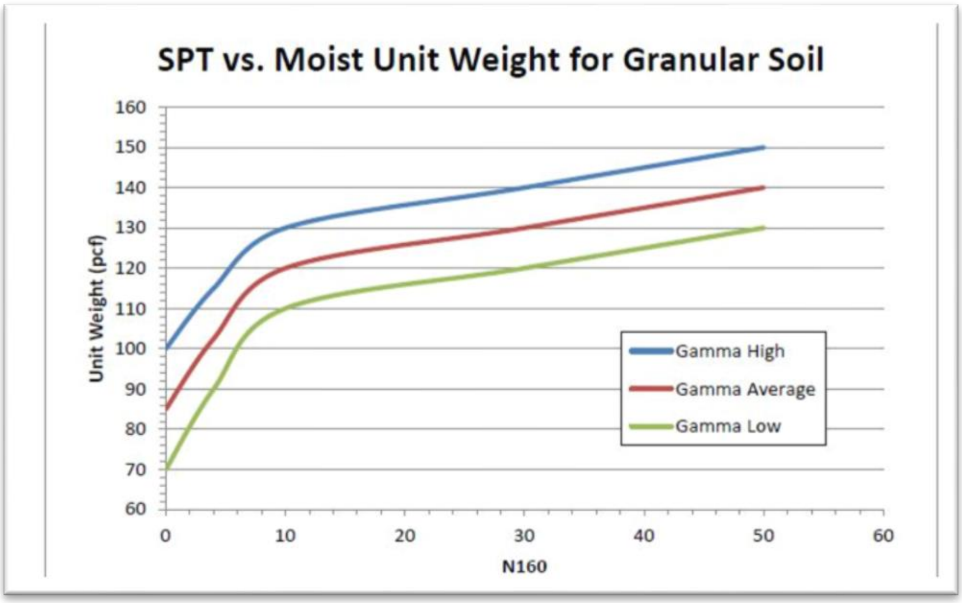

Figure 3.3: Correlation of SPT N_{160} with unit weight for cohesionless soil (after Bowles, 1977)

Figure 3.5: Photograph of a pile static load test

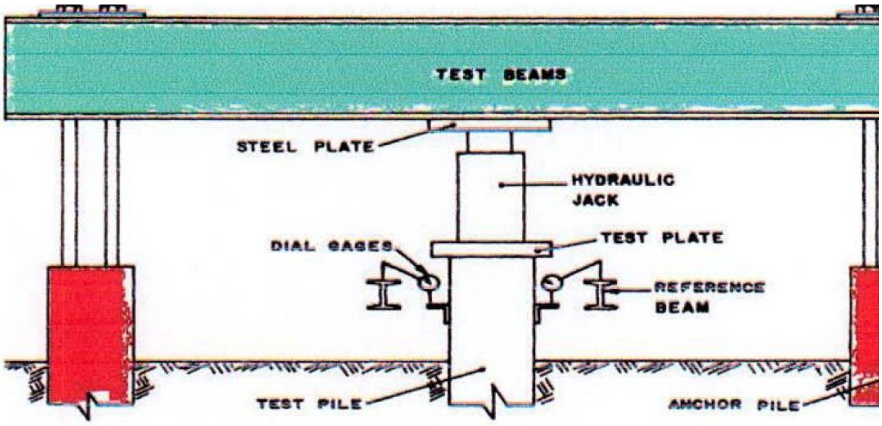

Figure 3.6: Schematic arrangement of the static load test.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 General

In this study, the main focus is to determine the compressive load capacity of piles using different existing methods of pile capacity estimation and compare with static load test. In this chapter, pile capacity calculation was done based on the Meyerhof (1976), NAVFAC DM 7.2 (1984), AASHTO (1986), O'Neill & Reese (1988), and Decourt (1995) methods for cast-in-situ bored piles and Drilled Shafts. And for calculation of capacity of driven piles based on Meyerhof (1976), API RP 2A (1993), Tomlinson (1994), Norwegian Pile Guideline (2005) and Indian Standard (2010) methods. For the selected fifteen precast driven pile eight cast in situ bored pile and seven Drilled shaft capacities was predicted and later compared with static load test. The later part of the study uses compared data for capacity and correlate with prediction and measured capacity for precast and cast in situ piles and Drilled shafts. Based on these compared (prediction and measured capacity) data selection of best theoretical methods for predicting the capacity of precast driven piles and cast in situ bored piles and Drilled shafts.

4.2 Determination of Pile Capacity by Theoretical Methods

4.2.1 Precast Driven Pile

In this study, fifteen numbers of precast driven piles have been selected from different projects all over Bangladesh in the time 1998 to 2018.By using the following methods their ultimate capacity has been measured. It has been discussed elaborately on the following methods in chapter two for both cohesive and cohesionless soil. The methods are:

- Meyerhof (1976)
- API RP 2A (1993)
- Tomlinson (1994)
- Norwegian Pile Guideline (2005)
- Indian Standard (2010)

Summary of the predicted ultimate capacity of fifteen precast drivren piles shown in table 4.2 the detailed calculation of these fifteen driven piles is included in the Appendix D.

4.2.2 Cast in Situ Bored Pile and Drilled Shaft

In this study, eight numbers of the cast in situ bored piles and seven numbers of drilled shaft have been selected from different projects all over Bangladesh in the time 1998 to 2018. By using the following methods their ultimate capacity has been measured. It has been discussed elaborately about the following methods in chapter two for both cohesive and cohesionless soil. The methods are:

- Meyerhof (1976)
- NAVFAC DM 7.2 (1984)
- AASHTO (1986)
- O'neill & Reese (1988)
- Decourt (1995)

Summary of the predicted ultimate capacity of eight casts in situ bored piles and seven drilled shafts shown in table 4.1. The detailed calculation of these fifteen bored piles is included in the Appendix D.

4.3 Determination of Pile Capacity from Load Test by Different Standards and Methods

The load was applied to piles in stages up to 200% to 300% of the design load. Pile settlement in mm against applied load in tons has been plotted for piles (Appendix-C). To measure the ultimate vertical load-carrying capacity of the test piles the following methods are followed. These are:

(i) Davisson's offset limit method- In Davisson's (1973) method, the failure load is defined as the load corresponding to the movement which exceeds the elastic compression of the pile, when considered as a free column, by a value of O.I 5in. plus a factor depending upon the diameter of the pile. This critical movement can be expressed as follows:

$$S_r = S + (0.15x25.4 + 0.008D) \qquad (4.1)$$

Where, S_r is the movement of the pile head (in mm), D is the pile diameter or width (inmm), and S is the elastic deformation of the total pile length (in mm). In this study to calculate the elastic compression of the pile the Modulus of Elasticity of the concrete (Ec) has been taken as

$$E_c = 57000\sqrt{f'c} \qquad (4.2)$$

Where, Ultimate compressive strength of the concrete, fc' = 4000 psi

(ii) Indian Standards (IS) code (1979) recommends that the ultimate capacity of pile is smaller of the following two-

a) Load corresponding to a settlement equal to 10% of the pile diameter
b) Load corresponding to a settlement of 12 mm

The same code states that the allowable pile capacity is smaller of the following:

a) Two-thirds of the final load at which total settlement is 12 mm.
b) Half of the final load at which total settlement is equal to 10% of the pile diameter.

(iv) The Bangladesh National Building Code (1993) recommends that the allowable load capacity of the pile shall not be more than one half of that test load which produces a permanent not settlement (i.e. gross settlement less rebound) of not more than 0.00028 mm/kg of test load nor 20mm.

The result of load tests on piles under this study is presented in Table 4.1 and Table 4.2. And compare of each pile with predicted theoretical capacities presented in Table 4.1, Table 4.2 and Appendix C.

Table 4.1: Summary of predicted and measured capacity of CTP (Bored pile and Drilled Shaft)

Index	Project Name and year	Pile information		Ultimate capacity (kips) From Load Test			Settlement	Ultimate capacity (kips) from static Analysis				
		Length (m)	Size (mm)	Davisson	Indian Standards	BNBC Code	mm	Meyerhof(1976)	NAVFAC(1984)	AASHTO (1986)	O'Neill & Reese (1988)	Decourt (1995)
CTP-01	Education Bored, Dhaka, 1998	14	ϕ-400	337.21	348.23	363	14	315.22	272.87	358.19	331.48	629.54
CTP-02	Education Bored, Dhaka, 1999	14	ϕ-400	233.62	244.64	301	11	300.47	240.46	335.73	311.21	611.54
CTP-03	JFIC Masjid, Chittagong, 1998	12	ϕ-500	268.88	246.84	253.46	9	394	367.51	414.04	352.24	820.06
CTP-04	JFIC Masjid, Chittagong, 1998	12	ϕ-500	220.21	233.62	250	7.5	206.99	207.96	266.8	211.85	446.86
CTP-05 (Drilled Shaft-1)	Kalshi Flyover, Dhaka, 2018	30	ϕ-1000	4231	2204	2821.12	41.6	885.82	1887.36	1734.71	2041.81	3564.94
CTP-06 (Drilled Shaft-2)	Kalshi Flyover, Dhaka, 2018	34	ϕ-1000	3085.6	1873.4	2380.32	31.8	1265.82	2701.57	1999.47	2583.91	3944.54

Table 4.1: (Continued) Summary of predicted and measured capacity of CTP (Bored pile and Drilled Shaft)

CTP-07 (Drilled Shaft-3)	Kumar Bridge, Keraniganj, Dhaka 2018	45.1	ϕ-1200	3923.12	2071.76	2777.0	42	2793.77	4116.57	3813.76	4185.23	5604
CTP-08 (Drilled Shaft-4)	Kumar Bridge, Keraniganj, Dhaka 2018	52.1	ϕ-1200	2644.8	1653	2027.6	36	2922.86	465.41	2948.87	5821.31	5826.94
CTP-09 (Drilled Shaft-5)	MRT, Dhaka 2017	30	ϕ-1000	3834.96	2204	2755	37	1925.59	2473.66	1674.75	2265.7	3894.41
CTP-10	NAM Village, Dhaka, 2000	14	ϕ-450	286.52	242.44	319.59	17	253.74	260.14	384.07	305.7	520.69
CTP-11	National Art Gallery, Dhaka, 1999	15	ϕ-510	282.11	270.23	311.41	13.5	315.66	343.95	418.75	351.65	662
CTP-12 (Drilled Shaft-6)	Postogola UP, Dhaka2018	32.1	ϕ-1200	925.68	881.6	969.76	19	2867.84	4585.43	2540.64	3246.37	5730.8
CTP-13	PG Hospital, Dhaka, 1997	18.5	ϕ-500	343.82	319.58	396.72	12	478.56	433.78	491.94	531.05	996.13
CTP-14	PG Hospital, Dhaka, 1997	18.5	ϕ-500	467.24	484.88	551	16	479.92	480.69	616.65	589.13	978.27
CTP-15 (Drilled Shaft-7)	Shibpur Bridge, Tanail, 2016	25	ϕ-1000	716.3	672.22	749.36	18	1606.98	2306.82	1606.14	1509.76	3443.33

Table: 4.2 Summary of the predicted and measured capacity of PTP

Index	Project Name and year	Pile information		Ultimate capacity (kips) From Load Test			Settlement	Ultimate capacity (kips) from static Analysis				
		Length (m)	Size (mm)	Davisson	Indian Standards	BNBC Code	mm	Meyerhof (1976)	API (1993)	Tomlinson (1994)	NPG (2005)	IS (2010)
PTP-01	BPATC, SAVAR, 2000	12	300x300	374.68	376	395	11	164.58	189.11	206.12	156.19	180.27
PTP-02	Court Building, Narail, 1998	7	175x175	24.9	18	20	51	17.25	23.94	30.98	26.9	30.67
PTP-03	Uttara Apartment, Dhaka,2014	30.5	400x400	1002.82	440.8	573.04	40	627.8	906.11	654.63	662.59	667.27
PTP-04	Uttara Apartment, Dhaka, 2014	30.5	400x400	859.56	462.84	617.12	37	646.3	897.93	702.11	661.27	603.81
PTP-05	Uttara Apartment, Dhaka, 2014	30.5	400x400	727.32	484	650	40	647.87	859.27	750.74	650.3	549.78
PTP-06	Dist. Jail Bling. Koulibiaca, 1998	7.5	175x175	51.35	41.87	51.35	20	78.46	31.58	69.58	45.68	46.66
PTP-07	Dist. Jail Bldng. Gopalganj, 2000	7.5	175x175	20.94	17.25	21	19	17.29	22.94	25.58	27.06	23.79
PTP-08	District Reg. & Sub reg. Off. Jhalokathi, 2001	9.2	400x400	55.1	60	68	18.5	71.53	84.3	101.94	91.13	105.29
PTP-09	Divisional HQ, Sylhet, 2000	7	175x175	33.06	25	28	43	23.3	30.79	29.18	33.56	33.46

Table: 4.2 (Continued) Summary of the predicted and measured capacity of PTP

Index	Project Name and year	Pile information		Ultimate capacity (kips) From Load Test			Settlement	Ultimate capacity (kips) from static Analysis				
		Length (M)	Size (mm)	Davisson	Indian Standards	BNBC Code	mm	Meyerhof (1976	API (1993)	Tomlinson (1994)	NPG (2005)	IS (2010)
PTP-10	Imam Training Centre, Khulna, PTP-14, PTP-11	15.5	350x350	240.23	220.4	253.46	14	234.54	205.61	275.54	184.91	192.88
PTP-11	Islamic Foundation, Dhaka, 2002	9.15	300x300	308.56	275.5	315.31	18	231.82	216.11	276.32	206.48	216.27
PTP-12	RDA, Bhaban, Rajshahi, 1997	10.6	300x300	242.44	245.01	264.48	11.5	149.31	136.65	159.69	102.22	178.5
PTP-13	Shishu Paribar, Munshiganj, 2000	12	350x350	143.26	125	138	38	172.09	146.53	169.82	137.73	174.33
PTP-14	Technical Training Centre, Patuakhali, 2002	7.5	300x300	114.61	121.22	128	6	114	58.51	71.11	63.95	79.35
PTP-15	Technical Training Centre, Patuakhali, 2002	7.5	300x300	121.22	133.14	140.24	6.9	140.03	74.71	127.3	78.79	125.85

4.3.1 Analysis of Load–Settlement Curves

As the load tests used in this study out of thirty piles (fifteen of the precast and fifteen of the cast in situ) ten numbers of piles (seven of bored piles) were not carried to failure which would have facilitated determination of the precise value of the ultimate load capacity of the piles, a method of extrapolation of load settlement curves has been used to estimate as nearly as possible; the failure load from load settlement curves. The assumptions used in the extrapolation are;

Load settlement curves follow the trend of a parabola after an initial straight portion. The piles under this study predominantly end bearing with a low to moderate contribution of frictional resistance. Before applying the extrapolation method, a clear idea of the nature of the load settlement curves of different types of piles is needed. The following discussions are provided to fulfill the above objective. As cited by Peck et al. (1980) the results of typical load tests are shown in Figure 4.1 in which the total load is plotted as a function of the settlement of the pile head. Curve 'a' represents a pile that slipped or plunged suddenly when the load reached a definite value termed as the ultimate pile load or pile capacity. The nature of the curve indicates that the pile under test is a friction pile with negligible end bearing. Curve ‘b’ does not show a well-defined break as in curve "a" and continues to penetrate the ground showing a predominant contribution of the end bearing. Curve 'c' on the other hand takes a parabolic shape after an initial straight portion showing both the contribution of friction and end bearing.

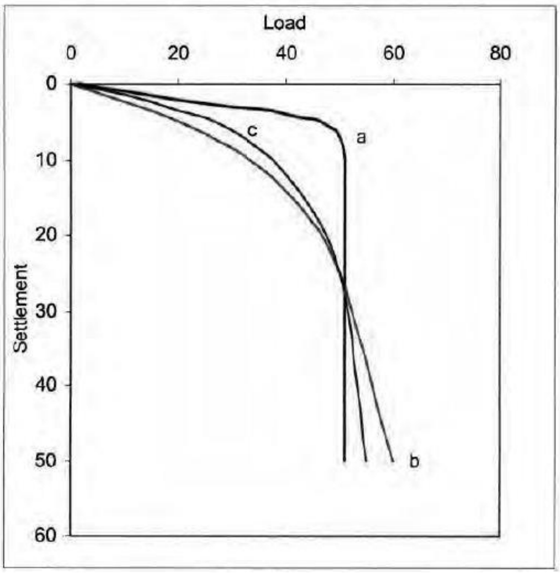

Figure 4.1: Typical results of load test on (a) friction pile, (b) end bearing pile and (c) pile deriving support from both end bearing and friction

Analysis of the bearing capacity of piles by static methods under this study in Chapter two shows that the piles are predominantly base resistance but has a good contribution from the friction since the embedded depth of the piles in layered soil. Thus, the piles of which the load test results are being discussed fall in the group 'c' as discussed in this section earlier.

4.3.2 Procedure of Extrapolation

- Step-1: With a careful examination of the load settlement curve, the parabolic portion of the curve is selected.
- Step-2: The general equation of the curve is taken as $y = ax^2 + bx + c$, where a, b, c are constants. Taking any three points on this curve the constants can be detel1nined. Therefore, the equation of the parabola is established.
- Step-3: With this equation the curve can be extrapolated up to the next load increment in the load settlement plot.
- Step-4: Using any three points on the extrapolated curve, another equation of parabola can be established. With this equation, the curve can be extrapolated up to the next load increment in the load settlement curve. Following the above procedure, the load settlement curves can be plotted up to a distinct break and using this curve estimates can be made of ultimate pile capacity using recommendations and standards as for full-scale load tests carried to failure some of the extrapolated load settlement curve shown in Figure 4.2 and Figure 4.3.

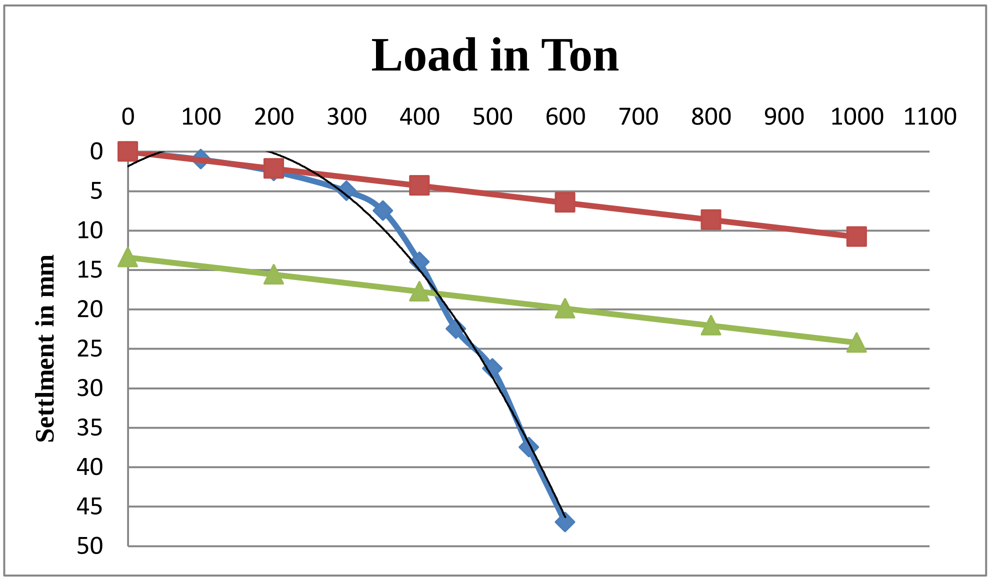

Figure 4.2: Load settlement curve of CTP-12 (Postogola under Pass)

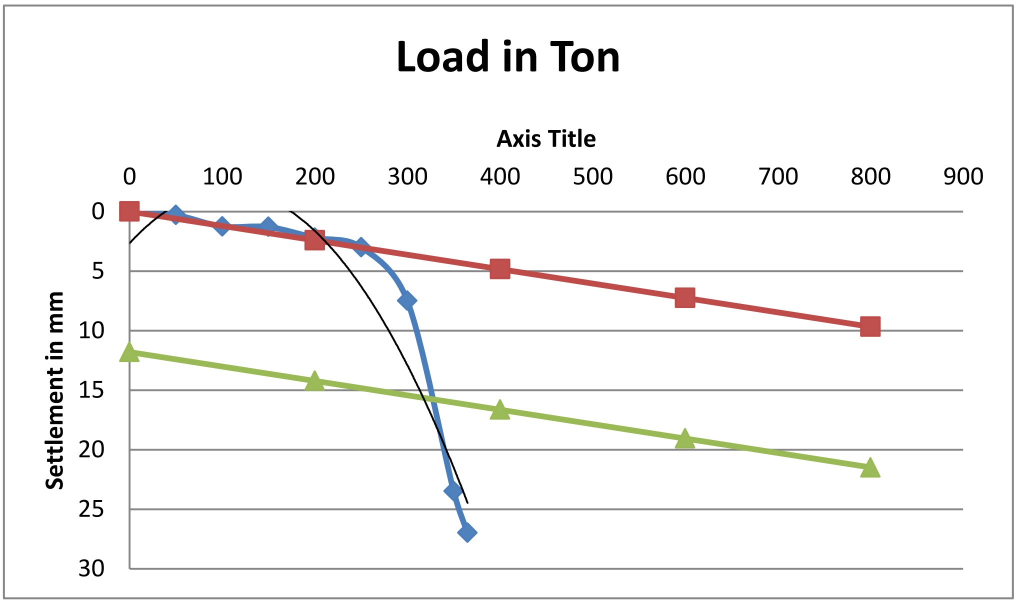

Figure 4.3: Load Settlement curve of CTP-15 (Shibpur Bridge, Tangail)

4.4 Statistical Analysis

For comparison of the prediction of the pile's bearing capacity estimation approaches and evaluation of their accuracy and efficiency, the Rank Index, RI was utilized. This index is calculated as follows:

RI=R1+R2+R3+R4+R5 (4.3)

Where R1 is the rank of the method based on the highest value of the coefficient of determination of Qp/Qm, R2 and R3 are the methods rank based on statistical analysis using the arithmetic mean and standard deviation, R4, and R5 is methods rank based on cumulative probability analysis. The lower the RI, the more precise would be the method. Table 4.3, Table 4.4 and Table 4.5 illustrates the variation of the predicted capacities with measured capacities for different methods. According to this figure, the solid line in each diagram reveals perfect agreement between predicted and measured pile capacity passing the origin with a slope equal to unity. Analyses of residual error, the difference between observed and predicted values, can be used to evaluate method performance by characterizing, i.e., systematic under or over-prediction. In this approach, the Coefficient of Determination (COD) or modeling efficiency is employed to check the compatibility of predictions and measured values. COD is measured by equation 4.4.

$$\text{COD}=1-\frac{\sum_{i=1}^{n}(Qpi-Qmi)2}{\sum_{i=1}^{n}(Qmi-Qma)2} \tag{4.4}$$

Where,

Qpi and *Qmi* are the predicted and measured values, and Qma is the mean of the measured values, respectively, and n is the number of samples. The COD provides a dimensionless statistic summary very similar to the coefficient of determination, R^2 from linear regression. It has been similarly interpreted as the proportional reduction in variation of observed values around the model expectation to variation around the observed mean value. Note *Qm* represents the "worst-case" regression line (slope = 0) indicating a lower bound of 0 for R^2, but Loehle pointed out that no such lower bound exists for COD. In the case of 100% accuracy in method predictions, the COD will be equal to one. The arithmetic average (μ) and standard deviation (σ) of the Qp/Qm values were calculated and utilized as a second-ranking criterion. The closer the arithmetic averages to one, the lower the methods prediction's error. Also, the closer the standard deviation to zero, the lower the scatter of the predictions.

The third approach employed to evaluate the accuracy of methods is the cumulative probability measure. According to the cumulative probability approach, the ratio of the predicted value (Qp) to the measured value (Qm) has been drawn versus cumulative probability. For a series of numerals, Qp/Qm has been set ascending and indexed with 1 ton. Then for each of the relative amounts, the cumulative probability factor has been calculated as follows:

$$P(\%)=\frac{i}{n+1}X100 \tag{4.5}$$

Where P is the cumulative probability factor, iis the index of the considered case, and n is the number of total cases. To determine the convergence or deviation tendency of the output of prediction, the following criteria have been referred: The value of Qp/Qm at the cumulative probability of 50% is a measurement of the tendency to overestimate or underestimate the pile capacity. The closer to a ratio of unity, the better the agreement. To estimate the average error the following equation can be used:

$$E_{ave}=(\frac{\text{Qp}}{\text{Qm}})_{\%50}\text{ - 1}$$

The slope of the line through the data points is a measurement of the dispersion or standard deviation. The flatter the line, the better the general agreement. Figure 4.4, Figure 4.5 and Figure 4.6 illustrates the cumulative probability analysis in this research.

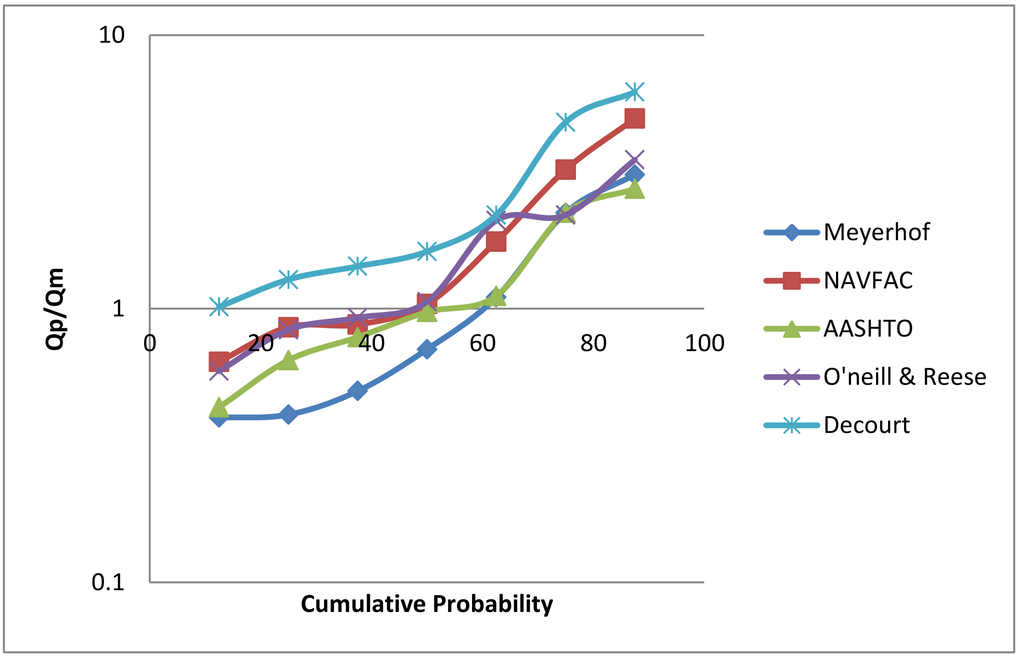

Figure 4.4: Cumulative probability for Drilled Shaft for different methods

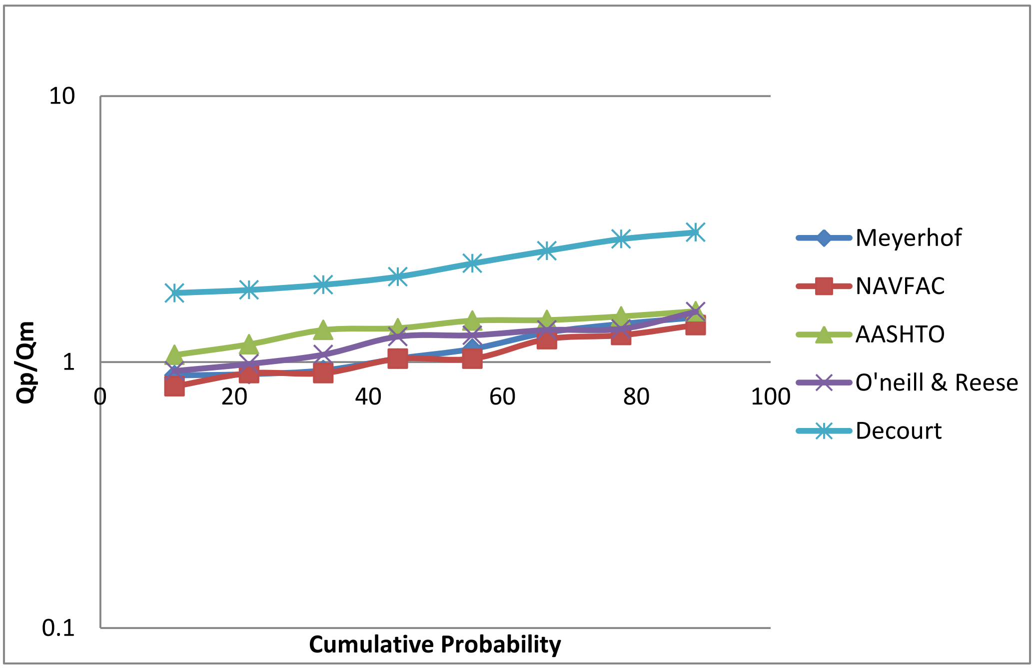

Figure 4.5: Cumulative Probability graph for bored pile for different methods

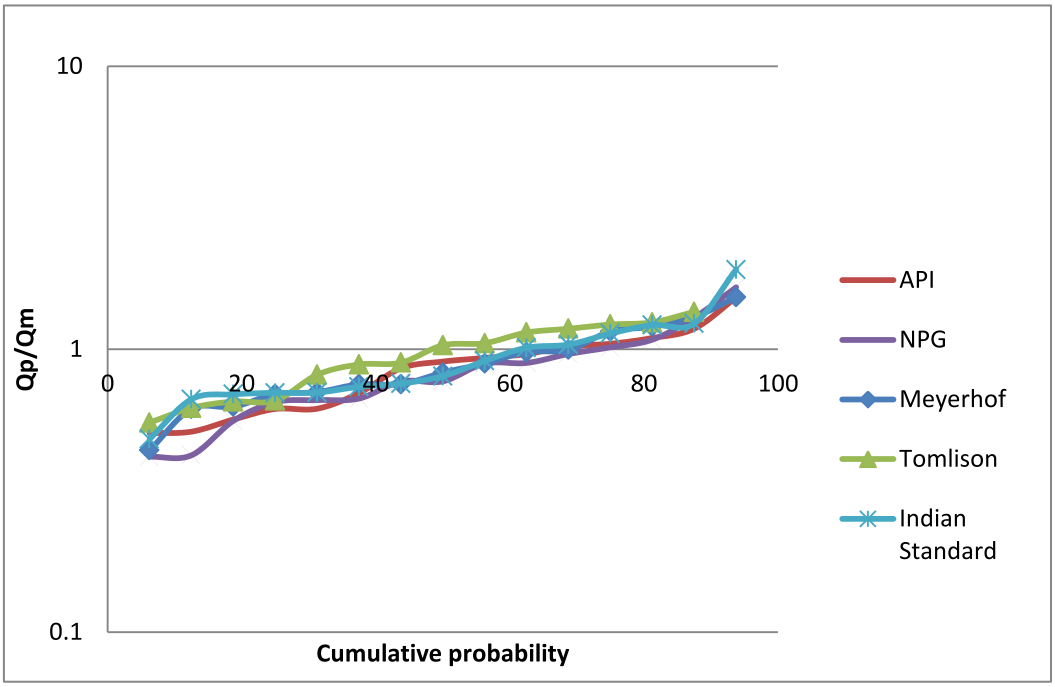

Figure 4.6: Cumulative Probability graph for driven pile for different methods

4.5 Establishment of Correlation

The establishment of a correlation between predicted capacity and measured capacity of PTP and CTP for different theoretical and semi-empirical methods is the goal of this study. For this purpose, Microsoft excel has been used here. A regression equation has been used to compare capacities. The R^2 values give this study the compatibility of these regression equations. The value of R^2 near to unity gives the most compatible equation. But it does not represent the accuracy of the statics equation to predict the capacity for different methods. This accuracy of the statics equation has been determined by a statistical analysis which has been discussed earlier.

Table 4.3: Statistical and probability analysis of PTP

Methods	**COD**	**R1**	**μ**	**R2**	**σ**	**R3**	**P_{50}**	**R4**	**P_{90}**	**R5**	**RI**	**Average Error**
Meyerhof (1976)	0.824	4	0.897	3	0.295	3	0.826	3	1.35	2	15	-17.40%
API (993)	0.937	1	0.869	4	0.288	2	0.904	2	1.22	1	10	-9.60%
Tomlinson (1994)	0.868	2	1.011	1	0.118	1	1.032	1	1.5	5	10	3.20%
Norwegian Pile Guideline (2005)	0.827	3	0.847	5	0.329	4	0.769	5	1.45	4	21	-23.10%
Indian Standard (2010)	0.814	5	0.933	2	0.349	5	0.803	4	1.36	3	19	-20%

Table 4.4: Statistical and probability analysis of CTP (Bored pile)

Methods	**COD**	**R1**	**μ**	**R2**	**σ**	**R3**	**P_{50}**	**R4**	**P_{90}**	**R5**	**RI**	**Average Error**
Meyerhof (1976)	0.824	4	0.897	3	0.295	3	0.826	3	1.35	2	15	-17.40%
API (993)	0.937	1	0.869	4	0.288	2	0.904	2	1.22	1	10	-9.60%
Tomlinson (1994)	0.868	2	1.011	1	0.118	1	1.032	1	1.5	5	10	3.20%
Norwegian Pile Guideline (2005)	0.827	3	0.847	5	0.329	4	0.769	5	1.45	4	21	-23.10%
Indian Standard (2010)	0.814	5	0.933	2	0.349	5	0.803	4	1.36	3	19	-20%

Table 4.5: Statistical and probability analysis of CTP (Drilled Shaft)

Methods	**COD**	**R1**	**μ**	**R2**	**σ**	**R3**	**P_{50}**	**R4**	**P_{90}**	**R5**	**RI**	**Average Error**
Meyerhof (1976)	0.242	2	1.21	1	1.05	2	0.71	1	3.12	2	8	-29.00%
NAVFAC (1984)	-0.145	4	1.91	4	1.6	3	1.04	3	4.98	4	18	4.00%
AASHTO (1986)	0.5	1	1.27	2	0.87	1	0.97	2	2.78	1	7	-3.00%
O'neill & Reese (1988)	0.02	3	1.61	3	1.05	2	1.06	4	3.55	3	15	6.00%
Decourt (1995)	-1.38	5	2.65	5	2.02	4	1.62	5	6.2	5	24	62.00%

4.5.1 Precast Driven Pile

4.5.1.1 Meyerhof (1976) Method

Comparing the predicted capacity from Meyerhof (1976) method with measured capacity from load test for fifteen numbers of precast driven piles gives a regression equation, $Q_p = 0.685Q_m + 24.98$ with $R^2 = 0.94$. High value of R^2 indicates a better correlation between

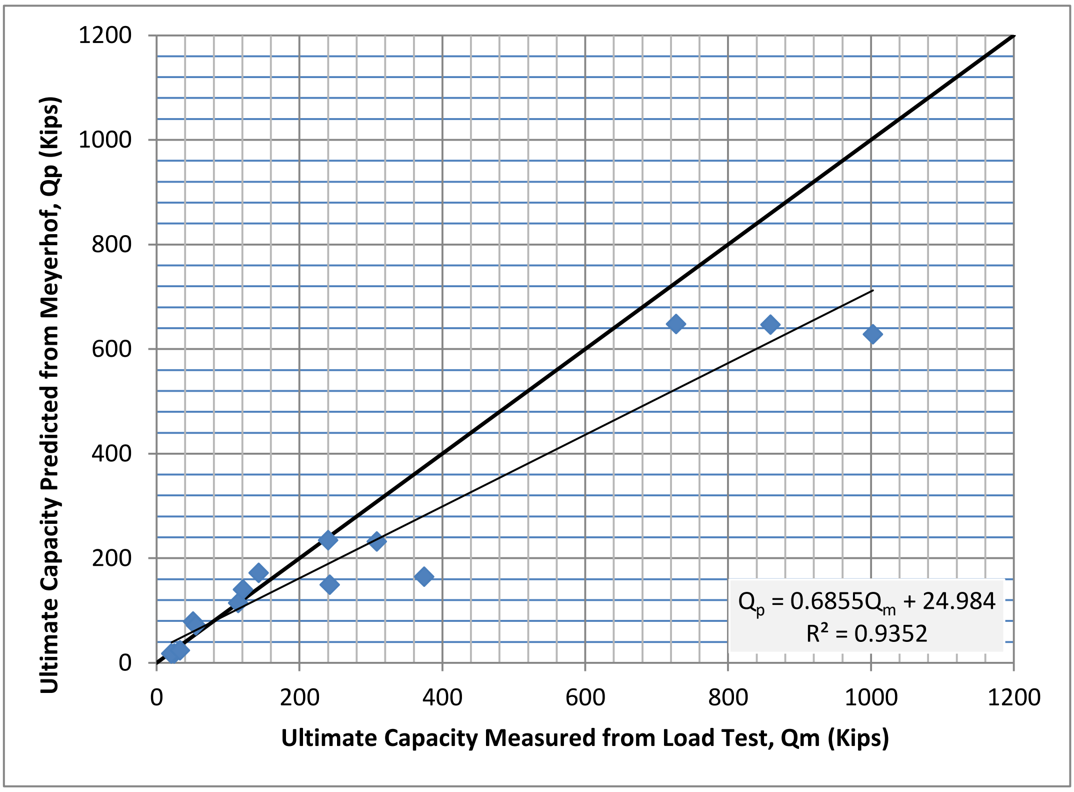

Figure 4.7: Correlation between Q_p and Q_m for Meyerhof method

Predicted and measured capacity Figure 4.7 represent the co-relation between predicted and measured capacity for Meyerhof method. From statistical analysis and cumulative probability (shown in Table 4.3), it has been observed that a moderate RI(RI=15) ranked this method third amongst all other methods in precast driven piles. Also, this method predicts to underestimate the ultimate capacity by 17.40%. Figure 4.6 justifies this error. COD value of 0.824 indicates the good compatibility of the predicted theory. Limiting value of effective stress after 10D for loose sand, 15D for mid dense sand, and 20D for dense to very dense sand recommended by Das control the skin resistance as well as end-bearing capacity. It is one of the major causes of under predict the capacity.

4.5.1.2 API RP 2A (1993) Method

Comparing the predicted capacity from API (1993) method with measured capacity from load test for fifteen numbers of precast driven piles gives a regression equation, $Q_p = 1.01Q_m - 31.95$ with $R^2=0.95$. High value of R^2 indicates a better correlation between

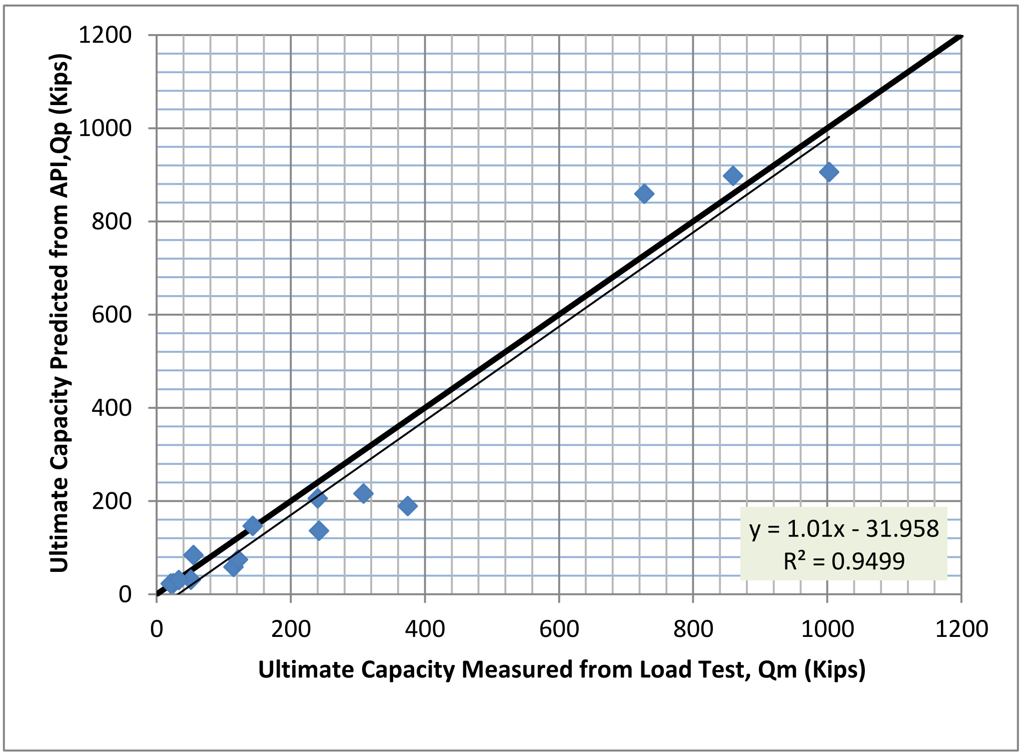

Figure 4.8: Correlation between Q_p and Q_m for API method

Predicted and measured capacity Figure 4.8 represent the co-relation between predicted and measured capacity for API method. From statistical analysis and cumulative probability (shown in Table 4.3), it has been observed that the lowest RI (RI=10) ranked this method second amongst all other methods in precast driven piles. Also, this method predicts to underestimate the ultimate capacity by 9.60%. Figure 4.6 justifies this error. COD value of 0.937 indicates very good compatibility of the predicted theory. No such limit for the determination of effective stress. For this large value of end bearing obtained for large displacement piles. But API recommended limiting unit skin friction and limiting unit end bearing keep the ultimate capacity justified with measured capacity.

4.5.1.3 Tomlinson (1994) Method

Comparing the predicted capacity from Tomlinson (1994) method with measured capacity from load test for fifteen numbers of precast driven piles gives a regression equation, Q_p=0.752Q_m+26.67 with R^2=0.92. High value of R^2 indicates a better correlation between

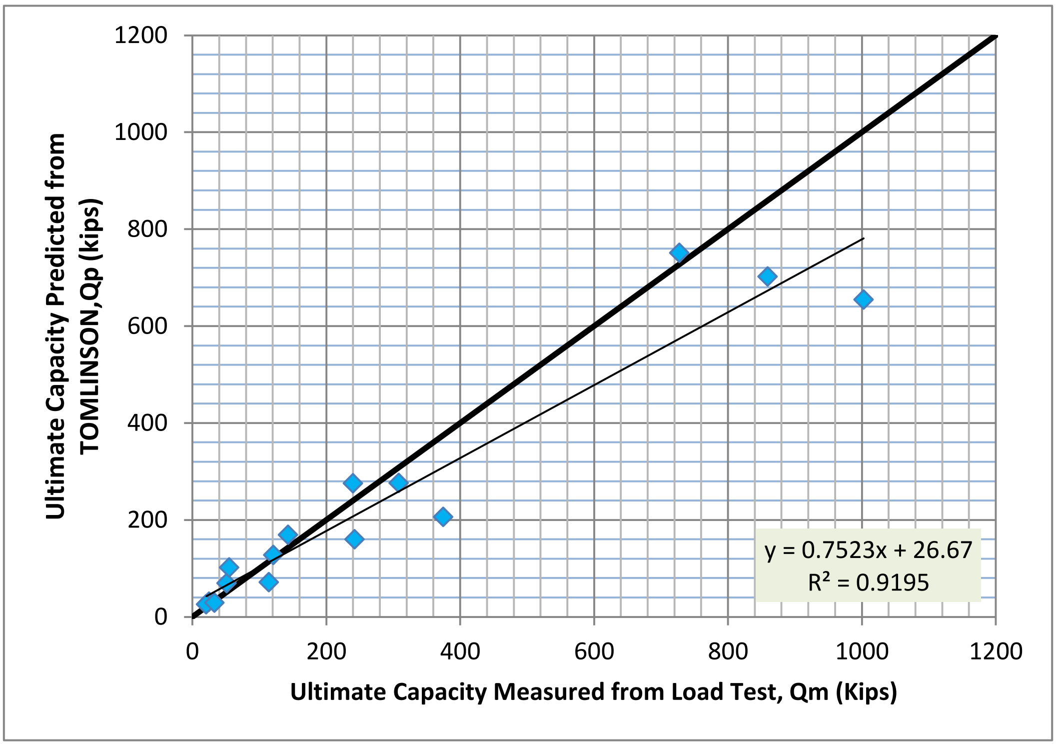

Figure 4.9: Correlation between Q_p and Q_m for Tomlinson method

Predicted and measured capacity Figure 4.9 represent the co-relation between predicted and measured capacity for Tomlinson method. From statistical analysis and cumulative probability (shown in Table 4.3), it has been observed that the lowest RI(RI=10) ranked this method first amongst all other methods in precast driven piles. This is the best method to predict the ultimate capacity of precast driven piles with 3.20% overestimation. Figure 4.6 justifies this error. The COD value of 0.868 indicates very good compatibility of the predicted theory. No such limit for the determination of effective stress. For this large value of end bearing obtained for large displacement piles. But Peck recommended limiting unit end bearing (11mn/m2) adopted by Tomlinson to keep the ultimate capacity justified with measured capacity.

4.5.1.4 Norwegian Pile Guideline (2005) Method

Comparing the predicted capacity from Norwegian Pile Guideline (2005) method with measured capacity from load test for fifteen numbers of precast driven piles gives a regression equation, Q_p =0.725Q_m-0.273 with R^2=0. 94. High value of R^2 indicates a better correlation between

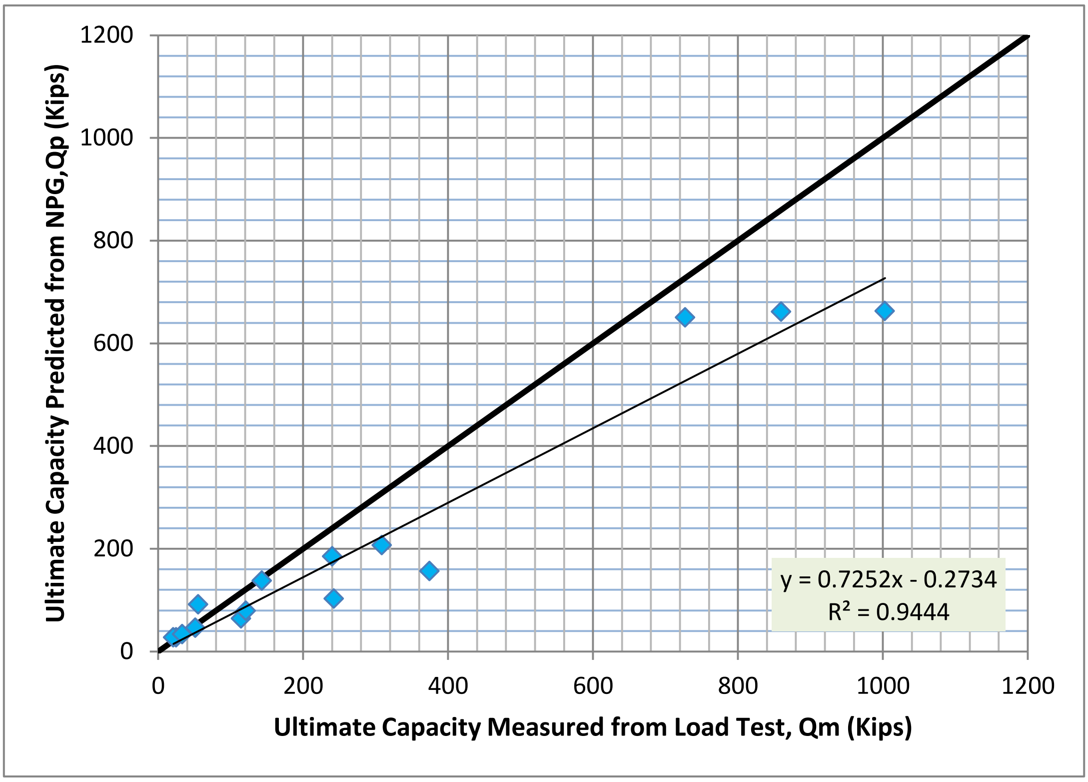

Figure 4.10: Correlation between Q_p and Q_m for NPG method

Predicted and measured capacity Figure 4.10 represent the co-relation between predicted and measured capacity for NPG method. From statistical analysis and cumulative probability (shown in Table 4.3), it has been observed that the highest RI (RI=21) ranked this method fifth amongst all other methods in precast driven piles. This is the worst conservative method to predict the ultimate capacity of precast driven piles with 23.10% underestimation. Figure 4.6 justifies this error. COD value of 0.827 indicates the good compatibility of the predicted theory. No such limit for the determination of effective stress. An effective stress method is used to calculate skin friction for cohesionless soil. The co-efficient of skin friction β is a function of OCR. Largely driven piles give very high side resistance for higher β value and highly effective stress value. But bearing capacity factor Nq is very conservative.

4.5.1.5 Indian Standard (2010) Method

Comparing the predicted capacity from Indian Standard (2010) method with measured capacity from load test for fifteen numbers of precast driven piles gives a regression equation, Q_p=0.659Q_m+23.98 with R^2=0.97. Highest value of R^2 indicates a better correlation between

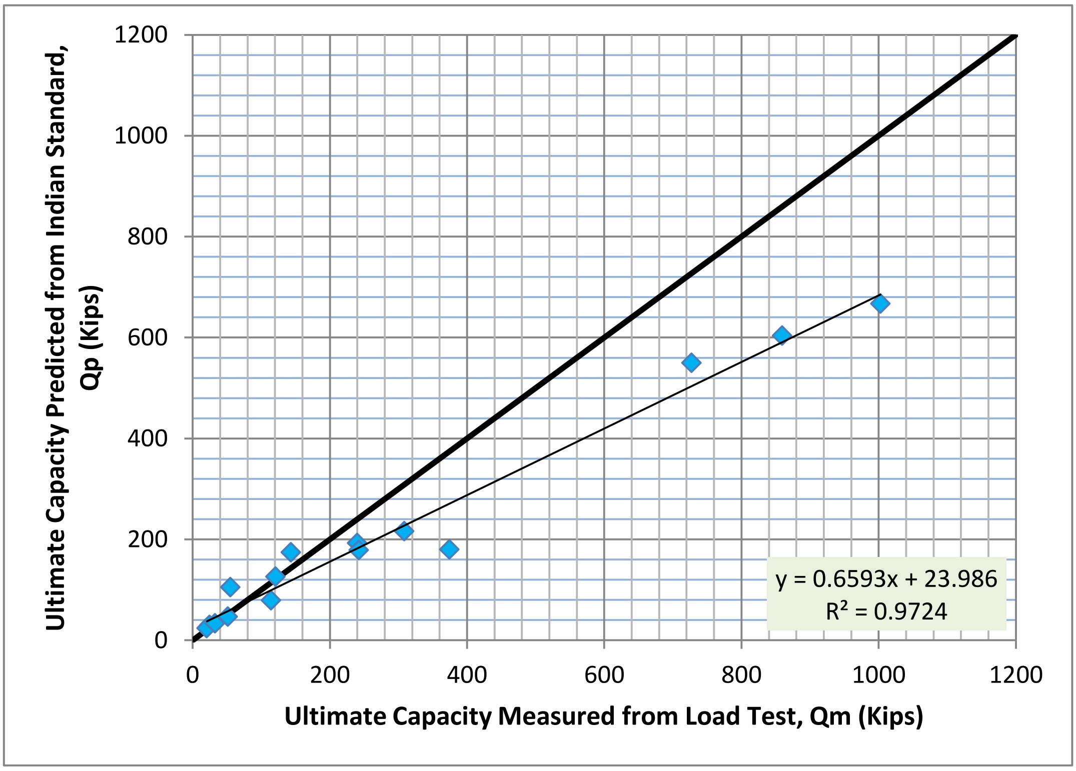

Figure 4.11: Correlation between Q_p and Q_m for Indian Standard method

Predicted and measured capacity Figure 4.11 represent the co-relation between predicted and measured capacity for Indian Standard method. From statistical analysis and cumulative probability (shown in Table 4.3), it has been observed that a high RI (RI=19) ranked this method fourth amongst all other methods in precast driven piles. This is a very conservative method to predict the ultimate capacity of precast driven piles with 20% underestimation. Figure 4.6 justifies this error. COD value of 0.814 indicates the good compatibility of the predicted theory. Indian Standard (2010) adopted Terzaghi Bearing Capacity equation to calculate the end bearing capacity of piles. Hence the length and shape of piles play an important role in the computation of capacity.

4.5.2 Cast in situ Bored Pile

4.5.2.1 Meyerhof (1976) Method

Comparing the predicted capacity from Meyerhof (1976) method with measured capacity from load test for eight numbers of the cast in situ bored piles gives a regression equation, Q_p=0.957Qm+50.32 with R^2=0.56. Moderate value of R^2 indicates a pretty good correlation between

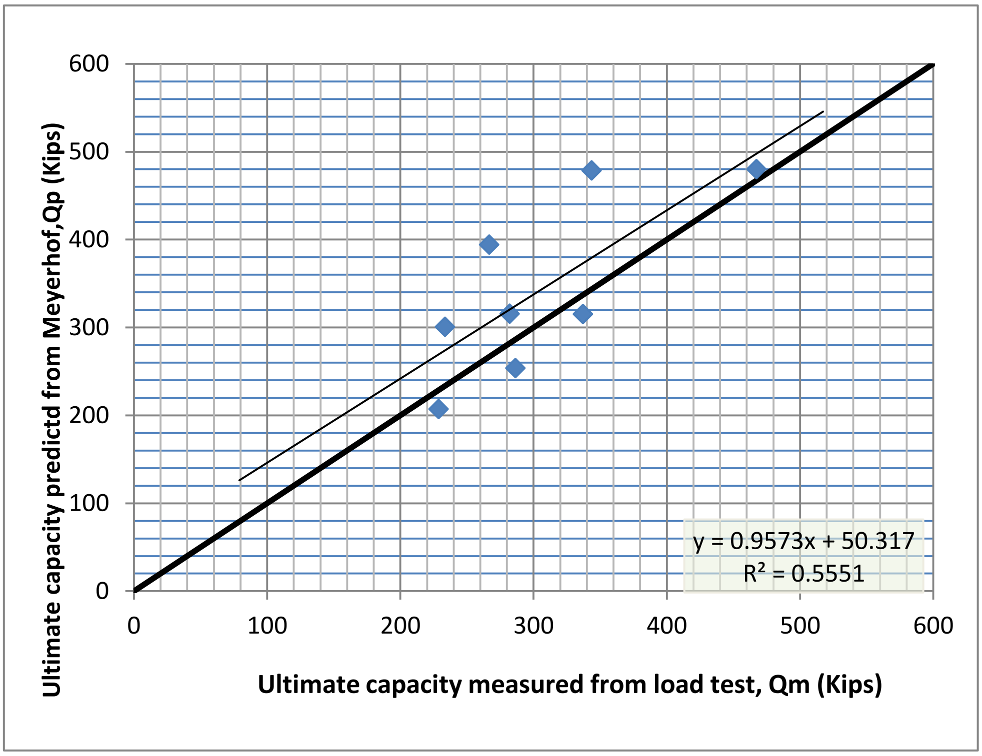

Figure 4.12: Correlation between Q_p and Q_m for Meyerhof method

Predicted and measured capacity Figure 4.12 represent the co-relation between predicted and measured capacity for Meyerhof method. From statistical analysis and cumulative probability (shown in Table 4.4), it has been observed that the lowest RI (RI=10) ranked this method second amongst all other methods in bored piles. Also, this method predicts to overestimate the ultimate capacity of 9%. Figure 4.5 justifies this error. This method to predict the capacity of bored piles is based on SPT. In this method field, SPT has been used. But for calculating the side friction field SPT has been used other hand for computing end bearing overburden pressure correction for SPT has been taken places. Also, Meyerhof limiting unit friction and limiting end bearing has been considered. This is kept the predicted capacity in minor error.

4.5.2.2 NAVFAC (1984) Method

Comparing the predicted capacity from NAVFAC (1984) method with measured capacity from load test for fifteen numbers of the cast in situ bored piles gives a regression equation, Q_p=0.995Q_m+21.62 with R^2=0.64. Moderate value of R^2 indicates a pretty good correlation between

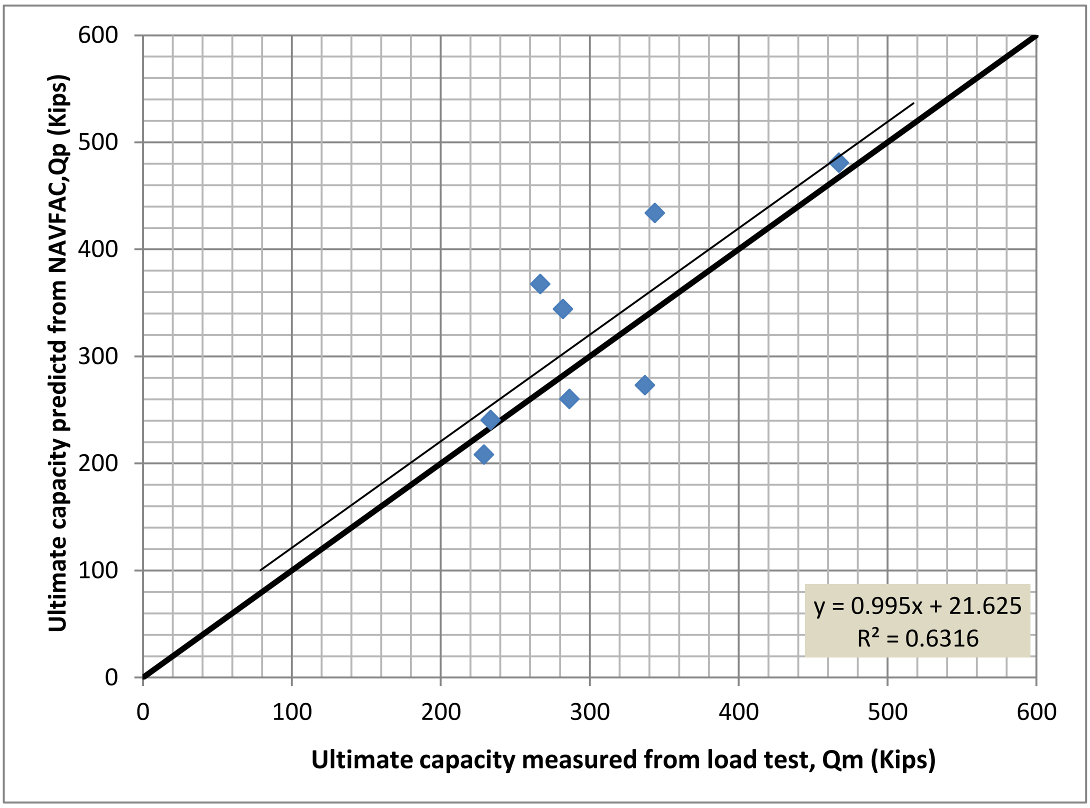

Figure 4.13: Correlation between Q_p and Q_m for NAVFAC method

Predicted and measured capacity Figure 4.13 represent the co-relation between predicted and measured capacity for NAVFAC method. From statistical analysis and cumulative probability (shown in table 4.4), it has been observed that a lower RI (RI=6) ranked this method first amongst all other methods in bored piles. Also, this method predicts to overestimate the ultimate capacity of 1%. Figure 4.5 justifies this error. COD value of 0.35 indicates the fair reliability of the predicted theory. This method predicts the capacity of bored piles in clay-based on total stress and in sand based on effective stress. In sand high bearing capacity factor for high SPT values gives the high capacity for long piles. But, limiting value of unit toe resistance and unit side resistance keep the predicted capacity relevant with measured capacity.

4.5.2.3 AASHTO (1986) Method

Comparing the predicted capacity from AASHTO (1986) method with measured capacity from load test for eight numbers of the cast in situ bored piles gives a regression equation, Q_p=1.22Q_m+37.75 with R^2=0.80. Higher value of R^2 indicates a good correlation between

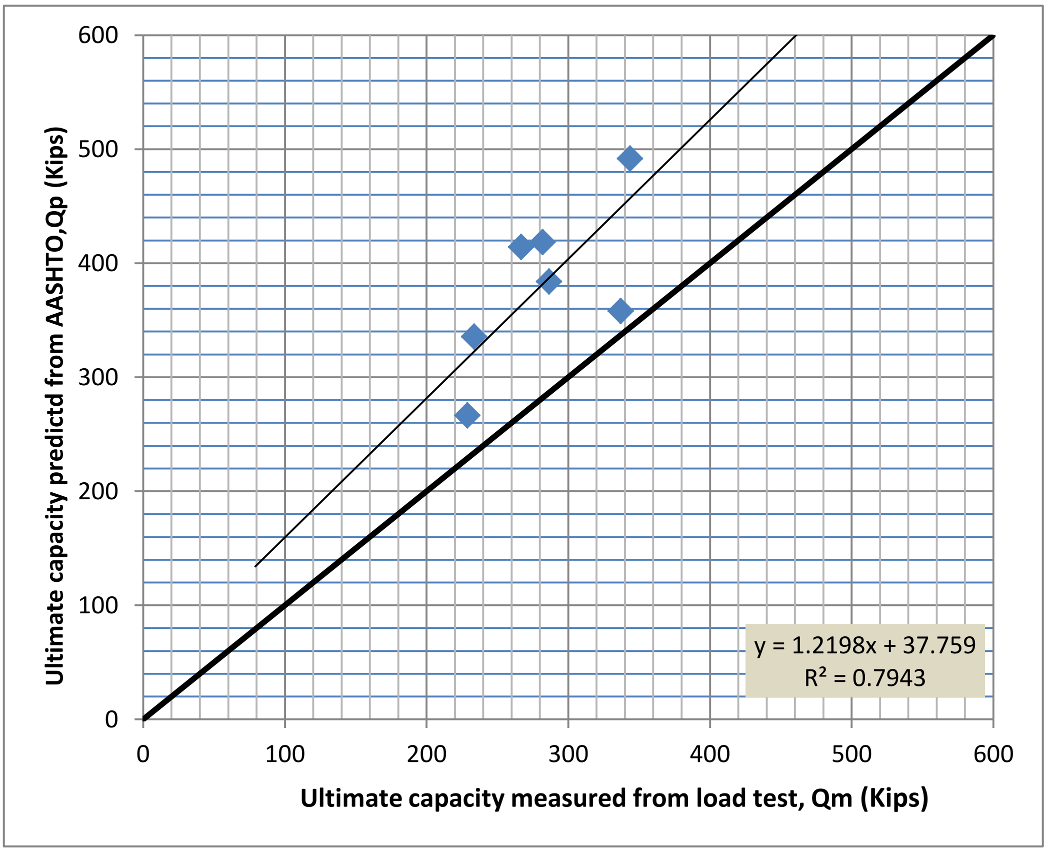

Figure 4.14: Correlation between Q_p and Q_m for AASHTO method

Predicted and measured capacity Figure 4.14 represent the co-relation between predicted and measured capacity for AASHTO method. From statistical analysis and cumulative probability (shown in table 4.4), it has been observed that a moderate RI (RI=17) ranked this method fourth amongst all other methods in bored piles. Also, this method predicts to overestimate the ultimate capacity by 40%. Figure 4.5 justify this error. This method predicts the capacity of bored piles in clay-based on total stress and in the sand based on effective stress. In sand side friction factor β is the function of the depth of the pile. It has an upper value of 1.2 for short length pile and a lower value of 0.25 after 87.5 feet depth of piles. Piles of low depth predict high value compare with long length piles. In sand end bearing capacity depends on SPT value. For high depth piles, it gives higher values.

4.5.2.4 O'neill & Reese (1988) Method

Comparing the predicted capacity from O'Neill & Reese (1988) method with measured capacity from load test for eight numbers of the cast in situ bored piles gives a regression equation, $Q_p=1.40Q_m-56$ with $R^2=0.76$. Higher value of R^2 indicates a good correlation between

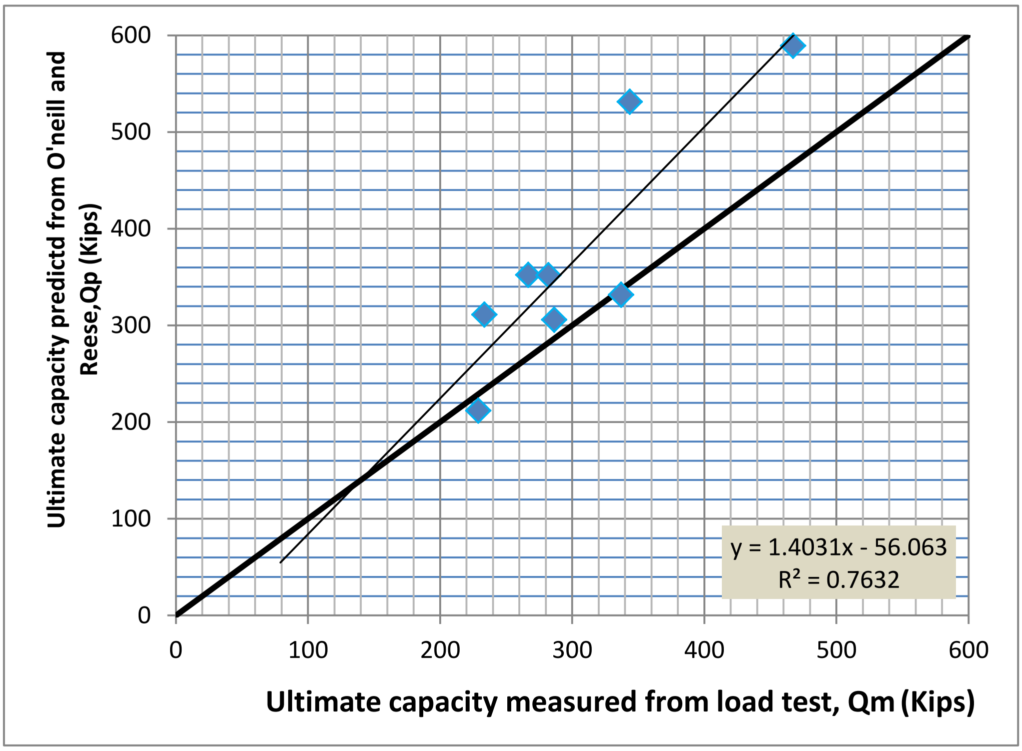

Figure 4.15: Correlation between Q_p and Q_m for O'Neill and Reese method

Predicted and measured capacity Figure 4.15 represent the co-relation between predicted and measured capacity for O'Neill and Reese method. From statistical analysis and cumulative probability (shown in Table 4.4), it has been observed that a lower RI (RI=13) ranked this method third amongst all other methods in bored piles. Also, this method predicts to overestimate the ultimate capacity by 24.60%. Figure 4.5 justifies this error. This method predicts the capacity of bored piles in clay-based on total stress and in the sand based on effective stress. In sand side friction factor β is the function of the coefficient of lateral earth pressure (k). This k is the function of pre consolidated stress and over consolidation ratio. With the increase of depth of pile effective stress increase abruptly. This causes higher depth piles to give more side friction due to excess vertical stress. Piles of high depth predict high value compare with short length piles. In sand end bearing capacity depends on SPT value. For high depth piles, it gives higher values.

4.5.2.5 Decourt (1995) Method

Comparing the predicted capacity from Decourt's (1995) method with measured capacity from load test for eight numbers of the cast in situ bored piles gives a regression equation, Q_p=1.89Q_m+130.54 with R^2=0.52. Moderate value of R^2 indicates a pretty good correlation between

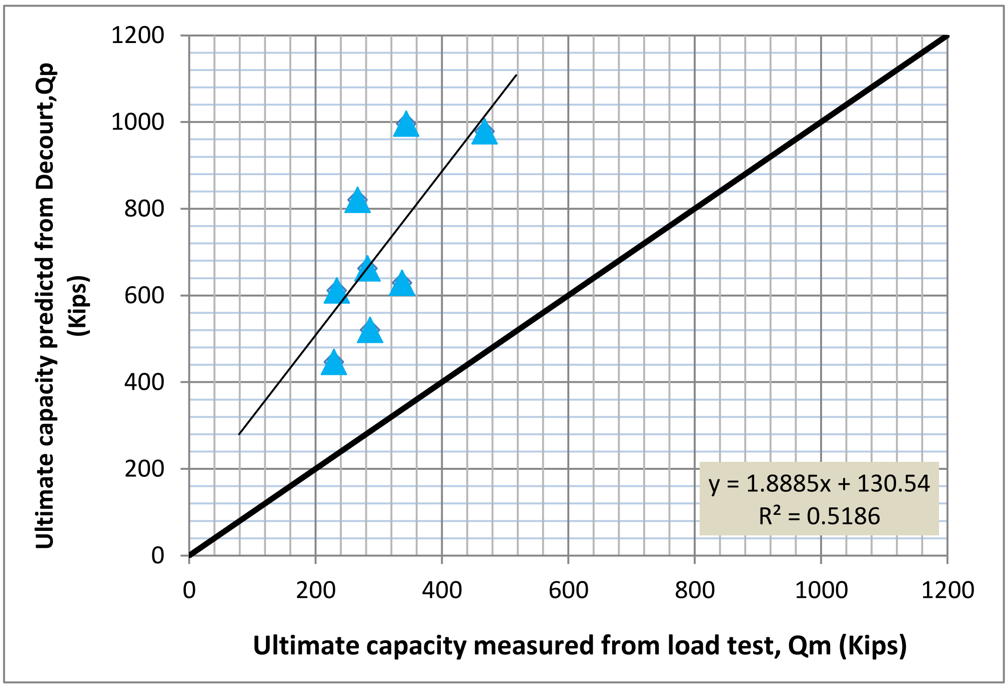

Figure 4.16: Correlation between Q_p and Q_m for Decourt method

Predicted and measured capacity Figure 4.16 represent the co-relation between predicted and measured capacity for Decourt method. From statistical analysis and cumulative probability (shown in Table 4.4, it has been observed that the highest RI (RI=23) ranked this method fifth amongst all other methods in bored piles. Also, this method predicts to overestimate the ultimate capacity of134%. Figure 4.5 justify this error. COD value of -33.84 indicates very poor reliability of the predicted theory. It is a SPT based empirical formula invents by Decourt with numbers of the load test. This empirical formula gives justified values for skin friction both clay and sand. But computation of end bearing capacity in the sand gives very high values due to the overestimate of end bearing co-efficient.

4.5.3 Cast in Situ Drilled Shaft

4.5.3.1 Meyerhof (1976) Method

Comparing the predicted capacity from Meyerhof (1976) method with measured capacity from load test for seven numbers of the cast in situ drilled shafts gives a regression equation, $Q_p=0.04Q_m+1937$ with $R^2=0$. Value of R^2 and other statistical data indicates the inability of this method to predict the capacity of drilled shaft. Figure 4.17 represent the co-relation between predicted and measured capacity for Meyerhof method.

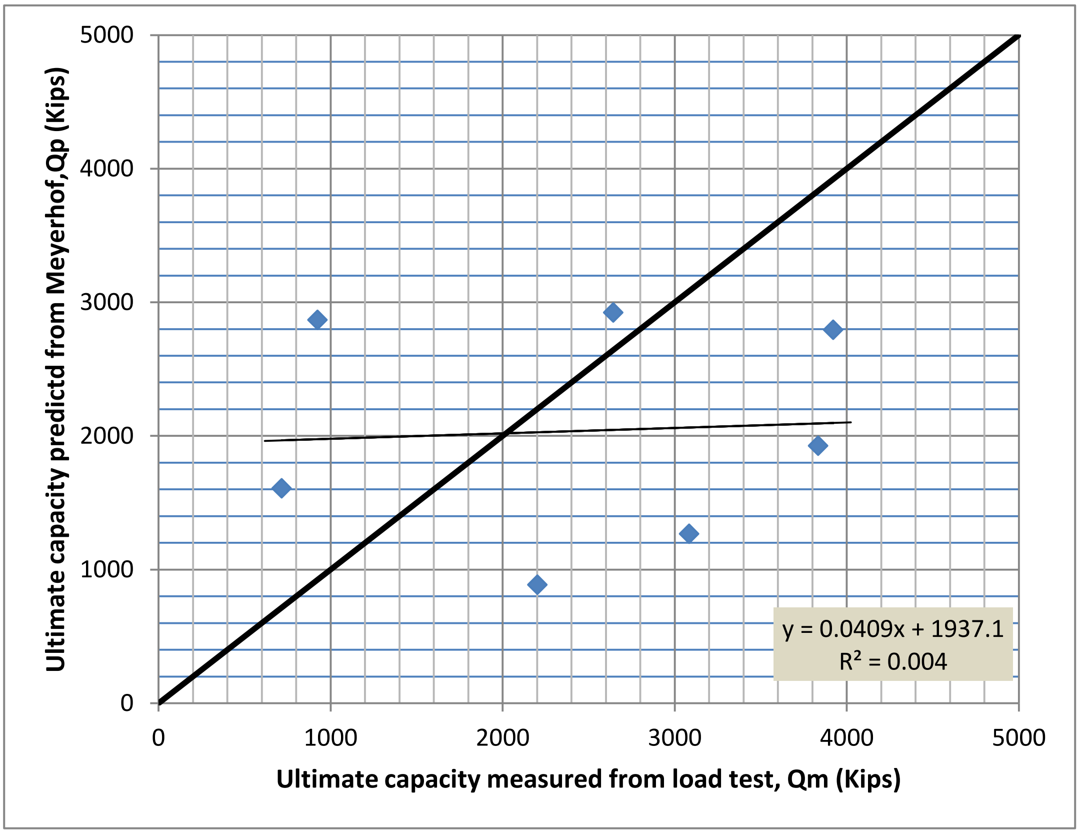

Figure 4.17: Correlation between Q_p and Q_m for Meyerhof method

4.5.3.2 NAVFAC (1984) Method

Comparing the predicted capacity from NAVFAC (1984) method with measured capacity from load test for seven numbers of the cast in situ drilled shafts gives a regression equation, Q_p=0.006Q_m+3231 with R^2=0. Value of R^2 and other statistical data indicates the inability of this method to predict the capacity of drilled shaft.

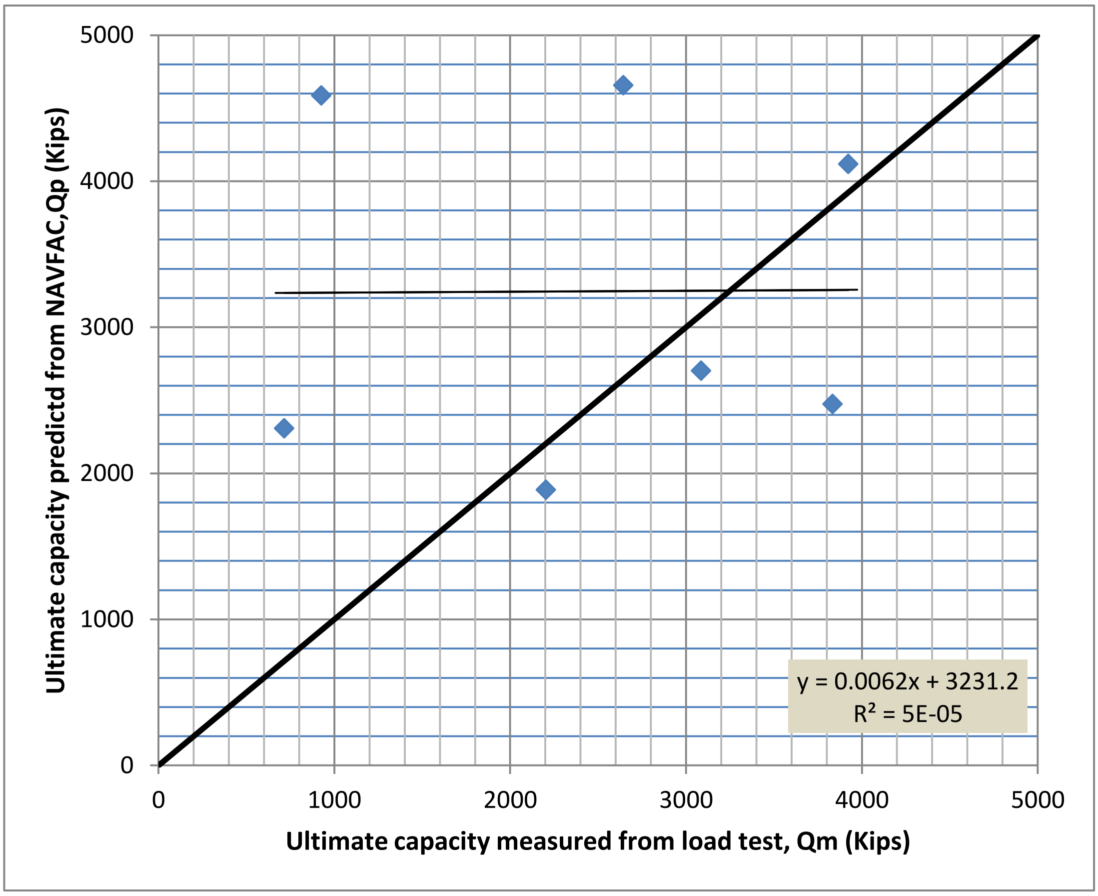

Figure 4.18: Correlation between Q_p and Q_m for NAVFAC method

This method predicts the capacity of bored piles in clay based on total stress and in sand based on effective stress. In sand high bearing capacity factor for high SPT values gives the irrelevant capacity for long piles. Figure 4.18 represent the co-relation between predicted and measured capacity for NAVFAC method.

4.5.3.3 AASHTO (1986) Method

Comparing the predicted capacity from AASHTO (1986) method with measured capacity from load test for seven numbers of the cast in situ drilled shafts gives a regression equation, Q_p=0.228Q_m+1764.6 with R^2=0.13. Low value of R^2 indicates a some correlation between

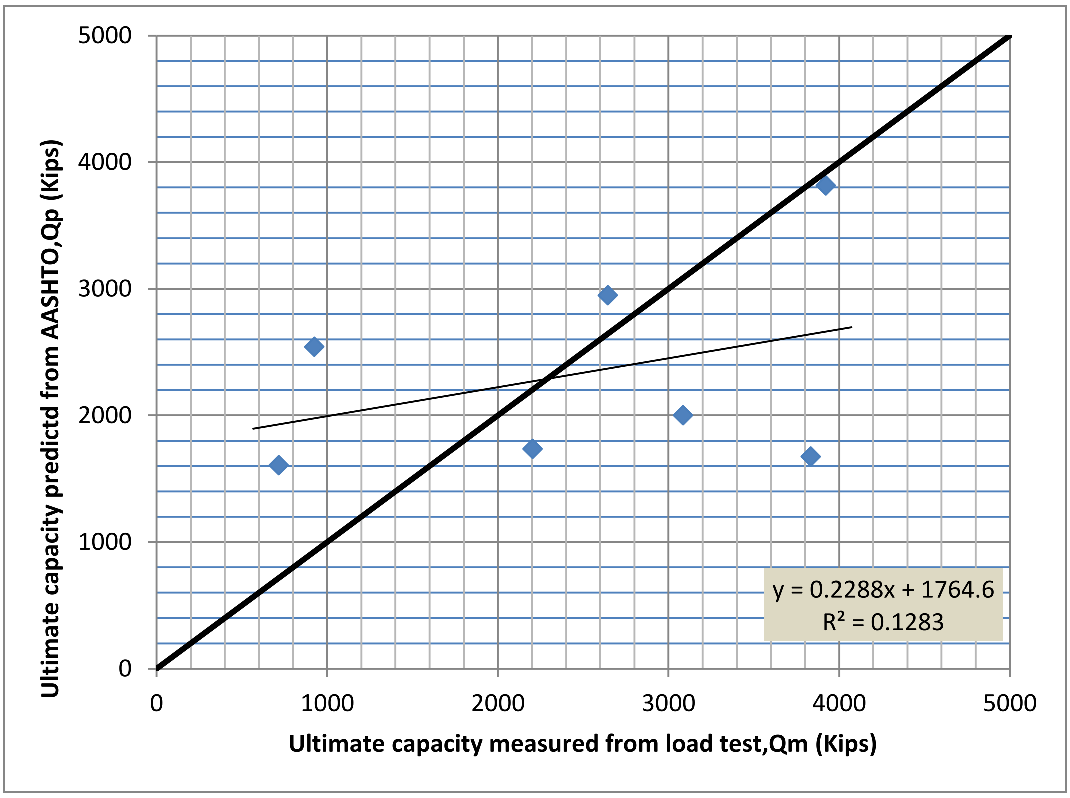

Figure 4.19: Correlation between Q_p and Q_m for AASHTO method

Predicted and measured capacity. From statistical analysis and cumulative probability (shown in Table 4.5), it has been observed that a lower RI (RI=7) ranked this method first amongst all other methods in drilled shaft. Also, this method predicts to underestimate the ultimate capacity by 3%. Figure 4.4 justifies this error. COD value of 0.5 indicates a good reliability of the predicted theory. Figure 4.19 represent the co-relation between predicted and measured capacity for AASHTO method.

4.5.3.4 O'neill & Reese (1988) Method

Comparing the predicted capacity from O’Neill & Reese (1988) method with measured capacity from load test for seven numbers of the cast in situ drilled shafts gives a regression equation, Q_p=0.347Q_m+2234 with R^2=0.1. Lower value of R^2 indicates some correlation between

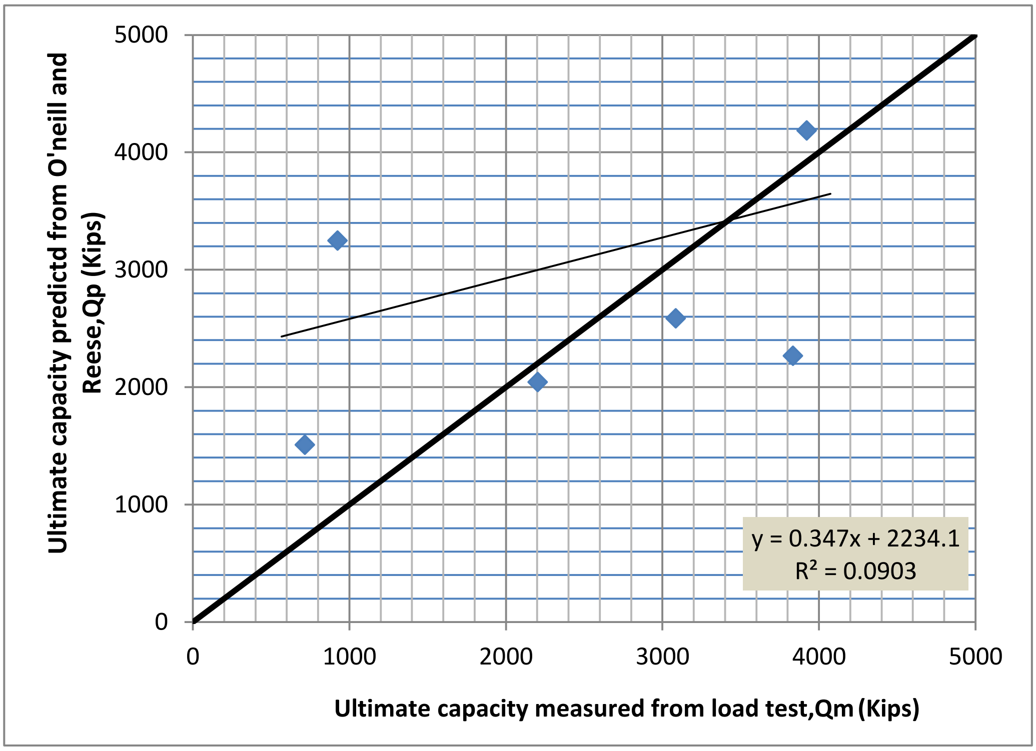

Figure 4.20: Correlation between Q_p and Q_m for O’Neill and Reese method

Predicted and measured capacity. From statistical analysis and cumulative probability (shown in Table 4.5), it has been observed that a lower RI (RI=15) ranked this method third amongst all other methods in drilled shafts. Also, this method predicts to overestimate the ultimate capacity by 6%. Figure 4.4 justifies this error. Figure 4.20 represent the co-relation between predicted and measured capacity for O’Neill and Reese method.

4.5.3.5 Decourt (1995) Method

Comparing the predicted capacity from Decourt's (1995) method with measured capacity from load test for seven numbers of the cast in situ drilled shafts gives a regression equation, $Q_p=0.087Q_m+4356$ with $R^2=0.01$.Very low value of R^2 indicates a poor correlation between

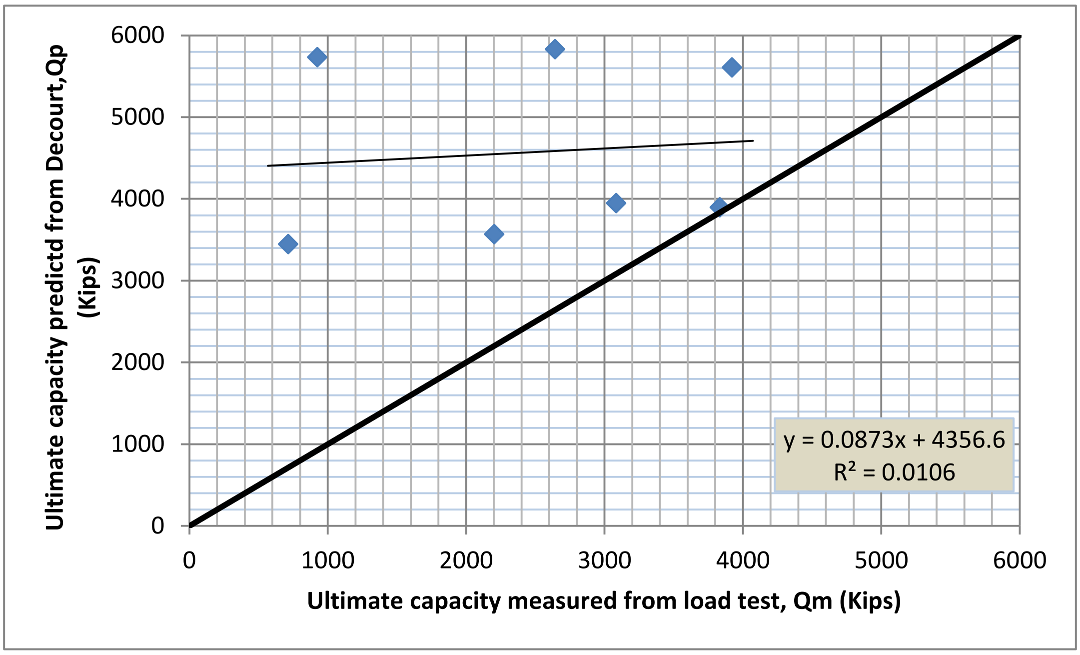

Figure 4.21: Correlation between Q_p and Q_m for Decourt method

Predicted and measured capacity. From statistical analysis and cumulative probability (shown in Table 4.5), it has been observed that the highest RI (RI=24) ranked this method fifth amongst all other methods in drilled shafts. Also, this method predicts to overestimate the ultimate capacity of 62%.Figure 4.4 justify this error. It is a SPT based empirical formula invents by Decourt with numbers of the load test. This empirical formula gives justified values for skin friction both clay and sand. But computation of end bearing capacity in the sand gives very high values due to the overestimate of end bearing co-efficient. Also, the uncorrected SPT value gives higher results for long piles. Figure 4.21 represent the co-relation between predicted and measured capacity for Decourt method.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In this study sub-soil investigation report and corresponding pile load test results have been collected from twenty-two projects all over the country. Among these projects, twelve projects have been selected where fifteen precast piles have been tested and ten projects have been selected where fifteen cast-in-situ piles have been tested. The tests are performed between 1997 to 2018 and funded by the Public Works Department (PWD), Bangladesh, RAJUK, R&H Department, Bangladesh, and Dhaka Mass Transit Company (MRT). Almost 70% pile load tests are carried out under the direct supervision of the Department of Civil Engineering, BUET, and the rest of the pile load test carried out by Icon Engineering Services, Dhaka. The findings of this study are as follows:

The length of fifteen cast in situ piles varies from 12 meters to 52 meters having diameters of 400 mm to 1200 mm. For convenience of analysis it has been divide into two categories named Bored pile and Drilled shaft. It has been observed that the long piles with large diameter generally conservatively predict the capacity of piles than the short piles (shorter than 18 m) for different statics methods except the Decourt (1995) method. It can be concluded for bored piles that the long and larger diameter pile capacity are governed by settlement rather than capacity.

Cast in situ piles larger than 600 mm diameter considered as drilled shafts according to AASHTO. In this study only AASHTO (1986) and O’neill and Reese (1988) methods are reliable to predict the ultimate capacity of drilled shaft. For the drilled shafts, the AASHTO (1986) and O’Neill and Reese (1988) methods provide relatively better correlation between predicted and measured capacity. Other theoretical methods describe here are not suitable to predict the capacity of drilled shafts. Drilled Shaft installation is a highly technological task. During boring of drilled shaft mud slurry has been used. This slurry creates a thin layer around the borehole of the drilled shaft. If this thin layer does not disappear during concreting it causes to reduce the skin friction between soil and Drilled Shaft. It lowers the measured capacity of the Drilled Shaft. In Drilled Shaft construction a crack is formed in the first 5 feet of the soil. This layer of cracked soil doesn't exhibit any skin friction. This consideration is ignored by most of the theoretical methods used in this study except AASHTO and Reese et. al. Caving, necking, and construction faults also reduce the measured capacity of Drilled Shaft. The above issues are not considered in most of the theoretical prediction. This is why correlation among predicted and measured capacity is low.

For the cast in situ bored piles, the AASHTO (1986), O'Neill and Reese (1988) and NAVFAC (1984) methods provide relatively reliable and justified correlation between predicted and measured capacity.

On the other hand, the length of fifteen precast piles varies from 7 meters to 30.5 meters having sizes of 175 mm x175mm to 400 mm x 400 mm. It has been found in this study that the predicted pile capacity using all the methods is relatively conservative. Pile driving energy plays an important role to increase the capacity of piles. In this study, it has been observed that the Tomlinson (1994), API (1993) and Meyerhof (1976) methods provide the most reliable results.

It has also been observed that the methods for predicting the ultimate capacity of precast piles give relatively higher reliability than bored piles. In all the cases, a reliable high correlation between the theoretical analysis of pile capacity and capacity of the pile from the static load test are found. This study has supported the idea to put a higher degree of confidence to use the statics formulae to find out the ultimate capacity of the precast piles.

5.2 Recommendations for Further Research

- Test piles should always be loaded until failure, only then ultimate capacity and corresponding settlement can be reliably estimated. This may sometimes reduce the total project cost.
- The database of the pile load test and subsoil exploration data should be updated from time to time and the correlation proposed here should also be updated to use them with more confidence.
- Public Works Department of Bangladesh is a national institute of repute, they should develop an archive in their Headquarters for the preservation of pile load test and other in-situ data, so that future generation can learn from those data and use them for engineering research and practical application.

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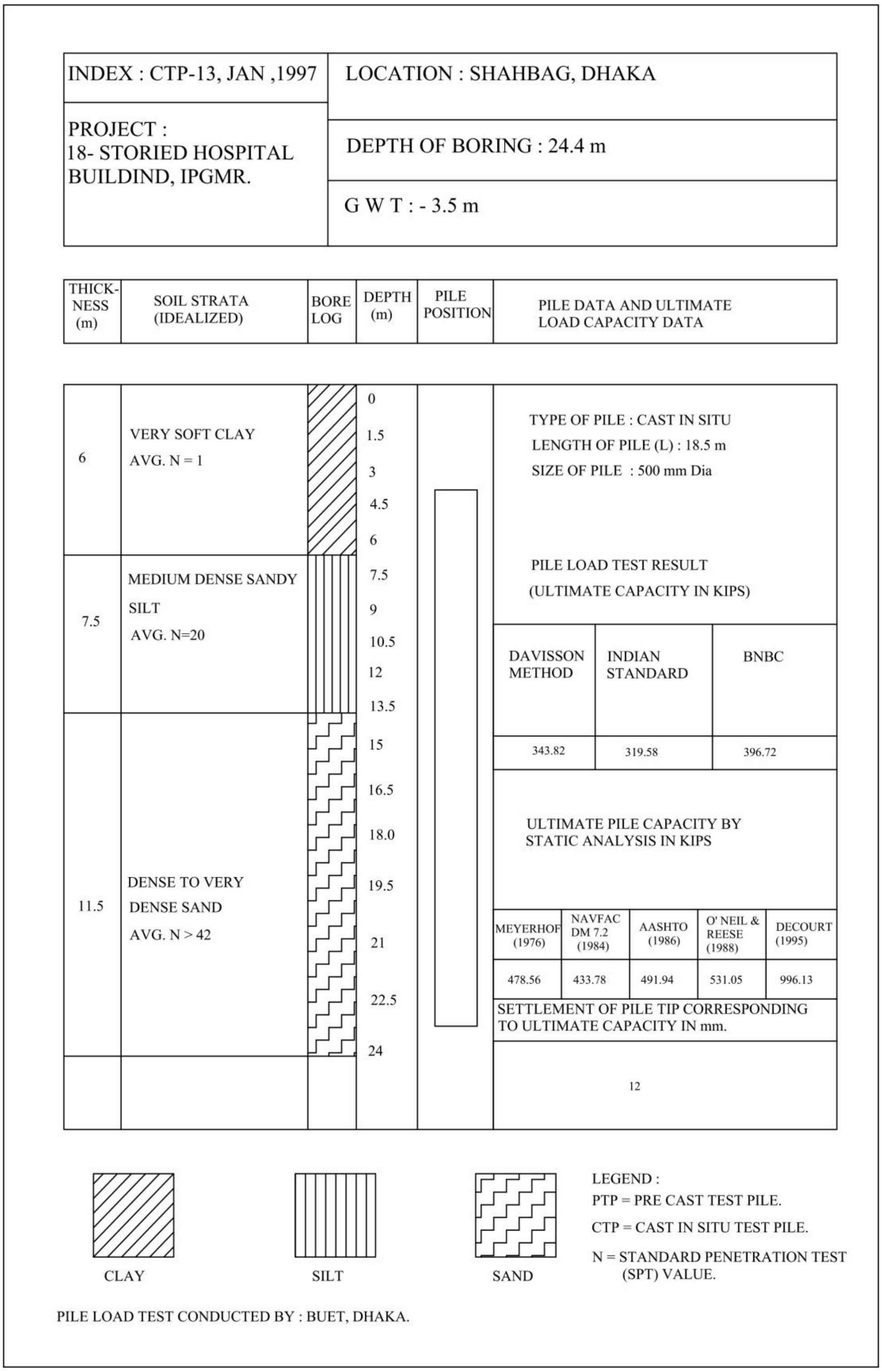
APPENDIX A

SOIL IDEALIZATION AND INDIVIDUAL PILE CAPACITY FOR BORED PILE AND DRIVEN PILE

CTP-01



INDEX : CTP-13, JAN ,1997	LOCATION : SHAHBAG, DHAKA
PROJECT : 18- STORIED HOSPITAL BUILDIND, IPGMR.	DEPTH OF BORING : 24.4 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
6	VERY SOFT CLAY AVG. N = 1	▨	0 1.5 3 4.5 6	
7.5	MEDIUM DENSE SANDY SILT AVG. N=20	▥	7.5 9 10.5 12 13.5	
11.5	DENSE TO VERY DENSE SAND AVG. N > 42		15 16.5 18.0 19.5 21 22.5 24	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 18.5 m
SIZE OF PILE : 500 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
343.82	319.58	396.72

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
478.56	433.78	491.94	531.05	996.13

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

12

▨ CLAY
▥ SILT
SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-02

INDEX : CTP-14	LOCATION : SHAHBAG, DHAKA
PROJECT : 18- STORIED HOSPITAL BUILDIND, IPGMR.	DEPTH OF BORING : 24.4 m G W T : - 3.0 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
4	MEDIUM STIFF TO STIFF CLAY AVG. N = 8	▨	0 1.5 3	
2	STIFF TO VERY STIFF CLAY AVG. N = 12	▨	4.5 6	
7	MEDIUM DENSE TO DENSE SANDY SILT AVG. N=24	▥	7.5 9 10.5 12 13.5	
11.4	DENSE TO VERY DENSE SAND AVG. N > 42	▦	15 16.5 18.0 19.5 21 22.5 24	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU

LENGTH OF PILE (L) : 18.5 m

SIZE OF PILE : 500 mm Dia

PILE LOAD TEST RESULT

(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
467.24	484.88	551

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
479.92	480.69	616.65	589.13	978.27

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

16

▨	▥	▦
CLAY	SILT	SAND

LEGEND :

PTP = PRE CAST TEST PILE.

CTP = CAST IN SITU TEST PILE.

N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-03

INDEX : CTP-03, APR , 1998	LOCATION : DAMPARA, CHITTAGONG
PROJECT : JAMIATUL FALAH ISLAMIC CENTRE AND MASJID	DEPTH OF BORING : 15.25 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
4	MEDIUM STIFF CLAY AVG. N = 6	▨	0 1 2 3
2	LOOSE SANDY SILT AVG. N = 9	▥	4 5 6
3	MEDIUM DENSE TO DENSE SILTY SAND AVG. N=25	▦	7 8 9
6.25	VERY DENSE SAND AVG. N > 45	▦	10 11 12 13 14 15

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 12 m
SIZE OF PILE : 500 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
268.88	246.84	253.46

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
394	367.51	414.04	352.24	820.06

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

9

▨	▥	▦
CLAY	SILT	SAND

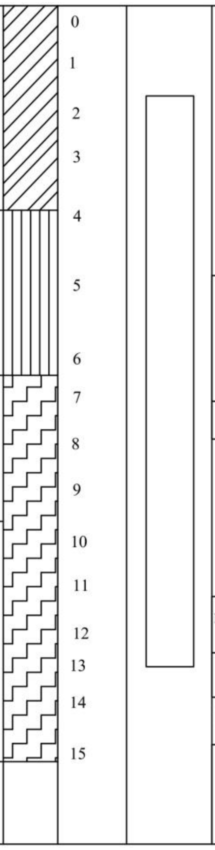
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-04

INDEX : CTP-04, APR , 1998	LOCATION : DAMPARA, CHITTAGONG
PROJECT : JAMIATUL FALAH ISLAMIC CENTRE AND MASJID	DEPTH OF BORING : 15.25 m G W T : - 3.5 m

THICKNESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG
4.5	SOFT SILTY CLAY AVG. N = 5	▨
3.5	MEDIUM STIFF CLAYEY SILT AVG. N = 9	▥
2	LOOSE SILTY SAND AVG. N=12	▦
5.25	MEDIUM DENSE TO DENSE SAND AVG. N > 45	▦



PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 12 m
SIZE OF PILE : 500 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
229.21	233.62	250.00

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
206.99	207.96	266.80	211.85	446.86

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

7.5

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-05

INDEX : CTP-01, JAN , 1998	LOCATION : DHAKA
PROJECT : EXAMINATION COMPLEX, EDUCATION BOARD, DHAKA	DEPTH OF BORING : 26 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
6.5	STIFF TO VERY STIFF CLAYEY SILT AVG. N = 12	▥	0 1.5 3 4.5 6		TYPE OF PILE : CAST IN SITU LENGTH OF PILE (L) : 14 m SIZE OF PILE : 400 mm Dia
4	MEDIUM DENSE TO DENSE SANDY SILT AVG. N=28	▥	7.5 9 10.5		
15.5	DENSE TO VERY DENSE SAND AVG. N > 45	▦	12 13.5 15 16.5 18.0 19.5 21 22.5 26		

PILE LOAD TEST RESULT

(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
337.21	348.23	363.00

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
315.22	272.87	358.19	331.48	629.54

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

14

▨ CLAY

▥ SILT

▦ SAND

LEGEND :

PTP = PRE CAST TEST PILE.

CTP = CAST IN SITU TEST PILE.

N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-06

INDEX : CTP-01, JAN , 1998	LOCATION : DHAKA
PROJECT : EXAMINATION COMPLEX, EDUCATION BOARD, DHAKA	DEPTH OF BORING : 26 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
6.5	STIFF TO VERY STIFF CLAYEY SILT AVG. N = 12	▥	0 1.5 3 4.5 6	
4	MEDIUM DENSE TO DENSE SANDY SILT AVG. N=28	▥	7.5 9 10.5	
15.5	DENSE TO VERY DENSE SAND AVG. N > 45	▦	12 13.5 15 16.5 18.0 19.5 21 22.5 26	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 14 m
SIZE OF PILE : 400 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
337.21	348.23	363.00

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
315.22	272.87	358.19	331.48	629.54

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

14

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-07

INDEX : CTP-02, JAN , 1999	LOCATION : DHAKA
PROJECT : EXAMINATION COMPLEX, EDUCATION BOARD, DHAKA	DEPTH OF BORING : 26 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
5	SOFT SILTY CLAY AVG. N = 3	▨	0 1.5 3 4.5 6	
4	MEDIUM DENSE TO DENSE SANDY SILT AVG. N=24	▥	7.5 9 10.5	
17	DENSE TO VERY DENSE SAND AVG. N > 45	▦	12 13.5 15 16.5 18.0 19.5 21 22.5 26	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 14 m
SIZE OF PILE : 400 mm Dia

PILE LOAD TEST RESULT (ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
233.62	244.64	301.00

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
300.47	240.46	335.73	311.21	611.54

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

11

▨ CLAY ▥ SILT ▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-08

INDEX : CTP-11, AUG, 1999	LOCATION : SEGUNBAGICHA, DHAKA
PROJECT : NATIONAL ART GALLERY AT SHILPAKALA ACADEMY COMPLEX	DEPTH OF BORING : 18.3 m G W T : - 3.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
7	MEDIUM STIFF CLAY AVG. N = 06	▨	0 1 2 3 4 5 6 7
4	MEDIUM DENSE SILTY SAND AVG. N=16	▦	8 9 10 11
15.5	MEDIUM DENSE TO DENSE SAND AVG. N= 31	▦	12 13 14 15 18

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 15 m
SIZE OF PILE : 510 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
282.11	270.23	311.41

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
315.66	343.95	418.75	351.65	662.00

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

13.5

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

CTP-09

INDEX : CTP-09,2017	LOCATION : AGARGAON, DHAKA
PROJECT : DHAKA MASS RAPID TRANSIT DEVELOPMENT	DEPTH OF BORING : 49 m
	G W T : - 1.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
6	GRAY TO BROWN, SOFT TO STIFF MEDIUM PLASTICITY, SANDY LEAN CLAY, CL AVG. N = 5	▨	0 3 6
24	BROWN, MEDIUM DENSE TO DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N=45	▦	8 12 16 20 25 30
5	BROWN DENSE TO VERY DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N=45	▦	33.5
1	BROWN, VERY STIFF, MEDIUM PLASTICITY, SANDY LEAN CLAY, CL AVG. N=24	▨	35
13	BROWN DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N > 40	▦	36 39 43 46 49

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 30 m
SIZE OF PILE : 1000 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
3834.96	2204	2755

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
1925.59	2473.66	1674.75	2265.70	3894.41

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

37

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-10

INDEX : CTP-07	LOCATION : PIER-01, KUMAR BRIDGE
PROJECT : DHAKA-KHULNA (N8) ROAD IMPROVMENT PROJECT	DEPTH OF BORING : 57 m G W T : - 0.62 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
2.25	DEBRIS AVG. N=1		1.5	
4.5	GRAY, VERY SOFT TO SOFT, MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N=1	▨	3 6	
1.5	GRAY, VERY SOFT, HIGH PLASTICITY, FLAT CLAY, CH AVG. N=1	▨	7.5	
12	GRAY, LOOSE TO MEDIUM DENSE, NONPLASTIC, SILT, ML, TRACE OF MICA AVG. N=11	▥	9 16 19.5	
18	GRAY, MEDIUM DENSE TO DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N=33	▦	21 31.5 37.5	
3	GRAY, MEDIUM DENSE TO DENSE, NONPLASTIC, SILT WITH SAND, ML , TRACE OF MICA AVG. N=25	▦	40.5	
15.75	GRAY, DENSE, SILTY SAND, SM , TRACE OF MICA AVG. N > 48	▦	42 45 52 55 57	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 45.10 m
SIZE OF PILE : 1200 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
3923.12	2071.76	2777.04

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
2793.77	4116.57	3813.76	4185.23	5604.00

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

42

▨ CLAY

▥ SILT

▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-11

INDEX : CTP-05,2018	LOCATION : KALSHI , MIRPUR, DHAKA PR-10
PROJECT : WIDENING AND IMPROVEMENT OF ROAD FROM ECB CIRCLE TO MIRPUR AND CONSTRUCTION OF FLYOVER AT KALSHI INTERSECTION PROJECT	DEPTH OF BORING : 31.5 m G W T : - 2.10 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
2.25	DEBRIS AVG. N=1		1.5	
4.5	BROWN SOFT TO MEDIUM STIFF, HIGH PLASTICITY, FAT CLAY, CH AVG. N=3	▨	3 6	
12	BROWN, LOOSE TO MEDIUM DENSE, SILT SAND , SM, TRACE OF MICA AVG. N=15	▦	7.5 9 10.5 12 15 18	
7.5	GRAY, MEDIUM DENSE TO DENSE, NONPLASTIC, SILT WITH SAND, ML , TRACE OF MICA AVG. N=12	▨	19.5 22.5 25.5	
15.75	GRAY, DENSE, SILTY SAND, SM , TRACE OF MICA AVG. N > 48	▦	27 29 31.5	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 30 m
SIZE OF PILE : 1000 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
4231	2204	2821.12

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
885.82	1887.36	1734.71	2041.81	3564.94

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

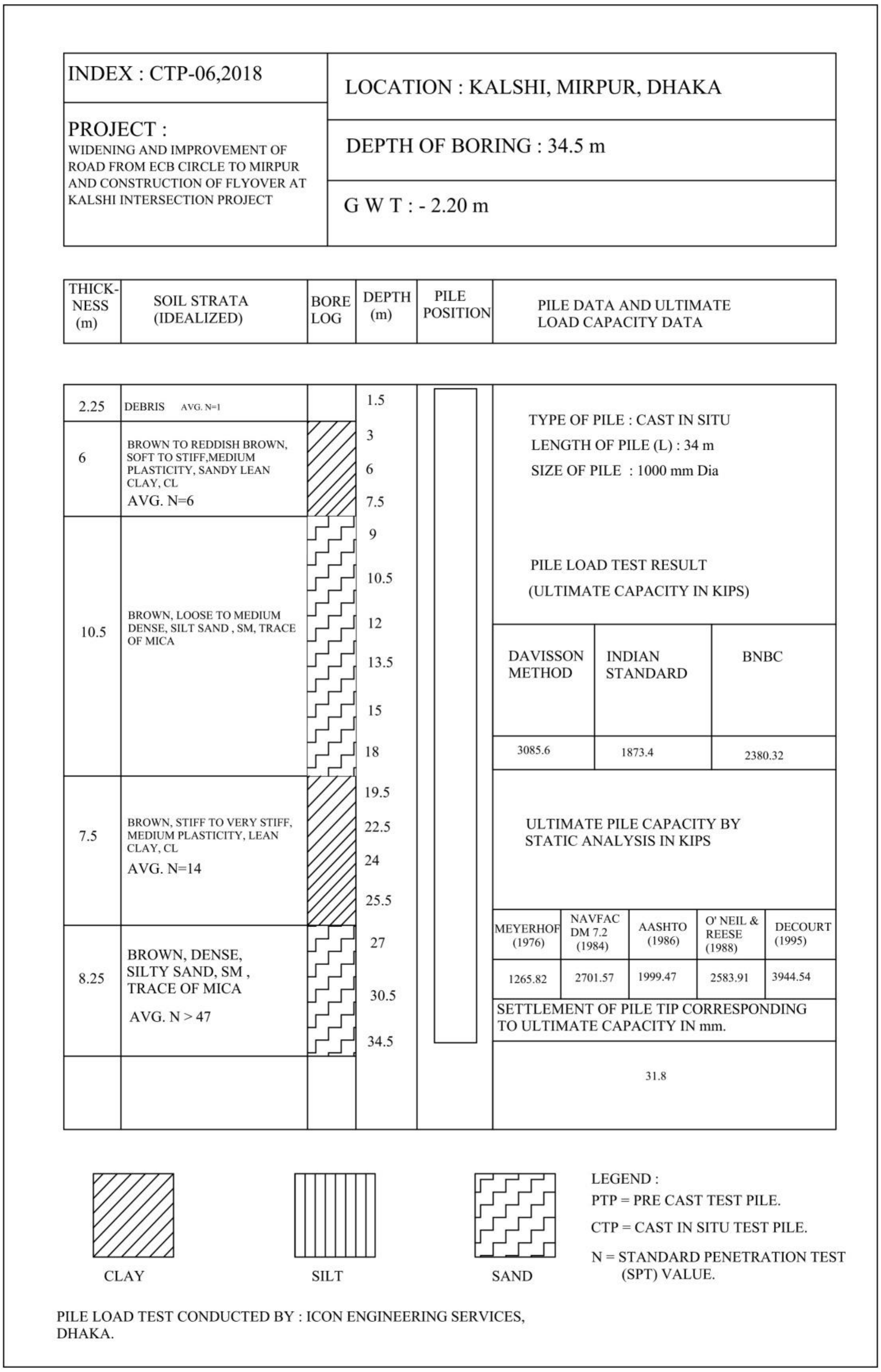
41.6

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-12



INDEX : CTP-06,2018	LOCATION : KALSHI, MIRPUR, DHAKA
PROJECT : WIDENING AND IMPROVEMENT OF ROAD FROM ECB CIRCLE TO MIRPUR AND CONSTRUCTION OF FLYOVER AT KALSHI INTERSECTION PROJECT	DEPTH OF BORING : 34.5 m G W T : - 2.20 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
2.25	DEBRIS AVG. N=1		1.5
6	BROWN TO REDDISH BROWN, SOFT TO STIFF,MEDIUM PLASTICITY, SANDY LEAN CLAY, CL AVG. N=6		3 6 7.5
10.5	BROWN, LOOSE TO MEDIUM DENSE, SILT SAND , SM, TRACE OF MICA		9 10.5 12 13.5 15 18
7.5	BROWN, STIFF TO VERY STIFF, MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N=14		19.5 22.5 24 25.5
8.25	BROWN, DENSE, SILTY SAND, SM , TRACE OF MICA AVG. N > 47		27 30.5 34.5

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 34 m
SIZE OF PILE : 1000 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
3085.6	1873.4	2380.32

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
1265.82	2701.57	1999.47	2583.91	3944.54

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

31.8

▨	▥	
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-13

INDEX : CTP-15,2016	LOCATION : SHIBPUR BRIDGE, TANGAIL
PROJECT : SASEC ROAD CONNECTIVITY PROJECT	DEPTH OF BORING : 27 m G W T : - 6.15 m

THICKNESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
8.25	GRAY, LOOSE, NONPLASTIC, SILT WITH SAND, ML, TRACE OF MICA AVG. N= 4	▨	1.5 3 4.5 6 7.5	
13.75	GRAY, MEDIUM DENSE TO DENSE, SILT SAND, SM, TRACE OF MICA AVG. N = 26	▨	9 12 13.5 15 18 19.5 22.5	
5	GRAY, DENSE,SAND, SM, TRACE OF MICA AVG. N = 50	▨	24 25.5 27	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 25 m
SIZE OF PILE : 1000 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
716.3	672.22	749.36

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
1606.98	2306.82	1606.14	1509.76	3443.33

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

20

▨ CLAY

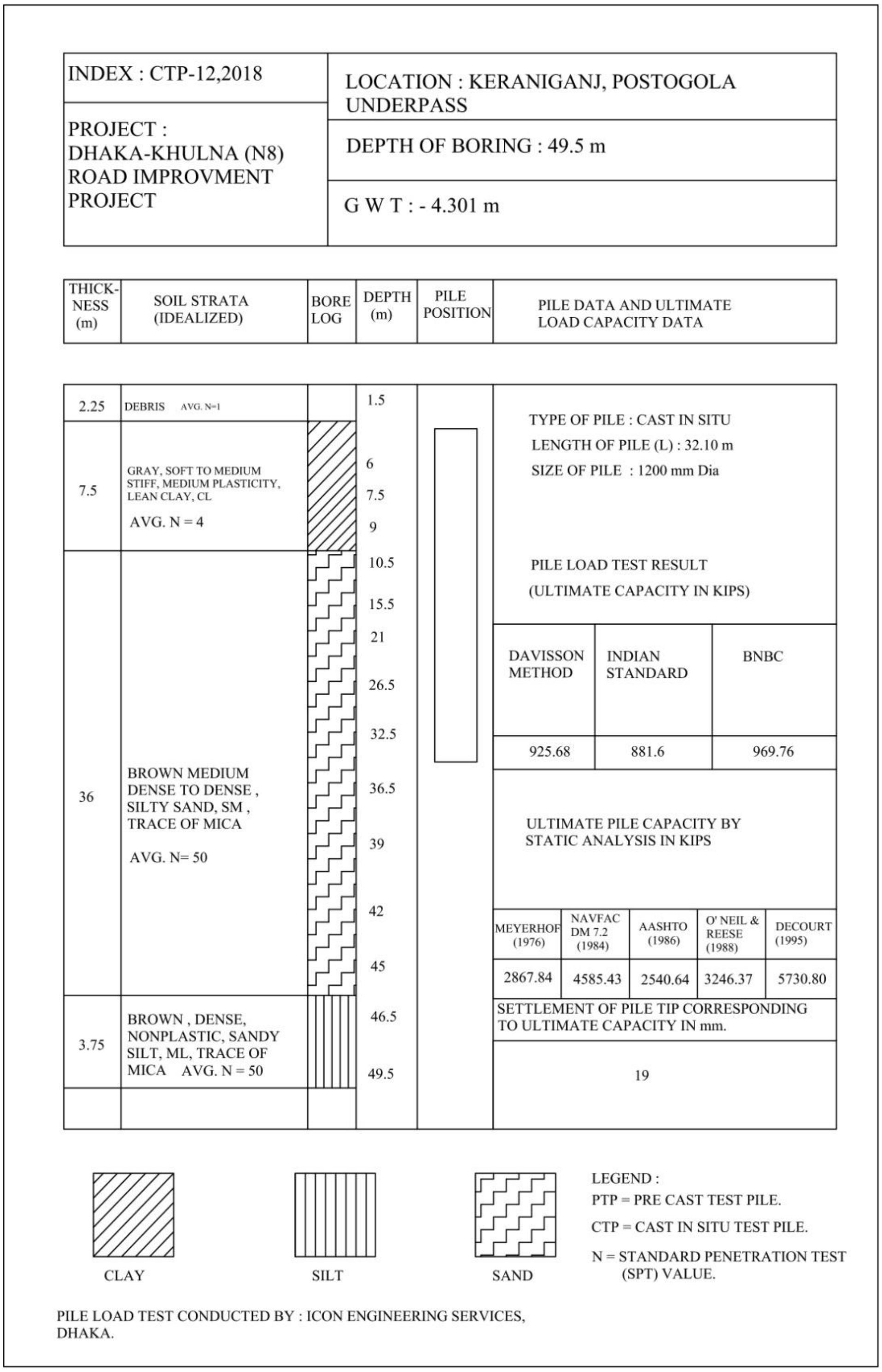
▥ SILT

▨ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-14



INDEX : CTP-12,2018	LOCATION : KERANIGANJ, POSTOGOLA UNDERPASS
PROJECT : DHAKA-KHULNA (N8) ROAD IMPROVMENT PROJECT	DEPTH OF BORING : 49.5 m
	G W T : - 4.301 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
2.25	DEBRIS AVG. N=1		1.5	
7.5	GRAY, SOFT TO MEDIUM STIFF, MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N = 4	▨	6 7.5 9	
36	BROWN MEDIUM DENSE TO DENSE , SILTY SAND, SM , TRACE OF MICA AVG. N= 50		10.5 15.5 21 26.5 32.5 36.5 39 42 45	
3.75	BROWN , DENSE, NONPLASTIC, SANDY SILT, ML, TRACE OF MICA AVG. N = 50		46.5 49.5	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 32.10 m
SIZE OF PILE : 1200 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
925.68	881.6	969.76

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
2867.84	4585.43	2540.64	3246.37	5730.80

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

19

CLAY

SILT

SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

CTP-15

INDEX : CTP-08,2018	LOCATION : ABUTMENT -1, KUMAR BRIDGE
PROJECT : DHAKA-KHULNA (N8) ROAD IMPROVMENT PROJECT	DEPTH OF BORING : 61.5 m G W T : - 0.584 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
2.25	DEBRIS AVG. N=1		1.5	
6	BROWN TO GRAY, SOFT TO MEDIUM STIFF, MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N = 4	▨	3 6 7.5	
3	GRAY, LOOSE , NONPLASTIC, SILT, ML, TRACE OF MICA AVG. N = 9	▥	9 10.5	
3	GRAY, LOOSE TO MEDIUM DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N =9	▦	12 13.5	
3	GRAY, SOFT , HIGH PLASTICITY, FAT CLAY, CH AVG. N = 4	▨	15 16.5	
44.25	GRAY, MEDIUM DENSE TO DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N = 34	▦	18 29.5 35.2 38.5 41.2 45.8 51.7 57.5 61.5	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : CAST IN SITU
LENGTH OF PILE (L) : 51.90 m
SIZE OF PILE : 1200 mm Dia

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
2644.8	1653	2027.68

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	NAVFAC DM 7.2 (1984)	AASHTO (1986)	O' NEIL & REESE (1988)	DECOURT (1995)
2922.86	4655.41	2948.87	5821.31	5826.94

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

36

▨ CLAY
▥ SILT
▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

PTP-01

INDEX : PTP -12, DEC ,1997	LOCATION : RAJSHAHI
PROJECT : RAJSHAHI DEVELOPMENT AUTHORITY BHABAN	DEPTH OF BORING : 18.6 m G W T : - 3.0 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
6	MEDIUM STIFF TO STIFF CLAYEY SILT AVG. N = 6	▥	0 1 2 3 4 5	
2	LOOSE SANDY SILT AVG. N=10	▥	6 7 8	
1.67	MEDIUM DENSE SILT FINE SAND AVG. N=12	▦	9	
2	DENSE SANDY SILT AVG. N=22	▥	10 11 12	
2.6	MEDIUM DENSE SILTY SAND AVG. N = 12	▦	13 14	
4.33	STIFF SILTY CLAY AVG. N = 10	▨	15 18	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CASE
LENGTH OF PILE (L) : 10.6 m
SIZE OF PILE : 300 mm X 300 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
242.44	245.01	264.48

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
149.31	136.65	159.69	102.22	178.50

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

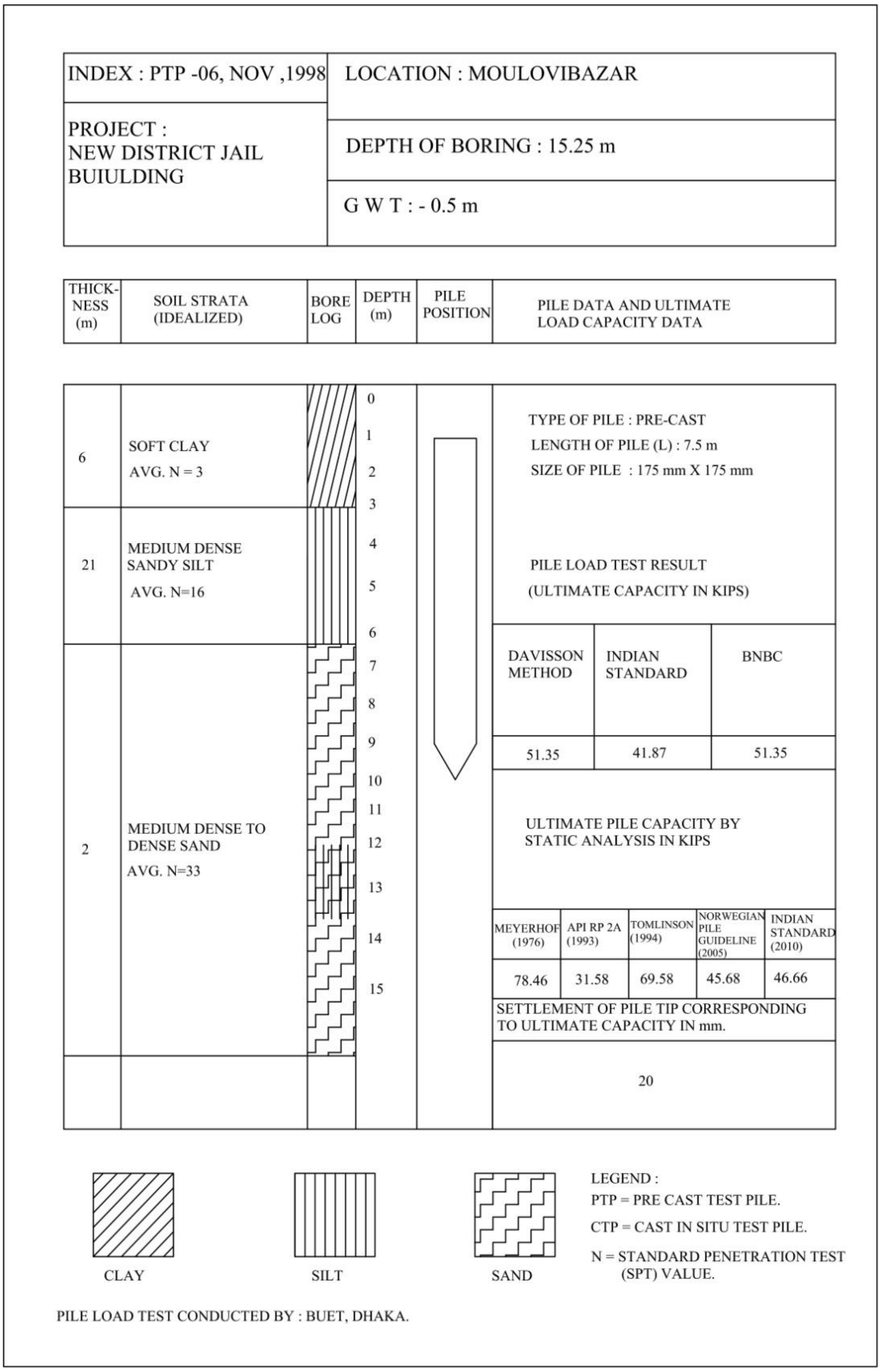
11.5

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-02



INDEX : PTP -06, NOV ,1998

LOCATION : MOULOVIBAZAR

PROJECT :
NEW DISTRICT JAIL
BUIULDING

DEPTH OF BORING : 15.25 m

G W T : - 0.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
6	SOFT CLAY AVG. N = 3		0 1 2 3	
21	MEDIUM DENSE SANDY SILT AVG. N=16		4 5 6	
2	MEDIUM DENSE TO DENSE SAND AVG. N=33		7 8 9 10 11 12 13 14 15	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 7.5 m
SIZE OF PILE : 175 mm X 175 mm

PILE LOAD TEST RESULT (ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
51.35	41.87	51.35

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
78.46	31.58	69.58	45.68	46.66

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

20

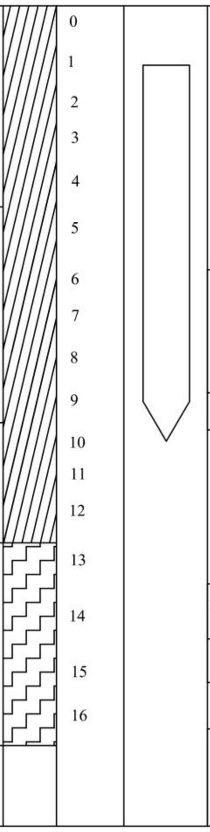
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-03

INDEX : PTP -02, JAN ,1998	LOCATION : NARAIL
PROJECT : COURT BUILDING	DEPTH OF BORING : 16.75 m G W T : - 2.95 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
4.5	MEDIUM STIFF CLAY AVG. N = 5				
5	VERY SOFT TO SOFT SILTY CLAY AVG. N=4				
4	DARK ORGANIC CLAY AVG. N=4				
3.25	LOOSE TO MEDIUM DENSE SILTY SAND AVG. N=12				



TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 7 m
SIZE OF PILE : 175 mm X 175 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
24.9	18	20

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
17.25	23.94	30.98	26.90	30.67

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

51

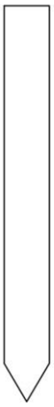
▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-04

INDEX : PTP -07, JAN ,2000	LOCATION : GOPALGANJ
PROJECT : DISTRICT JAIL	DEPTH OF BORING : 15.25 m G W T : -0.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
5.5	VERY SOFT DARK SILTY CLAY AVG. N = 3	▨	0 1 2 3 4 5	
2.5	SOFT CLAYEY SILT AVG. N=3	▥	6 7 8	
2	LOOSE FINE SAND AVG. N=7	▦	9	
2 2.6	MEDIUM STIFF CLAY AVG. N = 5	▥	10 11 12 13 14	
4.33	MEDIUM DENSE FINE SAND AVG. N = 17	▦	15	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CASE
LENGTH OF PILE (L) : 7.5 m
SIZE OF PILE : 175 mm X 175 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
20.94	17.25	21

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
17.29	22.94	25.58	27.06	23.79

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

19

▨ CLAY ▥ SILT ▦ SAND

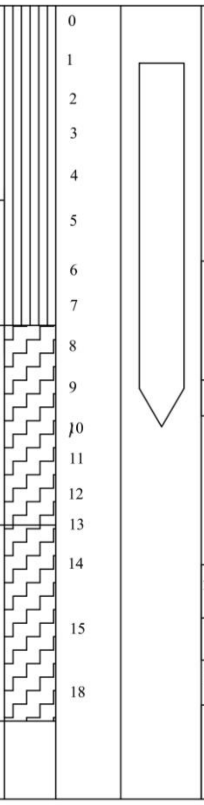
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-05

INDEX : PTP -01, MAR ,2000	LOCATION : SAVAR, DHAKA
PROJECT : INTERNATIONAL TRAINING COMPLEX, BPATC	DEPTH OF BORING : 18 m G W T : - 3 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
3	MEDIUM STIFF CLAYEY SILT AVG. N = 6				
4.5	MEDIUM STIFF TO STIFF CLAYEY SILT AVG. N=9				
5.5	MEDIUM DENSE SAND AVG. N=16				
5	MEDIUM DENSE TO DENSE SAND AVG. N=24				



TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 12 m
SIZE OF PILE : 300mm X 300 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

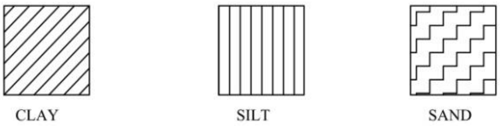
DAVISSON METHOD	INDIAN STANDARD	BNBC
374.68	376	395

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
164.58	189.11	206.12	156.19	180.27

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

11



LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-06

INDEX : PTP -09, MAR ,2000	LOCATION : ALAMPUR, SYLHET
PROJECT : DIVISIONAL HEADQUARTERS	DEPTH OF BORING : 18 m
	G W T : - 2 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
5	SOFT TO MEDIUM STIFF CLAY AVG. N = 4	▨	0 1 2 3 4	
4	VERY LOOSE FINE SAND AVG. N=8	▦	5 6 7 8 9	
5	BLACKISH MEDIUM STIFF TO STIFF CLAY AVG. N=8	▨	10 11 12 13 14	
5	MEDIUM DENSE TO DENSE SANDY SILT AVG. N=25	▥	15 18	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CAST

LENGTH OF PILE (L) : 7 m

SIZE OF PILE : 175mm X 175mm

PILE LOAD TEST RESULT (ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
33.06	25	28

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
23.30	30.79	29.18	33.56	33.46

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

43

▨ CLAY ▥ SILT ▦ SAND

LEGEND :

PTP = PRE CAST TEST PILE.

CTP = CAST IN SITU TEST PILE.

N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-07

INDEX : PTP -13, DEC ,2000	LOCATION : BHAGGAKUL, SREENAGAR, MUNSHIGANJ.
PROJECT : SHISHU PARIBAR, MUNSHIGANJ	DEPTH OF BORING : 25.5 m G W T : - 1.5 m

THICKNESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
5	SOFT SILTY CLAY AVG. N = 3	▨	0 2 4	
4	MEDIUM STIFF SILT AVG. N=7	▥	6 8	
6	MEDIUM DENSE SILTY SAND AVG. N=14	▦	10 12 14	
10.5	LOOSE TO MEDIUM DENSE SANDY SILT AVG. N=13	▥	16 18 20 22 24	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 12 m
SIZE OF PILE : 350 mm X 350 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
143.26	125	138

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
172.09	146.53	169.82	137.73	174.33

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

38

▨	▥	▦
CLAY	SILT	SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-08

INDEX : PTP -15, MAR ,2002	LOCATION : PATUAKHALI
PROJECT : TECHNICAL TRAINING CENTRE	DEPTH OF BORING : 18 m G W T : - 1.20 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
6	SOFT TO MEDIUM STIFF CLAY AVG. N = 4	▨	0 1 2 3 4 5 6		TYPE OF PILE : PRE-CAST LENGTH OF PILE (L) : 7.5 m SIZE OF PILE : 300 mm X 300 mm
2.5	LOOSE TO MEDIUM DENSE SAND AVG. N=12	SAND	7 8 9		
9	MEDIUM DENSE TO DENSE SAND AVG. N=22	SAND	10 11 12 13 14 15 18		

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
121.22	133.14	140.24

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
140.03	74.71	127.30	78.79	125.85

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

6.9

▨	▥	▦
CLAY	SILT	SAND

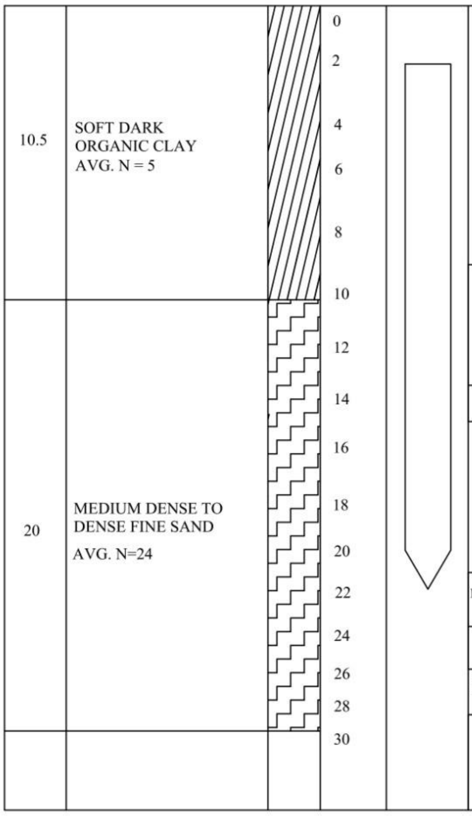
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-09

INDEX : PTP -10, NOV ,2001	LOCATION : KHULNA
PROJECT : IMAM TRAINING CENTRE	DEPTH OF BORING : 30.5 m
	G W T : - 0.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA



TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 15.5 m
SIZE OF PILE : 350 mm X 350 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
240.23	220.4	253.46

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
234.54	205.61	275.54	184.91	192.88

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

14

▨ CLAY

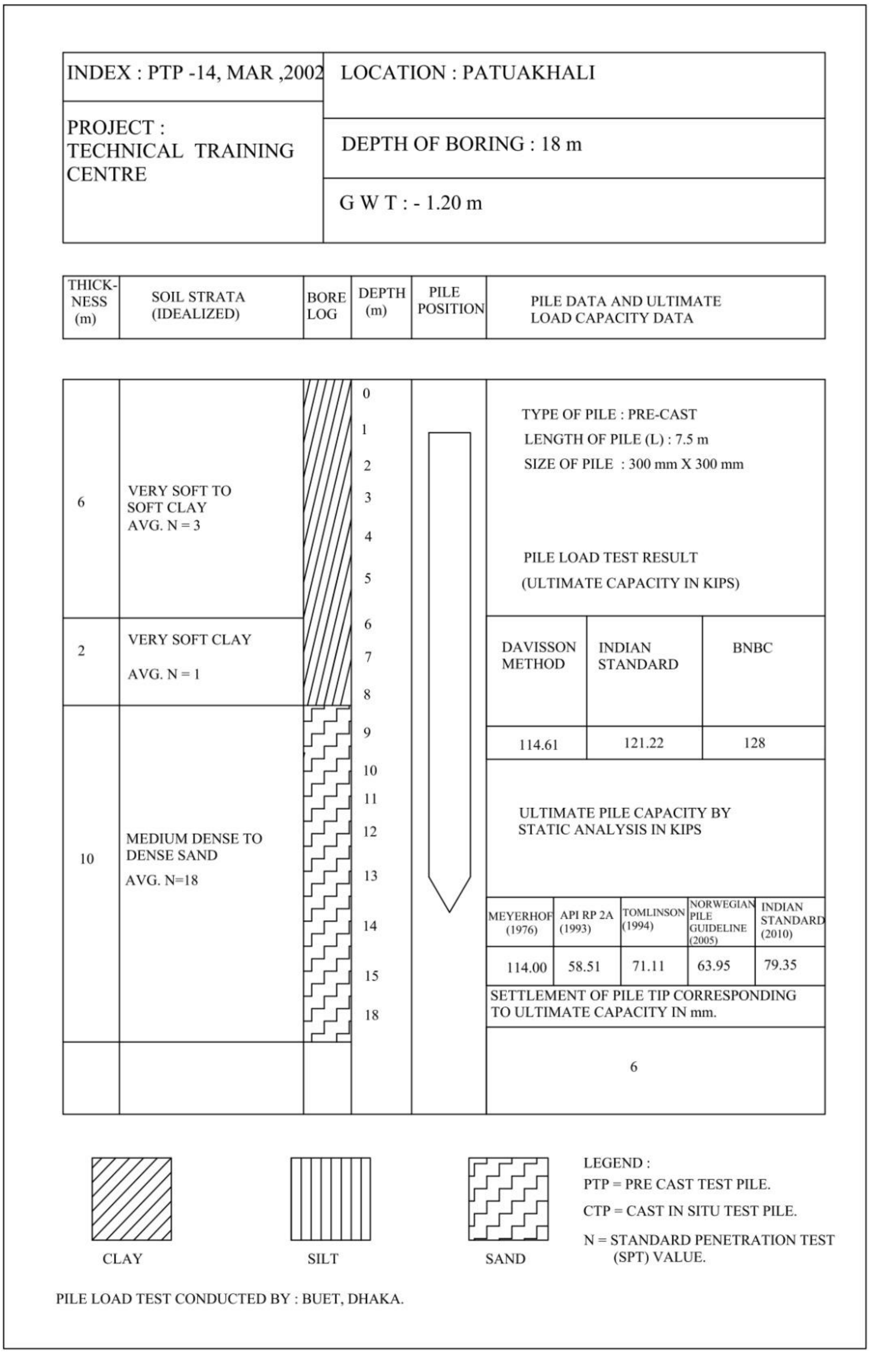
▥ SILT

▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-10



INDEX : PTP -14, MAR ,2002	LOCATION : PATUAKHALI
PROJECT : TECHNICAL TRAINING CENTRE	DEPTH OF BORING : 18 m
	G W T : - 1.20 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION
6	VERY SOFT TO SOFT CLAY AVG. N = 3	▨	0 1 2 3 4 5	
2	VERY SOFT CLAY AVG. N = 1	▨	6 7 8	
10	MEDIUM DENSE TO DENSE SAND AVG. N=18		9 10 11 12 13 14 15 18	

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 7.5 m
SIZE OF PILE : 300 mm X 300 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
114.61	121.22	128

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
114.00	58.51	71.11	63.95	79.35

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

6

▨ CLAY

▥ SILT

▦ SAND

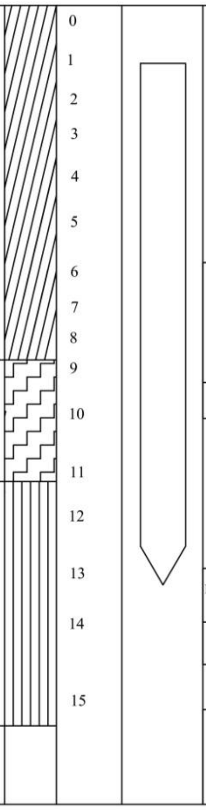
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-11

INDEX : PTP 08, DEC ,2001	LOCATION : JHALOKATHY
PROJECT : DISTRICT REGISTER AND SUB-REGISTER OFFICE	DEPTH OF BORING : 15.25 m G W T : - 1.5 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
9.5	SOFT SILTY CLAY AVG. N = 3				
2	VERY LOOSE FINE SAND AVG. N = 6				
3.75	CLAYEY SILT AVG. N=5				



TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 9.20 m
SIZE OF PILE : 400 mm X 400 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
55.1	60	68

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
71.53	84.30	101.94	91.13	105.29

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

1.85

▨ CLAY

▥ SILT

▦ SAND

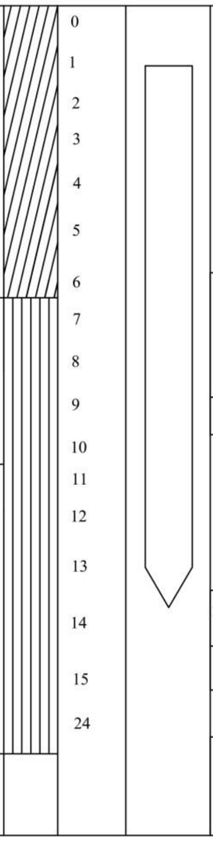
LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-12

INDEX : PTP -11, JUN ,2002	LOCATION : AGARGAON, S.B NAGAR, DHAKA
PROJECT : PRESS CUM- IMAM TRAINING BUILDING OF ISLAMIC FOUNDATION	DEPTH OF BORING : 24 m G W T :

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)	PILE POSITION	PILE DATA AND ULTIMATE LOAD CAPACITY DATA
6.5	STIFF TO VERY STIFF CLAY AVG. N = 14				
4	MEDIUM DENSE TO DENSE SAND AVG. N = 34				
13.5	DENSE TO VERY DENSE SAND AVG. N > 50				



TYPE OF PILE : PRE-CAST
LENGTH OF PILE (L) : 9.15 m
SIZE OF PILE : 300 mm X 300 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
308.56	275.5	315.31

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
231.82	216.11	276.32	206.48	216.27

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

18

▨ CLAY

▥ SILT

▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : BUET, DHAKA.

PTP-13

INDEX : PTP -05,2014	LOCATION : SECTOR-18, UTTARA,DHAKA
PROJECT : PROPOSED CONSTRUCTION OF RESIDENTIAL APARTMENT PROJECT FOR LOW & MIDDLE INCOME GROUP OF PEOPLE	DEPTH OF BORING : 33 m G W T : - 3.5 m

THICKNESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
6.75	GRAY, SOFT, HIGH PLASTICITY,FAT CLAY,CH AVG. N = 6	▨	1.5 3.0 4.5 5 6
7.5	LOOSE SANDY SILT AVG. N=8	▥	7.5 10.5 12 13.5
6	MEDIUM DENSE SILT FINE SAND AVG. N=8	▨	15 16.5 18 21
9.75	MID DENSE SANDY SILT AVG. N=20	▥	25 27
3	DENSE SANDY SILT AVG. N=42	▥	30 33

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CASE
LENGTH OF PILE (L) : 30.5 m
SIZE OF PILE : 400 mm X400 mm

PILE LOAD TEST RESULT (ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
727.32	484	650

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
647.87	859.27	750.74	650.30	549.78

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

40

▨ CLAY

▥ SILT

▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

PTP-14

INDEX : PTP -04,2014	LOCATION : SECTOR-18, UTTARA,DHAKA
PROJECT : PROPOSED CONSTRUCTION OF RESIDENTIAL APARTMENT PROJECT FOR LOW & MIDDLE INCOME GROUP OF PEOPLE	DEPTH OF BORING : 33 m G W T : - 1.0 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
2.25	GRAY, LOOSE , SILTY SAND, SM, TRACE OF MICA AVG. N = 1	▦	1.5
3	GRAY , SOFT , MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N = 2	▨	3.0 4.5
4.5	GRAY, NONPLASTIC, SILT, ML, TRACE OF MICA AVG. N=9	▥	6 7.5 9
6	GRAY, SOFT OT MEDIUM , MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N=5	▨	10.5 15
12.75	BROWN, MID DENSE SILTY SAND, SM, TRACE OF MICA AVG. N=17	▦	16.5 18 21 25 27
4.5	BROWN, DENSE, SILTY SAND, SM, TRACE OF MICA AVG. N=45	▦	30 33

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CASE
LENGTH OF PILE (L) : 30.5 m
SIZE OF PILE : 400 mm X 400 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
859.56	462.84	617.12

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
646.30	897.93	702.11	661.27	603.81

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

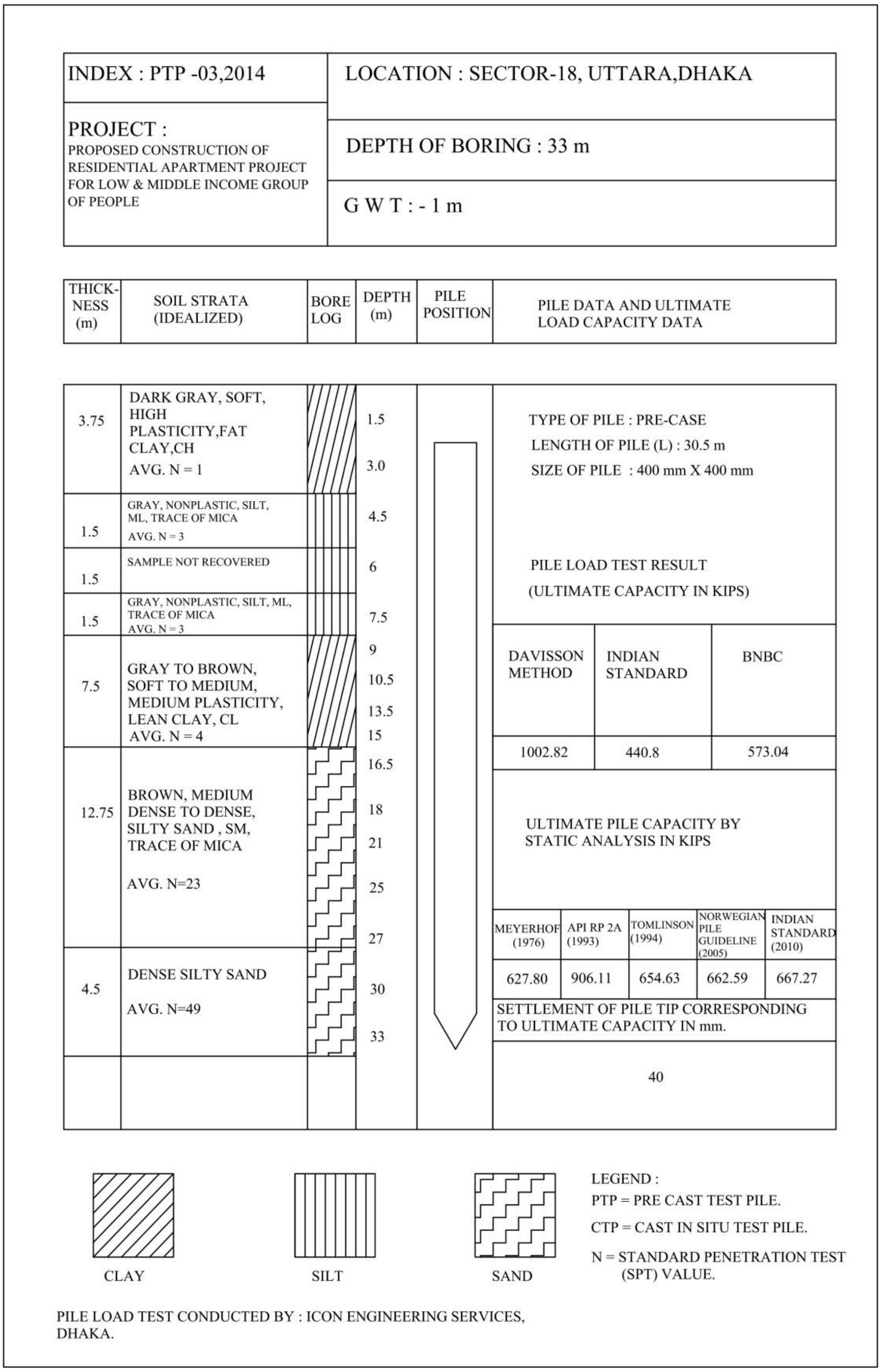
37

▨ CLAY
▥ SILT
▦ SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

PTP-15



INDEX : PTP -03,2014	LOCATION : SECTOR-18, UTTARA,DHAKA
PROJECT : PROPOSED CONSTRUCTION OF RESIDENTIAL APARTMENT PROJECT FOR LOW & MIDDLE INCOME GROUP OF PEOPLE	DEPTH OF BORING : 33 m G W T : - 1 m

THICK-NESS (m)	SOIL STRATA (IDEALIZED)	BORE LOG	DEPTH (m)
3.75	DARK GRAY, SOFT, HIGH PLASTICITY,FAT CLAY,CH AVG. N = 1	▨	1.5 3.0
1.5	GRAY, NONPLASTIC, SILT, ML, TRACE OF MICA AVG. N = 3	▥	4.5
1.5	SAMPLE NOT RECOVERED	▥	6
1.5	GRAY, NONPLASTIC, SILT, ML, TRACE OF MICA AVG. N = 3	▥	7.5
7.5	GRAY TO BROWN, SOFT TO MEDIUM, MEDIUM PLASTICITY, LEAN CLAY, CL AVG. N = 4	▨	9 10.5 13.5 15
12.75	BROWN, MEDIUM DENSE TO DENSE, SILTY SAND , SM, TRACE OF MICA AVG. N=23		16.5 18 21 25 27
4.5	DENSE SILTY SAND AVG. N=49		30 33

PILE POSITION

PILE DATA AND ULTIMATE LOAD CAPACITY DATA

TYPE OF PILE : PRE-CASE
LENGTH OF PILE (L) : 30.5 m
SIZE OF PILE : 400 mm X 400 mm

PILE LOAD TEST RESULT
(ULTIMATE CAPACITY IN KIPS)

DAVISSON METHOD	INDIAN STANDARD	BNBC
1002.82	440.8	573.04

ULTIMATE PILE CAPACITY BY STATIC ANALYSIS IN KIPS

MEYERHOF (1976)	API RP 2A (1993)	TOMLINSON (1994)	NORWEGIAN PILE GUIDELINE (2005)	INDIAN STANDARD (2010)
627.80	906.11	654.63	662.59	667.27

SETTLEMENT OF PILE TIP CORRESPONDING TO ULTIMATE CAPACITY IN mm.

40

CLAY

SILT

SAND

LEGEND :
PTP = PRE CAST TEST PILE.
CTP = CAST IN SITU TEST PILE.
N = STANDARD PENETRATION TEST (SPT) VALUE.

PILE LOAD TEST CONDUCTED BY : ICON ENGINEERING SERVICES, DHAKA.

APPENDIX B

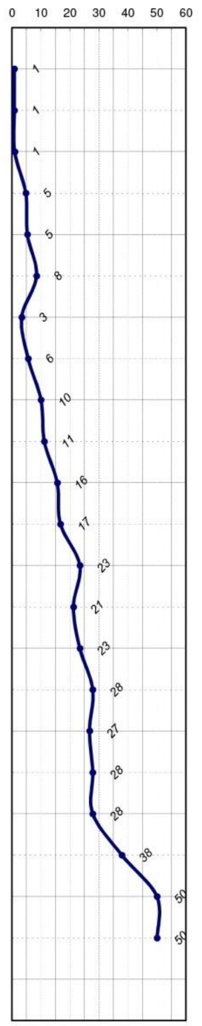
SOIL BORE LOG OF CTP AND PTP

Bore Log data for Driven Pile-1

Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People		**BORE LOG**
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.47
Co-ordinates(m):	23.85512N 90.35195E	Date Started:	12-Jun-14
Borehole No:	**1** (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.97	3.75	Gray, soft, medium plsaticity, ***Lean CLAY, CL***	▧	1.5						**1**
3.0	D-2	▩	25.47				3.0					33	**1**
4.5	D-3	▩	23.97	6	Gray, nonplasticity, ***SILT, ML,*** trace of mica	░	4.5	99					**1**
6.0	D-4	▩	22.47				6.0						**5**
7.5	D-5	▩	20.97				7.5						**5**
9.0	D-6	▩	19.47				9.0						**8**
10.5	D-7	▩	17.97	6	Gray, soft to stiff, high plasticity, **Fat CLAY, CH**	▧	10.5	99	66	44		27	**3**
12.0	D-8	▩	16.47				12.0					17	**6**
13.5	D-9	▩	14.97				13.5						**10**
15.0	D-10	▩	13.47				15.0						**11**
16.5	D-11	▩	11.97	17.25	Brown, medium dense to dense, Silty ***SAND, SM,*** trace of mica	░	16.5						**16**
18.0	D-12	▩	10.47				18.0						**17**
19.5	D-13	▩	8.97				19.5						**23**
21.0	D-14	▩	7.47				21.0						**21**
22.5	D-15	▩	5.97				22.5						**23**
24.0	D-16	▩	4.47				24.0						**28**
25.5	D-17	▩	2.97				25.5						**27**
27.0	D-18	▩	1.47				27.0	44					**28**
28.5	D-19	▩	(0.03)				28.5						**28**
30.0	D-20	▩	(1.53)				30.0						**38**
31.5	D-21	▩	(3.03)				31.5						**50**
33.0	D-22	▩	(4.53)				33.0						**50**
					End of BH RL (m) (4.53)								

Graphical Representation of Corrected SPT N-Value, N70



Legend:

▩ Split Spoon Sample

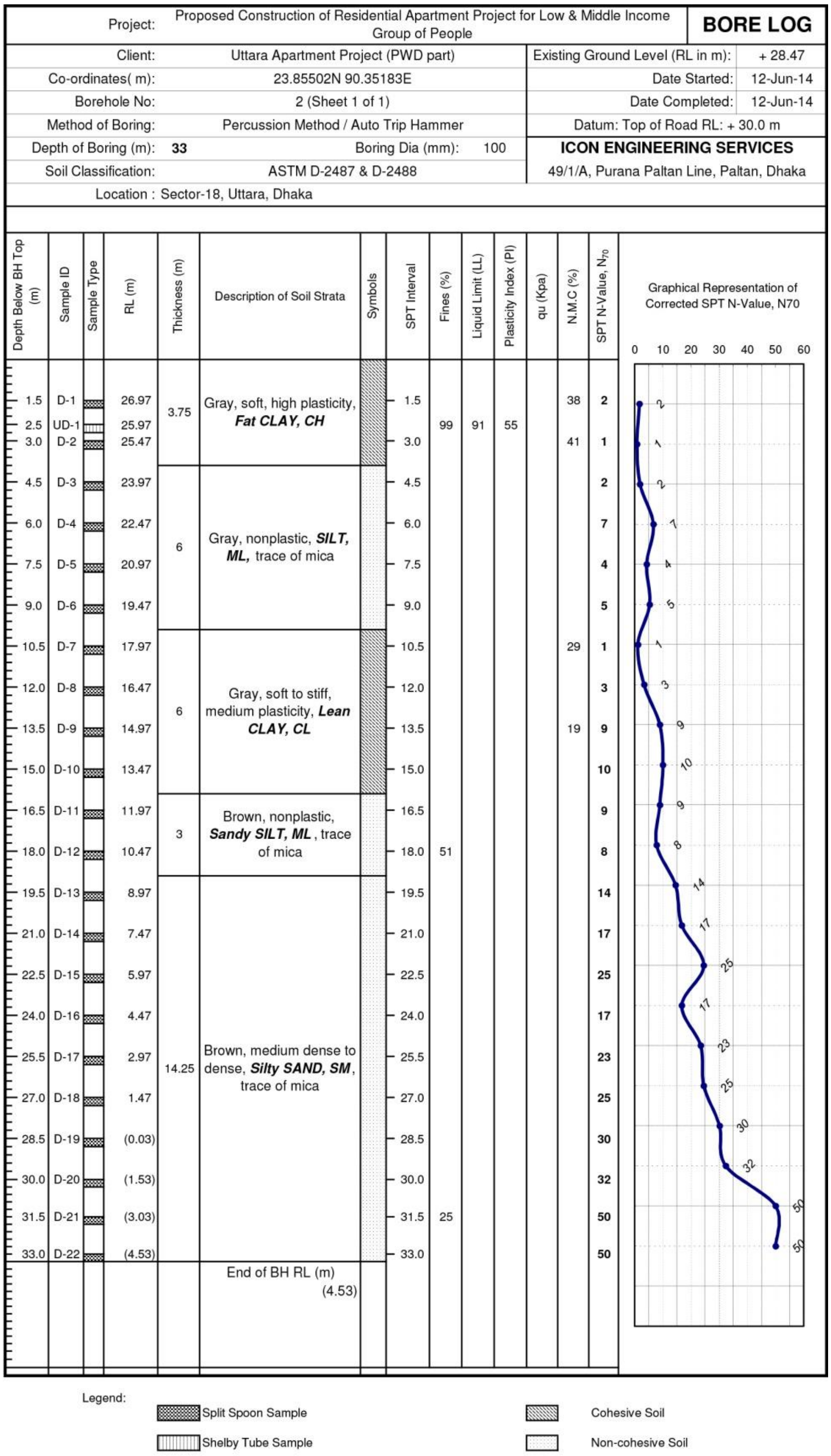
▥ Shelby Tube Sample

▧ Cohesive Soil

░ Non-cohesive Soil

Nahi

Bore Log Data for Driven Pile-2



BORE LOG

Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People		
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.47
Co-ordinates(m):	23.85502N 90.35183E	Date Started:	12-Jun-14
Borehole No:	2 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.97	3.75	Gray, soft, high plasticity, ***Fat CLAY, CH***	▨	1.5					38	**2**
2.5	UD-1	▥	25.97					99	91	55			
3.0	D-2	▩	25.47				3.0					41	**1**
4.5	D-3	▩	23.97	6	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	4.5						**2**
6.0	D-4	▩	22.47				6.0						**7**
7.5	D-5	▩	20.97				7.5						**4**
9.0	D-6	▩	19.47				9.0						**5**
10.5	D-7	▩	17.97	6	Gray, soft to stiff, medium plasticity, ***Lean CLAY, CL***	▨	10.5					29	**1**
12.0	D-8	▩	16.47				12.0						**3**
13.5	D-9	▩	14.97				13.5					19	**9**
15.0	D-10	▩	13.47				15.0						**10**
16.5	D-11	▩	11.97	3	Brown, nonplastic, ***Sandy SILT, ML***, trace of mica	░	16.5						**9**
18.0	D-12	▩	10.47				18.0	51					**8**
19.5	D-13	▩	8.97	14.25	Brown, medium dense to dense, ***Silty SAND, SM***, trace of mica	░	19.5						**14**
21.0	D-14	▩	7.47				21.0						**17**
22.5	D-15	▩	5.97				22.5						**25**
24.0	D-16	▩	4.47				24.0						**17**
25.5	D-17	▩	2.97				25.5						**23**
27.0	D-18	▩	1.47				27.0						**25**
28.5	D-19	▩	(0.03)				28.5						**30**
30.0	D-20	▩	(1.53)				30.0						**32**
31.5	D-21	▩	(3.03)				31.5	25					**50**
33.0	D-22	▩	(4.53)				33.0						**50**
					End of BH RL (m) (4.53)								

Legend:

▩ Split Spoon Sample

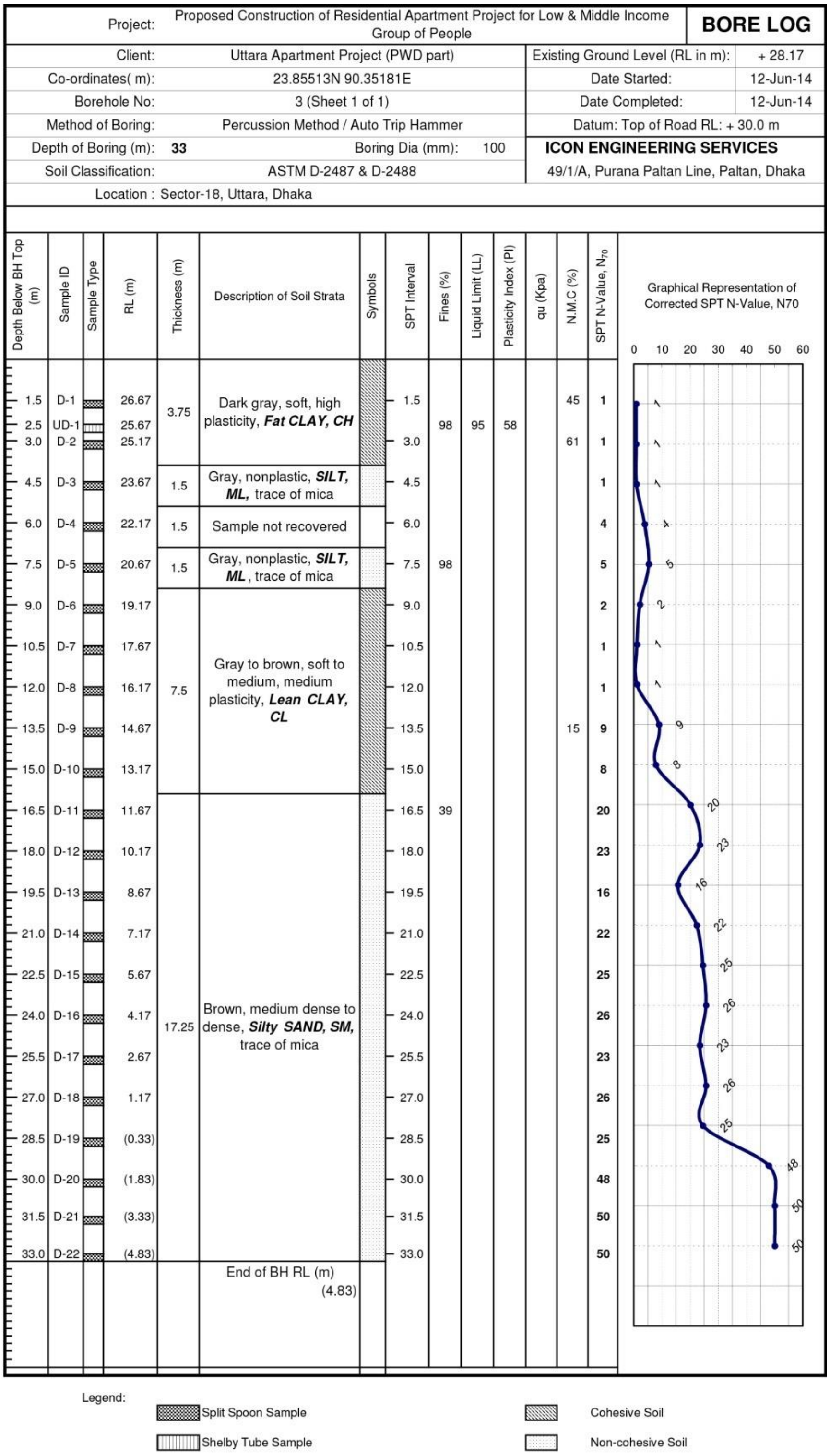
▥ Shelby Tube Sample

▨ Cohesive Soil

░ Non-cohesive Soil

Nahi

Bore Log Data for Driven Pile-3



BORE LOG

Project: Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People

Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.17
Co-ordinates(m):	23.85513N 90.35181E	Date Started:	12-Jun-14
Borehole No:	3 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Location : Sector-18, Uttara, Dhaka

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.67	3.75	Dark gray, soft, high plasticity, ***Fat CLAY, CH***	▨	1.5					45	**1**
2.5	UD-1	▥	25.67					98	95	58			
3.0	D-2	▩	25.17				3.0					61	**1**
4.5	D-3	▩	23.67	1.5	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	4.5						**1**
6.0	D-4	▩	22.17	1.5	Sample not recovered		6.0						**4**
7.5	D-5	▩	20.67	1.5	Gray, nonplastic, ***SILT, ML***, trace of mica	░	7.5	98					**5**
9.0	D-6	▩	19.17	7.5	Gray to brown, soft to medium, medium plasticity, ***Lean CLAY, CL***	▨	9.0						**2**
10.5	D-7	▩	17.67				10.5						**1**
12.0	D-8	▩	16.17				12.0						**1**
13.5	D-9	▩	14.67				13.5					15	**9**
15.0	D-10	▩	13.17				15.0						**8**
16.5	D-11	▩	11.67	17.25	Brown, medium dense to dense, ***Silty SAND, SM,*** trace of mica	░	16.5	39					**20**
18.0	D-12	▩	10.17				18.0						**23**
19.5	D-13	▩	8.67				19.5						**16**
21.0	D-14	▩	7.17				21.0						**22**
22.5	D-15	▩	5.67				22.5						**25**
24.0	D-16	▩	4.17				24.0						**26**
25.5	D-17	▩	2.67				25.5						**23**
27.0	D-18	▩	1.17				27.0						**26**
28.5	D-19	▩	(0.33)				28.5						**25**
30.0	D-20	▩	(1.83)				30.0						**48**
31.5	D-21	▩	(3.33)				31.5						**50**
33.0	D-22	▩	(4.83)				33.0						**50**
					End of BH RL (m) (4.83)								

Graphical Representation of Corrected SPT N-Value, N70

Legend:

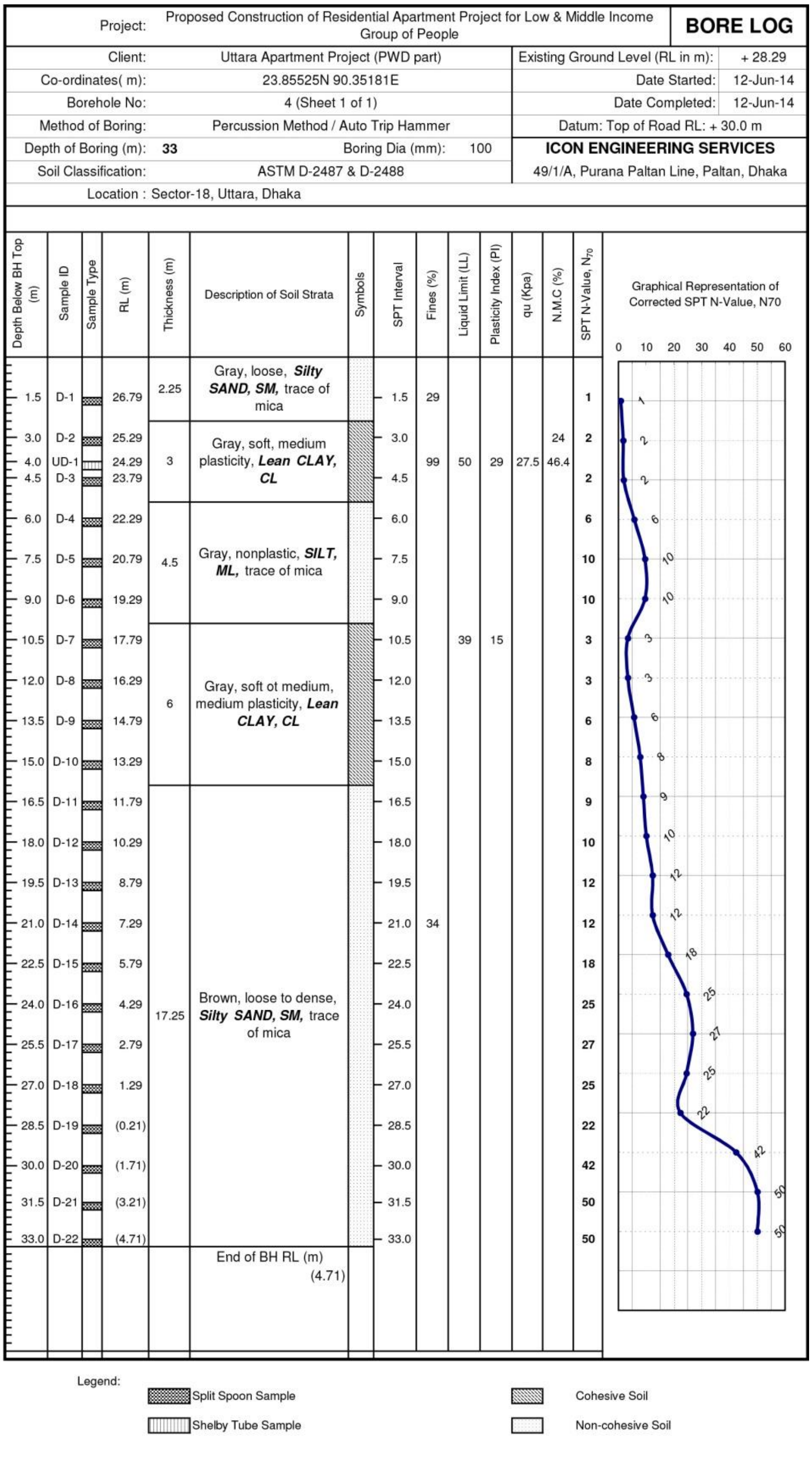
▩ Split Spoon Sample

▥ Shelby Tube Sample

▨ Cohesive Soil

░ Non-cohesive Soil

Bore Log Data for Driven Pile-4



BORE LOG

Project: Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People

Client: Uttara Apartment Project (PWD part)

Existing Ground Level (RL in m): + 28.29

Co-ordinates(m): 23.85525N 90.35181E

Date Started: 12-Jun-14

Borehole No: 4 (Sheet 1 of 1)

Date Completed: 12-Jun-14

Method of Boring: Percussion Method / Auto Trip Hammer

Datum: Top of Road RL: + 30.0 m

Depth of Boring (m): **33**

Boring Dia (mm): 100

ICON ENGINEERING SERVICES
49/1/A, Purana Paltan Line, Paltan, Dhaka

Soil Classification: ASTM D-2487 & D-2488

Location : Sector-18, Uttara, Dhaka

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.79	2.25	Gray, loose, ***Silty SAND, SM,*** trace of mica	░	1.5	29					**1**
3.0	D-2	▩	25.29	3	Gray, soft, medium plasticity, ***Lean CLAY, CL***	▨	3.0					24	**2**
4.0	UD-1	▥	24.29					99	50	29	27.5	46.4	
4.5	D-3	▩	23.79				4.5						**2**
6.0	D-4	▩	22.29	4.5	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	6.0						**6**
7.5	D-5	▩	20.79				7.5						**10**
9.0	D-6	▩	19.29				9.0						**10**
10.5	D-7	▩	17.79	6	Gray, soft ot medium, medium plasticity, ***Lean CLAY, CL***	▨	10.5		39	15			**3**
12.0	D-8	▩	16.29				12.0						**3**
13.5	D-9	▩	14.79				13.5						**6**
15.0	D-10	▩	13.29				15.0						**8**
16.5	D-11	▩	11.79	17.25	Brown, loose to dense, ***Silty SAND, SM,*** trace of mica	░	16.5						**9**
18.0	D-12	▩	10.29				18.0						**10**
19.5	D-13	▩	8.79				19.5						**12**
21.0	D-14	▩	7.29				21.0	34					**12**
22.5	D-15	▩	5.79				22.5						**18**
24.0	D-16	▩	4.29				24.0						**25**
25.5	D-17	▩	2.79				25.5						**27**
27.0	D-18	▩	1.29				27.0						**25**
28.5	D-19	▩	(0.21)				28.5						**22**
30.0	D-20	▩	(1.71)				30.0						**42**
31.5	D-21	▩	(3.21)				31.5						**50**
33.0	D-22	▩	(4.71)				33.0						**50**
					End of BH RL (m) (4.71)								

Legend:

▩ Split Spoon Sample

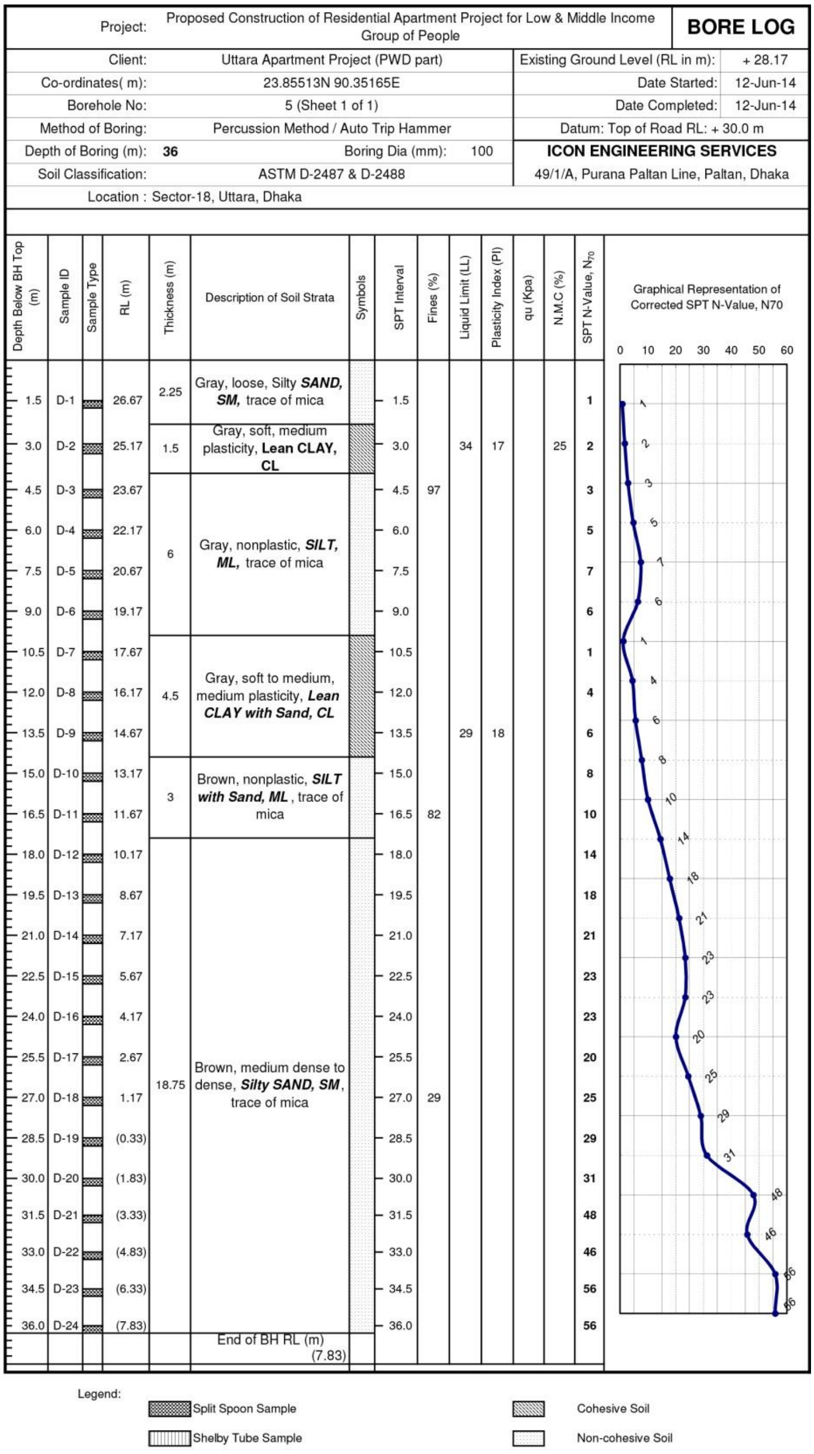
▥ Shelby Tube Sample

▨ Cohesive Soil

░ Non-cohesive Soil

Nahu

Bore Log Data for Driven Pile-5



BORE LOG

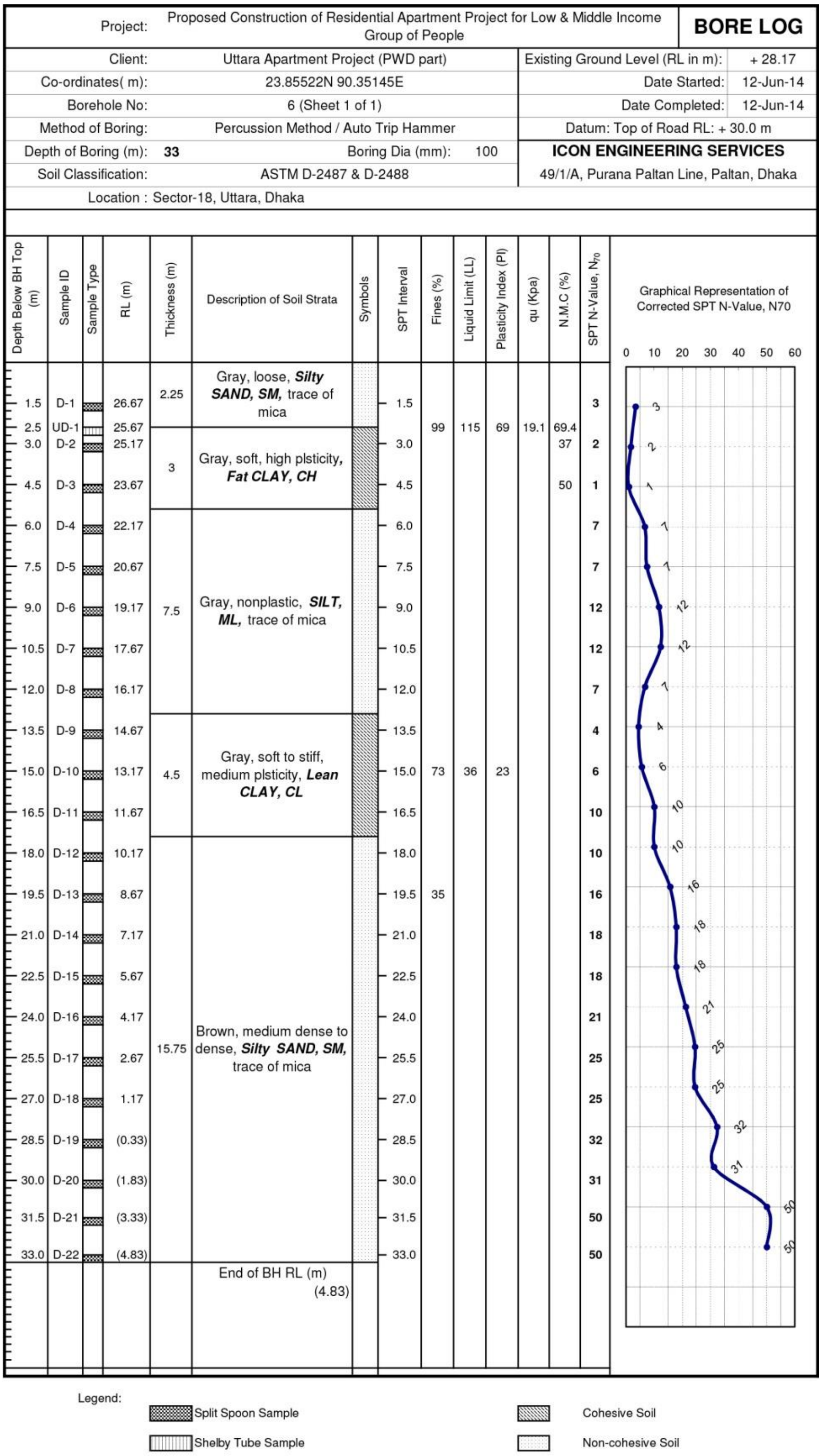
Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People		
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.17
Co-ordinates(m):	23.85513N 90.35165E	Date Started:	12-Jun-14
Borehole No:	5 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**36** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.67	2.25	Gray, loose, Silty ***SAND, SM,*** trace of mica	░	1.5						**1**
3.0	D-2	▩	25.17	1.5	Gray, soft, medium plasticity, **Lean CLAY, CL**	▨	3.0		34	17		25	**2**
4.5	D-3	▩	23.67	6	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	4.5	97					**3**
6.0	D-4	▩	22.17				6.0						**5**
7.5	D-5	▩	20.67				7.5						**7**
9.0	D-6	▩	19.17				9.0						**6**
10.5	D-7	▩	17.67	4.5	Gray, soft to medium, medium plasticity, ***Lean CLAY with Sand, CL***	▨	10.5						**1**
12.0	D-8	▩	16.17				12.0						**4**
13.5	D-9	▩	14.67				13.5		29	18			**6**
15.0	D-10	▩	13.17	3	Brown, nonplastic, ***SILT with Sand, ML*** , trace of mica	░	15.0						**8**
16.5	D-11	▩	11.67				16.5	82					**10**
18.0	D-12	▩	10.17	18.75	Brown, medium dense to dense, ***Silty SAND, SM***, trace of mica	░	18.0						**14**
19.5	D-13	▩	8.67				19.5						**18**
21.0	D-14	▩	7.17				21.0						**21**
22.5	D-15	▩	5.67				22.5						**23**
24.0	D-16	▩	4.17				24.0						**23**
25.5	D-17	▩	2.67				25.5						**20**
27.0	D-18	▩	1.17				27.0	29					**25**
28.5	D-19	▩	(0.33)				28.5						**29**
30.0	D-20	▩	(1.83)				30.0						**31**
31.5	D-21	▩	(3.33)				31.5						**48**
33.0	D-22	▩	(4.83)				33.0						**46**
34.5	D-23	▩	(6.33)				34.5						**56**
36.0	D-24	▩	(7.83)				36.0						**56**
					End of BH RL (m) (7.83)								

Legend:

▩	Split Spoon Sample	▨	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

Bore Log Data for Driven Pile-6



Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People		**BORE LOG**
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.17
Co-ordinates(m):	23.85522N 90.35145E	Date Started:	12-Jun-14
Borehole No:	6 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	26.67	2.25	Gray, loose, ***Silty SAND, SM,*** trace of mica	░	1.5						**3**
2.5	UD-1	▥	25.67	3	Gray, soft, high plsticity**,** ***Fat CLAY, CH***	▨		99	115	69	19.1	69.4	
3.0	D-2	▩	25.17				3.0					37	**2**
4.5	D-3	▩	23.67				4.5					50	**1**
6.0	D-4	▩	22.17	7.5	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	6.0						**7**
7.5	D-5	▩	20.67				7.5						**7**
9.0	D-6	▩	19.17				9.0						**12**
10.5	D-7	▩	17.67				10.5						**12**
12.0	D-8	▩	16.17				12.0						**7**
13.5	D-9	▩	14.67	4.5	Gray, soft to stiff, medium plsticity, ***Lean CLAY, CL***	▨	13.5						**4**
15.0	D-10	▩	13.17				15.0	73	36	23			**6**
16.5	D-11	▩	11.67				16.5						**10**
18.0	D-12	▩	10.17	15.75	Brown, medium dense to dense, ***Silty SAND, SM,*** trace of mica	░	18.0						**10**
19.5	D-13	▩	8.67				19.5	35					**16**
21.0	D-14	▩	7.17				21.0						**18**
22.5	D-15	▩	5.67				22.5						**18**
24.0	D-16	▩	4.17				24.0						**21**
25.5	D-17	▩	2.67				25.5						**25**
27.0	D-18	▩	1.17				27.0						**25**
28.5	D-19	▩	(0.33)				28.5						**32**
30.0	D-20	▩	(1.83)				30.0						**31**
31.5	D-21	▩	(3.33)				31.5						**50**
33.0	D-22	▩	(4.83)				33.0						**50**
					End of BH RL (m) (4.83)								

Legend:

▩ Split Spoon Sample

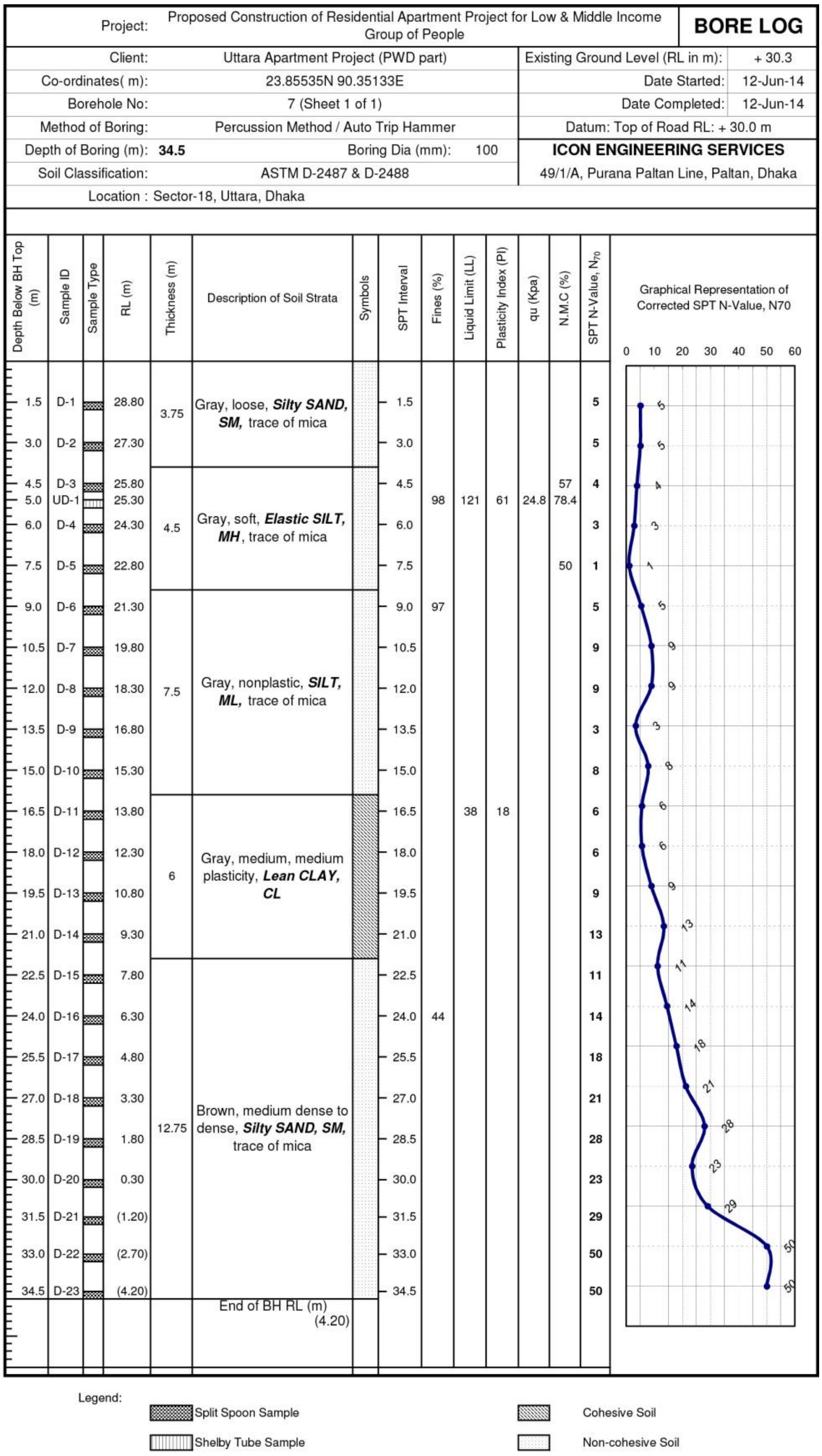
▥ Shelby Tube Sample

▨ Cohesive Soil

░ Non-cohesive Soil

Nahi

Bore Log Data for Driven Pile-7



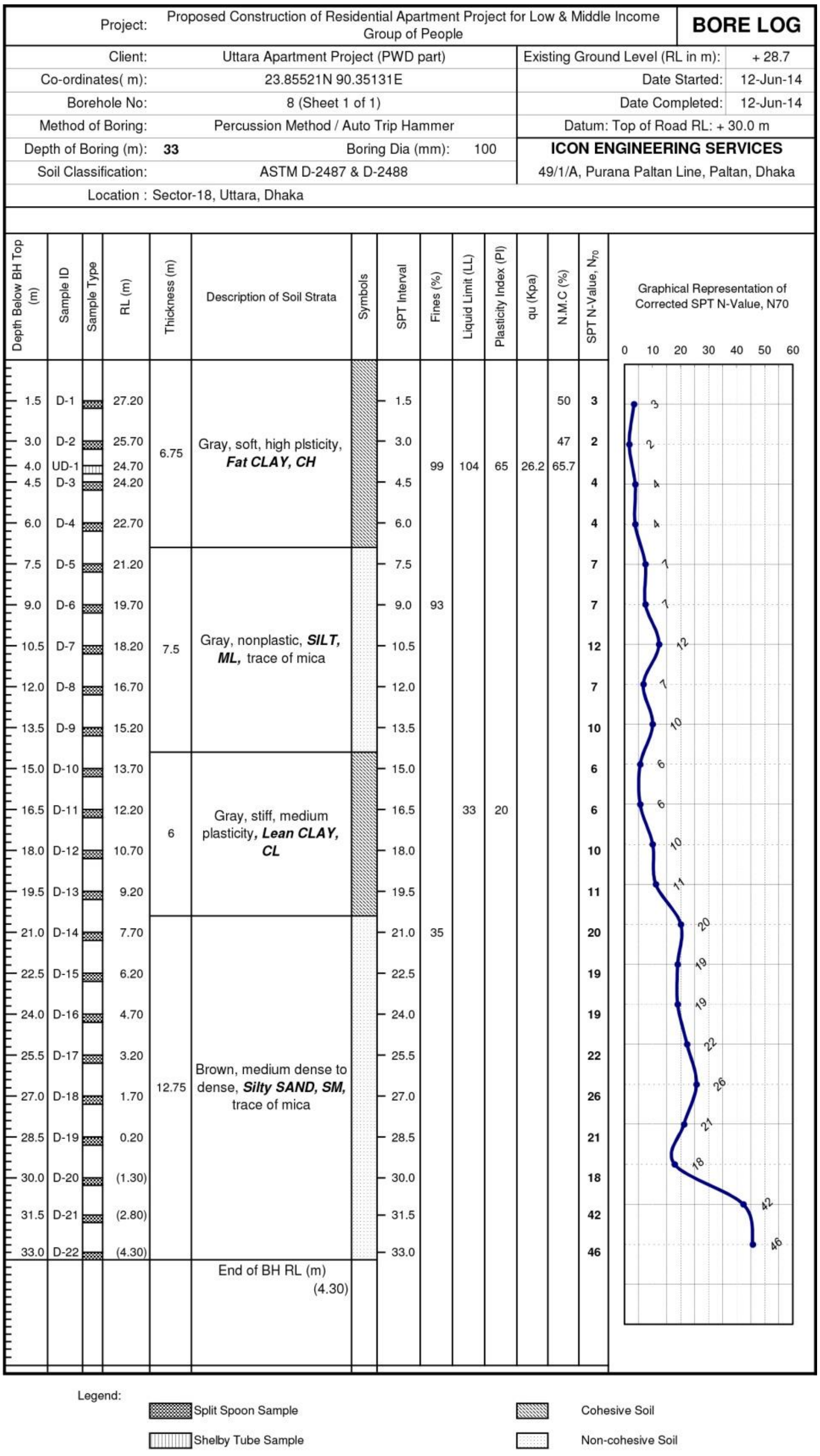
Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People		**BORE LOG**
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 30.3
Co-ordinates(m):	23.85535N 90.35133E	Date Started:	12-Jun-14
Borehole No:	7 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**34.5** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	28.80	3.75	Gray, loose, ***Silty SAND, SM,*** trace of mica	░	1.5						**5**
3.0	D-2	▩	27.30				3.0						**5**
4.5	D-3	▩	25.80	4.5	Gray, soft, ***Elastic SILT, MH***, trace of mica	░	4.5					57	**4**
5.0	UD-1	▥	25.30					98	121	61	24.8	78.4	
6.0	D-4	▩	24.30				6.0						**3**
7.5	D-5	▩	22.80				7.5					50	**1**
9.0	D-6	▩	21.30	7.5	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	9.0	97					**5**
10.5	D-7	▩	19.80				10.5						**9**
12.0	D-8	▩	18.30				12.0						**9**
13.5	D-9	▩	16.80				13.5						**3**
15.0	D-10	▩	15.30				15.0						**8**
16.5	D-11	▩	13.80	6	Gray, medium, medium plasticity, ***Lean CLAY, CL***	▨	16.5		38	18			**6**
18.0	D-12	▩	12.30				18.0						**6**
19.5	D-13	▩	10.80				19.5						**9**
21.0	D-14	▩	9.30				21.0						**13**
22.5	D-15	▩	7.80	12.75	Brown, medium dense to dense, ***Silty SAND, SM,*** trace of mica	░	22.5						**11**
24.0	D-16	▩	6.30				24.0	44					**14**
25.5	D-17	▩	4.80				25.5						**18**
27.0	D-18	▩	3.30				27.0						**21**
28.5	D-19	▩	1.80				28.5						**28**
30.0	D-20	▩	0.30				30.0						**23**
31.5	D-21	▩	(1.20)				31.5						**29**
33.0	D-22	▩	(2.70)				33.0						**50**
34.5	D-23	▩	(4.20)				34.5						**50**
					End of BH RL (m) (4.20)								

Legend:

▩	Split Spoon Sample	▨	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

Bore Log Data for Driven Pile-8



Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People	**BORE LOG**	
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.7
Co-ordinates(m):	23.85521N 90.35131E	Date Started:	12-Jun-14
Borehole No:	8 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	27.20	6.75	Gray, soft, high plsticity, ***Fat CLAY, CH***	▨	1.5					50	**3**
3.0	D-2	▩	25.70				3.0					47	**2**
4.0	UD-1	▥	24.70					99	104	65	26.2	65.7	
4.5	D-3	▩	24.20				4.5						**4**
6.0	D-4	▩	22.70				6.0						**4**
7.5	D-5	▩	21.20	7.5	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	7.5						**7**
9.0	D-6	▩	19.70				9.0	93					**7**
10.5	D-7	▩	18.20				10.5						**12**
12.0	D-8	▩	16.70				12.0						**7**
13.5	D-9	▩	15.20				13.5						**10**
15.0	D-10	▩	13.70	6	Gray, stiff, medium plasticity***, Lean CLAY, CL***	▨	15.0						**6**
16.5	D-11	▩	12.20				16.5		33	20			**6**
18.0	D-12	▩	10.70				18.0						**10**
19.5	D-13	▩	9.20				19.5						**11**
21.0	D-14	▩	7.70	12.75	Brown, medium dense to dense, ***Silty SAND, SM,*** trace of mica	░	21.0	35					**20**
22.5	D-15	▩	6.20				22.5						**19**
24.0	D-16	▩	4.70				24.0						**19**
25.5	D-17	▩	3.20				25.5						**22**
27.0	D-18	▩	1.70				27.0						**26**
28.5	D-19	▩	0.20				28.5						**21**
30.0	D-20	▩	(1.30)				30.0						**18**
31.5	D-21	▩	(2.80)				31.5						**42**
33.0	D-22	▩	(4.30)				33.0						**46**
					End of BH RL (m) (4.30)								

Graphical Representation of Corrected SPT N-Value, N70

Legend:

▩ Split Spoon Sample

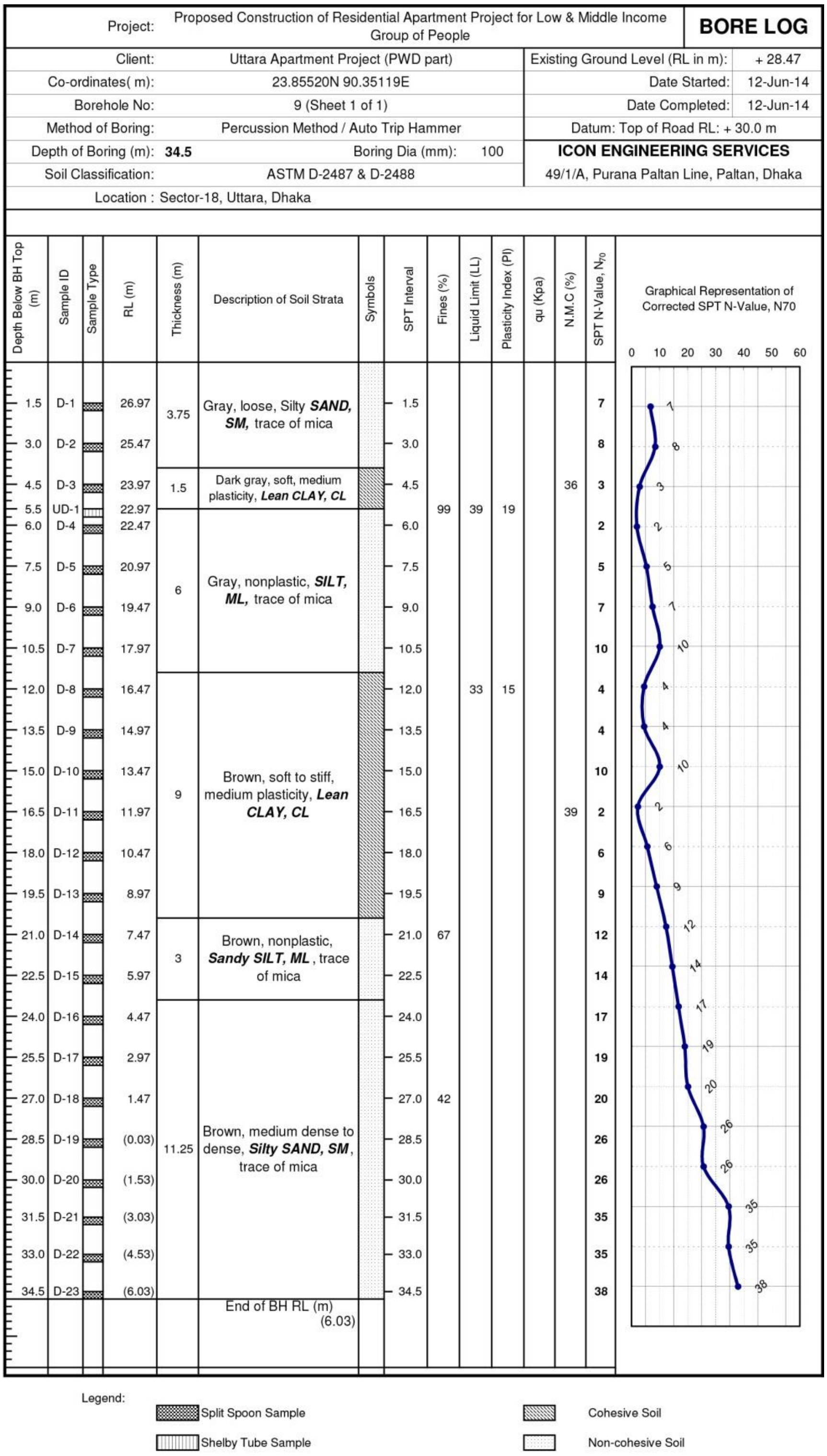
▥ Shelby Tube Sample

▨ Cohesive Soil

░ Non-cohesive Soil

Nahi

Bore Log Data for Driven Pile-9



Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People	**BORE LOG**	
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.47
Co-ordinates(m):	23.85520N 90.35119E	Date Started:	12-Jun-14
Borehole No:	9 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**34.5** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▦	26.97	3.75	Gray, loose, Silty ***SAND, SM,*** trace of mica	░	1.5						**7**
3.0	D-2	▦	25.47				3.0						**8**
4.5	D-3	▦	23.97	1.5	Dark gray, soft, medium plasticity, ***Lean CLAY, CL***	▨	4.5					36	**3**
5.5	UD-1	▥	22.97	6	Gray, nonplastic, ***SILT, ML,*** trace of mica	░		99	39	19			
6.0	D-4	▦	22.47				6.0						**2**
7.5	D-5	▦	20.97				7.5						**5**
9.0	D-6	▦	19.47				9.0						**7**
10.5	D-7	▦	17.97				10.5						**10**
12.0	D-8	▦	16.47	9	Brown, soft to stiff, medium plasticity, ***Lean CLAY, CL***	▨	12.0		33	15			**4**
13.5	D-9	▦	14.97				13.5						**4**
15.0	D-10	▦	13.47				15.0						**10**
16.5	D-11	▦	11.97				16.5					39	**2**
18.0	D-12	▦	10.47				18.0						**6**
19.5	D-13	▦	8.97				19.5						**9**
21.0	D-14	▦	7.47	3	Brown, nonplastic, ***Sandy SILT, ML***, trace of mica	░	21.0	67					**12**
22.5	D-15	▦	5.97				22.5						**14**
24.0	D-16	▦	4.47	11.25	Brown, medium dense to dense, ***Silty SAND, SM***, trace of mica	░	24.0						**17**
25.5	D-17	▦	2.97				25.5						**19**
27.0	D-18	▦	1.47				27.0	42					**20**
28.5	D-19	▦	(0.03)				28.5						**26**
30.0	D-20	▦	(1.53)				30.0						**26**
31.5	D-21	▦	(3.03)				31.5						**35**
33.0	D-22	▦	(4.53)				33.0						**35**
34.5	D-23	▦	(6.03)				34.5						**38**
					End of BH RL (m) (6.03)								

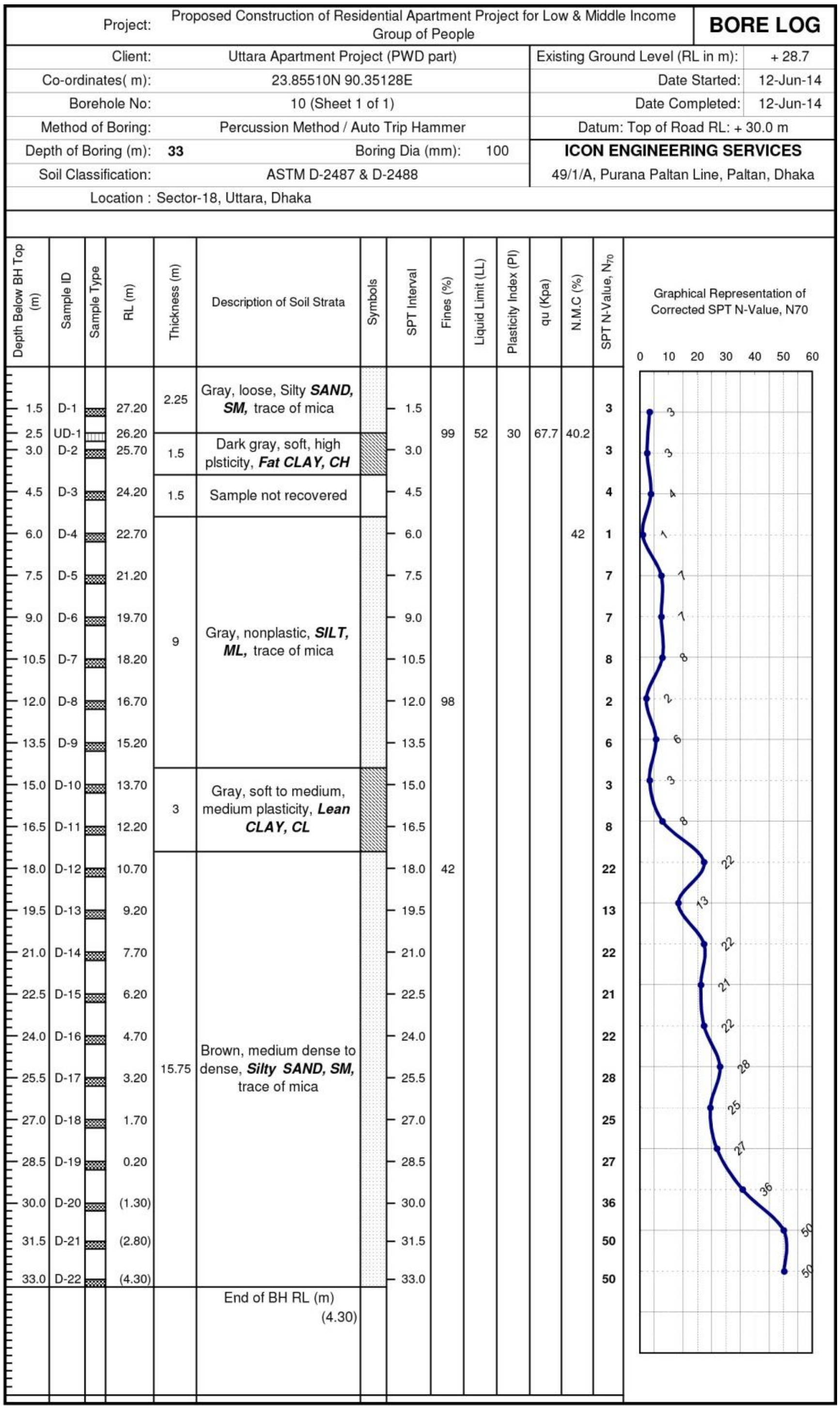
Graphical Representation of Corrected SPT N-Value, N70

Legend:

▦	Split Spoon Sample	▨	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

Nahi

Bore Log Data for Driven Pile-10



Project:	Proposed Construction of Residential Apartment Project for Low & Middle Income Group of People	**BORE LOG**	
Client:	Uttara Apartment Project (PWD part)	Existing Ground Level (RL in m):	+ 28.7
Co-ordinates(m):	23.85510N 90.35128E	Date Started:	12-Jun-14
Borehole No:	10 (Sheet 1 of 1)	Date Completed:	12-Jun-14
Method of Boring:	Percussion Method / Auto Trip Hammer	Datum: Top of Road RL: + 30.0 m	
Depth of Boring (m):	**33** Boring Dia (mm): 100	**ICON ENGINEERING SERVICES**	
Soil Classification:	ASTM D-2487 & D-2488	49/1/A, Purana Paltan Line, Paltan, Dhaka	
Location :	Sector-18, Uttara, Dhaka		

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value, N_{70}
1.5	D-1	▩	27.20	2.25	Gray, loose, Silty ***SAND, SM,*** trace of mica	░	1.5						**3**
2.5	UD-1	▥	26.20					99	52	30	67.7	40.2	
3.0	D-2	▩	25.70	1.5	Dark gray, soft, high plsticity, ***Fat CLAY, CH***	▨	3.0						**3**
4.5	D-3	▩	24.20	1.5	Sample not recovered		4.5						**4**
6.0	D-4	▩	22.70	9	Gray, nonplastic, ***SILT, ML,*** trace of mica	░	6.0					42	**1**
7.5	D-5	▩	21.20				7.5						**7**
9.0	D-6	▩	19.70				9.0						**7**
10.5	D-7	▩	18.20				10.5						**8**
12.0	D-8	▩	16.70				12.0	98					**2**
13.5	D-9	▩	15.20				13.5						**6**
15.0	D-10	▩	13.70	3	Gray, soft to medium, medium plasticity, ***Lean CLAY, CL***	▨	15.0						**3**
16.5	D-11	▩	12.20				16.5						**8**
18.0	D-12	▩	10.70	15.75	Brown, medium dense to dense, ***Silty SAND, SM,*** trace of mica	░	18.0	42					**22**
19.5	D-13	▩	9.20				19.5						**13**
21.0	D-14	▩	7.70				21.0						**22**
22.5	D-15	▩	6.20				22.5						**21**
24.0	D-16	▩	4.70				24.0						**22**
25.5	D-17	▩	3.20				25.5						**28**
27.0	D-18	▩	1.70				27.0						**25**
28.5	D-19	▩	0.20				28.5						**27**
30.0	D-20	▩	(1.30)				30.0						**36**
31.5	D-21	▩	(2.80)				31.5						**50**
33.0	D-22	▩	(4.30)				33.0						**50**
					End of BH RL (m) (4.30)								

Graphical Representation of Corrected SPT N-Value, N70

Legend:

▩ Split Spoon Sample

▥ Shelby Tube Sample

▨ Cohesive Soil

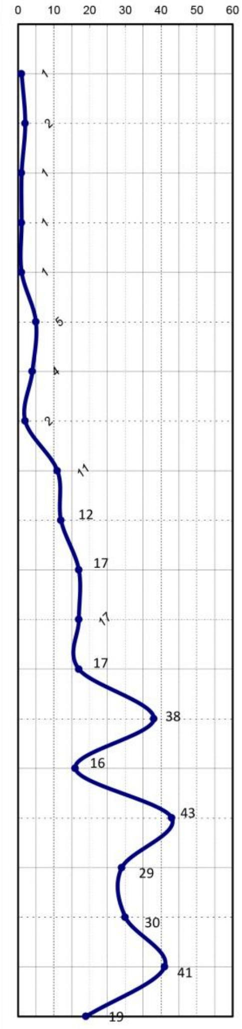
░ Non-cohesive Soil

BH NO-01, Kumar Bridge P-1

Project:	Dhaka-Khulna (N8) Road Improvement Project	**BORE LOG**	
Client:	24 Engineer Construction Brigade	Borehole Top RL, m:	+ 2.430
Co-ordinates:	N = 2586007.652 E = 498563.720	Date Started:	28-Dec-17
Borehole No:	**BH-01 (P1)** (Sheet 1 of 2)	Date Completed:	31-Dec-17
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL,m:	+ 0.62
Depth of Boring (m):	**57.0** Boring Dia (mm): 100	Structure: Bridge	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	CH-1+254.490, PIER-01, Kumar Bridge	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	$q_{u\ (Kpa)}$	N.M.C (%)	SPT N-Value / Panitration
1.5	D-1	▦	0.93	2.25	Debris	▦	1.5						**1**
3.0	D-2	▦	(0.57)	4.5	Gray, very soft to soft, medium plasticity, ***Lean CLAY, CL***	▒	3.0						**2**
4.5	D-3	▦	(2.07)				4.5						**1**
5.0	UD-1	▥	(2.57)					91	49.2	34.2	8.8	45.0	
6.0	D-4	▦	(3.57)				6.0						**1**
7.5	D-5	▦	(5.07)	1.5	Gray, very soft, high plasticity, ***Fat CLAY, CH***	▒	7.5						**1**
8.0	UD-2	▥	(5.57)					93	54.8	32.9	8	44	
9.0	D-6	▦	(6.57)	12	Gray, loose to medium dense, nonplastic, ***SILT, ML*** , trace of mica	░	9.0						**5**
10.5	D-7	▦	(8.07)				10.5						**4**
12.0	D-8	▦	(9.57)				12.0					17	**2**
13.5	D-9	▦	(11.07)				13.5						**11**
15.0	D-10	▦	(12.57)				15.0						**12**
16.5	D-11	▦	(14.07)				16.5						**17**
18.0	D-12	▦	(15.6)				18.0	98					**17**
19.5	D-13	▦	(17.1)				19.5						**17**
21.0	D-14	▦	(18.6)	18	Gray, medium dense to dense, ***Silty SAND, SM*** , trace of mica	░	21.0						**38**
22.5	D-15	▦	(20.1)				22.5						**16**
24.0	D-16	▦	(21.6)				24.0						**43**
25.5	D-17	▦	(23.1)				25.5						**29**
27.0	D-18	▦	(24.6)				27.0						**30**
28.5	D-19	▦	(26.1)				28.5	25					**41**
30.0	D-20	▦	(27.57)				30.0						**19**

Graphical Representation of field SPT N-Value



Legend:

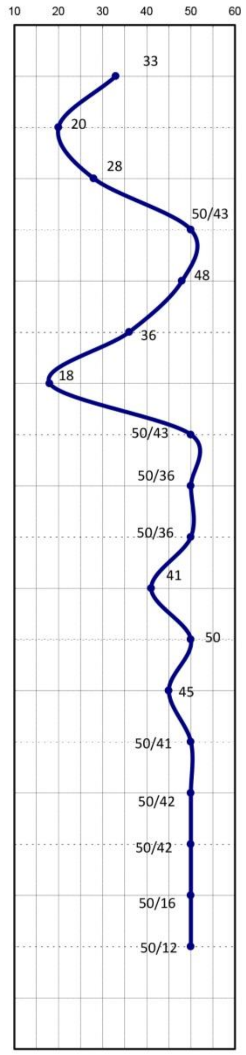
▦	Split Spoon Sample	▒	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

BH NO-01, Kumar Bridge P-1

Project:	Dhaka-Khulna (N8) Road Improvement Project		**BORE LOG**
Client:	24 Engineer Construction Brigade	Borehole Top RL, m:	+ 2.430
Co-ordinates:	N = 2586007.652 E = 498563.720	Date Started:	28-Dec-17
Borehole No:	**BH-01 (P1)** (Sheet 2 of 2)	Date Completed:	31-Dec-17
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL,m:	+ 0.62
Depth of Boring (m):	**57.0** Boring Dia (mm): 100	Structure: Bridge	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	CH-1+254.490, PIER-01, Kumar Bridge	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	q_u (Kpa)	N.M.C (%)	SPT N-Value / Panitration
31.5	D-21	▦	(29.07)	18	Gray, medium dense to dense, ***Silty SAND, SM***, trace of mica		31.5						**33**
33.0	D-22	▦	(30.57)				33.0						**20**
34.5	D-23	▦	(32.07)				34.5						**28**
36.0	D-24	▦	(33.57)				36.0						**50/43**
37.5	D-25	▦	(35.07)				37.5						**48**
39.0	D-26	▦	(36.57)	3	Gray, medium dense to dense, nonplastic, ***SILT with Sand, ML***, trace of mica		39.0	77					**36**
40.5	D-27	▦	(38.07)				40.5						**18**
42.0	D-28	▦	(39.57)	15.75	Gray, dense, ***Silty SAND, SM***, trace of mica		42.0						**50/43**
43.5	D-29	▦	(41.07)				43.5						**50/36**
45.0	D-30	▦	(42.57)				45.0						**50/36**
46.5	D-31	▦	(44.07)				46.5						**41**
48.0	D-32	▦	(45.57)				48.0						**50**
49.5	D-33	▦	(47.07)				49.5						**45**
51.0	D-34	▦	(48.57)				51.0	28					**50/41**
52.5	D-35	▦	(50.07)				52.5						**50/42**
54.0	D-36	▦	(51.57)				54.0						**50/42**
55.5	D-37	▦	(53.07)				55.5						**50/15**
57.0	D-38	▦	(54.57)				57.0						**50/12**
					End of BH RL (m) (54.57)								

Graphical Representation of field SPT N-Value



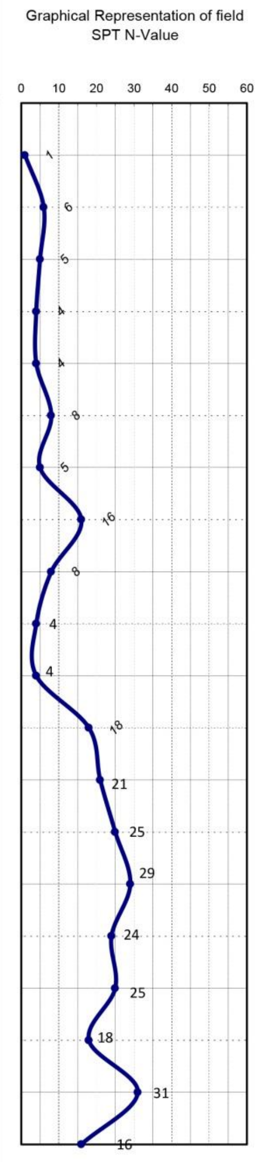
Legend:

▦	Split Spoon Sample	▨	Cohesive Soil
▥	Shelby Tube Sample	☐	Non-cohesive Soil

BH NO-02, Kumar Bridge, A-1

Project:	Dhaka-Khulna (N8) Road Improvement Project			**BORE LOG**
Client:	24 Engineer Construction Brigade		Borehole Top RL, m:	+ 7.181
Co-ordinates:	N = 2585918.071	E = 498563.898	Date Started:	01-Jan-18
Borehole No:	**BH-02 (A3)** (Sheet 1 of 2)		Date Completed:	04-Jan-18
Method of Boring:	Percussion Method / Auto Trip Hammer		Borehole Water RL,m:	+ 0.584
Depth of Boring (m): **61.5**	Boring Dia (mm):	100	Structure: Bridge	
Soil Classification:	ASTM D-2487 & D-2488		**ICON ENGINEERING SERVICES**	
Location :	CH-1+339.63, Abutment-3, Kumar Bridge		49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	q_u (Kpa)	N.M.C (%)	SPT N-Value / Panitration
1.5	D-1	▩	5.68	2.25	Debris	▓	1.5						**1**
3.0	D-2	▩	4.18	6	Brown to gray, soft to medium stiff, medium plasticity, ***Lean CLAY, CL***	▒	3.0						**6**
4.5	D-3	▩	2.68				4.5					16.0	**5**
5.0	UD-1	▥	2.18					99	49.8	28.1	59.3	34.5	
6.0	D-4	▩	1.18				6.0					14	**4**
6.5	UD-2	▥	0.68					97	43.6	25.6	21	34	
7.5	D-5	▩	(0.32)				7.5						**4**
9.0	D-6	▩	(1.82)	3	Gray, loose, nonplastic, ***SILT, ML*** , trace of mica	░	9.0						**8**
10.5	D-7	▩	(3.32)				10.5						**5**
12.0	D-8	▩	(4.82)	3	Gray, loose to medium dense, ***Silty SAND, SM*** , trace of mica	░	12.0						**16**
13.5	D-9	▩	(6.32)				13.5	24					**8**
15.0	D-10	▩	(7.82)	3	Gray, soft, high plasticity, ***Fat CLAY, CH***	▒	15.0						**4**
15.5	UD-3	▥	(8.32)					93	57	35.2		67	
16.5	D-11	▩	(9.32)				16.5					19	**4**
18.0	D-12	▩	(10.8)	44.25	Gray, medium dense to dense, ***Silty SAND, SM*** , trace of mica	░	18.0						**18**
19.5	D-13	▩	(12.3)				19.5						**21**
21.0	D-14	▩	(13.8)				21.0						**25**
22.5	D-15	▩	(15.3)				22.5						**29**
24.0	D-16	▩	(16.8)				24.0						**24**
25.5	D-17	▩	(18.3)				25.5						**25**
27.0	D-18	▩	(19.8)				27.0						**18**
28.5	D-19	▩	(21.3)				28.5						**31**
30.0	D-20	▩	(22.82)				30.0						**16**



Legend:

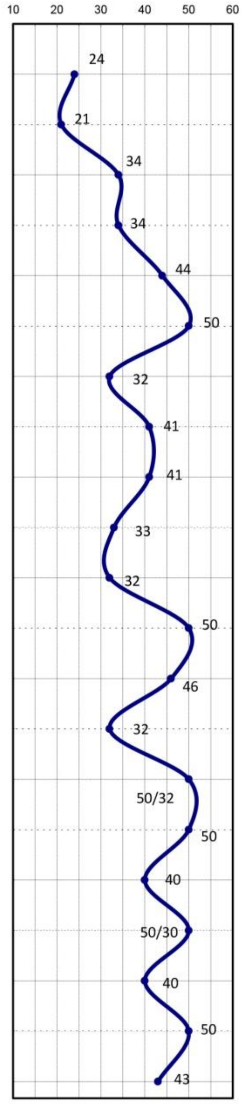
▩	Split Spoon Sample	▒	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

BH NO-02, Kumar Bridge, A-1

Project:	Dhaka-Khulna (N8) Road Improvement Project	**BORE LOG**	
Client:	24 Engineer Construction Brigade	Borehole Top RL, m:	+ 7.181
Co-ordinates:	N = 2585918.071 E = 498563.898	Date Started:	01-Jan-18
Borehole No:	**BH-02 (A3)** (Sheet 2 of 2)	Date Completed:	04-Jan-18
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL,m:	+ 0.584
Depth of Boring (m):	**61.5** Boring Dia (mm): 100	Structure: Bridge	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	CH-1+339.63, Abutment-3, Kumar Bridge	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	$q_{u\,(Kpa)}$	N.M.C (%)	SPT N-Value / Panitration
31.5	D-21	▨	(24.32)	44.25	Gray, medium dense to dense, ***Silty SAND, SM***, trace of mica		31.5						**24**
33.0	D-22	▨	(25.82)				33.0						**21**
34.5	D-23	▨	(27.32)				34.5						**34**
36.0	D-24	▨	(28.82)				36.0						**34**
37.5	D-25	▨	(30.32)				37.5						**44**
39.0	D-26	▨	(31.82)				39.0	30					**50**
40.5	D-27	▨	(33.32)				40.5						**32**
42.0	D-28	▨	(34.82)				42.0						**41**
43.5	D-29	▨	(36.32)				43.5						**41**
45.0	D-30	▨	(37.82)				45.0						**33**
46.5	D-31	▨	(39.32)				46.5						**32**
48.0	D-32	▨	(40.82)				48.0						**50**
49.5	D-33	▨	(42.32)				49.5						**46**
51.0	D-34	▨	(43.82)				51.0						**32**
52.5	D-35	▨	(45.32)				52.5						**50/32**
54.0	D-36	▨	(46.82)				54.0						**50**
55.5	D-37	▨	(48.32)				55.5						**40**
57.0	D-38	▨	(49.82)				57.0						**50/30**
58.5	D-39	▨	(51.32)				58.5						**40**
60.0	D-40	▨	(52.82)				60.0	25					**50**
61.5	D-41	▨	(54.32)				61.5						**43**
					End of BH RL (m) (49.82)								

Graphical Representation of field SPT N-Value



Legend:

▨ Split Spoon Sample

▥ Shelby Tube Sample

▒ Cohesive Soil

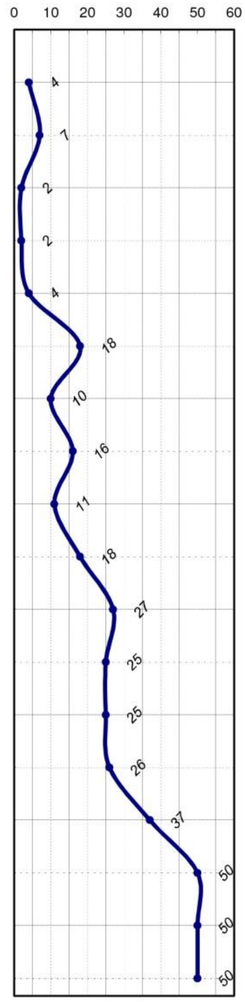
░ Non-cohesive Soil

BH NO-2-A2 Shibpur Bridge SASEC

Project:	SASEC Road Connectivity Project		**BORE LOG**
Client:	GDCL-DIENCO (JV)	Existing Ground Level (RL in m):	+ 9.918
Co-ordinates:	E = 4924581.665 N = 2685018.114	Date Started:	18-Apr-16
Borehole No:	**2** / **A2** (Sheet 1 of 1)	Date Completed:	19-Apr-16
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL (m):	+ 6.15
Depth of Boring (m):	**27** Boring Dia (mm): 100	TBM No: 120, RL: + 13.860	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	Chainage: 63+989.83m, Shibpur Bridge, Tangail	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below BH Top (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	qu (Kpa)	N.M.C (%)	SPT N-Value
1.5	D-1	▩	8.42	8.25	Gray, loose, nonplastic, ***SILT with Sand, ML***, trace of mica	░	1.5						4
3.0	D-2	▩	6.92				3.0						7
4.5	D-3	▩	5.42				4.5						2
6.0	D-4	▩	3.92				6.0						2
7.5	D-5	▩	2.42				7.5	84					4
9.0	D-6	▩	0.92	18.75	Gray, medium dense to dense, ***Silty SAND, SM***, trace of mica	░	9.0						18
10.5	D-7	▩	(0.58)				10.5						10
12.0	D-8	▩	(2.08)				12.0						16
13.5	D-9	▩	(3.58)				13.5						11
15.0	D-10	▩	(5.08)				15.0						18
16.5	D-11	▩	(6.58)				16.5						27
18.0	D-12	▩	(8.08)				18.0						25
19.5	D-13	▩	(9.58)				19.5	14					25
21.0	D-14	▩	(11.08)				21.0						26
22.5	D-15	▩	(12.58)				22.5						37
24.0	D-16	▩	(14.08)				24.0						50
25.5	D-17	▩	(15.58)				25.5						50
27.0	D-18	▩	(17.08)				27.0	24					50
					End of BH RL (m) (17.08)								

Graphical Representation of Field SPT N-Value



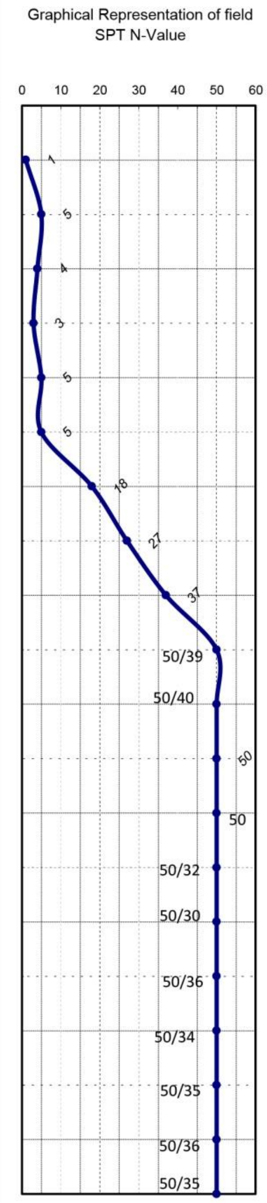
Legend:

▩	Split Spoon Sample	▨	Cohesive Soil
▥	Shelby Tube Sample	░	Non-cohesive Soil

Borehole 3 Postogola UP A-1

Project:	Dhaka-Khulna (N8) Road Improvement Project	**BORE LOG**	
Client:	17 ECB Special Works Organization(SWO-West)	Borehole Top RL, m:	+ 6.791
Co-ordinates:	X = 544201.927 Y = 2619748.366	Date Started:	07-Jan-18
Borehole No:	**BH-03** (Sheet 1 of 2)	Date Completed:	12-Jan-18
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL,m:	+ 4.301
Depth of Boring (m):	**49.5** Boring Dia (mm): 100	BM RL: + 6.870	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	Keraniganj, Postogola Underpass	49/1/A, Purana Paltan Line, Paltan, Dhaka	

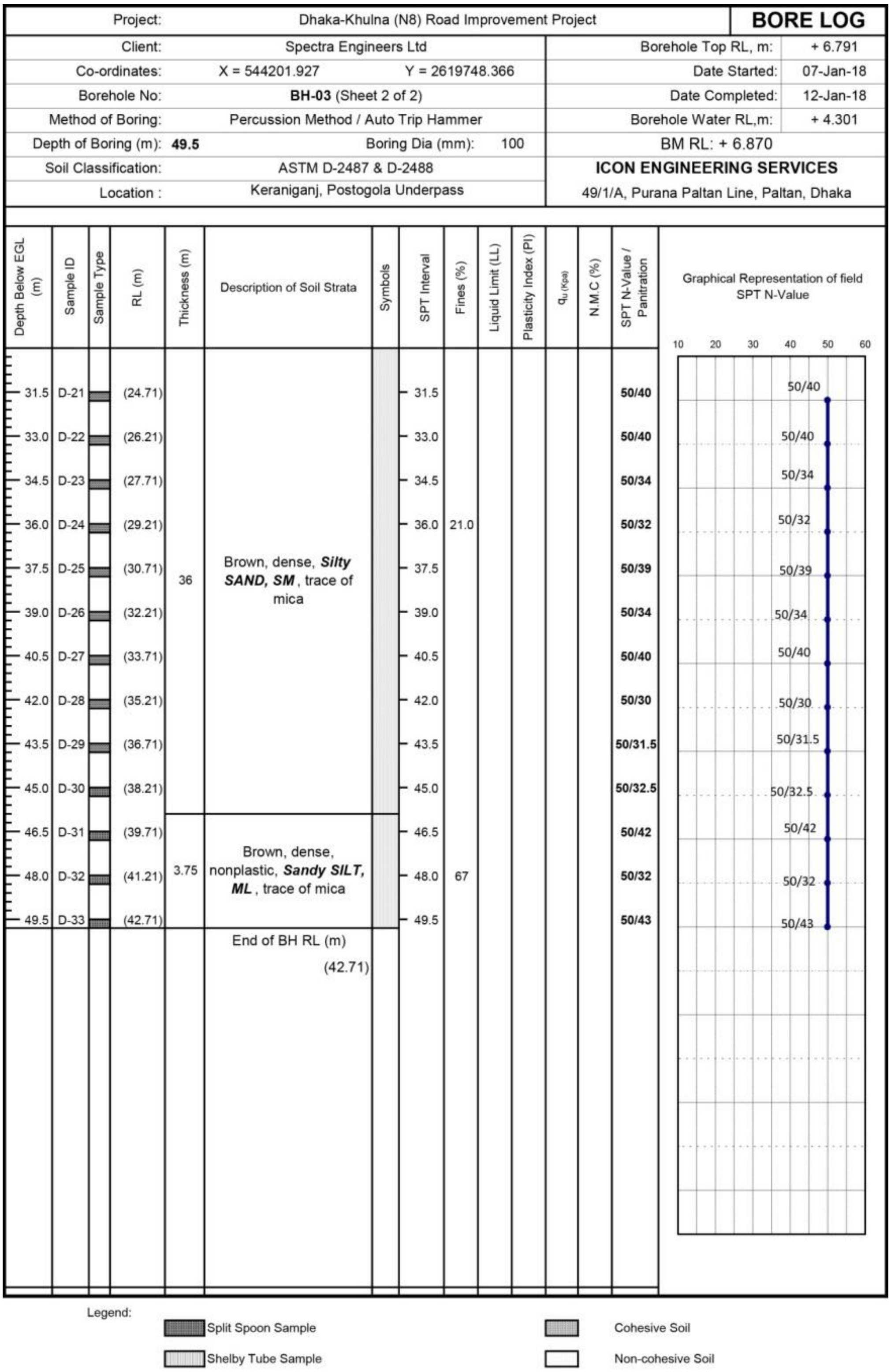
Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	$q_{u\ (Kpa)}$	N.M.C (%)	SPT N-Value / Panitration
1.5	D-1	▦	5.29	2.25	Debris	▒	1.5						1
3.0	D-2	▦	3.79	7.5	Gray, soft to medium stiff, medium plasticity, ***Lean CLAY, CL***	▒	3.0						5
4.5	D-3	▦	2.29				4.5						4
6.0	D-4	▦	0.79				6.0						3
7.5	D-5	▦	(0.71)				7.5						5
9.0	D-6	▦	(2.21)				9.0						5
9.5	UD-1	▥	(2.71)					93	41.1	22.2	71.2	27.9	
10.5	D-7	▦	(3.71)	36	Brown, medium dense to dense, ***Silty SAND, SM***, trace of mica	☐	10.5						18
12.0	D-8	▦	(5.21)				12.0						27
13.5	D-9	▦	(6.71)				13.5						37
15.0	D-10	▦	(8.21)				15.0						50/39
16.5	D-11	▦	(9.71)				16.5						50/40
18.0	D-12	▦	(11.2)				18.0						50
19.5	D-13	▦	(12.7)				19.5						50
21.0	D-14	▦	(14.2)				21.0	16					50/32
22.5	D-15	▦	(15.7)				22.5						50/30
24.0	D-16	▦	(17.2)				24.0						50/36
25.5	D-17	▦	(18.7)				25.5						50/34
27.0	D-18	▦	(20.2)				27.0						50/35
28.5	D-19	▦	(21.7)				28.5						50/36
30.0	D-20	▦	(23.21)				30.0						50/35



Legend:

▦	Split Spoon Sample	▒	Cohesive Soil
▥	Shelby Tube Sample	☐	Non-cohesive Soil

Borehole 3 Postogola Up A-1



Project:	Dhaka-Khulna (N8) Road Improvement Project	**BORE LOG**	
Client:	Spectra Engineers Ltd	Borehole Top RL, m:	+ 6.791
Co-ordinates:	X = 544201.927 Y = 2619748.366	Date Started:	07-Jan-18
Borehole No:	**BH-03** (Sheet 2 of 2)	Date Completed:	12-Jan-18
Method of Boring:	Percussion Method / Auto Trip Hammer	Borehole Water RL,m:	+ 4.301
Depth of Boring (m):	**49.5** Boring Dia (mm): 100	BM RL: + 6.870	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	Keraniganj, Postogola Underpass	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	q_u (Kpa)	N.M.C (%)	SPT N-Value / Panitration
31.5	D-21	▨	(24.71)	36	Brown, dense, ***Silty SAND, SM***, trace of mica	░	31.5						**50/40**
33.0	D-22	▨	(26.21)				33.0						**50/40**
34.5	D-23	▨	(27.71)				34.5						**50/34**
36.0	D-24	▨	(29.21)				36.0	21.0					**50/32**
37.5	D-25	▨	(30.71)				37.5						**50/39**
39.0	D-26	▨	(32.21)				39.0						**50/34**
40.5	D-27	▨	(33.71)				40.5						**50/40**
42.0	D-28	▨	(35.21)				42.0						**50/30**
43.5	D-29	▨	(36.71)				43.5						**50/31.5**
45.0	D-30	▨	(38.21)				45.0						**50/32.5**
46.5	D-31	▨	(39.71)	3.75	Brown, dense, nonplastic, ***Sandy SILT, ML***, trace of mica	░	46.5						**50/42**
48.0	D-32	▨	(41.21)				48.0	67					**50/32**
49.5	D-33	▨	(42.71)				49.5						**50/43**
					End of BH RL (m) (42.71)								

Graphical Representation of field SPT N-Value

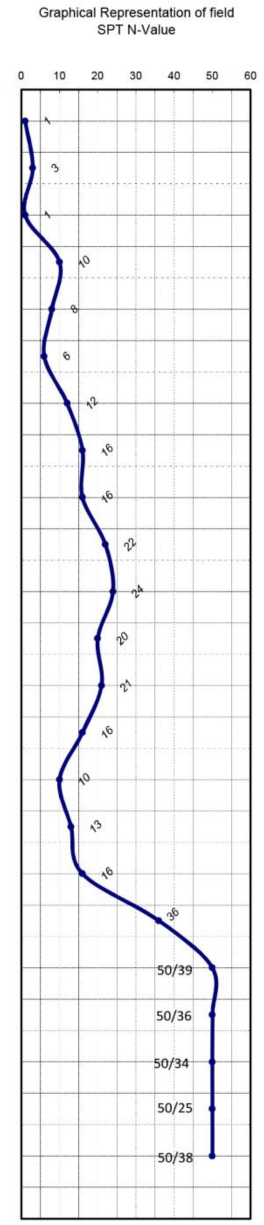
Legend:

▨	Split Spoon Sample	▒	Cohesive Soil
░	Shelby Tube Sample	☐	Non-cohesive Soil

Kalshi, BH-12, TP-3

Project:	Widening and improvement of road from ECB circle to Mirpur and construction of Flyover at Kalshi Intersection Project.	**BORE LOG**	
Client:	17 ECB	EGL (RL in m):	+ 7.702
Co-ordinates:	E = 232860.891 N = 2637120.731	Date Started:	20-Jul-18
Borehole No:	**SBH-12** (Sheet 1 of 1)	Date Completed:	22-Jul-18
Method of Boring:	Rotary	G.W.T (m):	+ 2.20
Depth of Boring (m):	**34.5** Boring Dia (mm): 150	TBM-(TP-14), RL=8.073, E = 232839.341, N = 2637122.983	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	Kalshi, Mirpur, Dhaka CH- 1+810.827	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	q_u (Kpa)	N.M.C (%)	SPT N Value / Penetration
1.5	D-1	▩	6.20	2.25	Debris	▨	1.5						**1**
3.0	D-2	▩	4.70	6	Brown to reddish brown, soft to stiff, medium plasticity, ***Sandy Lean CLAY, CL***	▨	3.0						**3**
4.5	D-3	▩	3.20				4.5						**1**
6.0	D-4	▩	1.70				6.0						**10**
6.5	UD-1	▥	1.20					65	46	33.7			
7.5	D-5	▩	0.20				7.5						**8**
9.0	D-6	▩	(1.30)	10.5	Brown, loose to medium dense, ***Silty SAND, SM***, trace of mica	☐	9.0	44					**6**
10.5	D-7	▩	(2.80)				10.5						**12**
12.0	D-8	▩	(4.30)				12.0						**16**
13.5	D-9	▩	(5.80)				13.5						**16**
15.0	D-10	▩	(7.30)				15.0						**22**
16.5	D-11	▩	(8.80)				16.5	31					**24**
18.0	D-12	▩	(10.30)				18.0						**20**
19.5	D-13	▩	(11.80)	7.5	Brown, stiff to very stiff, medium plasticity, ***Lean CLAY, CL***	▨	19.5						**21**
21.0	D-14	▩	(13.30)				21.0						**16**
22.5	D-15	▩	(14.80)				22.5						**10**
24.0	D-16	▩	(16.30)				24.0						**13**
25.5	D-17	▩	(17.80)				25.5						**16**
27.0	D-18	▩	(19.30)	8.25	Brown, dense, ***Silty SAND, SM***, trace of mica	☐	27.0						**36**
28.5	D-19	▩	(20.80)				28.5						**50/39**
30.0	D-20	▩	(22.30)				30.0						**50/36**
31.5	D-21	▩	(23.80)				31.5	35					**50/34**
33.0	D-22	▩	(25.30)				33.0						**50/25**
34.5	D-23	▩	(26.80)				34.5						**50/38**
					End of BH RL (m) (26.80)								



Legend:

EGL: Existing Ground Level WRL: Water Reduced Level RL: Reduced Level BH: Borehole PL: Plinth Level TBM : Temporary Bench Mark

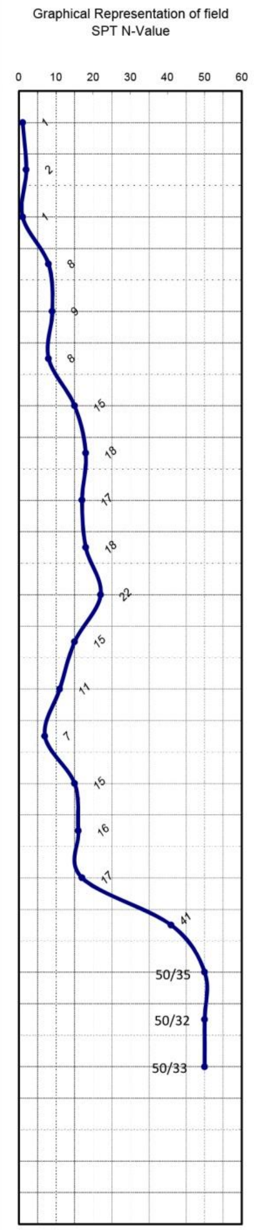
▩ Split Spoon Sample	▨ Cohesive Soil
▥ Shelby Tube Sample	☐ Non-cohesive Soil

Kalshi, BH-13, TP-2

Project:	Widening and improvement of road from ECB circle to Mirpur and construction of Flyover at Kalshi Intersection Project.	**BORE LOG**

Client:	17 ECB	EGL (RL in m):	+ 7.707
Co-ordinates:	E = 232852.464 N = 2637143.994	Date Started:	18-Jul-18
Borehole No:	**SBH-13** (Sheet 1 of 1)	Date Completed:	20-Jul-18
Method of Boring:	Rotary	G.W.T (m):	+ 2.10
Depth of Boring (m):	**31.5** Boring Dia (mm): 150	TBM-(TP-14), RL=8.073, E = 232839.341, N = 2637122.983	
Soil Classification:	ASTM D-2487 & D-2488	**ICON ENGINEERING SERVICES**	
Location :	Kalshi, Mirpur, Dhaka PR-10	49/1/A, Purana Paltan Line, Paltan, Dhaka	

Depth Below EGL (m)	Sample ID	Sample Type	RL (m)	Thickness (m)	Description of Soil Strata	Symbols	SPT Interval	Fines (%)	Liquid Limit (LL)	Plasticity Index (PI)	q_u (Kpa)	N.M.C (%)	SPT N Value / Penetration
1.5	D-1	▨	6.21	2.25	Debris	▒	1.5						**1**
3.0	D-2	▨	4.71	4.5	Brown, soft to medium stiff, high plasticity, ***Fat CLAY, CH***	▒	3.0						**2**
4.5	D-3	▨	3.21				4.5						**1**
6.0	D-4	▨	1.71				6.0						**8**
6.5	UD-1	▤	1.21						55	40.4			
7.5	D-5	▨	0.21	12	Brown, loose to medium dense, ***Silty SAND, SM***, trace of mica		7.5						**9**
9.0	D-6	▨	(1.29)				9.0						**8**
10.5	D-7	▨	(2.79)				10.5						**15**
12.0	D-8	▨	(4.29)				12.0	31					**18**
13.5	D-9	▨	(5.79)				13.5						**17**
15.0	D-10	▨	(7.29)				15.0						**18**
16.5	D-11	▨	(8.79)				16.5						**22**
18.0	D-12	▨	(10.29)				18.0						**15**
19.5	D-13	▨	(11.79)	7.5	Gray, medium stiff to very stiff, medium plasticity, ***Lean CLAY, CL***	▒	19.5						**11**
21.0	D-14	▨	(13.29)				21.0						**7**
22.5	D-15	▨	(14.79)				22.5	89	42	28			**15**
24.0	D-16	▨	(16.29)				24.0						**16**
25.5	D-17	▨	(17.79)				25.5						**17**
27.0	D-18	▨	(19.29)	5.25	Brown, dense, ***Silty SAND, SM***, trace of mica		27.0						**41**
28.5	D-19	▨	(20.79)				28.5						**50/35**
30.0	D-20	▨	(22.29)				30.0						**50/32**
31.5	D-21	▨	(23.79)				31.5	38					**50/33**
					End of BH RL (m) (23.79)								



Legend:

EGL: Existing Ground Level WRL: Water Reduced Level RL: Reduced Level BH: Borehole PL: Plinth Level TBM : Temporary Bench Mark

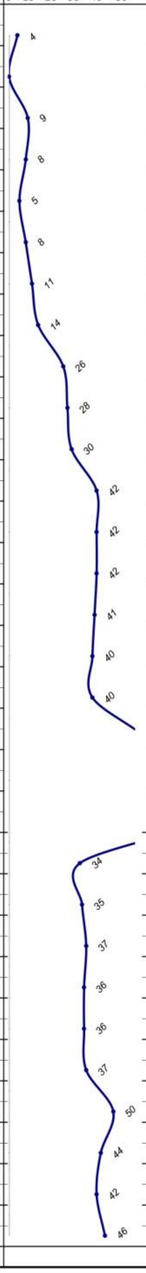
▨	Split Spoon Sample	▒	Cohesive Soil
▤	Shelby Tube Sample	☐	Non-cohesive Soil

MRT Borelog-PP8

ICON ENGINEERING SERVICES
49/1/A, Level-3, Purana Paltan Line, Purana Paltan, Dhaka-1000, Bangladesh.
Tel: + 880 2 831 9710, 933 6764, +880 1777775680, E-mail: info@iconengg.com

Project: Dhaka Mass Rapid Transit Development
Client: Italian-Thai Development (ITD)
Location: Agargaon, Dhaka.

Borehole No: PP-08	E: 232894.719m N: 2632531.706m Elev: 6. 836
W.D(m): + 4.836	Start: 22.07.2017 End: 27.07.2017



Grain Size				Atterberg Limits		Moisture Content	Unit Weight (Kn/m³)		Specific Gravity	Consolidation				Shear Strength		Standard Penetration Test				Sample No.	Depth (m)	Elev (m)	Layer Symbol	
Fines (%)	Sand (%)	Silt (%)	Clay (%)	L.L.	P.I.	W_n (%)	Y_t	Y_d	G_s	e_0	C_c	C_r	Pc (Kn/m²)	Type	qu (Kn/m²)	150mm	150mm	150mm	N-Value					
																						6.8		
																2	1	3	4	D1	1.0	5.8		
95	5			28.3	13.5	22.3									47.9					UD-1	2.0	4.8		
																2	4	5	9	D2	3.0	3.8		Gray to brown, soft to stiff, medium plasticity, ***Sandy Lean CLAY, CL***
																3	3	5	8	D3	4.0	2.8		
																2	2	3	5	D4	5.0	1.8		
																3	4	4	8	D5	6.0	0.8		
69	31			29.4	13.1	31.1									16.2					UD-2	6.5			
																3	5	6	11	D6	7.0	(0.2)		
69	31								2.6							3	6	8	14	D7	8.0	(1.2)		
																8	12	14	26	D8	9.0	(2.2)		
																8	12	16	28	D9	10.0	(3.2)		
																7	12	18	30	D10	11.0	(4.2)		
																14	19	23	42	D11	12.0	(5.2)		
25	75								2.63							10	18	24	42	D12	13.0	(6.2)		
																7	20	22	42	D13	14.0	(7.2)		
																14	22	19	41	D14	15.0	(8.2)		
																13	19	21	40	D15	16.0	(9.2)		
																17	16	24	40	D16	17.0	(10.2)		
																16	26	40	66	D17	18.0	(11.2)		
																15	28	50	78	D18	19.0	(12.2)		Brown, medium dense to dense, ***Silty SAND, SM*** , trace of mica
20	80								2.62							26	35	53	88	D19	20.0	(13.2)		
																11	14	20	34	D20	21.0	(14.2)		
																12	15	20	35	D21	22.0	(15.2)		
																13	15	22	37	D22	23.0	(16.2)		
																7	15	21	36	D23	24.0	(17.2)		
																10	14	22	36	D24	25.0	(18.2)		
																12	17	20	37	D25	26.0	(19.2)		
19	81								2.64							13	20	30	50	D26	27.0	(20.2)		
																14	21	23	44	D27	28.0	(21.2)		
																19	20	22	42	D28	29.0	(22.2)		
																16	22	24	46	D29	30.0	(23.2)		

Legend

L.L. = Liquid Limit
P.I. = Plasticity Index
W_n = Moisture Content
G_s = Spacific Gravity
Y_t = Total Unit Weight
Y_d = Dry Unit Weight
C_c = Compression Index
C_r = Re-compression Index
e_o = Initial Void Ratio
C_u = Undrained Cohension
S_u = Undrained Shear Strength
P_c = Preconsolidation Pressure
% = Strain Failure
D = Disturbed Sample
UD = Undisturbed Sample
W.d = Depth of water measured 24h after completion Drilling.
Fill 1 = Dredged Filled Sand Layer
Fill 2 = Human Filled Sand Layer
SM = Silty Sand
CL = Lean Clay

▨	Clay
☐	Silt
▨	Sand
■	Silty sand

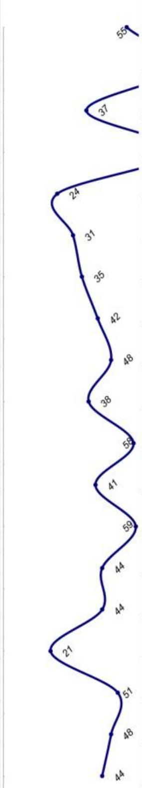
Drawn by : Saima Yeasmin
Check by: Nahid Kamal Kotha
Approved by: Abu Mohammed Masud
Date : 26.07.2017

MRT Borelog-PP8

ICON ENGINEERING SERVICES
49/1/A, Level-3, Purana Paltan Line, Purana Paltan, Dhaka-1000, Bangladesh.
Tel: + 880 2 831 9710, 933 6764, +880 1777775680, E-mail: info@iconengg.com

Project: Dhaka Mass Rapid Transit Development
Client: Italian-Thai Development (ITD)
Location: Agargaon, Dhaka.

Borehole No: PP-08	E: 232894.719m N: 2632531.706m Elev: 6. 836
W.D(m): + 4.836	Start: 22.07.2017 End: 27.07.2017



Grain Size				Atterberg Limits		Moisture Content	Unit Weight (Kn/m³)		Specific Gravity	Consolidation				Shear Strength		Standard Penetration Test									
Fines (%)	Sand (%)	Silt (%)	Clay (%)	L.L.	P.I.	W_n (%)	γ_t	γ_d	G_s	e_0	C_c	C_r	Pc (Kn/m²)	Type	Cu/Su (Kn/m²)	150mm	150mm	150mm	N-Value	Sample No.	Sample Rec. (mm)	Depth (m)	Elev (m)	Layer Symbol	
																							6.8		
																20	25	30	55	D30		31.0	(24.2)	▨	Brown, dense to very dense, ***Silty SAND, SM***, trace of mica
																25	36	41	77	D31		32.0	(25.2)	▨	
																8	16	21	37	D32		33.0	(26.2)	▨	
25	75								2.61							16	35	43	78	D33		34.0	(27.2)	▨	
69	31			37.4	21.3											8	10	14	24	D34		35.0	(28.2)	░	Brown, very stiff, medium plasticity, ***Sandy Lean CLAY, CL***
																12	13	18	31	D35		36.0	(29.2)	▨	Brown, dense, ***Silty SAND, SM***, trace of mica
																12	17	18	35	D36		37.0	(30.2)	▨	
																12	22	20	42	D37		38.0	(31.2)	▨	
																14	22	26	48	D38		39.0	(32.2)	▨	
																12	15	23	38	D39		40.0	(33.2)	▨	
30	70								2.63							15	23	35	58	D40		41.0	(34.2)	▨	
																11	22	19	41	D41		42.0	(35.2)	▨	
																18	28	31	59	D42		43.0	(36.2)	▨	
																18	18	26	44	D43		44.0	(37.2)	▨	
																13	21	23	44	D44		45.0	(38.2)	▨	
25	75			28.7	13.0											5	13	8	21	D45		46.0	(39.2)	▨	
																13	22	29	51	D46		47.0	(40.2)	▨	
																15	20	28	48	D47		48.0	(41.2)	▨	
																15	20	24	44	D48		49.0	(42.2)	▨	
																									End of Boring

Legend

L.L. = Liquid Limit
P.I. = Plasticity Index
W_n = Moisture Content
G_s = Spacific Gravity

γ_t = Total Unit Weight
γ_d = Dry Unit Weight
C_c = Compression Index
C_r = Re-compression Index

e_o = Initial Void Ratio
C_u = Undrained Cohension
S_u = Undrained Shear Strength
P_c = Preconsolidation Pressure

% = Strain Failure
D = Disturbed Sample
UD = Undisturbed Sample
W.d = Depth of water measured 24h after completion Drilling.

Fill 1 = Dredged Filled Sand Layer
Fill 2 = Human Filled Sand Layer
SM = Silty Sand
CL = Lean Clay

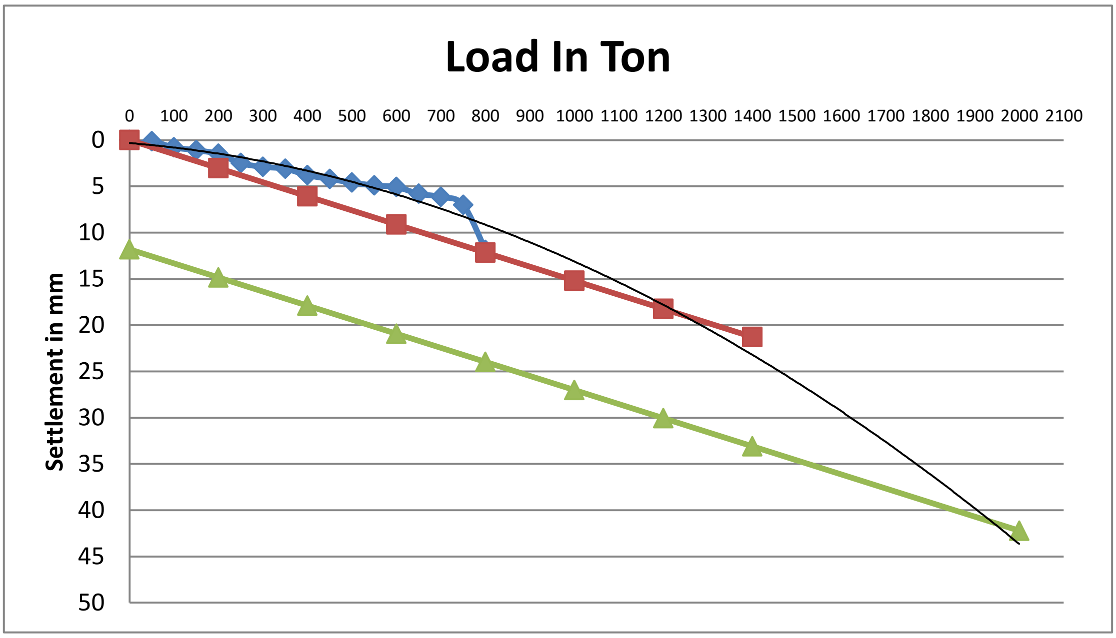
░	Clay
□	Silt
▦	Sand
▨	Silty sand

Drawn by : Saima Yeasmin
Check by: Nahid Kamal Kotha
Approved by: Abu Mohammed Masud
Date : 26.07.2017

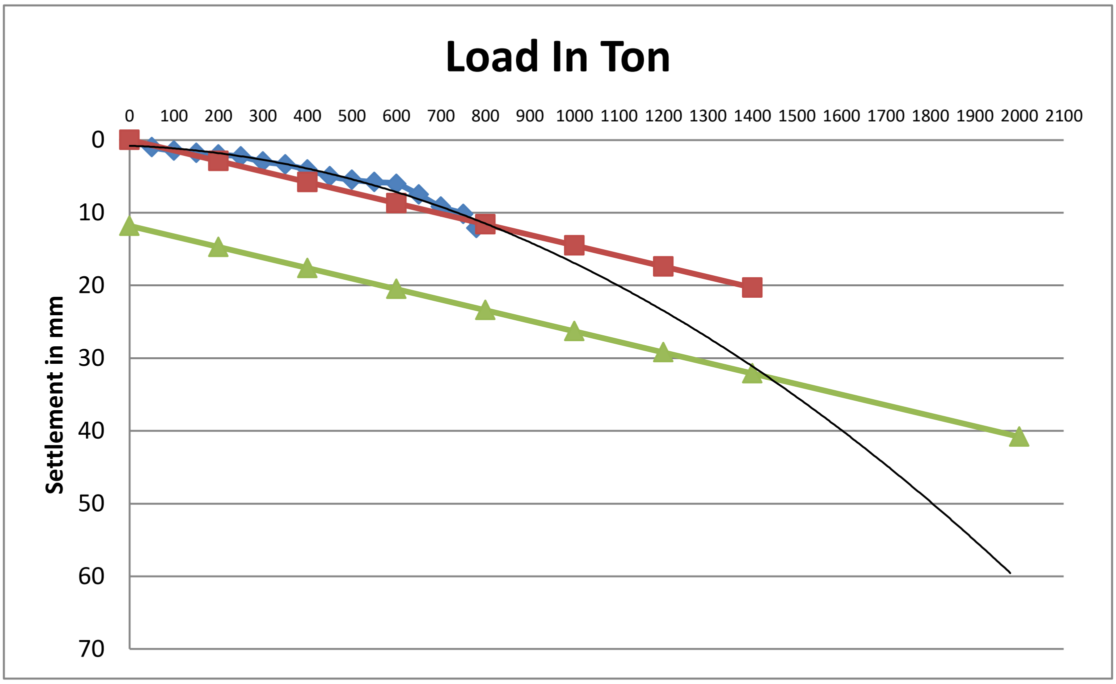
APPENDIX C

EXTRAPOLATION LOAD SETTLEMENT CURVE

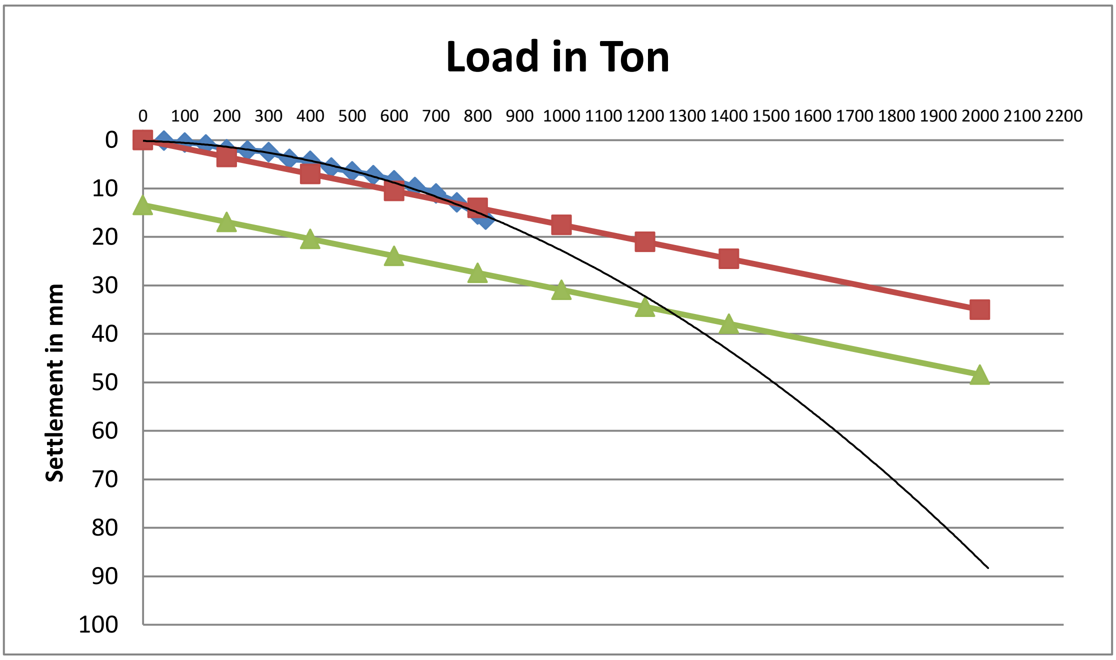
Kalshi, (TP-2) CTP-05, LT



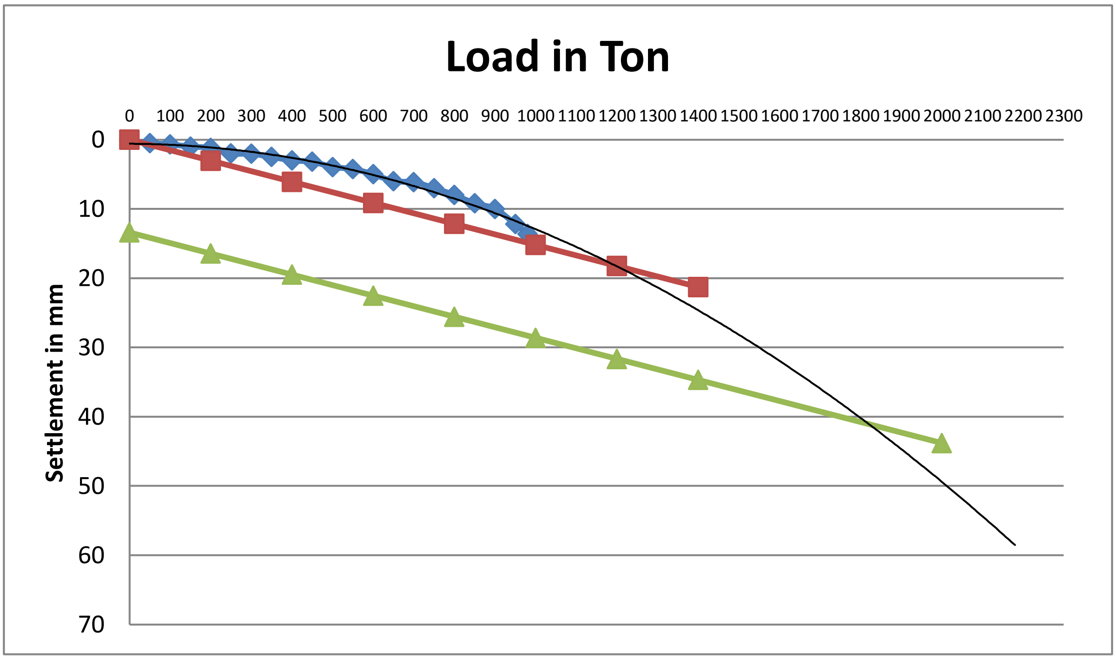
Kalshi, (TP-3) CTP-06, LT



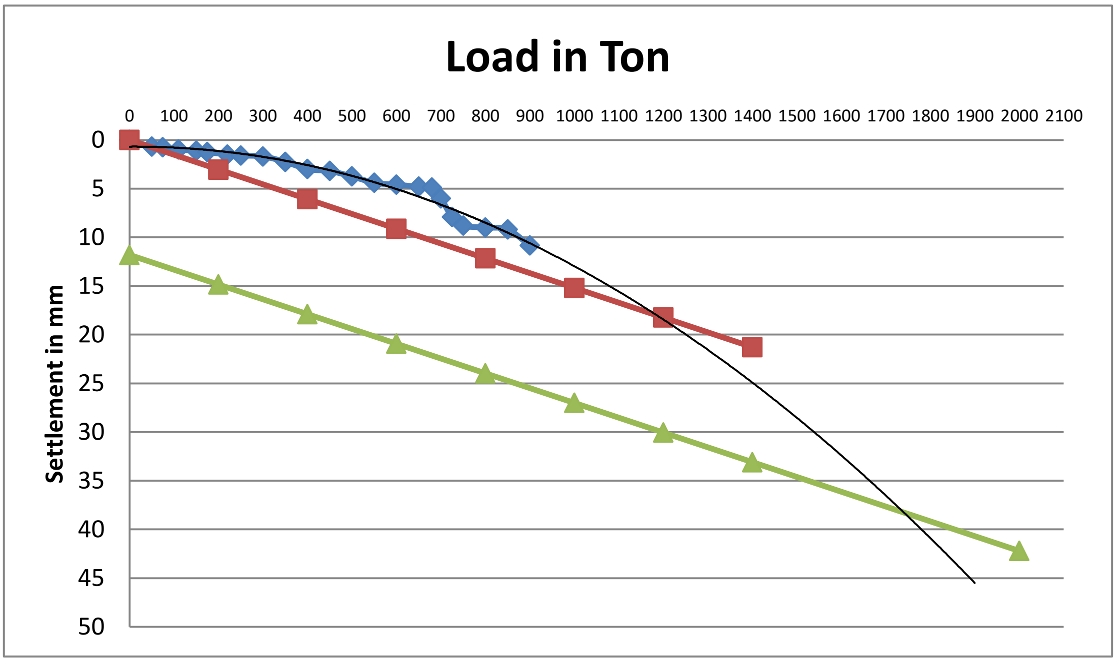
Kumar Bridge (A-1) CTP-08, LT



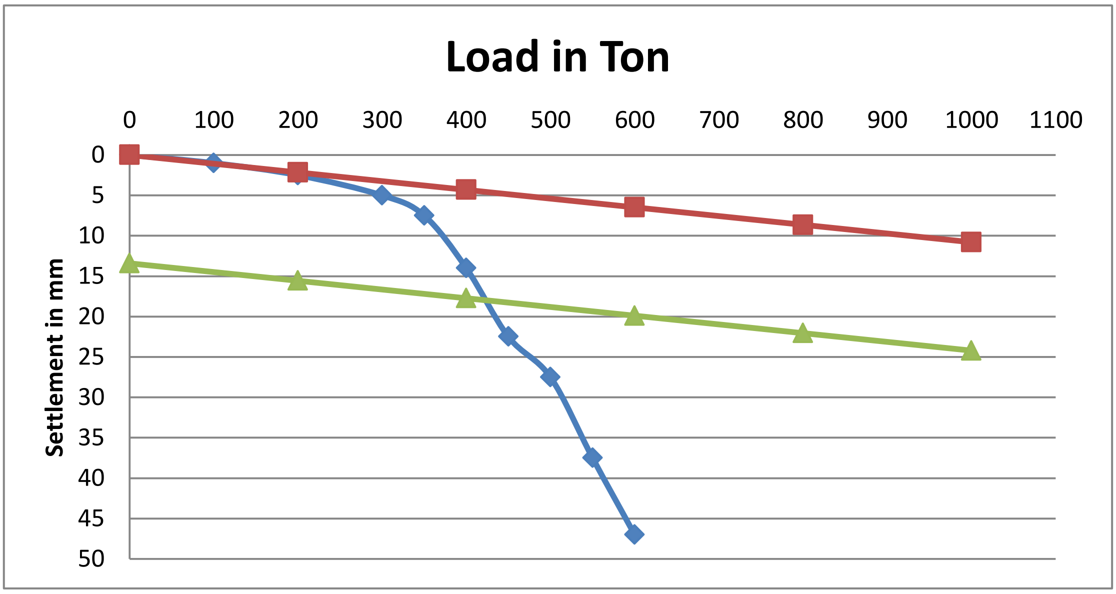
Kumar Bridge (P-1) CTP-07, LT



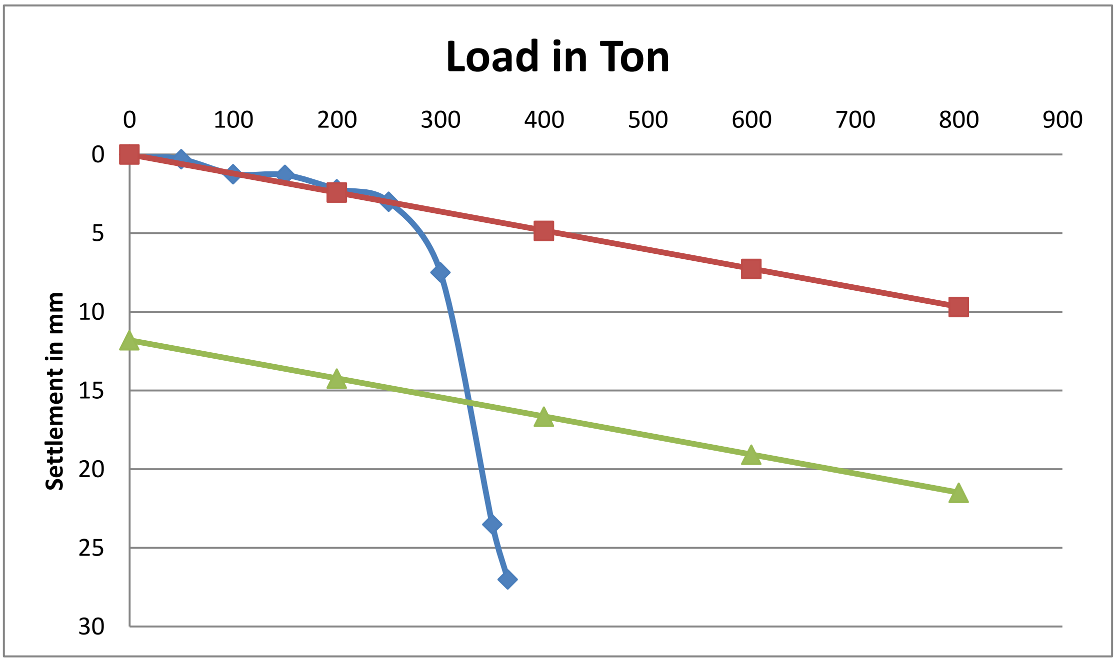
MRT CTP-09, LT



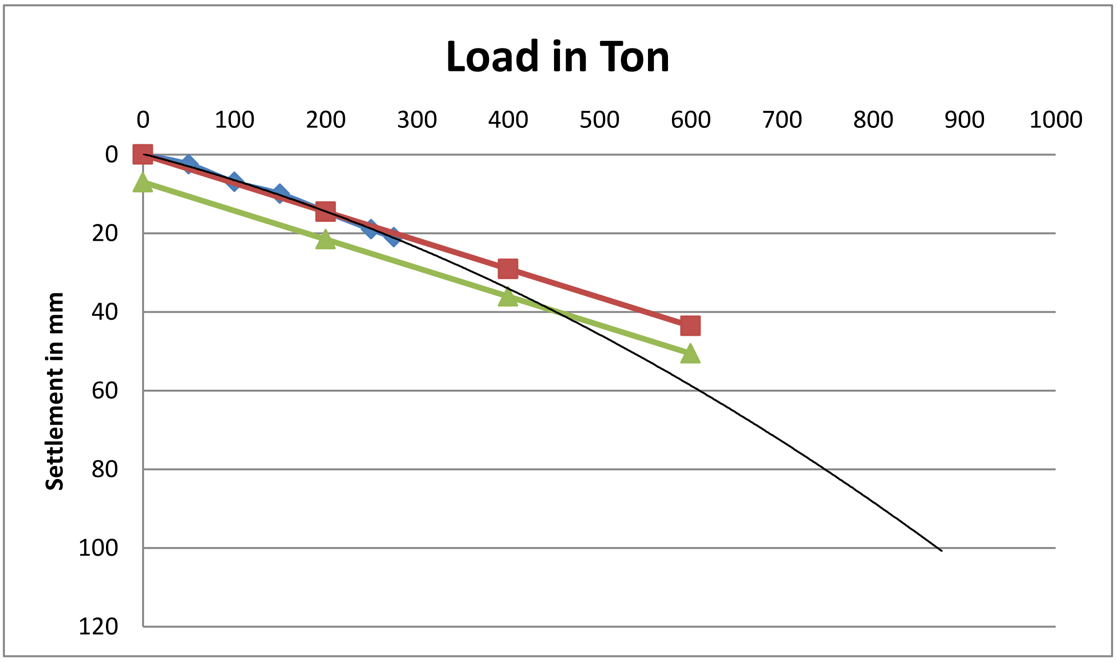
Postogola UP (A-1, TP1) CTP-12, LT Fail



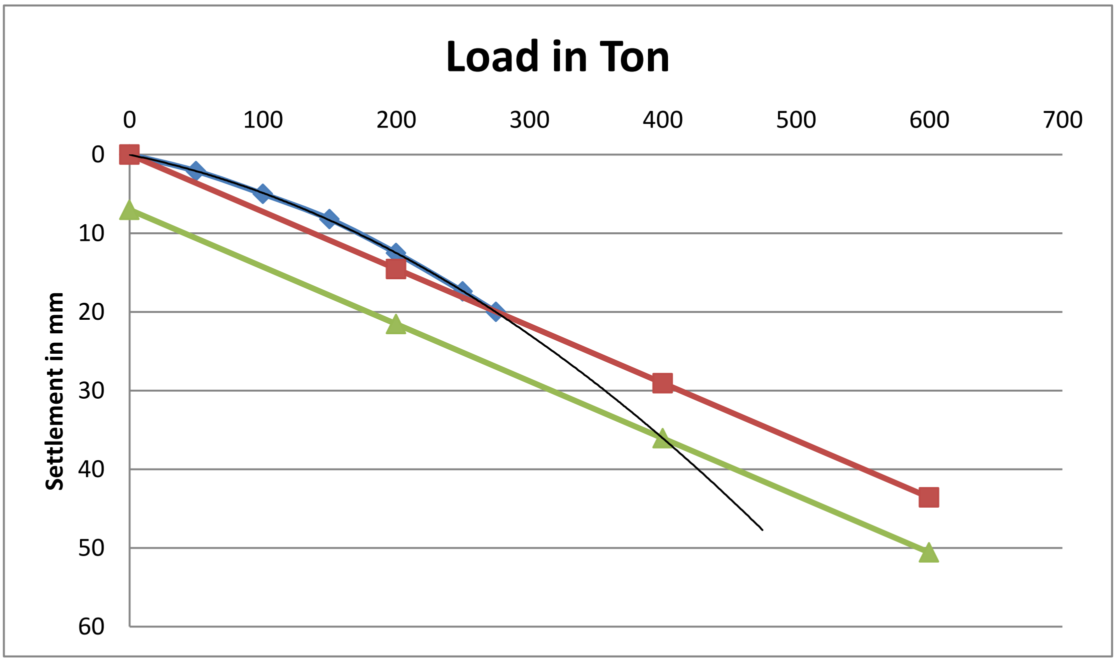
Shibpur Bridge, (A-2), CTP-15, LT – Fail



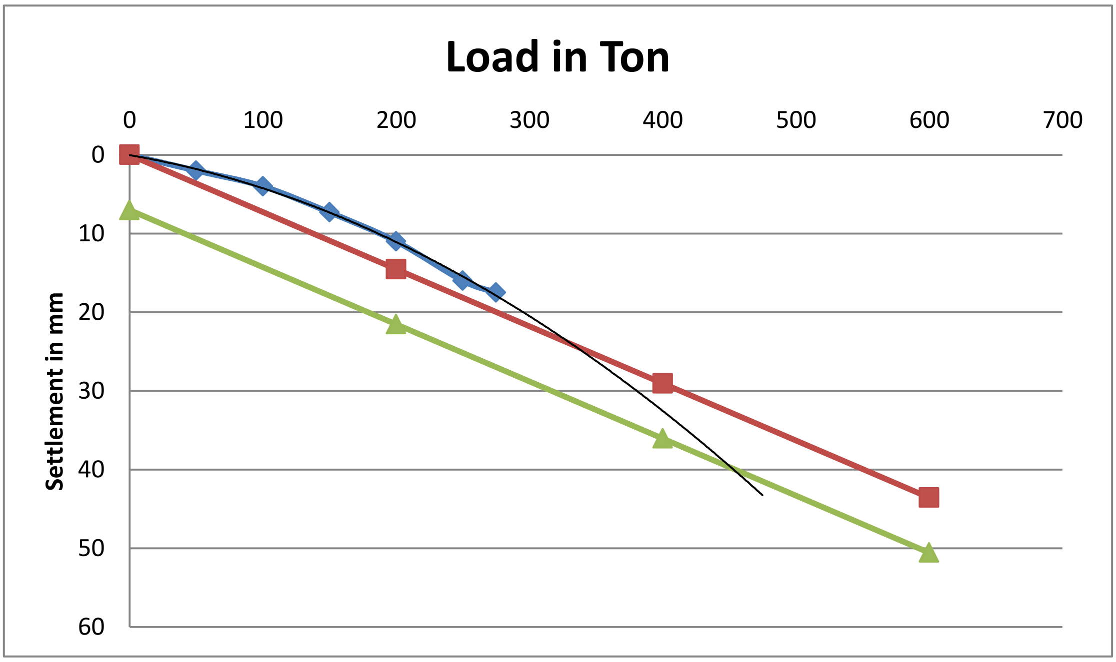
Uttara Apartment, PTP-3



Uttara Apartment, PTP-4



Uttara Apartment, PTP-5



APPENDIX D
SAMPLE CALCULATION

Kalshi, TP-2, BH-13-C1

Method of analysis: Meyerhof (1976)

Side friction

Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***	***fl (ksf)***	**As (ft^2)**	**Qs (kip)**
22.14'-61.5'	Silty sand	39.36	15	Medium Dense		0.33	1.10	405.38	134.02
61.5'-65.6'	stiff clay	4.1	9	1.125	0.36	*0.405*		42.22672	17.10182
65.6'-86.10'	stiff clay	20.5	14	1.75	0.36	*0.63*		211.1336	133.0142
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**		***fs***	***fl***	**As**	**Qs** (kip)
86.1'-87.5	Silty sand	1.4	48	Dense		1.06	1.10	14.42	15.25
87.5'-103.32'	Silty sand	15.82	48	Dense		1.06	1.10	162.93	172.37
Total side friction, Q$_s$									***483.24***

End bearing

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**		**qB(Ksf)**	**Pa** (ksf)	**qL**	**AB**	**QB** (kip)
87.5'-103.32'	Silty sand	17.22	48	30.9747572		47.67	2	140.70	8.45	**402.58**
Ultimate axial capacity, Qult										***885.82***

Method of analysis: NAVFAC DM 7.2(1986)												
Side friction												
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**C_A/c**	**C_A**		***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
4.92'-18.36'	**Soft clay**	**13.44**	**1**	mid. stiff	0.125	1.20	0.15		0.15		138.42	20.76
18.36'-22.14'	Mid. Stiff clay	3.78	3	mid. Stiff	0.375	1.05	0.39375		0.39		38.93	15.33
Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**Ncor**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
22.14'-61.5'	Silty sand	39.36	15	Medium Dense	13.82632	31	23.25	0.7	2.80	0.84	405.38	341.36
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**C_A/c**	**C_A**		***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
61.5'-65.6'	stiff clay	4.1	9	stiff	1.125	0.7	0.7875		0.79		42.23	33.25
65.6'-86.10'	stiff clay	20.5	14	stiff	1.75	0.55	0.9625		0.96		211.13	203.22
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**Ncor**	**ϕ**	**δ**	**Ks**	**σv(f)**	***fs***	**As**	**Qs (kip)**
86.1'-87.5	Silty sand	1.4	48	Dense	38.63987	38	28.5	0.7	3.97	1.51	14.42	21.76
87.5'-103.32'	Silty sand	15.82	48	Dense	38.63987	38	28.5	0.7	3.97	1.51	162.93	245.85
Total side friction, Q_s												***881.52***
End bearing												
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**Nq**	**σv**(ksf)	**qB**		**AB**		**QB** (kip)
87.5'-103.32'	Silty sand	17.22	48	38.005451	36	30	3.97	119.10		8.45		**1005.84**
Ultimate axial capacity, Qult												***1887.36***

Method of analysis:AASHTO (1986)											
Side friction											
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***		***fsz (ksf)***	**As (ft^2)**	**Qs (kip)**
4.92'-18.36'	Soft clay	13.44	1	soft	**0.125**	**0.55**	0.07		6.06	138.42	9.52
18.36'-22.14'	Mid. Stiff clay	3.78	3	mid. Stiff	0.375	0.55	0.21		6.06	38.93	8.03
Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**ϕ**	**β**	**σv**(ksf)	***fs(ksf)***	***fsl(ksf)***	**As**	**Qs** (kip)
22.14'-61.5'	Silty sand	39.36	15	Medium Dense	31	**0.63**	2.80	1.76	4.41	405.38	715.08
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***		***fsz(ksf)***	**As (ft^2)**	**Qs (kip)**
61.5'-65.6'	stiff clay	4.1	9	stiff	1.125	0.55	0.62		6.06	42.2267	26.13
65.6'-86.10'	stiff clay	20.5	14	stiff	1.75	0.55	0.96		6.06	211.134	203.22
Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**ϕ**	**β**	**σv**(ksf)	***fs(ksf)***	***f_{sl}(ksf)***	**As**	**Qs** (kip)
86.1'-87.5	Silty sand	1.4	48	Dense	38.00	0.25	5.29	1.33	4.41	14.42	19.15
87.5'-103.32'	Silty sand	15.82	48	Dense	38.00	0.25	5.34	1.34	4.41	162.93	217.52
Total side friction, Q_s											***1198.64***

<table>
<tr><td colspan="14"><i>Method of analysis: O'Neill & Reese (1988)</i></td></tr>
<tr><td colspan="14"><i>Side friction</i></td></tr>
<tr><td>Layer Range</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>c (ksf)</td><td colspan="3">α</td><td colspan="3"><i>fs (ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>4.92'-18.36'</td><td>Soft clay</td><td>13.44</td><td>1</td><td>soft</td><td>0.125</td><td colspan="3">0.55</td><td colspan="3">0.07</td><td>138.42</td><td>9.52</td></tr>
<tr><td>18.36'-22.14'</td><td>Mid. Stiff clay</td><td>3.78</td><td>3</td><td>mid. Stiff</td><td>0.375</td><td colspan="3">0.55</td><td colspan="3">0.21</td><td>38.93</td><td>8.03</td></tr>
<tr><td>Layer Range</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>ϕ</td><td>Φ' =δ</td><td>δ = Φ</td><td>K</td><td>σv (ksf)</td><td><i>fs(ksf)</i></td><td><i>fsl(ksf) below 87.5'</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>22.14'-61.5'</td><td>Silty sand</td><td>39.36</td><td>15</td><td>Medium Dense</td><td>31</td><td>38.3</td><td>30</td><td>0.8789</td><td>2.8</td><td>1.42</td><td></td><td>405.38</td><td>575.98</td></tr>
<tr><td>Layer Range</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>c (ksf)</td><td colspan="3">α</td><td colspan="3"><i>fs (ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>61.5'-65.6'</td><td>stiff clay</td><td>4.1</td><td>9</td><td>stiff</td><td>1.125</td><td colspan="3">0.55</td><td colspan="3"><i>0.61875</i></td><td>42.22672</td><td>26.13</td></tr>
<tr><td>65.6'-86.10'</td><td>stiff clay</td><td>20.5</td><td>14</td><td>stiff</td><td>1.75</td><td colspan="3">0.55</td><td colspan="3"><i>0.9625</i></td><td>228.62</td><td>220.04</td></tr>
<tr><td>Layer Range</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>ϕ</td><td>Φ' =δ</td><td>δ = Φ</td><td>K</td><td>σv(ksf)</td><td><i>fs(ksf)</i></td><td><i>fsl(ksf) below 87.5'</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>86.1'-87.5</td><td>Silty sand</td><td>1.4</td><td>48</td><td>Dense</td><td>37</td><td>42.9</td><td>37</td><td>0.9389</td><td>5.29</td><td>3.74</td><td></td><td>14.42</td><td>53.97</td></tr>
<tr><td>87.5'-103.32'</td><td>Silty sand</td><td>15.82</td><td>48</td><td>Dense</td><td>37</td><td>42.9</td><td>37</td><td>0.93</td><td>5.34</td><td>3.76</td><td></td><td>162.93</td><td>612.08</td></tr>
<tr><td colspan="13">Total side friction, Q_s</td><td><i>1505.74</i></td></tr>
<tr><td colspan="14"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td></td><td></td><td></td><td>q_p</td><td colspan="3">q_l</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>87.5'-103.32'</td><td>Silty sand</td><td>17.22</td><td>48</td><td>Dense</td><td></td><td></td><td></td><td>63.48</td><td colspan="3">66.12</td><td>8.45</td><td>536.07</td></tr>
<tr><td colspan="13"><i>Ultimate axial capacity, Qult</i></td><td><i>2041.81</i></td></tr>
</table>

<table>
<tr><td colspan="11">***Method of analysis: Decourt (1995)***</td></tr>
<tr><td colspan="11">***Side friction***</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height (ft)**</td><td>**N60**</td><td colspan="2">**α**</td><td colspan="2">***fs (ksf)***</td><td>***fl(ksf)***</td><td>**As (ft^2)**</td><td>**Qs (kip)**</td></tr>
<tr><td>22.14'-61.5'</td><td>Silty sand</td><td>39.36</td><td>15</td><td colspan="2"></td><td>0.5</td><td>0.54</td><td></td><td>405.38</td><td>220.07</td></tr>
<tr><td>**61.5'-65.6'**</td><td>**stiff clay**</td><td>**4.1**</td><td>**9**</td><td colspan="2">**1**</td><td colspan="2">0.73</td><td></td><td>42.23</td><td>31.04</td></tr>
<tr><td>**65.6'-86.10'**</td><td>**stiff clay**</td><td>**20.5**</td><td>**14**</td><td colspan="2">**1**</td><td colspan="2">1.03</td><td></td><td>211.13</td><td>216.90</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td></td><td>**α**</td><td colspan="2">***fs***</td><td>***fl***</td><td>**As**</td><td>**Qs** (kip)</td></tr>
<tr><td>86.1'-87.5</td><td>Silty sand</td><td>1.4</td><td>48</td><td></td><td>**0.55**</td><td colspan="2">1.66</td><td>2.15</td><td>14.42</td><td>23.91</td></tr>
<tr><td>87.5'-103.32'</td><td>Silty sand</td><td>15.82</td><td>48</td><td></td><td>**0.55**</td><td colspan="2">1.66</td><td>2.15</td><td>162.93</td><td>270.19</td></tr>
<tr><td colspan="10">**Total side friction, Q_s**</td><td>***814.06***</td></tr>
<tr><td colspan="11">***End bearing***</td></tr>
<tr><td>**Layer No.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td colspan="3" rowspan="2"></td><td>**K_B**</td><td>**qB**</td><td>**AB**</td><td>**QB** (kip)</td></tr>
<tr><td>87.5'-103.32'</td><td>Silty sand</td><td>17.22</td><td>48</td><td>0.325</td><td>325.73</td><td>8.45</td><td>**2750.89**</td></tr>
<tr><td colspan="10">***Ultimate axial capacity, Qult***</td><td>***3564.94***</td></tr>
</table>

Kumar Bridge, P1, TP-1, c-1

Method of analysis: Meyerhof (1976)										
Side friction										
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***		***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
14.76'-27.55'	clay	12.81	1	0.125	0.36	0.05		2.00	158.32	7.12
Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**			***fs***	***fl***	**As**	**Qs** (kip)
27.55'-65.60'	Non Plastic Silt	38.05	11	Medium Dense			0.24	1.10	470.26	114.01
65.60'-126.28'	sand	60.68	33	17.0529999			0.73	1.10	749.95	545.45
126.28'-136.77'	Non Plastic Silt	10.49	27	13.9524545			0.60	1.10	129.65	77.15
136.77'-162.69'	sand	25.92	48	24.8043635			1.06	1.10	320.35	338.90
Total side friction, Q$_s$										***1082.64***
End bearing										
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**		**qB(Ksf)**	**Pa** (ksf)	**qL**	**AB**	**QB** (kip)
27.55'-162.69'	sand	135.14	48	22.4721251		226.17	2	140.70	12.16	**1711.14**
Ultimate axial capacity, Qult										***2793.77***

<table>
<tr><th colspan="13">Method of analysis: NAVFAC DM 7.2(1986)</th></tr>
<tr><th colspan="13">Side friction</th></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>c (ksf)</th><th>C_A/c</th><th colspan="2">C_A</th><th colspan="2">fs (ksf)</th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>14.76'-27.55'</td><td>clay</td><td>12.81</td><td>1</td><td>Very Soft</td><td>0.125</td><td>1.18</td><td colspan="2">0.1475</td><td colspan="2">0.15</td><td>158.32</td><td>23.35</td></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>Ncor</th><th>ϕ</th><th>δ</th><th>Ks</th><th>σv (ksf)</th><th>fs</th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>27.55'-65.60'</td><td>Non Plastic Silt</td><td>38.05</td><td>11</td><td>loose</td><td>9.91011</td><td>30</td><td>22.5</td><td>0.7</td><td>2.98</td><td>0.86</td><td>470.26</td><td>406.33</td></tr>
<tr><td>65.62' -78.72</td><td>sand</td><td>13.1</td><td>33</td><td>Dense</td><td>25.9960</td><td>35</td><td>26.25</td><td>0.7</td><td>4.18</td><td>1.44</td><td>161.90</td><td>233.61</td></tr>
<tr><td>78.72'-126.28'</td><td>sand</td><td>47.56</td><td>33</td><td>Dense</td><td>25.2558</td><td>35</td><td>26.25</td><td>0.7</td><td>4.47</td><td>1.54</td><td>587.80</td><td>907.01</td></tr>
<tr><td>126.28'-136.77'</td><td>Non Plastic Silt</td><td>10.49</td><td>27</td><td>Mid Dense</td><td>20.6638</td><td>33</td><td>24.75</td><td>0.7</td><td>4.47</td><td>1.44</td><td>129.65</td><td>187.01</td></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td>Very Dense</td><td>36.7358</td><td>37</td><td>27.75</td><td>0.7</td><td>4.47</td><td>1.65</td><td>320.35</td><td>527.37</td></tr>
<tr><td colspan="12">Total side friction, Q_s</td><td>2051.07</td></tr>
<tr><th colspan="13">End bearing</th></tr>
<tr><th>Layer No</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Ncorr</th><th>ϕ</th><th colspan="2">Nq</th><th>σv (ksf)</th><th colspan="2">qB</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>27.55"-162.69'</td><td>sand</td><td>135.14</td><td>48</td><td>36.735803</td><td>37</td><td colspan="2">38</td><td>4.47</td><td colspan="2">169.86</td><td>12.16</td><td>2065.50</td></tr>
<tr><td colspan="12">Ultimate axial capacity, Qult</td><td>4116.57</td></tr>
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<table>
<tr><th colspan="12"><i>Method of analysis:AASHTO (1986)</i></th></tr>
<tr><th colspan="12"><i>Side friction</i></th></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>c (ksf)</th><th>α</th><th colspan="2"><i>fs (ksf)</i></th><th><i>fsz (ksf)</i></th><th>As (ft²)</th><th>Qs (kip)</th></tr>
<tr><td>14.76'-27.55'</td><td>clay</td><td>12.81</td><td>1</td><td>soft clay</td><td>0.125</td><td>0.55</td><td colspan="2">0.07</td><td>6.06</td><td>158.32</td><td>10.88</td></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>ϕ</th><th>β</th><th>σv (ksf)</th><th><i>fs (ksf)</i></th><th><i>fsl (ksf)</i></th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>27.55'-65.60'</td><td>Non Plastic Silt</td><td>38.05</td><td>11</td><td></td><td>30</td><td>0.58</td><td>2.98</td><td>1.73</td><td>4.41</td><td>470.26</td><td>812.80</td></tr>
<tr><td>65.60'-126.28'</td><td>sand</td><td>60.68</td><td>33</td><td></td><td>35</td><td>0.25</td><td>6.09</td><td>1.52</td><td>4.41</td><td>749.95</td><td>1141.79</td></tr>
<tr><td>126.28'-136.77'</td><td>Non Plastic Silt</td><td>10.49</td><td>27</td><td></td><td>33</td><td>0.25</td><td>8.65</td><td>2.16</td><td>4.41</td><td>129.65</td><td>280.36</td></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td>Dense</td><td>37</td><td>0.25</td><td>9.94</td><td>2.49</td><td>4.41</td><td>320.35</td><td>796.06</td></tr>
<tr><td colspan="11">Total side friction, Q_s</td><td><i>3041.90</i></td></tr>
</table>

<table>
<tr><th colspan="10"><i>End bearing</i></th></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>Bp (inch)</th><th>q_P</th><th>qpr (ksf)</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td>Dense</td><td>47.23</td><td>63.48</td><td>67.20</td><td>12.16</td><td>771.86</td></tr>
<tr><td colspan="9"><i>Ultimate axial capacity, Qult</i></td><td><i>3813.76</i></td></tr>
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<table>
<tr><th colspan="11"><i>End bearing</i></th></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Ncorr</th><th>Nc</th><th>c(ksf)</th><th>q_B (ksf)</th><th>q_p(ksf)</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>1</td><td>Clay</td><td>55</td><td>65</td><td>40</td><td>9</td><td>5</td><td>45</td><td>88.16</td><td>1.33</td><td>59.85</td></tr>
<tr><td colspan="10"><i>Ultimate axial capacity, Qult</i></td><td></td></tr>
</table>

<table>
<tr><td colspan="14"><i>Method of analysis: O'Neill & Reese (1988)</i></td></tr>
<tr><td colspan="14"><i>Side friction</i></td></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Den sity</th><th>c (ksf)</th><th colspan="3">α</th><th colspan="3"><i>fs (ksf)</i></th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>14.76'-27.55'</td><td>clay</td><td>12.81</td><td>1</td><td>very soft</td><td>0.125</td><td colspan="3">0.55</td><td colspan="3">0.07</td><td>158.32</td><td>10.88</td></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Den sity</th><th>ϕ</th><th>Φ'=δ</th><th>δ =Φ</th><th>K</th><th>σv(ksf)</th><th><i>fs (ksf)</i></th><th><i>fsl(ksf) below 87.5'</i></th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>27.55'-65.60'</td><td>Non Plastic Silt</td><td>38.05</td><td>11</td><td></td><td>30</td><td>37.0808127</td><td>30</td><td>0.75271702</td><td>2.98816</td><td>1.30</td><td></td><td>470.26</td><td>610.68</td></tr>
<tr><td>65.6'-87.5'</td><td>sand</td><td>21.9</td><td>33</td><td></td><td>35</td><td>41.4703282</td><td>35</td><td>0.61434619</td><td>4.29</td><td>1.85</td><td></td><td>270.66</td><td>499.48</td></tr>
<tr><td>87.5'-126.28'</td><td>sand</td><td>38.38</td><td>33</td><td></td><td>35</td><td>41.4703282</td><td>35</td><td>0.58201578</td><td>6.0915</td><td>2.48</td><td>2.48</td><td>474.33</td><td>1176.34</td></tr>
<tr><td>126.28'-136.77'</td><td>Non Plastic Silt</td><td>10.49</td><td>27</td><td></td><td>33</td><td>40.6685466</td><td>33</td><td>0.58954014</td><td>8.64519</td><td>3.31</td><td>2.48</td><td>129.65</td><td>321.52</td></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td>Very Dense</td><td>37</td><td>42.9674194</td><td>37</td><td>0.64245621</td><td>9.94</td><td>4.81</td><td>2.48</td><td>320.35</td><td>794.46</td></tr>
<tr><td colspan="13">Total side friction, Q_s</td><td><i>3413.37</i></td></tr>
<tr><td colspan="14"><i>End bearing</i></td></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Den sity</th><th colspan="3"></th><th>q_p</th><th colspan="3">q_l</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td>Very Dense</td><td colspan="3"></td><td>63.48</td><td colspan="3">66.12</td><td>12.16</td><td>771.86</td></tr>
<tr><td colspan="13"><i>Ultimate axial capacity, Qult</i></td><td><i>4185.23</i></td></tr>
</table>

<table>
<tr><th colspan="10"><i>Method of analysis: Decourt (1995)</i></th></tr>
<tr><th colspan="10"><i>Side friction</i></th></tr>
<tr><th>Layer Range.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th></th><th>α</th><th colspan="2"><i>fs (ksf)</i></th><th><i>fl(ksf)</i></th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>14.76'-27.55'</td><td>clay</td><td>12.81</td><td>1</td><td></td><td>1</td><td colspan="2">0.27</td><td>2.00</td><td>158.32</td><td>42.31</td></tr>
<tr><th>Layer Range.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th colspan="2"></th><th>α</th><th><i>fs</i></th><th><i>fl</i></th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>27.55'-65.60'</td><td>Non Plastic Silt</td><td>38.05</td><td>11</td><td colspan="2"></td><td>0.5</td><td>0.43</td><td></td><td>470.26</td><td>200.31</td></tr>
<tr><td>65.60'-126.28'</td><td>sand</td><td>60.68</td><td>33</td><td colspan="2"></td><td>0.5</td><td>1.07</td><td></td><td>749.95</td><td>801.73</td></tr>
<tr><td>126.28'-136.77'</td><td>Non Plastic Silt</td><td>10.49</td><td>27</td><td colspan="2"></td><td>0.5</td><td>0.89</td><td></td><td>129.65</td><td>115.86</td></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td colspan="2"></td><td>0.50</td><td>1.51</td><td>2.15</td><td>320.35</td><td>482.93</td></tr>
<tr><th colspan="10">Total side friction, Q_s</th><th><i>1643.15</i></th></tr>
<tr><th colspan="11"><i>End bearing</i></th></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th colspan="2"></th><th colspan="2">K_B</th><th>qB</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>136.77'-162.69'</td><td>sand</td><td>25.92</td><td>48</td><td colspan="2"></td><td colspan="2">0.325</td><td>325.73</td><td>12.16</td><td>3960.85</td></tr>
<tr><th colspan="10"><i>Ultimate axial capacity, Qult</i></th><th><i>5604.00</i></th></tr>
</table>

MRT PP-08-main

Method of analysis: Meyerhof (1976)									
Side friction									
Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***	***fl* *(ksf)***	**As (ft^2)**	**Qs (kip)**
0'-4.92'	clay	4.92	2	0.25	0.36	0.09	2.00	50.70	4.56
4.92'-24.928'	clay	20	6	0.75	0.36	0.27	2.00	206.08	55.64
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**		***fs***	***fl***	**As**	**Qs** (kip)
24.928'-98.4'	sand	73.47	45	28.1265371		0.99	1.10	757.04	750.84
Total side friction, Q_s									***811.04***
End bearing									
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**qB(Ksf)**	**Pa** (ksf)	**qL**	**AB**	**QB** (kip)
24.928'-98.4'	sand	73.47	45	20.3248404	133.45	2	131.91	8.45	**1114.55**
Ultimate axial capacity, Qult									***1925.59***

<table>
<tr><td colspan="13"><i>Method of analysis: NAVFAC DM 7.2(1986)</i></td></tr>
<tr><td colspan="13"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>c (ksf)</td><td>C_A/ c</td><td colspan="2">C_A</td><td colspan="2"><i>fs (ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>0'-4.92'</td><td>clay</td><td>4.92</td><td>2</td><td>soft</td><td>0.25</td><td>0.6</td><td colspan="2">0.15</td><td colspan="2"><i>0.15</i></td><td>50.6963</td><td>7.60</td></tr>
<tr><td>4.92'-24.928'</td><td>clay</td><td>20</td><td>6</td><td>Stiff</td><td>0.75</td><td>0.6</td><td colspan="2">0.45</td><td colspan="2">0.45</td><td>206.08</td><td>92.74</td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>Ncor</td><td>ϕ</td><td>δ</td><td>Ks</td><td>σv(ksf)</td><td><i>fs</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>24.928'-98.4'</td><td>sand</td><td>73.47</td><td>45</td><td>Dense</td><td>28.12</td><td>39</td><td>29.25</td><td>0.7</td><td>4.20</td><td>1.65</td><td>756.68</td><td>1245.86</td></tr>
<tr><td colspan="12">Total side friction, Q_s</td><td><i>1346.20</i></td></tr>
<tr><td colspan="13"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>ϕ</td><td colspan="2">Nq</td><td>σv (ksf)</td><td colspan="2">qB</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>24.928'-98.4'</td><td>sand</td><td>73.47</td><td>45</td><td>Dense</td><td>36</td><td colspan="2">30</td><td>4.45</td><td colspan="2">133.50</td><td>8.45</td><td>1127.45</td></tr>
<tr><td colspan="12"><i>Ultimate axial capacity, Qult</i></td><td><i>2473.66</i></td></tr>
</table>

Method of analysis:AASHTO (1986)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N 60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***	***fsz (ksf)***	**As (ft^2)**	**Qs (kip)**
0'-4.92'	clay	4.92	2	**soft**	**0.25**	**0**	***0***	6.06	50.7	0
4.92'-24.928'	clay	20	6	stiff	0.75	0.55	0.41	6.06	206.08	85.01

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N 60**	**Density**	**ϕ**	**β**	**σv**(ksf)	***fs(ksf)***	***fsl(ksf)***	**As**	**Qs** (kip)
24.928'-98.4'	sand	73.47	45	Dense	39	0.342	4.20	1.44	4.41	756.68	1086.90
Total side friction, Q_s											***1171.90***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**		**Bp(inch)**	**q_P**	**qpr(ksf)**	**AB**	**QB** (kip)
24.928'-98.4'	sand	73.47	45	Dense		39.36	59.51	75.59	8.45	**502.84**
Ultimate axial capacity, Qult										***1674.75***

<table>
<tr><td colspan="13">***Method of analysis: O'Neill & Reese (1988)***</td></tr>
<tr><td colspan="13">***Side friction***</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height (ft)**</td><td>**N60**</td><td>**Density**</td><td>**c (ksf)**</td><td colspan="3">**α**</td><td colspan="2">***fs (ksf)***</td><td>**As (ft^2)**</td><td>**Qs (kip)**</td></tr>
<tr><td>0'-4.92'</td><td>clay</td><td>4.92</td><td>2</td><td>**soft**</td><td>**0.25**</td><td colspan="3">**0**</td><td colspan="2">***0***</td><td>50.70</td><td>0</td></tr>
<tr><td>4.92'-24.928'</td><td>clay</td><td>20</td><td>6</td><td>Dense</td><td>0.75</td><td colspan="3">0.55</td><td colspan="2">0.41</td><td>206.08</td><td>85.01</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Density**</td><td>**ϕ**</td><td>**δ**</td><td>**δ =ϕ**</td><td>**K**</td><td>**σv**(ksf)</td><td>***fs***</td><td>**As**</td><td>**Qs** (kip)</td></tr>
<tr><td>24.928'-98.4'</td><td>sand</td><td>73.47</td><td>45</td><td>Loose</td><td>39</td><td>42.709</td><td>39</td><td>0.630</td><td>4.2</td><td>2.1</td><td>756.68</td><td>1621.98</td></tr>
<tr><td colspan="12">**Total side friction, Q_s**</td><td>***1706.98***</td></tr>
<tr><td colspan="13">***End bearing***</td></tr>
<tr><td>**Layer No.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Density**</td><td colspan="4" rowspan="2">**q_p59.51**</td><td colspan="2">**q_l**</td><td>**AB**</td><td>**QB** (kip)</td></tr>
<tr><td>24.928'-98.4'</td><td>sand</td><td>73.47</td><td>45</td><td>Dense</td><td colspan="2">66.12</td><td>8.45</td><td>**558.71**</td></tr>
<tr><td colspan="12">***Ultimate axial capacity, Qult***</td><td>***2265.70***</td></tr>
</table>

Method of analysis: Decourt (1995)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**		**α**	***fs (ksf)***	***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
0'-4.92'	clay	4.92	2		**1**	0.33		50.7	16.51441
4.92'-24.928'	clay	20	6		1	0.56	2.00	206.80	115.72
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**		**α**	***fs***	***fl***	**As**	**Qs** (kip)
24.928'-98.4'	sand	73.47	45		0.55	1.56	2.02	756.68	1181.80
Total side friction, Q_s									***1314.04***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**			**K_B**	**qB**	**AB**	**QB** (kip)
24.928'-98.4'	sand	73.47	45			0.325	305.37	8.45	**2580.38**
Ultimate axial capacity, Qult									***3894.41***

PostogolaUP,A-1, TP-1

<table>
<tr><td colspan="12"><i>Method of analysis: Meyerhof (1976)</i></td></tr>
<tr><td colspan="12"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>c (ksf)</td><td colspan="2">α</td><td colspan="2"><i>fs (ksf)</i></td><td><i>fl(ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>8.86'-32.8'</td><td>clay</td><td>23.94</td><td>4</td><td>0.5</td><td colspan="2">0.36</td><td colspan="2">0.18</td><td>2.00</td><td>295.88</td><td>53.26</td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td></td><td></td><td></td><td><i>fs</i></td><td><i>fl</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>32.8'-49.2'</td><td>Non Plastic Silt</td><td>16.4</td><td>33</td><td>Medium Dense</td><td></td><td></td><td></td><td>0.73</td><td>1.10</td><td>202.69</td><td>147.42</td></tr>
<tr><td>49.2'-78.72'</td><td>sand</td><td>29.52</td><td>50</td><td>Dense</td><td></td><td></td><td></td><td>1.10</td><td>1.10</td><td>364.84</td><td>402.05</td></tr>
<tr><td>78.72'-87.5'</td><td>Non Plastic Silt</td><td>8.78</td><td>50</td><td>Dense</td><td></td><td></td><td></td><td>1.10</td><td>1.10</td><td>108.51</td><td>119.58</td></tr>
<tr><td>87.5'-114.16</td><td>sand</td><td>26.66</td><td>50</td><td>Dense</td><td></td><td></td><td></td><td>1.10</td><td>1.10</td><td>329.49</td><td>363.10</td></tr>
<tr><td colspan="11">Total side friction, Q_s</td><td><i>1085.41</i></td></tr>
<tr><td colspan="12"><i>End bearing</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td colspan="2" rowspan="2"></td><td>qB(Ksf)</td><td>Pa (ksf)</td><td>qL</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>87.5'-114.16</td><td>sand</td><td>81.36</td><td>50</td><td>28.7658364</td><td>174.30</td><td>2</td><td>146.57</td><td>12.16</td><td>1782.43</td></tr>
<tr><td colspan="11"><i>Ultimate axial capacity, Qult</i></td><td><i>2867.84</i></td></tr>
</table>

Method of analysis: NAVFAC DM 7.2(1986)

Side friction

Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**C_A/c**	**C_A**	***fs (ksf)***	**As (ft^2)**	**Qs(kip)**
8.86'-32.8'	clay	23.94	4	Soft	0.5	0.95	0.475	0.48	295.88	140.54

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**Ncor**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
32.8'-49.2'	Silty Sand	16.4	33	Medium Dense	28.12	35	26.25	0.7	2.69	0.93	202.69	188.21
49.2'-78.72'	sand	29.52	50	Very Dense		38	28.5	0.7	4.25	1.62	364.84	589.32
78.72'-87.5'	sand	8.78	50	Very Dense		38	28.5	0.7	5.32	2.02	108.51	219.41
87.5'-114.16	sand	26.66	50	Very Dense		38	28.5	0.7	5.32	2.02	329.49	666.22
Total side friction, Q_s												***1803.71***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**ϕ**	**Nq**	**σv**(ksf)	**qB**	**AB**	**QB** (kip)
87.5'-114.16	sand	26.66	50	Very Dense	38	43	5.32	228.76	12.16	**2781.72**
Ultimate axial capacity, Qult										***4585.43***

Method of analysis:AASHTO (1986)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***	***fsz (ksf)***	**As (ft^2)**	**Qs (kip)**
8.86'-32.8'	clay	23.94	4	Soft	0.5	0.55	0.28	6.06	295.88	81.37

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**ϕ**	**β**	**σv**(ksf)	***fs(ksf)***	***fsl(ksf)***	**As**	**Qs** (kip)
32.8'-49.2'	Silty Sand	16.4	33	Medium Dense	35	**0.64**	2.69	1.7216	4.41	202.69	348.95
49.2'-78.72'	sand	29.52	50	Very Dense	38	**0.42**	4.25	1.785	4.41	364.84	651.24
78.72'-87.5'	sand	8.78	50	Very Dense	38	**0.27**	5.63	1.5201	4.41	108.51	164.95
87.5'-114.16	sand	26.66	50	Very Dense	38	0.25	5.95	1.4875	4.41	329.49	490.12
Total side friction, Q_s											***1736.62***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**		**Bp(inch)**	**q_P**	**qpr(ksf)**	**AB**	**QB** (kip)
87.5'-114.16	sand	26.66	50	Very Dense		47.23	66.12	70.00	12.16	**804.02**
Ultimate axial capacity, Qult										***2540.64***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**Nc**	**c(ksf)**	**q_B(ksf)**	**q_p(ksf)**	**AB**	**QB** (kip)
1	Clay	55	65	40	9	5	45	88.16	1.33	59.85
Ultimate axial capacity, Qult										

<table>
<tr><th colspan="14"><i>Method of analysis: O'Neill & Reese (1988)</i></th></tr>
<tr><th colspan="14"><i>Side friction</i></th></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>c (ksf)</th><th colspan="3">α</th><th colspan="3"><i>fs (ksf)</i></th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>8.86'-32.8'</td><td>clay</td><td>23.94</td><td>4</td><td>Soft</td><td>0.5</td><td colspan="3">0.55</td><td colspan="3">0.28</td><td>295.88</td><td>81.37</td></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>ϕ</th><th>Φ'=δ</th><th>δ =Φ</th><th>K</th><th>σv (ksf)</th><th><i>fs (ksf)</i></th><th><i>fsl(ksf below 87.5'</i></th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>32.8'-49.2'</td><td>Silty Sand</td><td>16.4</td><td>33</td><td>Medium Dense</td><td>35</td><td>41.4703282</td><td>35</td><td>1.19884666</td><td>2.69</td><td>2.26</td><td></td><td>202.69</td><td>457.69</td></tr>
<tr><td>49.2'-78.72'</td><td>sand</td><td>29.52</td><td>50</td><td>Very Dense</td><td>38</td><td>43.130524</td><td>38</td><td>0.66728058</td><td>4.25</td><td>2.22</td><td></td><td>364.84</td><td>808.37</td></tr>
<tr><td>78.72'-87.5'</td><td>sand</td><td>8.78</td><td>50</td><td>Very Dense</td><td>38</td><td>43.130524</td><td>38</td><td>0.55943811</td><td>5.63</td><td>2.46</td><td>2.48</td><td>108.51</td><td>267.02</td></tr>
<tr><td>87.5'-114.16</td><td>sand</td><td>26.66</td><td>50</td><td>Very Dense</td><td>38</td><td>43.130524</td><td>38</td><td>0.54051882</td><td>5.95</td><td>2.51</td><td>2.48</td><td>329.49</td><td>827.91</td></tr>
<tr><th colspan="13">Total side friction, Q_s</th><th><i>2442.35</i></th></tr>
<tr><th colspan="14"><i>End bearing</i></th></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><td colspan="3" rowspan="2"></td><th>q_p</th><th colspan="3">q_l</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>87.5'-114.16</td><td>sand</td><td>26.66</td><td>50</td><td>Very Dense</td><td>66.12</td><td colspan="3">66.12</td><td>12.16</td><td>804.02</td></tr>
<tr><th colspan="13"><i>Ultimate axial capacity, Qult</i></th><th><i>3246.37</i></th></tr>
</table>

Method of analysis: Decourt (1995)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**		**α**	***fs (ksf)***	***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
8.86'-32.8'	clay	23.94	4		1	0.44	2.00	295.88	130.97

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**		**α**	***fs***	***fl***	**As**	**Qs** (kip)
32.8'-49.2'	Non Plastic Silt	16.4	33		0.5	1.07		202.69	216.69
49.2'-78.72'	sand	29.52	50		0.5	1.57		364.84	571.34
78.72'-87.5'	Non Plastic Silt	8.78	50		0.5	1.57		108.51	169.93
87.5'-114.16	sand	26.66	50		0.50	1.57	2.24	329.49	515.98
Total side friction, Q_s									***1604.91***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**		**K_B**	**qB**	**AB**	**QB** (kip)
87.5'-114.16	sand	26.66	50		0.325	339.30	12.16	**4125.89**
Ultimate axial capacity, Qult								***5730.80***

Sahbag, CTP-2, c-1

<table>
<tr><td colspan="11"><i>Method of analysis: Meyerhof (1976)</i></td></tr>
<tr><td colspan="11"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>c (ksf)</td><td>α</td><td colspan="2"><i>fs (ksf)</i></td><td><i>fl(ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>9.84'-13.12'</td><td>mid. Stiff clay</td><td>3.28</td><td>8</td><td>1</td><td>0.36</td><td colspan="2">0.36</td><td>2.00</td><td>16.90</td><td>6.08</td></tr>
<tr><td>13.12'-19.68'</td><td>stiff clay</td><td>6.56</td><td>12</td><td>1.5</td><td>0.36</td><td colspan="2">0.54</td><td>2.00</td><td>33.78</td><td>18.24</td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td colspan="2" rowspan="4"></td><td><i>fs</i></td><td><i>fl</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>19.68'-32.80'</td><td>Mid. Dense silt</td><td>13.12</td><td>24</td><td>Mid. Dense</td><td>0.53</td><td>1.10</td><td>67.60</td><td>35.76</td></tr>
<tr><td>32.80'-42.64'</td><td>Mid. Dense silt</td><td>9.84</td><td>24</td><td>Mid. Dense</td><td>0.53</td><td>1.10</td><td>50.70</td><td>26.82</td></tr>
<tr><td>42.64'-70.52'</td><td>Dense sand</td><td>27.88</td><td>42</td><td>Dense</td><td>0.93</td><td>1.10</td><td>143.64</td><td>132.96</td></tr>
<tr><td colspan="10">Total side friction, Q_s</td><td><i>219.86</i></td></tr>
<tr><td colspan="11"><i>End bearing</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td rowspan="2"></td><td>qB(Ksf)</td><td>Pa (ksf)</td><td>qL</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>42.64'-70.52'</td><td>sand</td><td>27.88</td><td>42</td><td>31.833083</td><td>158.63</td><td>2</td><td>123.12</td><td>2.11</td><td>260.06</td></tr>
<tr><td colspan="10"><i>Ultimate axial capacity, Qult</i></td><td><i>479.92</i></td></tr>
</table>

Method of analysis: NAVFAC DM 7.2(1986)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**C_A/ c**	**C_A**	***fs (ksf)***	**As (ft^2)**	**Qs (kip)**
9.84'-13.12'	mid. Stiff clay	3.28	8	mid. Stiff	1	0.76	0.76	0.76	16.90	12.84
13.12'-19.68'	stiff clay	6.56	12	stiff	1.5	0.7	1.05	1.05	33.80	35.49

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Density**	**Ncor**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
19.68'-32.80'	**Mid. Dense silt**	**13.12**	**24**	**Mid. Dense**	24.70306	33	24.75	0.7	2.03	0.66	67.56	44.26
32.80'-42.64'	**Mid. Dense silt**	**9.84**	**24**	**Mid. Dense**	23.35928	33	24.75	0.7	2.4	0.77	50.67	39.25
42.64'-70.52'	Dense sand	27.88	42	**Dense**	40.87874	38	28.5	0.7	2.4	0.91	143.57	130.96
Total side friction, Q_s												***262.80***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**Nq**	**σv**(ksf)	**qB**	**AB**	**QB** (kip)
42.64'-70.52'	Dense sand	27.88	42	40.878741	38	43	2.4	103.20	2.11	**217.89**
Ultimate axial capacity, Qult										***480.69***

<table>
<tr><td colspan="12"><i>Method of analysis:AASHTO (1986)</i></td></tr>
<tr><td colspan="12"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>c (ksf)</td><td>α</td><td colspan="2"><i>fs (ksf)</i></td><td><i>fsz (ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>9.84'-13.12'</td><td>mid. Stiff clay</td><td>3.28</td><td>8</td><td>Mid. Stiff</td><td>1</td><td>0.55</td><td colspan="2">0.55</td><td>6.06</td><td>16.89</td><td>9.29</td></tr>
<tr><td>13.12'-19.68'</td><td>stiff clay</td><td>6.56</td><td>12</td><td>stiff</td><td>1.5</td><td>0.55</td><td colspan="2">0.83</td><td>6.06</td><td>33.78</td><td>27.87</td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td>ϕ</td><td>β</td><td>σv(ksf)</td><td><i>fs(ksf)</i></td><td><i>fsl(ksf)</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>19.68'-32.80'</td><td>Mid. Dense silt</td><td>13.12</td><td>24</td><td>Mid. Dense</td><td>33</td><td>0.81</td><td>2.03</td><td>1.64</td><td>4.41</td><td>67.56</td><td>111.09</td></tr>
<tr><td>32.80'-42.64'</td><td>Mid. Dense silt</td><td>9.84</td><td>24</td><td>Mid. Dense</td><td>33</td><td>0.67</td><td>2.69</td><td>1.80</td><td>4.41</td><td>50.67206</td><td>91.33</td></tr>
<tr><td>42.64'-70.52'</td><td>Dense sand</td><td>27.88</td><td>42</td><td>Dense</td><td>40</td><td>0.48</td><td>3.77</td><td>1.81</td><td>4.41</td><td>143.5708</td><td>259.81</td></tr>
<tr><td colspan="11">Total side friction, Q_s</td><td><i>499.39</i></td></tr>
<tr><td colspan="12"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Density</td><td rowspan="2"></td><td>Bp(inch)</td><td colspan="2">q_P</td><td>qpr(ksf)</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>42.64'-70.52'</td><td>Dense sand</td><td>27.88</td><td>42</td><td>Dense</td><td>19.68</td><td colspan="2">55.54</td><td>141.11</td><td>2.11</td><td>117.27</td></tr>
<tr><td colspan="11"><i>Ultimate axial capacity, Qult</i></td><td><i>616.65</i></td></tr>
</table>

<table>
<tr><td colspan="13">***Method of analysis: O'Neill & Reese (1988)***</td></tr>
<tr><td colspan="13">***Side friction***</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height (ft)**</td><td>**N60**</td><td>**Density**</td><td>**c (ksf)**</td><td colspan="3">**α**</td><td colspan="2">***fs (ksf)***</td><td>**As (ft^2)**</td><td>**Qs (kip)**</td></tr>
<tr><td>9.84'-13.12'</td><td>mid. Stiff clay</td><td>3.28</td><td>8</td><td>mid stiff</td><td>1</td><td colspan="3">0.55</td><td colspan="2">0.55</td><td>16.90</td><td>9.29</td></tr>
<tr><td>13.12'-19.68'</td><td>stiff clay</td><td>6.56</td><td>12</td><td>stiff</td><td>1.5</td><td colspan="3">0.55</td><td colspan="2">0.83</td><td>33.78</td><td>27.87</td></tr>
<tr><td>**Layer Range**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Density**</td><td>**ϕ**</td><td>**δ**</td><td>**δ =ϕ**</td><td>**K**</td><td>**σv**(ksf)</td><td>***fs***</td><td>**As**</td><td>**Qs** (kip</td></tr>
<tr><td>**19.68'-32.80'**</td><td>**Mid. Dense silt**</td><td>**13.12**</td><td>**24**</td><td>**Mid. Dense**</td><td>33</td><td>40.1979434</td><td>33</td><td>0.87104556</td><td>2.03</td><td>1.15</td><td>67.56</td><td>77.58</td></tr>
<tr><td>**32.80'-42.64'**</td><td>**Mid. Dense silt**</td><td>**9.84**</td><td>**24**</td><td>**Mid. Dense**</td><td>33</td><td>40.1979434</td><td>33</td><td>0.74841318</td><td>2.69</td><td>1.31</td><td>50.67</td><td>66.25</td></tr>
<tr><td>42.64'-70.52'</td><td>Dense sand</td><td>27.88</td><td>42</td><td>**Dense**</td><td>40</td><td>42.4338935</td><td>40</td><td>0.64043516</td><td>3.77</td><td>2.03</td><td>143.57</td><td>290.87</td></tr>
<tr><td colspan="12">**Total side friction, Q_s**</td><td>***471.86***</td></tr>
<tr><td colspan="13">***End bearing***</td></tr>
<tr><td>**Layer No.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Density**</td><td colspan="4">**q_p**</td><td colspan="2">**q_l**</td><td>**AB**</td><td>**QB** (kip)</td></tr>
<tr><td>42.64'-70.52'</td><td>Dense sand</td><td>27.88</td><td>42</td><td>Dense</td><td colspan="4">**55.54**</td><td colspan="2">66.12</td><td>2.11</td><td>**117.27**</td></tr>
<tr><td colspan="12">***Ultimate axial capacity, Qult***</td><td>***589.13***</td></tr>
</table>

Method of analysis: Decourt (1995)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**		**α**	***fs (ksf)***	***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
9.84'-13.12'	mid. Stiff clay	3.28	8		1	0.68	2.00	16.89	11.43
13.12'-19.68'	stiff clay	6.56	12		1	0.91		33.78	30.75
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**		**α**	***fs***	***fl***	**As**	**Qs** (kip)
19.68'-32.80'	**Mid. Dense silt**	**13.12**	**24**		0.55	0.89	1.08	67.56	59.90
32.80'-42.64'	**Mid. Dense silt**	**9.84**	**24**		0.55	0.89		50.67	44.92
42.64'-70.52'	Dense sand	27.88	42		0.60	1.60		143.57	229.51
Total side friction, Q_s									***376.51***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**		**K_B**	**qB**	**AB**	**QB** (kip)
42.64'-70.52'	Dense sand	27.88	42		0.325	285.01	2.11	**601.76**
Ultimate axial capacity, Qult								***978.27***

Court Building, Narail, PTP-04

Method of analysis: Meyerhof (1976)									
Side friction									
Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***	***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
4.92-11.48	Mid. Stiff Clay	6.56	5	0.625	0.549125	0.34	2.20	15.06	5.17
11.48-14.76	Mid. Stiff Clay	3.28	5	0.625	0.549125	0.34	2.20	7.53	2.58
14.76-27.88	Mid. Stiff Clay	13.12	4	0.5	0.5325	0.27	2.20	30.12	8.02
Total side friction, Q_s									***15.77***

End bearing											
End bearing											
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**Nc**	**σv**(ksf)	**c(ksf)**	**qB**	**qL(Ncorr)**	**AB**	**QB** (kip)
1	Clay	13.12	4	4.25	9	1.3	0.50	4.5	9.37	0.33	**1.48**
Ultimate axial capacity, Qult											***17.25***

Method of analysis: API (1993)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***	**As (ft^2)**	**Qs (kip)**
4.92-11.48	Mid. Stiff Clay	6.56	5	Miid. Stiff	0.625	0.6	0.38	15.06	5.65
11.48-14.76	Mid. Stiff Clay	3.28	5	Miid. Stiff	0.625	0.69	0.43	7.53	3.25
14.76-27.88	Mid. Stiff Clay	13.12	4	Miid. Stiff	0.5	0.9	0.45	30.12	13.56

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)

Total side friction, Q_s	***22.45***

End bearing

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**Nc**	**σv**(ksf)	**c(ksf)**	**qB**	**AB**	**QB** (kip)
1	Clay	13.12	4	4.205703	9	1.9	0.50	4.5	0.33	1.49

Ultimate axial capacity, Qult	***23.94***

Method of analysis: Tomlinson (1994)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**Ca**	***fs (ksf)***	**As (ft^2)**	**Qs (kip)**
4.92-11.48	Mid. Stiff Clay	6.56	5	soft	0.636	1	0.64	15.06	9.58
11.48-14.76	Mid. Stiff Clay	3.28	5	soft	0.636	1	0.64	7.53	4.79
14.76-27.88	Mid. Stiff Clay	13.12	4	soft	0.502	1	0.50	30.12	15.12

Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs**(kip)

Total side friction, Q_s	***29.49***

End bearing

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**Nc**	**σv**(ksf)	**c(ksf)**	**qB**	**AB**	**QB** (kip)
1	Clay	13.12	4	4.205703	9	1.9	0.50	4.518	0.33	1.49

Ultimate axial capacity, Qult	***30.98***

Method of analysis: Norwegian Pile Guideline(1991) (1993)											
Side friction											
Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**c/σ'**	**α**	***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
4.92-11.48	Mid. Stiff Clay	6.56	5	stiff	0.625	0.69	0.7	0.44		15.06	6.59
11.48-14.76	Mid. Stiff Clay	3.28	5	stiff	0.625	0.53	0.8	0.50		7.53	3.77
14.76-27.88	Mid. Stiff Clay	13.12	4	stiff	0.5	0.31	1	0.50		30.12	15.06
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**σ'$_P$**	**OCR**	**β**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
Total side friction, Q_s											***25.42***
End bearing											
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**Nq**	**σv**(ksf)	**qB**		**AB**	**QB** (kip)
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**Nc**		**σv**(ksf)	**c(ksf)**	**qB**	**AB**	**QB** (kip)
1	Clay	13.12	4	4.205703	9		1.9	0.50	4.5	0.33	1.49
Ultimate axial capacity, Qult											***26.90***

Method of analysis: Indian Standard (2010)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**			**α**		***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
4.92-11.48	Mid. Stiff Clay	6.56	5	0.625			1		0.63		15.06	9.41
11.48-14.76	Mid. Stiff Clay	3.28	5	0.625			1		0.63		7.53	4.71
14.76-27.88	Mid. Stiff Clay	13.12	4	0.5			1		0.50		30.12	15.06
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**		**ϕ** *(from table 2.1)*	**δ** *(from fig 2.3)*	**Ks** *(from fig 2.2)*	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
Total side friction, Q_s												***29.18***

End bearing

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**γ**	**Nc**		**σv**(ksf)	**c (ksf)**	**qB**	**AB**	**QB** (kip)
1	Clay	13.12	4	4.20570299	0.125	9		1.9	0.50	4.50	0.33	**1.49**
Ultimate axial capacity, Qult												***30.67***

Dist. Jail Bldng, Moulovibazar, PTP-05

Method of analysis: Meyerhof (1976)

Side friction

Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***	***fl (ksf)***	**As (ft^2)**	**Qs (kip)**
3.28'-9.84'	soft clay	6.56	3	0.375	0.515875	0.19	2.20	15.06	2.91

Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ** *(from table 2.1)*	**δ** *(from fig 2.3)*	**Ks** *(from fig 2.2)*	**σv**(ksf)	***fs***	***fl***	**As**	**Qs** (kip)
9.84'-11.48'	Mid. Dense sandy silt	1.64	16	22.9902329	38.25	25.5	2.25	0.60	0.64	0.71	3.77	2.42
11.48'-19.68'	Mid. Dense sandy silt	8.20	16	21.1266152	37.78	25.1867	2.25	0.64	0.68	0.71	18.83	12.75
19.68'-27.88'	Dense sandy silt	8.20	33	38.7163951	42.17	28.1133	2.5	0.64	0.85	1.45	18.83	16.09
Total side friction, Q_s												***34.18***

End bearing

Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ** *(from table 2.1)*	**Nq** *(from fig 2.3)*	**σv**(ksf)	**qB**	**Pa** (ksf)	**qL**	**AB**	**QB** (kip)
19.68'-27.88'	Dense sandy silt	8.20	33	36.7323021	41.68	210	0.64	134.40	2	186.97	0.33	**44.28**
Ultimate axial capacity, Qult												***78.46***

Method of analysis: API (1993)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**α**	***fs (ksf)***	**As (ft^2)**	**Qs (kip)**
3.28'-9.84'	soft clay	6.56	3	soft	0.375	0.52	0.20	15.06	2.94

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
9.84'-11.48'	Mid. Dense sandy silt	1.64	16	22.990233	33	20	1	0.60	0.22	3.77	0.82
11.48'-19.68'	Mid. Dense sandy silt	8.20	16	21.126615	33	20	1	0.85	0.31	18.83	5.82
19.68'-27.88'	Dense sandy silt	8.20	33	38.716395	36	25	1	1.32	0.62	18.83	11.59
Total side friction, Q_s											***21.17***

End bearing

Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**δ**	**Nq**	**σv**(ksf)	**qB**	**q_l**	**AB**	**QB** (kip)
19.68'-27.88'	Dense sandy silt	8.20	33	36.732302	25	20	1.58	31.60	100.00	0.33	**10.41**
Ultimate axial capacity, Qult											***31.58***

Method of analysis: Tomlinson (1994)											
Side friction											
Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**Ca**		***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
3.28'-9.84'	soft clay	6.56	3	soft	0.368	1		0.37		15.06	5.54
Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**δ**	**Ks**	**σv**(ksf)	***fs***	**As**	**Qs**(kip)
9.84'-11.48'	Mid. Dense sandy silt	1.64	16	22.990233	34	30.6	0.57	0.60	0.10	3.77	0.38
11.48'-19.68'	Mid. Dense sandy silt	8.20	16	21.126615	33.5	30.15	0.57	0.85	0.14	18.83	2.65
19.68'-27.88'	Dense sandy silt	8.20	33	38.716395	38.2	34.38	0.44	1.32	0.20	18.83	3.74
Total side friction, Q_s											***12.31***
End bearing											
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**Nq**	**σv**(ksf)	**qB(ksf)**	**$q_{l(ksf)}$ peck value**	**AB**	**QB** (kip)
19.68'-27.88'	Dense sandy silt	8.20	33	36.732302	37.5	110	1.58	173.80	229.68	0.33	**57.26**
Ultimate axial capacity, Qult											***69.58***

Method of analysis: Norwegian Pile Guideline(1991) (1993)											
Side friction											
Layer Range	**Layer Type**	**Layer Height (ft)**	**N60**	**Density**	**c (ksf)**	**c/σ'**	**α**	***fs (ksf)***		**As (ft^2)**	**Qs (kip)**
3.28'-9.84'	soft clay	6.56	3	stiff	0.375	0.94	0.55	0.21		15.06	3.11
Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**$σ'_P$**	**OCR**	**β**	**σv**(ksf)	***fs***	**As**	**Qs** (kip)
9.84' - 11.48'	Mid. Dense sandy silt	1.64	16	22.990233	8.92676	14.8779341	1.07563977	0.60	0.65	3.77	2.43
11.48'-19.68'	Mid. Dense sandy silt	8.20	16	21.126615	8.413844	9.89864025	0.73123463	0.85	0.62	18.83	11.70
19.68'-27.88'	Dense sandy silt	8.20	33	38.716395	12.85701	9.74015862	0.72535731	1.32	0.96	18.83	18.03
Total side friction, Q_s											***35.27***
End bearing											
Layer No.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ**	**Nq**	**σv**(ksf)	**qB**		**AB**	**QB** (kip)
19.68'-27.88'	Dense sandy silt	8.20	33	36.732302	36.5	20	1.58	31.60		0.33	**10.41**
Ultimate axial capacity, Qult											***45.68***

<table>
<tr><td colspan="13"><i>Method of analysis: Indian Standard (2010)</i></td></tr>
<tr><td colspan="13"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td colspan="3">c (ksf)</td><td colspan="2">α</td><td colspan="3"><i>fs (ksf)</i></td><td>As (ft</td></tr>
<tr><td>3.28'-9.84'</td><td>soft clay</td><td>6.56</td><td>3</td><td colspan="3">0.375</td><td colspan="2">1</td><td colspan="3">0.38</td><td>15.0</td></tr>
<tr><td></td><td></td><td></td><td></td><td colspan="3"></td><td colspan="2"></td><td colspan="3"></td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td colspan="3"></td><td colspan="2"></td><td colspan="3"></td><td></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td colspan="2">Ncorr</td><td>ϕ <i>(from table 2.1)</i></td><td>δ <i>(from fig 2.3)</i></td><td>Ks <i>(from fig 2.2)</i></td><td>σv(ksf)</td><td colspan="2"><i>fs</i></td><td>As</td></tr>
<tr><td>9.84'-11.48'</td><td>Mid. Dense sandy silt</td><td>1.64</td><td>16</td><td colspan="2">23</td><td>34</td><td>25.50</td><td>1.5</td><td>0.60</td><td colspan="2">0.43</td><td>3.7</td></tr>
<tr><td>11.48'-19.68'</td><td>Mid. Dense sandy silt</td><td>8.20</td><td>16</td><td colspan="2">21.12</td><td>33.5</td><td>25.13</td><td>1.5</td><td>0.64</td><td colspan="2">0.45</td><td>18.8</td></tr>
<tr><td>19.68'-27.88'</td><td>Dense sandy silt</td><td>8.20</td><td>33</td><td colspan="2">38.71</td><td>38.2</td><td>28.65</td><td>2</td><td>0.64</td><td colspan="2">0.70</td><td>18.8</td></tr>
<tr><td colspan="13">Total side friction, Q_s</td></tr>
<tr><td colspan="13"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td>γ</td><td>ϕ <i>(from table 2.1)</i></td><td>Nq <i>(from fig 2.3)</i></td><td>σv(ksf)</td><td>N_γ</td><td>D or B</td><td>qB</td><td>AB</td></tr>
<tr><td>19.68'-27.88'</td><td>Dense sandy silt</td><td>8.20</td><td>33</td><td>36.73</td><td>0.125</td><td>37.5</td><td>80</td><td>0.64</td><td>75.00</td><td>0.574</td><td>53.89</td><td>0.3</td></tr>
<tr><td colspan="13"><i>Ultimate axial capacity, Qult</i></td></tr>
</table>

Islamic Foundation, Dhaka, PTP-19

Method of analysis: Meyerhof (1976)

Side friction

Layer Range.	**Layer Type**	**Layer Height (ft)**	**N60**	**c (ksf)**	**α**	***fs (ksf)***	***fl(ksf)***	**As (ft^2)**	**Qs (kip)**
4.42'-19.68'	Stiff Clay	15.26	14	1.75	0.69875	1.22	2.20	60.06	73.45
19.68'-21.32	Stiff Clay	1.64	14	1.75	0.69875	1.22	2.20	6.46	7.89

Layer Range	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ** *(from table 2.1)*	**δ** *(from fig 2.3)*	**Ks** *(from fig 2.2)*	**σv**(ksf)	***fs***	***fl***	**As**	**Qs** (kip)
21.32'-34.44'	Dense sand	13.12	34	34.2902455	41	27.3333	0.55	1.65	0.47	1.50	51.64	24.22

Total side friction, Q_s	***105.56***

End bearing

Layer Range.	**Layer Type**	**Layer Height** (ft)	**N60**	**Ncorr**	**ϕ** *(from table 2.1)*	**Nq** *(from fig 2.3)*	**σv**(ksf)	**qB**	**Pa** (ksf)	**qL**	**AB**	**QB** (kip)
21.32'-34.44'	Dense sand	13.12	34	32.4030228	41	150	1.65	247.50	2	130.39	0.97	**126.25**

Ultimate axial capacity, Qult	***231.82***

<table>
<tr><td colspan="12">***Method of analysis: API (1993)***</td></tr>
<tr><td colspan="12">***Side friction***</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height (ft)**</td><td>**N60**</td><td>**Density**</td><td>**c (ksf)**</td><td colspan="2">**α**</td><td colspan="2">***fs (ksf)***</td><td>**As (ft^2)**</td><td>**Qs (kip)**</td></tr>
<tr><td>4.42'-19.68'</td><td>Stiff Clay</td><td>15.26</td><td>14</td><td>stiff</td><td>1.75</td><td colspan="2">0.45</td><td colspan="2">0.79</td><td>60.06</td><td>47.30</td></tr>
<tr><td>19.68'-21.32</td><td>Stiff Clay</td><td>1.64</td><td>14</td><td>stiff</td><td>1.75</td><td colspan="2">0.5</td><td colspan="2">0.88</td><td>6.46</td><td>5.65</td></tr>
<tr><td>**Layer Range.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Ncorr**</td><td>**ϕ**</td><td>**δ**</td><td>**Ks**</td><td>**σv**(ksf)</td><td>***fs***</td><td>**As**</td><td>**Qs** (kip)</td></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>34.290246</td><td>38</td><td>30</td><td>1</td><td>2.16</td><td>1.25</td><td>51.64</td><td>64.40</td></tr>
<tr><td colspan="11">**Total side friction, Q_s**</td><td>***117.35***</td></tr>
<tr><td colspan="12">***End bearing***</td></tr>
<tr><td>**Layer No.**</td><td>**Layer Type**</td><td>**Layer Height** (ft)</td><td>**N60**</td><td>**Ncorr**</td><td>**δ**</td><td>**Nq**</td><td>**σv**(ksf)</td><td>**qB**</td><td>**q_l**</td><td>**AB**</td><td>**QB** (kip)</td></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>32.403023</td><td>30</td><td>40</td><td>2.55</td><td>102.00</td><td>200.00</td><td>0.97</td><td>**98.76**</td></tr>
<tr><td colspan="11">***Ultimate axial capacity, Qult***</td><td>***216.11***</td></tr>
</table>

<table>
<tr><th colspan="12"><i>Method of analysis: Tomlinson (1994)</i></th></tr>
<tr><th colspan="12"><i>Side friction</i></th></tr>
<tr><th>Layer Range.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>c (ksf)</th><th colspan="2">Ca</th><th colspan="2"><i>fs (ksf)</i></th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>4.42'-19.68'</td><td>Stiff Clay</td><td>15.26</td><td>14</td><td>Mid stiff</td><td>1.842</td><td colspan="2">0.81</td><td colspan="2">1.49</td><td>60.06</td><td>89.62</td></tr>
<tr><td>19.68'-21.32</td><td>Stiff Clay</td><td>1.64</td><td>14</td><td>Mid stiff</td><td>1.842</td><td colspan="2">0.81</td><td colspan="2">1.49</td><td>6.46</td><td>9.63</td></tr>
<tr><th>Layer Range.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Ncorr</th><th>ϕ</th><th>δ</th><th>Ks</th><th>σv(ksf)</th><th><i>fs</i></th><th>As</th><th>Qs (kip)</th></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>34.290246</td><td>37</td><td>33.3</td><td>0.52</td><td>2.16</td><td>0.37</td><td>51.64</td><td>19.05</td></tr>
<tr><td colspan="11">Total side friction, Q_s</td><td><i>118.30</i></td></tr>
<tr><th colspan="12"><i>End bearing</i></th></tr>
<tr><th>Layer No.</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Ncorr</th><th>ϕ</th><th>Nq</th><th>σv(ksf)</th><th>qB(ksf)</th><th>$q_{l(ksf)}$ peck value</th><th>AB</th><th>QB (kip)</th></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>32.403023</td><td>36.4</td><td>64</td><td>2.55</td><td>163.20</td><td>229.68</td><td>0.97</td><td>158.02</td></tr>
<tr><td colspan="11"><i>Ultimate axial capacity, Qult</i></td><td><i>276.32</i></td></tr>
</table>

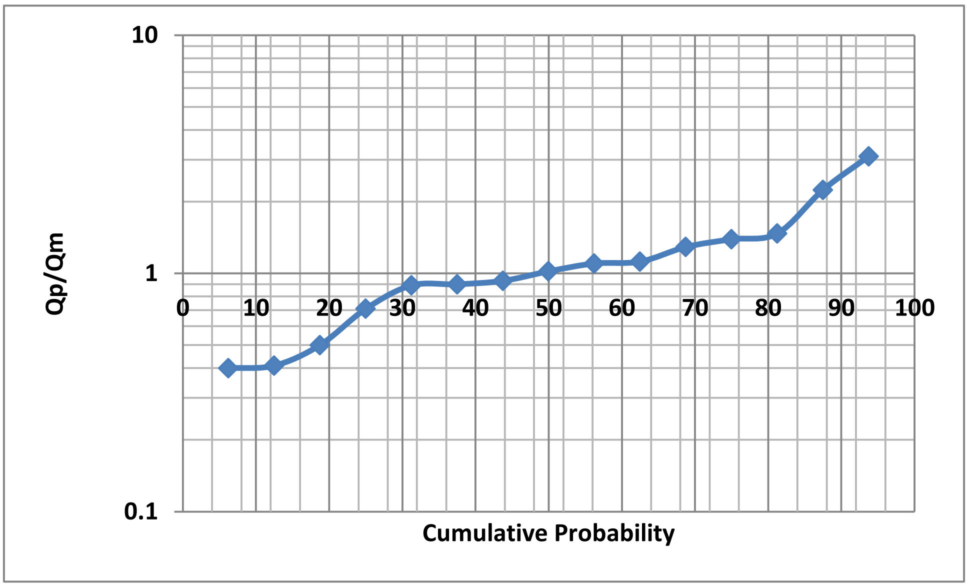
<table>
<tr><th colspan="12"><i>Method of analysis: Norwegian Pile Guideline(1991) (1993)</i></th></tr>
<tr><th colspan="12"><i>Side friction</i></th></tr>
<tr><th>Layer Range</th><th>Layer Type</th><th>Layer Height (ft)</th><th>N60</th><th>Density</th><th>c (ksf)</th><th>c/σ'</th><th>α</th><th colspan="2"><i>fs (ksf)</i></th><th>As (ft^2)</th><th>Qs (kip)</th></tr>
<tr><td>4.42'-19.68'</td><td>Stiff Clay</td><td>15.26</td><td>14</td><td>stiff</td><td>1.75</td><td>1.22</td><td>0.55</td><td colspan="2">0.96</td><td>60.06</td><td>57.81</td></tr>
<tr><td>19.68'-21.32</td><td>Stiff Clay</td><td>1.64</td><td>14</td><td>stiff</td><td>1.75</td><td>1.04</td><td>0.55</td><td colspan="2">0.96</td><td>6.46</td><td>6.21</td></tr>
<tr><td>Layer Range</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td>σ'$_P$</td><td>OCR</td><td>β</td><td>σv (ksf)</td><td><i>fs</i></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>34.290246</td><td>11.80954</td><td>5.46737908</td><td>0.50243901</td><td>2.16</td><td>1.09</td><td>51.64</td><td>56.04</td></tr>
<tr><td colspan="11">Total side friction, Q$_s$</td><td><i>120.07</i></td></tr>
<tr><td colspan="12"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td>ϕ</td><td>Nq</td><td>σv (ksf)</td><td colspan="2">qB</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>21.32' - 34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>32.403023</td><td>41</td><td>35</td><td>2.55</td><td colspan="2">89.25</td><td>0.97</td><td>86.42</td></tr>
<tr><td colspan="11"><i>Ultimate axial capacity, Qult</i></td><td><i>206.48</i></td></tr>
</table>

<table>
<tr><td colspan="14"><i>Method of analysis: Indian Standard (2010)</i></td></tr>
<tr><td colspan="14"><i>Side friction</i></td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td colspan="3">c (ksf)</td><td colspan="2">α</td><td colspan="3"><i>fs (ksf)</i></td><td>As (ft^2)</td><td>Qs (kip)</td></tr>
<tr><td>4.42'-19.68'</td><td>Stiff Clay</td><td>15.26</td><td>14</td><td colspan="3">1.75</td><td colspan="2">0.55</td><td colspan="3">0.96</td><td>60.06</td><td>57.81</td></tr>
<tr><td>19.68'-21.32</td><td>Stiff Clay</td><td>1.64</td><td>14</td><td colspan="3">1.75</td><td colspan="2">0.55</td><td colspan="3">0.96</td><td>6.46</td><td>6.21</td></tr>
<tr><td>Layer Range.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td colspan="2">Ncorr</td><td>ϕ <i>(from table 2.1)</i></td><td>δ <i>(from fig 2.3)</i></td><td>Ks <i>(from fig 2.2)</i></td><td>σv(ksf)</td><td><i>fs</i></td><td></td><td>As</td><td>Qs (kip)</td></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td colspan="2">34.29</td><td>37</td><td>27.75</td><td>1</td><td>1.65</td><td>0.87</td><td></td><td>51.64</td><td>44.83</td></tr>
<tr><td colspan="13">Total side friction, Q_s</td><td><i>108.85</i></td></tr>
<tr><td colspan="14"><i>End bearing</i></td></tr>
<tr><td>Layer No.</td><td>Layer Type</td><td>Layer Height (ft)</td><td>N60</td><td>Ncorr</td><td>γ</td><td>ϕ <i>(from table 2.1)</i></td><td>Nq <i>(from fig 2.3)</i></td><td>σv(ksf)</td><td>$N_γ$</td><td>D or B</td><td>qB</td><td>AB</td><td>QB (kip)</td></tr>
<tr><td>21.32'-34.44'</td><td>Dense sand</td><td>13.12</td><td>34</td><td>32.4</td><td>0.125</td><td>36.5</td><td>65</td><td>1.65</td><td>60.00</td><td>0.984</td><td>110.94</td><td>0.97</td><td>107.42</td></tr>
<tr><td colspan="13"><i>Ultimate axial capacity, Qult</i></td><td><i>216.27</i></td></tr>
</table>

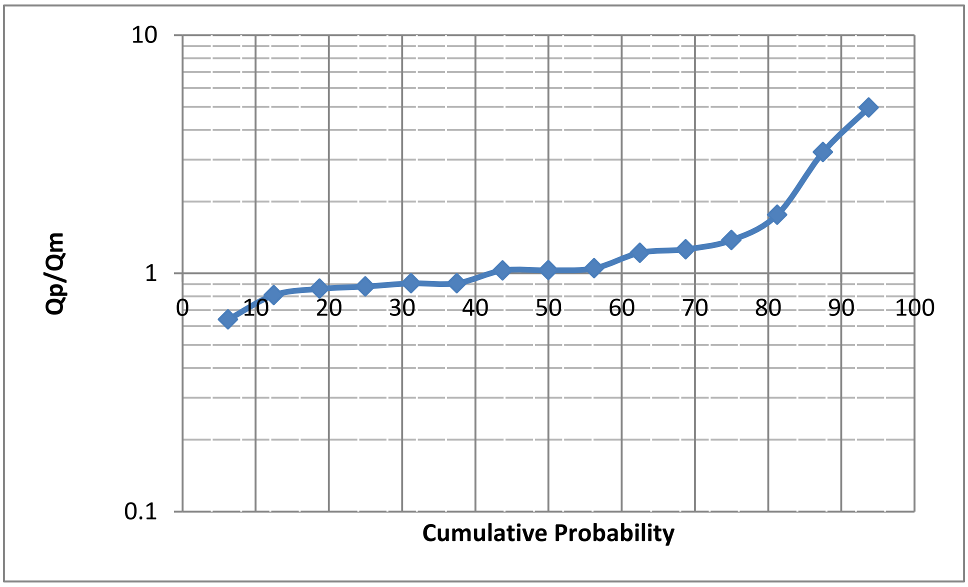
APPENDIX E

Individual Stastical Curve for CTP and PTP OF Different Static Theory

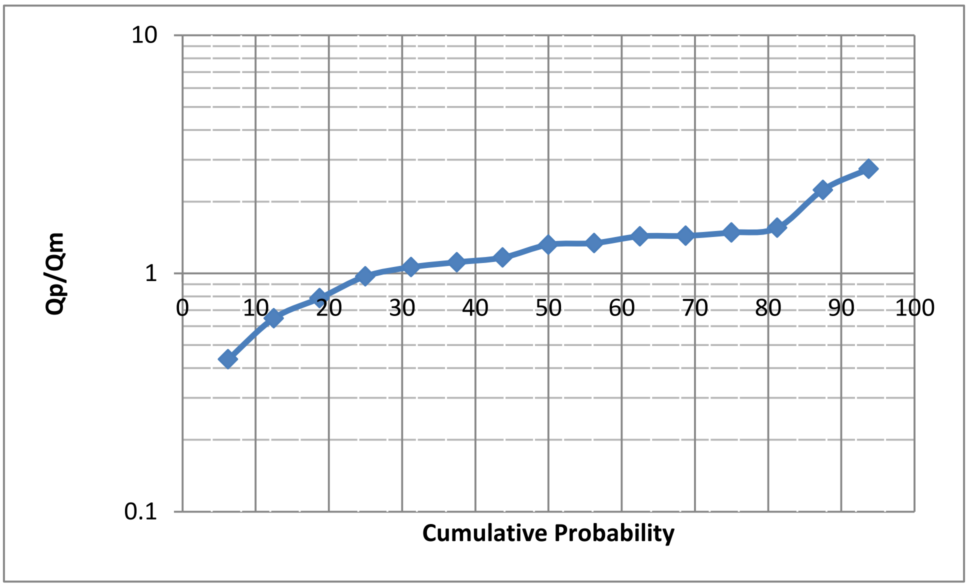
Result and discussion, CTP



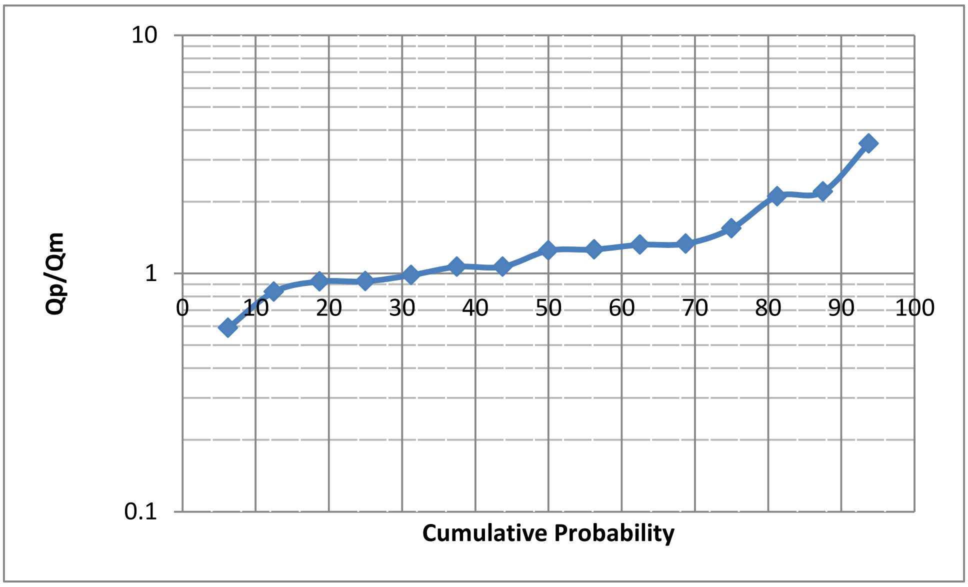
Meyerhof CTP



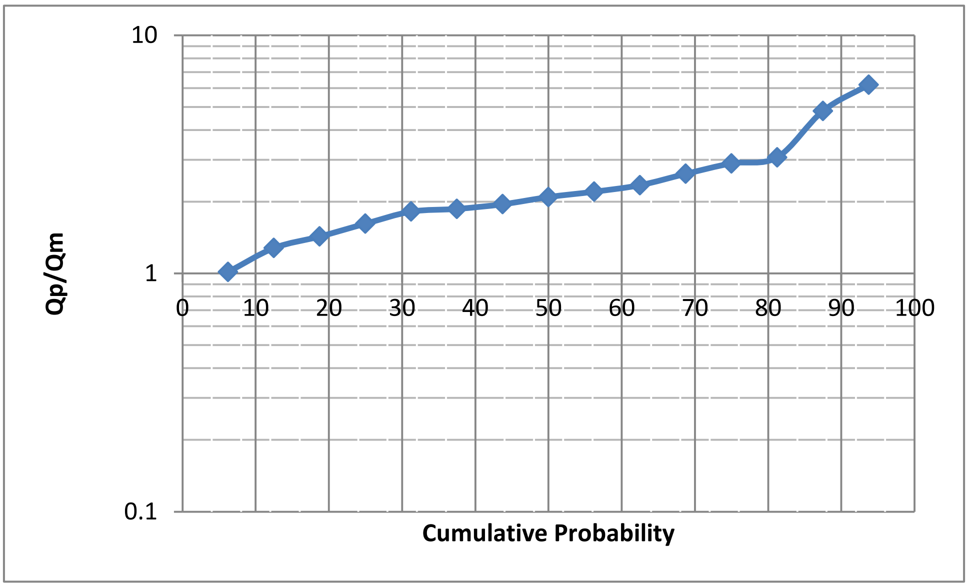
NAVFAC CTP



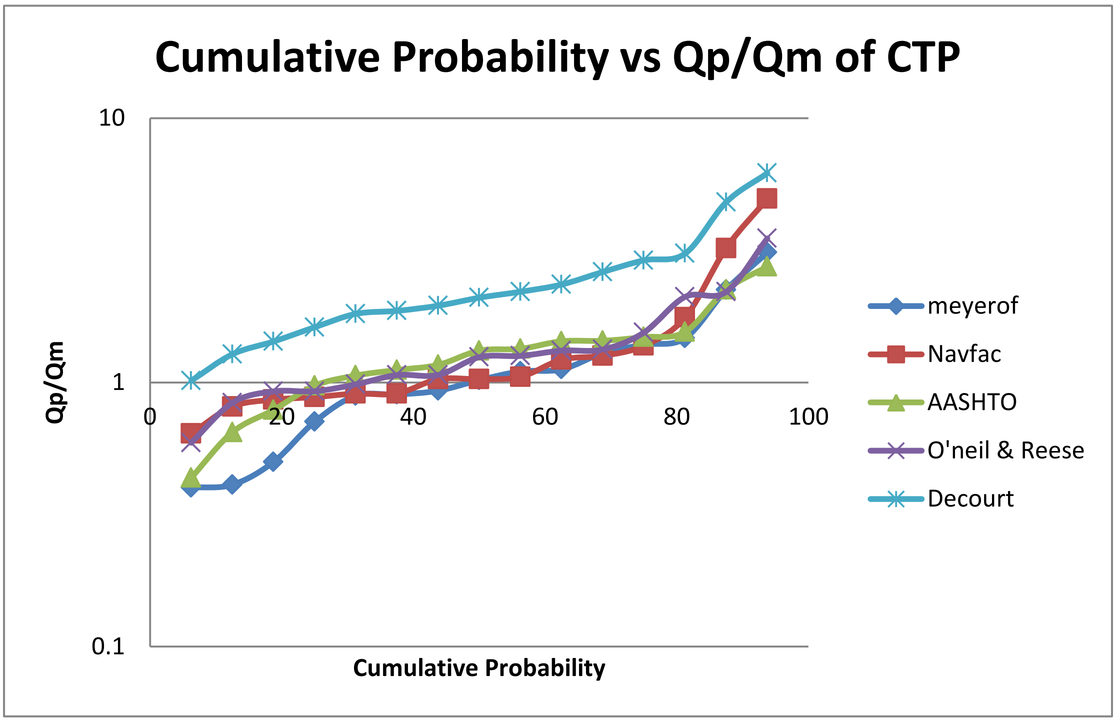
AASHTO CTP



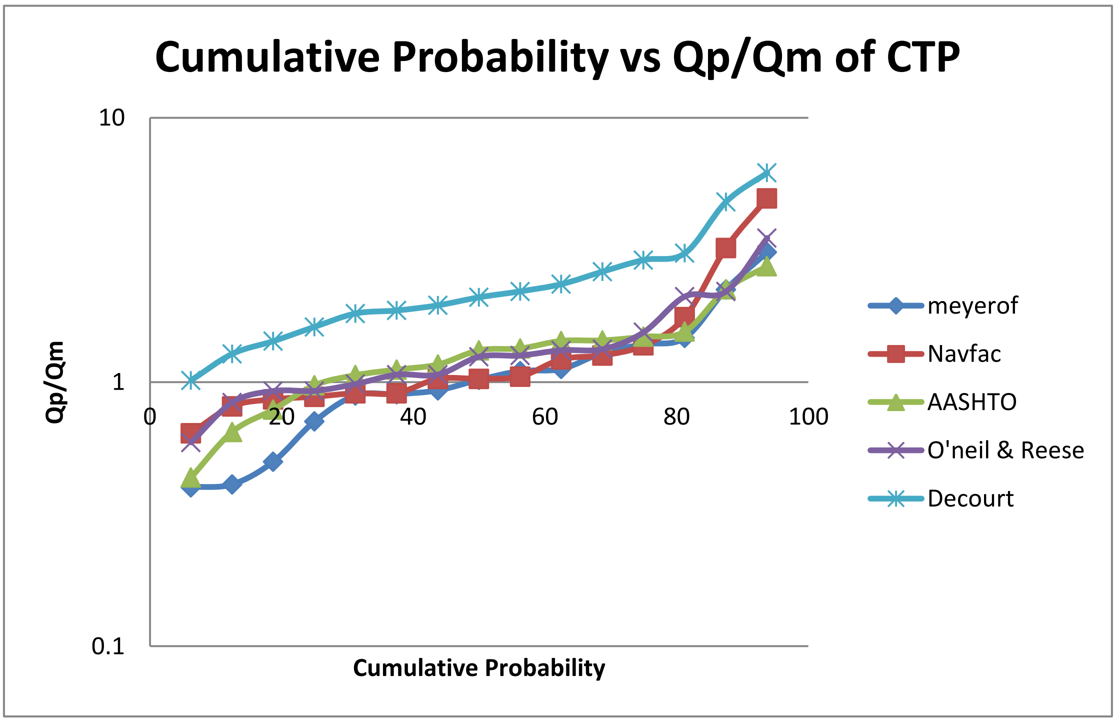
O’neill& Reese CTP



Decourt CTP

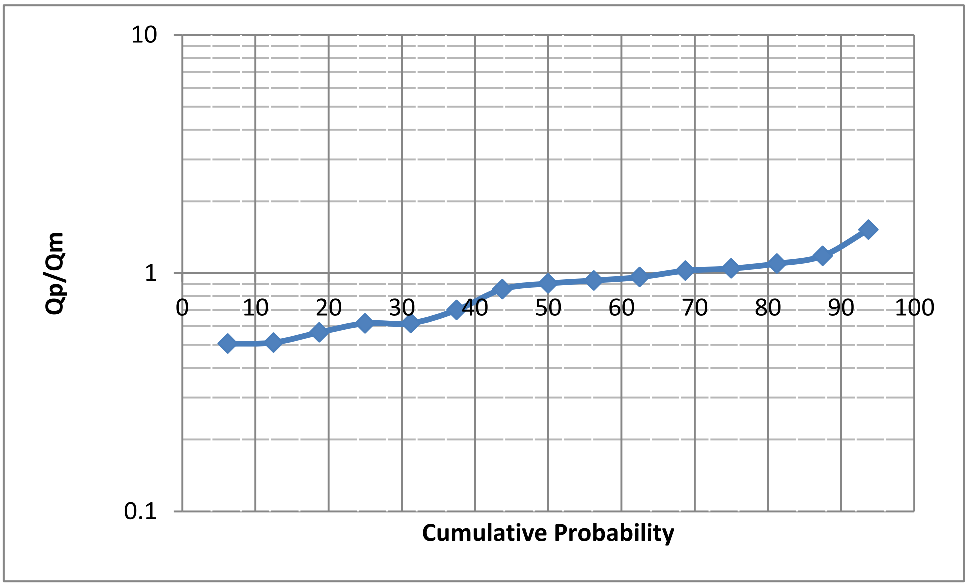


Sheet 2

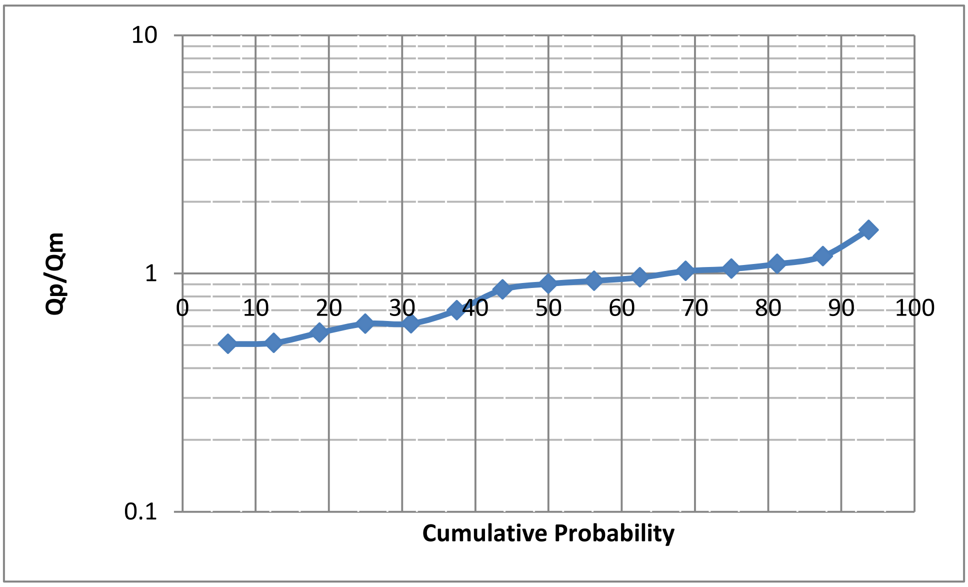

Cumulative Probability vs Qp/Qm of CTP
10
1
0.1
Qp/Qm
0
20
40
60
80
100
Cumulative Probability
meyerof
Navfac
AASHTO
O'neil & Reese
Decourt

Sheet 3

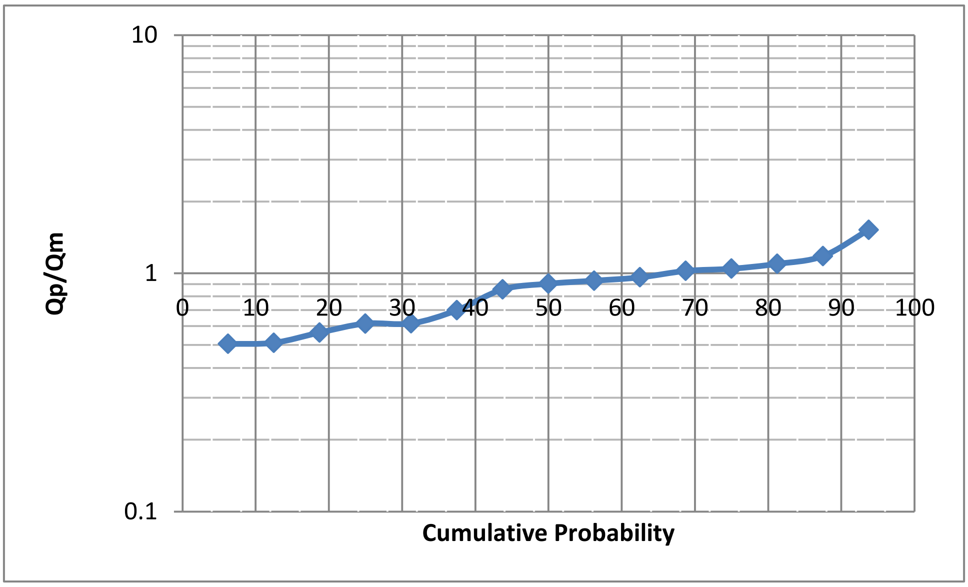
Result and Discussion, PTP



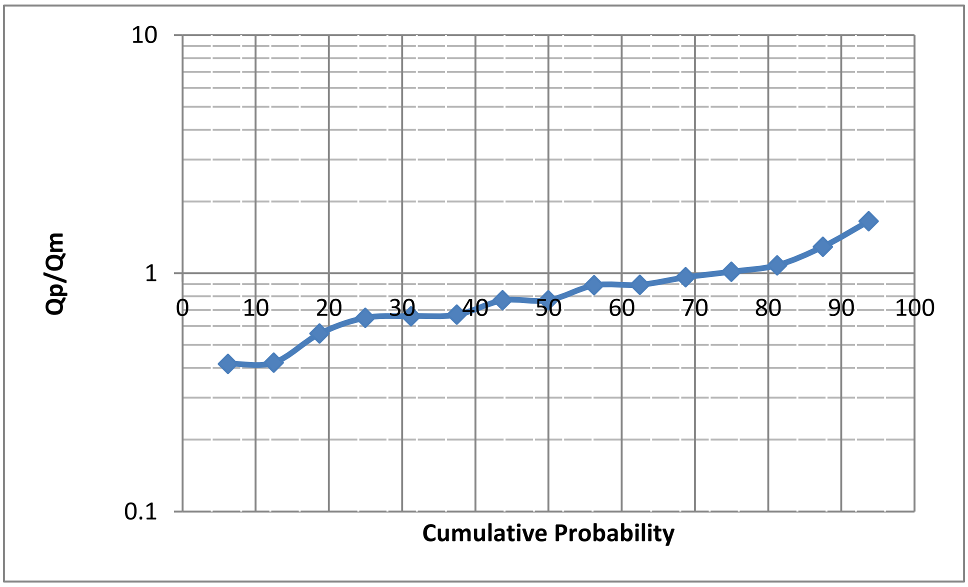
Meyerh of PTP



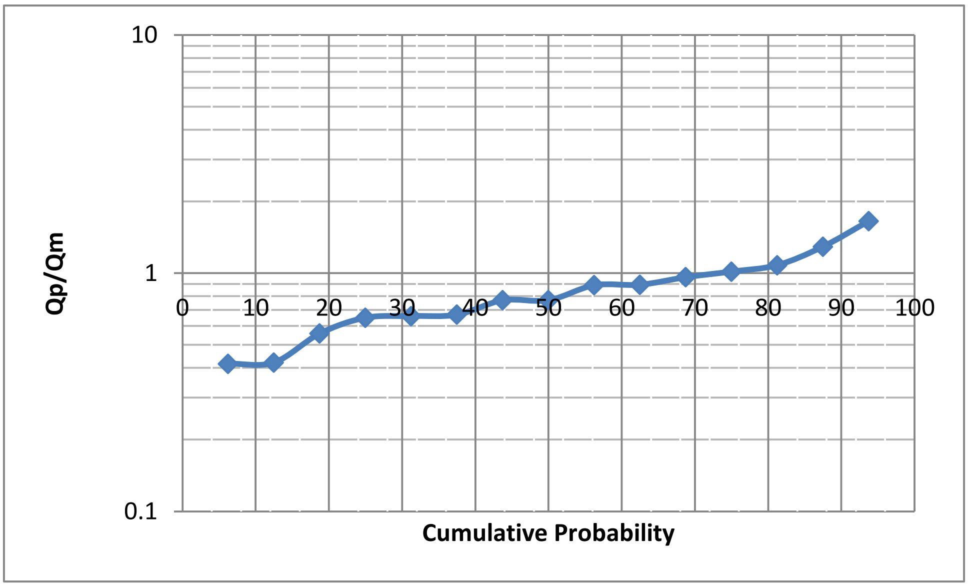
API, PTP



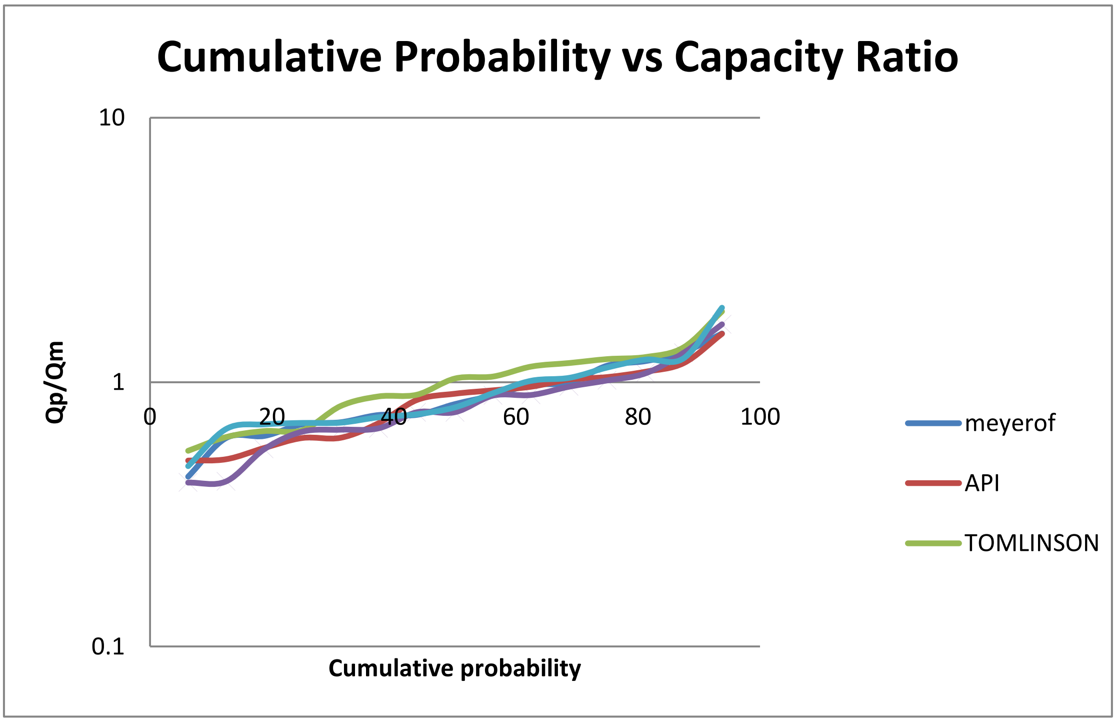
Tomlinson, PTP



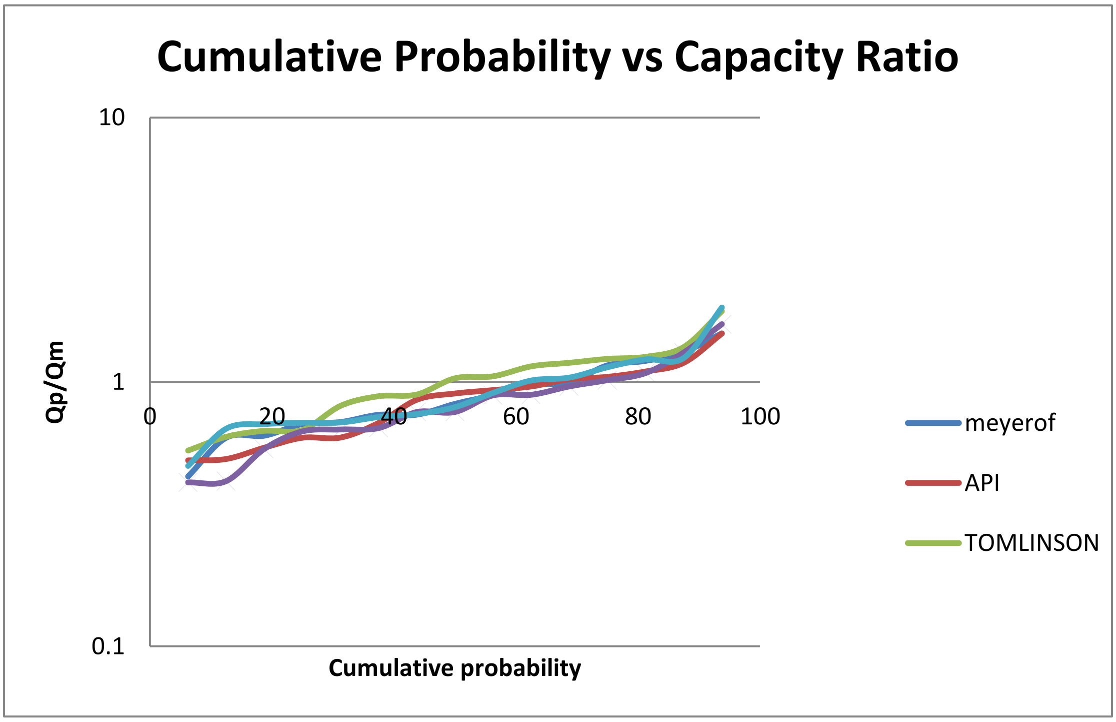
NPG, PTP



Indian Standard, PTP



Sheet 2


Cumulative Probability vs Capacity Ratio
10
1
0.1
Qp/Qm
0
20
40
60
80
100
Cumulative probability
meyerof
API
TOMLINSON