EVALUATION OF DESIGN METHODS FOR UPLIFT CAPACITY OF BORED PILES

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A Thesis Submitted by

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DECLARATION

It is thereby declared that except for the contents where specific reference have been made to the work of others, the study contained in this thesis are the result of investigation carried out by the author under the supervision of Dr. Mohammad Shariful Islam, Associate Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology BUET, Dhaka-1000.

No part of this thesis has been submitted to any other university or other educational establishment for a degree, diploma or other qualification (except for publication).

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Yasser Abou Zeid El Sayed

The thesis titled **"EVALUATION OF DESIGN METHODS FOR UPLIFT CAPACITY OF BORED PILES",** submitted by Yasser Abou Zeid El Sayed, Roll No. 100704253P, Session October 2007 has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master of Science in Civil Engineering on 25 July 2012.

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ABSTRACT

Shaft resistance is a major design factor for piles supporting structures such as transmission towers, harbor structures and offshore platforms. Several studies have been conducted to correlate the estimated capacity of piles with the actual capacity determined from load test in compression. However, few studies focused on the determination of the tension capacity of piles. The main objectives of this study are to compare the estimated tension capacity of bored piles with results obtained from uplift load tests and to evaluate the current design methods used to determine the uplift capacity of bored piles.

In this study, uplift load tests were conducted on 21 bored piles in several sites. Capacity of the piles was determined from load test data. Capacity of the piles was also estimated from the sub-soil characteristics using five methods–Meyerhof (1968), Murthy (1992), Tomlinson (1977), the German Code of Practice (DIN 4014), and British/American Method (1974). The first three methods use two soil parameters–cohesion, c and angle of internal friction, φ. The British/American method uses δ (based on φ) while the German Code of Practice (DIN 4014) uses only SPT N-value for determining the pile capacity.

The results of the pile load test correlate reasonably well with the results estimated from theory. Linear regression of the experimental data showed that the capacity estimated by Meyerhof and Murthy equation and the German Code of Practice (DIN 4014) need to be multiplied by factors of 1.08, 1.22 and 1.28 respectively to get the actual pile capacity. The Tomlinson equation and the British/American method over estimated the pile capacity. The capacity estimated by the Tomlinson equation and the British/American method need to be multiplied by factors of 0.86 and 0.81 respectively to get the actual pile capacity. As far as regression is concerned, the British/American method provided the best regression.

The results of these experiments are based on 21 bored pile load test data. The large scatter in the experimental data suggests that a larger sample size is required for better correlation. Nevertheless, the study conducted in this thesis provides an appropriate ground work for further study in this area.

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NOTATIONS

Symbol	Description
W_n	Natural moisture content
LL	Liquid limit
PL	Plastic limit
PI	Plasticity index
q_u	Unconfined compressive strength
ε_f	Failure strain
G_s	Specific gravity
c	Cohesion
c'	Effective soil cohesion
ϕ'	Effective angle of internal friction
γ	Unit weight of soil
γ'	Submerged unit weight of soil
γ_w	Unit weight of water
F_c	Fines content (% passing sieve # 200)
D_{50}	Mean grain size
σ_n	Normal stress
τ_{max}	Peak shear stress
R	Pile capacity (N)
R_t	Shaft resistance
m, n	Constants depending on type of pile (driven or bored piles)
N	SPT N-value at the pile toe
N′	Average SPT N-value along the pile
N_t	Bearing-capacity coefficient as recommended by Canadian Foundation Engineering Manual
A_t	Pile toe cross sectional area
A_s	Shaft area per unit length of pile
D	Embedment length of the pile in soil
r_t	Unit toe resistance
r_s	Unit shaft resistance along the pile
σ'_D	Unit effective vertical stress at the pile toe
σ'_z	Effective vertical stress at depth z
β	Shaft resistance coefficient = K_s M tanϕ'

K_s	Ratio between the horizontal effective soil stress to the vertical effective soil stress at the pile shaft
ϕ'	Angle of Internal Friction
$\tan \phi'$	Soil friction
$\tan \delta'$	Soil-pile friction
q_s	Design ultimate unit skin friction of an individual pile
K_s	Coefficient of horizontal soil stress
K_0	Coefficient of earth pressure at rest
σ'_{vo}	Average effective overburden pressure over length of the soil layer
δ	Angle of wall friction
Q_b	Base resistance of pile
N_q	Bearing capacity coefficient
A_b	Base area of pile
Q_u	Ultimate axial load capacity of a drilled pier in sand
Q_p	The base resistance
Q_s	The side resistance
K	A factor used to reduce the value of base resistance considering Settlement criterion
C	Circumference of bored pile (ft)
H	Total depth of embedment (ft)
p'	Effective overburden pressure (psf) at mid depth
ϕ'	Effective friction angle of soil
q_i	Base resistance to downward movement of 5% of diameter
α_{avg}	A factor that allows correlation with experimental results
Q_T	In cohesion less soils the ultimate tip resistance in soil
q_{TR}	Ultimate tip resistance for piles reduced for size effects
Q_s	The ultimate side resistance of axially loaded drilled pile in cohesion less soil
B	Diameter of pile(ft)
γ'_i	Effective soil unit weight in i^{th} interval (kcf)
z_i	Depth to midpoint of i^{th} interval (ft)
β_i	Load transfer factor in the i^{th} interval
Δz_i	i^{th} increment of pile length (ft)

Q_u	The total uplift resistance of the pile group
L	The overall length of the group pile
B	The overall width of the group
H	The depth of the block of soil below pile cap level
$\bar{c}_u$	The value of average undisturbed un-drained cohesion of the soil
Q_u	The total uplift resistance of the plate
B	The diameter of the plate
H	Height of the block of soil lifted by the pile
s	Shape factor
D	Depth of the plate
K_u	A constant
ϕ	The angle of shearing resistance of the soil
W	The weight of the soil resisting uplift by the plate.
ER_n	Standard rod energy ratio
n_d	Dynamic efficiency depends on anvil weight (0.87-0.6)
ER_v	Velocity energy ratio
(fs)i	Shaft resistance per unit area of pile surface in the i^{th} layer
(Asp)i	Surface area of the pile shaft in the i^{th} layer
α	adhesion factor
q	Effective overburden pressure at the mid height of the layer
K	Coefficient of lateral earth pressure
δ	Effective friction angle between pile surface and soil in the layer
N_f	Field SPT N-value
χ_m	A constant
N_{60}	The corrected (for overburden) SPT N-value for 60% energy
Q_p	The point resistance
Fs	The shaft resistance in kN/m^2
N	The SPT value
N_p	SPT value of tip
$\overline{N}$	Average SPT on the region of the shaft

Chapter One

INTRODUCTION

1.1 General

Piles are extensively used as deep foundation in unfavourable soil conditions to provide axial and lateral support to structures. Shaft resistance is a major design factor for pile supporting structures such as transmission towers, harbor structures, and offshore platforms. Estimation of shaft resistance depends largely on the correct determination of the sub-soil behaviour. In geotechnical engineering, major advances have been made in understanding the behavior of soils. These advances have stemmed largely from extensive in-situ tests, laboratory tests and sophisticated analysis methods. From many studies, some semi-empirical equations, for example, equations based on SPT N-value, have emerged in realistic analysis and design. However, because of the complexity, it is still very difficult to evaluate the strength and other characteristics of soils and soil-structure interactions. In recent years, especially after the severe earthquakes, geotechnical engineers have learned that it is necessary to find more reasonable methods to estimate the interaction between the soil and foundation from elastic to large deformation levels. Therefore, it is essential to develop new techniques other than the SPT method to evaluate the strength and compressibility characteristics of soils to evaluate the bearing capacity of both the shallow and deep foundations.

However, compared to the element tests in the laboratory, the stresses or the strains in in-situ tests are not simple. The properties of soils determined by in-situ tests may be different from those of the element tests in a laboratory owing to the induced stress that it undergoes, and owing to the type of test. The deformation problems and/or strength problems in in-situ tests are, as a matter of fact, considered as a kind of boundary value problem. Consequently, it is vital to investigate the process of the mobilized strength in in-situ tests corresponding to the failure mechanism.

Regarding in-situ tests, pressure meter tests and torsion meter tests are typically used to estimate the deformation and strength of soils. Such techniques have already been suggested and are applied in practice. Test approaches to evaluate the friction behavior of soil or the coefficient of sub grade shear reaction of soils.

There are many kinds of infrastructures e.g., building, bridge, jetty, tower etc. Now communications towers are widely used all over the world. Also in the country population is very high; that's why the demand for telecommunication mobile has increased abnormally. Therefore, many government and non-government companies are establishing or constructing such structures all over Bangladesh. Foundation of such structures uses mostly concrete piles. For such cases tension load is governed. Practice for designing such foundation, mostly based on available theories that use SPT-results. However, due to lack of information it may be over designed or under designed. There should be some uplift tests on such piles to get actual capacity. Similarly, many offshore structures are being constructed at coastal area or river banks. Estimation of pile capacity in uplift needs to be studied widely.

Some researches have been conducted to correlate the estimated capacity of concrete pile and actual one from load test in compression (Khan, 1997 and Khan, 2002). Although several studies (Krabbenhoft et al., 2006; Shooshpasha et al., 2009) have been taken in different countries, none of the local researches focused on the determination of the tension capacity of piles in local level in Bangladesh. Very few literatures describe the estimation of pile capacity in tension from sub-soil characteristics.

Review of literature reveals that in-depth studies of uplift capacity of piles are important for proper design of foundations where tension load governs. So, this is felt necessary to conduct research to correlate the tension capacity of piles determined from load test with that one estimated from the soil properties using available methods. Thus to evaluate the suitability of the available methods for estimating pile capacity in tension.

1.2 Background of the Study

Piles used in large towers and chimneys or in dry docks can go in tension under uplift loads and overturning moments. It is essential to understand the distribution of shaft resistance with depth for identifying the nature of load transfer between the pile and the soil. However, the quality of a pile depends significantly on the installation or construction technique, on equipment, and on workmanship. Such parameters cannot always be quantified nor taken into account in normal design procedures. Consequently, it is desirable to design bored piles on the basis of test loading of actual foundation units and to monitor construction details to ensure that the design requirements are fulfilled. Only a few projects, however, are large enough to warrant full-scale testing during design phase, and, in most cases, tests (proof-tests) are performed only during or even after construction of the foundation.

Therefore, design methods have been developed to estimate the axial load capacity of piles based on soil parameters and construction procedure. A number of methods are available to estimate the ultimate axial load capacity of bored piles in compression. These methods are based on N-values obtained from Standard Penetration Test (SPT) and on angle of internal friction of sand (such as Canadian Foundation Engineering Manual, 1985; Tomlinson Method, 1995; Reese, 1978 and AASHTO, 1992). Past experience shows that in sands the resistance of pile in tension is only two-third of that of skin friction value in compression. In clays they develop more or less the same skin friction as in compression. As a rule, it can be assumed that the ultimate tension capacity of a friction pile is two-thirds its ultimate skin friction capacity in compression (Varghese, 2005).

In recent times, communications towers are being widely used all over the world. As also the population density in Bangladesh is very high; consequently the demand for mobile phone has increased enormously. Moreover, telecommunication and power transmission towers are being constructed in a large number. For that reason many government and non government companies are constructing such tower structures all over Bangladesh. Foundation of such structures uses mostly bored concrete piles. The design of such pile is generally governed by tension load. Common practices of designing such foundation are mostly based on available theories that use SPT-results.

However, due to the empirical nature of such methods, the pile may be over designed or under designed. It is necessary to conduct uplift tests on such piles to estimate actual tension capacity of piles.

A number of works have been done in different countries to evaluate the tension capacity of piles (Krabbenhoft et al., 2006; Shooshpasha et al., 2009). A few studies have been conducted locally to correlate the estimated capacity of concrete piles with its actual capacity determined from load tests in compression (Khan, 1997 and Khan, 2002, Rahman, 2008). Yasin et al. (2009) reported the case study of the pile capacity in a soft clay deposit of Bangladesh. But none of these local studies have focused on the determination of the tension capacity of piles. Thus it is felt necessary to conduct research to correlate the tension capacity of piles obtained from field tests with that obtained from soil properties.

1.3 Objectives with Specific Aims and Possible Outcome

The main objectives of the research are as follows:

a) To compare the estimated tension capacity of piles with results obtained from uplift load tests.
b) To evaluate the current design methods used to determine the uplift capacity of bored piles.

Possible outcome of this reaches is as follows:

It may result in improvement in current design methods for estimation of uplift capacity of bored piles.

1.4 Outline of Methodology

The whole research is conducted according to the following phases:

a) In this study, two uplift tests were conducted on bored piles at two different sites. At first, geotechnical investigation that includes at least two boreholes up to 30 m depth at each site was conducted. During drilling, SPT was taken at 1.5 m intervals as well as disturbed and undisturbed samples were collected. Detailed laboratory investigations including grain size analysis, Atterberg limit tests, unconfined compression tests and direct shear tests were conducted on the collected samples of various depths of each boring. Sub-soil profile was determined based on the test results.

b) Uplift tests were conducted on several piles (both solid and hollow section) in this study to determine the actual capacity of the pile in tension using the method described in ASTM D 3689 (ASTM, 1989).

c) Other than these uplift tests, load tests data and geotechnical investigation reports were collected from different organizations.

d) Tension capacity of piles were estimated using both SPT-N value and soil parameters based on different theoretical methods such as Tomlinson, or (α) method, The German Code of Practice – DIN 4014, Meyerhof (1956) and Murthy's (1992).

e) Comparisons were made between the estimated tension capacity of piles and actual tension capacity of piles determined from uplift tests.

f) Finally, the results are analyzed to evaluate the available design methods for uplift capacity of both cast-in-situ and precast piles.

1.5 Organization of the Thesis

The complete research work for achieving the stated objectives is divided in number of chapters so that it becomes easier to understand the chronological development of the work. The brief contents of each chapter are presented below:

Chapter One describe the background of this study, objectives and the methodology of the research. Finally, the organization of the thesis is summarized in this chapter.

Chapter Two discuss the types of piles, construction methods of different piles, available formulas and methods of estimating capacity of piles in both tension and compression using sub-soil characteristics. Determination of pile capacity both in tension and compression from load test data are also described.

Sites selected for the research have been discussed in Chapter Three. Experimental program is also discussed here. Brief descriptions of the field and laboratory tests are also provided. Test procedures in tension and compression are also described. Finally, the methods for determining the ultimate capacity of piles from load test data and estimating the piles capacity from soil parameters have been included. A test has scheme has been provided.

Chapter Four presents detail site characteristics such as geographic condition, sub-soil characteristics, and pile load test results. Pile capacity determined from load tests and estimated from the sub-soil properties has been described and discussed elaborately. Pile capacity obtained from load test has been compared with that obtained from equations. Finally, a method has been developed for estimating pile capacity in tension.

Chapter Five includes the conclusions and limitations of the study. Recommendations for future studies have also been listed in this chapter on the basis of present study.

Chapter Two

LITELRATURE REVIEW

2.1 General

The main objective of this chapter is to review the available literature related to axial capacity of piles in uplift and compression. This chapter also deals with past researches that are related to pile capacity in Bangladesh and other parts of the world. Classification of pile types based on material, construction techniques and installation are presented here briefly. Uses of different piles are also discussed in this chapter.

2.2 Types of Pile

The British Standard Code of Practice for Foundations (BS 8004: 1986) places piles in three categories. These are as follows:

Large displacement piles

Comprise solid-section piles or hollow-section piles with a closed end, which are driven or jacked into the ground and thus displace the soil. All types of driven and cast-in-place piles come into this category. Large diameter screw piles and rotary displacement auger piles are increasingly used for piling in contaminated land and soft soils.

Small displacement piles

Small displacement piles are also driven or jacked into the ground but have a relatively small cross-sectional area. They include rolled steel H- or I- sections and pipe or box sections driven with an open end such that the soil enters the hollow section. Where these pile types plug with soil during driving they become large displacement types.

Replacement piles

Replacement piles are formed by first removing the soil by boring using a wide range of drilling techniques. Concrete may be placed into an unlined or lined hole, or the lining may be withdrawn as the concrete is placed. Preformed elements of timber, concrete or steel may be placed in drilled holes. Continuous flight auger (CFA) piles have become the dominant type of pile in the United Kingdome for structures on land.

Euro code 7 (EC7) does not categorize piles, but Clause 7 applies to the design of all types of load-bearing piles. When piles are used to reduce settlement of a raft or spread foundation (e.g. Love, 2003), as opposed to supporting the full load from a structure, then the provisions of EC7 may not apply directly. A basic classification with examples of displacement piles is given in BS EN 12699: 2000 Execution of special geotechnical work – Displacement piles.

Types of piles in each of the BS 8004 categories can be listed as follows:

Large displacement piles (driven types)

The following types of piles are under the large displacement piles (driven types):

a) Timber (round or square section, jointed or continuous)
b) Precast concrete (solid or tubular section in continuous or jointed units)
c) Pre-stressed concrete (solid or tubular section)
d) Steel tube (driven with closed end)
e) Steel box (driven with closed end)
f) Fluted and tapered steel tube
g) Jacked-down steel tube with closed end
h) Jacked-down solid concrete cylinder

Large displacement piles (driven and cast-in-place types)

The following types of piles are under the large displacement piles (*driven and cast-in-place types*):

a) Steel tube driven and withdrawn after placing concrete
b) Steel tube driven with closed end, left in place and filled with reinforced concrete
c) Precast concrete shell filled with concrete
d) Thin-walled steel shell driven by with draw able mandrel and then filled with concrete
e) Rotary displacement auger and screw piles
f) Expander body

Small displacement piles

The following types of piles are under the small displacement piles

a) Precast concrete (tubular section driven with open end)
b) Pre-stressed concrete (tubular section driven with open end)
c) Steel H-section
d) Steel tube section (driven with open end and soil removed as required)
e) Steel box section (driven with open end and soil removed as required)

Replacement piles

The replacement piles are generally

a) Concrete placed in hole drilled by rotary auger, baling, grabbing, airlift or reverse circulation methods (bored and cast-in-place)
b) Tubes placed in hole drilled as above and filled with concrete as necessary
c) Precast concrete units placed in drilled hole
d) Cement mortar or concrete injected into drilled hole
e) Steel sections placed in drilled hole
f) Steel tube drilled down

Composite piles

Numerous types of piles of composite construction may be formed by combining units in each of the above categories or by adopting combinations of piles in more than one category. Thus composite piles of a displacement type can be formed by jointing a timber section to a precast concrete section, or a precast concrete pile can have an H-section jointed to its lower extremity. Composite piles consisting of more than one type can be formed by driving a steel or precast concrete unit at the base of a drilled hole or by driving a tube and then drilling out the soil and extending the drill hole to form a bored and cast-in-place pile.

Again, piles are divided in to the following types based on materials and construction practice:

Steel Piles

Steel piles are generally rolled H-pile used in point bearing. H-pile is available in many sizes, and is designated by the depth of the member and the weight per unit length. For example, an HP 12X74 is an H-pile which is 12 inch deep and weighs 74 pounds per foot. H-piles are well adapted to deep penetration and close spacing due to their relatively small point area and small volume displacement. They can also be driven into dense soils, coarse gravel and soft rock without damage. In some foundation materials, it may be necessary to provide pile points to avoid damage to the pile. In some instances it may become necessary to increase the length of H-Pile by welding two pieces together. Figure 2.1 shows both plugged and unplugged plan view of the steel piles.

If this is the case, splicing must be done in accordance with Guide Lines for Splicing International Building Code, IBC states that splices shall develop not less than 50% of the pile bending capacity. If the splice is occurring in the upper 10 ft of the pile, eccentricity of 3 inch should be assumed for the column load. The splice should be capable of withstanding the bending moment and shear forces due to a 3 inch eccentricity.

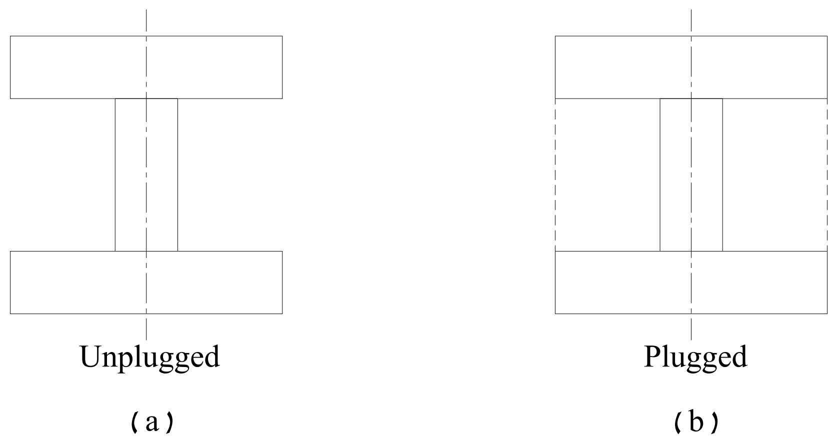


Figure 2.1: Plan view of the steel pile: (a) unplugged and (b) plugged

Cast-in-place pipe pile

Cast-in-place pipe pile is considered as displacement (friction) type pile. Closed-end pipe piles are formed by welding a watertight plate on the end to close the tip end of the pile. The shell is driven into the foundation material to the required depth and then filled with concrete. Thus both concrete and steel share in supporting the load. After the shell is driven and before filling with concrete, the shell is inspected internally its full length to assure that damage has not occurred during the driving operation. Pipe pile may be either spiral or longitudinally welded or seamless steel. Pipe piles are normally used in foundation footings. Their use for above ground pile bents is not recommended. Pipe piles are considered concrete pile for bidding and on the Standard Pile sheet. Figure 2.2 shows both the open and closed end pipe piles.

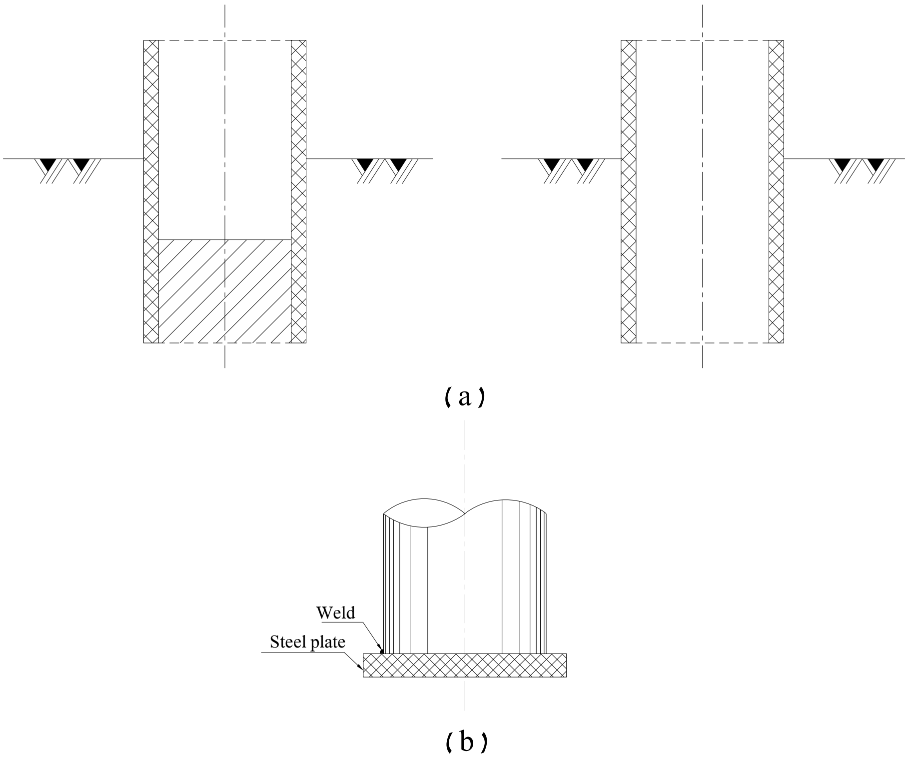


Figure 2.2: Plan view of the pipe pile: (a) open–end pipe pile and (b) closed-end pipe pile

Timber Piles

Timber piles are used for comparatively light axial and lateral loads and where conditions indicate they will not be damaged by driving. Figure 2.3 shows a concrete-timber composite pile. Timber piles are rarely used on permanent bridge structures today, but they are used for temporary structures such as false work construction. Care should be taken when driving false work piling to avoid underground utilities. For permanent installations, untreated timber pile is used below water line (pile will be continually wet) and treated timber at all other locations. Untreated pile may be used on temporary structures. Pile points for timber pile are unnecessary unless hard driving is anticipated.

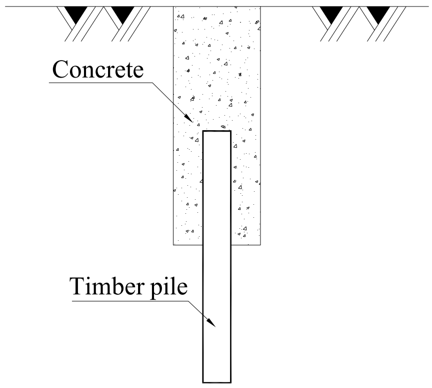


Figure 2.3: Concrete-timber composite pile

Concrete Piles

Concrete piles come in precast, pre-stressed, cast-in-place, or composite construction form. Composite concrete piles are very rarely used in construction and therefore are not discussed in this study.

Precast piles

Precast piles are cast at a production site and shipped to the project site. The Contractor should take special care when moving these piles as not to create tension cracks.

Pre-stressed Piles

Pre-stressed piles are produced in the same manner as a pre-stressed concrete beam. The advantage of pre-stressed piles is their ability to handle large loads while maintaining a relatively small cross section. Also, a pre-stressed pile is less likely to develop tension cracks during handling.

Cast-in-Place-Piles

Cast-in-place pressure grouted piles are constructed by drilling with a continuous-flight, hollow-shaft auger to the required depth. A non-shrinking mortar is then injected, under pressure, through the hollow shaft as the rotating auger is slowly withdrawn. A reinforcing steel cage is placed in the shaft immediately after the auger is withdrawn. When a shell or casing is used the contractor must make sure that the inside of the casing is free of soil and debris before placing the concrete. This system is used when hammer noise or vibration could be detrimental to adjacent footings or structures. This type of pile is commonly used in Bangladesh.

2.3 Selection of Pile Types

Once the geotechnical engineer has decided to use piles, the next question is which piles to be used? Many types of piles are available. Timber piles are cheap but difficult to install in hard soil. However, the use of timber piles is discouraged now a day because of its durability. Steel piles may not be good in marine environments owing to corrosion. The selection of the appropriate type of pile from any of the above categories depends on the following three principal factors:

1) The location and type of structure
2) The ground conditions
3) Durability

Considering the first factor, some form of displacement pile is the first choice for a *marine structure*. A solid precast or pre-stressed concrete pile can be used in fairly shallow water, but in deep water a solid pile becomes too heavy to handle and either a steel tubular pile or a tubular precast concrete pile is used. Steel tubular piles are preferred to H-sections for exposed marine conditions because of the smaller drag forces from waves and currents. Large-diameter steel tubes are also an economical solution to the problem of dealing with impact forces from waves and berthing ships. Timber piles are used for temporary works in fairly shallow water. Bored and cast-in-place piles would not be considered for any marine or river structure unless used in a composite form of construction, say as a means of extending the penetration depth of a tubular pile driven through water and soft soil to a firm stratum.

Piling for a structure on *land* is open to a wide choice in any of the three categories. Bored and cast-in-place piles are the cheapest type where unlined or only partly lined holes can be drilled by rotary auger. These piles can be drilled in very large diameters and provided with enlarged or grout-injected bases, and thus are suitable to withstand high working loads. Augered piles are also suitable where it is desired to avoid ground heave, noise and vibration, i.e., for piling in urban areas, particularly where stringent noise regulations are enforced. Driven and cast-in-place piles are economical for land structures where light or moderate loads are to be carried, but the ground heave, noise and vibration associated with these types may make them unsuitable for some environments.

Timber piles are suitable for light to moderate loadings in countries where timber is easily obtainable. Steel or precast concrete-driven piles are not as economical as driven or bored and cast-in-place piles for land structures. Jacked-down steel tubes or concrete units are used for underpinning work.

The second factor, *ground conditions*, influences both the material forming the pile and the method of installation. Firm to stiff fine-grained soils (silts and clays) favour the augered bored pile, but augering without support of the borehole by bentonite slurry cannot be performed in very soft clays or in loose or water-bearing granular soils, for which driven or driven and cast-in-place piles would be suitable. Piles with enlarged bases formed by auger drilling can be installed only in firm to stiff or hard fine-grained soils or in weak rocks. Driven and cast-in-place piles can neither be used in ground containing boulders or other massive obstructions, nor can they be used in soils subject to ground heave, in situations where this phenomenon must be prevented.

Driven and cast-in-place piles which employ withdraw able tube cannot be used for very deep penetrations because of the limitations of jointing and pulling out of the driving tube. For such conditions a driven pile would be suitable. For hard driving conditions, for example, boulder clays or gravely soils, a thick-walled steel tubular pile or a steel H-section can withstand heavier driving than a precast concrete pile of solid or tubular section.

Some form of drilled pile, such as a drilled-in steel tube, would be used for piles taken down into a rock for the purpose of mobilizing resistance to uplift or lateral loads.

When piling in *contaminated land* using boring techniques, the disposal of arising to licensed tips and measures to avoid the release of aerosols are factors limiting the type of pile which can be considered and can add significantly to the costs. Precautions may also be needed to avoid creating preferential flow paths while piling which could allow contaminated groundwater and leachates to be transported downwards. Hollow tubular steel piles can be expensive for piling in contaminated ground when compared with other displacement piles, but they are useful in overcoming obstructions which could cause problems when driving precast concrete or boring displacement piles. Large displacement piles are unlikely to form transfer conduits for contaminants, although untreated wooden piles may allow 'wicking' of volatile organics. End-bearing H-piles can form long-term flow conduits into aquifers (particularly when a driving shoe is needed) and it may be necessary for the piles to be hydraulically isolated from the contaminated zone.

The factor of *durability* affects the choice of material for a pile. Although timber piles are cheap in some countries they are liable to decay above groundwater level, and in marine structures they suffer damage by destructive mollusk-type organisms. Precast concrete piles do not suffer corrosion in saline water and rich well-compacted concrete can withstand attack from quite high concentrations of sulphates in soils and ground waters. Cast-in-place concrete piles are not so resistant to aggressive substances because of difficulties in ensuring complete compaction of the concrete, but protection can be provided against attack by placing the concrete in permanent linings of coated light-gauge metal or plastics.

Steel piles can have a long life in ordinary soil conditions, if they are completely embedded in undisturbed soil, but the portions of a pile exposed to sea water or to disturbed soil must be protected against corrosion by catholic means, if a long life is required.

2.4 Axial Load Capacity of Pile in Compression

The quality of a pile depends significantly on the installation or construction technique, on equipment, and on workmanship. Such parameters cannot always be quantified nor taken into account in normal design procedures. Consequently, it is desirable to design bored piles on the basis of test loading of actual foundation units and to monitor construction details to ensure that the design requirements are fulfilled.

Only a few projects, however, are large enough to warrant full-scale testing during design phase, and, in most cases, tests (proof-tests) are performed only during or even after construction of the foundation. Therefore, design methods have been developed to estimate the axial load capacity of piles based on soil parameters and construction procedure. However, it is necessary to study the capacity of piles in different geographic locations. Now-a-days, the quality or integrity of piles is evaluated by pile integrity tests (ASTM, 1989). But the capacity of the piles cannot be determined/ evaluated from such tests.

2.4.1 Estimation of Ultimate Axial Load Capacity of Bored Pile

A number of methods are available to estimate the ultimate axial load capacity of bored piles. These methods are based on N-values obtained from Standard Penetration Test (SPT) and on angle of internal friction of sand. Ultimate axial load capacity of a single bored pile can be determined using the following methods:

(i) Method based on the Standard Penetration Test (Canadian Foundation Engineering Manual, 1985)

(ii) Method based on Theory of Plasticity (Canadian Foundation Engineering Manual, 1985)

(iii) Tomlinson Method (1995)

(iv) Method Proposed by Reese et al. (1976) and Reese (1978)

(v) Method Recommended by American Association of State and Transportation Officials, AASHTO (1992)

2.4.1.1 Method Based on the Standard Penetration Test

This method is based on N-values obtained from Standard Penetration Test (SPT). This method has been described in Canadian Foundation Engineering Manual (1985) published by Canadian Geotechnical Society.

The capacity of a single pile in granular soils can be estimated from the results of SPT using the following expression as suggested by Meyerhof (1976).

$$R = m\, N\, A_t + nN'\, D\, A_s \qquad (2.1)$$

where,

R = pile capacity (N)

m, n = constants depending on type of pile (driven or bored piles)

N = SPT N-value at the pile toe

A_t = pile toe area

N' = average SPT N-value along the pile

D = pile embedment length

A_s = pile unit shaft area

The Standard Penetration Test is subject to a multitude of errors, and a lot of care must be exercised when using the test results. For this reason, in this method a minimum factor of safety of 4 should be applied to calculate allowable capacity of a bored pile.

2.4.1.2 Method Based on the Theory of Plasticity

This method has been described in Canadian Foundation Engineering Manual (1985). The capacity of a single pile may be determined from the friction angle of the soil by use of the theory of plasticity (or bearing-capacity theory).

The capacity of a pile in a soil of uniform density increases in a linear manner with increase in effective overburden pressure at least to a certain depth called the critical depth. Investigations of single piles indicate that there is very little increase in toe resistance or unit shaft resistance below the critical depth. The ratio of the critical depth to the pile diameter increases with increase in the angle of shearing resistance. For most applications, the ratio ranges between a value of 7 at $\phi' = 30°$ to a value of 22 at $\phi' = 45°$.

The ultimate static resistance, R of a single pile is a function of the sum of the toe and shaft resistance, R_t and R_s, as follows:

$$R = R_t + R_s \qquad (2.2)$$

$R_t \quad = A_t\, r_t = A_t \sigma'_D\, N_t$ (2.3)

where,

A_t = cross-sectional area of pile at toe

r_t = unit toe resistance = $\sigma'_D\, N_t$

σ'_D = unit effective vertical stress at the pile toe = γ'D (below the critical depth, D = D_c)

γ' = submerged unit weight of soil

D = embedment length of the pile in soil

N_t = bearing-capacity coefficient as recommended by Canadian Foundation Engineering Manual

The expression for shaft resistance is as follows:

$$R_s = \sum_{Z=0}^{D} A_s r_s = \sum_{Z=0}^{D} A_s \beta \sigma'_z = \sum_{Z=0}^{D} A_s M K_s \tan\phi' \sigma'_z \qquad (2.4)$$

where,

A_s = shaft area per unit length of pile

r_s = unit shaft resistance along the pile

σ'_Z = effective vertical stress at depth z (below the critical depth, use σ'_Z)

β = shaft resistance coefficient = K_s M $\tan\phi'$

K_s = ratio between the horizontal effective soil stress to the vertical effective soil stress at the pile shaft

M = $\tan \delta' / \tan \phi'$

$\tan \phi'$ = soil friction

$\tan \delta'$ = soil-pile friction

The value of K_s is influenced by the angle of shearing resistance, the method of installation, the compressibility and original state of stress in the ground, and the size and shape of the pile. It increases with the in-situ density and angle of shearing resistance of the soil and with the amount of displacement. It is higher for displacement-type piles than for low-displacement-type piles such as H-piles. For bored piles, the value of K_s is usually assumed equal to the coefficient of earth pressure at rest, K_0. For driven displacement-type piles, the value of K_s is normally assumed to be twice the value of K_0. The value of M ranges from 0.7 to 1.0, depending on the pile material (steel, concrete, wood) and method of installation (Bozozuk et al., 1978). The combined shaft resistance coefficient, β, is generally assumed to range from 0.3 to 0.8, where the lower value is used in clay and silt, and the higher value in coarse and dense soils (Burland, 1973).

Terzaghi and Peck (1967) reported typical values of angle of internal friction for different types of sands which are shown in Table 2.1.

Table 2.1 Typical values of angle of internal friction for different types of sand (after Terzaghi and Peck, 1967)

Type of Sand	Angle of Internal Friction, φ′	
	Loose	Dense
Uniform sand, rounded particles	27°	35°
Well graded sand, angular particles	33°	45°
Sandy gravels	35°	50°
Silty sands	27° to 30°	30° to 34°
Inorganic silts	27° to 30°	30° to 35°

2.4.1.3 Tomlinson Method

This method of estimating ultimate axial load capacity of a single pile has been described by Tomlinson (1995). In this method, the design ultimate unit skin friction of an individual pile is given by the following expression:

$$q_s = K_s\, \sigma'_{vo} \tan\delta \tag{2.5}$$

where,

K_s = coefficient of horizontal soil stress

σ'_{vo} = average effective overburden pressure over the length of the soil layer

δ = angle of wall friction

The value of coefficient K_s is related to the coefficient of earth pressure at rest (K_0) and also to the method of installation of the piles. Values of coefficient of horizontal soil stress (K_s) are shown in Table 2.2 while values of the angle of pile to soil friction (δ) for various interface conditions are shown in Table 2.3.

The equation for estimating ultimate skin friction implies that in a uniform cohesionless soil the unit skin friction continues to increase linearly with increasing depth. This is not the case. Vesic (1970) showed that at some penetration depth between l0 and 20 pile diameters, a peak value of unit skin friction is reached which is not exceeded at greater penetration depths.

Table 2.2 Values of coefficient of horizontal soil stress, K_s (after Kulhawy, 1984)

Installation method	K_s / K_0
Driven piles, large displacement	1-2
Driven piles, small displacement	0.75-1.75
Bored and cast-in-place piles	0.71-1.0
Jetted piles	0.5-0.7

K_s = coefficient of horizontal soil stress and K_0 = coefficient of earth pressure at rest

Table 2.3 Values of the angle of pile to soil friction (δ) for various interface conditions (after Kulhawy, 1984)

Pile/soil interface condition	Angle of pile to soil friction (δ)
Smooth (coated) steel/sand	$0.5\phi'$ to $0.7\phi'$
Rough (corrugated) steel/sand	$0.7\phi'$ to $0.9\phi'$
Precast concrete/sand	$0.8\phi'$ to $1.0\phi'$
Cast-in-place concrete/sand	$1.0\phi'$
Timber/sand	$0.8\phi'$ to $0.9\phi'$

ϕ'= angle of internal friction

Research has not yet established whether the peak value is a constant in all conditions, or is related to factors such as soil grain size or angularity. A peak value of 110 kN/m^2 has been recommended by Tomlinson (1995) for straight-sided piles.

The base resistance is obtained from the following equation:

$$Q_b = q_b A_b = N_q \sigma'_{vo} A_b \tag{2.6}$$

where,

N_q = bearing capacity coefficient

σ'_{vo} = average effective overburden pressure over the length of the soil layer

A_b = base area of pile

Comparisons of observed base resistances of piles by Nordlund (1963) and Vesic (1964) have shown that N_q values established by Berezantsev (1961) which take into account the depth to width ratio of the pile most nearly conform to practical criteria of pile failure.

2.4.1.4 Method Proposed by Reese et al. (1976) and Reese (1978)

This method has been discussed in detail by a number of investigators (Das; 1984; Bowles, 1988; Cernica, 1995)

The ultimate axial load capacity of a drilled pier in sand may be computed by

$$Q_u = Q_p + Q_s \tag{2.7}$$

Based on Reese et. al. (1976), the base resistance Q_p and side resistance Q_s may be computed as

$$Q_p = \frac{A_b}{K} q_i \tag{2.8}$$

$$Q_s = \alpha_{avg} C \int_o^H p'_z \tan\phi'_z dz \tag{2.9}$$

$$Q_u = \frac{A}{K} q_i + \alpha_{avg} C \int_o^H p'_z \tan\phi'_z dz \tag{2.10}$$

where,

A_b = cross-section of base (ft^2)

K = a factor used to reduce the value of base resistance considering settlement criterion

C = circumference of bored pile (ft)

H = total depth of embedment (ft)

p' = effective overburden pressure (psf) at mid depth

ϕ' = effective friction angle of soil

q_i = base resistance to downward movement of 5% of diameter

= 0 for loose sand (relative density designation)

= 32,000 psf for medium density (relative density designation)

= 80,000 psf for dense formations (relative density designation)

α_{avg} = a factor that allows correlation with experimental results

α_{avg} = 0.7 for H < 25 ft

= 0.6 for H = 25 ft to 40 ft

= 0.5 for H > 40 ft

dz = differential element of length (ft)

2.4.1.5 Method Recommended by AASHTO (1992)

This method has been described in detail by American Association of State Highway and Transportation Officials, AASHTO (1992).

For axially loaded drilled shafts in cohesionless soils the ultimate tip resistance in soil (Q_T) may be estimated using the following expression:

$$Q_T = q_T A_t \tag{2.11}$$

where,

q_T = unit end bearing in ksf

A_t = pile toe area in ft^2

The value of q_T can be determined from the results of Standard Penetration Tests using uncorrected N-value. If B_t (diameter of pile) is greater than 50 inches, the value of q_T: should be reduced to q_{TR}. The expression for q_{TR} is as follows:

$$q_{TR} = \frac{50 q_T}{12 B_T} ksf \tag{2.12}$$

where,

q_{TR} = ultimate tip resistance for piles reduced for size effects

Recommended values of q_T are shown in Table 2.4 (Resse and O'Neill, 1988).

For shafts in cohesionless soils, the ultimate side resistance of axially loaded drilled shafts may be estimated using the following expression:

$$Q_s = \pi B \sum_{i=1}^{N} \gamma_i' z_i \beta_i \Delta z_i \tag{2.13}$$

where,

B = diameter of pile (ft)

γ'_i = effective soil unit weight in *ith* interval (pcf)

z_i = depth to midpoint of *ith* interval (ft)

β_i = load transfer factor in the *ith* interval

Δz_i = *ith* increment of pile length (ft)

The value of β may be determined using the following:

$$\beta = 1.5 - 0.135\sqrt{z_i},\ 1.2 > \beta i > 0.25 \tag{2.14}$$

The limiting value of unit skin friction in cohesionless soils is 4 ksf. According to this method, the top 5 feet and the bottom one diameter of pile are noncontributing to skin friction.

Table 2.4 Recommended values of q_T (after Resse and O'Neill, 1988)

N-value (uncorrected)	Value of q_T (ksf)
0 to 75	1.20 N
Above 75	90

q_T = unit end bearing

2.4.1.6 Recommended Factor of Safety of Different Methods

Recommended factor of safety on the methods based on the Standard Penetration Test (Canadian Foundation Engineering Manual, 1985) and on Theory of Plasticity (Canadian Foundation Engineering Manual, 1985) are 4.0 and 3.0, respectively. American Association of State Highway and Transportation Officials, AASHTO (1992) also recommends different values of factor of safety depending on ultimate axial load capacity based on specified construction control. The values of factor of safety recommended by AASHTO (1992) are shown in Table 2.5. Considering the values of factor of safety recommended by Canadian Foundation Engineering Manual (1985) and AASHTO (1992), a minimum factor of safety of 3.0 should be used to estimate the allowable axial load capacity of bored pile.

In order to ensure quality control, static axial load test on service pile should be carried out with a minimum test load of 1.5 times the allowable axial load capacity of a single bored pile.

Table 2.5 Recommended factor of safety on ultimate axial load capacity based on specified construction control (after AASHTO, 1992)

	Increasing Construction Control				
Subsurface Exploration	X[(1)]	X	X	X	X
Static Calculation	X	X	X	X	X
Dynamic Formula	X				
Wave Equation		X	X	X	X
Dynamic Measurement and Analysis			X		X
Static Load Test				X	X
Factor of Safety	3.50	2.75	2.25	2.00[(2)]	1.90

(1) X = Construction Control Specified on Contract Plans

(2) For any combination of construction control that includes an approved static load test, a factor of safety of 2.0 may be used.

2.5 Axial Pile Capacity in Tension

The simplest method of restraining piles against uplift is to employ a pile shaft that is sufficiently long to take the whole of the uplift in skin friction. However, where there is rock beneath a shallow soil overburden it may not be possible to drive the piles deeply enough to mobilize the required skin friction resistance. In such cases the shaft resistance must be augmented by adding dead weight to the pile to overcome the uplift load, or by anchoring the pile to the rock adding dead weight to counteract uplift loading is not usually feasible or economical. The piles may be required to carry alternating uplift and compressive loading, in which case the added dead load weight would result in a large increase in the compressive loading.

2.5.1 The Uplift Resistance of Friction Piles

The resistance of straight-straight piles in skin friction to uplift loads is calculated in exactly the same way as the skin friction on compression piles. The details about the uplift capacity determination have been described in Tomlinson (1977). Some published test results show that the uplift resistance in skin friction on a pile shaft is less than its resistance in compression, possibly up to 50% less in a granular soil. In the short term, the uplift resistance of a bored pile in clay is likely to be equal to its skin-frictional resistance in compression. However, Radhakrishna and Adams (1973) noted a 50% reduction in the uplift resistance of cylindrical augured footings and a 30 to 50% reduction in belled footings in clay when sustained loads were carried over a period of 3 to 4 months. It was considered that the reduction in uplift was due to a loss of suction beneath the pile base and the dissipation of negative pore pressures set up at the initial loading stage. These authors pointed out that such reduction are unlikely for piles with depth/width ratio are greater than 5.

A safety factor of 3 on the ultimate resistance as calculated for compression piles is usually adequate. An upward movement of only 0.5 to 1.0% of the pile width is required to mobilize the peak resistance. Because this movement is very small the tapered pile is unlikely to show any significant difference between the skin-frictional resistance in compression and tension, although it is clear that the latter falls off very quickly as the tapered pile is lifted out of the ground.

Where vertical piles are arranged in closely-spaced groups the uplift resistance of the complete group may not be equal to the sum of the resistance's of the individual piles. This is because, at ultimate load conditions, the block of soil enclosed by the pile group is lifted. The manner in which the load is transferred from the pile to the soil is complex and depends on the elasticity of the pile, the layering of the soil, and the disturbance to the ground caused by installing the piles. A spread of load of 1 in 4 from the pile to the soil provides a simplified and conservative estimate of the volume of a cohesionless ($c = 0$) or partly cohesive (c-ϕ) soil available to be lifted by the pile group, as shown in Figure 2.4.

For simplicity in calculation, the weight of the pile embedded in the ground is assumed to be equal to that of the volume of soil it displaces. If the weight of the block of soil is calculated by using a diagram of the type shown in Figure 2.4, then the safety factor against uplift can be taken as unity, since skin friction around the periphery of the group is ignored in the calculation. The submerged weight of the soil should be taken below ground water level.

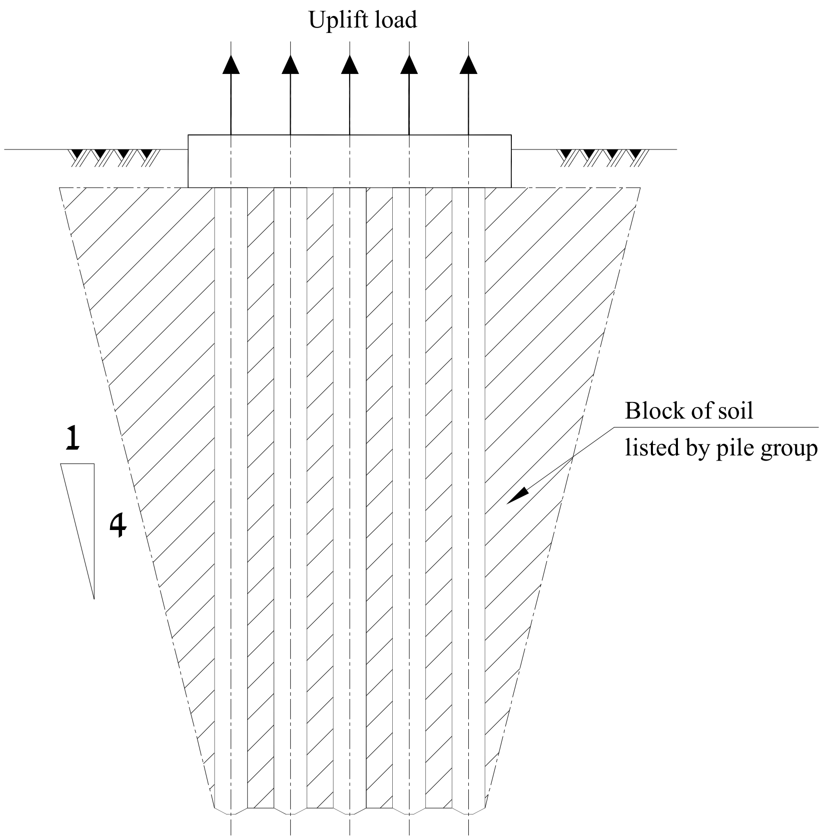


Figure 2.4: Uplift of group of closely-spaced piles in cohesionless soils

In the case of cohesive ($\phi = 0$) soils the uplift resistance of the block of soil enclosed by the pile group in Figure 2.5 is given by the equation:

$$Q_u = (2LH + 2BH)\bar{C}_u + W \tag{2.15}$$

where, Q_u is the total uplift resistance of the pile group, L and B, are the overall length and width of the group, respectively. H is the depth of the block of soil below pile cap level,

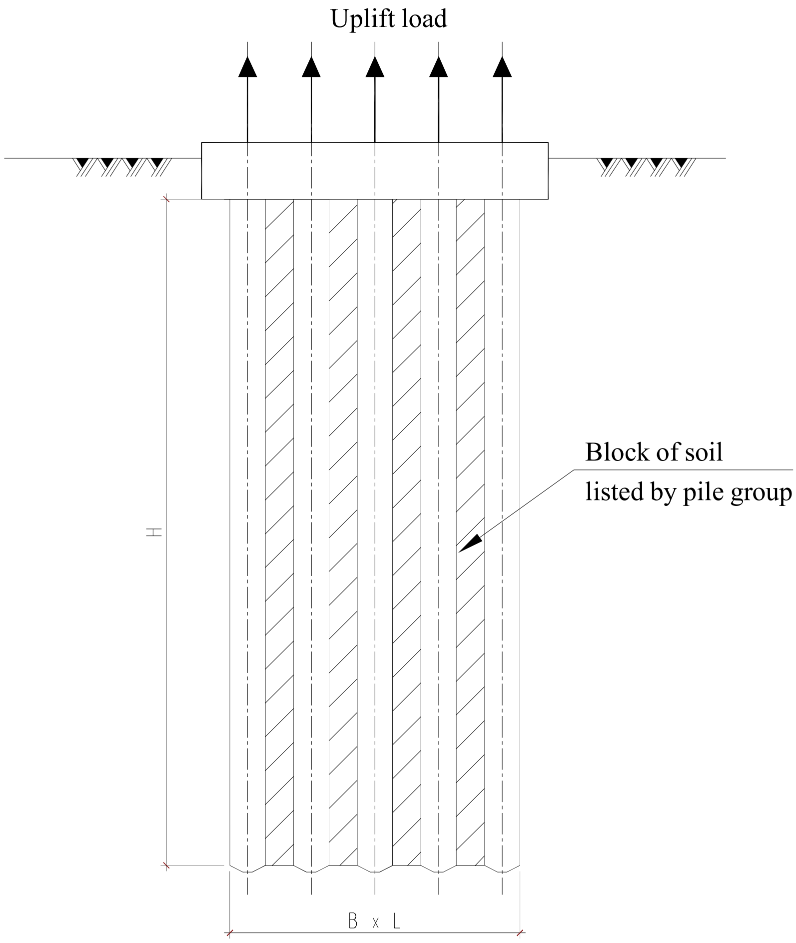


Figure 2.5: Uplift of group of piles in cohesive soils

$\bar{c}_u$ is the value of average undisturbed un-drained cohesion of the soil around the sides of the group and W is the combined weight of the block of soil enclosed by pile group plus the weight of the piles and pile cap.

A safety factor of 2 should be used with Equation 2.15 to allow for the possible weakening of the soil around the pile group caused by the method of installation. For long-term sustained loading a safety factor of 2.5 to 3.0 would be appropriate.

If either of the above two methods is used to calculate the combined uplift resistance of a pile group, the allowable resistance must not be greater than that provided by the sum of the skin-frictional resistance of the individual piles in the group divided by the appropriate safety factor.

2.5.2 Piles with Base Enlargement

Tension piles for towers can be constructed with enlargement of the base in which case strength of a part of the soil above the base of the pile can also be made to resist the uplift forces as in the case of under reamed piles. Drilled in rock anchors are also commonly used to resist tension forces and details of their design are explained in Tomlinson (1977). When bored piles are constructed in clay soils, base enlargements can be formed to anchor the piles against uplift. The enlargements are made by the belling tools. Enlargements cannot be formed in cohesionless soils unless the borehole is drilled with the support of bentonite slurry. The size and stability of an enlargement made in this way is problematical and it cannot be inspected to check for size. Full-scale loading tests are essential to prove the reliability of the bentonite method for any particular site. Reliably predictions cannot be made of the size angle shape of base enlargements formed by hammering out a bulb concrete at the bottom of a driven-and cast-in-situ pile. End enlargements formed on precast concrete or steel piles, although providing a substantial increase in compressive resistance when driven to a dense or hard stratum, do not offer much uplift resistance since a gap of loosened soil is formed around the shaft as the pile is driven down. Figure 2.6 shows the uplift of single pile with base enlargement in cohesive soil ($\phi = 0$).

Meyerhof and Adams (1968) investigated the uplift resistance of a circular plane embedded in a partly cohesive (c-ϕ) soil and established the formula:

$$Q_u = \pi cBH + s \times \frac{1}{2}\pi \times B(2D - H)\,HK_u \tan\phi + W \qquad (2.16)$$

where,

Q_u = the total uplift resistance of the plate,

B = the diameter of the plate,

H = the height of the block of soil lifted by the pile,

c = the cohesive strength of the soil, s is a shape factor,

D = the depth of the plate, K_u is a constant,

ϕ = the angle of shearing resistance of the soil, and

W = the weight of the soil resisting uplift by the plate.

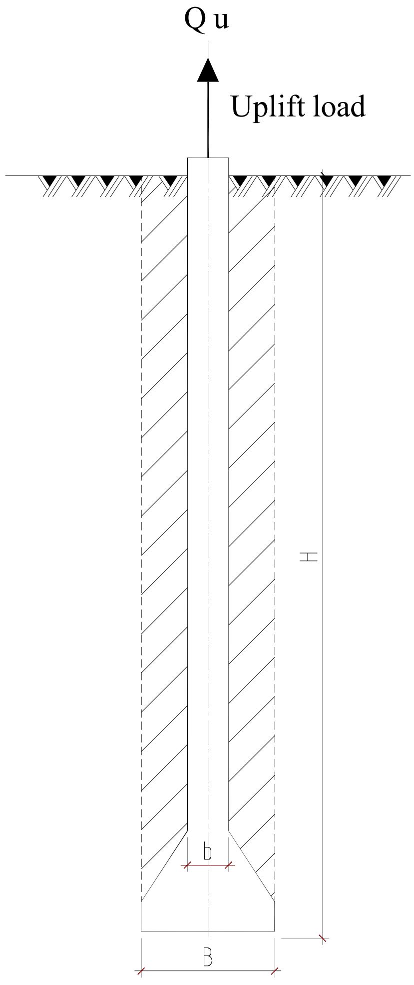


Figure 2.6: Uplift of single pile with base enlargement in cohesive soil (ϕ = 0).

Finally, it can be said that the uplift resistance of straight sided friction piles are calculated in the same way as explained for compression piles when the L/d ratio is greater than 5 (bearing resistance of the pile is absent when the pile is in pure tension). In the case of shorter lengths (when L/d < 5) there is a likelihood of reduction of the frictional resistance. A factor of safety of 3 is recommended for tensile strength. It should also be noted that generally the movement necessary to mobilize the skin friction and hence tension in pile is small.

Past experience shows that in sands the resistance in tension is only 2/3 that of skin friction value in compression. In clays they develop more or less the same skin friction as in compression. As a rule we may assume that the ultimate tension capacity of a friction pile in two-third (2/3) its ultimate skin friction capacity in compression. The tension capacity of piles in different soil and length versus diameter ratio are presented in Table 2.6.

Table 2.6 Tension capacity of pile in different soil condition

Case	L/d>5	L/d<5	Soil Type
1	Tension capacity = 2/3 of skin friction value of Compression	------	Sand
2	Tension capacity = skin friction value of Compression	------	Clay
3	Tension capacity = 2/3 of skin friction value of Compression	------	For any soil

2.6 Piles Subjected to Lateral Loads

Vertical piles resist lateral loads or moments by deflecting until the necessary reaction in the surrounding soil is mobilized. The behavior of the foundation under such loading conditions depends essentially on the strength and stiffness of the pile and the strength of the soil.

The lateral load capacity of vertical piles may be limited in three different ways:

(i) The capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;

(ii) The bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles;

(iii) The deflections of the pile heads may be too large to be compatible with the superstructure.

All the three modes of failure must be considered in design.

The methods presently available for design of pile foundations subjected to horizontal loads must be regarded as highly empirical. The input soil data are associated with a high degree of uncertainty. Therefore, these methods must be used with great caution and with due consideration of their limitations.

2.6.1 Deflections and Moments in a Pile

In most cases other than short rigid piles, the maximum horizontal load that may be safely applied to a vertical pile is limited, not only by the capacity of the surrounding soil, but by the magnitude of the deflection of the pile and of the resulting bending moments in the pile.

The analysis of the behavior of horizontally loaded piles is based on the concept of elastic reaction. In this concept it is assumed that the soil around a pile can be simulated by a series of horizontal springs, each spring representing the behavior of a layer of soil of unit height. When the pile is forced against the soil under the action of horizontal loads, the soil deforms and generates an elastic reaction assumed to be identical to the force that would be generated by the simulating spring subjected to the same deformation. With the further assumption that the soil is homogeneous, i.e., all springs are identical, the soil's behavior can be determined if the equivalent spring constant is known. This spring constant is called the coefficient of subgrade reaction, k_s (dimension: force/volume).

2.6.2 Coefficient of Subgrade Reaction

Though simple in its definition, the coefficient of subgrade reaction has proved to be a very difficult parameter to evaluate. This is because it cannot be measured in laboratory tests, but must be back calculated from full-scale field tests.

Investigations have shown it to be variable not only with the soil type and mechanical properties, but also with stress level and the geometry of the pile.

In the absence of better information, the coefficient of horizontal subgrade reaction may be estimated by the following,

In cohesionless soil (Terzaghi, 1955)

$$k_s = n_h \ (z/d) \qquad (2.17)$$

where,

k_s = coefficient of horizontal subgrade reaction (force per unit volume)

z = depth

d = pile diameter

n_h = coefficient related to soil

Terzaghi's recommended values of n_h are given in Table 2.7.

Reese et al (1974) proposed criteria for cohesionless soils for analyzing the behaviour of piles under static and cyclic loading. His recommended values of n_h are given in Table 2.8.

It may be noted here that the n_h values as proposed by the investigators are not constants. It varies with the soil properties, (density for granular material and shear strength for the clay soils), the width or diameter of pile, the flexural stiffness of the pile material and the deflection of the pile. It is a very complex phenomenon, which depends on various factors.

Table 2.7 Values of n_h for cohesionless soils (after Terzaghi, 1955)

Soil Compactness Condition	n_h in MN/m^3	
	Above groundwater	Below groundwater
Loose	2.2	1.3
Compact	6.6	4.4
Dense	18.0	11.0

Table 2.8 Values of n_h for cohesionless soils (after Reese, 1974)

Soil Compactness Condition	n_h in MN/m^3	
	Above groundwater	Below groundwater
Loose	7	6
Compact	25	15
Dense	60	30

2.7 Past Researches

Many researches have been conducted to study the pile behavior in case of compression load, but only few researches have been conducted to study the piles against uplift load application. Studies related to pile capacity in tension are discussed briefly below.

Several researches are conducted in many places of the world. Some of them are summarized below:

Ismael (2001) conducted field testing program on bored piles and pile groups in medium dense, weakly cemented sands in South Surra and Kuwait. The program included single piles in tension and compression and two pile groups, each consisting of five piles, in compression. He observed that the soil deposit at the site consists of medium dense, weakly cemented sands with strength parameters c′ and ϕ' is 20 kPa and 35º, respectively, and a unit weight of; 18 kN/m^3. The single piles in compression resisted 70% of the applied load at failure in side friction and 30% in base resistance. The axial load distribution along the piles in compression was nearly linear. This indicates uniform side friction along the pile shafts. He observed that friction in compression and tension was very similar. For the two pile groups, each consisting of five piles installed at a pile spacing of two and three pile diameters, the group efficiency was 1.22 and 1.93, respectively. This is attributed to the increased side friction along the pile shafts of the groups.

Krabbenhoft et. al. (2006) conducted field testing program on bored piles. The lengths of the bored piles varied from 2 m to 6 m and all were of a diameter of 140 mm. The piles were tested to failure in tension and the load-displacement relations were recorded. The investigation has shown pronounced differences between the load bearing capacities in sand obtained by different design methods. But they seem to be no difference in side friction for piles in tension and compression.

Shooshpasha et al. (2009) in the present study, different theoretical and empirical methods were used to evaluate shaft friction capacity of piles. These methods show differences in tension capacity value of piles buried in sand. Field measurement results provided in full scale are in the range of results obtained from some methods. Differences observed in results of these methods are made because of different parameters influential in shaft tension capacity in sandy soils and lack of enough suitable information from tests conducted in full scale especially on open-ended pipe pile and each method considered a few parameters to estimate shaft friction capacity. However, assessment on pile tests show that two methods of ICP-05 and UWA-05 (Lehanc et al. 2005) gives the shaft friction capacity closed to field measurements in full scale.

Few researches have been conducted in Bangladesh. These are described below.

Khan (1997) studied the performance of axially loaded small size prestressed concrete piles and the possibility to use it as substitute of timber piles especially for light structure. He uses piles with size of 175mm × 175mm in cross section and 5.0m to 7.5m in length. His objectives from study were:

a) To investigate the methods of construction and installation of small size prestressed piles.
b) To drive and carry out pile load test at four different locations.
c) To compare the capacity of piles from load test with the estimated value from different methods.

The outcomes of his studies are:

a) The ultimate capacity of pile determined from load test using the criteria of Terzaghi's method (1942), IS: 2911-1979, BS: 8004-1986 and Davisson's method (1973) are almost equal.

b) The measured ultimate capacity of piles driven through Dhaka clay is in close agreement with the predicted value using λ method [Vijayvergiya and Focht (1972)]. The predicated ultimate capacity of pile using α method [Tomlison (1974)] and α_2 method [Peck (1971)] is slightly smaller than the tested value.

Khan (2002) study the ultimate capacity of piles in Bangladesh. Objectives of his study were:

a) To analyses the pile theoretically with the help of sub-soil investigation report.
b) To predict the ultimate load carrying capacity by studying result of field load test.
a) To correlate the pile load capacity obtained from theoretical analysis with that of the result from load test.
b) To draw a conclusion reading the theoretical pile capacity in context with Bangladesh soil.

The outcomes of his studies are:

a) For piles in Dhaka City about 85% to 90% of pile capacity is contributed by the sandy layer.
b) For Khulna soil the organic layer dose not contribute any value to the pile capacity. Most part of the capacity can be obtained from the underlying sandy layer.
c) Two correlations are proposed to obtain ultimate pile capacity from static analysis for Bangladesh.

For precast piles (in ton): $Q_{ultimate} = 2.1331 + 0.8315\ Q_{static}$ ($r= 0.96$, $\sigma= 17.54$)

For in-situ piles (in ton): $Q_{ultimate} = 34.43 + 0.7582\ Q_{static}$ ($r= 0.86$, $\sigma= 28.40$)

where:

r = correlation factor

σ = standard deviation

These relations should be used with some caution until more data prove their validity.

Yasin et. al. (2009) presented a case study in Khulna, Bangladesh where soft soil exists to depths greater than about 40 m. Based on static pile load test data on cast-in-situ bored RC piles in soft clay, in which it is attempted to correlate ultimate pile load capacity from static load tests with estimated capacity using SPT-N value. Also a comparison has been made between the load capacities of cast-in-situ bored pile. And the study result showed that ultimate compressive load capacity of cast-in-situ and driven RC piles fully embedded in soft soil can be reliably estimated using a set of empirical correlations between soil parameters and SPT-N value. Also, allowable load capacity determined following BNBC (1993), IS and double tangent method are compared. It is found that allowable compressive load capacity by these three different methods is nearly same.

Ahmed et al. (2012) evaluate the dynamic pile head stiffness using a soil-structure interaction analysis tool based on Thin Layered Element Method (TLEM). They evaluated the frequency dependent pile head stiffness by TLEM software for two selected sites of Dhaka city, namely; Mirpur Defense Officers' Housing Scheme (Mirpur DOHS) and Uttara site. They used RC pile dimensions ranging from 457 to 610 mm diameter with lengths varying from 9 to 25 m for the analysis. They developed dynamic stiffness curves for selected sites provide general approximation of pile head stiffness up to predominant frequency of 10Hz. They observed that stiffness decreases with the increase of frequency. The pile head stiffness becomes almost independent of pile diameter at a certain range of frequency (23 to 28 Hz) for different lengths. Larger diameter piles exhibit greater damping in comparison to piles of smaller diameters. They also present the effect of soil layer homogeneity that influences the soil-pile-soil interaction effect. Dense sand layer of Uttara site has shown almost four times higher stiffness than that of Mirpur DOHS site. More details are available in Ahmed (2010).

From these, it is clear that although several researches are conducted about the capacity of pile in compression none of this study locally conducted is related to the determination of pile capacity in tension. It is necessary to conduct research to determine the uplift capacity of pile.

2.8 Summary

The main objective of this chapter is to review the types of piles, selection criteria of piles, available literature related to axial capacity of piles and the past researches related to this study. These can be summarized in the following:

a) Piles are generally divided into large displacement piles, small displacement piles, replacement piles and composite piles. Again due to materials and construction practice piles are divided in to steel piles, cast-in-place pipe piles, timber piles, concrete piles, precast piles, pre-stressed piles and cast-in-place-piles etc.

b) The selection of the appropriate type of pile is another important issue. The selection of pile(s) generally depends on the location and type of structure, the ground conditions and durability of pile.

c) The quality of a pile depends significantly on the installation or construction technique, on equipment, and on workmanship. Such parameters cannot always be quantified nor taken into account in normal design procedures. Only a few projects, however, are large enough to warrant full-scale testing during design phase, and, in most cases, tests (proof-tests) are performed only during or even after construction of the foundation. Therefore design methods have been developed to estimate the axial load capacity of piles based on soil parameters and construction procedure.

d) A number of methods are available to estimate the ultimate axial load capacity of bored piles in compression. These methods are based on N-values obtained from Standard Penetration Test (SPT) and on angle of internal friction of sand. Ultimate axial load capacity of a single bored pile can be determined using the methods like method based on the Standard Penetration Test, method based on Theory of Plasticity, Tomlinson method, method Proposed by Reese et. al. and method Recommended by American Association of State and Transportation Officials etc.

e) The uplift resistance of straight sided friction piles are calculated in the same way as explained for compression piles when the L/d ratio is greater than 5. In the case of shorter lengths (when L/d < 5) there is a likelihood of reduction of the frictional resistance. Past experience shows that in sands the resistance in tension is only 2/3 that of skin friction value in compression. In clays they develop more or less the same skin friction as in compression. As a rule we may assume that the ultimate tension capacity of a friction pile in two-third (2/3) its ultimate skin friction capacity in compression. A factor of safety of 3 is recommended for tensile strength.

f) Many researches have been conducted to study the pile behavior in case of compression load, but only few researches have been conducted to study the piles capacity against uplift load application. It is necessary to conduct research to determine the uplift capacity of pile.

Chapter Three

EXPERIMENTAL PROGRAM AND METHODOLOGY

3.1 General

The objective of this chapter is to describe the experimental program that was carried in this research. The selected sites for the research are described. All the field and laboratory test procedures are discussed. Methodologies of the load tests (both uplift and compression) are also presented. Methods for determining the ultimate and allowable capacity of piles from load tests are also described. Methods for estimating pile capacities from sub-soil characteristics are presented. Finally, a test plan is provided.

3.2 Selected Areas for the Research

Sites are selected at different areas of Bangladesh. The locations of the sites are presented on Bangladesh map in Figure 3.1. The descriptions of the sites are presented in Table 3.1.

Load test results of thirteen sites are presented in this study. In these thirteen sites, 19 piles are tested under uplift load. Among these 19 piles, 2 piles are tested in this study, 7 pile load test results are collected from BRTC, BUET. Other pile load test results are collected from Power Grid Company of Bangladesh and ICON Engineering Services, Dhaka. Site S-01 is located at Baridhara in Dhaka city. Site S-02, S-04, S-09 and S-13 are located in eastern part of Dhaka. Site S-03, S-05 and S-11 are located in western part of Dhaka. Site S-06 and S-07 are located in western part of Bangladesh. Site S-08 is located in Khulna. Site S-10 and S-12 are located in Rajshahi, the west part of Bangladesh. Besides, these two plies are tested at the site S-01 under axial compression.

Sub-soil investigations at the vicinity of tested piles were also conducted in order to obtain the property of the soil very close to tested piles location. Samples were also collected during field investigation and those samples were tested in the Geotechnical Engineering Laboratory of BUET.

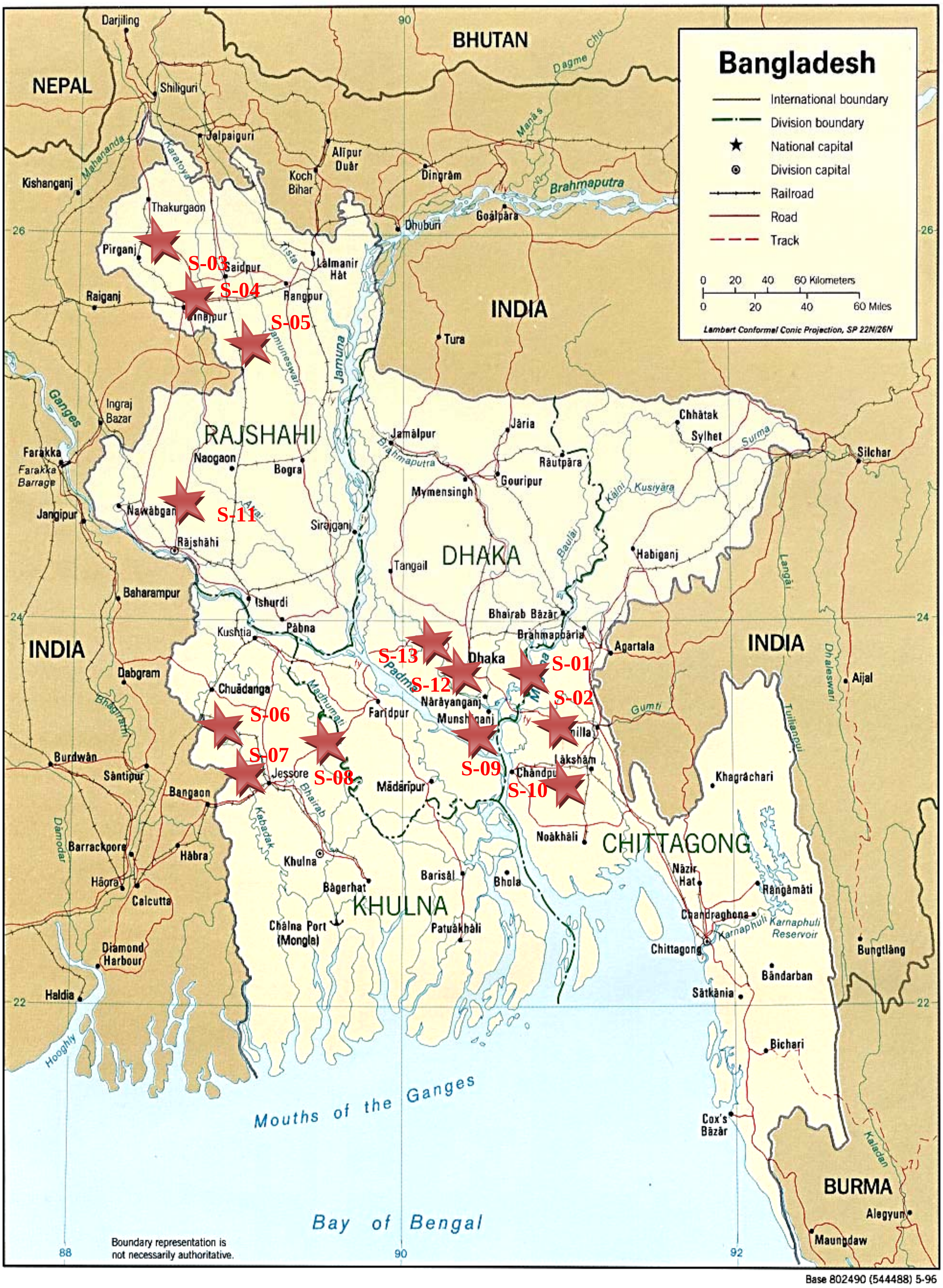


Figure 3.1: Map showing the selected areas for the research

Table 3.1: Selected areas for the research

Site No	Site Code	Site Location	Type and Number of Test		Source of Data
			Tension	Compression	
1	S-01	Baridhara, Dhaka	2	2	Conducted in this study
2	S-02	Manikganj Grid Sub-Station	1	-	BRTC, BUET*
3	S-03	Thakurgaon-Panchagar T. L. Tower No 56	1	-	PGCB**
4	S-04	Thakurgaon-Panchagar T. L. Tower No 77	1	-	
5	S-05	Thakurgaon-Panchagar T. L. Tower No 147	1	-	
6	S-06	Jhinaidah-Chuadanga T. L. Tower No 25	1	-	
7	S-07	Jhinaidah-Chuadanga T. L. Tower No 28	1	-	
8	S-08	Jhinaidah-Chudanga T.L. Tower No 78	1	-	
9	S-09	Jhinaidah-Magura T. L. Tower No 17	1	-	
10	S-10	Jhinaidah-Magura T. L. Tower No 53	1	-	
11	S-11	Naogaon-Niamatpur T. L. Tower No 112	1	-	
12	S-12	Amin Bazar, Savar T. L. Tower No 51	1	-	ICON E.S***
13	S-13	Modanpur, Narayanganj	6	-	BRTC, BUET
Total No. of Tests			19	2	

*BUREAU OF RESEARCH, TESTING & CONSULTATION, BANGLADESH UNIVERSITY OF ENGINEERING & TECHNOLOGY, DHAKA
** POWER GRID COMPANY, BANLADESH
***ICON ENGINEERING SERVICES, DHAKA, BANGLADESH

3.3 Sub-soil Investigation

Sub-soil investigation is conducted at the selected sites. Borings were conducted near the pile locations. Figure 3.2 shows the location of boreholes on the pile layout plan of Baridhara site. Figure 3.3 shows the photograph of sub-soil investigation at Baridhara site (i.e., site S-01).

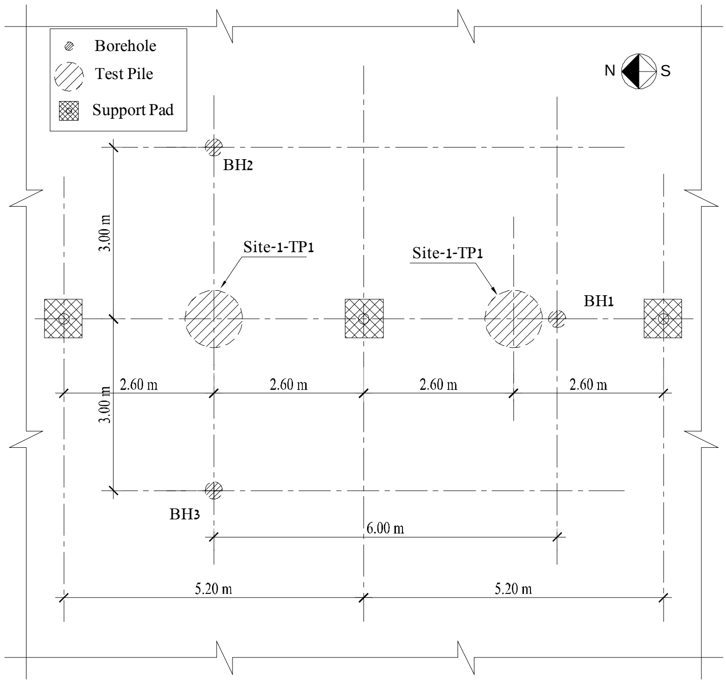


Figure 3.2: Layout of boreholes and test piles at Baridhara site (S-01)

Figure 3.3: Photograph showing sub-soil investigation at Baridhara site (S-01)

3.4 Test Procedure

In each location, Standard Penetration Tests (SPT) was conducted. SPT-N values were recorded during the test. Disturbed and un-disturbed soil samples were collected during SPT. The collected soil samples were tested in the Geotechnical Engineering Laboratory of BUET. Mainly Grain size distribution, Atterberg limit and un-confined compression tests were conducted. Besides direct shear tests were also conducted. All the tests were conducted according to ASTM standards (ASTM, 1989). Procedures followed during the field tests are described below in brief.

3.4.1 Standard Penetration Test

Standard Penetration Test is widely used for the estimation of pile capacity. To utilize SPT results in estimating pile capacity, SPT was conducted according to ASTM D 1586 (ASTM, 1989) in all areas as described in Section 3.2. The main objectives were as below:

(a) Boring and recording of soil stratification
(b) Sampling (both disturbed and undisturbed)
(c) Recording of SPT-N value
(d) Recording of ground water table

Table 3.2: Recommended SPT procedure (ASTM D1586)

Equipment's	Short procedure
Borehole size	65 mm < Diameter < 1 15 mm
Borehole support	Casing for 3m length and drilling mud
Drilling	• Wash boring • Side discharge bit rotary boring • Side or upward discharge bit clean bottom of bore hole
Drill rods	A or AW for depths of less than 15m N or NW for greater depths
Sampler	Standard O.D. 51mm +/- 1mm, I.D. 35mm +/- 1mm and length > 457mm
Penetration resistance	Record number of blows for each 150mm; N = number of blows from 150 to 450mm penetration
Blow count rate	30 to 40 blows per minute

Procedure of SPT Test

The Standard Penetration Test uses a 50 millimeter diameter pipe (split spoon) driven with a 63.5 kilogram hammer at a drop of 750 millimeters. The test is described in ASTM D1586 (ASTM, 1989). A short procedure of SPT N-value test is described in Table 3.2.

Drilling Method

The borehole should be made by mud rotary techniques using a side or upward discharge bit. Hollow-stem-auger techniques generally are not recommended, because unless extreme care is taken, disturbance and heave in the hole is common. However, if a plug is used during drilling to keep the soils from heaving into the augers and drilling fluid is kept in the hole when below the water table (particularly when extracting the sampler and rods), hollow-stern techniques may be used. If water is used as the fluid in a hollow-stem hole, and it becomes difficult to keep the fluid in the hole or to keep the hole stable, it may be necessary to use a drilling fluid (consisting of mud or polymers). With either technique, there is a need for care when cleaning out the bottom of the borehole to avoid disturbance. Prior to extracting the drill string or auger plug for each SPT test, the driller should note the depth of the drill hole and upon lowering of the sampler to the bottom of the hole, the depth should be carefully checked to confirm that no caving of the walls or heaving of the bottom of the hole has occurred.

Hole Diameter

Preferably, the borehole should not exceed 115 mm in diameter, because the associated stress relief can reduce the measured N-value in some sands. However, if larger diameter holes are used, the factors listed in Table 3.2 can be used to adjust the N-values for them. When drilling with hollow-stem augers, the inside diameter of the augers is used for the borehole diameter in order to determine the correction factors provided in Table 3.2.

Drive-rod Length

The energy delivered to the SPT can be very low for an SPT performed above a depth of about 10 m due to rapid reflection of the compression wave in the rod. The energy reaching the sampler can also become reduced for an SPT below a depth of about 30 m due to energy losses and the large mass of the drill rods. Correction factors for those conditions are listed in Table 3.3.

Sampler Type

If the SPT sampler has been designed to hold a liner, it is important to ensure that a liner is installed, because a correction of up to about 20% may apply if a liner is not used. In some cases, it may be necessary to alternate samplers in a boring between the SPT sampler and a larger-diameter ring/liner sampler. The ring/liner samples are normally obtained to provide materials for normal geotechnical testing. Although the use of a plastic sample catcher may have a slight influence on the SPT N-values that influence is thought to be insignificant and is commonly neglected.

Energy Delivery

One of the single most important factors affecting SPT results is the energy delivered to the SPT sampler (Table 3.4). This is normally expressed in terms of the rod energy ratio (ER). An energy ratio of 60% has generally been accepted as the reference value. The value of ER (%) delivered by a particular SPT setup depends primarily on the type of hammer/anvil system and, the method of hammer release. Values of the correction factor used to modify the SPT results to 60% energy (ER/60) can vary from 0.3 to 1.6, corresponding to field values of ER of 20% to 100%. Table 3.4 provides guidance for summary of rod energy ratios provide specific recommendations for energy correction factors.

Standard Rod energy ratio = 60%

$$ER_n = n_d ER_v \tag{3.1}$$

where,

n_d is dynamic efficiency depends on anvil weight (0.87-0.6); ER_v is Velocity energy ratio

$$N_{60} = (N.\ ER_{v)}/\ 60 \tag{3.2}$$

Down-hole hammers, raised and lowered using a cable wire-line, should not be used unless adequately designed and documented correlation studies have-been performed with the specific equipment being used. Even then, the use of such equipment typically results in highly variable results, thereby making their results questionable.

Table 3.3: Borehole, sampler and correction factors (Skempton, 1986)

Factor	Equipment Variable	Symbol	Correction value
Rod length	3 m to 4 m	C_R	0.75
	4 m to 6m		0.85
	6 m to 10 m		0.95
	>10 m		1.00
Sampling method	Standard Sampler	C_S	1.00
	U.S. Sampler without liners		1.20
Borehole Diameter	65 mm to 115mm	C_B	1.00
	150 mm		1.05
	200 mm		1.15

Table 3.4: SPT hammer efficiencies (Clayton, 1990)

Country	Hammer	Release	ER_v(%)	ER_v/60
Japan	Donut	Tombi	78	1.30
	Donut	2 turns of rope	65	1.10
China	Pilcon type	Trip	60	1.00
	Donut	Manual	55	0.90
USA	Safety	2 turns of rope	55	0.90
	Donut	2 turns of rope	45	0.75
UK	Pilcon, Dando	Trip	60	1.00
	Old standard	2 turns of rope	50	0.80

3.4.2 Laboratory Tests

Disturbed and undisturbed samples were collected during SPT tests. Collected samples were tested at the Geotechnical Engineering Laboratory of BUET. List of tests conducted are presented in Table 3.5. These tests were performed according to the procedure specified by American Society for Testing Materials (ASTM) standard. Laboratory tests that were conducted are grain size analysis, Atterberg limit test, unconfined compression test and direct shear test. The details of the test procedures are available at the ASTM manuals (ASTM, 1989).

Table 3.5: List of tests conducted as ASTM designation

Name of test	Procedures
Grain size analysis	ASTM D 422
Atterberg limit test	ASTM D 4318
Unconfined compression test	ASTM D 2166
Direct Shear Test	ASTM D 3080

3.5 Procedure of Load Tests

The actual load capacity of a pile-soil system can best be determined by testing. Two types of tests are common to determine pile capacity. Depending on the type of load applied to the pile, these are called compression test and tension or uplift test. The brief procedures of the tests are described below.

3.5.1 Uplift Test

This test method covers procedures for testing vertical or batter piles, individually or in groups, to determine response of the pile or pile group to a static tensile toad applied axially to the pile or pile group. This test method is applicable to all deep foundation units that function in a manner similar to piles regardless of their method of installation.

Testing measures the response of a pile-soil system to loads and may provide data for research and development, engineering design, quality assurance' or acceptance or rejection in accordance with the specifications and contract documents. Testing as covered herein, when combined with and acceptance criterion, is suitable for assurance of pile foundation design and installation under building codes, standards and other regulatory statutes.

Apparatus for Applying Loads

Where feasible, the immediate area of the test pile or pile group shall be excavated to the proposed pile cut-off elevation. The test pile(s) shall be cut off or built up to the proper grade as necessary to permit construction of the load-application apparatus, placement of the necessary testing and instrumentation equipment and observations of the instrumentation.

The clear distance between the test pile and the reaction pile(s) or cribbing shall be at least five times the butt diameter or diagonal dimension of the test pile, but not less than 2.5 m.

Bearing plates, hydraulic jack ram and load cell shall be centered on test beam, cap beam. Reaction member, reactions piles, or cribbing, bearing plates shall be set perpendicular to the longitudinal axis of the pile. Plates shall be set in high-strength, quick-setting grout for concrete reaction piles, or-welded to steel reaction piles. Or, in the case of timber reaction piles, set on the pile top which shall be sawed off on a plane perpendicular to the longitudinal axis of the pile. Bearing plates on cribbing shall be set in a horizontal plane.

Testing Equipment

Hydraulic jacks including their operation shall conform to ASTM D3689 recommendation, complete jacking system including the hydraulic jack's hydraulic pump and pressure gage shall be calibrated as a unit before each test or series of tests in a test program to an accuracy of not less than 5% of the applied load. The hydraulic jack shall be calibrated over its complete range of ram travel for increasing and decreasing applied loads.

Load-Applied to Pile by Hydraulic Jack

Center over the test pile a test beam of sufficient size and strength to avoid excessive deflection under load with adequate space between the bottom flange of the test beam (including any projecting parts of the connection system to the reaction frame) and the top of the test pile to provide for the total anticipated upward movement of the test pile under test. Support the ends of the test beam with reaction piles or cribbing. If two or more reaction piles are used at each end of the test beam, they shall be capped with a suitable steel beam set on the piles or on bearing plates.

Anchor the reaction frame to the test pile by means of straps or bars welded to the pile or by bars or cables embedded in the pile. Tension connections between test pile and reaction frame shall be constructed so as to prevent slippage, rupture, or excessive elongation of the connection under the maximum required test load.

Loading Procedures

Standard Loading Procedure: Unless failure occurs first, load the pile to 200% of the anticipated pile design load for tests on individual piles or to 150% of the group design load for tests on pile groups, applying the load in increments of 25% of the individual pile or group design load. Maintain each load increment until the rate of movement is not greater than 0.25 mm/h but not longer than 2 h. Here we apply the Cyclic Loading Method which is (optional) but it common in Bangladesh For the first application of test load increments, After the application of loads equal to 50, 100, and 150% of the pile design load for tests on individual piles, or 50 and 100% of the group design load for tests on pile groups, maintain the total load in each case for 1 h and remove the applied load decrements equal to the loading increments, allowing 20 min between decrements. After removing each total applied load, reapply the load to each preceding load level in increments equal to 50% of the design load, allowing 20 min between increments. After the total required test load has been applied, hold and remove the test load.

3.5.2 Compression Load Test

This test method covers procedures for testing vertical or batter piles individually or groups of vertical piles to determine response of the pile or pile group to a static compressive load applied axially to the pile or piles within the group. This test method is applicable to all deep foundation units that function in a manner similar to piles regardless of their method of installation.

Apparatus for Applying Loads

The apparatus for applying compressive loads to the test pile or pile group shall be as described in ASTM D3689 or as otherwise specified and shall be constructed so that the loads are applied to the central longitudinal axis of the pile or pile group to minimize eccentric loading.

For a test on an individual pile, a steel bearing plate of sufficient thickness to prevent it from bending under the loads involved (but not less than 50 mm) shall be centered on the pile or pile cap and set perpendicular to the longitudinal axis of the pile or piles within the group, except that for tests on pile groups involving the use of two or more separate loading points, a test plate shall be used at each loading point and such plates shall be arranged symmetrically about the centroid of the group. For tests on individual piles, the size of the test plate shall be not less than the size of the pile but not less than the area covered by the base of the hydraulic jack; for tests on pile groups, the size of the test plate shall be not less than twice the area covered by the base of the hydraulic jack.

The hydraulic jack shall be centered on the test plate with a steel bearing plate of adequate thickness between the top of the jack ram and the bottom of the test beam. If a load cell or equivalent device is to be used, it shall be centered on the bearing plate above the ram with another steel bearing plate of sufficient thickness between the load cell or equivalent device and the bottom of the test beam. Bearing plates shall be of sufficient size to accommodate the jack ram and the load cell or equivalent device and properly bear against the bottom of the test beam.

Load- Applied to Pile by Hydraulic Jack

Center over the test pile or pile group a test beam of sufficient size and strength to avoid excessive deflection under load allowing sufficient clearance between the top of the test pile or pile cap and the bottom of the beam after deflection under load to accommodate the necessary bearing plates, hydraulic jack (and load cell if used). Support the ends of the test beam on temporary cribbing or other devices.

Center a box or platform on the test beam with the edges of the box or platform parallel to the test beam supported by cribbing or piles placed as far from the test pile or pile group as practicable but in no case less than a clear distance of 1.5 m. If cribbing is used, the bearing area of the cribbing at ground surface shall be sufficient to prevent adverse settlement of the weighted box or platform.

Load the box or platform with any suitable material such as soil, rock, concrete, steel, or water-filled tanks with a total weight (including that of the test beam and the box or platform) at least 10% greater than the anticipated maximum test load.

Apply the test loads to the pile or pile group in accordance with the standard procedure in ASTM D3689 or as otherwise specified with the hydraulic jack reacting against the test beam.

Install a sufficient number of anchor piles or suitable anchoring device(s) so as to provide adequate reactive capacity and a clear distance from the test pile or pile group at least five times the maximum diameter of the largest anchor or test pile but not less than 2 m. When testing individual batter piles, the anchor piles shall be battered in the same direction and angle as the test pile.

Center over the test pile or pile group a test beam of sufficient size and strength to avoid excessive deflection under load with sufficient clearance between the bottom flange of the test beam and the top of the test pile or pile group to provide for the necessary bearing plates, hydraulic jack (and load cell if used). When applying axial loads to an individual batter pile, the test beam should be oriented perpendicular to the direction of batter. For test loads of high magnitude requiring several anchors, a steel framework may be required to transfer the applied loads from the test beam to the anchors.

Attach the test beam (or reaction framework if used) to the anchoring devices with connections designed to adequately transfer the applied loads to the anchors so as to prevent slippage, rupture or excessive elongation of the connections under maximum required test load.

Apply the test load in accordance with the standard loading procedure 5.1 or as otherwise specified to the test pile or pile group with the hydraulic jack reacting against the test beam.

Testing Equipment

Hydraulic jacks including their operation shall conform to ASTM D3689 recommendation unless a calibrated load cell is used, the complete jacking system including the hydraulic jack, hydraulic pump, and pressure gage shall be calibrated as a unit before each test or series of tests in a test program to an accuracy of not less than 5% of the applied load. The hydraulic jack shall be calibrated over its complete range of ram travel for increasing and decreasing applied loads. If two or more jacks are to be used to apply the test load, they shall be of the same ram diameter, connected to a common manifold and pressure gage, and operated by a single hydraulic pump.

When accuracy greater than that obtainable with the jacking system is required, a properly constructed load cell or equivalent device shall be used in series with the hydraulic jack. Load cell or equivalent device shall be calibrated prior to the test to an accuracy of not less than 2% of the applied load and shall be equipped with a spherical bearing.

If the hydraulic jack pump is to be left unattended at any time during the test, it shall be equipped with an automatic regulator to hold the load constant as pile settlement occurs.

Calibration reports shall be furnished for all testing equipment for which calibration is required, and shall show the temperature at which the calibration was done.

Loading Procedures

Standard Loading Procedure—Unless failure occurs first, load the pile to 200% of the anticipated pile design load for tests on individual piles or to 150% of the group design load for tests on pile groups, applying the load in increments of 25% of the individual pile or group design load. Maintain each load increment until the rate of settlement is not greater than 0.25 mm/h but not longer than 2 h. Provided that the test pile or pile group has not failed, remove the total test load any time after 12 h if the butt settlement over a one-hour period is not greater than 0.25 mm; otherwise allow the total load to remain on the pile or pile group for 24 h. After the required holding time, remove the test load in decrements of 25% of the total test load with 1 h between decrements. If pile failure occurs continue jacking the pile until the settlement equals 15% of the pile diameter or diagonal dimension.

Direct Loading Method—when using the loading method described in ASTM D3689, include in the first load increment the weight of the test beam and the platform adding or removing load increments, tighten the wedges along the platform edges to stabilize the platform. Place or remove load increments in a manner which avoids impact and maintains the load balanced at all times. After each load increment has been added, loosen (but do not remove) the wedges and keep them loose to permit the full load to act on the pile as settlement occurs.

3.6 Test Set-up for Uplift Test

Different techniques are used for applying load depending on the available equipments and the value of maximum applied load. Also it depends on the extent of technological progress and the importance of the project for which the piles are used. In this study, two methods were used for conducting the uplift load test. These methods are briefly described below.

3.6.1 Method 1: Support on Ground

Two uplift load tests are conducted at site S-01 using the procedure described in Figure 3.4 and Figure 3.5. Figure 3.4 and Figure 3.5 show the schematic diagram and photograph of actual test set-up for the uplift load test, respectively. In this case, two supports pads were used as shown in the figure. It is to be noted here that, the surrounding ground was strong enough to get the necessary reaction using supporting pads.

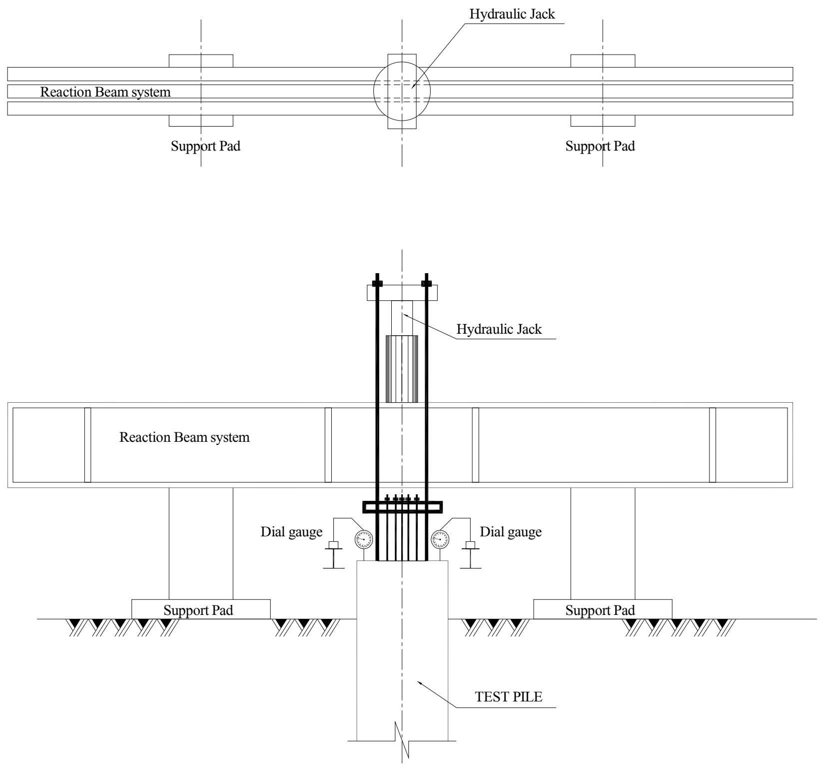


Figure 3.4: Schematic diagram of the test set-up and arrangement for site S-01

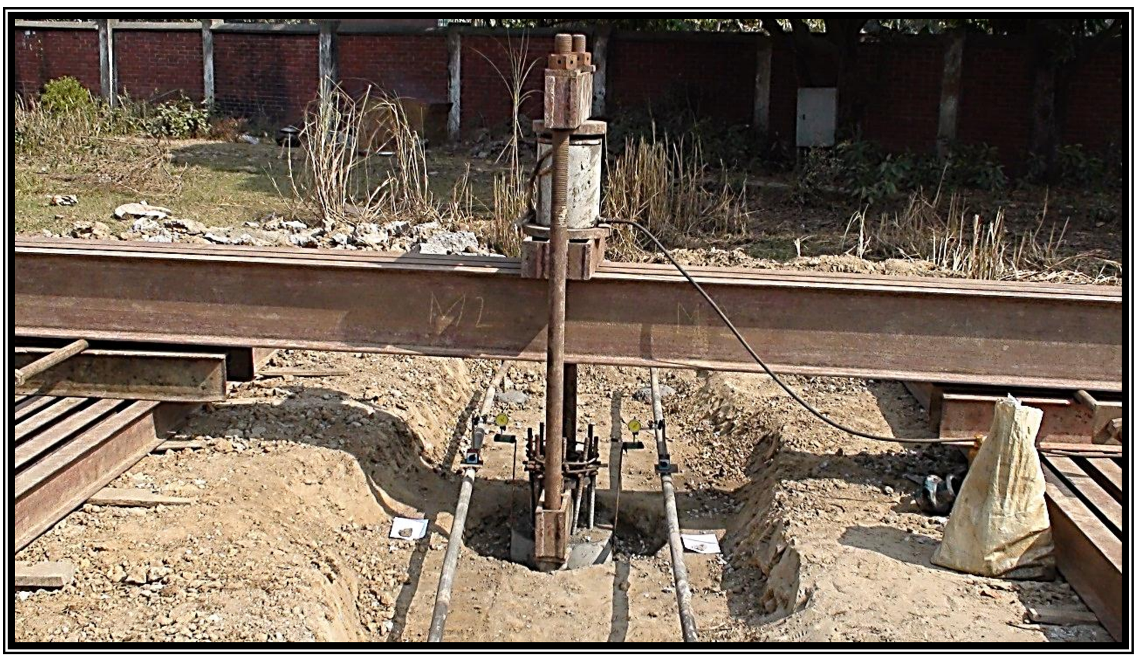


Figure 3.5: Photograph showing test set-up for uplift test at Baridhara site (S-01)

3.6.2 Method 2: Reaction Pile

Cast in-situ piles having 500mm diameter and 14.0m length were used for the foundation of 70m high Tower in Manikganj. Figure 3.6 shows the location of pile related to reaction piles. Figure 3.7 shows the schematic diagram of the test set-up. Two uplift load tests were conducted at site, S-02. It is seen from the figure that other piles were used as the reaction piles.

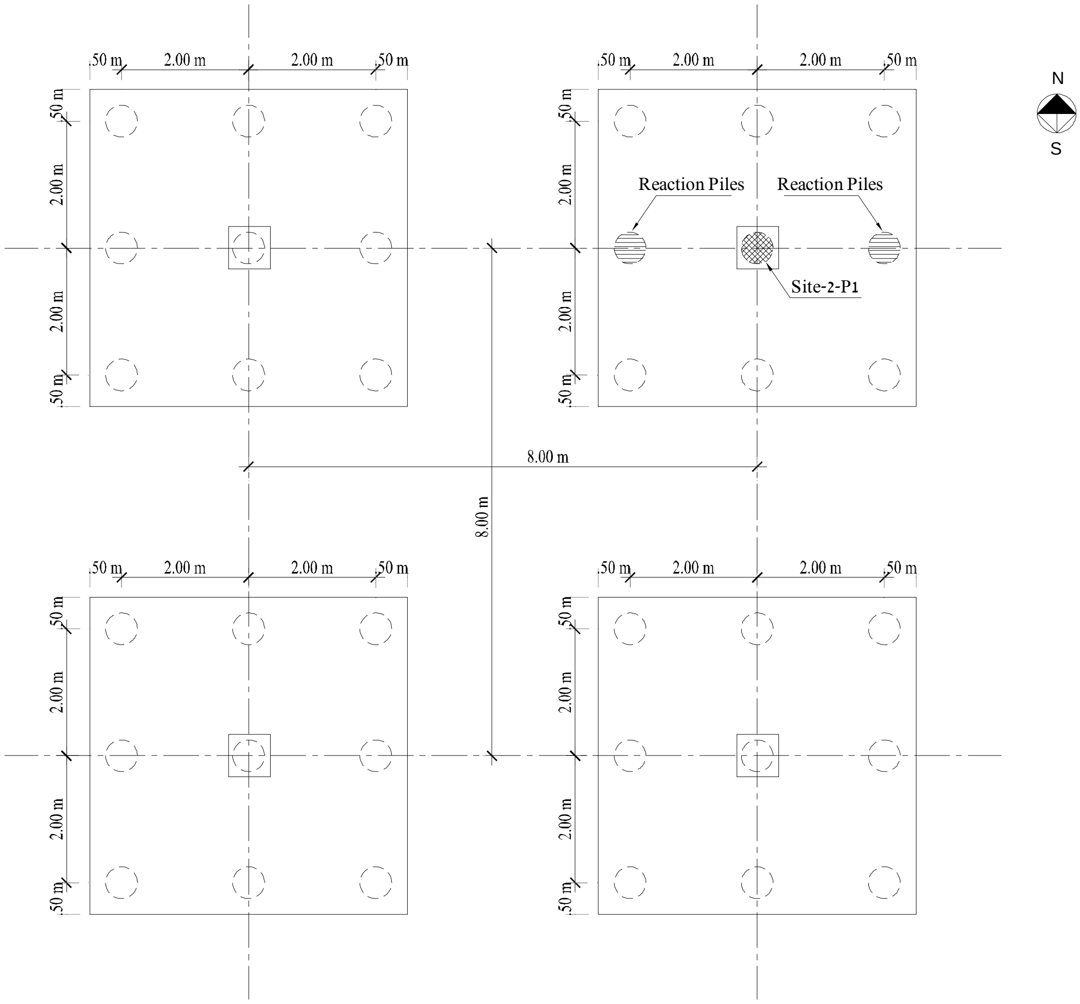


Figure 3.6: Location of test piles on pile layout at Manikganj site (S-02)

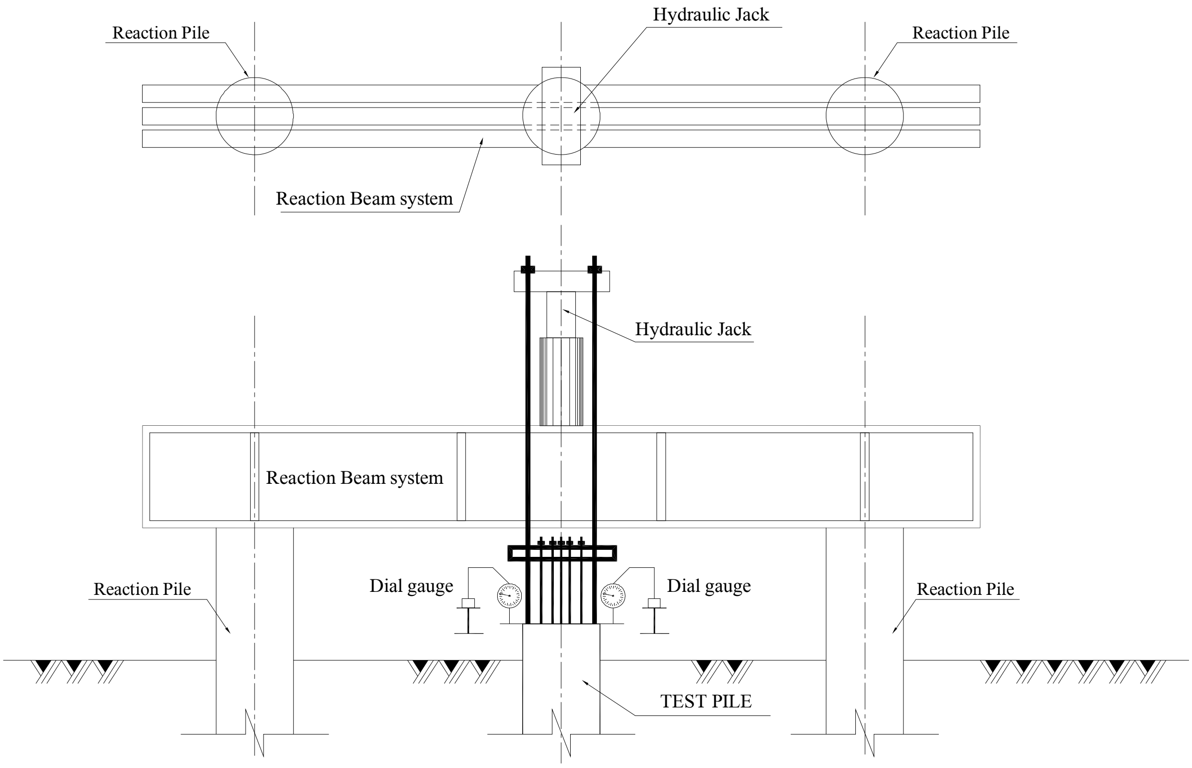


Figure 3.7: Schematic diagram of the test set-up using reaction pile at Manikganj site (S-02)

3.7 Determination of Pile Capacity from Load Tests

3.7.1 Compression Capacity

A number of arbitrary or empirical methods are used to serve as criteria for determining the allowable and ultimate load carrying capacity from pile load test. Some are based on maximum permissible gross or net settlement as measured at the pile butt while the others are based on the performance of the pile during the progress of testing (Chellis, 1961; Whitaker, 1976; Poulos and Davis, 1980; Fuller, 1983). Most commonly practiced criteria used for evaluating the ultimate and allowable load carrying capacity of piles in Bangladesh are given below:

(1) A very useful method of computing the ultimate failure load has been reported by Davisson (1973). This method is based on offset method that defines the failure load. The elastic shortening of the pile, considered as point bearing,

free standing column, is computed and plotted on the load-settlement curve, with the elastic shortening line passing through the origin. The slope of the elastic shortening line is 20°. An offset line is drawn parallel to the elastic line. The offset is usually 0.15 inch plus a quake factor, which is a function of pile tip diameter. For normal size piles, this factor is usually taken as 0.1D inch, where D is the diameter of pile in foot. The intersection of the offset line with the gross load-settlement curve determines the arbitrary ultimate failure load.

(2) Terzaghi (1942) reported that the ultimate load capacity of a pile may be considered as that load which causes a settlement equal to 10% of the pile diameter. However, this criterion is limited to a case where no definite failure point or trend is indicated by the load-settlement curves (Singh, 1990). This criterion has been incorporated in BS 8004: 1986 which recommends that the ultimate capacity of pile should be that which causes the pile to settle a depth of 10% of pile width or diameter.

(3) According to IS: 2911 (Part-VI)-1979 ultimate capacity of pile is smaller of the following two:

(a) Load corresponding to a settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed or large diameter cast in-situ pile.

(b) Load corresponding to a settlement of 12 mm.

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

(5) According to Indian Standard Code of practice (IS: 2911 – 1979), allowable pile capacity is smaller of the following:

(a) Two thirds of the final load at which the total settlement attains a value of 12 mm.

(b) Half of the final load at which total settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed pile.

(6) According to the Code of Practice 2004 of British Standards Institution (1972), the allowable pile capacity should be 50% of the final load, which causes the pile to settle a depth of 10% of pile width or diameter.

3.7.2 Uplift Capacity

Two methods have been selected for determining the uplift capacity of pile in tension in this study.

Method 1: Load Corresponding to Settlement 0.5 to 1.0% of Pile Diameter, $\Delta_{0.05Dto0.10D}$

Tomlinson (1995) stated that an upward movement of 0.5 to 1.0% of the pile width is required to mobilize the peak resistance. Because this movement is very small the tapered pile is unlikely to show any significant difference between the skin-frictional resistance in compression and tension, although it is clear that the latter falls off very quickly as the tapered pile is lifted out of the ground.

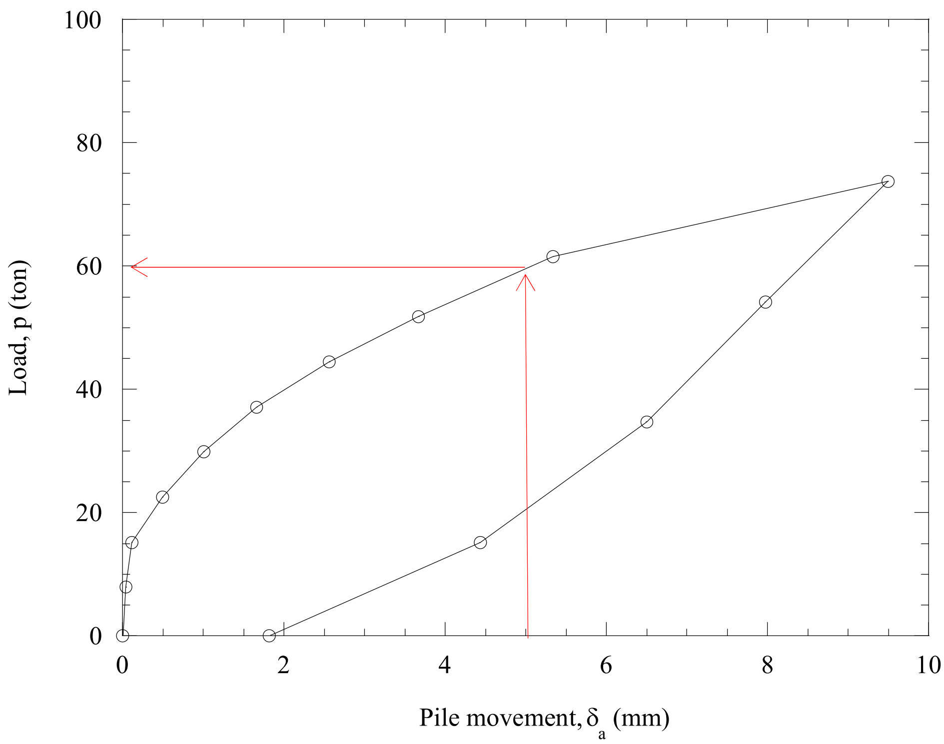


Figure 3.8: Ultimate load determination based on settlement criteria (Tomlinson, 1995)

Method 2: Double tangent method (after Butler and Hoy, 1976)

The slope of the tangent method-states that the intersection point of the tangent at the initial straight portion of the load-settlement curve and the tangent at a slope point of 1.27 mm/ton determines the arbitrary ultimate failure load (Butler and Hoy, 1976).

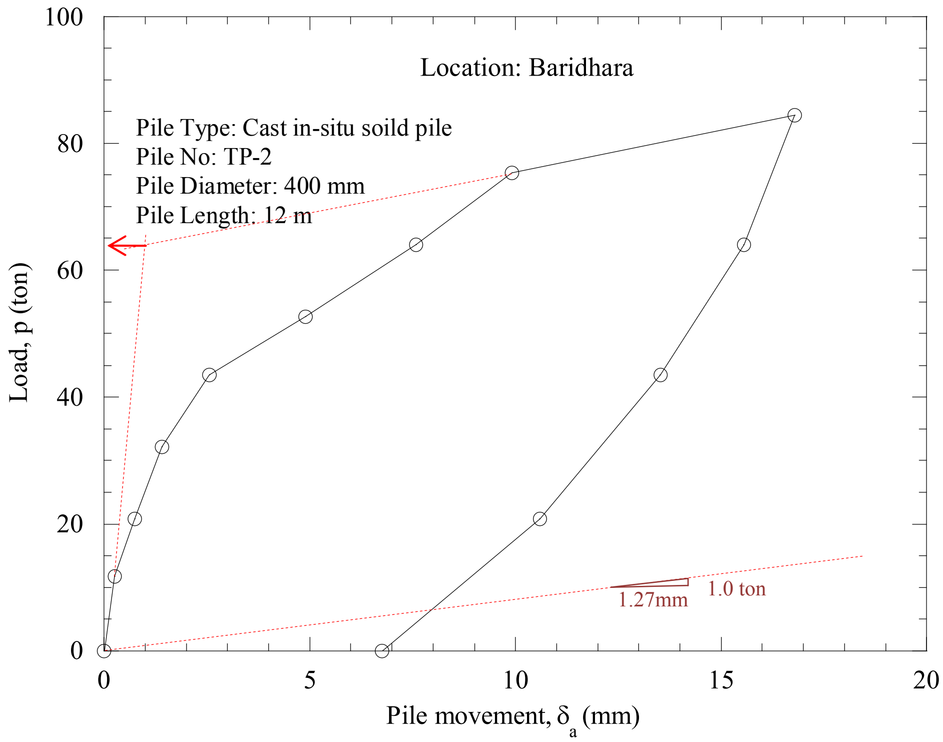


Figure 3.9: Double tangent method for determining uplift capacity of pile (Butler and Hoy, 1976)

3.8 Estimation of Pile Capacity from Sub-soil Characteristics

Piles which are used in large towers and chimneys or dry docks can go in tension under uplift loads and overturning moments. This subject is fully described by Tomlinson (1977). Similarly, expansion of top layers of expansive soils like black cotton soils can produce uplift piles. The uplift resistance of straight sided friction piles are calculated in the same way as explained for compression piles when the L/d ratio is greater than 5 (Bearing resistance of the pile is absent when the pile is in pure tension). In the case of shorter lengths (when L/d< 5) there is a likelihood of reduction of the frictional resistance. A factor of safety of 3 is recommended for tensile strength. It should also be noted that generally the movement necessary to mobilize the skin friction and hence tension in pile is small.

Tension piles for towers can be constructed with enlargement of the base in which case strength of a part of the soil above the base of the pile can also be made to resist the uplift forces as in the case of under reamed piles. Drilled in rock anchors are also commonly used to resist tension forces and details of their design are explained in Tomlinson (1977).Past experience shows that in sands the resistance in tension is only 2/3 that of skin friction value in compression. In clays they develop more or less the same skin friction as in compression. As a rule we may assume that the ultimate tension capacity of a friction pile in two-third (2/3) its ultimate skin friction capacity in compression. The tension capacity of piles in different soil and length versus diameter ratio are presented in Table 3.6.

Table 3.6: Tension capacity of pile in different soil condition

Case	L/d>5	L/d<5	Soil Type
1	Tension capacity= 2/3 of skin friction value of Compression	------	Sand
2	Tension capacity= skin friction value of Compression	------	Clay
3	Tension capacity= 2/3 of skin friction value of Compression	------	For any soil

Some of the common methods for determining pile capacity are described below.

Method 1: Tomlinson, or (α) method

The ultimate pile capacity, Q_u of a pile is considered as the summation of shaft resistance Q_s and point resistance, Q_p. In tension case the point or end bearing do not play any role in the capacity unless the construction or the shape of the pile have special design to resist tension by point action.

$$Q_u = Q_s + Q_p \tag{3.3}$$

The shaft resistance is considered to be derived from cohesion property and friction property of the soil layer. Thus the shaft resistance is taken as

$$Qs = \sum_{i=1}^{n} (f_s)_i * (A_{sp})_i \tag{3.4}$$

Here, $(fs)_i$ = shaft resistance per unit area of pile surface in the ith layer and $(Asp)_i$ = surface area of the pile shaft in the ith layer. For unit shaft resistance the following equation for α method (Tomlinson, 1971) is adopted.

$$fs = \alpha c + qK \tan\delta \qquad (3.5)$$

where α = adhesion factor, c = average cohesion for the soil strata, q = effective overburden pressure at the mid height of the layer, K = coefficient of lateral earth pressure and δ = effective friction angle between pile surface and soil in the layer.

The adhesion factor α is reported to depend on cohesion c, Over Consolidation Ratio (OCR) of the cohesive layer and the aspect ratio, L/d of the pile (Tomlinson 1986; NAVFAC, 1982). In the present analysis α is considered to linearly vary from 1.0 for

c = 0 to 0.5 for c = 200 kPa i.e.

$$\alpha = -0.0025\, c + 1.0 \qquad (3.6)$$

The cohesion c may be obtained from N value (Meyerhof, 1976) as

$$C = \chi_m . N_f \qquad (3.7)$$

where, N_f is field SPT N-value and χ_m is a constant.

In the present analysis $\chi_m = 5$ is used for unit of c in kPa.

The coefficient of earth pressure K depends on the soil type, initial soil density, method of pile installation (bored cast-in-situ, driven), volume of soil displacement etc. For cast-in-situ piles the value of K may be smaller than K_0 (coefficient of lateral earth pressure at rest) due to inward yielding of surrounding soil. In case of precast driven piles, surrounding soil is compressed and densified. As a result, in that case K may be assumed to be in between K_o and K_p (coefficient of lateral earth pressure at passive condition). Thus K was taken as:

$$K = m\, K_0 = m\,(1-\mathrm{Sin}\varphi) \qquad (3.8)$$

Value of m can be in the range of 1 to 3 depending on volume of soil displaced by pile. In the present analysis m = 1.8 was assumed for the precast driven piles and m = 1.0 for cast-in-situ piles.

The value of δ depends on the pile material, soil type and on the normal pressure at the pile-soil interface and should not exceed the limiting value of φ'. In the present analysis δ is taken as

$$\delta = 0.75\ \varphi' \tag{3.9}$$

The effective angle of internal friction of the layer possessing frictional property is taken as

$$\varphi' = \sqrt{18N_{60}} + 15 \tag{3.10}$$

N_{60} is the corrected (for overburden) SPT N-value for 60% energy. The equipment and practice of SPT used in Bangladesh is considered to be consistent with an energy ratio of 60% and therefore, field SPT-N value, N_f after correction for overburden pressure is regarded as N_{60}. For sand layers the N_{60} values were corrected for overburden pressure.

r is reduction factor for 'bentonite slurry' or 'drill mud' used for borehole drilling. The drill mud develops a thin soil zone of reduced friction around the pile. Therefore, r = 0.7 is taken for cast-in-situ piles whereas for driven piles r =1.0 is used.

The bottom of the piles rested in clay layer and following Bowels (1996) the point resistance Q_p is computed as:

$$Qp = c\ N_c\ d_c + q\ N_q d_q \tag{3.11}$$

Method 2: The German Code of Practice – DIN 4014

This code does not distinguish between the shaft resistance in tension and compression. The relation is based on a large number of tests for both cased and uncased borings. The shaft friction can be obtained based on SPT as:

$$F_s = 2.86N \tag{3.12}$$

In which Fs is the shaft resistance in kN/m^2 and N is the SPT value.

Method 3: British/American Method

It is suggested to calculate the unit side friction from the following equation

$$\tau_S = \sigma'_r . \tan \delta = k . \sigma'_v . \tan \delta \tag{3.13}$$

where, k = 0.90 for all sands and 0.6 for silt, σ_v is the vertical effective stress and δ is the angle of friction in the interface between the pile and the soil which can be taken between φ_{peak} to φ_{cv}. No distinction is made between the values in tension and compression.

Based on 41 piles test, the following equation is proposed for the unit side friction

$$\tau S = \beta . \sigma' z \tag{3.14}$$

where $\beta = 1.5 - 0.245\ Z^{0.5}$, z is the depth below ground level and, σ'_Z is the vertical effective stress at depth z. It is assumed that $0.25 < \beta < 1.20$ and $\tau s < 200$ kPa.

For SPT values lower than 15 it is recommended to scale down the side resistance by the reduction factor N/15.

Method 4: Meyerhof (1956)

Meyerhof's Formula for Driven Piles in Sand

Meyerhof's formula is given in IS 29l 1for driven piles in sands. The capacity of piles in sand is to be calculated form results of SPT values of soil. In 1959, Meyerhof proposed the following formula for the ultimate bearing capacity of driven piles in cohesionless soils. In this formula the value of N used should be the corrected SPT values.

$$Q_u = 4N_p A_p + \frac{\overline{N} A_s}{50} \text{ (U.S. tons with areas expressed in square feet)} \tag{3.15}$$

where N_p = SPT value of tip and $\overline{N}$ = average SPT on the region of the shaft.

When, A_p and A_s are expressed in square meters, we get the approximate value in metric tons

$$Q_u = 40NA_p + \left(\frac{\overline{N}}{5}\right)A_s \quad \text{(Assume 1 square meter = 10 square feet)} \quad (3.16)$$

This formula is valid for piles with L/D equal to or greater than 10. In order to take the effect of lower I/D values also into account, the formula is also expressed (with A_p and A_s are in m^2) as follows

$$Q_u = 4\left(\frac{L}{D}\right)N_p A_p + \frac{\overline{N}A_s}{50} \quad (3.17)$$

where, $\frac{L}{D}$ is not greater than 10.

According to the above formula, the shaft friction value in sand is $\left(\frac{\overline{N}}{5}\right)$ t/m^2. However an upper limit of 6 tons/m^2 corresponding to $\overline{N}$ = 30 is suggested for the shaft resistance in sand.

Similarly, the limiting base bearing capacity works out to (40N) t/m^2. However, a limiting value 1000t/m is usually adopted for bearing value. This works out as N = 25 as the critical value.

Extension of Meyerhof's formula to Non-plastic silt and Fine Sand

For non-plastic silt and fine sand, the above formula can be modified as follows (IS 2911)

$$Q_u = 3\left(\frac{L}{D}\right)NA_p + \frac{\overline{N}A_s}{6} \text{ tons (where areas are in m}^2\text{)} \quad (3.18)$$

$$Q_u = 30NA_p + \left(\frac{\overline{N}}{5}\right)A_s \text{ tons for } \frac{L}{D} > 10 \quad (3.19)$$

where, A_p and A_s are in m^2.

Meyerhof's Approach Extended to Clay

It should be remembered that Meyerhof proposed his formula for bearing capacity of piles in cohesionless soils only. However, we may assume SPT value N as a measure of the consistency of clays and thus indirectly its cohesion values as c = N/20 kg/cm^2 or N/2 t/m^2.

Using the relationship 1 kg/cm^2 = 10 t/m^2, we have the following

$$\begin{aligned} Q_u &= 9cA_p + \alpha c A_s \\ &= 9(\frac{N}{20})(10)A_p + \alpha(\frac{\overline{N}}{20})(10)A_s \text{ (tons)} \qquad (3.20) \\ &= 4.5NA_p + (\frac{\overline{N}}{2})\, A_s \text{ metric tons (assuming } \alpha = 1) \end{aligned}$$

Where, A_p and A_s are in m^2.

Thus, we may redefine the formula as follows (in metric tons):

a) End bearing in:

Sand = 40N t/m^2 (not greater than 1000 t/m^2) (3.21)

Silt = 30N t/m^2 (3.22)

Clay = 4.5N t/m^2 (3.23)

b) Side friction in:

Sand = $\frac{\overline{N}}{5}$ (not greater than 6 t/m^2) (3.24)

Silt = $\frac{\overline{N}}{6}$ t/m^2 (3.25)

Clay = $\frac{\overline{N}}{6}$ (not greater than 7 t/m^2) (3.26)

where, N is SPT-N value.

Even though these formula for design of piles in saturated sand only, many designers use it for a preliminary estimate of pile capacity in all types of soil as shown above.

Method 5: Murthy (1992)

This method stated that the ultimate skin resistance in cohesive soil is expressed as.

$$Q_f = A_s\, q_o'\, (K_s \tan \delta)$$

where: A_s = surface area of the embedded length of the pile

q_o' = average effective overburden pressure over the depth of the pile

K_s = average lateral earth pressure coefficient

δ = angle of wall friction.

And based on SPT values the ultimate skin resistance in granular soil for bored piles is expressed as.

$$Q_u = 0.60\, N_{cor}\, A_s$$

where: Q_u = ultimate total skin resistance in kN

N_{cor} = corrected average SPT value along the pile shaft

A_s = shaft surface area in m^2

3.9 Summary

The main objective of this chapter is to highlight the experimental programs that had been carried out to conduct the research. Here, the locations of the sites for the research have been described. Field and laboratory test procedures for determining sub-soil characteristics has been discussed. Methodologies of the load tests are also presented. Different methods to determine the ultimate capacity of pile in both tension and compression using the result from load tests and the methods based on the soil properties are also presented. The discussion in the chapter can be summarized as follows:

1) Total 21 pile load tests data are presented in this study. Among these 19 piles are tested under uplift load and other two piles are tested under axial compression. Study areas are situated at different geographic locations with variable sub-soil properties.
2) Test procedure, test set-up for both the uplift load test and axial compression tests are presented briefly according to ASTM.
3) Several methods for the determination of capacity of piles both in tension and compression have been presented.
4) Five methods such as Tomlinson, the German Code of Practice (DIN 4014), Meyerhof, British/American Method and Murty's are described in this chapter.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 General

The main objective of this study is to evaluate the design methods for determining uplift capacity of bored piles. For that uplift load tests have been conducted at different sites located in different geographic locations of Bangladesh. The capacity of piles has been determined from load tests and compared with that obtained from different methods (based on sub-soil characteristics) as described. In order to achieve this objective sub-soil investigations have also been carried out.

The results are presented for each site. Ultimate tension capacity of pile from both load test and estimated from sub-soil soil characteristic data has been compared.

Load test results of thirteen sites are presented in this study. In these thirteen sites, 19 piles are tested under uplift load. Among these 19 piles, 2 piles are tested in this study, 7 pile load tests result are collected from BRTC, BUET. Other pile load test results are collected from Power Grid Company of Bangladesh and ICON Engineering Services, Dhaka. Site S-01 is located at Baridhara in Dhaka city. Site S-02, S-04, S-09 and S-13 are located in eastern part of Dhaka. Site S-03, S-05 and S-11 are located in western part of Dhaka. Site S-06 and S-07 are located in western part of Bangladesh. Site S-08 is located in Khulna. Site S-10 and S-12 are located in Rajshahi, the west part of Bangladesh. Besides, these two plies are tested at the site S- 01 under axial compression. The description of the piles is presented in Table 4.1.

4.2 Pile Capacity

4.2.1 Site-1 (Baridhara)

In this site, two piles are tested under uplift and two piles are tested under compression load. Three boreholes were drilled for determining the sub-soil characteristics.

Table 4.1 List of sites and piles characteristics

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Section Type	L/D
Bored	Site-1-P1	P01	12.00	400	Solid	30
	Site-1-P2	P02	12.00	400	Solid	30
	Site-2-P1	P03	14.00	500	Solid	28
	Site-3-Tower-56	P04	13.00	500	Solid	26
	Site-4-Tower-77	P05	9.00	500	Solid	18
	Site-5-Tower-147	P06	9.50	500	Solid	19
	Site-6-Tower-25	P07	15.00	500	Solid	30
	Site-7-Tower-28	P08	15.00	500	Solid	30
	Site-8-Tower-78	P09	11.00	500	Solid	22
	Site-9-Tower-17	P10	14.00	500	Solid	28
	Site-10-Tower-53	P11	14.50	500	Solid	29
	Site-11-Tower-112	P12	15.00	500	Solid	30
	Site-12-Tower-51	P13	14.25	500	Solid	29
Driven pile in bored hole	Site-13-TP-12-1	P14	12.00	300 Thick. 65	Hollow	40
	Site-13-TP-12-2	P15	12.00	300 Thick. 65	Hollow	40
	Site-13-TP-12-3	P16	12.00	300 Thick. 65	Hollow	40
	Site-13-TP-9-1	P17	9.00	300 Thick. 65	Hollow	30
	Site-13-TP-9-2	P18	9.00	300 Thick. 65	Hollow	30
	Site-13-TP-9-3	P19	9.00	300 Thick. 65	Hollow	30

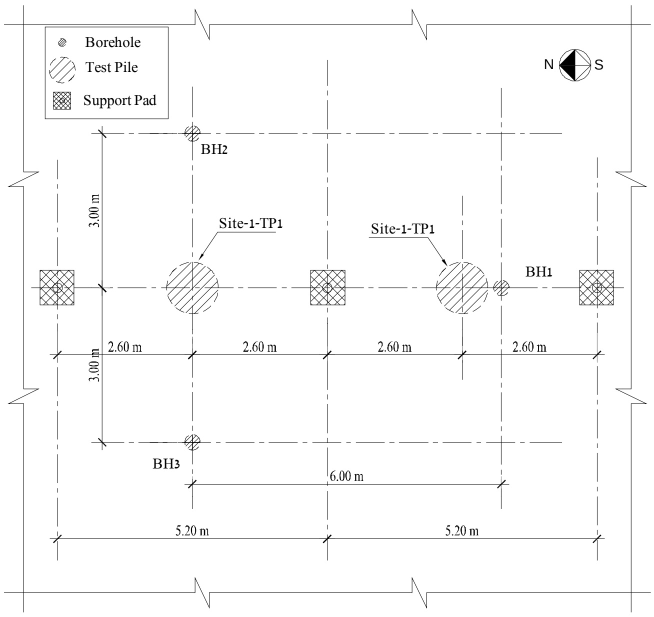


Figure 4.1: Layout plan showing the test pile location and borehole locations

Three boreholes were drilled at the site. SPT was conducted during drilling. Both disturbed and undisturbed samples were collected during boring. Sub-soil characteristics i.e. SPT and laboratory test results are presented in this section.

4.2.1.1 Sub-soil Characteristics

The site is situated at the north-east of Dhaka city. It is a private land owned by the Saudi Embassy in Bangladesh (Fig. 3.1, Chapter 3).

SPT Results

SPT was conducted at the site following procedure described in ASTM D1586. The soil profiles of the boreholes are shown in Fig. 4.1. The SPT N-values of the boreholes are shown in the Tables 4.1 to 4.3. The general soil profile of this area is grey silty clay layer with depth varying from ground level to 1.5 m from Existing Ground Level (EGL). The brown clay layer exists from 1.5 to 6.0 m depth from EGL. Silty sand layer is found at 7.5 m from EGL at BH-1. The uncorrected SPT-N value of the clay layer varies from 1 to 5. The SPT-N value of the sand layer varies from 8 to 48.

Index Properties

Typical grain size distributions of sands obtained from various depths of BH-1 are presented in Fig. 4.2. The variations of SPT N-values, mean grain size, D_{50} and fines content, F_c with depth from all boreholes are shown in Fig. 4.5. Results from grain size analysis of the soil samples are presented in Tables 4.1 to 4.3.

From Fig. 4.2, 4.3 and 4.4, it is seen that sand percentage of various samples varies from 63 to 90%. Among them, coarse sand are 2% which is common in all samples, medium sand varies from 23 to 49% and fine sand varies from 33 to 34% according to Unified Soil Classification System (USCS).

Table 4.2 Soil type, SPT N-value and index properties at BH-1

Depth (m)	Description of Soil	SPT N-Value	F_c (%)	PI (%)	D_{50} (mm)
1.5	Grey Silty Clay	1			
3.0	Silty Clay	3		31	
4.5	Silty Clay	7		29	
6.0	Silty Clay	5			
7.5	Silty sand	8	37		0.18
9.0	Silty sand	7			
10.5	Silty sand	13	33		0.19
12.0	Silty sand	16			
13.5	Silty sand	18	28		0.17
15.0	Silty sand	28			
16.5	Silty sand	33	24		0.2
18.0	Silty sand	48			

Table 4.3 Soil type, SPT N-value and index properties at BH-2

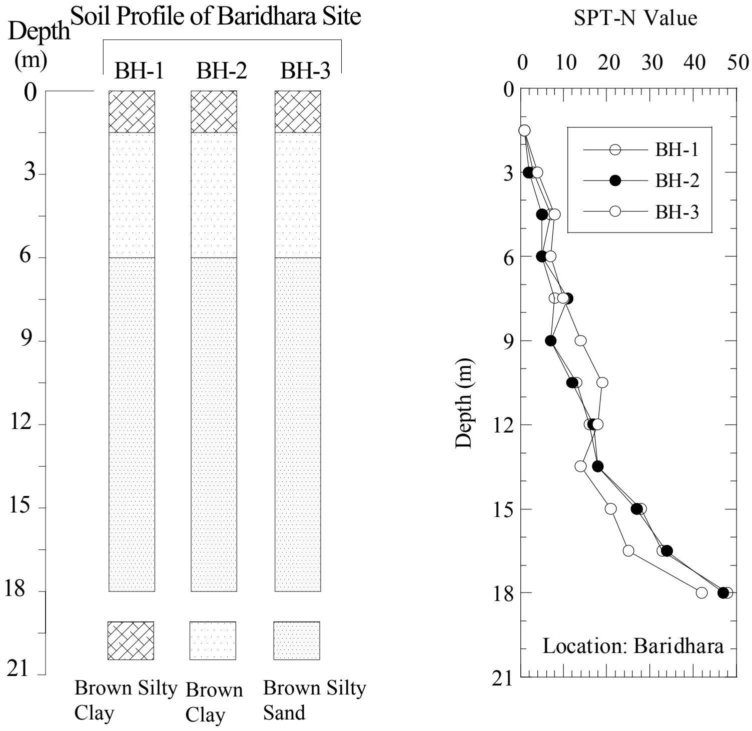
Depth (m)	Description of Soil	SPT N-Value	F_c (%)	PI (%)	D_{50} (mm)
1.5	Grey Silty Clay	1			
3.0	Silty Clay	3		28	
4.5	Silty Clay	7			
6.0	Silty Clay	5		23	
7.5	Silty sand	8			
9.0	Silty sand	7	32		0.16
10.5	Silty sand	13			
12.0	Silty sand	16	27		0.16
13.5	Silty sand	18			
15.0	Silty sand	28			
16.5	Silty sand	33			
18.0	Silty sand	48	20		0.19

Table 4.4 Soil type, SPT N-value and index properties at BH-3

Depth (m)	Description of Soil	SPT N-Value	F_c (%)	PI (%)	D_{50} (mm)
1.5	Grey Silty Clay	1			
3.0	Silty Clay	3			
4.5	Silty Clay	7		29	
6.0	Silty Clay	5			
7.5	Silty sand	8		30	
9.0	Silty sand	7			
10.5	Silty sand	13			
12.0	Silty sand	16	28		0.18
13.5	Silty sand	18			
15.0	Silty sand	28	26		0.19
16.5	Silty sand	33			
18.0	Silty sand	48	26		0.20

Strength Parameters

Unconfined compression tests and direct shear tests were performed on soil samples collected from two boreholes. Unconfined compressive strength varies from 5.6 kPa to 7.5 kPa of the soil samples collected from a depth about 5.5 to 7.0 m. The Figure 4.6 shows the unconfined strength graph for Baridhara soil. Direct shear tests were



(a) (b)

Figure 4.2: Sub-soil profile of Baridhara site (site-01): (a) sub-soil profile and (b) variation of SPT-N value with depth

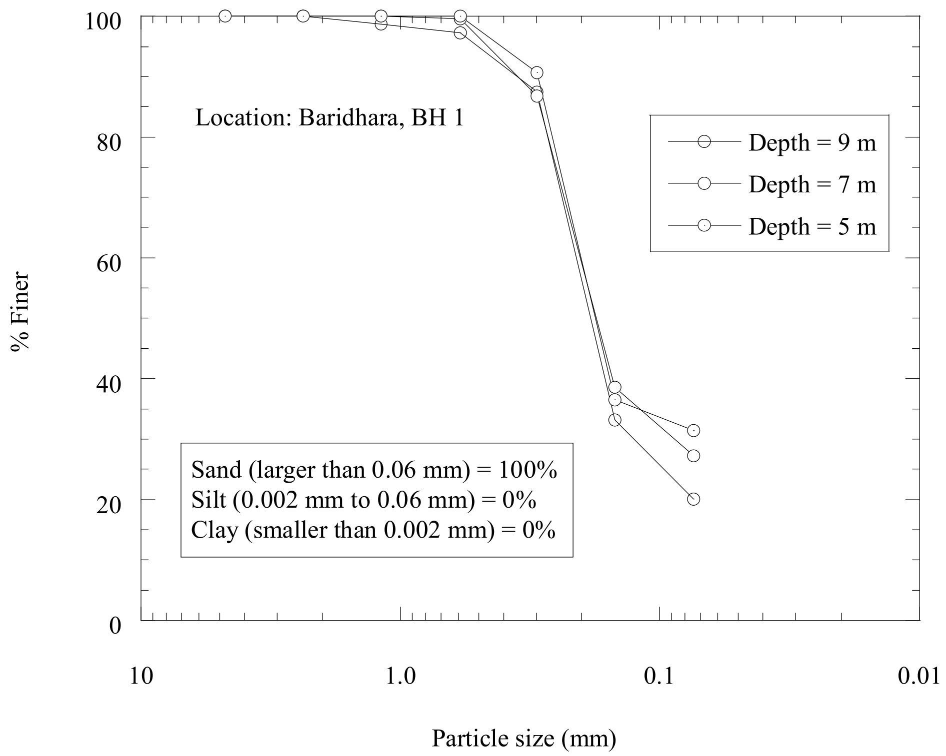


Figure 4.3: Grain size distributions of samples collected from various depths at BH-1, Baridhara

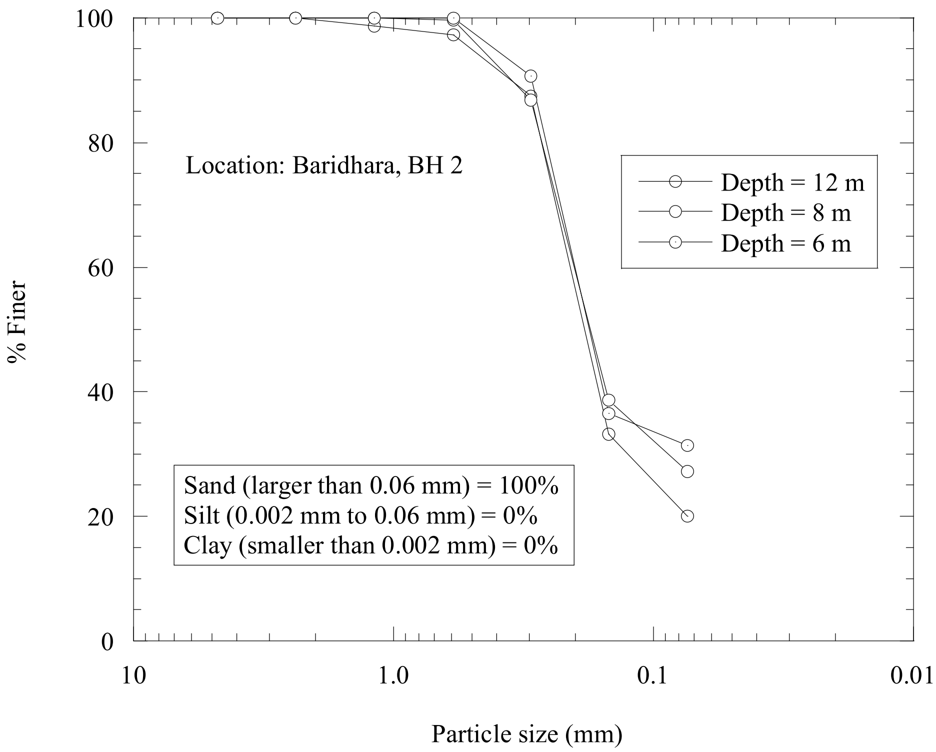


Figure 4.4: Grain size distributions of samples collected from various depths at BH-2, Baridhara

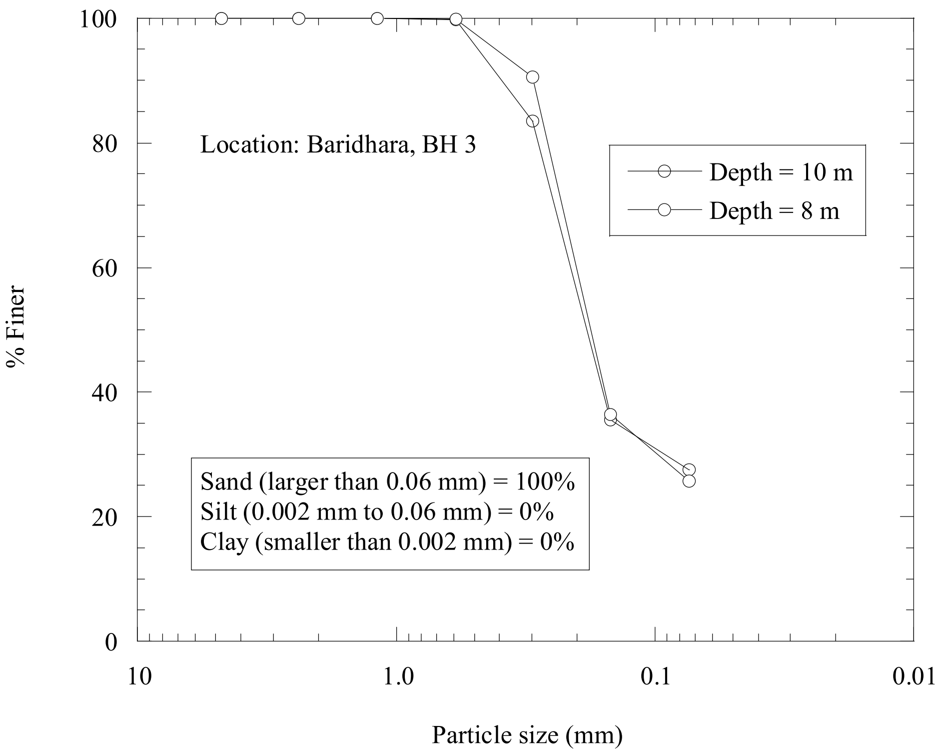


Figure 4.5: Grain size distributions of samples collected from various depths of BH-3, Baridhara

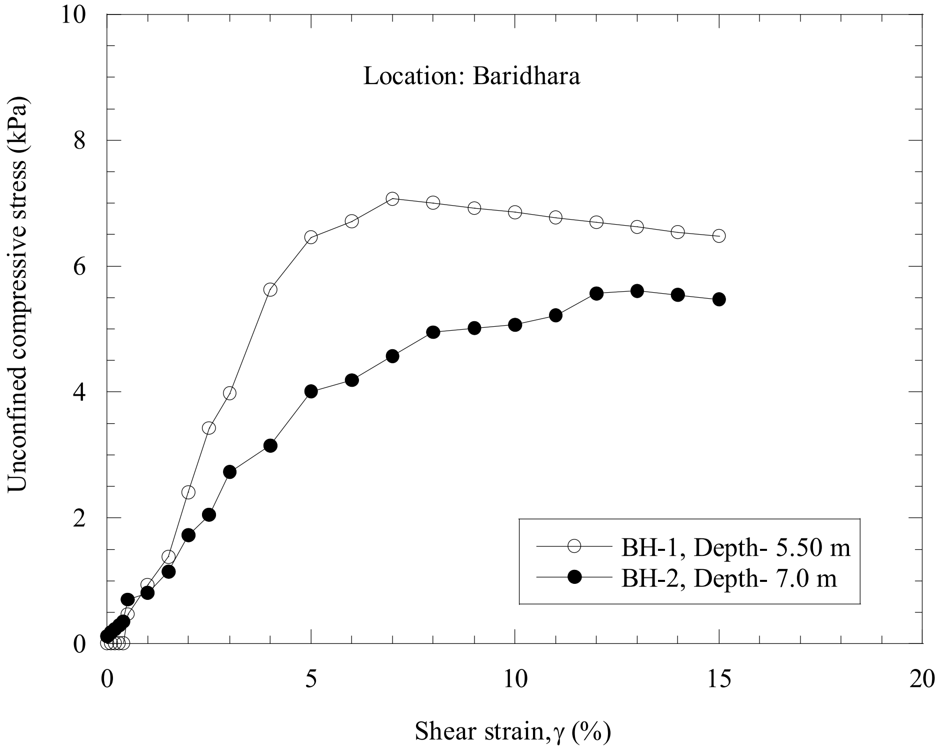


Figure 4.6: Unconfined compressive strength of soil samples collected from BH-1 and BH-2 at Baridhara site

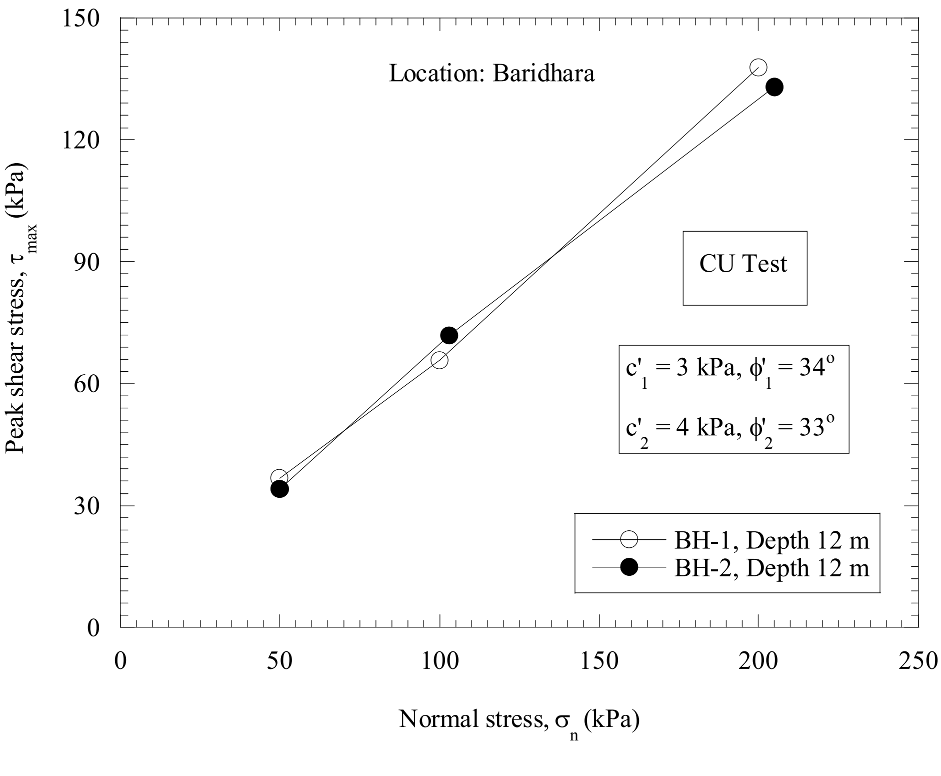


Figure 4.7: Peak shear stress versus normal stress graph for Baridhara soil

performed on two samples collected from 12 m depth from EGL from borehole one and two. The Figure 4.7 shows the peak shear stress versus normal stress graph for Baridhara soil. From the Figure 4.7, it is found that the cohesion varies from 3 to 4 kPa and angle of internal friction varies from 33° to 34°.

4.2.1.2 Load Test Results

Pile Description

Two piles were tested under uplift load at this site. The length and diameter of the piles are 12 m and 400 mm, respectively. Constructed bored piles were tested after one month of construction. Details of the pile are presented in Figure 4.8. Uplift tests were conducted at the site using the Method-1 (support on the ground) as described in Chapter 3 (Art 3.6.1). Again, two piles were tested under compression. The length and diameter of the piles are 14 m and 400 mm, respectively. Tests were conducted using the method as described in Chapter 3 (Art. 3.5.1).

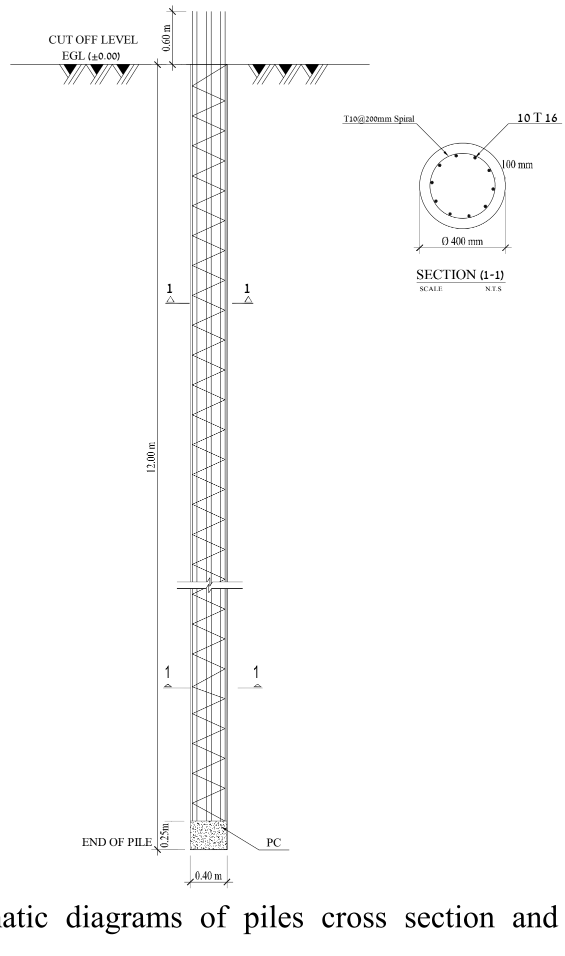


Figure 4.8: Schematic diagrams of piles cross section and length for site S-01, Baridhara

Results obtained from the load tests are presented in Figures 4.9 to Figure 4.21. Figure 4.9 presents the load versus elapsed time graph of the pile TP-1. Figure 4.10 presents the pile movement versus elapsed time graph for the pile TP-1. Load versus pile movement graph of the pile TP-1 is presented in Figure 4.11. From this graph, the ultimate capacities of the piles are determined using the methods described in Chapter 3 (Art. 3.7.2).

Figure 4.12 presents the load versus elapsed time graph of the pile TP-2. Figure 4.13 presents the pile movement versus elapsed time graph for the pile TP-2. Load versus pile movement graph of the pile TP-2 is presented in Figure 4.14. From this graph, the ultimate capacity of the piles is determined using the methods described in Chapter 3 (Art.3.7.2).

Figure 4.15 presents the load versus elapsed time graph of the pile CP-1 in compression. Figure 4.16 presents the pile movement versus elapsed time graph for the pile CP-1. Load versus pile movement graph of the pile CP-1 is presented in Figure 4.17. From this graph, the ultimate capacity of the piles is determined using the methods described in Chapter 3 (Art. 3.7.2).

Figure 4.18 presents the load versus elapsed time graph of the pile CP-2 in compression. Figure 4.19 presents the pile movement versus elapsed time graph for the pile CP-2. Load versus pile movement graph of the pile CP-2 is presented in Figure 4.20. From this graph, the ultimate capacities of the piles are determined using the methods described in Chapter 3 (Art. 3.7.2).

Uplift Capacity from Load Test

Uplift capacity of the tested piles is determined using two methods described in Chapter 3 (Art 3.7.2). Determination of results using two methods is described in Figure 4.21. The ultimate capacities of the piles are determined using the same methods for all piles.

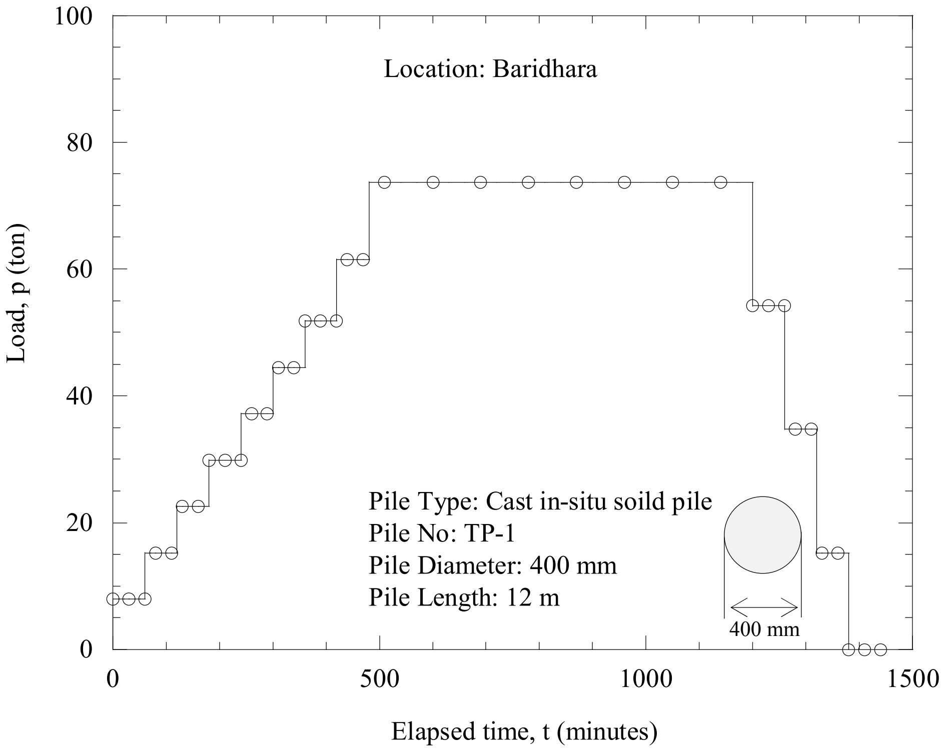


Figure 4.9: Elapsed time vs. load curve of the test pile TP-1 at Baridhara site

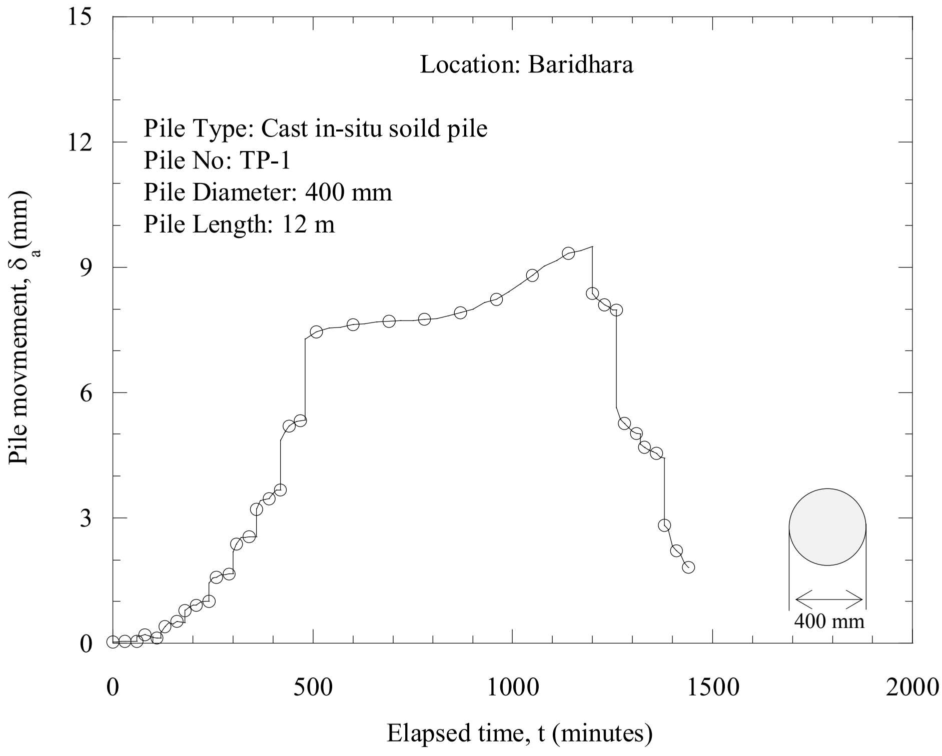


Figure 4.10: Pile movement vs. elapsed time curve of pile TP-1 at Baridhara site

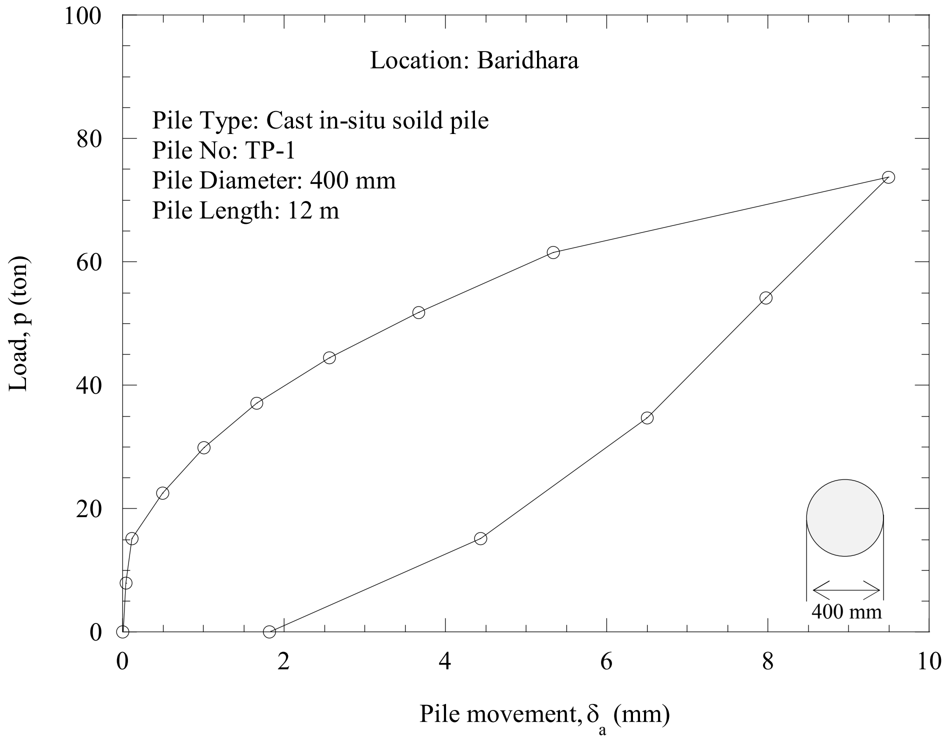


Figure 4.11: Load vs. pile movement curve of the pile TP-1 at Baridhara site

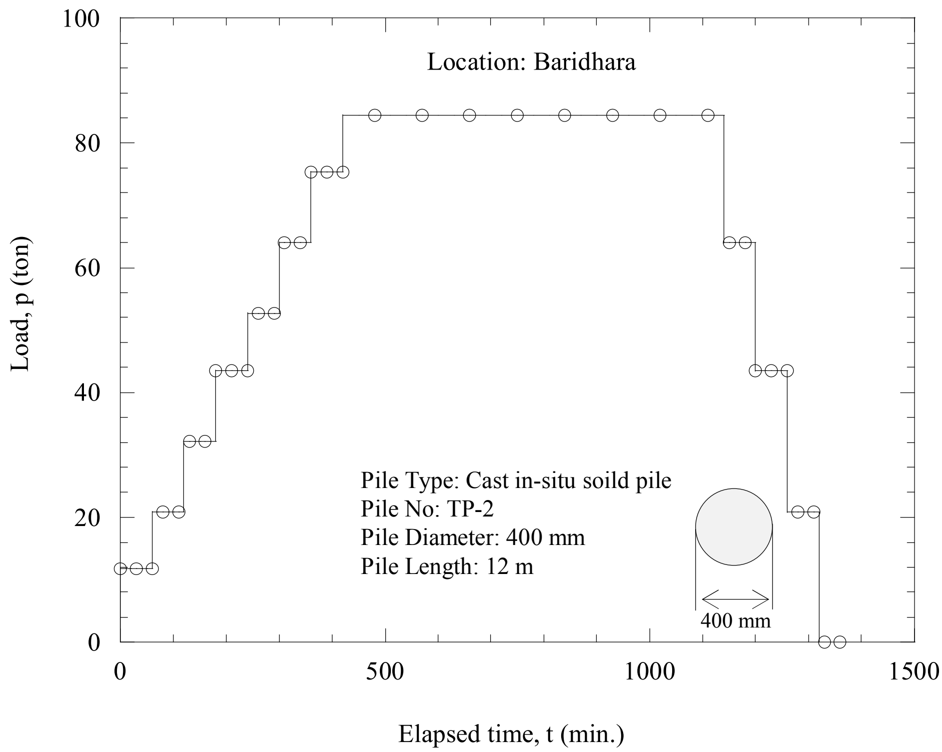


Figure 4.12: Elapsed time vs. load curve of the test pile TP-2 at Baridhara site

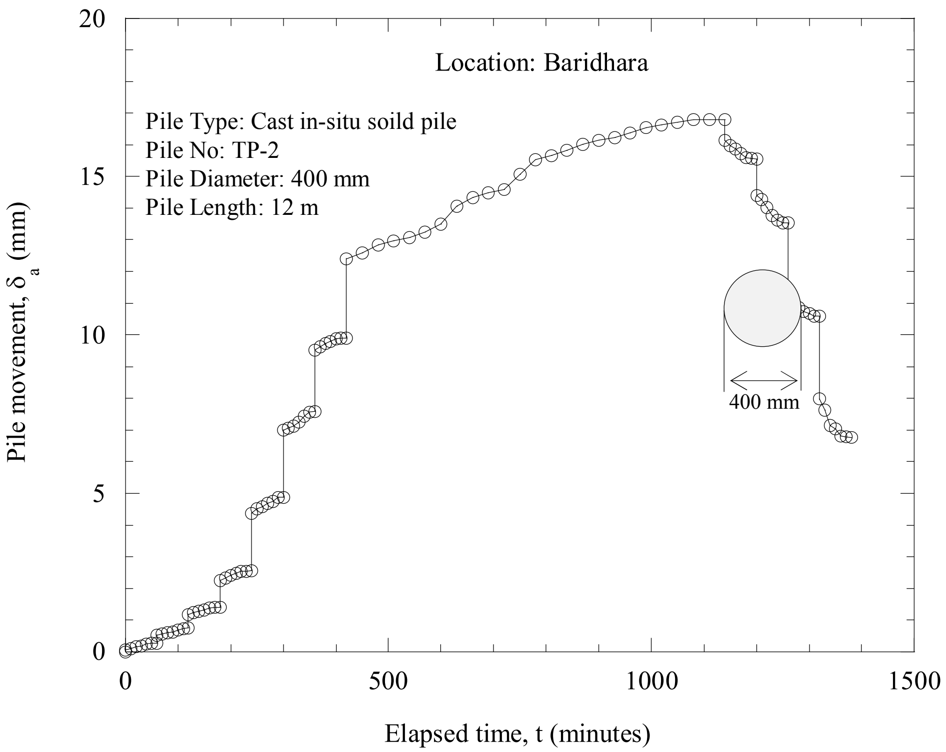


Figure 4.13: Pile movement vs. elapsed time curve of pile TP-2 at Baridhara site

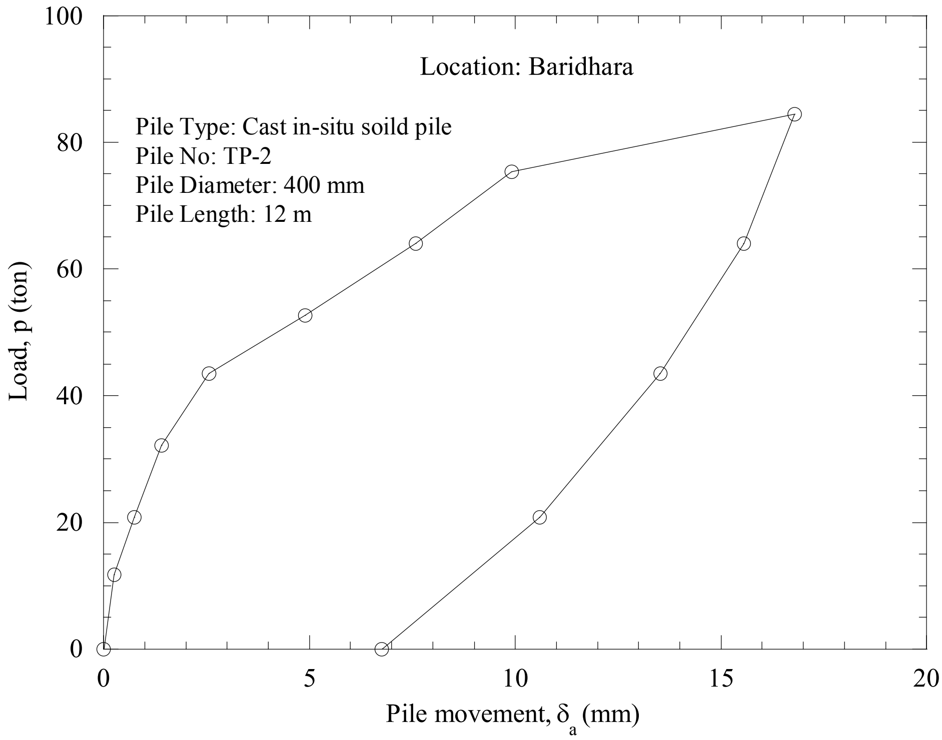


Figure 4.14: Load vs. pile movement curve of the pile TP-2 at Baridhara site

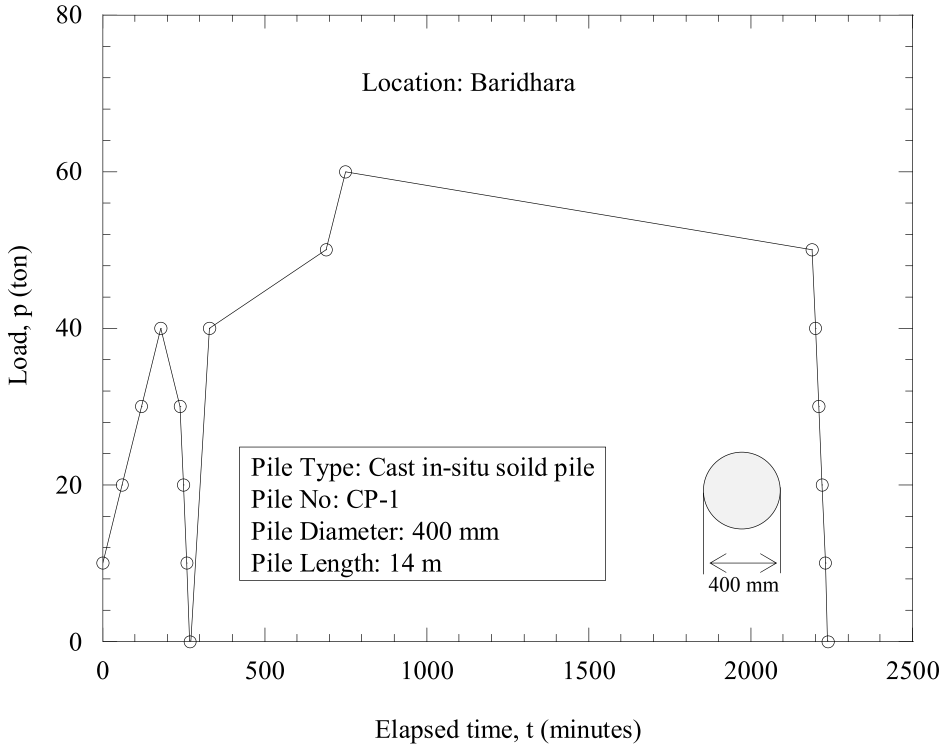


Figure 4.15: Load vs. elapsed time curve of the pile CP-1 at Baridahara site

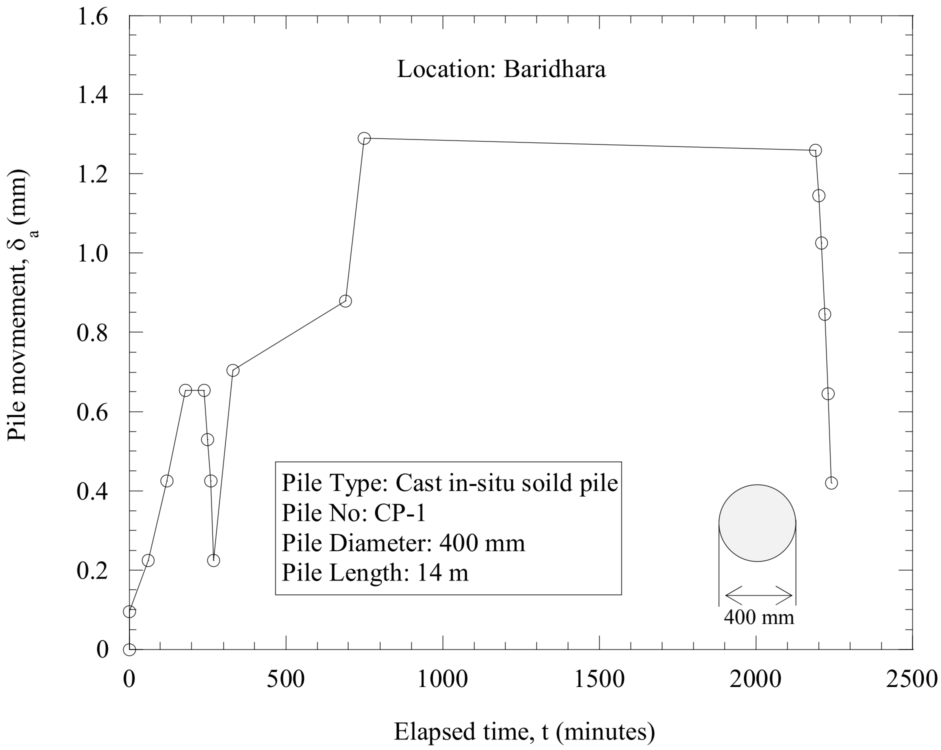


Figure 4.16: Pile movement vs. elapsed time curve of the pile CP-1 at Baridahara site

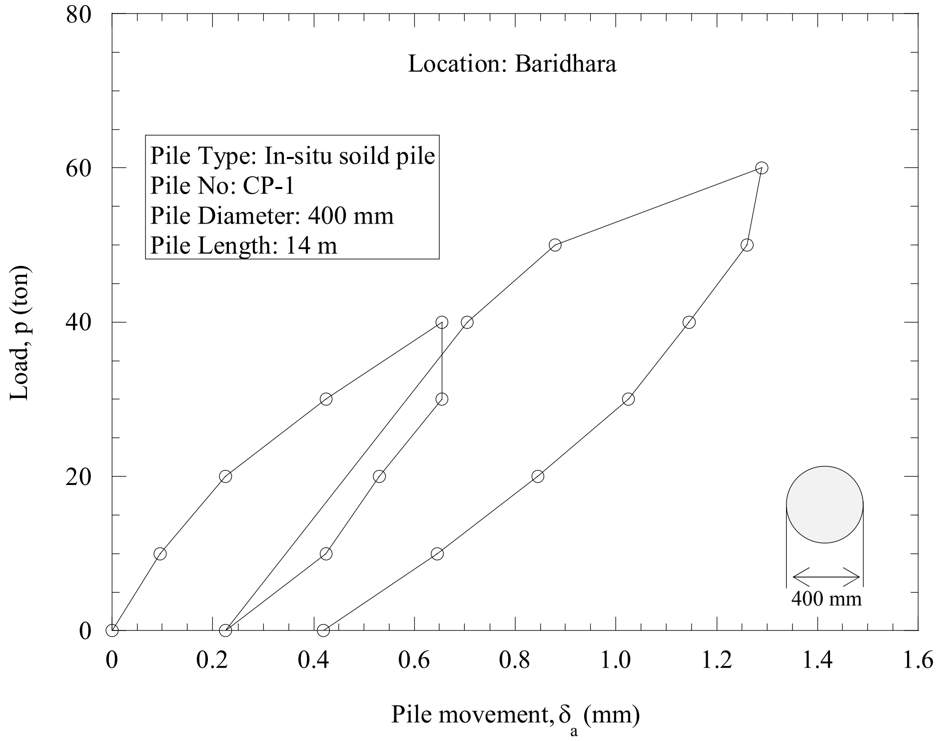


Figure 4.17: Load vs. pile movement curve of the pile CP-1 at (Baridhara site)

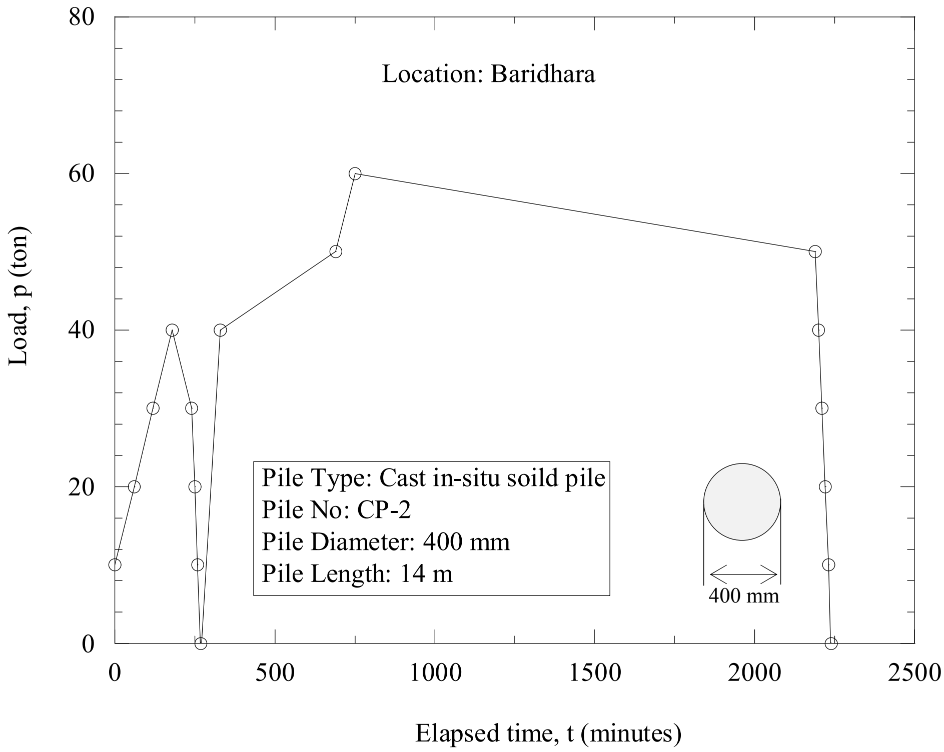


Figure 4.18: Elapsed time vs. load curve of the pile CP-2 at Baridahara site

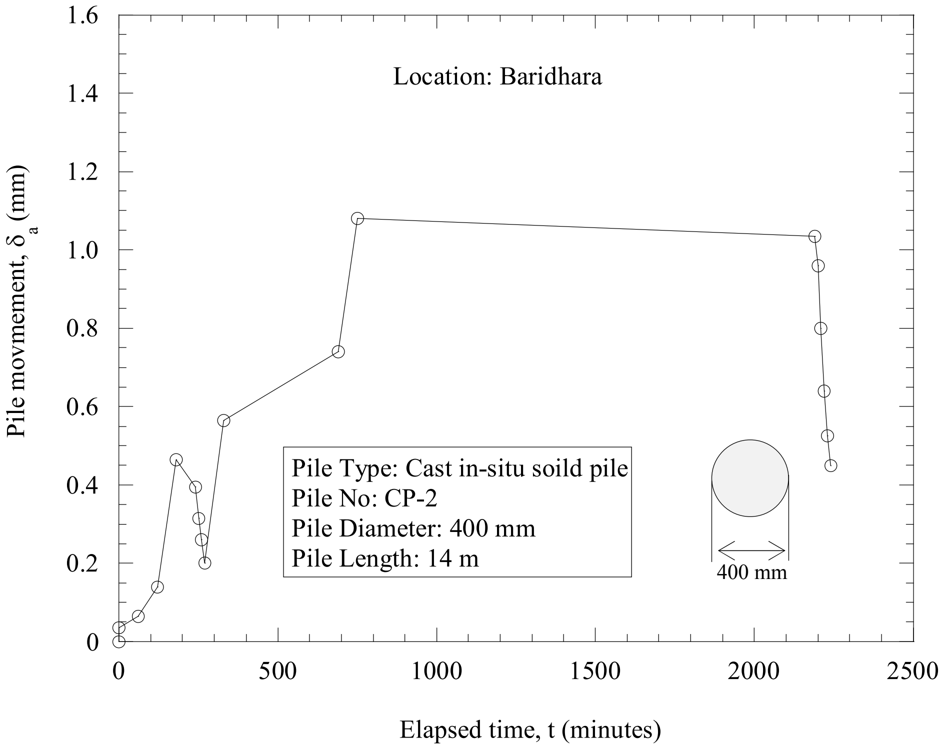


Figure 4.19: Pile movement vs. elapsed time curve of the pile CP-2 at Baridahara site

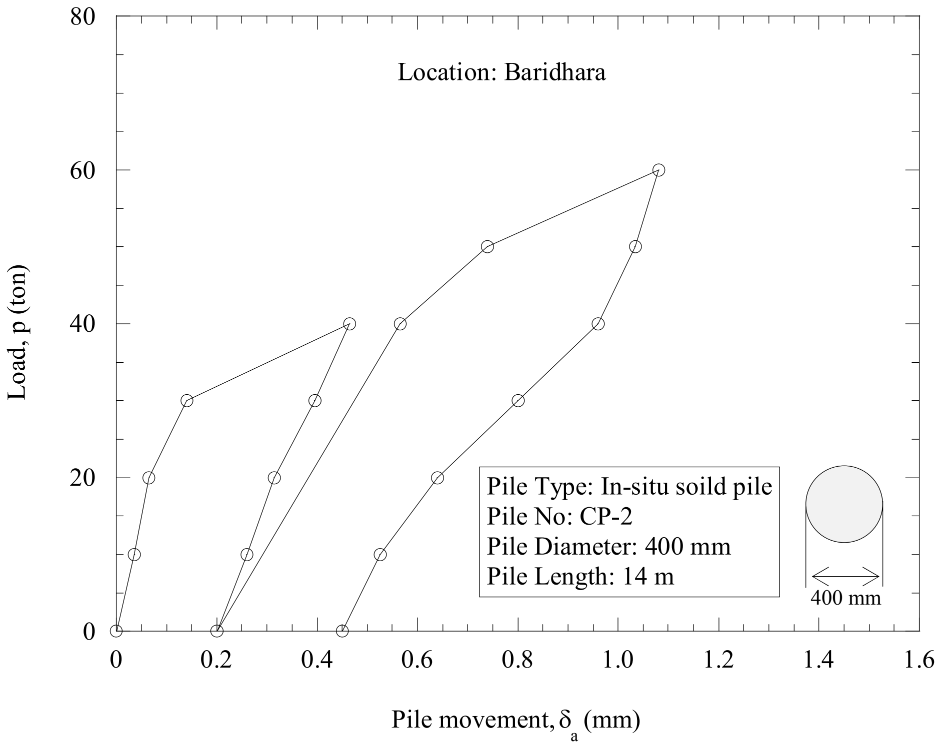


Figure 4.20: Load vs. pile movement curve of the pile CP-2 at Baridhara site

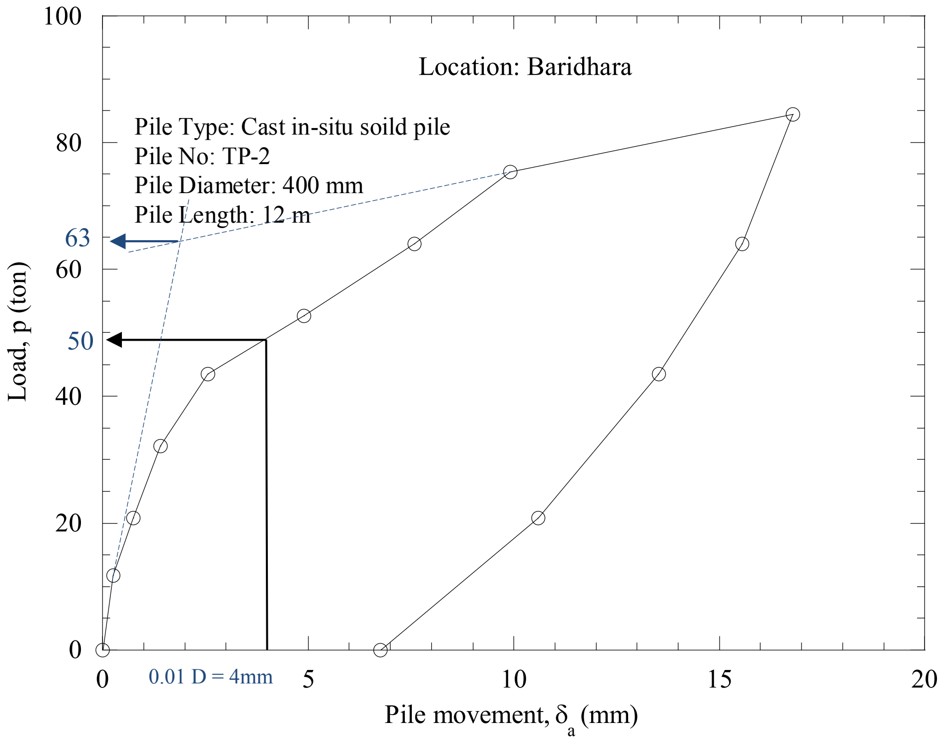


Figure 4.21: Uplift capacity of the pile TP-1 at Baridhara site

Test results of compression tests are presented in Table 4.5. It is seen that these piles were not failed at the applied load of 60 Ton. These results are presented to compare with the uplift capacities of piles.

Table 4.5 Test results of compression tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity (Ton)
Bored	Site-1-CP1	CP-1	14	400	Solid	Not failed
	Site-1-CP2	CP-2	14	400	Solid	Not failed

Uplift Capacity Estimated from Sub-soil Characteristics

Using the five methods described in Chapter 3 (Art. 3.8), pile capacity has been estimated for the piles. Uplift capacity determined from the estimation is presented in Table 4.6, it is seen that the results obtained from the load test and estimation are significantly different. The result obtained based on the SPT-N value is the lowest. This is because it only depends on one factor. The results obtained from other equations (such as Meyerhof, Murthy and Tomlinson etc.) are almost similar because these equations consider the soil parameters such as friction angle and cohesion. We cannot compare with the compression capacity of the piles with the tension capacity of piles because the piles did not reach the failure in compression test.

Table 4.6 Comparison of pile capacity determined from load test and estimation

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec .Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-1-P1	P01	12	400	Solid	51	58	24	19	27	12	26
	Site-1-P2	P02	12	400	Solid	50	63	28	22	29	13	33

4.2.2 Site-2 (Manikganj)

In this site, one pile is tested under uplift load. Three boreholes were drilled for determining the sub-soil characteristics. SPT was conducted and disturbed as well as undisturbed samples were collected from these locations. Sub-soil characteristics i.e. SPT and laboratory test results of these samples are presented in this section. Location of the test pile and reaction piles are presented in Figure 4.22.

4.2.2.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the site following the procedure described in ASTM D1586. Three boreholes were drilled at the site. The soil profiles of the boreholes are shown in Fig. 4.23. It is seen that the top 4.5 m from the EGL is brown soft to medium stiff clayey silt from Existing Ground Level (EGL). The next layer is medium dense to very dense silty sand up to the drilled depth. The SPT-N value of the clayey silty layer varies from 2 to 7. The SPT-N value of the silty sand layer varies from 14 to 50.

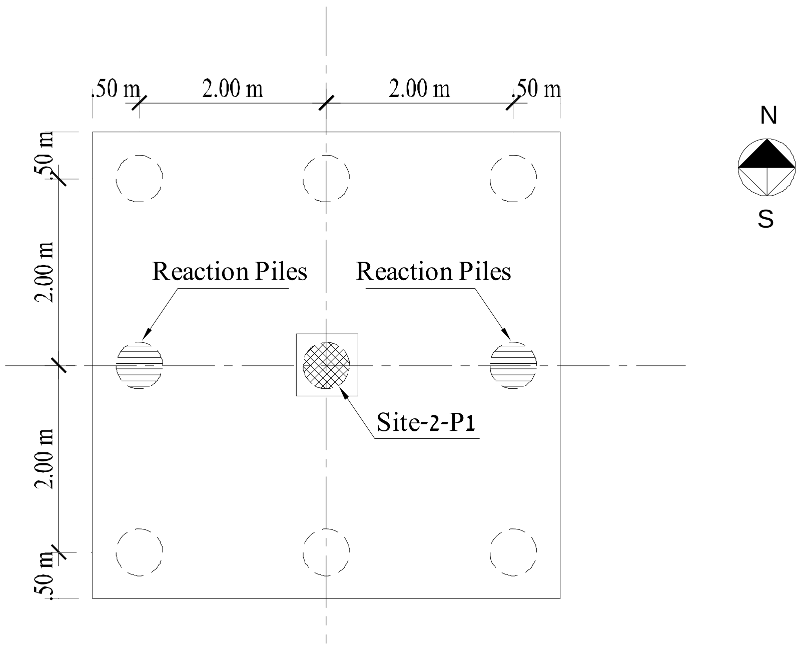


Figure 4.22: Locations of test pile and reaction piles at Manikganj site

Index Properties

Typical grain size distributions of sands obtained from various depths of BH-1and BH-2 are presented in Figs. 4.24 and 4.25, respectively. From the grain size distribution, it is seen that the soil is mainly silty sand.

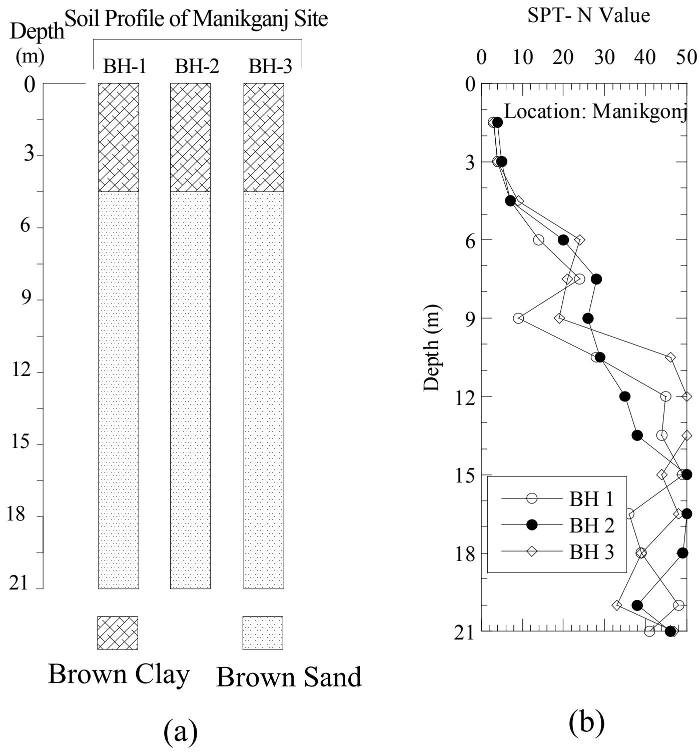


Figure 4.23: Sub-soil profile of Manikganj site (Site-2): (a) sub-soil profile and (b) variation of SPT-N value with depth

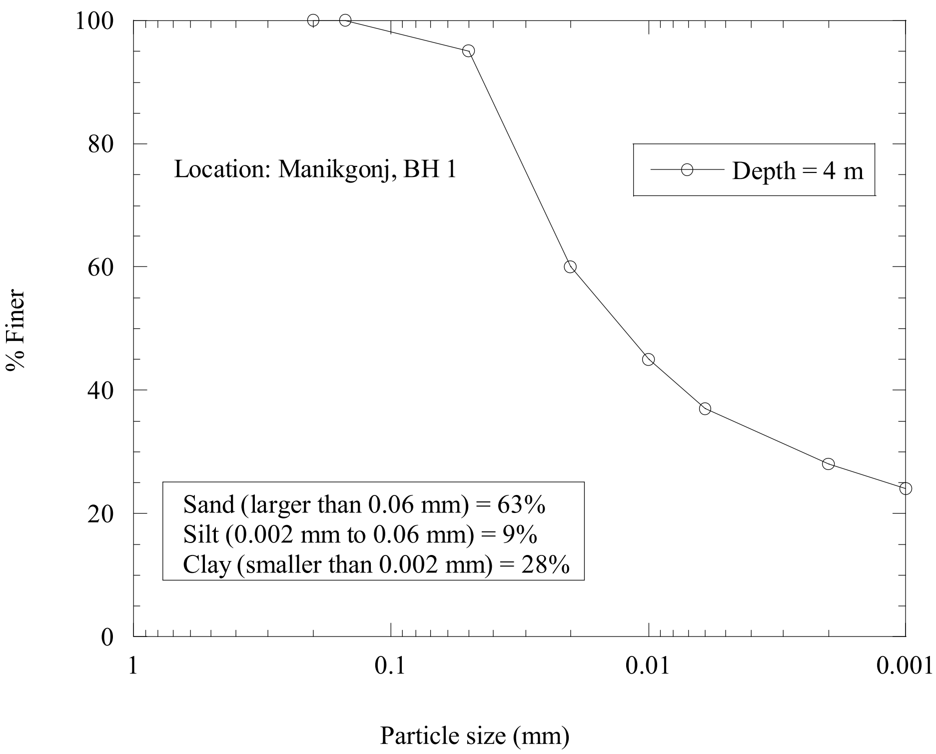


Figure 4.24: Grain size distributions of samples collected from BH-1

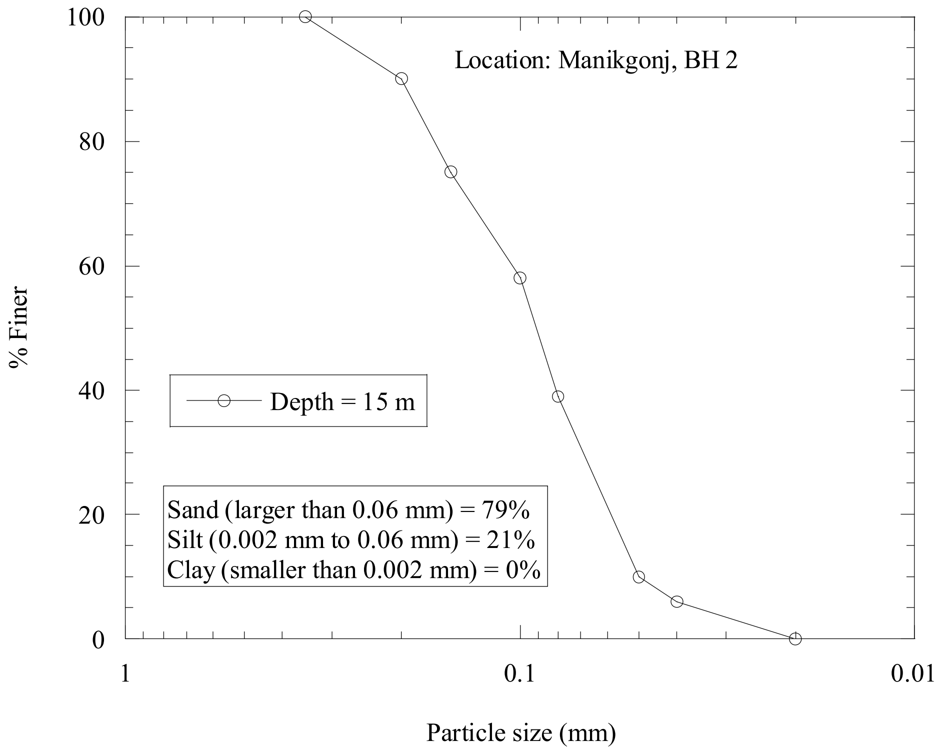


Figure 4.25: Grain size distributions of samples collected from BH- 2

Strength Parameters

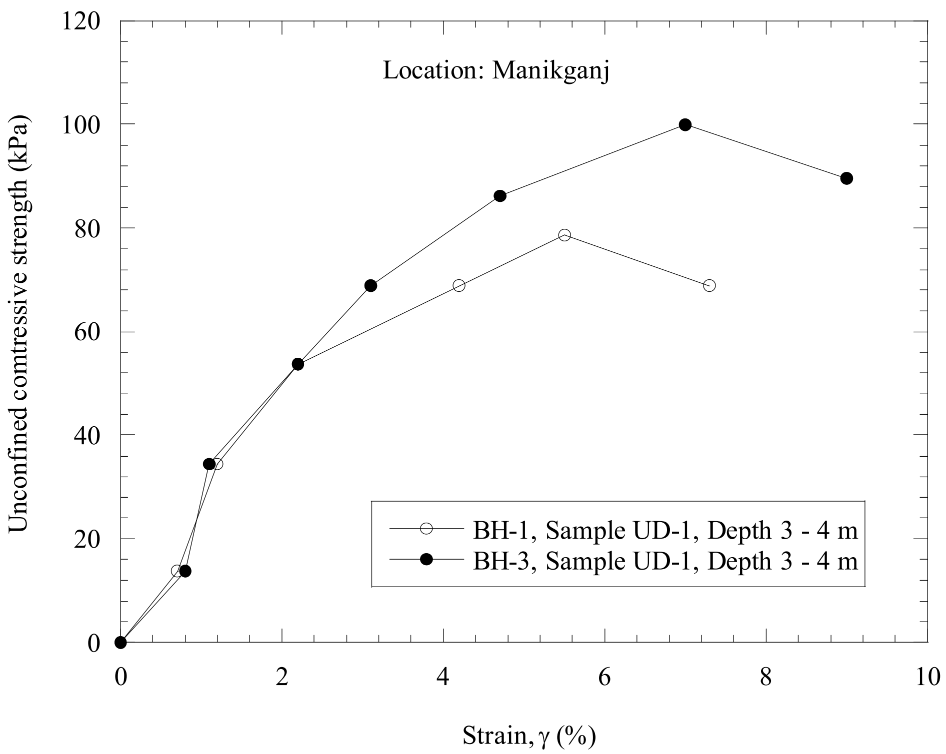


Figure 4.26: Unconfined compressive strength of Manikganj soil

Unconfined compression tests were conducted on two soil samples collected from the top layer (3 to 4 m from EGL) of BH-1 and BH-3. Figure 4.26 shows the unconfined compression test results of Manikganj soil. From the Figure 4.26, it is sheen that the unconfined compressive strength of these soil samples varies from 79 to 100 kPa.

4.2.2.2 Load Test Results

Pile Description

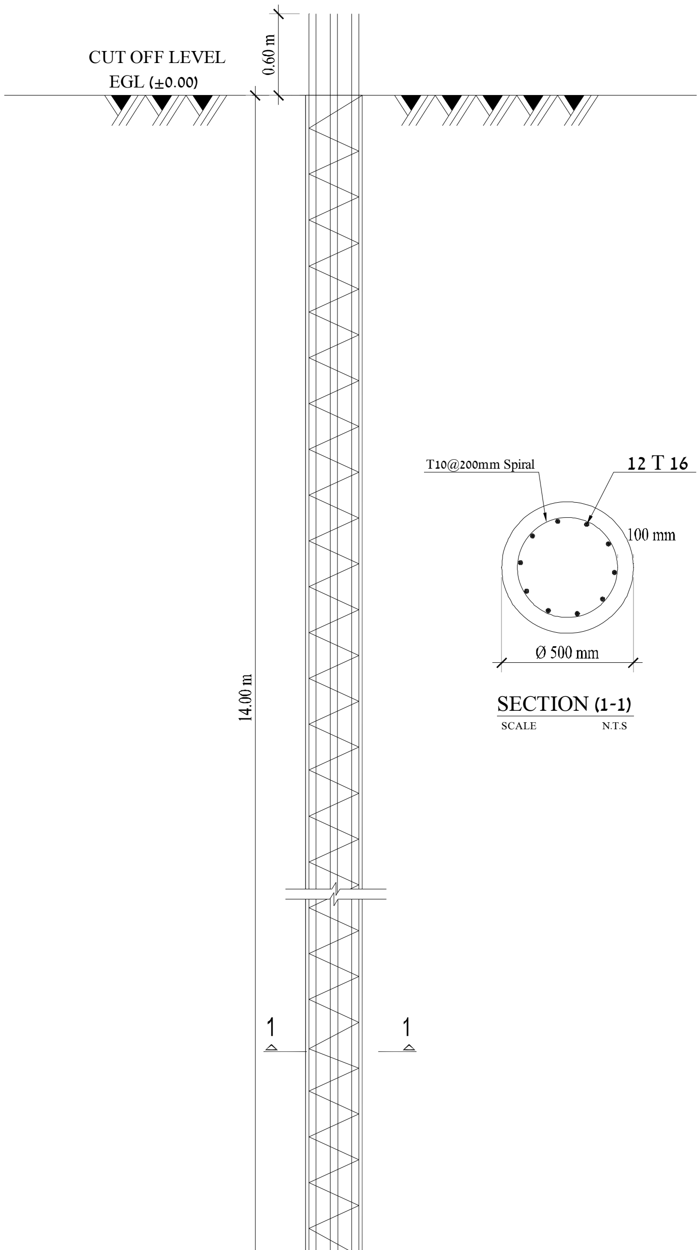


Figure 4.27: Schematic diagrams of piles cross section and length for site S-02

One pile is tested under uplift load at this site. The length and diameter of the pile are 14 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Details of the pile are presented in Figure 4.27.

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (using reaction pile) as described in Chapter 3 (Art. 3.6.2). Figures 4.28, Figure 4.29 and Figure 4.30 show the load versus elapsed time, pile movement versus elapsed time and load versus pile movement curve for pile the P3, respectively. Results obtained from the load tests are presented in the Table 4.9.

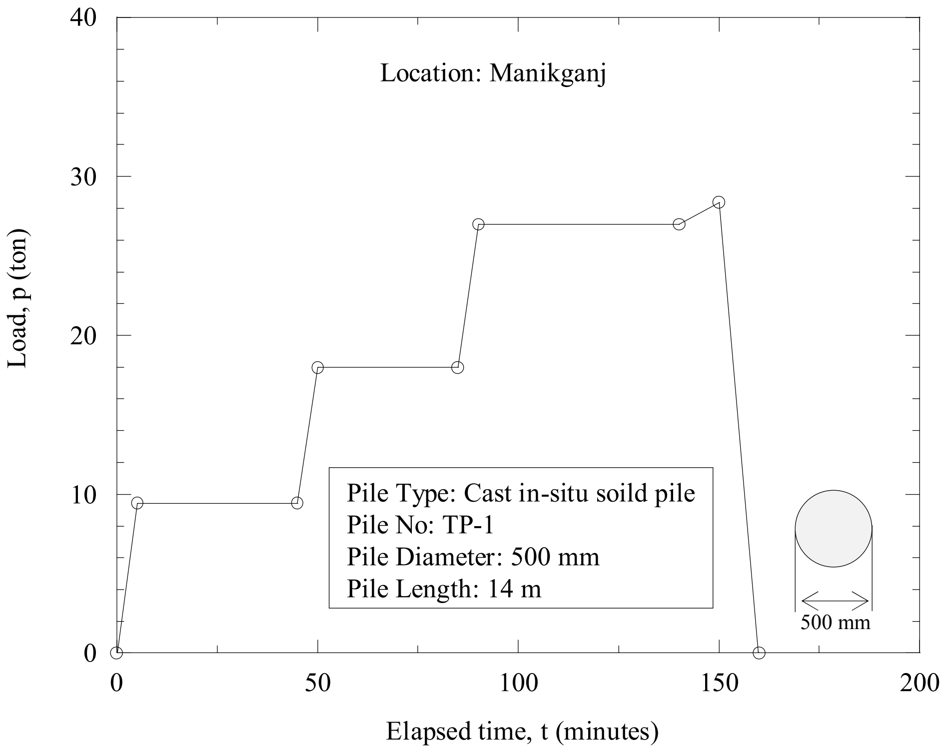


Figure 4.28: Elapsed time vs. load curve of the test pile TP-1 at site-2

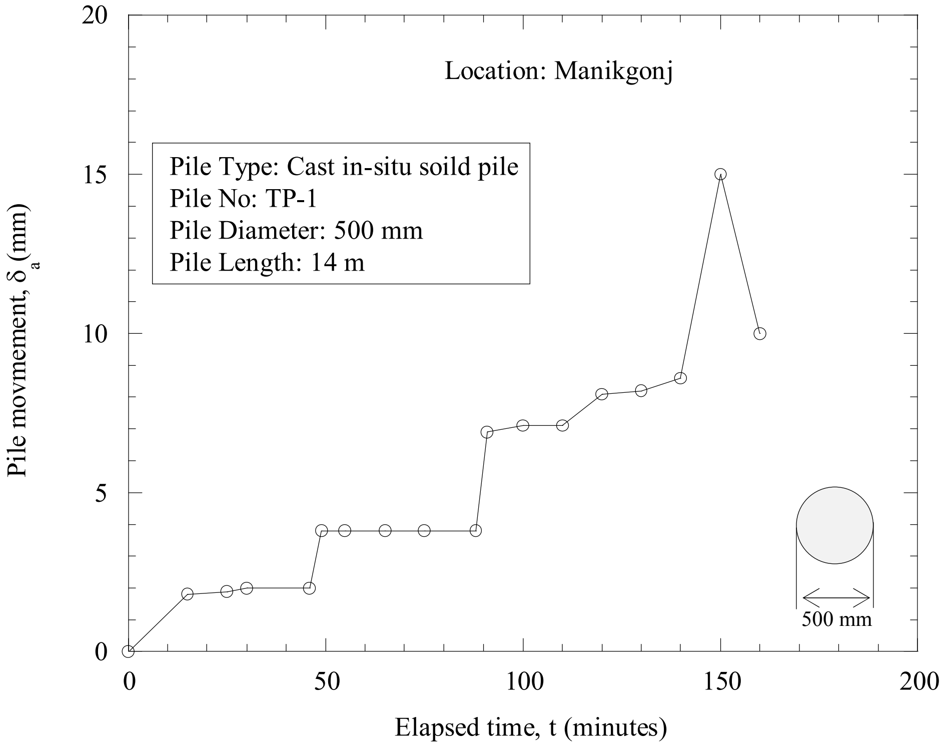


Figure 4.29: Pile movement vs elapsed time curve of the pile TP-1 at site-2

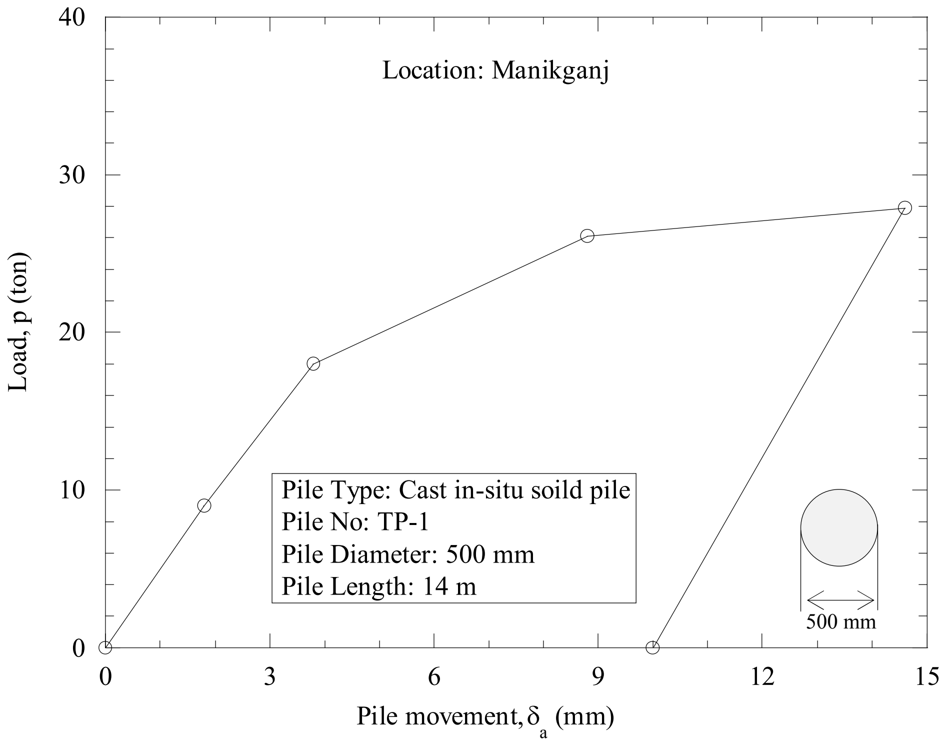


Figure 4.30: Load vs. pile movement curve of the pile TP-1 at site-2

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods as mentioned in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.7.

Table 4.7: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-2-P1	P03	14	500	Solid	20	23	64	43	47	37	100

From the results presented in Table 4.7, it is seen that the results obtained from the load test and estimation based on Meyerhof, Tomlinson and British/American method are close. However, the results from load test are smaller than those obtained from estimation based on soil parameters. The pile capacity determined from DIN method is smaller than that obtained from load test. This might be due to the fact that this method only considers the SPT N-value.

4.2.3 Site-3-Tower-56 (Thakurgaon)

In this site, one pile is tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.3.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in Figure 4.31.

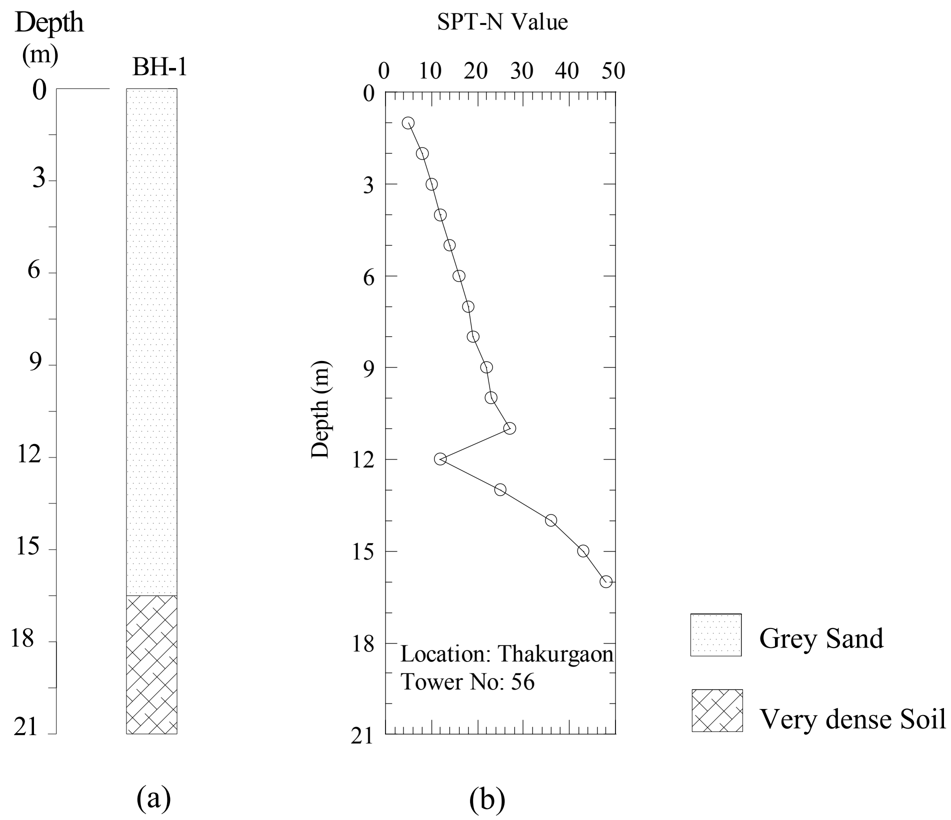


Figure 4.31: Sub-soil profile of Thakurgaon site (Site-3): (a) sub-soil profile and (b) variation of SPT-N value with depth

The general soil profile of this area is grey fine sand. A very dense soil layer is found below the 16.5 m from EGL. The uncorrected SPT-N value of the top layer varies from 5 to 48.

4.2.3.2 Load Test Results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are 13 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the Method-2 (using reaction pile) as described in Chapter 3 (Art. 3.6.2).

Uplift Capacity from Load Test

Uplift test was conducted at the site using the method-2 (reaction pile) as described in Chapter 3 (Art. 3.6.2). Figure 4.32, Figure 4.33 and Figure 4.34 show the load versus elapsed time, pile movement versus elapsed time and load versus pile movement curve for pile P4, respectively. Results obtained from the load tests are presented in the Table 4.8. From the table it is found that the pile capacity from % dia method and double tangent method is 26 ton and 35 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacity of the tested pile is estimated using the five methods mentions in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.8.

From the results presented in Table 4.8, it is seen that the results obtained from the load test and estimation are different. The result from DIN method is close to the capacity obtained from load test. The capacity obtained from the Murty’s method is lower than the capacity obtained from load test. However, the capacity obtained from the method British/American is significantly higher than all other methods.

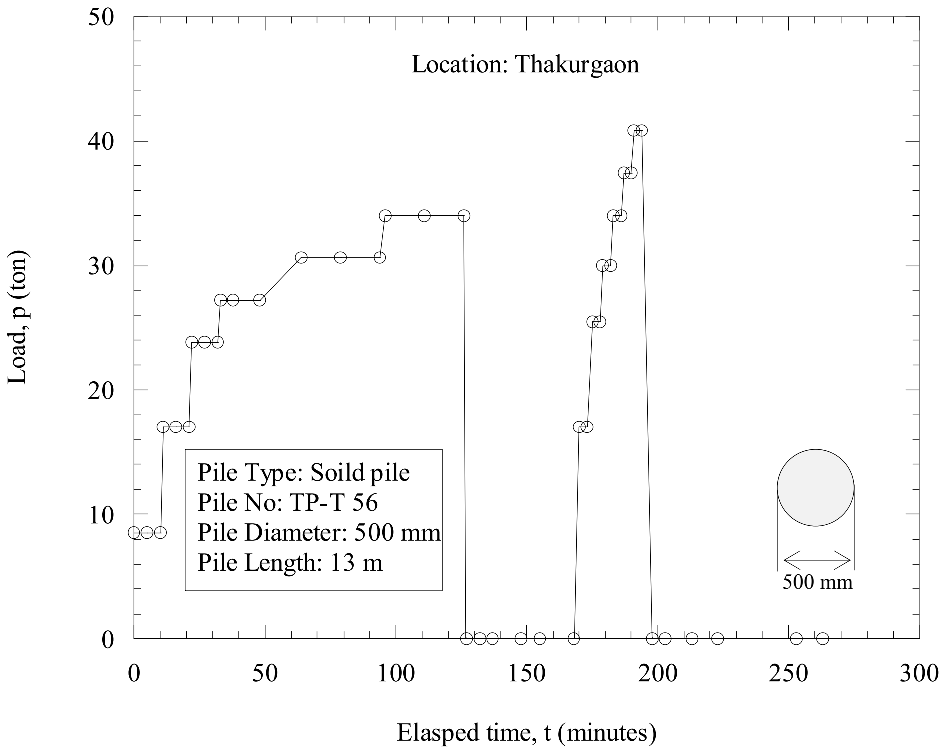


Figure 4.32: Elapsed time vs. load curve of the test pile TP-T 56 at Site-3

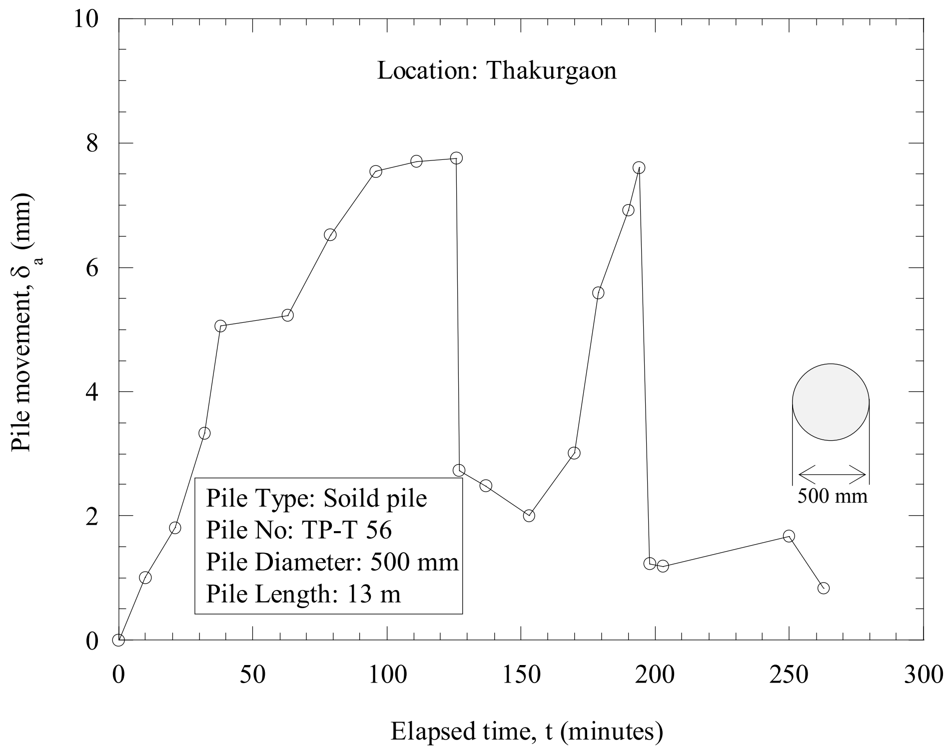


Figure 4.33: Pile movement vs. elapsed time curve of pile TP-T 56 at Site-3

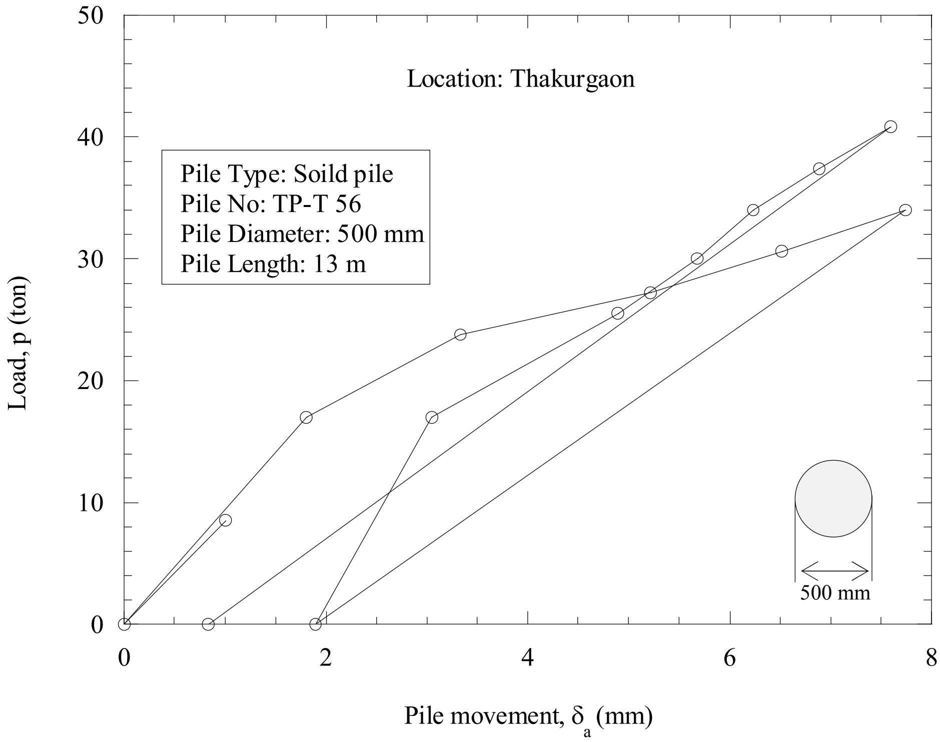


Figure 4.34: Load vs. pile movement curve of the pile TP-T 56 at Site-3

The values of SPT N-values are high. The capacity obtained from British/American methods is significantly high as the correction on N value is not applied for the most of the layers as N >15 (Chapter 3, Art.3.8)

Table 4.8: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-3 Tower- 56	P04	13	500	Solid	30	29	40	20	39	27	95

4.2.4 Site-4: Tower-77

In this site, one pile was tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.4.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.35.The general soil profile of this area is grey medium dense to dense fine sand. The top 1.5 m is brown silt. A grey loose fine sand layer exists from 1.5 to 15 m depth from EGL. A very dense layer is found below the 15 m from EGL. The uncorrected SPT-N value of this sand layer varies from 4 to 46.

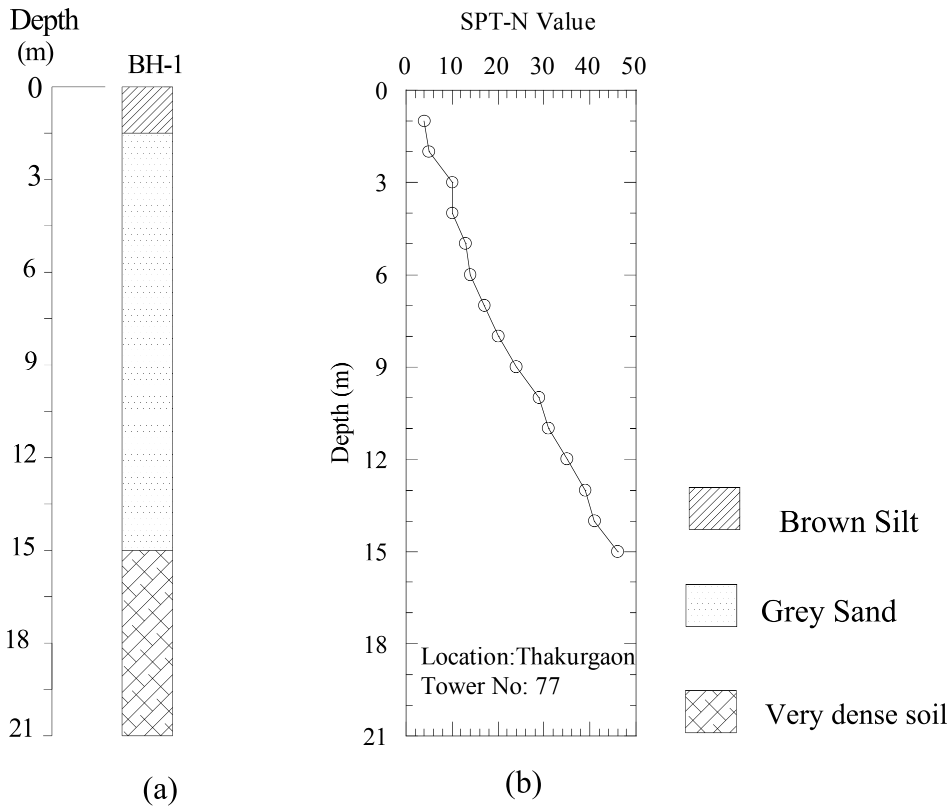


Figure 4.35: Sub-soil profile of Thakurgaon-Panchagar site (site-4): (a) sub-soil profile and (b) variation of SPT-N value with depth

4.2.4.2 Load Test results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are 9 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method-2 (reaction pile) as described in Chapter 3 (Art. 3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2). Figure 4.36 shows the load versus pile movement curve for pile P5. Results obtained from the load tests are presented in the Table 4.15. From the table it is found that the pile capacity from % dia method and Double tangent method is 22 ton and 19 ton, respectively.

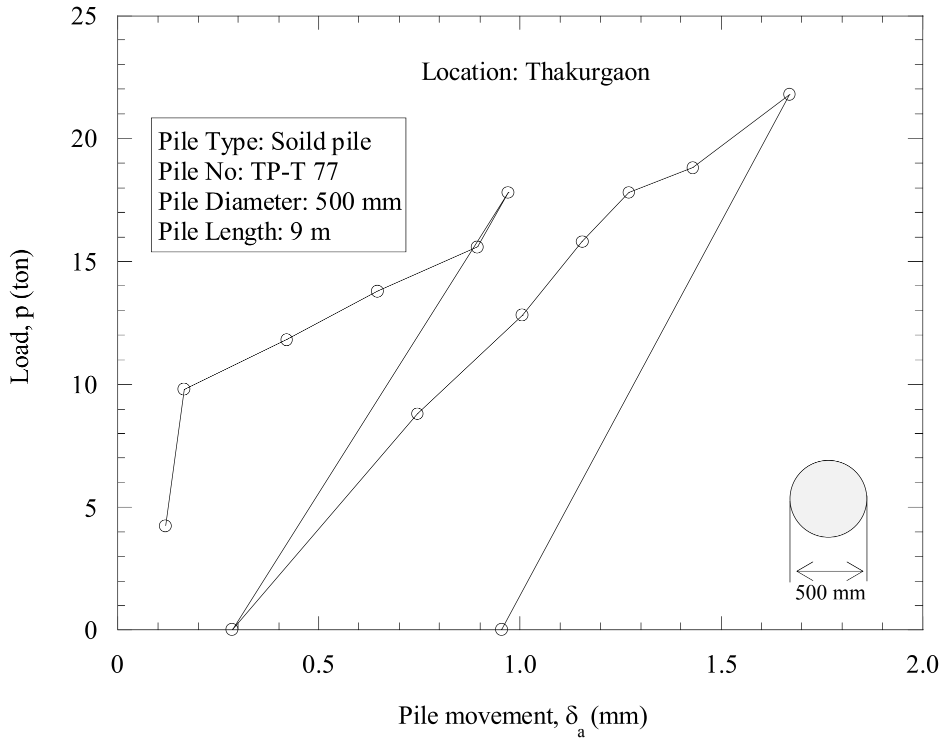


Figure 4.36: Load vs. pile movement curve of the pile TP-T 77 at Site-4

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.9. From the results presented in Table 4.9, it is seen that the results obtained from the load test and estimation are convergent. The result from Murthy is very small as the major length of the pile embedded in sandy layer. Again British/American method gives high value related to high value for N.

Table 4.9: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-4-P1	P05	13	500	Solid	22	19	25	11	20	21	45

4.2.5 Site-5-Tower-147 (Jhinaidah)

In this site, one pile was tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.5.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.37.

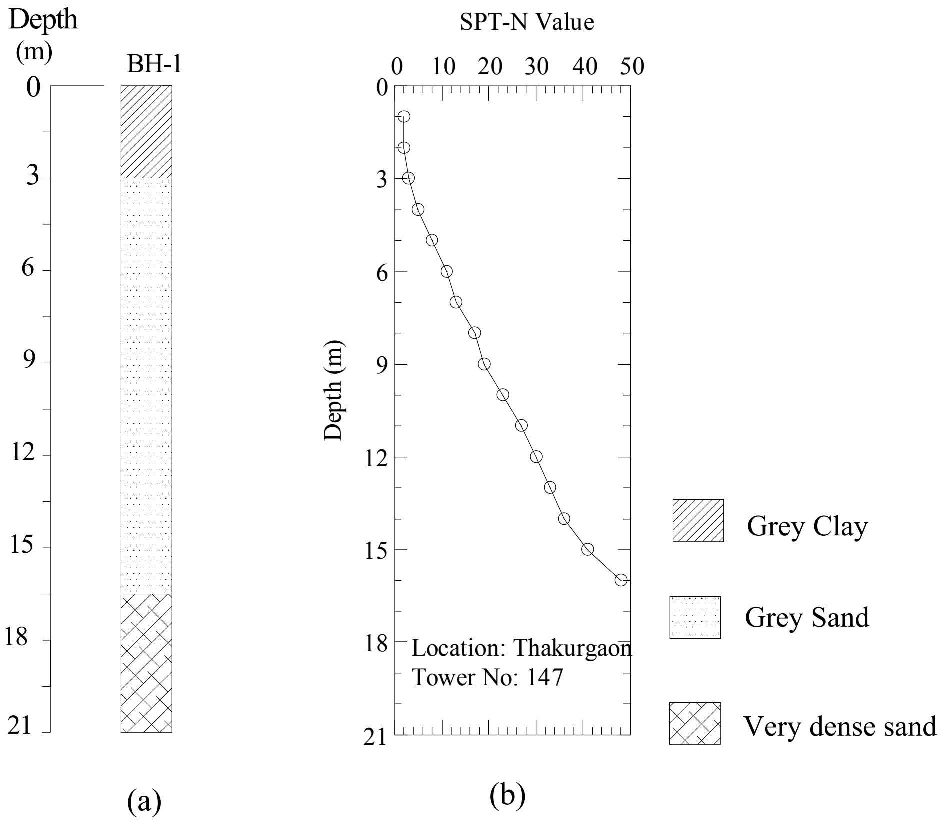


Figure 4.37: Sub-soil profile of Jhinaidah site (Site-5): (a) sub-soil profile and (b) variation of SPT-N value with depth

The general soil profile of this area is grey clay and fine sand. The top 3 m is gray soft clay. A gray fine sand layer exists from 3 to 16.5 m depth from EGL. A very dense sand layer is found below the 16.5 m from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 48.

4.2.5.2 Load Test Results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are 9.5 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2). The Figure 4.38 shows the load versus pile movement curve for pile P6. Results obtained from the load tests are presented in the Table 4.10. From the table it is found that the pile capacity from % dia method and Double tangent method is 17 ton and 18 ton, respectively.

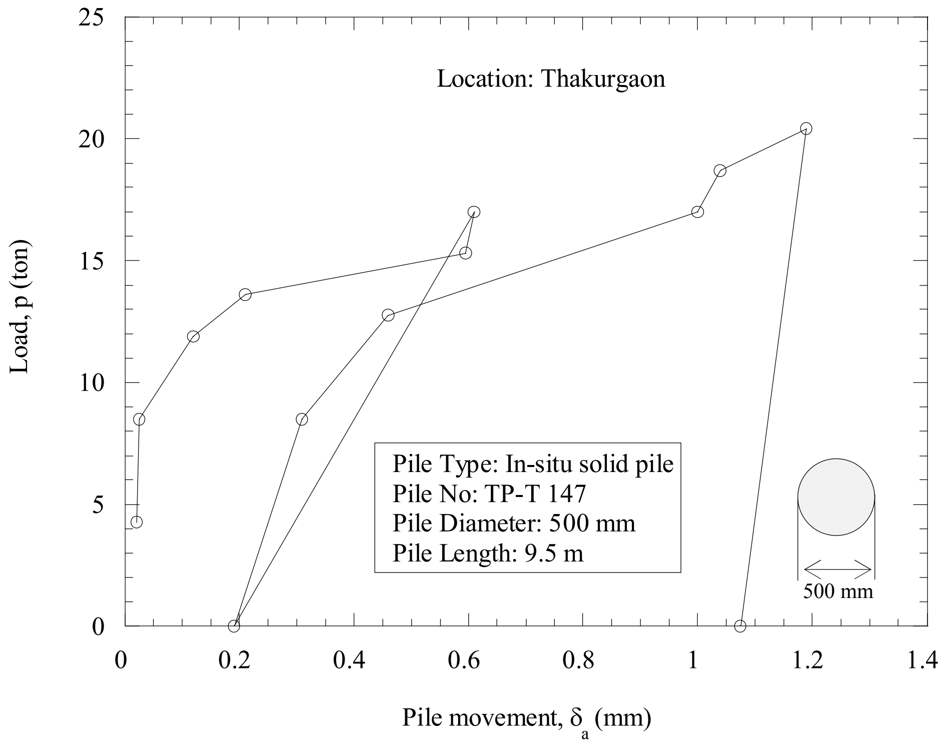


Figure 4.38: Load vs. pile movement curve of the pile TP-T 77 at Site-4

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.10.

Table 4.10: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-5 Tower-147	P06	9.5	500	Solid	17	18	22	12	21	18	41

From the results presented in Table 4.10, it is seen that the results obtained from the load test and estimation are slightly different except the capacity determined from the Murthy's method and British/American method this may because the top clay layer with small N value. The result from British/American method is very high as all length of the pile is embedded in a sandy layer.

4.2.6 Site-6-Tower-25 (Jhinaidah)

In this site, one pile was tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.6.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.39.

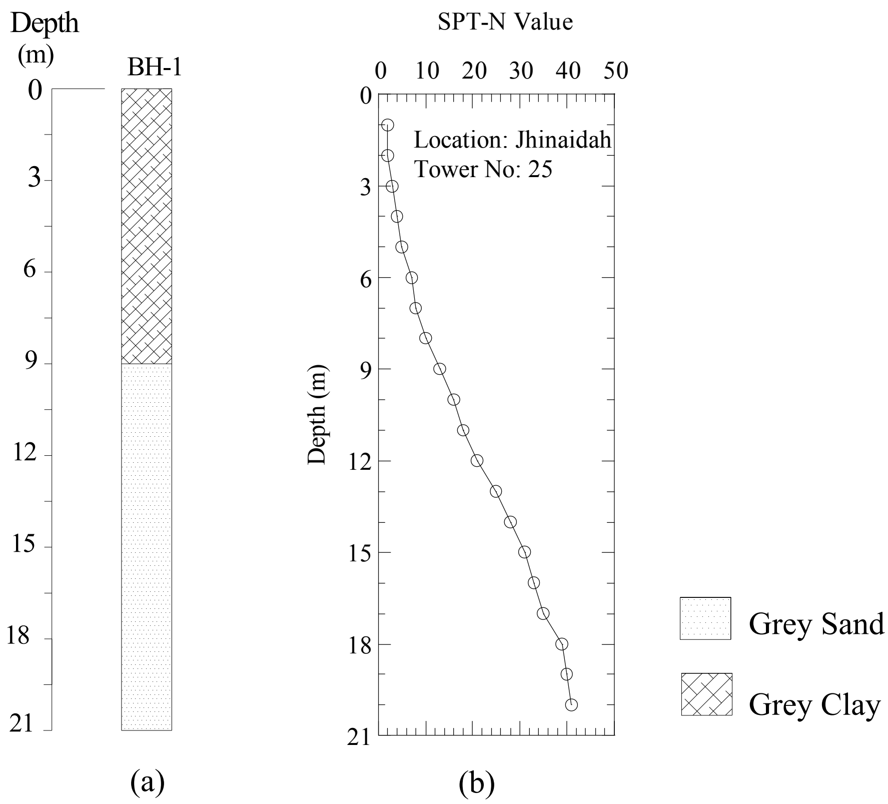


Figure 4.39: Sub-soil profile of Jhinaidah site (site-6): (a) sub-soil profile and (b) variation of SPT-N value with depth

The general soil profile of this area is grey clay and sand. The top 9 m is gray soft clay. A grey sand layer exists from 9 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 41.

4.2.6.2 Load Test Results

Pile Description

One pile was tested under uplift load at this site. The length and diameter of the piles are 15 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2). Figure 4.40, Figure 4.41 and Figure 4.42 show the load versus elapsed time, pile movement versus elapsed time and load versus pile movement curve for pile P7, respectively. Results obtained from the load tests are presented in the Table 4.11. From the table it is found that the pile capacity from % dia method and Double tangent method is 41 ton and 32 ton, respectively.

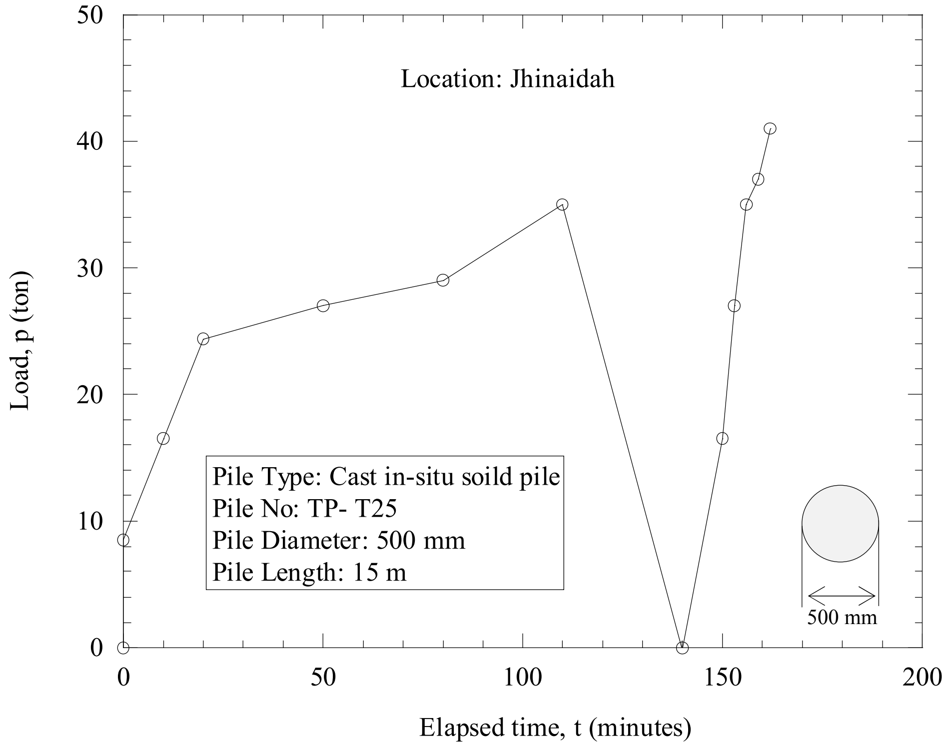


Figure 4.40: Elapsed time vs. load curve of the test pile TP-T 25 at Site-6

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.11.

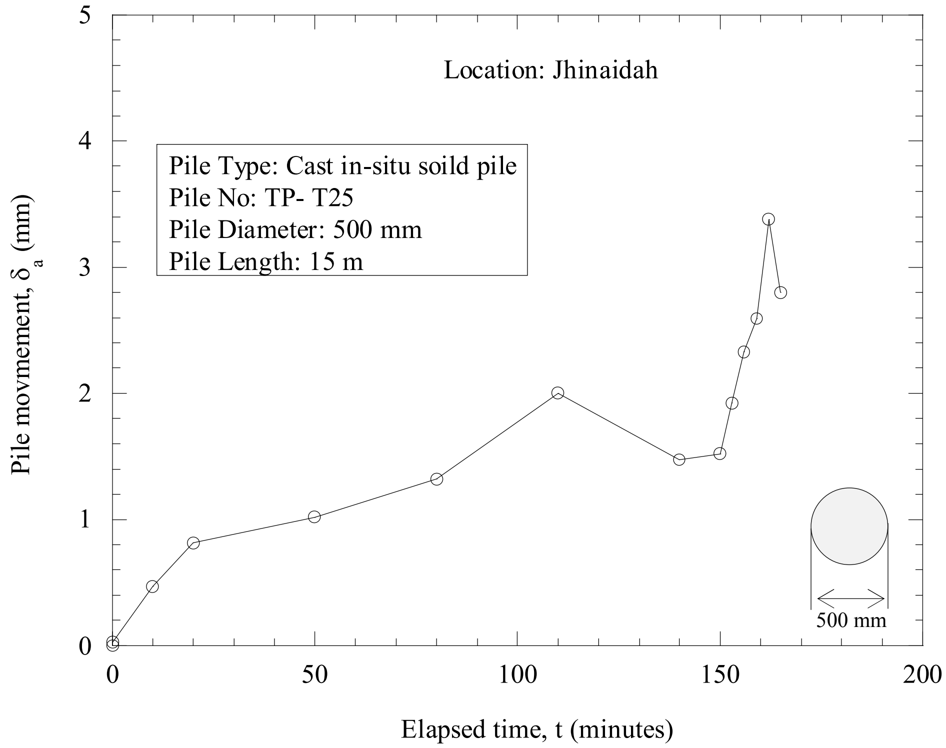


Figure 4.41: Pile movement vs. elapsed time curve of pile TP-T 25 at Site-6

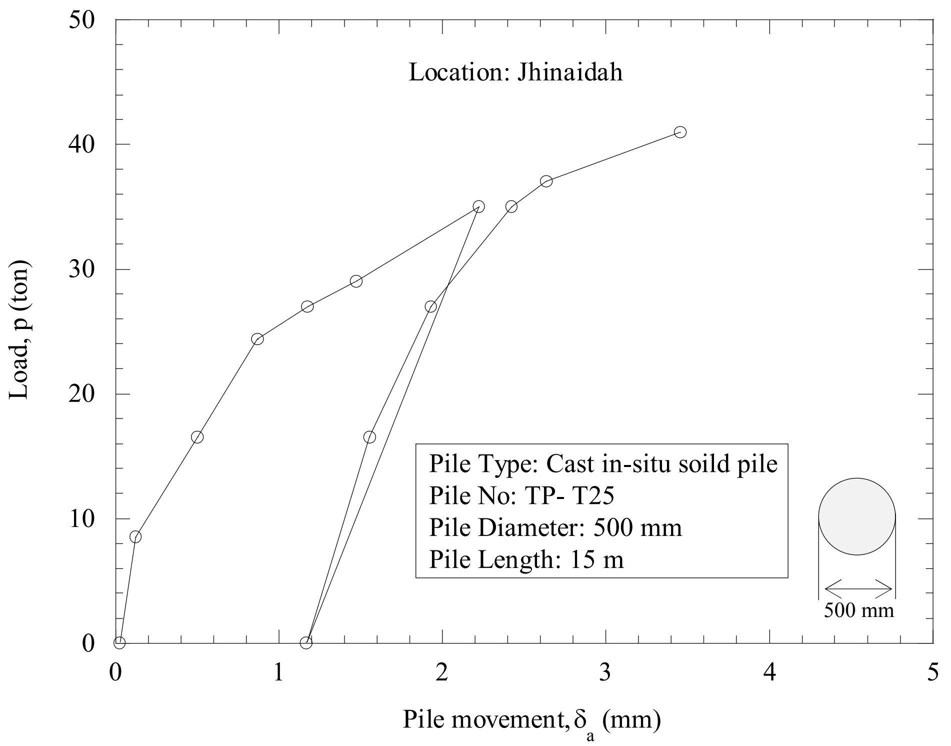


Figure 4.42: Load vs. pile movement curve of the pile TP-T 25 at Site-6

Table 4.11: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-6 Tower-25	P07	15	500	Solid	33	38	65	60	63	28	90

From the results presented in Table 4.11, it is seen that the results obtained from the load test and estimation are significantly different. All results from estimation are high except the pile capacity estimated from DIN 4014 as the top layer for the soil is clay with lower SPT- N values.

4.2.7 Site-7-Tower-28 (Jhinaidah)

In this site, one pile is tested in uplift. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.7.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.43.

The general soil profile of this area is grey clay sand. The top 9 m is gray soft clay layer. A gray sand layer exists from 9 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 36.

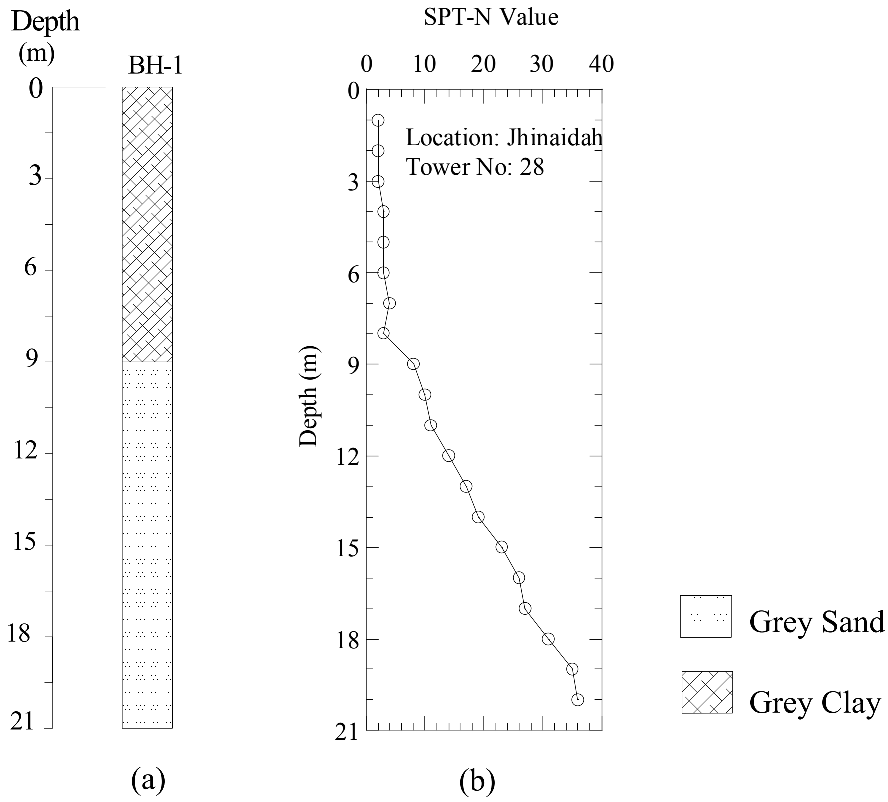


Figure 4.43: Sub-soil profile of Jhinaidah site (Site-7): (a) sub-soil profile and (b) variation of SPT-N value with depth.

4.2.7.2 Load Test Results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are 15 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.44, the Figure 4.45 and the Figure 4.46 shows the load versus elapsed time, pile movement versus elapsed time and load versus pile movement curve for pile P8, respectively. Results obtained from

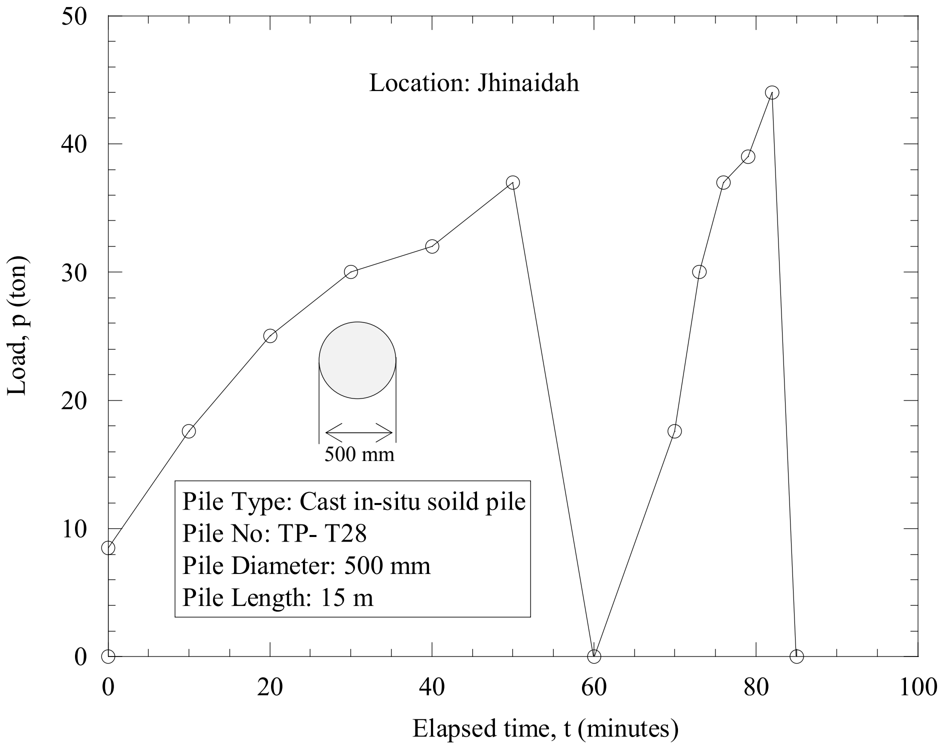


Figure 4.44: Elapsed time vs. load curve of the test pile TP-T 28 at Site-7

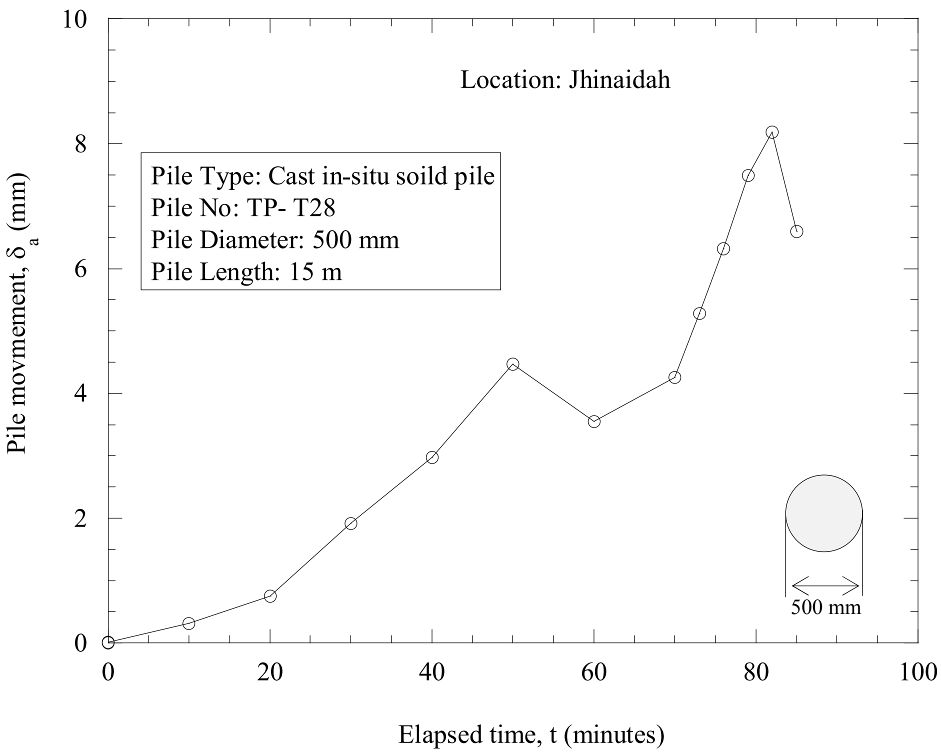


Figure 4.45: Pile movement vs. elapsed time curve of pile TP-T 28 at Site-7

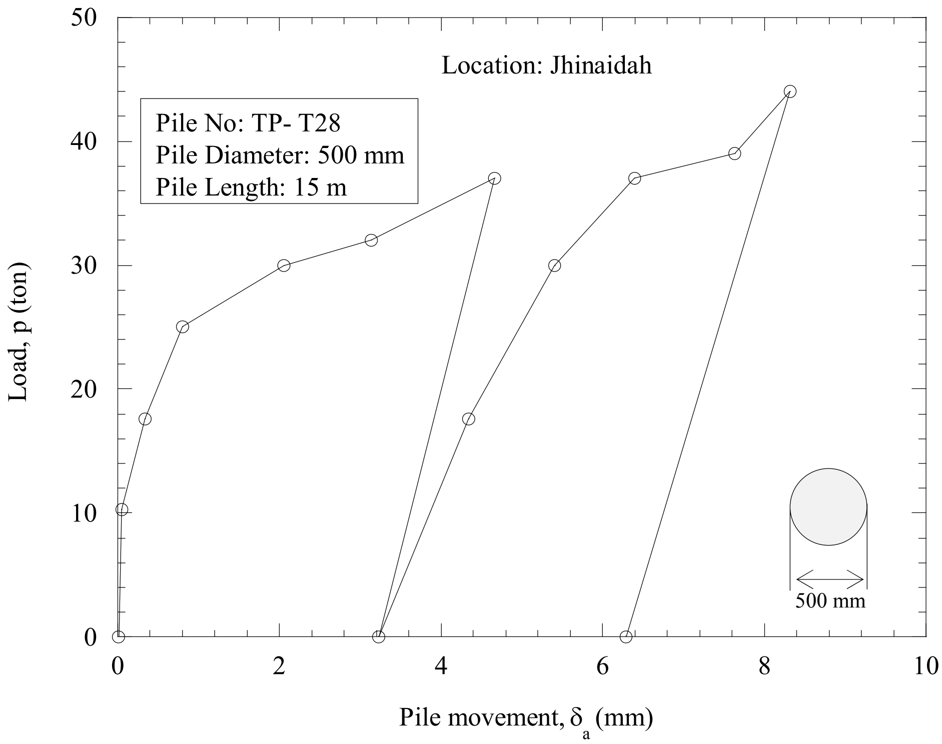


Figure 4.46: Load vs. pile movement curve of the pile TP-T 28 at Site-7

the load tests are presented in the Table 4.12. From the table it is found that the pile capacity from % dia method and Double tangent method is 25 ton and 34 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.12.

***Table 4.12*:** Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-7 Tower-28	P08	15	500	Solid	31	35	38	33	50	23	56

From the results presented in Table 4.12, it is seen that the results obtained from the load test and estimation are significantly different for the methods DIN and British/American method. Results from estimation are high except the value from DIN 4014 as the top layer for the depth around pile is clay soil. However, the pile capacity determined from Meyerhof and Murty's method is close to that obtained from load test.

4.2.8 Site-8-Tower-78 (Jhinaidah)

In this site, one pile was tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.8.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.47. The top 18 m layer is grey clay. The gray fine sand layer exists from 18 to drilled depth from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 38.

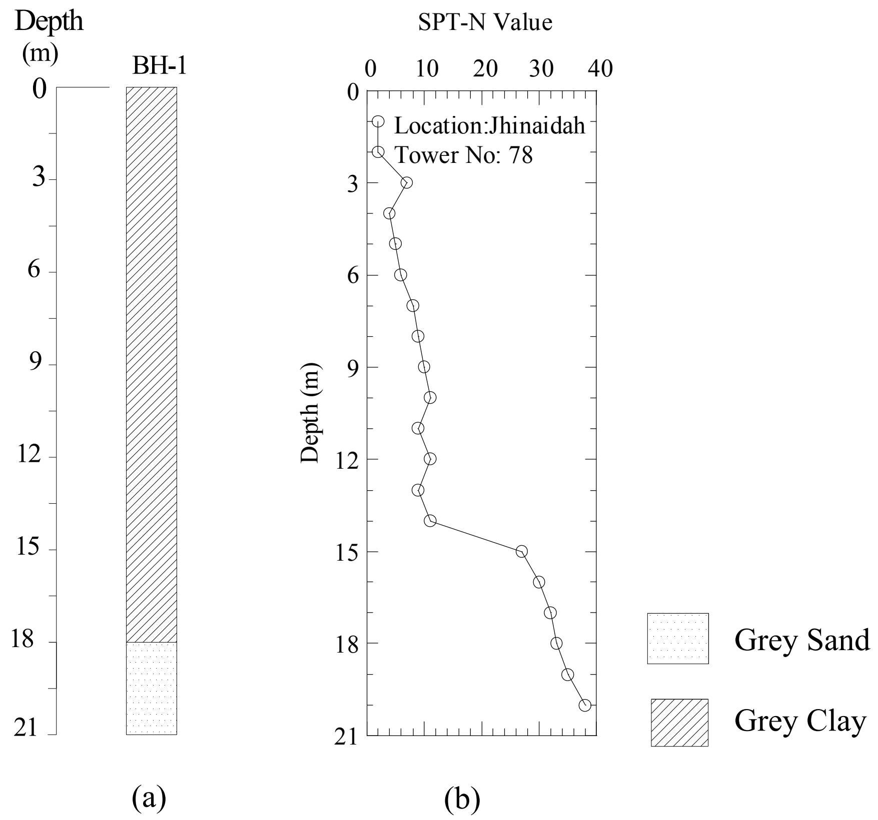


Figure 4.47: Sub-soil profile of Jhinaidah site (Site-8): (a) sub-soil profile and (b) variation of SPT-N value with depth.

4.2.8.2 Load Test Results

Pile Description

One pile was tested under uplift load at this site. The length and diameter of the piles are 11 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. 3.6.2). The Figure 4.48 shows the load versus pile movement curve for pile P9. Results obtained from the load tests are presented in the Table 4.13. From the table it is found that the pile capacity from % dia method and Double tangent method is 22 ton and 12 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.13.

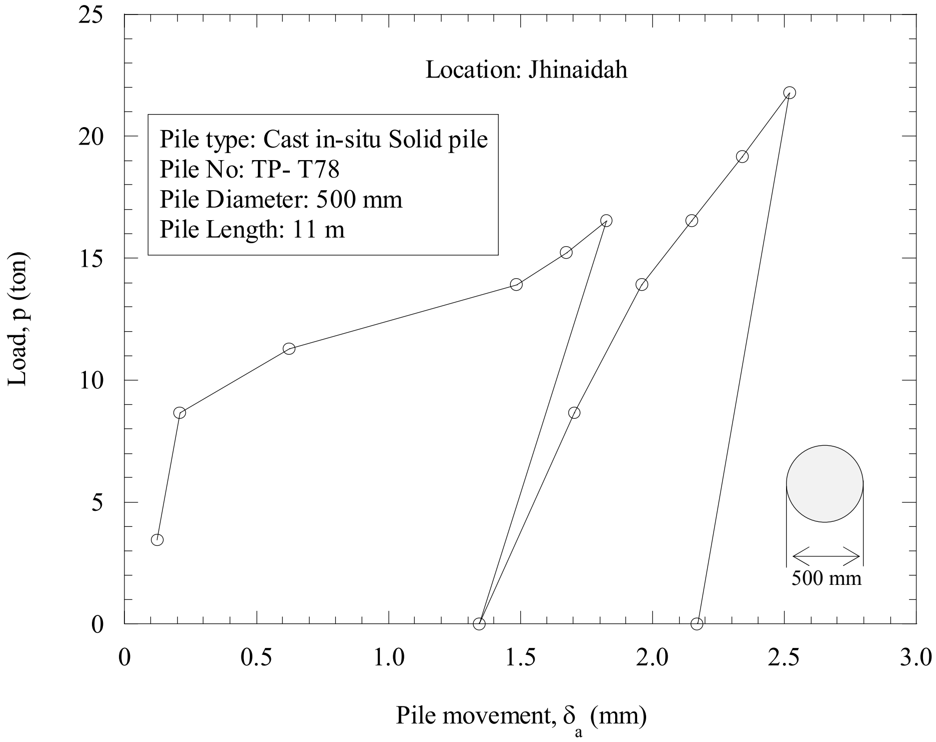


Figure 4.48: Load vs. pile movement curve of the pile TP-T 78 at Site-8

From the results presented in Table 4.13, it is seen that the results obtained from the load test and estimated from British/American method is close. However, the pile capacity determined from Meyerhof, Murthy and Tomlinson are very higher than that obtained from load test. The capacity obtained from DIN method is lower than that obtained from load test thus the all pile impeded in clay layer with small N values.

Table 4.13: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-8 Tower-78	P09	11	500	Solid	18	18	63	63	44	12	17

4.2.9 Site-9-Tower-17 (Magura)

In this site, one pile was tested in uplift. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.9.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.49. The general soil profile of this area is grey clay and fine sand. The top 4.5 m is gray soft clay. The gray fine sand layer exists from 4.5 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 36.

4.2.9.2 Load Test Results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are

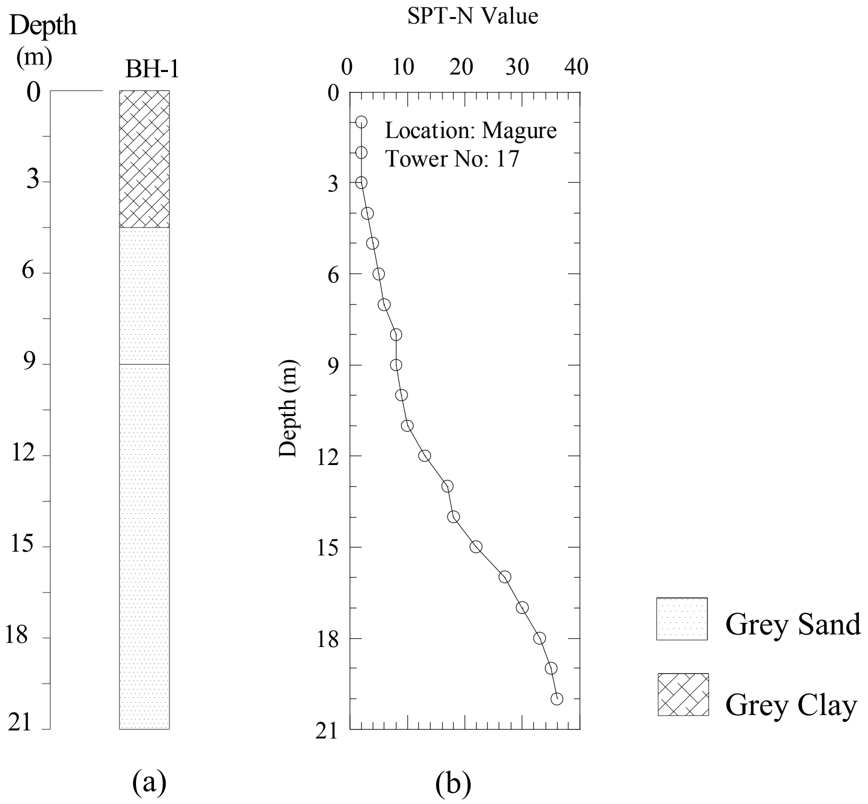


Figure 4.49: Sub-soil profile of Magura site (Site-9): (a) sub-soil profile and (b) variation of SPT-N value with depth

14 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.50 shows the load versus pile movement curve for pile P10. Results obtained from the load tests are presented in the Table 4.14. From the table it is found that the pile capacity from % dia method and Double tangent method is 34 ton and 37 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.14.

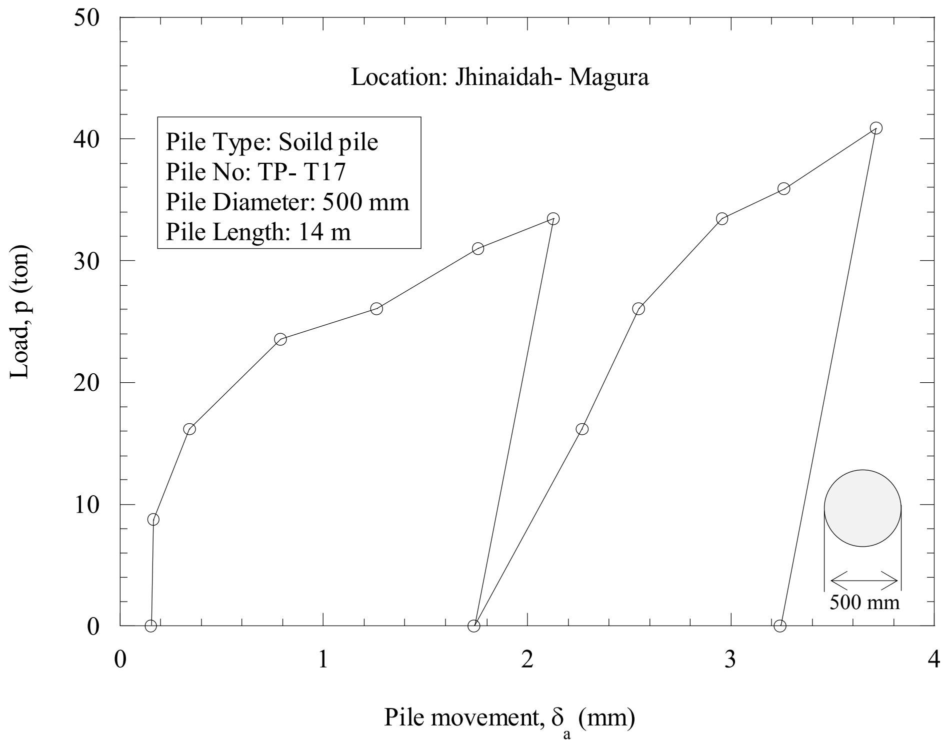


Figure 4.50: Load vs. pile movement curve of the pile TP-T 17 at Site-9

From the results presented in Table 4.14, it is seen that results from estimation are high except the value from DIN 4014 and Murthy as the overall layer for the depth around pile is clay soil with small SPT N values also the values from pile test is significantly small which is not make sense.

Table 4.14: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-9 Tower-17	P10	14	500	Solid	32	37	25	17	41	19	38

4.2.10 Site-10-Tower-53 (Jhinaidah)

In this site, one pile is tested in uplift. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.10.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.51.

The general soil profile of this area is grey clay and fine sand. The top 6 m is gray soft clay. The gray fine sand layer exists from 6 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 2 to 46.

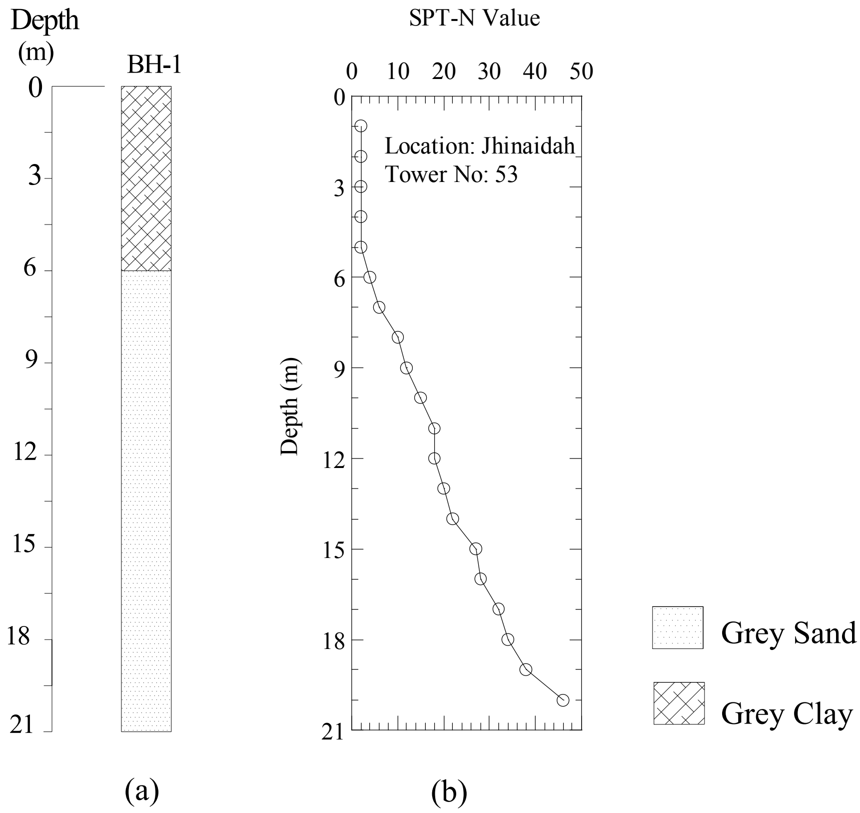


Figure 4.51: Sub-soil profile of Jhinaidah site (site-10): (a) sub-soil profile and (b) variation of SPT-N value with depth

4.2.10.2 Load Test Results

Pile Description

One pile was tested under uplift load at this site. The length and diameter of the piles are 14.5 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.52 shows the load versus pile movement curve for pile P11. Results obtained from the load tests are presented in the Table

4.15. From the table it is found that the pile capacity from % dia method and Double tangent method is 40 ton and 39 ton, respectively.

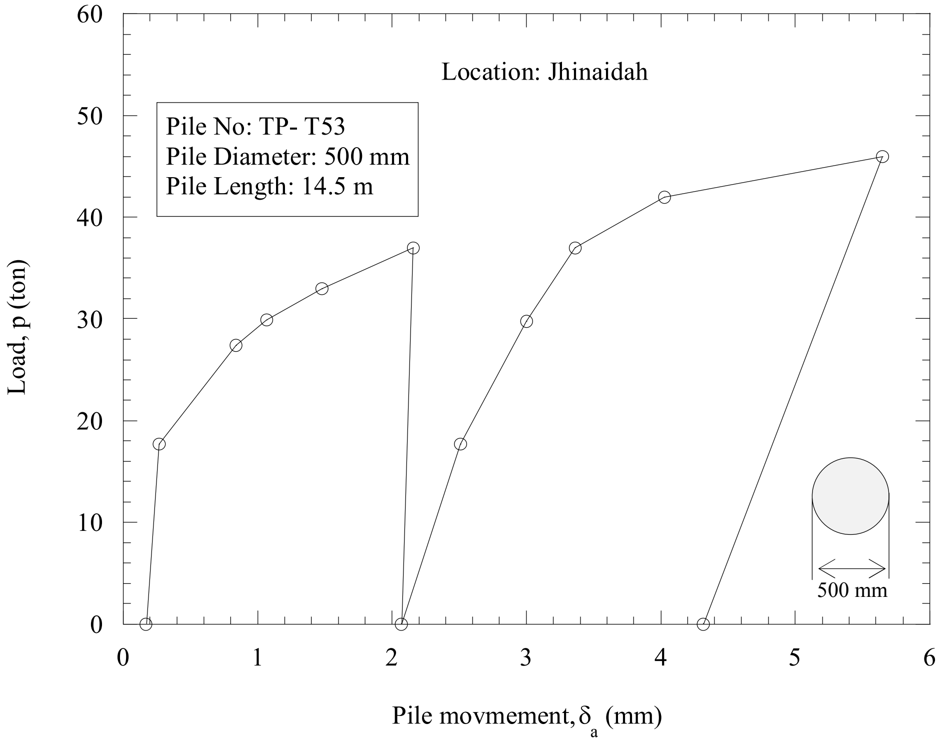


Figure 4.52: Load vs. pile movement curve of the pile TP-T 53 at Site-10

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.15.

From the results presented in Table 4.15, it is seen that the results obtained from the load test and estimation are slightly different, the result based on the SPT-N value only is high because it only depends on SPT-N value the result from British/American Method is significantly high case the pile length is high and N value.is high in the lower sandy layer.

Table 4.15: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-10 Tower-53	P11	14.5	500	Solid	40	39	32	25	48	25	88

4.2.11 Site-11-Tower-112 (Naogaon)

In this site, one pile was tested under uplift load. One borehole was drilled for determining the sub-soil characteristics. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

4.2.11.1 Sub-soil Characteristics

SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.53.

The general soil profile of this area is brown clay and grey fine sand. The brown soft clay layer exists top 3 m depth from EGL. The gray fine sand layer exists from 3 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 1 to 31.

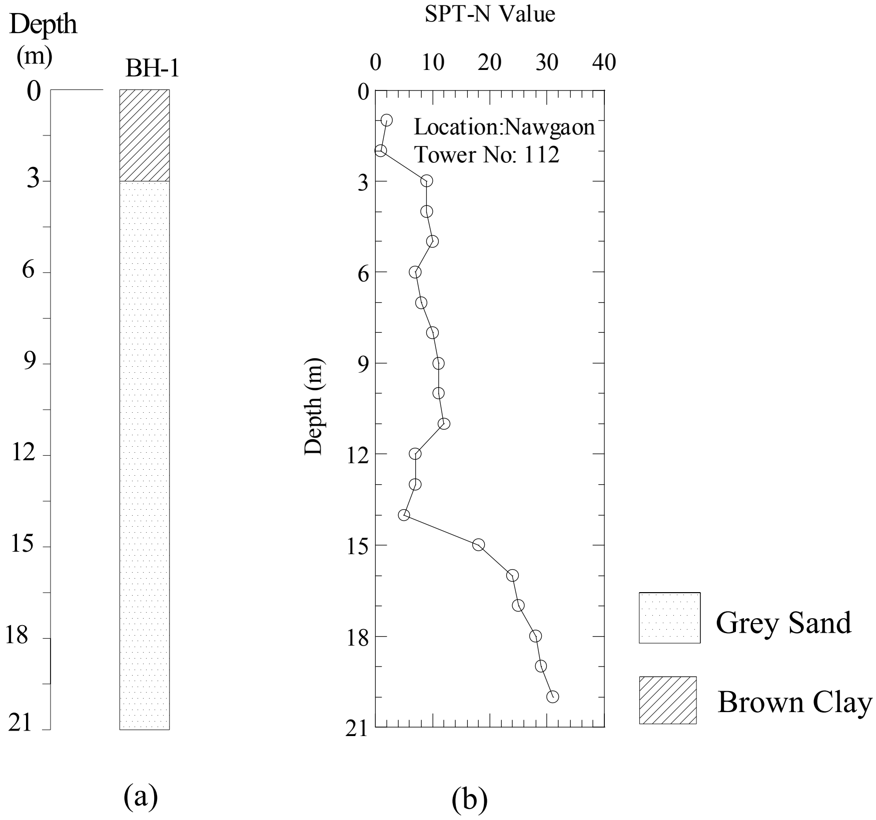


Figure 4.53: Sub-soil profile of Naogaon site (site-11): (a) sub-soil profile and (b) variation of SPT-N value with depth

4.2.11.2 Load Test Results

Pile Description

One pile was tested under uplift load at this site. The length and diameter of the piles are 15 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.54 shows the load versus pile movement curve for pile P12. Results obtained from the load tests are presented in the Table

4.16. From the table it is found that the pile capacity from % dia method and Double tangent method is 24 ton and 25 ton, respectively.

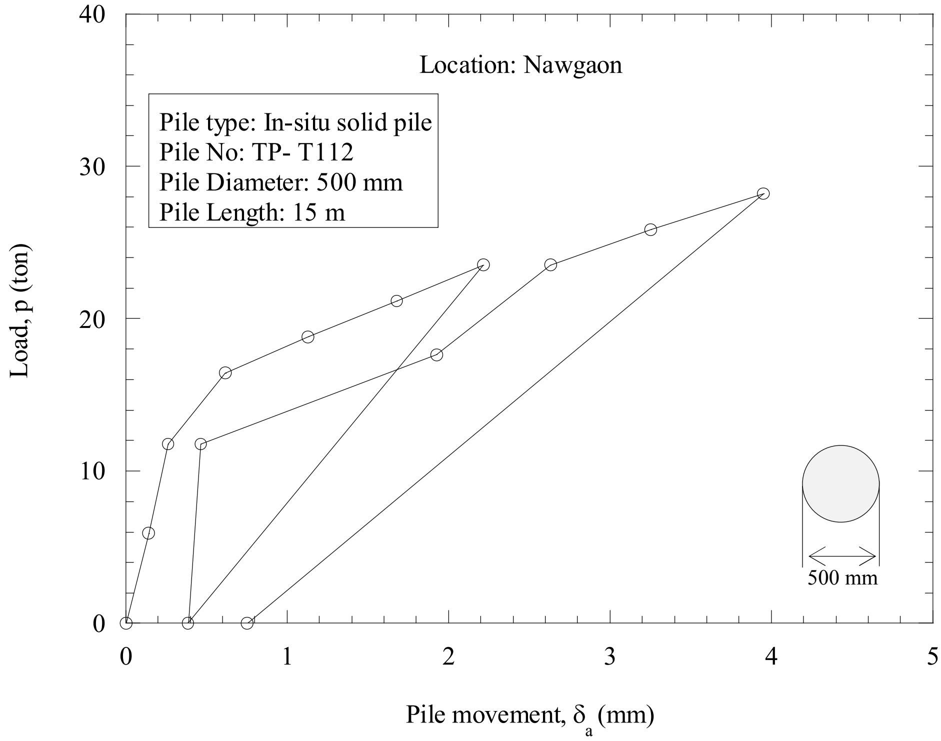


Figure 4.54: Load vs. pile movement curve of the pile TP-T 112 at Site-11

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.16.

From the results presented in Table 4.16, it is seen that the results obtained from the load test significantly small, as the top layer is loose sand with small N values, also the DIN Method.

Table 4.16: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-11 Tower-112	P12	15	500	Solid	24	25	45	39	61	19	44

4.2.12 Site-12-Tower-51 (Amin bazar)

4.2.12.1 Load Test Results

Pile Description

One pile is tested under uplift load at this site. The length and diameter of the piles are 14.25 m and 500 mm, respectively. Constructed bored piles were tested after one month of construction. Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).

Uplift Capacity from Load Test

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2).The Figure 4.55 shows the load versus pile movement curve for pile P13. Results obtained from the load tests are presented in the Table 4.17. From the table it is found that the pile capacity from % dia method and Double tangent method is 20 ton and 23 ton, respectively.

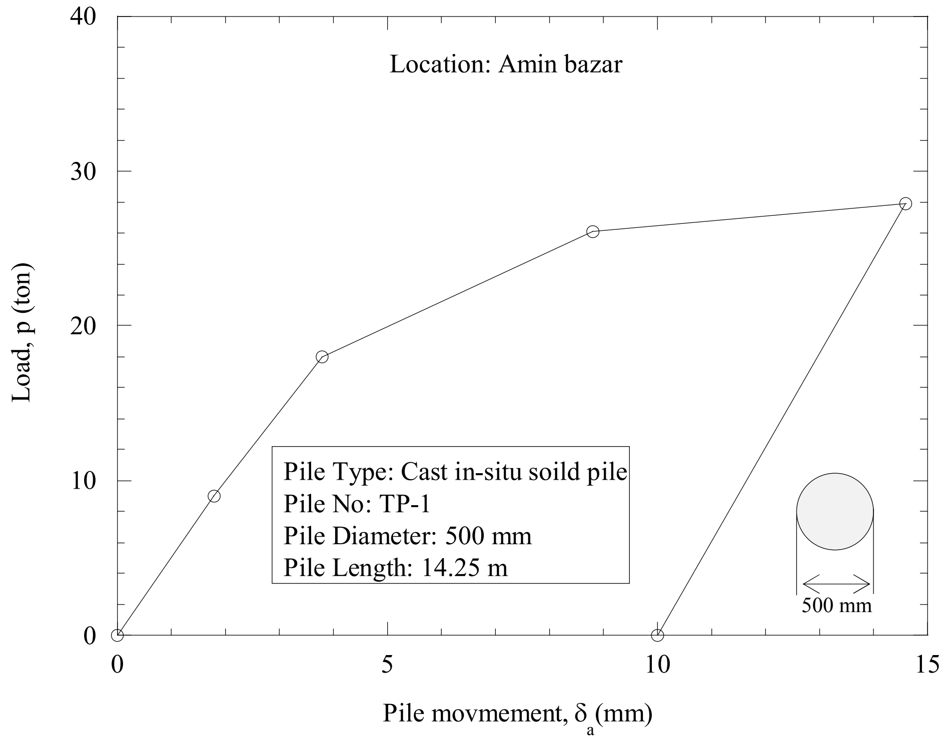


Figure 4.55: Load vs. pile movement curve of the pile TP-T 51 at Site-12

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.17.

Table 4.17: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-12 Tower-51	P13	14. 25	500	Solid	20	23	29	21	42	14	40

From the results presented in Table 4.17, it is seen that the results of methods consider the other soil parameters like friction angle and cohesion provides very close value as obtained in the load tests and those govern by N value is slightly high.

4.2.13 Site-13 (Baraba)

In this site, six piles are tested in uplift and five boreholes were drilled for determining the sub-soil characteristics. The Figure 4.56 shows the site map for test pile locations. SPT was conducted and disturbed and undisturbed samples were collected. The soil profile and the variation of SPT-N values with depth are presented here.

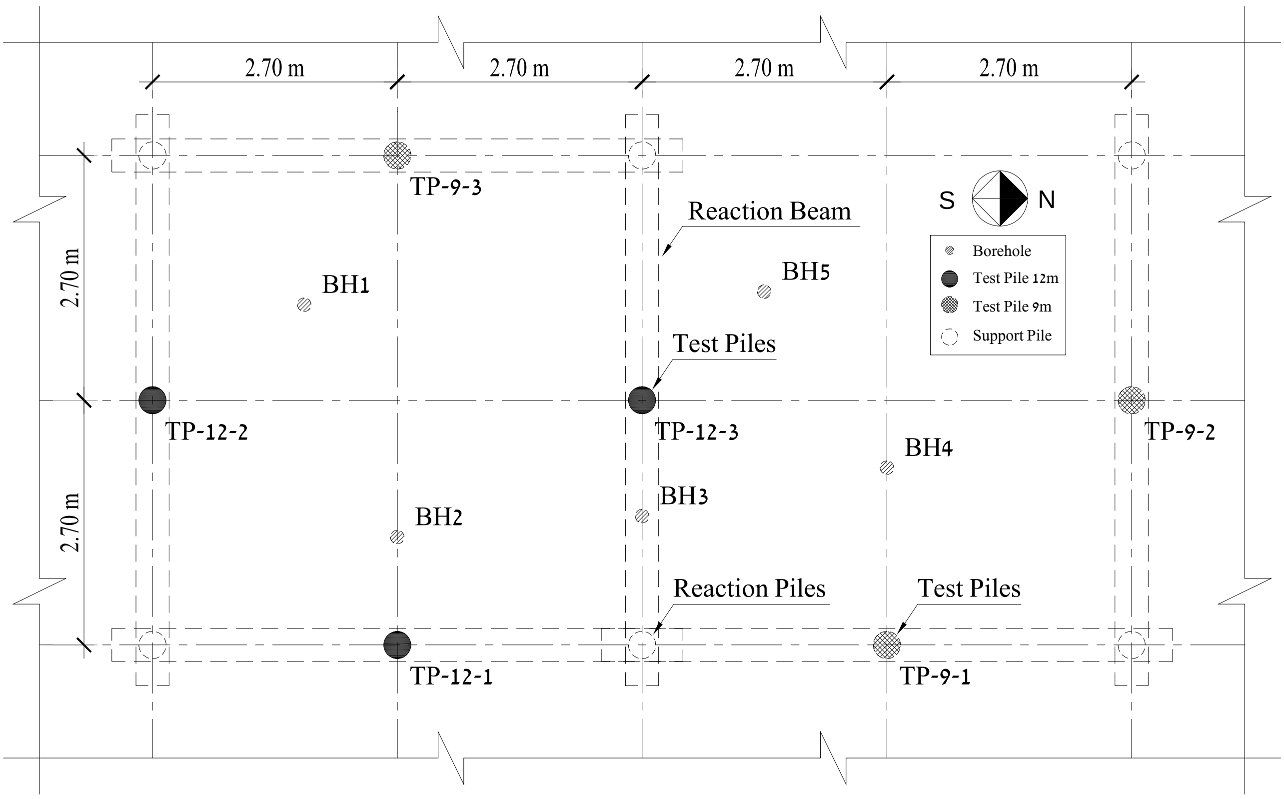


Figure 4.56: Layout showing the test pile and bore hole locations on the site map

4.2.13.1 Sub-soil Characteristics

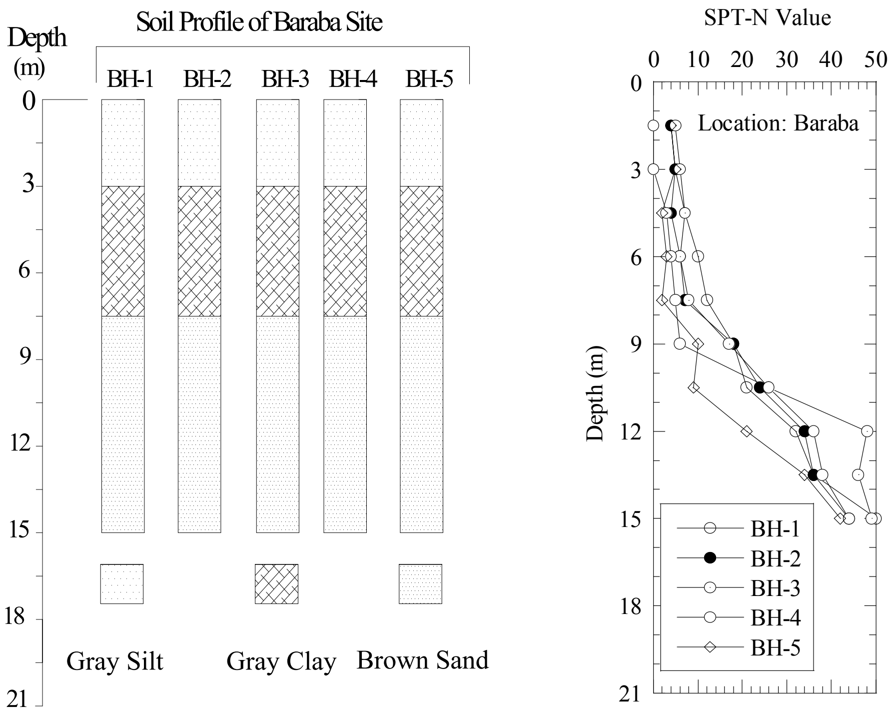
SPT Results

SPT was conducted in the area following procedure described in ASTM D1586. The soil profiles and the variation of SPT-N values with depth are shown in the Figure 4.57. The general soil profile of this area is grey clay, gray silt and fine sand. The gray silt layer exists top 3 m depth from EGL. The gray clay layer exists from 3 to 7.5 m depth from EGL. The sand layer exists from 7.5 to the drilling depth from EGL. The uncorrected SPT-N value of this sand layer varies from 3 to 50.

4.2.13.2 Load Test Results

Pile Description

Six piles are tested under uplift load at this site. Two types of hollow driven piles are tested here. The length and diameter of one type is 9 m, and 300 mm, respectively. The wall thickness of the pile is 65mm. The length and diameter of the other type is 12 m, and 300 mm, respectively. The thickness of this pile is also 65mm. At first, borings of 200-250 mm diameter were made. After that precast piles were driven on those holes. Then piles were tested after one month of installation. Details of the pile are presented in Figure 4.58.



(a) (b)

Figure 4.57: Sub-soil profile of Baraba site (Site-13): (a) sub-soil profile and (b) variation of SPT-N value with depth

Uplift Capacity from Load Test (Site 13-TP-12-1)

Uplift tests were conducted at the site using the method-2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.59 shows the load versus pile movement curve for pile P14. Results obtained from the load tests are presented in the Table 4.18. From the table it is found that the pile capacity from % dia method and Double tangent method is 38 ton and 43 ton, respectively.

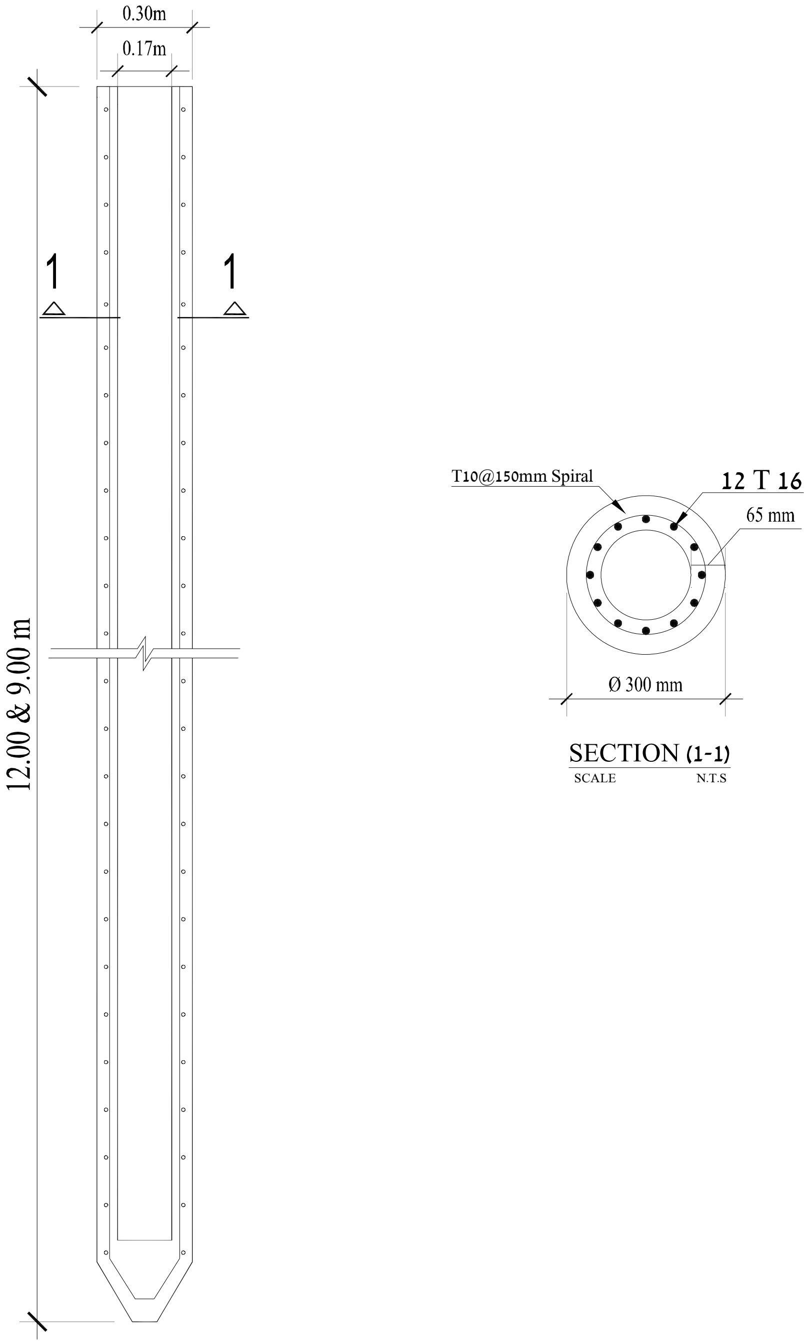


Figure 4.58: Schematic diagrams of piles cross section and length for site S-13, Baraba

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.18.

From the results presented in Table 4.18, it is seen that the results obtained from the load test and estimation are convergent, piles are hollow and driven in boreholes The result based on the SPT-N value only is the lowest case.

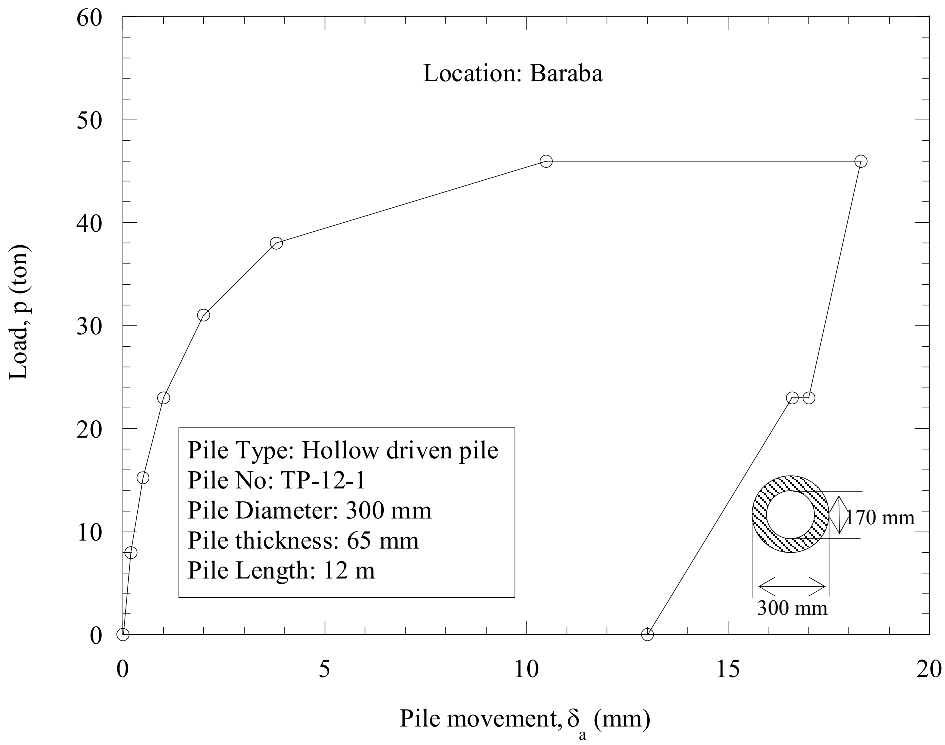


Figure 4.59: Load vs. pile movement curve of the pile TP-12-1 at Site-13.

Table 4.18: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-12-1	P14	12	300	Hollow	38	43	29	25	25	14	38

Uplift Capacity from Load Test (Site 13-TP-12-2)

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.60 shows the load versus pile movement curve for pile P15. Results obtained from the load tests are presented in the Table 4.19. From the table it is found that the pile capacity from % dia method and Double tangent method is 43 ton and 44 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.19.

From the results presented in Table 4.19, it is seen that the results obtained from the load test and estimation are significantly different, the results from load test are convergent and most of the estimated method but the British/American Method is slightly high for the high N value.

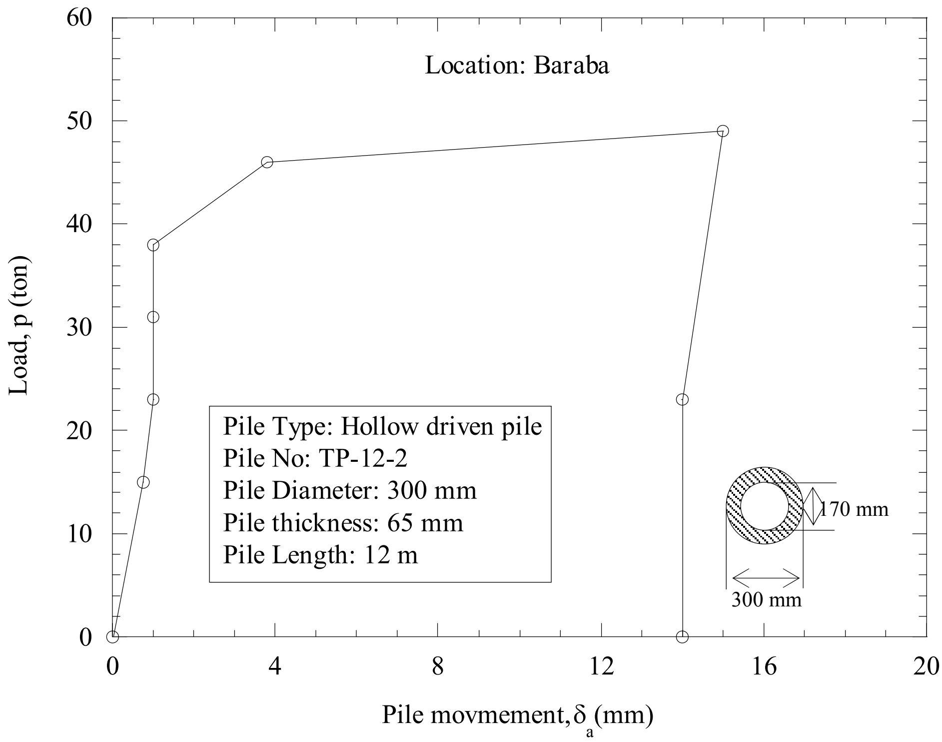


Figure 4.60: Load vs. pile movement curve of the pile TP-12-2 at Site-13

Table 4.19: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-12-2	P15	12	300	Hollow	43	46	24	20	20	15	36

Uplift Capacity from Load Test (Site 13-TP-12-3)

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.61 shows the load versus pile movement curve for pile P16. Results obtained from the load tests are presented in the Table 4.20. From the table it is found that the pile capacity from % dia method and Double tangent method is 31 ton and 38 ton, respectively.

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.20

From the results presented in Table 4.20, it is seen that the results obtained from the load test and estimation are convergent.

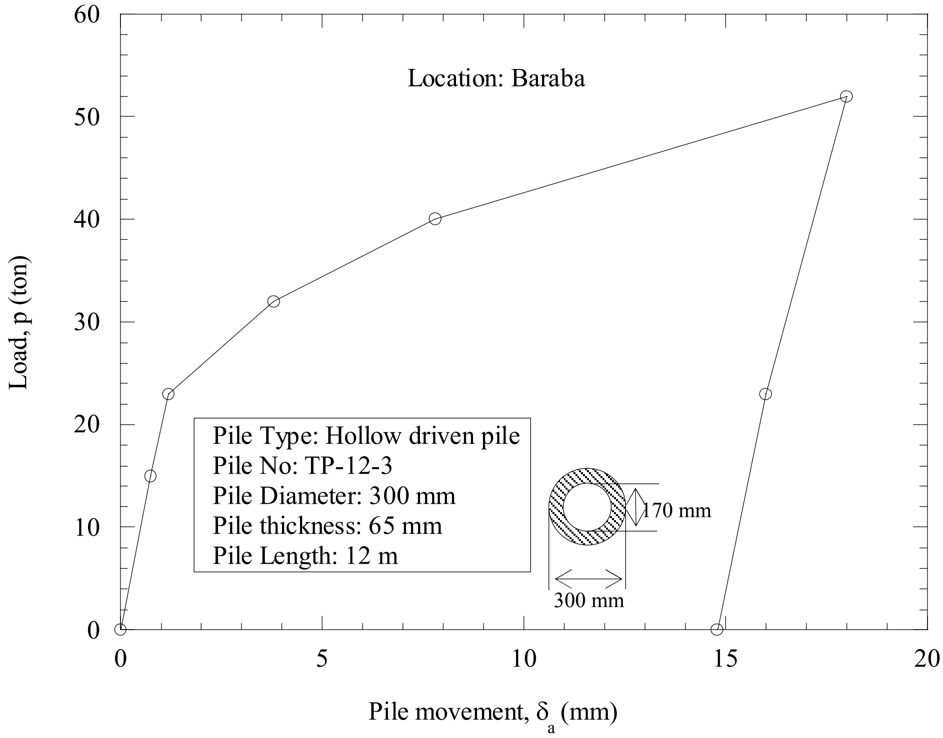


Figure 4.61: Load vs. pile movement curve of the pile TP-12-3 at Site-13

Table 4.20: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-12-3	P16	12	300	Hollow	31	38	29	28	22	21	37

Uplift Capacity from Load Test (Site 13-TP-9-1)

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.62 shows the load versus pile movement curve for pile P17. Results obtained from the load tests are presented in the Table 4.21. From the table it is found that the pile capacity from % dia method and Double tangent method is 33 ton and 38 ton, respectively.

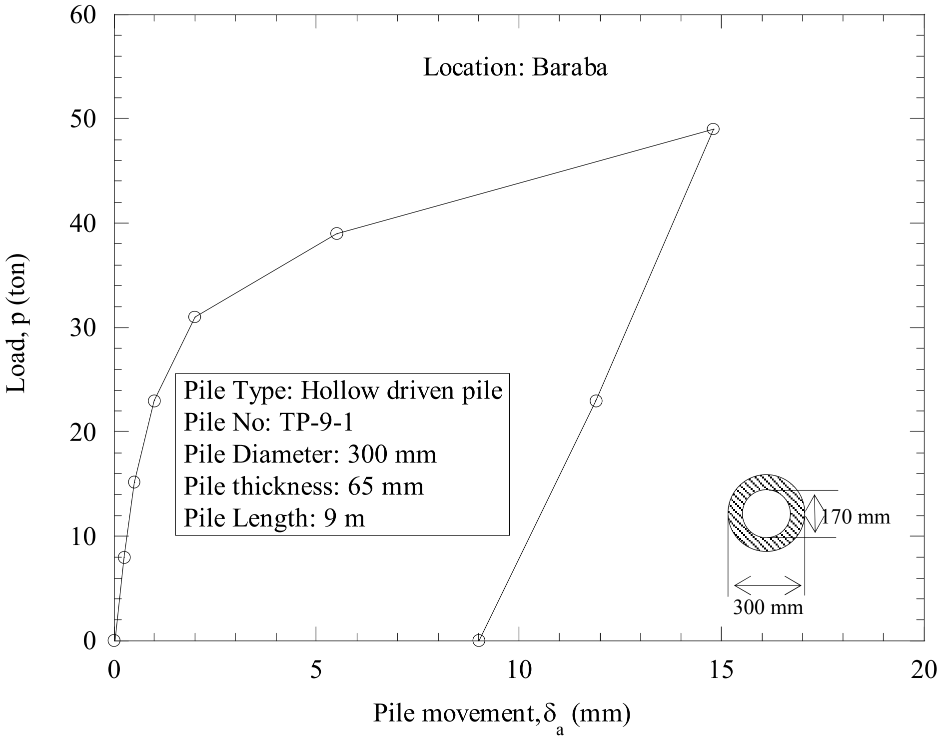


Figure 4.62: Load vs. pile movement curve of the pile TP-9-1 at Site-13

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.21.

Table 4.21: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-9-1	P17	9	300	Hollow	33	38	9	9	10	3	4

From the results presented in Table 4.21, it is seen that the results obtained from the load test and estimation are significantly different, as the length of pile become smaller 9.0 m the skin friction decrease but the pile Expressed more resist value than that estimated from different methods.

Uplift Capacity from load test (Site 13-TP-9-2)

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.63 shows the load versus pile movement curve for pile P18. Results obtained from the load tests are presented in the Table 4.22. From the table it is found that the pile capacity from % dia method and Double tangent method is 39 ton and 46 ton, respectively.

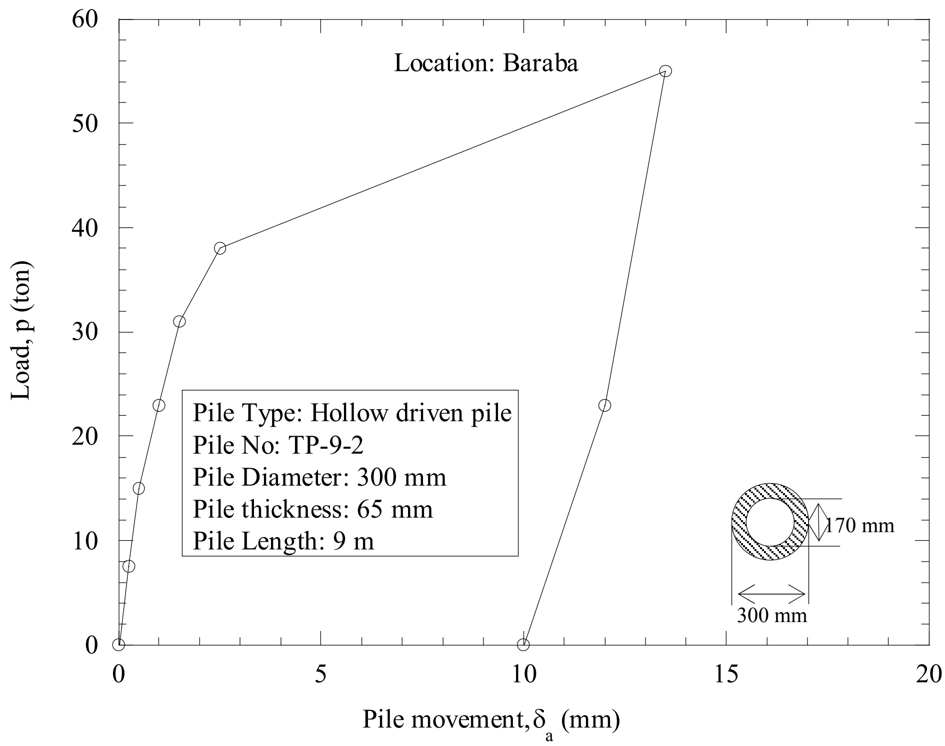


Figure 4.63: Load vs. pile movement curve of the pile TP-9-2 at Site-13

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in Chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.22.

Table 4.22: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity From Load test (ton)		Capacity Estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-9-2	P18	9	300	Hollow	39	46	9	6	8	5	5

From the results presented in Table 4.22, it is seen that the results obtained from the load test and estimation are significantly different pile gives more resist values from load test SPT-N values is small witch give a reason of low capacity from estimation methods.

Uplift Capacity from Load Test (Site 13-TP-9-3)

Uplift tests were conducted at the site using the method -2 (Reaction Pile) as described in Chapter 3 (Art. No.3.6.2). The Figure 4.64 shows the load versus pile movement curve for pile P19. Results obtained from the load tests are presented in the Table 4.23. From the table it is found that the pile capacity from % dia method and Double tangent method is 31 ton and 30 ton, respectively.

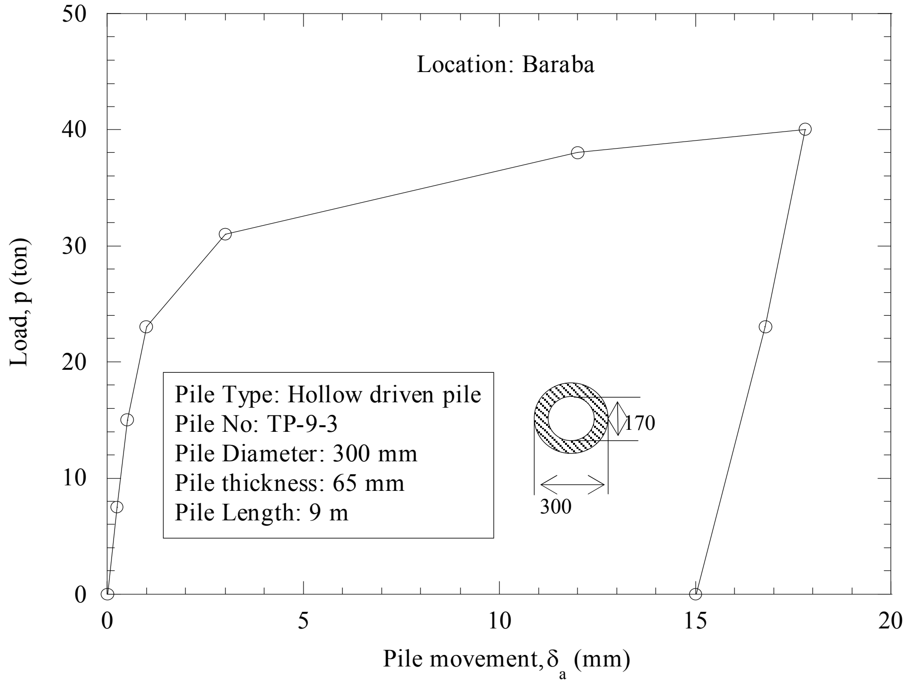


Figure 4.64: Load vs. pile movement curve of the pile TP-9-3 at Site-13

Uplift Capacity Estimated from Sub-soil Characteristics

Pile capacities of the tested piles are estimated using the five methods mentions in chapter 3 (Art. 3.8). The comparison of pile capacity from load tests and different methods is presented in the Table 4.23.

Table 4.23: Results obtained from estimation and load tests

Type of construction	Pile Name	Pile Code	Length (m)	Diameter (mm)	Sec. Type	Capacity from load test (ton)		Capacity estimated from different methods (ton)				
						% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/ American Method
Bored	Site-13 TP-9-3	P19	9	300	Hollow	31	30	9	8	9	4	5

From the results presented in Table 4.23, it is seen that the results obtained from the load test and estimation are significantly different.

Table 4.24: Results obtained from estimation and load tests for all sites

PILE CAPACITY FROM LOAD TEST AND ESTIMATED FROM METHODS														
Type of construction	Pile Name	Pile Code	Length (m)	Dia. (mm)	Sec. Type	L/D	Capacity From Load Test (ton)		Estimated Capacity From Different Methods (ton)					Averages From Load Test (ton)
							% of Dia	Double Tangent method	Meyerhof	Murthy	Tomlinson	DIN 4014	British/American Method	Average of predicted
Bored	Site-1-P1	P01	12.0	400	Solid	30	51	58	24	19	27	12	26	22
	Site-1-P2	P02	12.0	400	Solid	30	50	63	28	22	29	13	33	25
	Site-2-P1	P03	14.0	500	Solid	28	20	23	64	43	47	37	100	58
	Site-3-Tower-56	P04	13.0	500	Solid	26	30	29	40	20	39	27	95	44
	Site-4-Tower-77	P05	9.0	500	Solid	18	22	19	25	11	20	21	45	24
	Site-5-Tower-147	P06	9.5	500	Solid	19	17	18	22	12	21	18	41	23
	Site-6-Tower-25	P07	15.0	500	Solid	30	33	38	65	60	63	28	90	61
	Site-7-Tower-28	P08	15.0	500	Solid	30	31	35	38	33	50	23	56	40
	Site-8-Tower-78	P09	11.0	500	Solid	22	18	18	63	63	44	12	17	40
	Site-9-Tower-17	P10	14.0	500	Solid	28	32	37	25	17	41	19	38	28
	Site-10-Tower-53	P11	14.5	500	Solid	29	40	39	32	25	48	25	88	44
	Site-11-Tower-112	P12	15.0	500	Solid	30	24	25	45	39	61	19	44	42
	Site-12-Tower-51	P13	14.3	500	Solid	29	20	23	29	21	42	14	40	29
Driven pile in bored Hole	Site-13-TP-12-1	P14	12.0	300	hollow	40	38	43	29	25	25	14	38	26
	Site-13-TP-12-2	P15	12.0	300	hollow	40	43	46	24	20	20	15	36	23
	Site-13-TP-12-3	P16	12.0	300	hollow	40	31	38	29	28	22	21	37	27

4.3 Correlation of Results

Correlation between Pile Capacities Obtained from Meyerhof's Equation and Load Test Results

The ultimate capacity of piles obtained from Meyerhof equation and load test is compared in Figure 4.65. From the figure, it is seen that the capacity of the pile determined from the Meyerhof equation is either close to the actual or smaller than the actual one. It seems that to determine the pile capacity from Meyerhof equation, the uplift capacity of the pile determined from the equation is to be multiplied by a factor of 1.08.

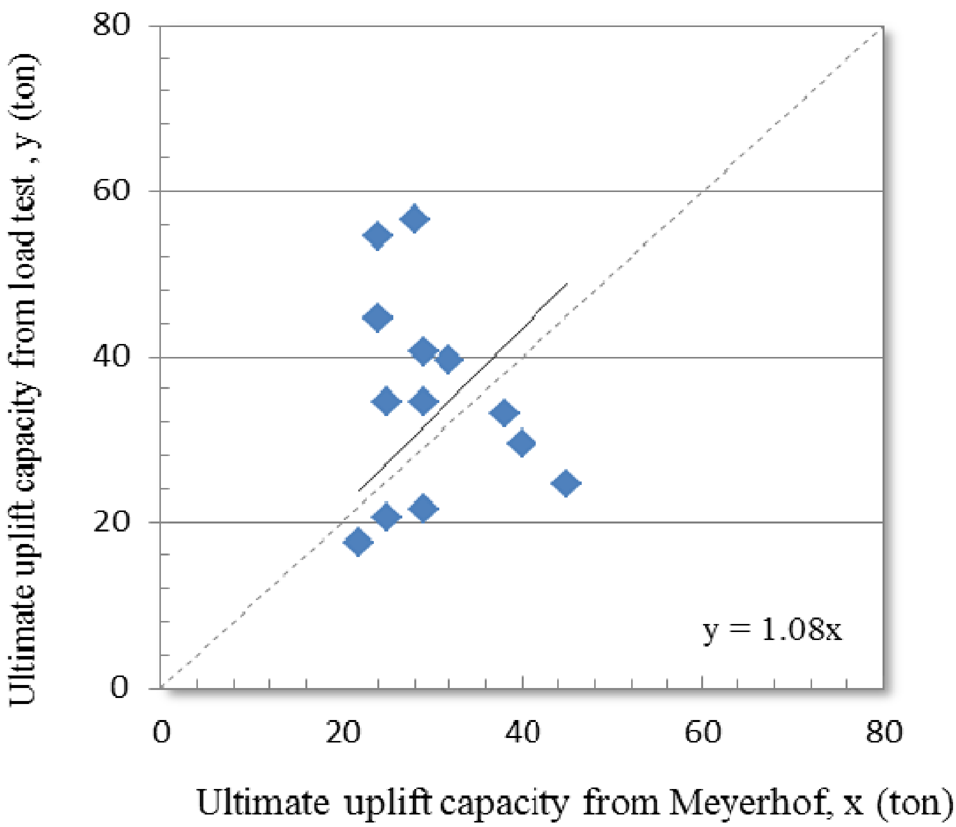


Figure 4.65: Correlation between ultimate uplift capacity from load test and the estimated by Meyerhof

Correlation between Pile Capacities Obtained from Murthy's Equation and Load Test Results

The ultimate capacity of pile obtained from Murthy's equation and load test is compared in Figure 4.66. From the figure, it is seen that the capacity of the pile determined from the Murthy's equation is lower than the actual capacity except few cases. It seems that to determine the pile capacity from Murty's equation, the uplift capacity of the pile determined from the equation is to be multiplied by a factor of 1.22.

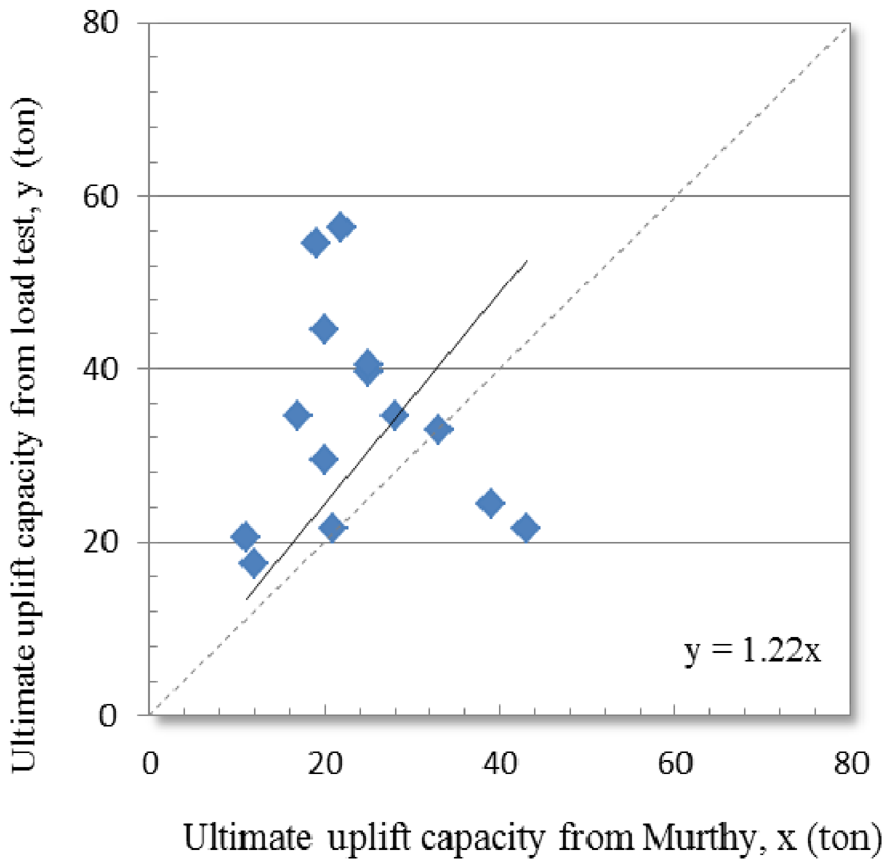


Figure 4.66: Correlation between ultimate uplift capacity from load test and the estimated by Murthy

Correlation between Pile Capacities Obtained from Tomlinson's Equation and Load Test Results

The ultimate capacity of pile obtained from Tomlinson method and load test is compared in Figure 4.67. From the figure, it is seen that the capacity of the pile determined from the Tomlinson equation is higher than the actual capacity in most cases. Hence to determine the actual capacity of the pile, the uplift capacity of the pile determined from the equation is to be multiplied by a factor 0.86.

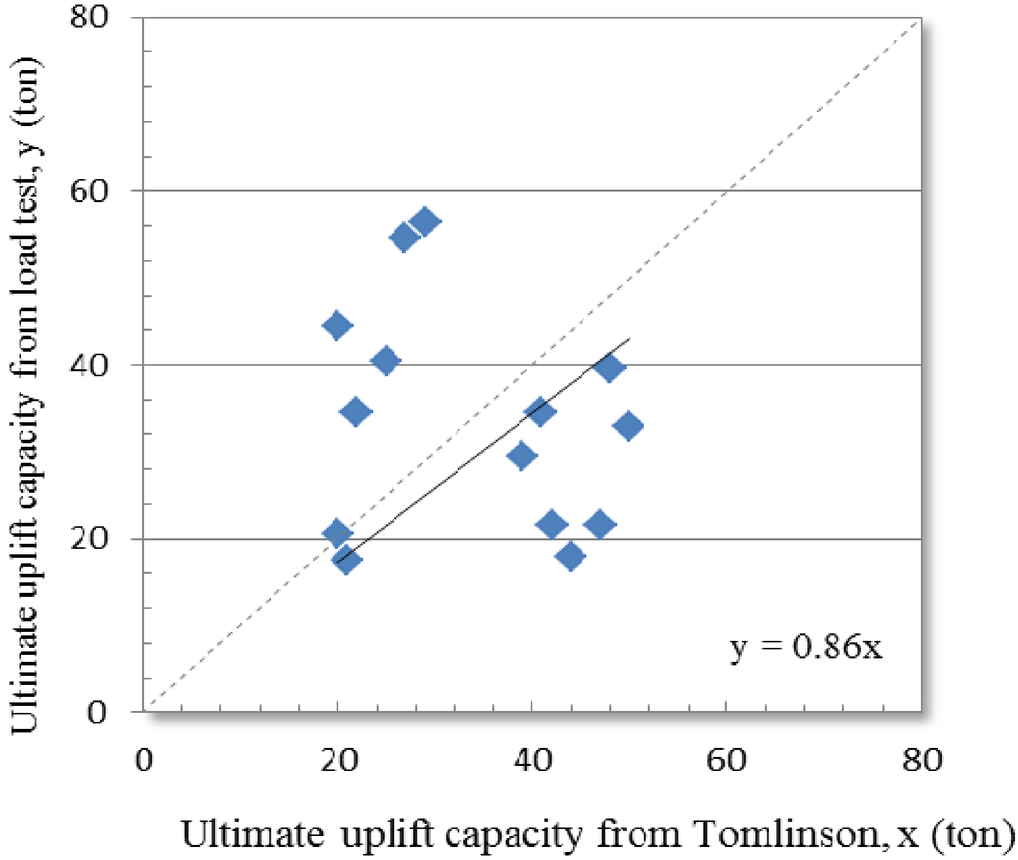


Figure 4.67: Correlation between ultimate uplift capacity from load test and the estimated by Tomlinson

Correlation between the Pile Capacities Obtained from DIN-4014 Method and Load Test Results

The ultimate capacity of pile obtained from DIN-4014 and load test is compared in Figure 4.68. From the figure, it is seen that the capacity of the pile determined from the DIN-4014 equation is lower than the actual capacity. However, the capacity determined from this method is close to that obtained from Murty's equation, to determine the uplift capacity of the pile, the pile capacity determined from the equation is to be multiplied by a factor 1.28.

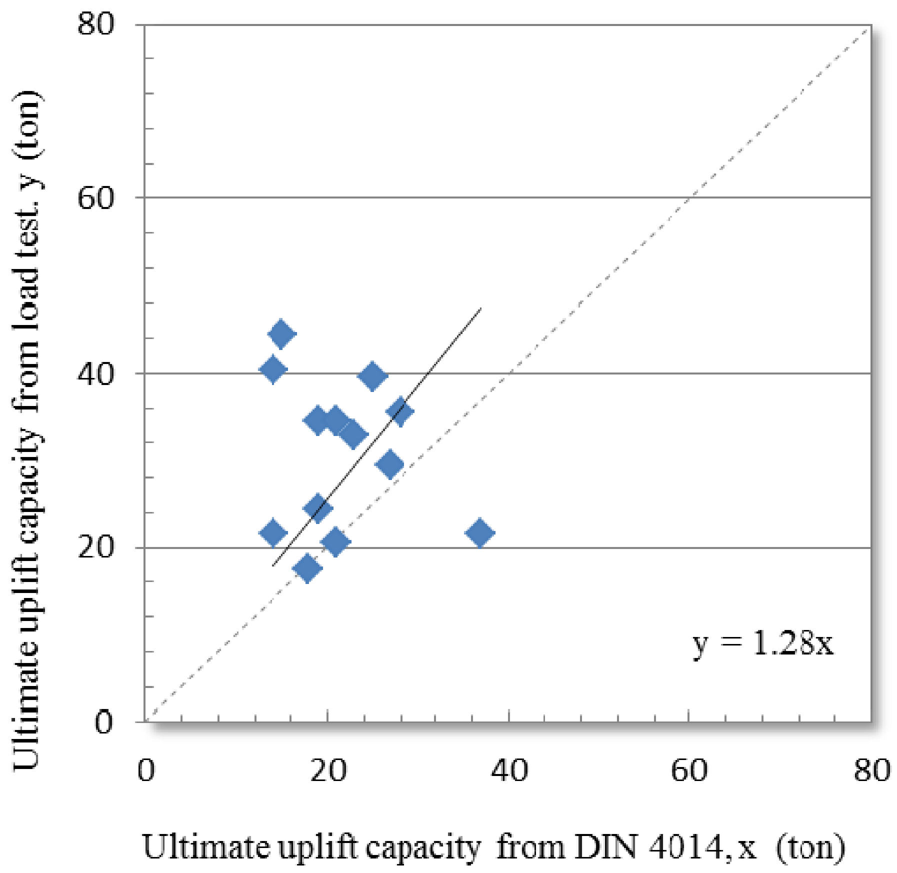


Figure 4.68: Correlation between ultimate uplift capacity from load test and the estimated by DIN-4014

Correlation between the Pile Capacities Obtained from British/American Method and Load Test Results

The ultimate capacity of pile obtained from British/American Method and load test is compared in Figure 4.69. From the figure, it is seen that the capacity of the pile determined from the British/American Method is close to that obtained from load test. It seems that to get the actual capacity of the pile, the pile capacity determined from this equation is to be multiplied by a factor 0.81.

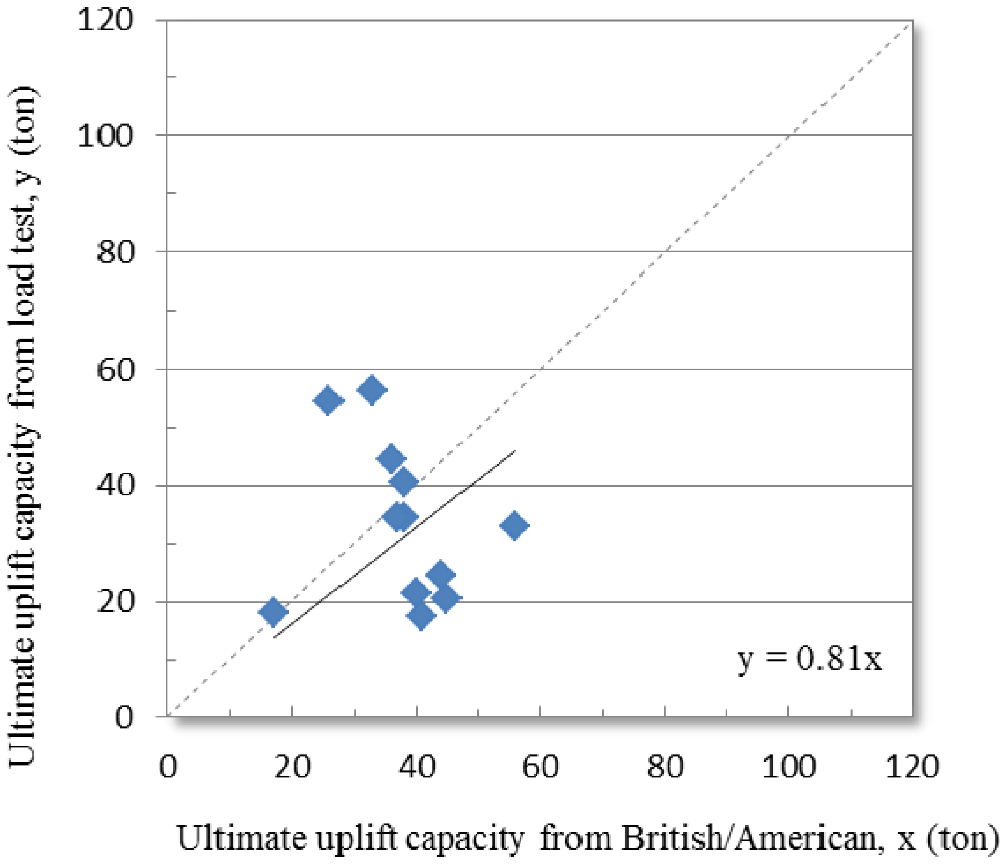


Figure 4.69: Correlation between ultimate uplift capacity from load test and the estimated by British/American Method.

Linear regression of the experimental data showed that to get the actual pile capacity, the capacity estimated by Meyerhof and Murthy equation and the German Code of Practice (DIN 4014) method to be multiplied by a factor of 1.08, 1.22 and 1.28, respectively. On the other hand, the Tomlinson equation and the British/American method over estimated the pile capacity. To get the actual pile capacity, the capacity estimated by the Tomlinson equation and the British/American method to be multiplied by a factor of 0.86 and 0.81, respectively. As far as regression is concerned, the British/American method provided the best regression.

The results of these experiments are based on 19 bored pile load test data. But the large scatter in the experimental data suggests that even a larger sample size is required for a more definitive study to develop a better correlation. Nevertheless, the study conducted in this thesis provides the perfect working ground for future study in this area of research.

Correlation of Results for different L/D Ratio

In this study, the L/D ratio ranges between 18 and 40, the pile capacity depends on L/D ratio and it is clearly seen in site 13 witch have the L/D equal to 18.

Correlation of Results for L/D Equal 40

The ultimate capacity of pile obtained from all estimated methods and includes the one from load test for L/D ratio 40 is compared in Figure 4.70. From the figure, it is seen that the capacity of the pile determined from the Methods consider c and φ with long depth gives more uplift capacity than those depends on only SPT N-value. The estimated capacity is close to the actual capacity.

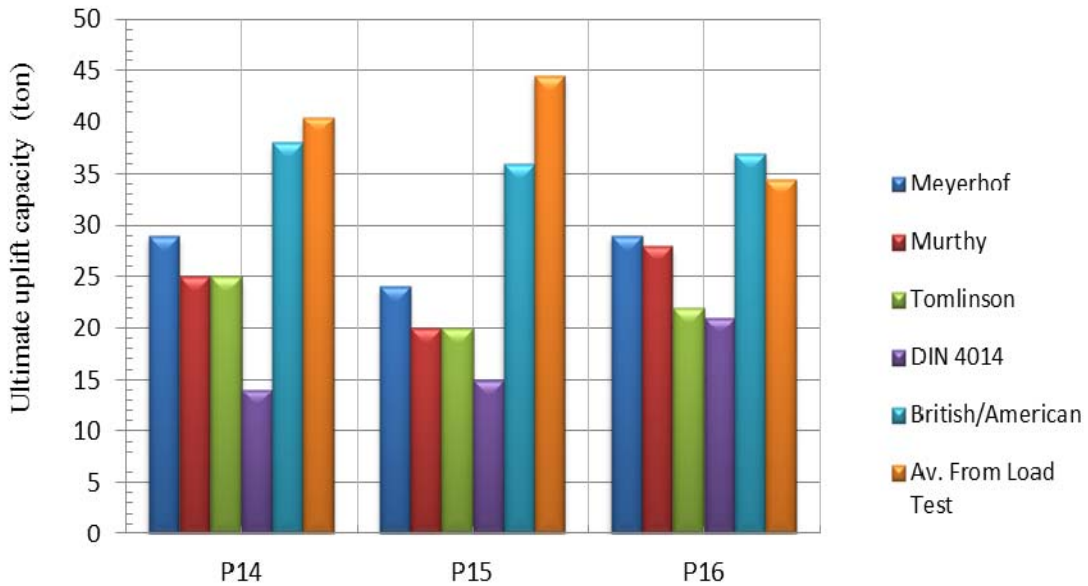


Figure 4.70: Correlation between ultimate uplift capacity from Estimation and load test for L/D = 40

Correlation of Results for L/D Equal 30

The ultimate capacity of pile obtained from all estimated methods and includes the one from load test for L/D ration 30 is compared in Figure 4.71. From the Figure, it is seen that the uplift capacity estimated from British/American method is very high compare to the results from load test this may be due to the soil condition as this method is based on the SPT-N values.

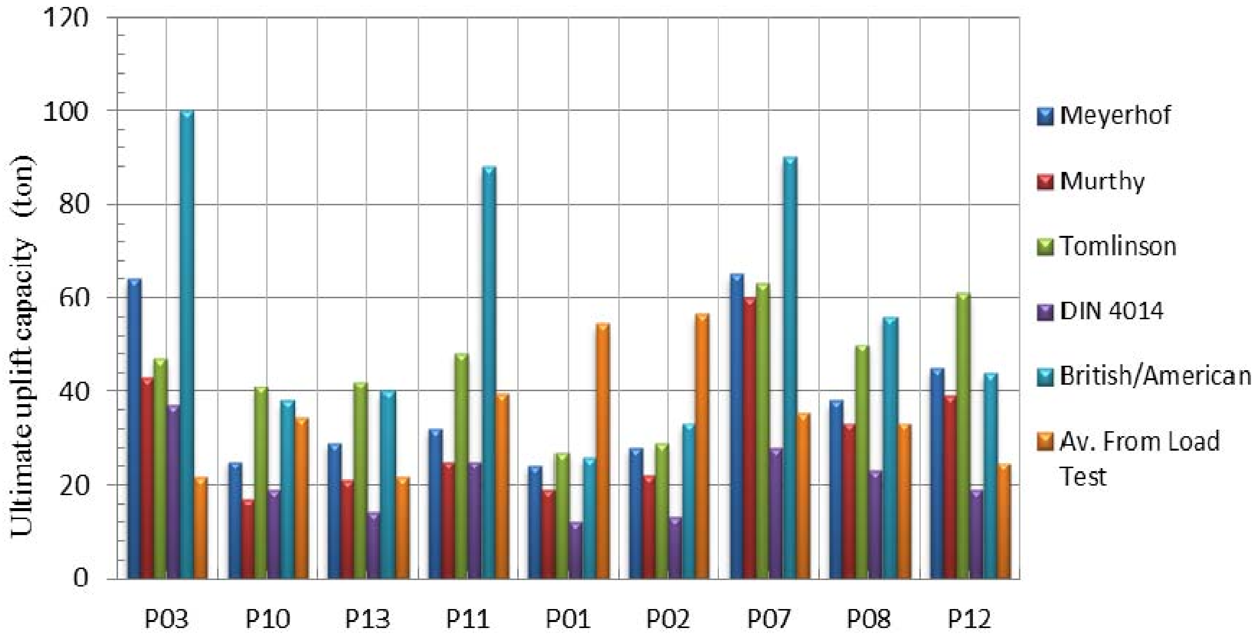


Figure 4.71: Correlation between ultimate uplift capacity from estimation and load test for L/D = 30

Correlation of Results for L/D Equal 25

The ultimate capacity of pile obtained from all estimated methods and includes the one from load test for L/D ratio 25 is compared in Figure 4.72. From the figure, it is seen that the uplift capacity for the short length and small diameter of piles is small compare to the estimated values from equations.

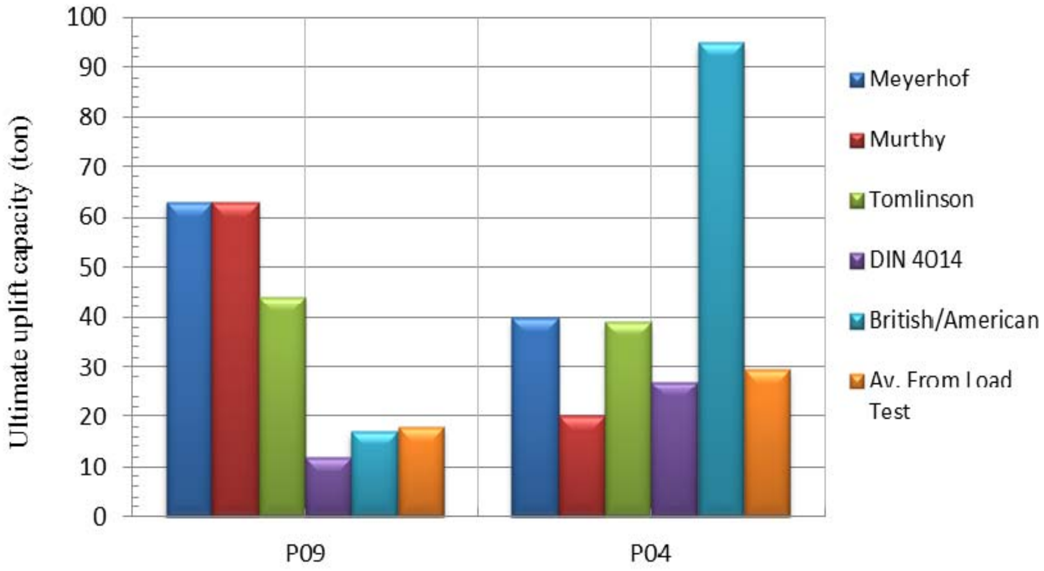


Figure 4.72: Correlation between ultimate uplift capacity from estimation and load test for L/D = 25

Correlation of Results for L/D Equal 18

The ultimate capacity of pile obtained from all estimated methods and includes the one from load test for L/D ratio 18 is compared in Figure 4.73. From the figure, it is seen that the uplift capacity for this two piles is smaller compare to the estimated values from equations except Murthy's equation.

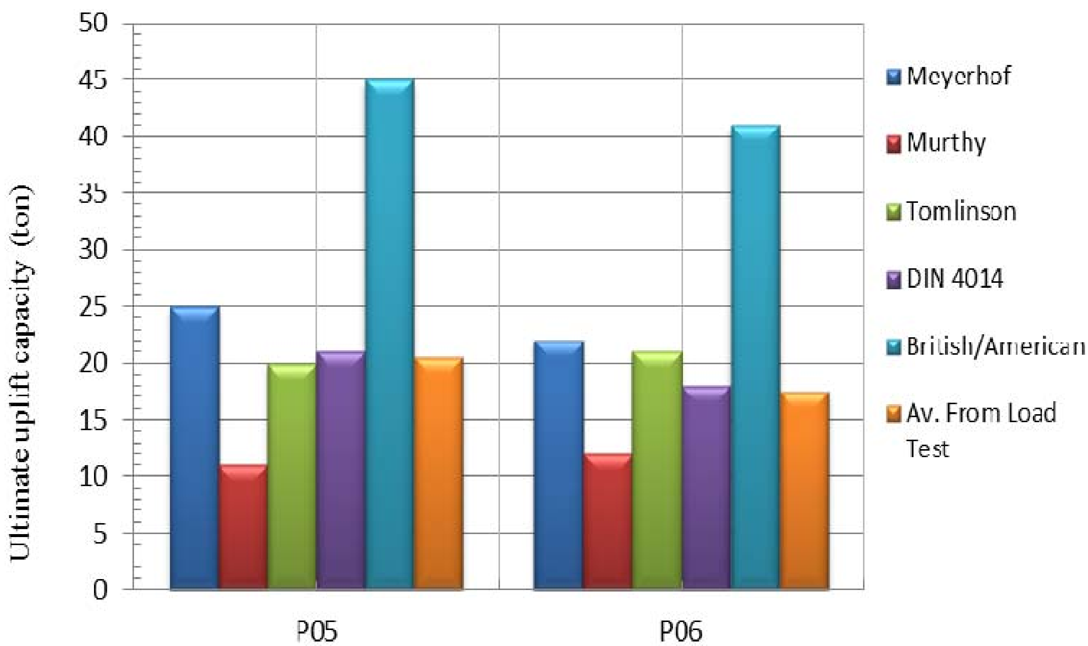


Figure 4.73: Correlation between ultimate uplift capacity from estimation and load test for L/D = 18

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 General

The main objective of this research is to evaluate the methods for determining the uplift capacity of bored piles. To this context, 21 pile load test data under uplift condition has been presented in this thesis. Sites were selected in different parts of Bangladesh. Besides, the capacity of the piles was estimated from the sub-soil characteristics using five methods Meyerhof (1968), Murthy (1992), Tomlinson (1977), the German Code of Practice (DIN 4014), and British/American Method (1974). The capacity of the piles determined from load test and estimated from sub-soil characteristics is compered after excluding the irregular results. The main findings of this study and recommendations for future study have been presented in this chapter.

5.2 Evaluation of the Methods

Five methods – Meyerhof (1968), Murthy (1992), Tomlinson (1977), The German Code of Practice (DIN 4014), and British/American Method (1974) are used for estimating the pile capacity. The first three methods use soil parameters cohesion, c and angle of internal friction, φ for determine the pile capacity. British/American method uses mainly δ (based on φ). The other method i.e., DIN 4014 uses only SPT N-value. The main findings of this study are presented below.

- The pile capacity estimated from Meyerhof equation is close to that obtained from uplift pile test. But in general, the values are less than the results that obtained from load tests. It seems that to determine the pile capacity from Meyerhof equation, the uplift capacity of the pile determined from the equation is to be multiplied by a factor of 1.0839.

- The pile capacity determined from Murthy's equation is smaller than that of the value obtained from uplift load test except two cases. It seems that to determine the pile capacity from Murty's equation, the uplift capacity of the pile determined from the equation is to be multiplied by a factor 1.2227.

- The pile capacity determined from Tomlinson's equation is higher than that obtained from load test. Therefore, to determine the actual capacity of the pile, the uplift capacity of the pile determined from the equation is to be multiplied by a factor of 0.8615.

- Also the pile capacity determined from DIN 4014 method is very close to that obtained from Murty's equation. Therefore, to determine the uplift capacity of the pile, the pile capacity determined from the equation is to be multiplied by a factor of 1.2854.

- Finally, the pile capacity determined from British/American Method is close to that obtained from load test. It seems that to get the actual capacity of the pile, the pile capacity determined from this equation is to be multiplied by a factor 0.8197. This equation is based on pile load test results of 41 piles. Therefore, the determination of pile capacity using this method closer to the real one.

Finally, it can be said that the results of the pile load tests in general showed reasonably good agreement with the theoretical equations. However, linear regression of the experimental data showed that to get the actual pile capacity, the capacity estimated by Meyerhof and Murthy equation and the German Code of Practice (DIN 4014) method to be multiplied by a factor of 1.08, 1.22 and 1.28, respectively. On the other hand, the Tomlinson equation and the British/American method over estimated the pile capacity. To get the actual pile capacity, the capacity estimated by the Tomlinson equation and the British/American method to be multiplied by a factor of 0.86 and 0.81, respectively. As far as regression is concerned, the British/American method provided the best regression.

The results of these experiments are based on 21 bored pile load test data. But the large scatter in the experimental data suggests that even a larger sample size is required for a more definitive study to develop a better correlation.

5.3 Future Recommendations

During the study, it is felt that the following researches can be conducted in future.

- In this study, available equations have been evaluated only for bored pile as it is common in Bangladesh. This study can be extended to determine the uplift capacity of driven piles/precast piles.

- The pile capacity depends on L/D ratio. In this study, the L/D ratio ranges between 18 and 40. So, study can be conducted to evaluate the methods for shorter piles with L/D ratio less than 5.

- Uplift load test can be conducted using micro piles installed in either sand or clay to evaluate the methods more precisely.
- This study deals with the determination of single pile. The methods should also be evaluated for group piles.

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