FOUNDATION ALTERNATIVES IN FILL SOILS OVERLAYING ORGANIC LAYER

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DEDICATED

TO

MY PARENTS

The thesis titled **"FOUNDATION ALTERNATIVES IN FILL SOILS OVERLAYING ORGANIC LAYER",** submitted by Marufa Nasrin, Roll No. 100704249F, Session: October 2007 has been accepted as satisfactory in partial fulfilment of the requirement for the degree of Master of Science in Civil Engineering on 15 May 2010.

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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. Mohammed Shariful Islam, Associate Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology.

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).

Marufa Nasrin

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ABSTRACT

The main objectives of this research are to determine the sub-soil characteristics (especially the properties of the soft organic layer beneath the filling layer) of selected reclaimed areas and to propose suitable alternative foundation systems for such sub-soil condition.

In order to identify the geotechnical parameters, investigations have been carried out at some selected reclaimed areas. Total fourteen soil borings have been conducted at four reclaimed areas of Dhaka city. Besides, data have been collected for six reclaimed areas within Dhaka city. A silty clay or a dense sand layer exists under the filling layer in such areas. The depth of filling layer of sand varies from 1.5~7.5 m from existing ground level (EGL). The thickness of the soft organic clay varies from 3.0~13.5 m from EGL. Beneath this soft organic layer, a soft silty clay and dense sand have been found up to 18 m depth from EGL. Groundwater table exists at 0.6~7.0 m below the EGL. The uncorrected SPT N-value of the filling sand layer varies from 1~11. The mean grain size (D_{50}) and fine content (F_c) was found to vary from 0.015~0.210 mm and 12~31%, respectively.

The uncorrected SPT N-value of the organic clay varies from 1~4. Dry unit weight, moisture content and organic content have been found to vary from 4.6~12.1 kN/m^3, 29~140% and 7.9~29.4%, respectively. It indicates that, this clay is very soft in consistency. Liquid limit (LL) and plasticity index (PI) of organic clay varies from 38~190% and 11~62%, respectively. From the above results, it is observed that this layer contains organic silt (OL) and organic clay (OH) which are medium to highly compressible. Unconfined compressive strength of the clay varies from 6~66 kPa. It has been found that initial void ratio and compression index of soft organic layer vary from 0.87~3.88 and 0.28~1.25, respectively. The coefficient of consolidation (c_v), coefficient of volume compressibility (m_v) and permeability (k) vary from 0.25~17.25 m^2/yr, 0.06×10^{-3}~7.30×10^{-3} kN/m^2 and 9.06×10^{-12}~4.97×10^{-9} m/sec, respectively.

Attempt has been made to correlate unconfined compressive strength (q_u) with SPT N-value, plasticity index (PI) and organic content (OC). Correlation does not exist between q_u and SPT N-value. No definite correlation has been found between q_u and PI as well as OC. However, it has been found that q_u increases with increase of PI and decreases with the increase of OC. Attempt has also been made to correlate compression index (C_c) with OC and initial void ratio (e_o). In this case also no definite correlation was obtained. The increasing tendency of C_c has been observed with the increase of OC. However, a correlation between C_c and e_o has been found.

Total consolidation settlement and time dependent settlement of organic layer due to the surcharge of the filling layer have been estimated. It has been found that total consolidation settlement of the study area varies from 447~1734 mm. It has been observed that settlement time (considering single drainage) varies from 6~106 years.

Performance of foundation systems in similar sub-soil condition has been studied. Some foundation systems are suggested for the study areas. Spread footing with Rammed Aggregate Pier (RAP), buoyancy raft foundation and pile foundation have been suggested. The sub-soil condition indicates that negative skin friction may develop in case of pile foundation in such sub-soil condition. Possible mitigation methods for negative skin friction have been discussed.

Keyword: Reclaimed area, fill soil, soft organic clay, strength and compressibility characteristics.

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NOTATIONS

Symbol	Description
σ_v	Total overburden stress
$\overline{q}$	Total effective overburdened pressure
γ	Unit weight of soil enclosed in the pile group to depth L_f
p'_g	Perimeter of the pile group
ϕ	Angle of internal friction
γ_d	Dry density
ω_n	Natural moisture content
ℓ/d	Pile slenderness ratio
Δ	Immediate settlement
ΔH	Consolidation settlement
ΔS	Secondary (creep) settlement
μ_o	Effective overburdened area
A	Area of pile group enclosed in the perimeter p'_g
A	Surface area of raft foundation
a_{max}	Peak horizontal acceleration on the ground surface
A_p	Area of pile shaft
A_p	Area of the pile tip
A_{si}	Perimeter of pile
B	Width of the footing
B	Breadth of the raft

c	Cohesion
c'	Cohesion of the pile supporting to the pile tip
c_α	Coefficient of secondary consolidation
C_c	Compression index
C_r	Recompression index
C_R	Virgin compression ratio
c_v	Coefficient of consolidation
d	Depth factor
D_{50}	Mean grain size
D_f	Depth of embedment
D_f	Depth of raft
D_k	Characteristics downdrag force
D_r	Relative density
E_{60}	60% of the theoretical potential energy
$E_{measured}$	Measured energy
e_o	Initial void ratio
e_o	Initial void ratio
f	Unit friction resistance at depth z
F_c	Fines content (% passing sieve # 200)
F_d	Total design load
FS	Factor of safety
$f_{s\text{-}ve}$	Negative friction per unit area
G_s	Specific gravity
H_t	Thickness of consolidating layer

i	Inclination of load factor
I	Influence factor (Poulos and Devis, 1974)
k	Coefficient of permeability
k	Coefficient of permeability
K	Effective earth pressure
k_f	Overall foundation stiffness
k_P	Pile group stiffness
L	Length of the raft
LL	Liquid limit
m_v	Coefficient of volume compressibility
m_v	Coefficient of volume compressibility
N	Field SPT N-value
N_c, N_q, N_γ	Dimensionless bearing capacity factors
N_f	Total negative skin friction
N_{field}	Field observed N-value
OC	Organic matter content
OCR	Over consolidation ratio
P	Perimeter of the pile section
p'_c	Maximum preconsolidation pressure
p'_o	Vertical effective pressure
PI	Plasticity index
PL	Plastic limit
P_{nf}	Amount of negative skin friction resistance carried by the point
Q	Total structural load

q'	Effective vertical stress at the level of pile tip
Q_N	Total negative friction on the pile per unit length
Q_p	Load carrying capacity of the pile point
Q_p	Frictional resistance
Q_p	Unit point resistance
q_u	Unconfined compressive strength
R	Shaft resistance
R_k	Characteristics shaft resistance
S	Shape factor
s/d	Pile spacing ratio
S_c	Total consolidation settlement
s_u	Undrained shear strength
V_k	Characteristics applied load
α	Empirical adhesion factor
α_{cp}	Average interaction factor between the pile and pile cap
β	Reduction factor for negative skin friction
γ	Unit weight of soil
δ	Soil pile friction angle
ΔL	Incremental pile length over which P and f are taken to be constant
$\Delta\sigma$	Increase of stress due to structural load
ε_f	Failure strain
λ	Pile stiffness ratio
ν	Poisson's ratio
σ_o'	Effective overburdened pressure

σ_o'	Effective vertical stress
φ	Angle of internal friction

Chapter ONE
INTRODUCTION

1.1 GENERAL

Over the past 30~40 years, Dhaka city has experienced a rapid growth of urban population and it will continue in future due to several unavoidable reasons. Hence, most of the areas of Dhaka city have already been occupied. As a result, different new areas are being reclaimed inside and near Dhaka city by both government and private agencies. General practice for reclaiming such areas is to fill low lands (ditches, lakes etc.) inside and near the city.

In most cases, the practice for developing new areas is just to fill low land by dredge fill materials. Different filling procedures are in practice to develop such land. One of them is to carry soil by vehicles from remote sources and manually dumped at the filling site. Due to huge traffic congestion, most widely used method is hydraulic filling procedure. In this procedure, soil is collected from riverbeds/riverbank by cutter-suction dredging into a barge, which is carried to the nearest river site. Soil is then pumped through the pipes in a slurry form after mixing with water, and transferred to the point of deposition, upon the surface and being filled. Large volume of fill material is required for the land filling in reclaimed areas. Because of huge amount needed, such fill materials are usually collected from river bed and river bank.

Filling material is dumped directly upon the marshy low land. The organic content beneath the filling is decomposed and produces a very soft organic clay layer. This very soft organic clay layer may cause excessive settlement problem to the structures having shallow foundation. As well as, it may cause geotechnical problems such as negative skin friction to the deep pile foundation. Negative skin friction produces a drag load which can be very large for long piles. Johannessen and Bjerrum (1965), and Bozozuk (1972) reported measurements of drag loads. In addition, it exceeds the allowable loads that ordinarily would have been applied to the piles in case of marine clay.

In most cases, the dredged material is almost silty sand with high fine content (Ahamed, 2005). The presence of fines in hydraulic fill means greater compressibility and greater difficulty in compaction of the fill. Fine also reduce permeability and hence the rate of drainage is slow. Therefore, consolidation rate is also slow (Ahamed, 2005). Since Dhaka city exists in seismic Zone 2 (peak ground acceleration, a_{max} = 0.15g) of Bangladesh (BNBC, 1993) this silty sand layer may liquefy if an earthquake of sufficient magnitude occurs.

Some studies have been carried out locally to understand the characteristics of silty sand layer (Ahamed, 2005; Hossain, 2009). These studies mainly focused on the liquefaction problem/potential of such areas. Studies did not investigate the problem that may occur due to the presence of organic clay layer. Islam et al. (2004) investigated mechanical properties of soft organic Dhaka clay. However, properties and foundations problems in similar soils of Khulna city had been investigated in such study (Ferdous, 2007; Khan and Ferdous, 2004). Islam et al. (2007) investigated compressibility properties of reconstituted organic soils at Khulna region of Bangladesh. And other study also have investigated which is based on marine clay deposited reclaimed ground (Kim, 2008).

Therefore, it is clear that this very soft organic clay layer, in reclaimed areas demand special attention for designing foundation systems on it. So, it is felt necessary to carry out research to know the characteristics of the soft organic clay layer of such reclaimed areas and propose suitable alternatives for foundation systems on such soil.

1.2 RECLAIMATION PROCEDURES

Due to rapid urbanization, many areas are being reclaimed in and around Dhaka city. Many of the low-lying areas around the city of Dhaka are now being filled for housing by developers. Huge volume of soil is required to fill for these reclaimed areas. Because of the severe road traffic congestion around the sites, the filling soil has to come through river that has to be collected from nearest river beds or river banks.

The following methods are presently used for filling these low-lying areas:

a) Soil being carried by country boats from remote sources and manually dumped at the filling site.

b) Soil being carried by trucks from remote sources and manually dumped at the filling site.

c) Soil is collected from riverbeds by dredging into a barge, which is carried to the nearest river site. Soil is then pumped through pipes in a slurry form after mixing with water, and transferred to the point of deposition.

d) Soil is dredged from riverbed by dredging and directly pumped to the filling site thorough discharge pipes.

In most cases in our country where large volume of fill materials is required, a hydraulic filling procedure similar to method No. (c) or (d) is followed (Ahmed, 2005). Method No. (a) or (b) are used where small volume of filled material is required. In hydraulic filling, soil suspended in water is pumped through a pipe and the mixture discharged upon the surface being filled. It has been mentioned that the presence of fines in a hydraulic fill means greater compressibility together with greater difficulty in compaction of the fill. Fines also reduce permeability and hence the rate of drainage. For these reasons, contracts governing the placement of hydraulic fill generally contain a specification aimed at minimizing the amount of fines in the resulting fill. It is especially important to avoid ponding of water where fines might settle out to form soft pockets or layers.

1.3 BACKGROUND OF THE PRESENT RESEARCH

The followings are the background of the present research

- Dhaka city has experienced a rapid growth of urban population and it will continue in future due to several unavoidable reasons. Hence, most of the areas of Dhaka city have already been occupied. As a result, different new areas are being reclaimed inside and near Dhaka city by both government and private agencies.

- In most cases, the dredged material is almost silty sand with fine content (Ahamed, 2005). The presence of fines in hydraulic fill means greater compressibility and greater difficulty in compaction of the fill. Fines also reduce permeability and hence the rate of drainage is slow therefore self-weight consolidation rate is also slow.
- General practice for reclaiming such areas inside and near Dhaka city is to fill low lands (ditches, lakes etc.). Filling material is dumped directly upon the marshy low land.

  After a certain time, the organic content beneath the previous surface water is decomposed and produces a soft organic clay layer.
- Some studies have been carried out which is mainly focused on the mechanical properties of soft organic clay in a particular place in Dhaka and liquefaction problem/potential of such reclaimed areas in Dhaka city. Studies did not investigate the problem that may occur due to the presence of organic clay layer beneath the filling layer. However, the properties of similar soil of Khulna city has investigated and foundation alternatives have proposed for that area. Similar investigations may be conducted for the reclaimed areas of Dhaka city.

Therefore, it is clear that this very soft organic clay layer, in reclaimed areas demand special attention for designing foundation system on it. So, it is felt necessary to carry out research to know the characteristics of the soft organic clay layer of such reclaimed areas and propose suitable alternatives for foundation systems on such soil.

1.4 SCOPE AND OBJECTIVES

Based on the above-mentioned background the main objective of this research was to propose the suitable foundation alternatives in fill soils overlaying soft organic soils. However, in many cases necessary geotechnical data were limited. With this limitation, the objectives of the present study are as follows:

a) To study the filling procedure of reclaimed areas and to characterize the sub-soil of selected reclaimed sites.

b) To make an attempt to correlate the shear strength parameters and compressibility properties with index properties.

c) To estimate the total settlement and time dependent settlement of soft organic clay layer due to the filling in reclaimed areas.

d) To suggest foundation alternatives for the areas having similar sub-soil condition.

Possible outcome of this research are as follows:

a) Sub-soil characteristics of such reclaimed areas will clarified from this study.

b) It is expected that the results obtained from the research can be used for foundation design in reclaimed areas to minimize future possible hazards that may occur due to excessive settlement, negative skin friction, earthquake induced liquefaction, etc.

c) Foundation alternatives can be recommended for such areas.

1.5 THE RESEARCH SCHEMES

The whole researches have been conducted according to the following steps:

a) Field surveys have been conducted to know the development/ reclamation procedure of the reclaimed areas of Dhaka city.

b) Field investigation that includes approximately 14 (fourteen) boreholes at different locations of Dhaka city have been conducted. During drilling, SPT have been taken at 1.5 m intervals and disturbed and undisturbed samples have been collected.

c) Laboratory tests such as sieve analysis, hydrometer test, Atterberg′s limits, unconfined compression test, consolidation test have been performed in order to determine index properties, strength properties and compressibility properties (Compression index, C_c; coefficient of consolidation, c_v; coefficient of volume compressibility, m_v, etc.) of different soil layers with a special attention to the soft organic clay layer.

d) By using all data, correlation between shear strength parameters and compressibility properties with index properties have been obtained.

e) Total settlement and time dependent settlement of soft organic clay layer due to the filling in reclaimed areas have been calculated.

f) By analyzing all results, suitable alternative foundation systems in reclaimed areas of Dhaka city have been suggested.

1.6 ORGANIZATION OF THE THESIS

The thesis work conducted for achieving the stated objectives is presented in several chapter of this thesis. The thesis consists of six chapters. A brief description of each chapter is as follows.

Chapter One describes geology of Dhaka, reclamation procedure of reclaimed area within Dhaka city, background of this study, objectives with scope and research schemes.

Chapter Two discusses about the development procedure of reclaimed area of Dhaka city, past research related to this study, properties of organic soils and correlations, organic layer in reclaimed area, analysis procedures of bearing capacity, settlement and negative skin friction. Possible solutions to mitigate problem due to organic layer also have been discussed in this chapter. At the end of this chapter, some case study of typical foundation for soft soil has also been described. Typical foundation for soft soil includes Rammed Aggregate Pier (RAP) foundation, Mattress foundation, Floating/ buoyancy raft foundations, Piled raft foundation and Deep pile foundation.

Selected study areas for the research have been discussed in Chapter Three. Experimental programs including test procedures and methods have been described. Summary of field tests and laboratory investigation that have been conducted in this study is presented in this chapter.

Chapter Four discusses detail sub-soil characteristics of study areas including field and laboratory tests. Field tests include Standard Penetration Tests (SPT). Laboratory tests such as grain size analysis, organic content test, Atterberg limit and unconfined compression tests, consolidation tests are described in this chapter. Correlations of index properties with strength properties and compressibility properties have been presented in this chapter. At last settlement of organic layer due to filling layer has been presented in this chapter.

Chapter Five contains general soil profiles and suggested alternatives foundation systems such as RAP foundation, mattress foundation, floating/ buoyancy raft foundations, deep pile foundation and piled-raft foundation. Techniques of reduce negative skin friction of piles have also been discussed here.

Chapter Six presents the conclusions of sub-soil characteristics, correlations, observed settlement and foundation alternatives. Recommendations for future studies have also been discussed in this chapter on the basis of the present study.

Chapter TWO
LITERATURE REVIEW

2.1 GENERAL

Past researches related to organic clay in home and abroad have been analyzed in this chapter. In addition to that, detail estimation procedures of bearing capacity, consolidation settlement and negative skin friction have been discussed here. Possible mitigation methods have also been discussed in this chapter including ground improvement techniques and foundation systems with special attention to the soft organic layer. At the end of this chapter, some typical case studies of different types of foundation system for soft clays have been presented.

2.2 PREVIOUS INVESTIGATIONS

Various type of research works have been conducted in different time by different authors. Summary of some past researches in home and abroad related to this study have been described in this part.

Islam et al. (2004) investigated mechanical properties of soft organic clay in a particular location in Dhaka. It was found that top 8~9 m of the sub-soil consists of soft organic clay of high plasticity. Liquid limit, plasticity index of the soft organic clay samples are in the range of 72 to 94% and 42 to 57%, respectively. Natural moisture content and initial void ratio of the samples were in the range of 46 to 83% and 1.16 to 2.01, respectively. Strength, compressibility, swelling and permeability properties of undisturbed samples were also evaluated. Unconfined compressive strength of the samples indicated that these samples were very soft. The values of compression index found to vary between 0.33 to 0.59 while swelling index varied between 0.02 and 0.13. The coefficient of consolidation and permeability of the samples were also determined from the consolidation test. Permeability change index (C_k) was determined from approximate linear e-logk relationships. C_k and the ratio C_c/C_k vary between 0.22 to 0.55 and 0.98 to 1.54, respectively. Permeability parameters (n, C) were determined from approximate linear $\log_{10}e$-$\log_{10}[k(1+e)]$ relationships. In addition, permeability parameters n and C were found to vary between 4.8 to 13.0 and 0.09×10^{-10} to 11.2×10^{-10} m/sec. The relation, $k = C_{en}/(1+e)$ were used to predict permeability-void ratio relation for organic soils of selected place in Dhaka. The relationship between compression index (C_c) and initial void ratio of the organic samples were found in this study can be express by C_c=0.25(e_o+0.194).

Khan and Ferdous (2004) investigated erratic soft sub-soil deposits and presence of organic layer in Khulna City Corporation (KCC) area. It is situated at the North side of the world's largest Mangrove-Sundarban within southwest part of Bangladesh. It poses potential challenges to the design and construction of foundation for building structures within the area. At the end of this study, an attempt was made to perform a systematic geotechnical investigation consisting of different field and laboratory tests. Sub-soil investigations were carried out and subsequent laboratory investigations were performed in order to identify the index properties, shear strength parameters, and compressibility characteristics of different soil layers.

Bo et al. (2004) investigated reclamation and soil improvement of ultra-soft soil. In recent years, more and more reclamations have been carried out on waste ponds or mine tailing ponds. The deposits in these ponds are ultra-soft and may still be undergoing self-weight consolidation. Reclamation and soil improvement works in these areas are very challenging. They require special attention. Conventional reclamation and soil improvement methods are no longer workable. This paper highlights the experience obtained from reclamation and soil improvement works on such soil in a project in Singapore. The fill was placed using a sand spreader, and the ultra-soft deposit was strengthened with a high-strength geotextile. Prefabricated vertical drains and staged construction were used to accelerate the consolidation process. Monitoring of settlement and pore pressure requires modification of conventional instruments to cater for the low strength of the ultra-soft soil and the large strain of deformation. The paper also outlines a new method to estimate the settlement based on large-strain theory. Finally, a case study is presented to validate the proposed method in the determination of settlement magnitude and the time rate of settlement.

Ahamed (2005) conducted preliminary evaluation of liquefaction potential of some selected reclaimed areas of Dhaka city. The purpose of this research was to determine the sub-soil characteristics and liquefaction potential of reclaimed areas of Dhaka city. The research included survey of development procedure of reclaimed areas and laboratory and field tests to determine the sub-soil characteristics of such areas. The study has extended up to determination of liquefaction potential of the selected areas. It was observed that some parts of the reclaimed areas are susceptible to liquefaction and some parts are not liquefiable.

Ferdous (2007) investigated the geotechnical characteristics of the sub-soil in Khulna City Corporation (KCC) area. Erratic soft soil deposit and presence of soft organic layer in KCC area pose potential challenge to the design and construction for foundation for building structures within this area. This research investigated sub-soil characteristics of this area. Based on geotechnical characteristics identification in this study, it was suggested that Rammed aggregate pier (RAP) may be useful ground improvement technique for low to medium rise building for this study area. Preloading with Vertical Drains (PVD) may be considered to be the suitable. Piled raft foundation system also suggested for tall building where basement is required.

Islam et al. (2007) investigated the compressibility properties of reconstitute organic soils content at Khulna region of Bangladesh. Reconstitute soils were prepared manually to have the soil samples of required organic with wide variation. Different samples of Reconstitute soils for the organic contents of about 5 to 42% performed in the laboratory by adjusting the different proportion of inorganic and organic soil samples with a water content equal to 1.25 times of liquid limits. The results showed that the compressibility properties of reconstituted soils increased significantly with the increases of percentage of organic contents.

Rahman (2008) investigated strength and deformation characteristics of cement and lime treated soft clay. This study presented experimental and numerical investigations for the effect of cement and lime treatment on compressibility, permeability, stress–strain, strength and stiffness behaviour of three soft clays. Clay samples were collected from Gazipur, Gopalgonj and Khulna districts in Bangladesh. Soils were collected from a depth of 2 to 3 m from EGL. Set of variables considered in the testing program include a wide range for type of clay (PI =13 to 47%), type of admixture, clay-water/cement ratio (2 to 30), curing time (1 to 104 weeks), mixing water content (120 to 250%) and effecting confining pressure(50 to 400 kPa). The significant decreases in compression index (C_c) and swelling index (C_s) were observed with increasing admixture (cement/lime) content and increasing curing time. C_c and C_s values were also increased significantly with the increases of mixing clay-water content. The lime treated clays gained comparatively higher void ratio and volumetric strains and lower yields stress than those of cement-treated clays. Permeability of lime treated clays were found to be higher than those cement treated clays.

Hossain (2009) conducted the liquefaction potential of selected areas of Dhaka city based on both Standard Penetration Test (SPT) and Shear Wave Velocity. Small Scale Microtremor Measurements (SSMM) was conducted to determine shear wave velocity. It was seen that there is a probability of liquefaction to occur in reclaimed areas of Dhaka city especially for the locations reclaimed by dredged soil up to the filling depth. It was also found that estimated results of liquefaction potential of the studied locations were different for different methods. The uncorrected SPT N-value, uncorrected shear wave velocity, mean grain size (D_{50}) and fines content (F_c) of liquefiable soil were varied from 2 to 11, 70 to 125 m/sec, 0.14 to 0.19 mm and 12 to 28%, respectively. A relation between shear wave velocity and SPT N-value was also obtained in this study.

2.3 PROPERTIES OF ORGANIC SOILS AND CORRELATIONS

The available geotechnical data i.e., the index properties, shear strength properties and compressibility properties obtained from different sources are presented in the following subsections.

Organic Matter Content

Organic deposits are due to decomposition of organic matters and found usually in topsoil and marshy place. A soil deposit in organic origin is said to peat if it is at the higher end of the organic content scale (75% or more according to some other), organic soil at the low end, and muck in between. Peat soil is usually formed of fossilized plant minerals and characterized by fiber content and lower decomposition.

However, there are many other criteria existed to classify the organic deposits and it remains still as controversial issue with numerous approaches available for varying purpose of classification. Soil from organic deposits and it refers to a distinct mode of behavior different than traditional soil mechanics in certain respects. A possible approach is being considered by the American society for Testing and Materials for classify (Table 2.1) soils having organic contents (OC) which may states follows (Edil, 1997).

Table 2.1 Classification of organic soils having organic content (OC) according to ASTM (Edil, 1997)

Organic content (OC)	Description according to (ASTM)
< 5 %	Little effect on behavior; considered inorganic soil.
6 ~ 20 %	Effects properties but behavior is still like mineral soils; organic silts and clays.
21 ~ 74 %	Organic matter governs properties; traditional soil mechanics may be applicable; silty or clayey organic soils.
> 75 %	Displays behavior distinct from traditional soil mechanics especially at low stress.

Peats have certain characteristics that set them apart form moist mineral soils and required special considerations for construction over them. This special characteristic includes:

- High natural moisture content (up to 1500%)
- High compressibility including significant secondary and even tertiary compression.
- Low strength in natural content.

Razzaque and Alamgir (1999) reported that the organic matter content at different depths of boreholes at Gollahmari (Khulna University) area varies from 3.27 to 49.81%. Ferdous (2007) reported that organic content of the sub-soil of the south and north part of KCC area varies from 13 to 43.82%. Range of organic matter content has been presented in Table 2.2.

Specific Gravity

The usual values of specific gravity of organic clay vary from 2.14 to 2.17 (BRTC, 2003) or may be even less than 2.0 (Bowles, 1978). Ferdous (2007) reported that specific gravity of organic clay in KCC area varies from 1.61 to 2.29.

Table 2.2 Range of organic matter content

Organic Content (%)	Soil description	Reference
2.49 ~ 49.81	Organic clay	Razzaque and Alamgir (1999)
13.00 ~ 43.82	Organic clay	Ferdous (2007)

Table 2.3 Ranges of specific gravity

Specific gravity	Soil description	Reference
2.14 ~ 2.49	Organic clay, Khulna	BRTC (2003)
1.61 ~ 2.29	Organic clay, Khulna	Ferdous (2007)
1.94 ~ 2.53	Organic clay, Dhaka	Islam et al. (2004)
Variable and may be < 2	Organic clay	Bowels (1978)

After an investigate in at a particular location within Dhaka city, Islam et al. (2004) reported that specific gravity of organic clay varies from 1.94 to 2.53. Range of specific gravity of sub-soil of different locations has been shown in Table 2.3.

Moisture Content

Razzaque and Alamgir (1999) reported that natural moisture content might be as high as 134% for organic clay. Moisture content ranges have been shown in Table 2.4. Siddique et al. (2002) also reported the moisture content within the upper 6 m in Khulna University area to be as high as 400%. That moisture content of organic layer at Chota Boyra (Khulna Medical College) area was found to be in the range of 118 to 222% (BRTC, 2003). Islam et al. (2004) found moisture of organic clay varies from 46 to 83% within Dhaka city. Range of moisture content of organic clay at Farazipara varied from 157 to 212% (Ferdous, 2007).

Atterberg Limits

Islam et al. (2004) investigated mechanical properties of organic clay in a selected area within Dhaka city. Liquid limit (LL) and plasticity index (PI) varied from 72 to 94% and 42 to 57%, respectively. LL and PI of sub-soil of KCC area was found to vary from 109 to 114% and 54 to 57%, respectively (Ferdous, 2007). Value of LL, PL and PI of sub-soil of different locations has been shown in Table 2.5.

Table 2.4 Ranges of moisture content

Moisture content (%)	Soil description	Reference
118 ~ 222	Organic clay, Khulna	BRTC (2003)
157 ~ 212	Organic clay, Khulna	Ferdous (2007)
46 ~ 83	Organic clay, Dhaka	Islam et al. (2004)
As high as 134	Organic clay, Khulna	Razzaque and Alamgir (1999)
As high as 400	Organic clay, Khulna	Siddque et al. (2002)

Table 2.5 Range Liquid limit (LL), plastic limit (PL), plasticity index (PI)

Atterberg limits			Soil description	Reference
LL	PL	PI		
80 ~ 352	55 ~ 171	24 ~ 181	Organic clay, Khulna	Hossain and Rahman (2005)
109 ~ 114	55 ~ 57	54 ~ 57	Organic clay, Khulna	Ferdous (2007)
72 ~ 94	30 ~ 46	42 ~ 57	Organic clay, Dhaka	Islam et al. (2004)

Table 2.6 (a) Correlations between consistency, N-value and q_u of cohesive inorganic fine-grained soils (after Terzaghi and Peck, 1967)

Consistency	N-value	Unconfined compressive strength, q_u (kPa)*
Very soft	0-2	< 25
Soft	2-4	25-50
Medium stiff	4-8	50-100
Stiff	8-15	100-200
Very stiff	15-30	200-400
Hard	> 30	> 400

*Terzaghi and Peck (1967) used the unit in ton/sq. ft. which is converted here to kPa, assuming 1 ton/sq. ft. = 100 kPa.

Table 2.6 (b) Undrained shear strength and consistency of clays (based on BS 5930)

Consistency	Undarined shear strength, q_u (kPa)*
Very soft	< 20
Soft	20-40
Soft to firm	40-50
Firm	50-75
Firm to stiff	75-100
Stiff	100-150
Very stuff to hard	> 150

Correlation between q_u and SPT N-value

The approximate correlation between SPT N-value and unconfined compressive strength, q_u for cohesive soils as suggested by Terzaghi and Peck (1948 & 1967) is q_u

= *f*. N (kPa), Where, f = 12.50 (for very soft to soft clay) and 13.33 (for the medium stiff to hard clay). On the basis of this, the consistency of clayey silts may be described as shown in Table 2.6 (a) and Table 2.6 (b).

The correlation between N and q_u as obtained by Sowers (1953 and 1962) for cohesive soils have been presented in Table 2.7. It is seen that the average values of f range from 6.7 to 24. Sanglerat (1972) proposes the following relationships between N and q_u for different soil types with the values of f ranging from 13.33 to 25.0 have been shown in Table 2.8.

Table 2.7 Correlations between N-Value and unconfined compressive strength for different soil types (after Sowers, 1953 and 1962)

Soil types	Unconfined compressive strength, q_u (kPa)		
	Minimum	Average	Maximum
Highly plastic inorganic clay	14.4 N	24 N	33.6 N
Medium to low plastic clay	9.6 N	14.4 N	19.2 N
Plastic silts, clays with failure planes	4.8 N	6.7 N	9.6 N

Note: N = Uncorrected SPT N-value; q_u = unconfined compressive strength

Table 2.8 Correlations between N-Value and unconfined compressive strength for different soil types (after Sanglerat, 1972)

Soil types	Unconfined compressive strength, q_u (kPa)
Clay	25 N
Silty clay	20 N
Silty sandy soil	13.33 N

Note: N = Uncorrected SPT N-value

Correlation between C_c and Organic Content (OC)

The relation between C_c and OC was established for reconstituted organic soils at Khulna region of Bangladesh (Islam et al. 2007). Correlations for reconstituted organic soils at Khulna region have been shown in Eq. 2.1 and Eq.2.2.

$$C_c = 0.0054OC + 0.3202, R = 0.8 \text{ (OC: 5 to 25 \%)} \tag{2.1}$$

$$C_c = 0.015OC + 0.0812, R = 0.2 \text{ (OC: 25 to 42 \%)} \tag{2.2}$$

Correlation between C_c and e_o

Correlations between C_c and e_o that were established by different authors have been listed in Table 2.9.

2.4 ORGANIC LAYER IN RECLAIMED AREA

2.4.1 Reasons of Presence of Organic Layer

Filling material is dumped directly upon the marshy low land. Presence of fines in a hydraulic fills means greater compressibility together with greater difficulty in compaction of the fill. Fines also reduce permeability. Due to low permeability, drainage time is long. Therefore, after a certain time, the organic material beneath the previous surface and water is decomposed. Thus, it produces a soft organic clay layer.

Table 2.9 Correlations between C_c and e_o

Correlations	Soil types	Reference
$C_c = 1.15(e_o-0.35)$	Clays	Nishida (1956)
$C_c = 0.44(e_o-0.36)$	Fine-grained soils	Serajuddin and Ahmed (1967)
$C_c = 0.42(e_o-0.34)$	Costal soils	Amin et al. (1987)
$C_c = 0.25(e_o+0.194)$	Organic Clay	Islam et al. (2004)

Note: e_o = Initial void ratio; C_c = Compression index

2.4.2 Problem Due to Organic Layer

Presence of this very soft organic layer beneath filling layer may cause several geotechnical problems. Such as:

a) Bearing capacity problem

b) Excessive settlement for shallow foundation and

c) Negative skin friction for deep foundation.

2.5 ANALYSIS PROCEDURES

2.5.1 Bearing Capacity

The ability of a soil to support a load from a structural foundation without failing in shear is known as its bearing capacity. The stability of foundation depends on:

a) The bearing capacity of the soil beneath the foundation.

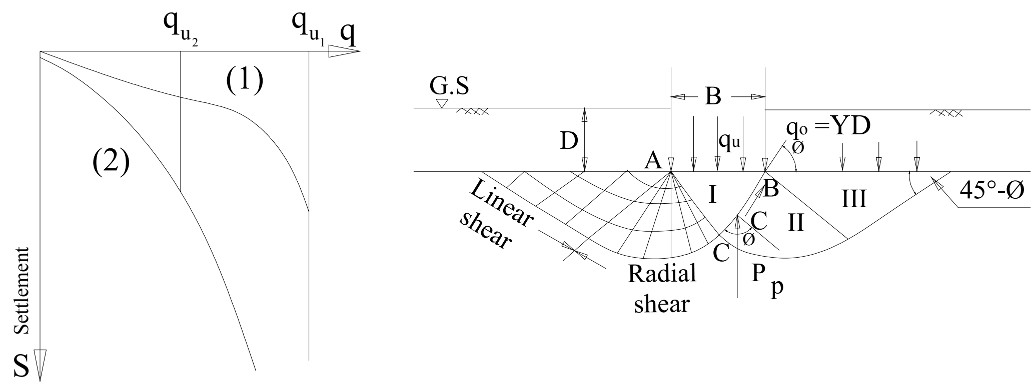
b) The settlement of the soil beneath the foundation.

There are, therefore, two independent stability conditions to be fulfilled since the shearing resistance of the soil provides the bearing capacity and the consolidation properties determine the settlement. The safe bearing capacity is the maximum pressure which the soil can carry safely without risk of shear failure. Theories established by deferent authors are presented below:

Terzaghi's Bearing Capacity Theory

Based on Prandtl's theory (1920) for plastic failure of metal under rigid punches Terzaghi derived a general bearing capacity equation. All soils are covered in this method by two cases which are designated as general shear and local shear failures.

General shear is the case, wherein the loading test curve for the soil under consideration comes to a vertical ultimate condition at relatively small settlement as shown by curve 1 in Figure 2.1(a). Local shear is the case wherein settlements are relatively large and there is not a definite vertical ultimate to the curve as in curve 2 in Figure 2.1(a). Terzaghi's mechanism of failure is shown in Figure 2.1(b).



(a) (b)

Figure 2.1 (a) Load settlement curves [(1) general shear failure and (2) local shear failure]; (b) Terzaghi's mechanism of failure

The following assumptions comes after this analysis. Terzaghi presented the following equations (Eq. 2.3 to Eq. 2.9) of bearing capacity expression for general shear failure:

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma \tag{2.3}$$

$q_u = 1.3\,cN_c + qN_q + 0.3\,\gamma B N_\gamma$, for circular footing where B = dia of the footing (2.4)

$q_u = 1.3\,cN_c + qN_q + 0.4\,\gamma B N_\gamma$, for rectangular footing (2.5)

Here, Q = γD, B = least lateral dimension of footing; Nc, Nq, Nγ = dimensionless beaaring capacity factors and

$$N_c = \cot\varphi \left\{ \frac{a^2}{2\cos^2\left(45 + \frac{\varphi}{2}\right)} - 1.0 \right\} \tag{2.6}$$

$$N_q = \frac{a^2}{2\cos^2\left(45 + \frac{\varphi}{2}\right)} \tag{2.7}$$

$$N_\gamma = \frac{1}{2}\tan\varphi\left(\frac{K_{pr}}{\cos^2\varphi} - 1.0\right) \tag{2.8}$$

here, φ = angle of internal fiction of soil

$$a = \exp\left(\frac{3\pi}{4} - \frac{\varphi}{2}\right)\tan\varphi \tag{2.9}$$

Meyerhof's Bearing Capacity Theory

The bearing capacity of shallow foundations has been derived by Meyerhof (1951) and is expressed as Eq. 2.10:

$$q_u = cN_c + q_o N_q + \frac{1}{2}\gamma B N_\gamma \tag{2.10}$$

Where, N_c, N_q and N_γ are the general bearing capacity factors which depend on foundation depth, shape and roughness and the angle of internal friction.

Skempton's (1951) Bearing Capacity for Clays

Skempton (1951) recommended the following shape and depth factors, and values of N, for surface footing on clays. The ultimate bearing capacity is given (Eq. 2.11 to Eq. 2.14) by:

$$q_u = cN_{cq} d_c S_c \tag{2.11}$$

(i) Surface footings (D = 0)

NC $\approx$ 5 (for strip footing) and 6 (for square or circular footing)

(ii) At depth D

$$d_c = \left(1 + 0.2\frac{D}{B}\right) for \frac{D}{B} \langle 2.50 \tag{2.12}$$

$$d_c = 1.50 \, for \, \frac{D}{B} \rangle 2.50 \tag{2.13}$$

(iii) At any depth, for rectangular footings, BXL

$$S_c = \left(1 + 0.2\frac{B}{L}\right) \tag{2.14}$$

Brinch Hansen's Bearing Capacity Theory

A theory, somewhat similar to the Terzaghi's, has been proposed by Hansen (1961). The ultimate bearing capacity according to this theory is given as Eq. 2.15 to Eq. 2.18.

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma \tag{2.15}$$

Where, S = shape factor; d = depth factor; i = inclination of load factor

$$N_c = \left(N_q - 1\right)\cot\varphi \tag{2.16}$$

$$N_q = \tan^2\left(45 + \frac{\varphi}{2}\right)\exp\left(\pi \tan\varphi\right) \tag{2.17}$$

$$N_\gamma = 1.8\left(N_q - 1\right)\tan\varphi\left(appox.\right) \tag{2.18}$$

2.5.2 Settlement

The total settlement of a foundation can be divided into the following three components:

a) The immediate settlement Δ, that are takes place due to elastic deformation of soil without change in water content.

b) The consolidation settlement ΔH, that are takes place in clayey soil mainly due to the expulsion of the pore water in the soil.

c) Secondary (creep) settlement ΔS, which are takes place over long periods due to viscous resistance of soil under constant compression.

Only consolidation settlements have been discussed in this study.

Settlement Calculation for Normally Consolidated Clay

Total consolidation settlement of normally consolidated clay can be calculated from the value of the compression index of the soil obtained from oedometer tests (Eq. 2.19 and Eq. 2.20).

$$S_c = \frac{\Delta e}{1+e_o} H_t \tag{2.19}$$

and,

$$\Delta e = C_c \log \frac{\sigma'_o + \Delta\sigma}{\sigma'_o} \tag{2.20}$$

where, S_c=total consolidation settlement; Δe=changes of void ratio = e_1-e_0; e_o=initial void ratio; H_t=total consolidating layer; C_c=Compression index; σ_o'=effective overburdened pressure; Δσ=increases of stress due to structural load. Figure 2.2 shows the determination of compression index for normally consolidated clay.

Settlement of Over Consolidated Clay

Total consolidation settlement over consolidated clay can be calculated from the value of the compression index of the soil obtained from oedometer tests (Eq. 2.21 and Eq. 2.23).

$$S_c = \frac{\Delta e}{1+e_o} H_t \tag{2.21}$$

And,

$$\Delta e = C_r \log \frac{\sigma_o' + \Delta\sigma}{\sigma_o'} \text{ (for, } \sigma_o' + \Delta\sigma \leq \sigma_c'\text{)} \tag{2.22}$$

$$\Delta e = C_r \log \frac{\sigma_c'}{\sigma_o'} + C_c \log \frac{\sigma_o' + \Delta e}{\sigma_c'} \text{ (for, } \sigma_o' < \sigma_c' < \sigma_o' + \Delta\sigma\text{)} \tag{2.23}$$

where, S_c = total consolidation settlement; Δe = changes of void ratio = e_1-e_0; e_o = initial void ratio; H_t = total consolidating layer; C_c = compression index; C_r = recompression index; σ_o' = effective overburdened pressure; σ_c' = preconslidation pressure; $\Delta\sigma$ = increases of stress due to structural load.

Determination of preconsolidation pressure can be found out from Figure 2.3. Figure 2.4 shows the determination of compression index and recompression index for over consolidated clay.

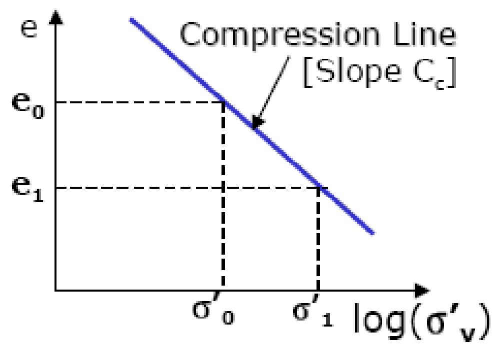


Figure 2.2 Determination of compression index for normally consolidated clay (after Das, 1985)

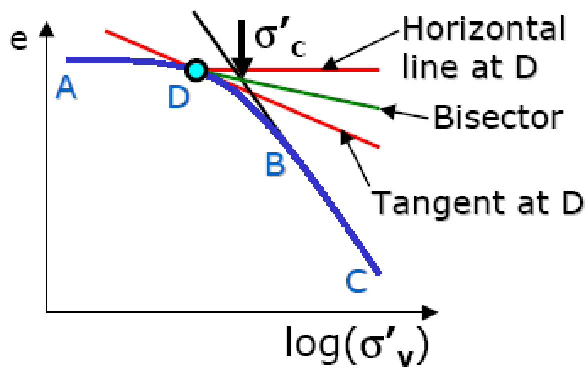


Figure 2.3 Estimation of pre-consolidation pressure from e-logP graph (after Das, 1985)

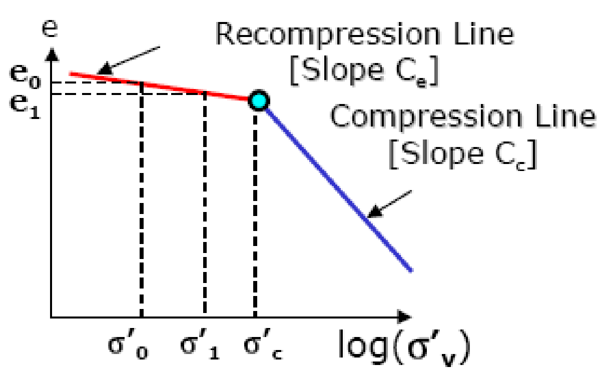


Figure 2.4 Determination of compression index for over consolidated clay (after Das, 1985)

Methods to Eliminate or Reduce Excessive Settlement of Foundation

a) By reducing the load on the soil by removing soil and adopting basement floor (i.e., adopt a floating or compensated foundation).

b) By reducing the load on the soil by using lighter building material like ribbed floors, lightweight wall panels can reduce settlement.

c) Properly designed pile foundation can reduce settlement.

d) Ground improvement technique(s) may be adopted.

e) Extension of the construction can reduce the damage on building.

f) Structure should be well designed so that the different settlement is small.

g) Lateral strain should be prevented by providing by suitable structure like sheet pile walls.

h) Jacking arrangements under columns should be provided well so that the settlements can be adjusted by jacking and proving additional extension to the column to the foundation.

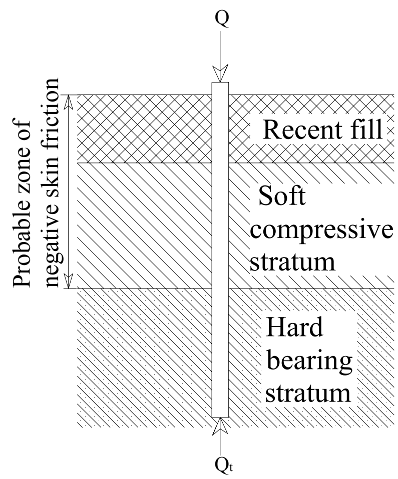
2.5.3 Negative Skin Friction

When the soil surrounding the pile shaft moves downward relative to the pile shaft, the friction develops along the pile shaft is in the downward direction and it is then known as negative skin friction or down drag. This drag acts as a load on pile. Development of Negative skin friction is shown in Figure 2.5. Generally, the following three cases sited under which negative friction may be developed (Varghese, 2005).

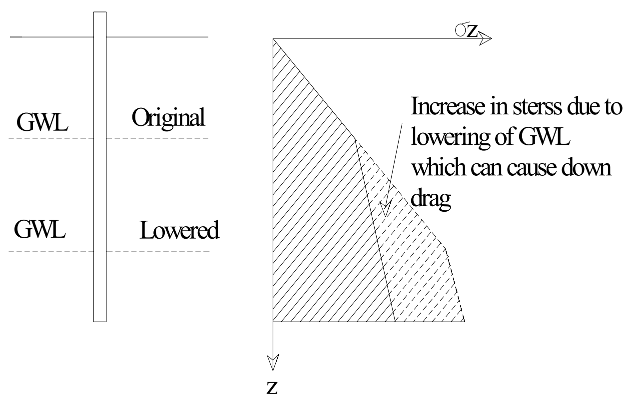
a) Recent fill on soft compressible stratum

b) Lowering of ground water table

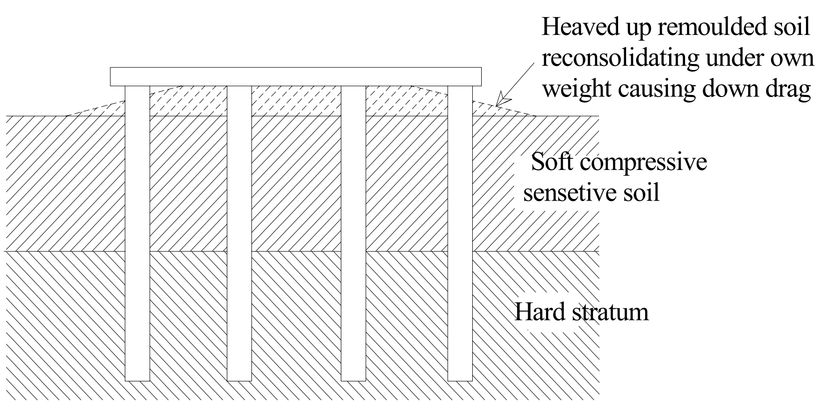
c) Driving of pile through soft compressible soil



(a)



(b)



(c)

Figure 2.5 Three general causes of negative skin friction development: (a) recent fill on soft compressible stratum; (b) lowering of ground water table and (c) driving of pile through soft compressible soil (reproduced after Nayak, 1996)

Estimation of Negative Skin Friction:

(i) Estimation of Negative Skin Friction for Single Pile

Case 1. The pile may rest on a hard stratum without any positive downward movement of the pile but the soil around it settles. Here full negative friction acts (Varghese, 2005) on piles and that have been shown in Eq. 2.24.

$$N_f = cA_p, \tag{2.24}$$

where, N_f=Total negative skin friction; c = Cohesion and A_p = Area of pile shaft.

Case 2. Pile resist on a comparatively compressible strata to that the pile settles to the same extend with the surrounding soil. In this case the settlement of pile is large, the soil around the pile will have to pull the pile up in 'positive friction'. Hence, judgment must be used when estimating these forces (Eq. 2.25 and Eq. 2.26).

$$f_{s\text{-}ve} = \beta\mu_o \tag{2.25}$$

and

$$Q_N = \Sigma(\beta\mu_o)A_{si}, \tag{2.26}$$

where, $f_{s\text{-}ve}$ = negative friction per unit area; μ_o = effective overburdened area.

β = the reduction factor. Mayerhof recommends values depending on length of the piles, 0.3, 0.2, and 0.2 for the length 15, 40 and 60 metres of pile lengths, respectively; Q_N = total negative friction on the pile per unit length; A_{si} = perimeter of pile.

(ii) Estimation Negative Skin Friction for Single Pile and Pile Group

The amount of negative skin friction obtained (Bowels, 1988) according to Eq. 2.27:

$$P_{nf} = \int_0^{L_1} \alpha' p' \bar{q} K \, dz \tag{2.27}$$

Below the neutral point (Figure 2.6), if there is one, positive friction is generated as where P_{nf} = amount of negative skin friction resistance carried by the point. where point-bearing piles are used. General form of $\bar{q}$ is

$$\bar{q} = \overline{q_o} + \gamma'_z \tag{2.28}$$

Where, $\bar{q}$ =Total effective overburdened pressure. In addition, it may be necessary to adjust the integration limits of the soil is stratified to obtained a summation of negative skin contributations.

If we take $\alpha' = \alpha'_2$, and take a flooting pile where $P_{np} \cong 0$ and equate Equations After integration, and for the limits shown, we obtain

$$\alpha' p' \left(\overline{q_o} L_1 + \frac{\gamma' L_1^{\ 2}}{2} \right) k = \alpha' p' \overline{q_o} (L - L_1) K' + \alpha' p' \gamma' \left(L^2 - L_1^{\ 2} \right) \frac{K}{2} \tag{2.29}$$

From which L_1 the distance the neutral point is

$$L_1 = \frac{L}{L_1} \left(\frac{L}{2} + \frac{\overline{q_o}}{\gamma'} \right) - \frac{2\overline{q_0}}{\gamma'} \tag{2.30}$$

which reduces $\bar{q} = 0$ to

$$L_1 = \frac{L}{\sqrt{2}} \tag{2.31}$$

P_{np} = point bearing

When the piles are spaced at small *s/D* ratios, the negative friction force may act effectively on the block perimeter rather than on the individual piles to obtain two modes of stressing requiring investigation.

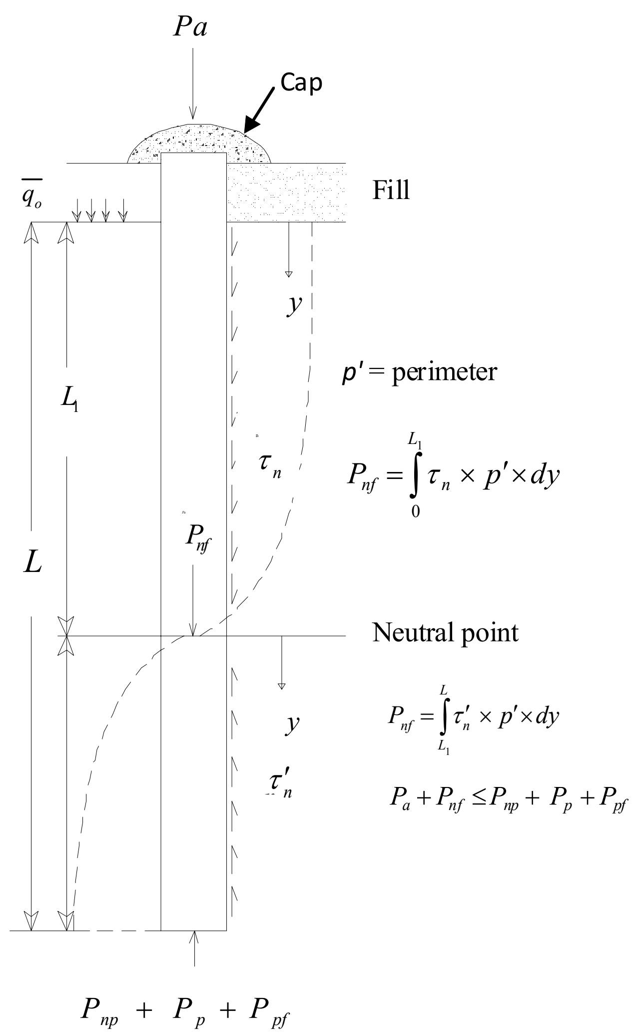


Figure 2.6 Location of neutral point to satisfy the statics of vertical equilibrium with negative skin friction acting on pile (reproduced after Bowels, 1988)

a) Total group negative skin resistance as the sum from the individual piles

$$Q_n = \sum P_{nf} \tag{2.32}$$

b) The block skin resistance based on shear resistance on the block perimeter + weight of block trapped between the piles

$$Q_n = f_s L_f p'_g + \gamma L_f A \tag{2.33}$$

where, γ = unit weight of soil enclosed in the pile group to depth L_f; A =Area of pile group enclosed in the perimeter p'_g; $f_s = \alpha' \overline{q} K$ = effective skin resistance on the group perimeter; p'_g = perimeter of the pile group.

Methods to Eliminate/ Reduce Negative Skin Friction

There are different ways to reduce the negative skin friction of pile (Figure 2.7). These methods summarized below:

a) Coating the surface of the precast pile with thick coat of bituminous paint.

b) In the area of negative skin friction, the space between pile and casing can be filled with viscous material and the casing is withdrawn after installing the pile.

c) Increasing pile length where the shaft resistant pile is used.

d) Decreasing the pile diameter could be improved the situation. In Holland, they have successfully experiment with precast concrete piles with shaft of smaller cross sectional area along its length as compared with the base. The solution is possible with bearing piles only where we do not depend with shaft resistance.

e) Electro-osmosis can be used.

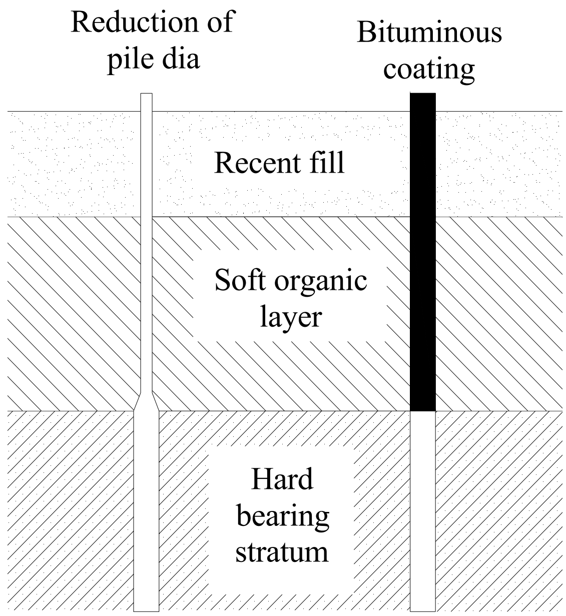


Figure 2.7 Methods of reducing negative skin friction
(reproduced after Varghese, 2005)

2.6 POSSIBLE SOLUTIONS TO MITIGATE PROBLEM DUE TO ORGANIC LAYER

2.6.1 Foundation Alternatives

To mitigate problems due to organic layer, foundation is the most important considerations. The choice of the appropriate type of foundation is governed by some important factors such as:

a) Foundation that can reduced settlement and
b) Foundation that can be reduced negative skin friction.

Foundation types have been discussed in below.

2.6.2 Ground Improvement

The erection of structures on subsiding and derelict ground is becoming a major task in today's civil engineering work. Increasing loads as well as the need to work in areas with soft and incapable soils necessitate the improvement of the actual ground condition. The choice of the appropriate technique has to be made depending on the type of soil, the loads applied and the time available for the improvement process. A rough classification can be undertaken by distinguishing ground improvement solely by compact the existing ground and the improvement by reinforcing the soil with additional material. In the latter case there are techniques with and without certain displacing effects, hence the improvement becomes a combination of compaction and reinforcement. Table 2.11 gives an overview on the existing methods of ground improvement.

Table 2.10 Classification of ground improvement

Type	Technique
Compaction by static methods	- Pre-loading - Pre-loading with consolidation aid - Compaction grouting - Influencing ground water table
Compaction by dynamic methods	- Vibro-compaction - Compaction using vibratory hammers - Dynamic compaction (drop weight) - Compaction by blasting - Air pulse compaction
Reinforcement with displacing effect	- Vibro stone columns - Sand compaction piles - Lime/cement columns installed by displacing methods
Reinforcement without displacing effect	- Mixed-in-place methods - Jet grouting - Permeation grouting - Ground freezing

2.7 FOUNDATION TYPES

2.7.1 Shallow Foundations

Shallow foundations are those executed near the ground surface or at shallow depths. As mentioned before in the previous chapter, shallow foundations are used when sub-soil exploration proves that all soil strata affected by the building could resist the superimposed stresses (Δp) without causing excessive settlements. Shallow foundations are either

a) Footing

- Wall or Strip footing
- Isolated column footing
- Combined Column Footing

b) Raft foundation.

2.7.2 Deep Foundations

Pile Foundations

Piles are structural member that used to transmit surface loads to the lower level in the soil mass (Bowels, 1988). Piles foundations are most commonly used in soft sub-soil condition.

Caissons/ Pier foundations

A cassion/ pier foundation is an underground structural member that serves the same purpose as the footing. Usually, it is used the concrete or masonry for the superstructures of the bridge. The ratio of the depth of the foundation to the base width of the pier is usually greater than 4.

2.8 TYPICAL FOUNDATIONS FOR SOFT SOIL

In this section, the foundation systems which may be employed in the study area are presented. These are:

a) Rammed Aggregate Pier (RAP) foundation
b) Mattress foundation
c) Buoyancy raft foundation
d) Piled-raft foundation
e) Deep pile foundation

2.8.1 Rammed Aggregate Pier (RAP) Foundation

Theoretical Background

Rammed Aggregate Pier (RAP) is used to improve the bearing capacity of peat and organic layer. In this method, very stiff short aggregate piers are installed in cavities. The cavity is filled with crushed stone in a number of layers and being compacted with high-energy impact rammer (not vibration energy). During the densification process, stone chips is pushed laterally into the sidewall of the cavities and the soil surrounded the piers are stressed laterally. These combined actions causes an increase of the confining pressure of the matrix soils, thus providing additional load carrying capacity of the RAP.

The pier are designed and constructed to underlie approximately 35% or more of the footing area of the overlying footing. The load carrying capacity of the RAP depends on the friction angle of the material used in the cavities and the amount of confining pressure affording by the surrounding media. Again, the friction angle will depend on the interlocking properties and the relative density of the aggregate. The friction angle of densified crushed stone was measured as high as 50 degrees or more from full-scale field test (Fox and Cowell, 1998).

The settlement of RAP can be calculated using two-zone method, the upper zone (the aggregate pier-matrix soil Zone) and the underlying lower zone. Figure 2.8 shows the upper zone and lower zone concept. The settlement of the upper zone methods uses a spring analogy and considered the stiff pier acting as a stiff spring, while the less stiff matrix soil acts as a soft spring (Lawton et al, 1994). The settlement can be calculated by the Eq. 2.34.

$$s = \frac{\dfrac{qR_s}{(R_a R_s + 1 - R_a)}}{k_{gp}} \tag{2.34}$$

where, s = Settlement of the upper-zone; q = composite footing bearing pressure; R_s = Stiffness ratio of Geopier element to surrounding soil; R_a = Ratio of geopier area to footing area; k_{gp} = Geopier stiffness modulus.

The settlement component of the lower zone is computed using conventional geotechnical settlement analysis on the assumption that the vertical stress intensity within the lower zone is the same as that of a bare footing without the stiffened upper zone. The combination of the settlements of these two-zone presents the total long-term settlement.

Case Study of Rammed Aggregate Pier (RAP) at Oregon, USA

Foundation systems initially considered are presented in Figure 2.9. Conventional spread footings (Option A), even if designed for low bearing pressures of 48 kPa, were estimated to settle approximately 400 mm, with settlement occurring over a period of several years. Based on experience in the general site location, deep foundations (Option C) were estimated to be on the order of 24 to 30 m long. Because of the soft conditions, limited support capacities, and length of elements, costs of the deep foundations were estimated to be approximately $50 to $70 U.S. per ton of load, approximately three times the typical cost of normal construction. Over-excavation (Option B) was deemed impractical because of shallow groundwater and difficult dewatering conditions. Vibro-replacement Stone Columns were ruled out because of capacity limitations in peat and organic soils. A system of RAP (Option D) was considered and eventually selected for foundation support. This system spreads the load similar to that afforded by option B, while eliminating groundwater problems and providing a lighter surcharge to supporting soils.

Geotechnical Data

Total footprint area of about 11, 150 m^2. The top 1.5 m depth layer was uncontrolled filling layer. The layer between 1.5 and 2.5 m depth was peat. The underlying layer between 2.5 and 4.5 m depth was silt with organic matter. The following layer was firm to stiff silt. The soil profile of the site is shown in Figure 2.10.

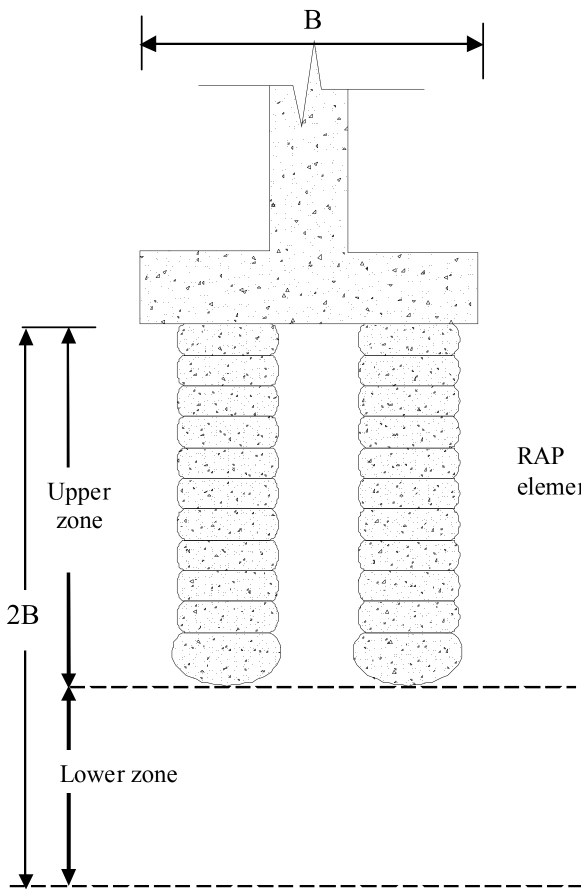


Figure 2.8 Upper zone and lower zone concept of RAP

(reproduced after Farrell et al., 2004)

Foundation System

The aggregate piers were made by drilling 762 mm diameter holes to the depth of 5.0 m from EGL. A small volume of “clean stone” (crushed stone chips without fines, maximum diameter 50 mm) was placed at the bottom of each drilled cavity. This aggregate was then densified with high-energy impact rammer to form the bottom bulb, Figure 2.11. An undulated-sided pier shaft was formed in 300 mm (12 inch) thick lift by using well-graded stone chips that was highly dendified by the ramming action of the penetrated, beveled, tamper head.

Column loads were moderately heavy for this four-storey office building and ranged from 5000 to 40000 kPa. Modulus load tests confirmed capacities of 4000 kPa per pier for the relatively long piers up to 7 m in length. The piers penetrated the fills and soft highly organic silt and peat soils and extended into medium stiff sandy silt and medium dense sandy silt. Allowable composite footing bearing pressure was confirmed at the relatively high value of 336 kPa.

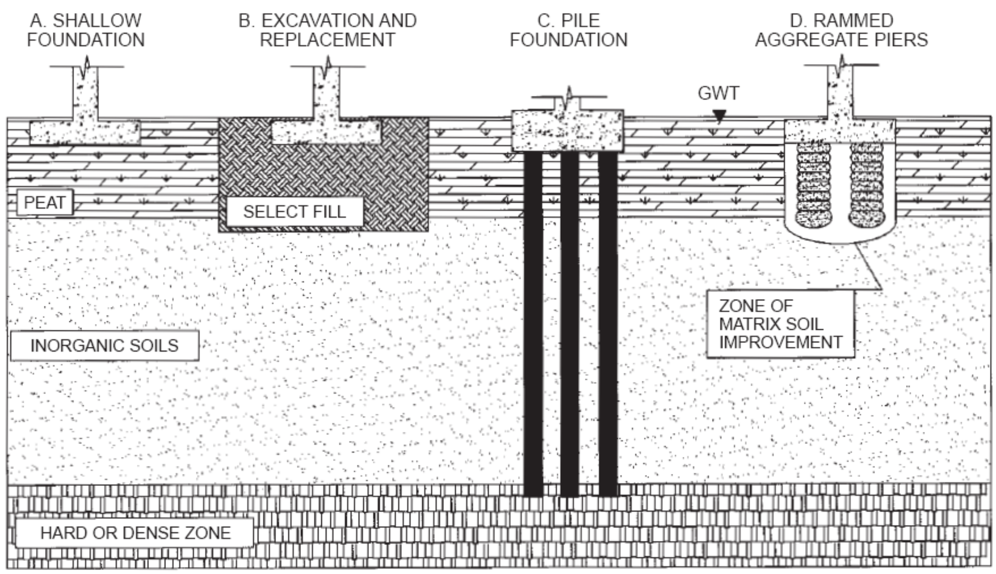


Figure 2.9 Foundation option in peat in United States (after Fox, 2000)

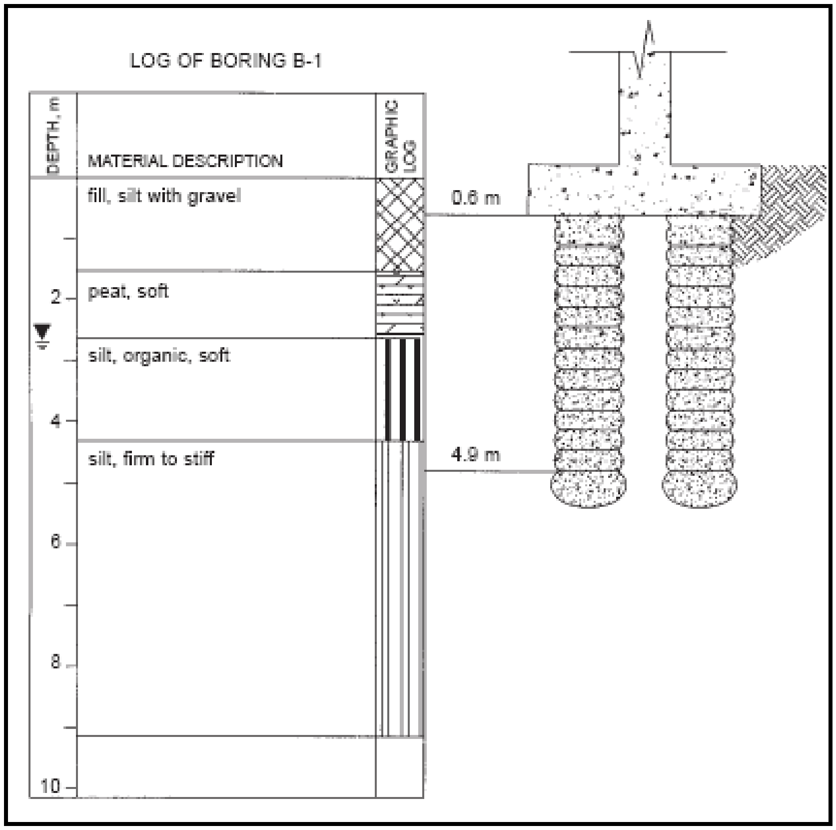
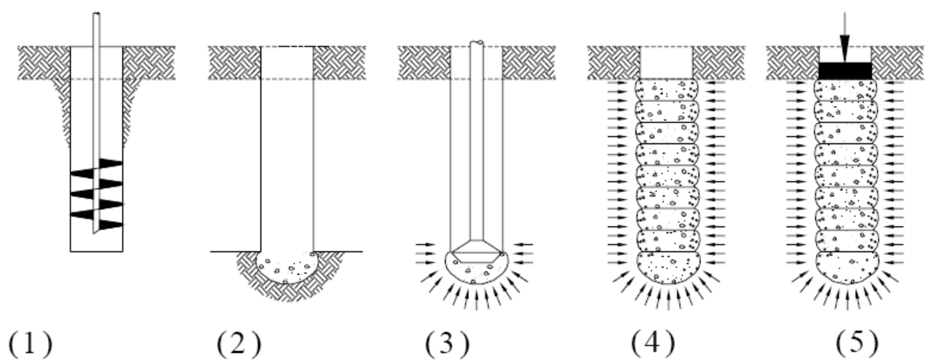


Figure 2.10 Soil profile at site Beaverton, Oregon, USA (after Fox, 2000)



(1) Drill cavity.
(2) Place stone at bottom of cavity.
(3) Ram stone to form bottom bulb.
(4) Densify stone in lifts to form undulated-shaft.
(5) Preload top of the rammed aggregate element.

Figure 2.11 Construction process of rammed aggregate pier (after Fox, 2000)

Performance

The building experienced excellent settlement performance with observed total settlements of less than 20 mm. The modulus load test data indicated that the pier did not bulge appreciably during increased load intensity. The pier provided significant side friction to resist essentially full load up to a stress intensity of about 960 kPa top of pier stress.

2.8.2 Mattress Foundation

A typical spread footing with mattress foundation is shown in Figure 2.12. The function of granular mattress can be described as the following.

a) Increase in bearing capacity by allowing very fast dissipation of pore pressure, especially from the region immediately underneath the geotextile layer.

b) Distribution of stress over a large area.

c) Minimize total and differential settlement.

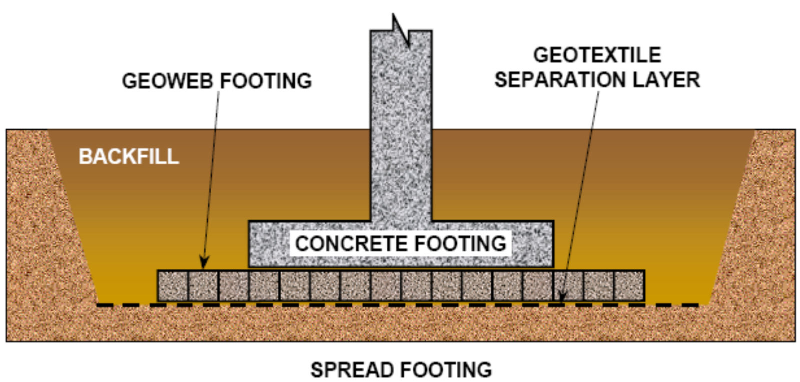


Figure 2.12 Typical spread footing with mattress

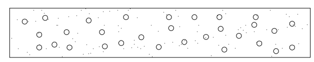
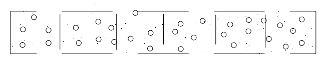
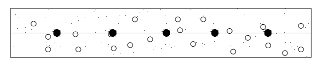
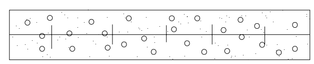
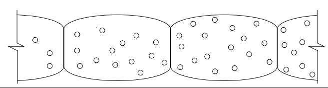
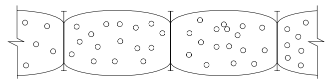
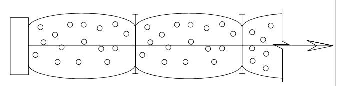
Type	Description	
	Plain	Plain
	Geosynthetic Geocell	Reinforced
	Geosynthetic Reinforced	
	Multi anchored Reinforced	
	Geosynthetic Encapsulated	
	Geosynthetic 1-D prestressed	Prestressed
	Geosynthetic M-D prestressed	

Figure 2.13 Types of granular mattresses (reproduced after Kabir et al., 1992)

Several types of granular mattresses foundations are available on which spread foundation may be placed. These include plain reinforced and prestressed types. Schematic representation of such types has been presented in Figure 2.13. The thickness, material properties, depth of placement, consistency and heterogeneity of the underlying soil layer, spacing of footing, magnitude of applied loading and the stiffness of superstructure will influence the behavior of such foundations (Kabir et al., 1992).

Case Study, Khulna, Bangladesh

Granular mattress used in column and wall footing were designed and constructed for Khulna Medical College building.

Geotechnical Data

A general soil profile and soil characteristics of the site is presented in Figure 2.14. The DPL (Dynamic Probing Light) tests were conducted to ascertain the depth of soft top layer more precisely.

Foundation System

Granular mattress used in column and wall footing were designed and constructed for Khulna Medical College building, Figure 2.15. The aggregate layer consisted of 2 parts crushed brick aggregates and 1 part course sand. The crushed brick aggregate consisted of 25 mm down graded aggregate. The coarse sand used having fineness modulus /2.5.

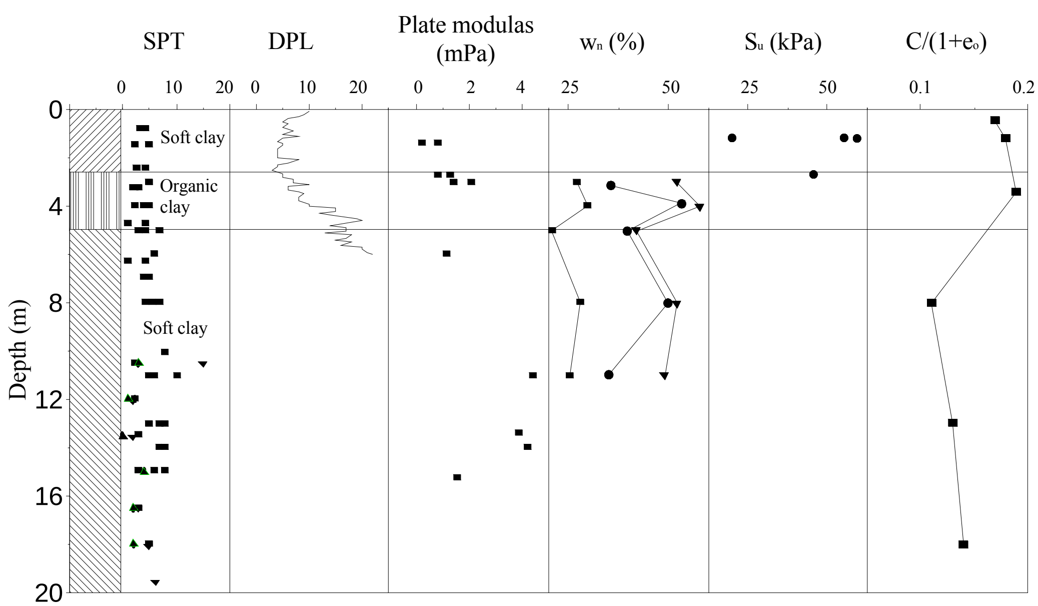


Figure 2.14 Typical borelog at Khulna Medical College
(reproduced after Kabir et al., 1997)

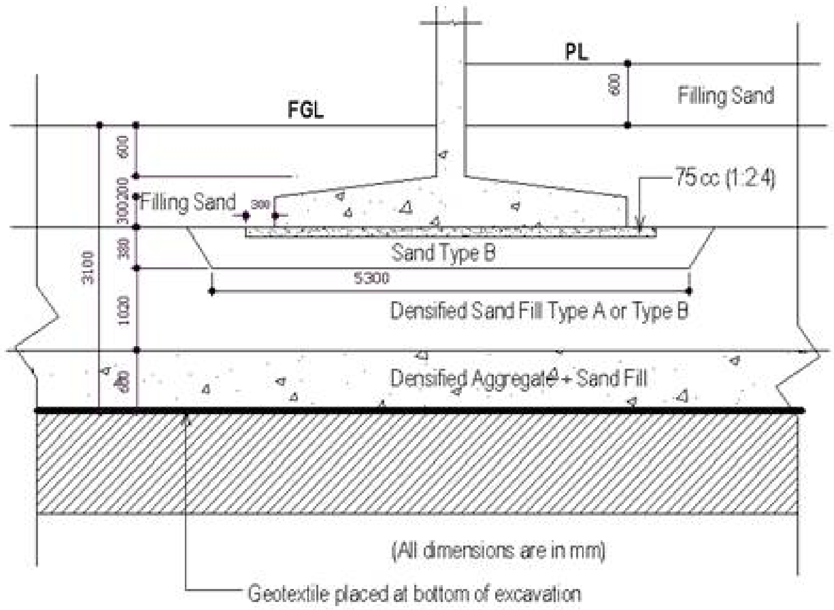


Figure 2.15 Detail of column footing on mattress (after Kabir et al., 1997)

The fill layer consisted local sand having Fineness Modulus (FM) greater than 1.0. Fines passing number 200 sieve was limited to 5 % for FM up to 1.5 and 10 % for FM greater than 1.5. The sand and aggregate were densified in layers by using twin steel drum vibratory rollers. The densities of the materials were monitored by TRL dynamic penetrometer.

A geotextile separator and filter layer was placed at the bottom of the excavation of the soft clay layer. A nonwoven needle punched geotextile was used. The weight, grab tensile strength and permeability were greater than or equal to 200 gsm, 750 N and 1 x 10^{-3} m/s, respectively.

Performance

The reduction of differential settlement with the increase in confining pressure is more profound for higher value of thickness ratio (H/B = 1.5) than the lower value (H/B = 1.0) (Kabir et al., 1992).

2.8.3 Floating/ Buoyancy Raft Foundation

Theoretical Background

The buoyancy raft works on a similar principle to that of a floating structure where the supporting pressure for the raft is obtained by displacing the weight of earth or overburden by the volume of a large voided foundation. It is designed so that sufficient overburden is removed to allow the superstructure load to be applied to the ground with little or no increase in the original stress, which existed on the sub-strata prior to excavation and construction

Floating foundations are also known by other names such as "Deep cellular raft", "Buoyancy rafts", "Compensated foundation". In theory floating foundations are fully compensated foundations in which an amount is excavated from below the foundation of soil is equal to the weight of the building and a basement raft on floating raft is provided to the structures. The weight of excavated earth is fully compensates the weight of the building (Varghese, 2005).

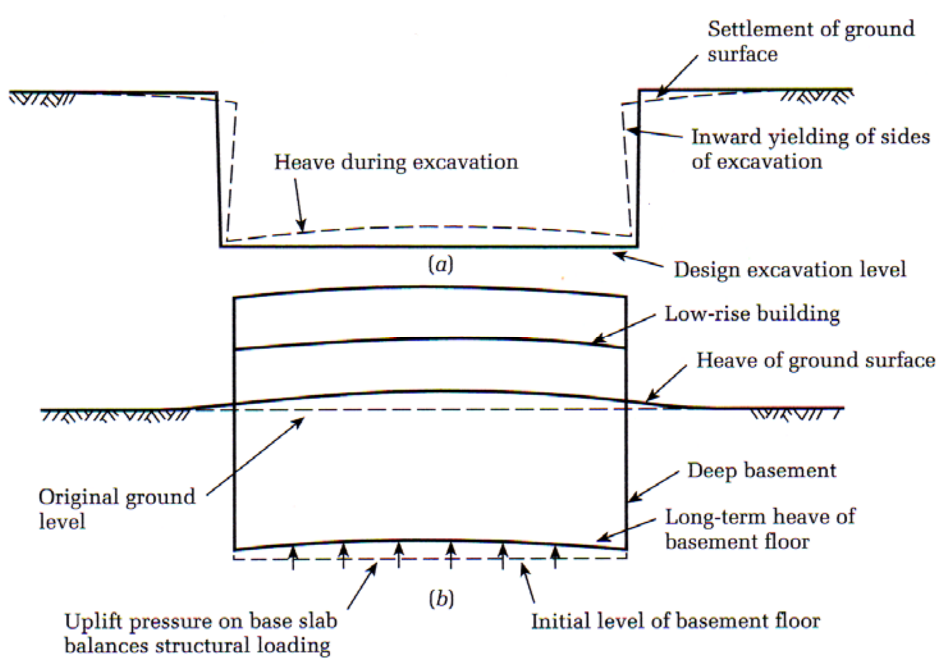


Figure 2.16 Movements around a deep basement supporting a low-rise structure (a) on completion of excavation (b) final movements of completed structure (Tomlinson, 2001)

Where the soil is predominantly cohesive, the reduction in ground stress will result in heave i.e., settlement in reverse. As with settlement, heave will have short-term (elastic) component and long-term (consolidation) component. The short-term heave normally occurs during the excavation period. Where its magnitude is considered significant, the formation is then trimmed down to its required level. The long-term heave is dealt with in the same way as long-term settlement. The anticipated amount of differential heave is calculated, and the structure is designed to accommodate this movement. The heave of the foundation has been shown in Figure 2.16.

The bottom of slab may be the basement of the proposed building and combined with the ground slab and retaining walls. The raft design considers eccentricity of loads and aims to keep differential settlements and tilting within the acceptable limits. The net average applied pressure on soil is shown in Eq. 2.35.

$$q = \frac{Q}{A} - \gamma D_f \tag{2.35}$$

The factor of safety against bearing capacity failure for partially compensated foundations may be given as Eq. 2.36 (Das, 1990).

$$FS = \frac{q_{u(net)}}{q} = \frac{q_{u(net)}}{\frac{Q}{A} - \gamma D_f} \tag{2.36}$$

where, D_f =the depth of embedment; Q=total structural load; A=surface area of raft foundation; γ=unit weight of soil; D_f=depth of raft; FS=factor of safety.

Case Study of Buoyancy Raft Foundation System, Bangladesh

Buoyancy raft foundation system for a five-storey hospital building was analyzed and designed by the Design Division-3, Public Works Department (PWD).

Soil profile

A typical soil profile of Goalkhali area is shown in Figure 2.17. The upper 1.52 m depth layer is very soft clay. The layer between 1.52 and 3.05 m is soft clay with organic matter. The underlying layer between 3.05 and 12.19 m depth is silty clay. The layer between 12.19 and 18.29 m is clay. The uncorrected SPT N-value is very low and it varies from 2 to 5 up to the depth of 18.29 m from EGL. The unconfined compressive strength, q_u varies from 11.53 to 66.33 kPa. The initial void ration and compression index in the silty clay layer varies from 0.89 to 1.0 and 0.32 to 0.48, respectively.

Foundation

The foundation system is shown in Figure 2.18. The foundation base was placed below the organic layer. As the sub-soil strata below the organic layer is very soft clay with high void ratio and high ground water table, there was possibility of heaving of soil after excavation. To minimize heaving, compacted sand with crushed aggregate was used below the mat. The detail analysis and calculation of heaving at the bottom of excavation was not available. No settlement plates were inserted to measure the settlement at periodical intervals.

Performance

Another, buoyancy raft foundation was analysed and designed by Shahedullah and Associates Limited, Dhaka at Gollahmari (Khulna University area) for 4 storey.

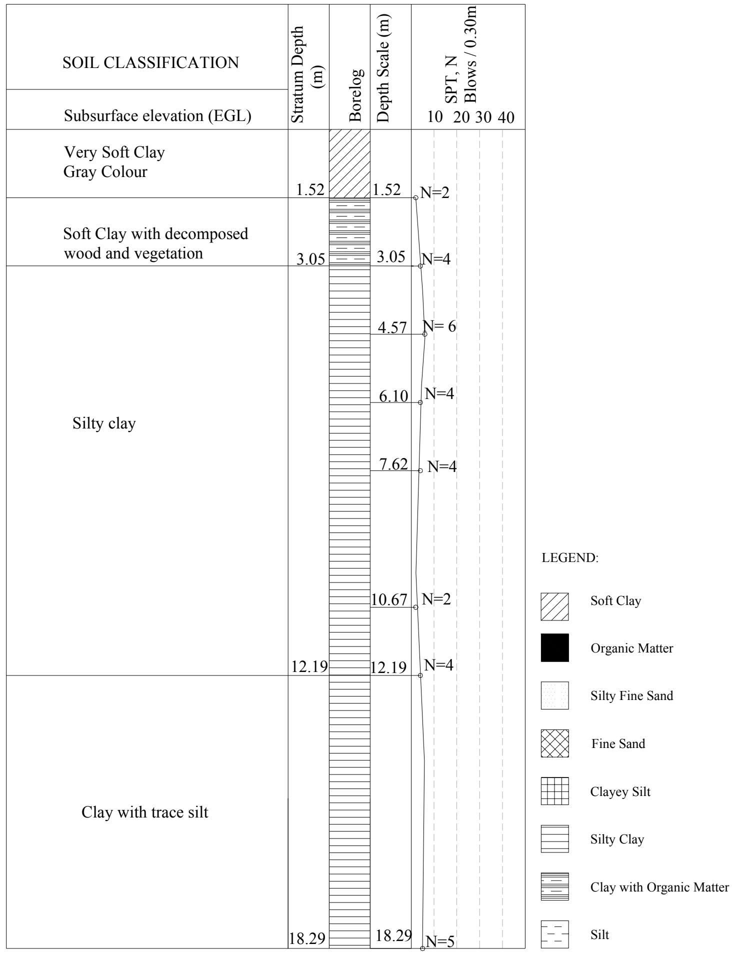


Figure 2.17 Typical sub-soil properties in at Goalkhali, Khulna (after PWD, 2000)

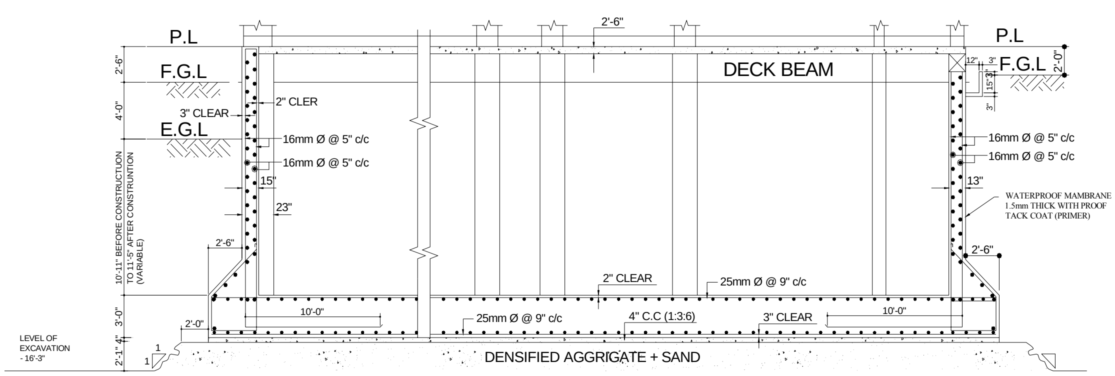


Figure 2.18 Buoyancy raft foundation system at Goalkhali for five-storey hospital building (after PWD, 2000)

Academic Building-II. Razzaque and Alamgir (1999) reported that the recorded settlement was as negligible as 20 mm only.

2.8.4 Piled Raft Foundation

Theoretical Background

The load on a footing or raft affects the foundation soil for a depth only approximately equal to 1 to 2 times its breadth. A pile foundation transfers load to deeper layers. A piled raft foundation comprises both piles and a pile cap that itself transmit load directly to the ground. The aim of such a foundation is to reduce the number of piles compared with a more conventional piled foundation. A concept of settlement reducing piles is shown in Figure 2.19. The piled raft is two types:

a) Piled raft for settlement reduction
b) Piled raft for load transfer

a) Piled raft for settlement reduction: When the raft is safe from bearing capacity considerations but it suffers from excessive settlement, a few numbers of piles under the raft is used to relieve the raft of a part of the total load. As the piles do not have to take all the loads, the number of piles require will be much smaller than the traditional piled foundation. Because of some relief of the load, the raft settlement will also fall within allowable limits (Varghese, 2005).

b) Piled raft for load transfer: These types of piled rafts are the conventional types which are used in situations where the sub-soil is very weak with high water level and raft have to be adopted. These rafts should resists the bouncy forces from the ground water and must transmit all the net loads from the structures to the piles to be carried to deeper and stronger layers to the foundations (Varghese, 2005).

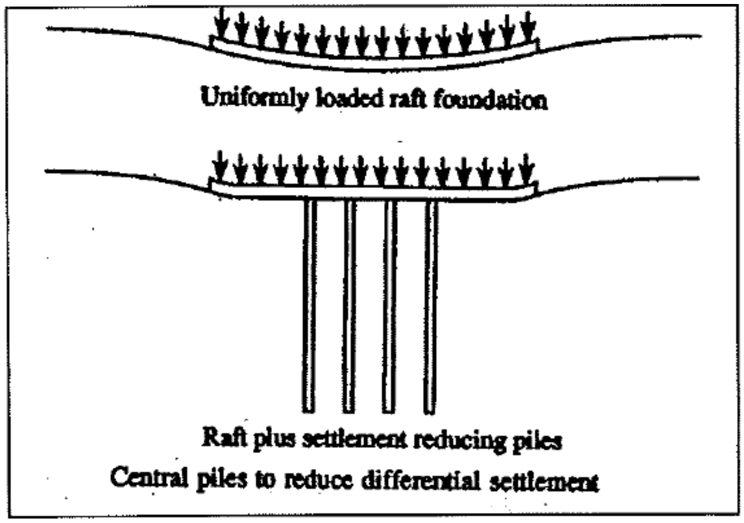


Figure 2.19 Concept of settlement reducing piles (after Randolph, 1994)

Poulos-Davis-Randolph (PDR) Method

The ultimate capacity of the piled raft foundation can be taken as the lesser of the two values:

a) The sum of the ultimate capacities of the raft plus all the piles

b) The ultimate capacity of a block containing the piles and the raft, plus that of the portion of the raft outside the periphery of the piles (Impe, 2001)

An approach to combining the separate stiffness of the raft and the pile group has been suggested by Randolph (1983).

$$k_f = \frac{k_p + k_c(1 - 2\alpha_{cp})}{1 - \dfrac{\alpha_{cp}^2 k_c}{k_p}} \tag{2.37}$$

where, α_{cp} = average interaction factor between the pile and pile cap, pile cap stiffness; k_P = pile group stiffness; k_f = overall foundation stiffness.

The overall foundation stiffness (k_f) approach is based on the Eq. 2.37. Proportion of load carried by the pile cap (P_c) and the pile group (P_p) is given in Eq.2.38(Fleming et al., 1992).

$$\frac{p_c}{\left(p_c + p_p\right)} = \frac{k_c(1-\alpha_{cp})}{k_p + k_c(1-2\alpha_{cp})} \qquad (2.38)$$

The stiffness, k_p, of the pile group (load divided by settlement) may be expressed as a fraction η_ω of the sum of the individual pile stiffness, k. Thus for a group of n piles, $k_p = \eta_\omega nk$ while the group efficiency can be expressed as $\eta_\varphi = n^{-e}$. The exponent e lies between 0.40 and 0.60 for most pile groups. The actual e will depend on:

Pile slenderness ratio, ℓ/d,

Pile stiffness ratio, $\lambda = E_p / G_l$

Pile spacing ratio, s/d,

Homogeneity of soil characterized by ρ

and Poisson's ratio, ν.

The average stiffness of the pile cap may be estimated (Poulos and Davis, 1974) as follows (Eq. 2.39).

$$k_c = \frac{2G}{I(1-\nu)}\sqrt{B \times L} \qquad (2.39)$$

Where, Shear modulus, $\frac{G}{c_u} = 200$, for London clay (Simpson et al., 1979); I=Influence factor (Poulos and Devis, 1974) ; B=Breadth of the raft; L= Length of the raft

Optimization, analysis and design of piled- raft foundation can be done through PLAXIS-3D or GARP6 computer programmes. Behavior of piled raft foundation depends on the number, length, diameter, disposition of piles, raft thickness and geo-technical properties of the site (Cunha et al., 2001).

Case Study of Piled Raft Foundation, Malaysia

Geotechnical Data

The alluvial deposits at the site consisted of very soft to firm silty clay up to a depth of 25 to 30 m with presence of intermediate sandy layers. The silty clay stratum was underlaing by silty sand. Klang Clay could be divided into two distinct layers at depth of 15 m.

Foundation System

Both temporary surcharging and preloading technique was adopted to control long-term settlement of the sub-soil. Then the building was placed on top of it. The net fill height at the site was about 0.5 to 1.0 m. The temporary surcharging heights remained in the range of 2 to 5 m. After the sub-soil had achieved the required percentage of settlement and verified using Asaoka's method (Asaoka, 1978), the temporary earth fills were removed and the construction of the foundation begins.

The loadings of 5-storey apartments were highest at the columns and ranges from 100 to 750 kN. The line load from the brick wall was 9 kN/m (4.5" brick wall) and the uniform live load acting on the ground floor raft was 2.7 kN/m2 (1.5 kN/m2 live load +1.2 kN/m2 floor finishing) as per recommended values given by British Standard 6399 (1996). The main design criterion was to limit the relative rotation (angular distortion) to 1/350 (Skempton and MacDonald, 1956) to prevent cracking in walls and partitions.

The objective of the design was to provide an optimum piled raft foundation system that takes into consideration the bearing capacity contribution of the raft and the piles introduced mainly to limit differential settlement. The approach was to increase the stiffness of areas where the settlement is expected to be the largest by introducing settlement-reducing piles. Horikoshi and Randolph (1998) suggested that for uniformly loaded raft, piles distributed over the central 16-25% of the raft area is sufficient to produce an optimum design and for piled raft subjected to non-uniform vertical loads, the use of piles with varying length would give the most optimum design (Reul and Randolph, 2004). The foundation system adopted for low cost apartments consists of 200 x 200 mm reinforced concrete square piles with pile length varying from 18m to 24m interconnected with 350 x 700 mm strips and 300 mm thick raft. Figure 2.20 shows typical section of the strip-raft foundation system.

The locations of the strips were adjusted during detailed design to ensure they pass beneath all the columns (i.e. concentrated loads) for optimum structural design. Two cases were considered in the detailed analysis of the foundation system, i.e.:

(a) Case 1: Overall settlement behaviour
(b) Case 2: Pile-soil-structure interaction

Case 1 considered the overall settlement behaviour of the piled raft foundation system in order to predict the settlement profile of the structural design. The settlement analysis was carried out based on Terzaghi's 1-D consolidation theory. Approximate adjustments were made to the pressure imposed on the sub-soil due to distribution of the super structure load by the piles using concept of equivalent raft. The settlement profiles were then used to determine the spring stiffness or Winkler's modulus to generate the overall stress on the foundation raft due to the settlement profile.

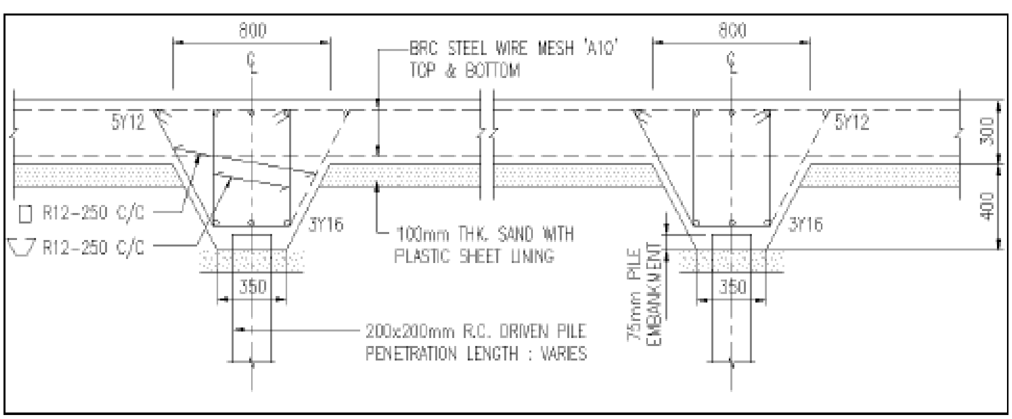


Figure 2.20 Typical section of the strip-raft foundation system (after Tan et al., 2004)

Case 2 considered the interaction between the pile-soil-structure (foundation raft) of the foundation system in order to determine the load distribution and local settlement of the piles. The pile-soil-structure interaction can be carried out iteratively using elastic pile interaction software (e.g. PIGLET/PIGEON) together with finite element structural analysis software (e.g. SAFE) until convergence of results was achieved (typically 6 10%). The iterative approach was proposed due to limitations of available software in modelling pile-soil-structure interaction. The analysis could be carried out using 3-dimensional finite element method (FEM) software (e.g., PLAXIS 3-D Foundation) that could model 3-dimensional pile-soil-structure interaction. The solution for pile interaction proposed by Randolph and Worth (1979) was based on the solution for single pile and extended for pile groups based on the principle of superposition. For cases with different pile lengths, the interaction of pile bases at different levels was very complicated and its effects to shear stress along the pile shaft were unknown. However, for the current application in soft ground, the pile capacity was derived primarily from shaft/skin friction with very little end bearing contribution.

Performance

Settlement monitoring works was carried out when the building has been completed for more than six months. The monitoring results showed that the maximum differential settlement recorded was 27.02 mm. The relative maximum local angular distortion recorded is 1/1215. The monitoring results also showed the building experiences marginal tilt of approximately 1/1000. However, the value was well within the limits of 1/250 to 1/500 (Charles and Skinner, 2004) for it to be noticeable.

2.8.5 Deep Pile Foundation

Theoretical Background

Pile foundation may be used to transmit the super structure load to the firm strata. The total capacity of a pile is due to both end and side resistance. Where the soil layer at greater depth consists of dense layer, the cast-in-situ pile may be expected to have better load resistance than driven piles.

The ultimate carrying capacity Q_u of a pile may be estimated by Eq. 2.40, Eq. 2.41 and Eq. 2.42, respectively.

$$Q_u = Q_p + Q_s \tag{2.40}$$

where, Q_p = load carrying capacity of the pile point; Q_p = Frictional resistance.

$$Q_p = A_p Q_p = A_p\,(c'N_c + q'N_q) \tag{2.41}$$

where, A_p = area of the pile tip; Q_p = Unit point resistance; c' = Cohesion of the pile; q' = Effective vertical stress at the level of pile tip; N_c, N_q = Bearing capacity factor.

$$Q_s = \sum P \Delta L f \tag{2.42}$$

where, P = perimeter of the pile section; ΔL = Incremental pile length over which P and f are taken to be constant; f = Unit friction resistance at depth z.

Unit friction resistance, f (for cohesionless) soil can be calculated from the Eq. 2.43:

$$f = K\sigma_o{}' \tan\delta \tag{2.43}$$

where, K=effective earth pressure; $\sigma_o{}'$=Effective vertical stress; δ=Soil pile friction angle; φ=Angle of internal friction.

Table 2.11 Average value of K for different pile type

Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\varphi$
Low−displacement driven	$\approx K_o = 1.4(1 - \sin\varphi)$
High−displacement driven	$\approx K_o = 1.8(1 - \sin\varphi)$

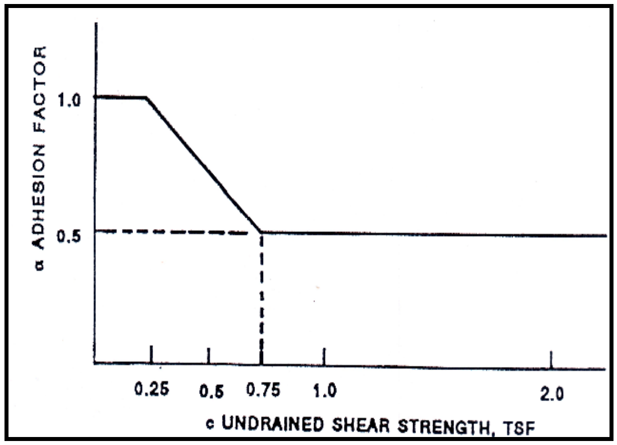


Figure 2.21 Values of α versus undrained shear strength (after US Army Corps of Engineers, 1994)

In reality, the magnitude of K varies with depth; it is approximately equal to the Rankine passive earth pressure, k_p, at the top of the pile and may be less than the at-rest pressure coefficient, k_o, at a greater depth. Table 2.12 shows the average value of K.

Unit friction resistance, f (for cohesive soil) soil can be calculated from the Eq. 2.44.

$$f = \alpha c_u \tag{2.44}$$

where, α = Empirical adhesion factor. The approximate variation of the value of α is shown in Figure 2.21.

Case Study of Deep Pile Foundation

Geotechnical Data

The upper layer between 4.5 and 6.0 m depth is soft organic clay. The layer between 6.0 and 13.75 m depth is silty clay. The layer between 13.75 and 21.50 m depth is fine sand. The SPT-N value of organic clay, silty clay and fine sand layers was found in the range of 2 to 3, 4 to 6 and 22 to 48, respectively. Typical borelog is shown in Figure 2.22. The unconfined compressive strength of organic clay and silty clay was found in the range of 18 to 90 kPa and 12.0 to 29 kPa, respectively. Cohesion(c) and angle of internal friction (φ) of fine sand layer vary from 5.75 to 7.5 kPa and 29.10 to 32.60, respectively.

Foundation System

Four-storied residential building was designed by Public Works Department (PWD). The foundation system was precast reinforced concrete pile, the effective length of pile was 45'-0" and the dimension of this pile was 12" x 12", Figure 2.23. The allowable bearing capacity of was 50 kip and the pile is end bearing.

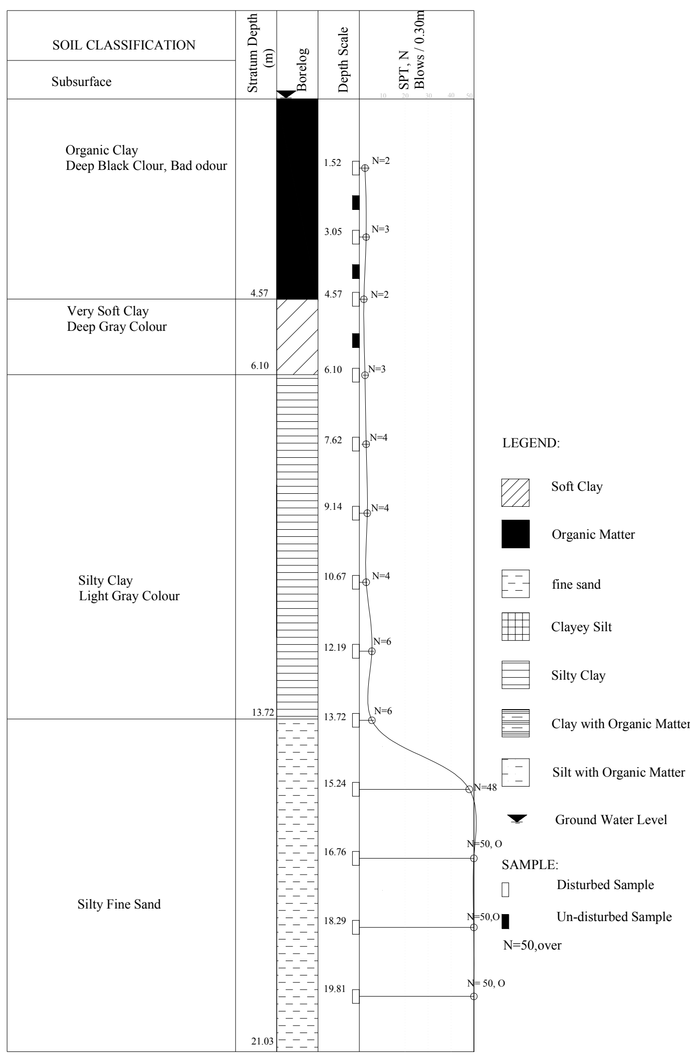


Figure 2.22 Typical borelog at Sonadanga area (PWD, 2006)

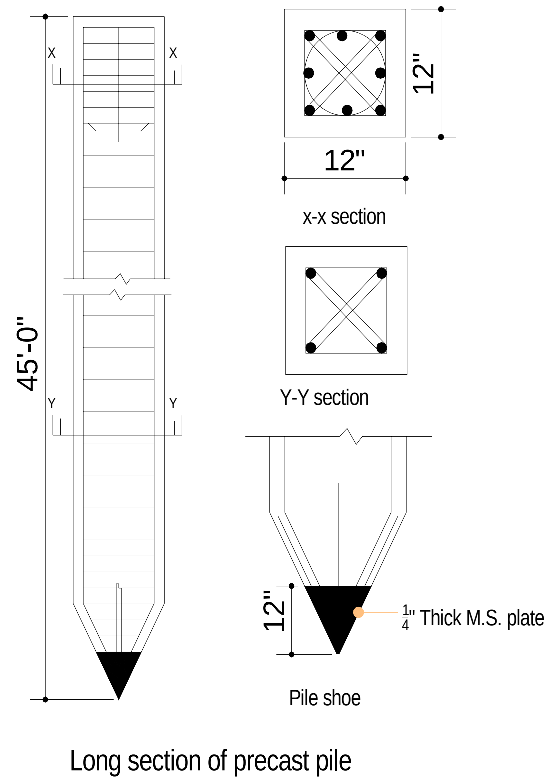


Figure 2.23 Longitudinal section of precast concrete pile (after PWD, 2006)

Except these foundation systems, some ground improvement technique may be adopted for avoid structural damage due to soft organic layer. The ground improvement techniques are Preloading without vertical drain, Preloading with vertical drain, grouting etc.

2.9 SUMMARY

Due to rapid urbanization, many areas are being reclaimed in and around Dhaka city by dredge filling. This filling material is directly dumped on marshy low land.

Some studies have been carried out to understand the characteristics of silty sand layer of reclaimed area (Ahamed, 2005 and Hossain, 2009). These studies mainly focused on the liquefaction problem/potential of such areas. However, properties and foundations problems in similar soils of Khulna city had been investigated in such study (Ferdous, 2007 and Khan and Ferdous, 2004).

In reclaimed areas, filling material is dumped directly upon the marshy low land. Therefore, the organic material beneath the previous surface and water is decomposed. Thus, it produces a soft organic clay layer. Presence of this very soft organic layer beneath filling layer may cause several geotechnical problems (i. e., excessive settlement, negative skin friction etc.). Ahamed (2005) and Hossain (2009) were worked on reclaimed soil whereas; their main focused was on filling soil and liquefaction. Due to poor geological data and lack of study of organic layer in reclaimed area, it felts necessary to study on filling layer overlaying soft organic layer.

Some typical case studies of different types of foundation system have been presented which may be employed in the study area. These are Rammed Aggregate Pier (RAP) foundation, mattress foundation, buoyancy raft foundation, piled-raft foundation and deep pile foundation.

Chapter THREE
EXPERIMENTAL PROGRAM

3.1 GENERAL

The objective of this chapter is to describe the experimental programs that have been carried out to conduct the research. Here, the locations of the selected area of this research have been shown. Then, all the field and laboratory test procedure has been discussed. During field investigation, disturbed and undisturbed samples were collected. Laboratory tests were performed at the geotechnical laboratory of Civil Engineering Department of Bangladesh University of Engineering and Technology (BUET). Detail experimental program has been presented in this chapter.

3.2 SELECTED STUDY AREAS

Total six areas of the Dhaka city have been selected for this research as shown in Fig. 3.1 and Table 3.1. The main targeted areas are reclaimed lands since some of these lands were found very soft organic layer just beneath the filling layer (Ahmed, 2005 and Hossain, 2009).

All these six areas are given code name as shown in Table 3.1 and these code names are used subsequently in this thesis paper instead of their original name.

Table 3.1 Selected study areas of Dhaka city

Sl. No.	Locations	CODE
1	Mirpur DOHS	S-1
2	Banasree	S-2
3	Pink city	S-3
4	Hatir Jheel	S-4
5	Uttara	S-5
6	Nikunja-1	S-6

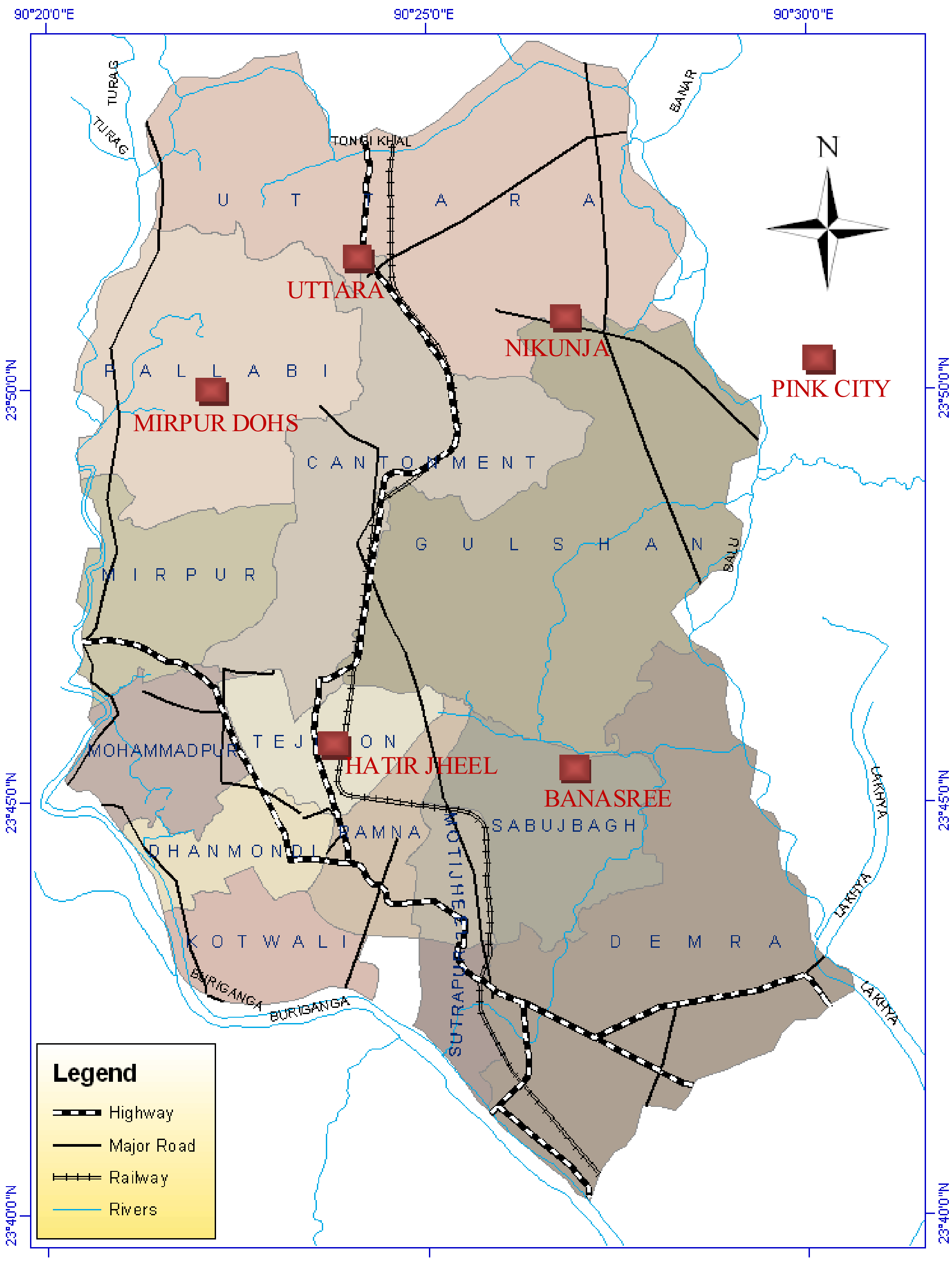


Figure 3.1 Location of selected study areas on Dhaka city map

These areas are Mirpur DOHS, Banasree, Pink city, Hatir Jheel, Uttara and Nikunja-1. Fig. 3.1 shows the location map of these six selected areas in Dhaka city.

3.3 TEST PROGRAM

At all the six locations, both field investigation and laboratory investigations have been carried out for this study. Standard Penetration Test (SPT) has been conducted and disturbed and undisturbed soil samples have been collected. These soil samples have been tested in the BUET geotechnical laboratory. The procedures followed during these field and laboratory tests are described below in brief.

3.3.1 Field Investigations

Standard Penetration Test (SPT)

This test is probably the most widely used field test. It has the advantages of simplicity, the availability of a wide variety of correlations for its data and the fact that a sample is obtainable with each test. The main objectives of SPT are as follows:

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

Standard split barrel sampler is advanced into the soil by dropping a 140-pound (63.5-kilogram) safety or automatic hammer on the drill rod from a height of 30 inches (760 mm). The sampler is advanced a total of 18 inches (450 mm). The number of blows required to advance the sampler for each of three 6-inch (150 mm) increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (blows per foot {300 mm}). Tests shall be performed in accordance with ASTM D 1586. Table 3.2 shows the recommended SPT procedure. During design, the N-values may need to be corrected for overburden pressure. Correlation factor for those conditions are listed in

Table 3.3. The type of hammer (safety or automatic) shall be noted on the boring logs, since this will affect the actual input driving energy. A method to measure the energy during the SPT has been developed (ASTM D 4633). Since there is a wide variability of performance in SPT hammers, this method is useful to evaluate an individual hammer's performance. The SPT installation procedure is similar to pile driving because it is governed by stress wave propagation. As a result, if force and velocity measurements are obtained during a test, the energy transmitted can be determined. Once this is known, the N-values from that SPT can be modified to a standard N_{60} using the following equation:

$$N_{60}\, E_{60} = N_{field}\, E_{measured}$$

where,

E_{60} = 60% of the theoretical potential energy (210 ft-pounds {285 N-m})

N_{field} = field observed N-value

$E_{measured}$ = measured energy

Table 3.2 Recommended SPT procedure (ASTM D1586)

Equipments	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 15 m N or NW for greater depths
Sampler	Standard O.D. 51 mm +/- 1 mm, I.D. 35 mm +/- 1 mm and length > 457 mm
Penetration resistance	Record number of blows for each 150 mm; N = number of blows from 150 to 450 mm penetration
Blow count rate	30 to 40 blows per minute

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

3.3.2 Geotechnical Laboratory Investigations

After conducting field investigation of soil samples, all samples have been taken into BUET geotechnical laboratory for geotechnical laboratory investigation.

Disturbed samples have been used in order to identify the index properties, i.e. grain size analysis, specific gravity (G_s) and organic matter content. Besides this, moisture content (w_n), Atterberg's limit tests have been performed on disturbed soil samples.

Unconfined compressive strength tests have been performed on undisturbed organic soil samples to determine of shear strength parameters, i.e., cohesion and ϕ.

For the determination of the consolidation parameters such as compression index (C_c), coefficient of consolidation (c_v), coefficient of volume compressibility (m_v) etc., one-dimensional consolidation tests have been preformed on undisturbed organic soil samples. Permeability properties of organic soils were also calculated from the one-dimensional consolidation tests results.

The relevant ASTM, BS and AASHTO designations of the laboratory tests performed in the present study are shown in Table 3.4. Table 3.5 shows the list of sample collected during SPT test. It this table all the boreholes are given serial no and these serial no are used subsequently in Table 3.6 and table 3.7. Summary of laboratory test have been shown in Table 3.6 and Table 3.7.

Table 3.4 Name of the laboratory tests and the standards followed

Sl. No.	Name of experiment	Test Standard
1.	Standard Test Method for Particle–Size Analysis of Soil	ASTM D 422-63 (Reapproved 1998)
2.	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	ASTM D 2488-84
3.	Determination of Organic Content in Soils by Loss on Ignition	AASHTO Designation: T 267-86 (1993)
4.	**Standard Test Method for Specific Gravity of Soils (Inorganic Soils)**	ASTM D 854-98
5.	**Determination of the Specific Gravity of Soil Particles (Organic Matter)**	BS 1377:1975
6.	Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils	ASTM D – 4318-86
7.	Standard Test Method for Unconfined Compressive Strength Test of Cohesive Soil	ASTM D – 2166-86
8.	Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	ASTM D– 4186-89 (Reapproved 1988)

Table 3.5 List of samples collected during SPT test

Borehole Locations	BH No.	BH Sl. No. (For Table 3.6 and 3.7)	RL of EGL (m)	Sample depth (m)		
				Disturbed	Undisturbed	
					UD-1	UD-2
S-1	01	01	1.22	21.05	5.5	7.0
	02	02	1.22	21.05	4.0	-
S-2	01	03	0.30	19.55	5.5	7.0
	02	04	0.30	19.55	5.5	7.0
	03	05	0.30	19.55	5.5	7.0
S-3	01	06	0.90	18.05	5.5	7.0
	02	07	0.90	18.05	5.5	7.0
	03	08	0.90	18.05	5.0	5.5
	04	09	0.90	18.05	5.0	5.5
S-4	03	10	1.07	16.55	4.0	5.5
	09	11	1.37	16.55	4.0	5.5
	10	12	3.81	12.05	2.0	4.0
	15	13	1.37	16.55	4.0	-
	16	14	5.18	12.05	4.0	-
S-5	01	15	0.40	21.50	2.0	-
	02	16	0.30	21.50	2.0	-
	03	17	0.40	21.50	4.0	-
	04	18	0.30	21.50	2.0	-
	05	19	0.36	21.50	4.0	-
S-6	01	20	0.20	18.60	2.0	-
	02	21	0.20	18.60	2.0	-
	03	22	0.20	18.60	2.0	-

EGL= (-) Existing Ground Level

Table 3.6 Summary of the laboratory test of disturbed samples

Depth (m)	BH Sl. No.	Name of laboratory test
1.5	1, 2, 12, 15, 16	Sieve & Hydrometer Analysis
3.0	3, 4, 5, 6, 8	Sieve & Hydrometer Analysis
	10, 14	Organic content test
	12, 14	Atterberg's limit test
4.5	1, 3, 7	Sieve & Hydrometer Analysis
	2	Organic content test
6.0	1, 2, 4,14	Sieve & Hydrometer Analysis
	6, 8, 22	Atterberg's limit test
7.5	2, 14, 20, 21	Sieve & Hydrometer Analysis
9.0	2	Sieve & Hydrometer Analysis
	1	Organic content test
	12	Atterberg's limit test
12.0	2, 16	Sieve & Hydrometer Analysis
13.5	20	Sieve & Hydrometer Analysis
15.0	1, 17, 19, 21	Sieve & Hydrometer Analysis
16.5	2, 15, 17	Sieve & Hydrometer Analysis
18.0	1, 19, 22	Sieve & Hydrometer Analysis
21.0	2, 18	Sieve & Hydrometer Analysis

Table 3.7 Summary of the laboratory test of undisturbed samples

Depth (m)	BH Sl. No.	Name of laboratory test
2.0	12, 15, 22	Specific Gravity, Consolidation test, Unconfined compression test
4.0	2, 10, 11, 13, 14,19	Specific Gravity, Atterberg Limit, Consolidation, Unconfined Compression and Grain Size Analysis
5.0	8, 9	Specific Gravity, Atterberg Limit, Consolidation, Unconfined compression and Grain Size Analysis
5.5	1, 3, 4, 5, 6, 7	Specific Gravity, Atterberg Limit, Consolidation, Unconfined compression and Grain Size Analysis
7.0	1, 3, 4, 5, 6, 7	Specific Gravity, Atterberg Limit, Consolidation, Unconfined compression and Grain Size Analysis

Chapter FOUR

SUB-SOIL CHARACTERISTICS

4.1 GENERAL

The main objective of this chapter is to present detail results of the sub-soil investigations. Field and laboratory test results obtained for different locations has been presented in this Chapter. In order to identify sub-soil characteristics, both disturbed and undisturbed samples have been collected using wash boring technique from selected reclaimed areas of Dhaka city. Specific gravity, natural moisture content, grain size distribution, organic matter content, liquid limit (LL) and plasticity index (PI) of the samples have been determined. Unconfined compression tests and one-dimensional consolidation tests have been conducted on undisturbed samples in special attention to very soft organic layer. This chapter presents the sub-soil characteristics of the selected reclaimed areas of Dhaka city.

4.2 SUB-SOIL CHARACTERISTICS

Total six locations are selected for this research (Fig. 3.1, Chapter 3). Standard Penetration Test (SPT) has been conducted and disturbed as well as undisturbed samples have been collected during drilling from all these locations. The depth of drilling varies from 18.0 to 30.0 m. Sub-soil characteristics i.e., SPT and laboratory test results of these samples are presented in this section.

4.2.1 Sub-soil Characteristics of Mirpur DOHS (S-1)

Mirpur DOHS is situated at north-west of Dhaka city. It is a Defense officer’s housing project where land areas were developed by dredge fill material. Fig. 4.1 shows the location map of the study area S-1. In the study area S-1, two boreholes were drilled at plot 332 (BH-1 and BH-2). In addition, data of twelve borehles of plot 332(BH-3 to BH-5), plot 333(BH-6 to BH-10) and plot 349(BH-11 to BH-14) has been collected from different sources (Ahmed, 2005 and Hossain, 2009).

Figure 4.1 Location map of the study area S-1

Soil profiles of these fourteen boreholes are shown in Fig. 4.2(a), Fig. 4.2(b) and Fig. 4.2(c). The general soil profile of this area is filling sand overlaying clay or organic clay layer. Filling sand overlaying soft organic clay layer has been found at all the borehole locations of plot 332 and at one borehole location of plot 349. In the other plot, no such layer has been found. Detail test results of these boreholes (where soft layer has been found) are described and presented below.

SPT Results

SPT has been conducted in the area following the procedure described in ASTM D1586. The soil profiles of the boreholes of the plots 332, 333 and 349 have been presented in Fig. 4.2(a), Fig. 4.2(b) and Fig. 4.2 (c). The variations of SPT N-values with depth of these boreholes have been presents in Fig. 4.3.

The general soil profile of this area is filling sand overlying soft clay or organic clay layer. The depth of filling sand varies from 2 to 6 m from Existing Ground Level (EGL). Soft organic clay layer exists in all borehole locations of plot 332 and one borehole location of plot 349. Depth of this organic layer varies from 2.0 to 12.0 m. Thickness of the organic layer varies from 2.0 to 6.0 m.

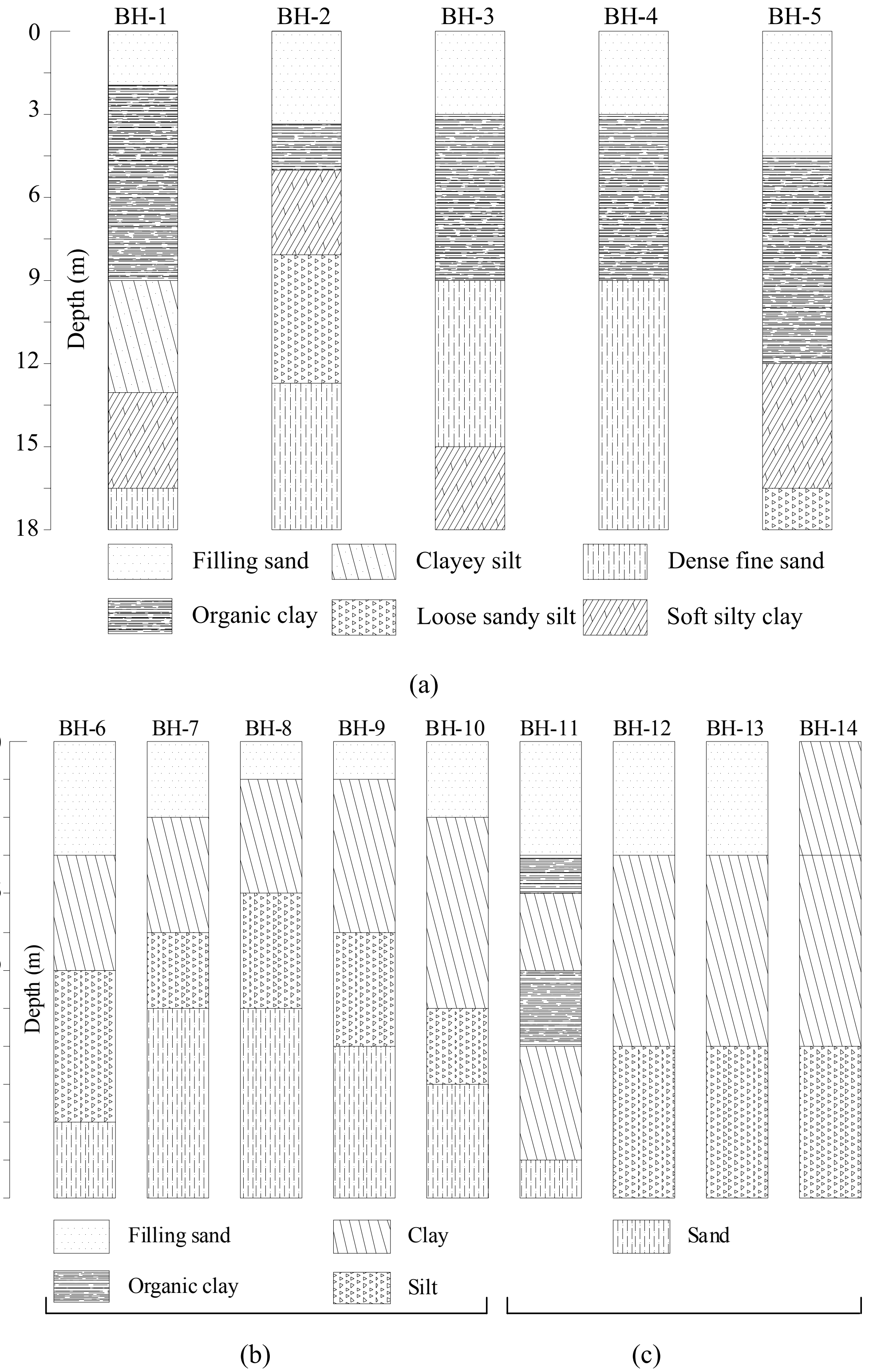


Figure 4.2 Soil profile of the boreholes of the study area S-1: (a) plot 332 (BH-1 to BH-5; filling sand overlaying organic clay); (b) plot 333 (BH-6 to BH-10; filling sand overlaying clay) and (c) plot 349 (BH-11 to BH-14)

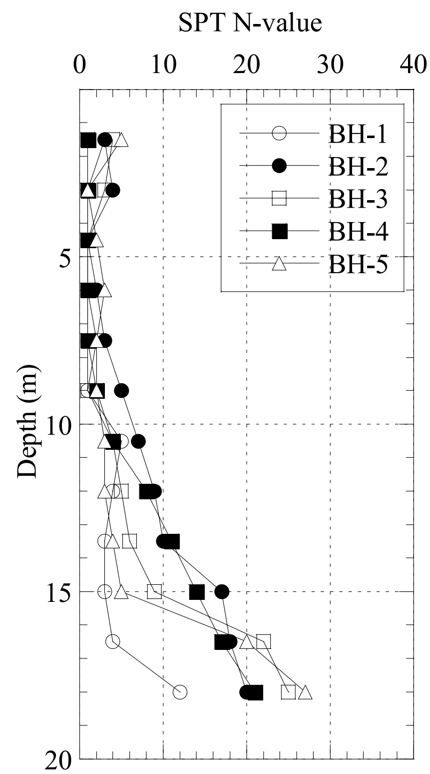


Figure 4.3 Depth vs. SPT N-Value of the study area S-1

In all borehole locations of the plot 333 and two borehole locations of plot 349, a grey silty clay layer exists just below this filling layer. This clay layer varies from 3.0 to 16.5 m from EGL. In three borehole locations of plot 332 and all boreholes locations of plot 333 and 349, silt layer have been found beneath this clay layer. This silt layer varies from 7.5 to 18.0 m from EGL. Below this silt layer, dense sand layer has been found at 9.0 to 18.0 m from EGL at various locations.

Fig. 4.3 presents the variations of SPT N-values with depth (from EGL) of the five borehole locations (BH-1 to BH-5) that contains filling soil overlaying organic clay. The SPT N-value of the filling sand varies from 1 to 5. The SPT N-value of organic clay layer varies from 1 to 3. It is seen that this organic clay is very soft. The SPT N-value of clay layers varies widely from 2 to 25. The SPT N-value of silt layer varies from 6 to 24. The SPT N-value of the dense sand layer varies from 3 to 35.

Among these fourteen boreholes, analyses are not conducted at eight borehole locations in details since it is found that these locations do not contain organic clay. At five boreholes in plot 332 and one borehole in plot 349 filling sand have been found overlaying soft organic clay. Sub-soil characteristics of these five boreholes of plot 332(BH-1 to BH-5) are described below.

Physical and index properties

Typical grain size distributions of sands of various depth of S-1 are shown in Fig. 4.4. Specific gravity (G_s) of filling soil is 2.65 whereas specific gravity of organic clay varies from 2.29 to 2.36. Results of index properties of the sand and clay samples have been presented in Tables 4.1 and Table 4.2, respectively.

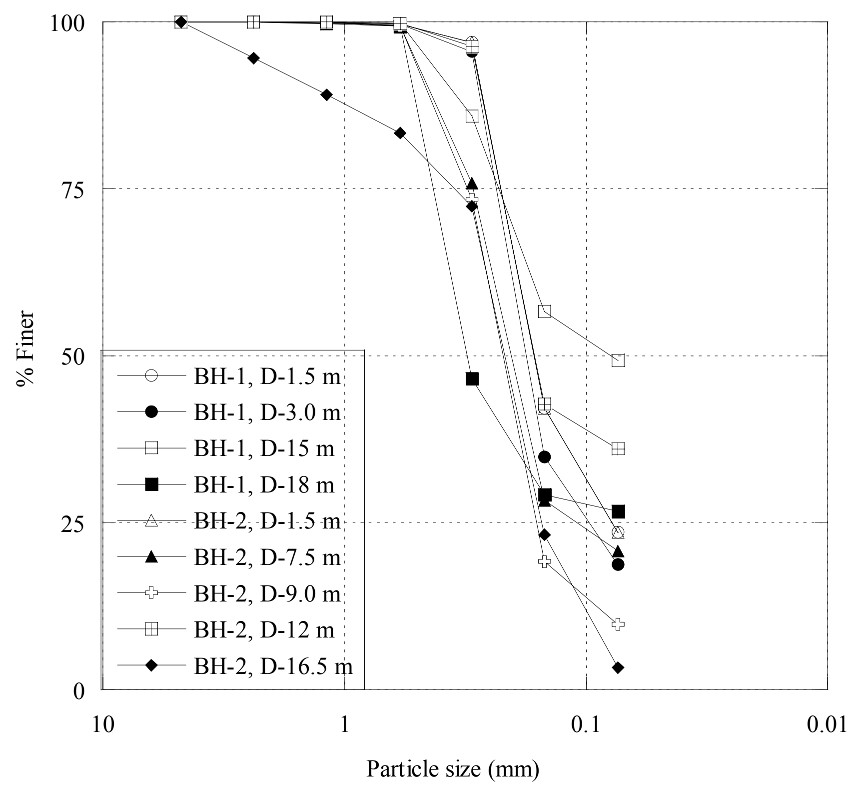


Figure 4.4 Typical grain size distributions of the sand samples collected from the study area S-1

Table 4.1 Summary of physical properties of sand samples collected from the study area S-1

BH No./Sample No./Depth (m)	D_{50} (mm)	F_c (%)
BH-1/D-1/1.5	0.180	23.6
BH-1/D-3/4.5	0.010	100.0
BH-1/D-4/6.0	0.011	100.0
BH-1/D-10/15.0	0.080	49.3
BH-1/D-12/18.0	0.300	26.7
BH-2/D-1/1.5	0.180	23.6
BH-2/D-4/6.0	0.009	100.0
BH-2/D-5/7.5	0.210	20.8
BH-2/D-6/9.0	0.230	9.8
BH-2/D-18/12.0	0.180	36.1
BH-2/D-11/16.5	0.220	3.3
BH-2/D-14/21.0	0.170	38.1
BH-3/D-2/3.0	0.180	24.0
BH-3/D-3/4.5	0.002	94.0
BH-3/D-7/10.5	0.085	35.0
BH-3/D-11/16.5	0.001	99.0
BH-4/D-2/3.0	0.170	19.0
BH-4/D-3/4.5	0.002	92.0
BH-4/D-7/10.5	0.090	32.0
BH-5/D-2/3.0	0.170	12.0
BH-5/D-7/10.5	0.002	96.0
BH-5/D-11/16.5	0.001	100.0

Note: D_{50} = Mean grain size; F_c = Fines content

Table 4.2 Summary of physical properties of clay samples collected from the study area S-1

BH No./Sample No./Depth(m)	W_n (%)	e_0	γ_d (kN/m^3)	LL (%)	PL (%)	PI (%)
BH-1/UD-1/5.0	60.0	1.50~3.70	4.90~9.30	144	99	45
BH-2/UD-1/4.0	29.0	2.85~3.88	4.60~5.80	190	128	62
BH-2/UD-2/6.0	81.0~82.0	2.02	7.40	–	–	–

Note: w_n= Natural moisture content; e_o = Initial void ratio; γ_d = Dry density; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index.

Table 4.3 Organic matter content of the samples collected from the study area S-1

BH No./Sample No./Depth (m)	Soil description	Organic content (%)
BH-1/D-6/9.0	Black clay	29.4
BH-2/D-3/4.5	Black clay	13.2

The mean grain size (D_{50}) and fines content (F_c) of filling sand vary from 0.170 to 0.180 mm and 12 to 24%, respectively. The mean grain size (D_{50}) and fines content (F_c) of organic clay vary from 0.002 to 0.011 mm and 92 to 100%, respectively. The mean grain size (D_{50}) and fines content (F_c) of dense sand vary from 0.080 to 0.300 mm and 3.3 to 49.3%, respectively.

Atterberg's limits test results have been shown in Table 4.2. It has been found that top filling layer is non plastic sand. Liquid limit (LL) and plasticity index (PI) of organic clays vary from 144 to 190% and 45 to 62%, respectively which are highly plastic. LL and PI of grey clay layer vary 40 to 55% and 13 to 29%, respectively.

Organic content of the sample from black very soft clay layer has been determined using loss of ignition method. Organic content of the samples are presented in Table 4.3. It is seen that organic content of the soils varies from 13.2 to 29.4%. This soil may be classified as ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils (Table 2.1, chapter 2).

Shear strength properties

Unconfined compression tests have been conducted on organic clay samples collected from the BH-1 and BH-2. It was observed that organic samples were very soft and sample preparation was very difficult. Special care was taken to trim the samples.

Detail test results of samples and axial stress versus axial strain graphs are presented in Table 4.4 and Fig. 4.5, respectively. Dry unit weight and moisture content of the samples collected from organic layer vary between 4.6 to 4.9 kN/m^3 and 29 to 60%, respectively. It is seen that dry unit weight is low comparison to inorganic clay.

Undrained shear strength of the organic samples vary between and 8 to 25 kPa. Based on BS 5930, it has been found that, consistency of this soil is very soft (Bowels, 1982). It is seen that undrained shear strength of the clay samples of the area S-1 is less than that of Dhaka clay (s_u = 82.5~201.5 kPa; Hossain, 2009). In addition, it is similar with the soft organic Dhaka clay (s_u = 13~19 kPa; Islam et al., 2004). Failure strain varies from 13 to 15%.

Table 4.4 Summary of unconfined compression test results of the clay samples collected from the study area S-1

BH No./Sample No./Depth (m)	W_n (%)	γ_{dry} (kN/m^3)	q_u (kPa)	s_u (kPa)	ε_f (%)	Consistency (BS 5930)
BH-1/UD-1/6.5	60	4.90	16	8	15	Very soft
BH-2/UD-1/4.0	29	4.60	50	25	13	Very soft

Note: w_n = Natural moisture content; q_u = Unconfined compressive strength; s_u = Undrained shear strength; ε_f = Failure strain.

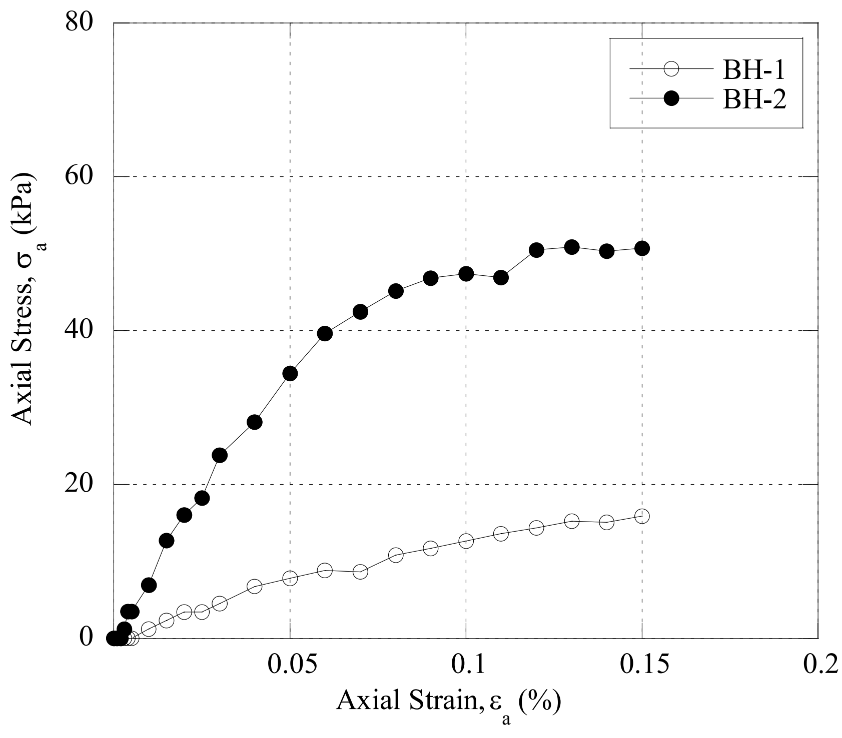


Figure 4.5 Axial stress vs. axial strain graph of organic clay samples collected from the study area S-1

Compressibility properties

One-dimensional consolidation tests have been conducted on undisturbed soil samples collected from organic layers of BH-1 and BH-2 of the study area S-1. Typical e-logP curves are presented in Fig. 4.6. From the unloading part of the e-logP curves, it is seen that in most of the cases, the rebound is very small in comparison to the inorganic clay soils. It means that the elastic rebound in such cases is very low and the plastic deformation is high. It means that this type of soil will undergo for large displacement/settlement.

Table 4.5 presents the one-dimensional consolidation test results. It is found that initial void ratio, compression index and recompression index of soft organic soil vary from 1.50 to 3.88, 0.44 to 1.25 and 0.13 to 0.44, respectively. It is seen that initial void ratio (e_o) of study area S-1 is less than that of Khulna soil (e_o varies from 3.11~6.15 for Khulna soil).

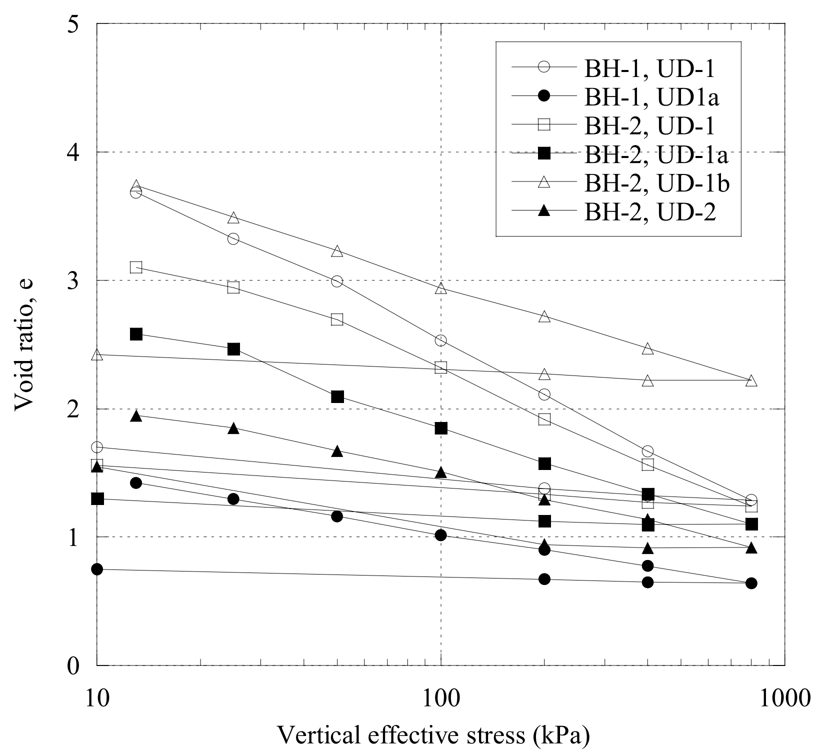


Figure 4.6 Typical e-logP curves of organic samples collected from the study area S-1

Table 4.5 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-1

BH No./Sample No./Depth (m)	e_o	C_c	C_r	c_v (m^2/yr)	m_v (kN/m^2*10^3)
BH-1/UD-1/5.5	1.50~3.70	0.44~1.25	0.16~0.44	0.34~3.86	0.13~6.13
BH-2/UD-1/4.0	2.85~3.88	0.77~1.07	0.29~0.32	0.30~5.08	0.13~4.05
BH-2/UD-2/6.0	2.02	0.73	0.13	0.25~4.32	0.18~2.36

Note: e_o = Initial void ratio; γ_d = Dry density; C_c = Compression index; C_r = Recompression index; c_v = Coefficient of consolidation; m_v= Coefficient of volume compressibility.

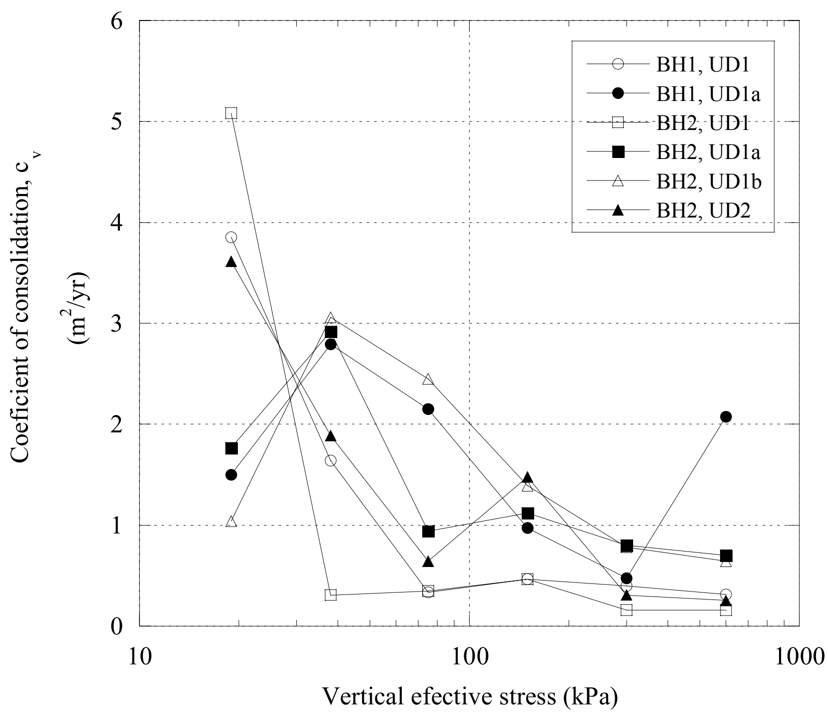


Figure 4.7 Coefficient of consolidation (c_v) vs. vertical effective stress of the samples collected from area S-1

Results and graphs of coefficient of consolidation (c_v) has been presented in the Table 4.5 and Fig. 4.7, respectively. From the table, it is seen that coefficient of consolidation (c_v) widely varies from 0.25 to 5.08 m^2/yr. Fig. 4.7 shows that the values of c_v is variable with effective stress, whereas this variation does not follow any particular trend line.

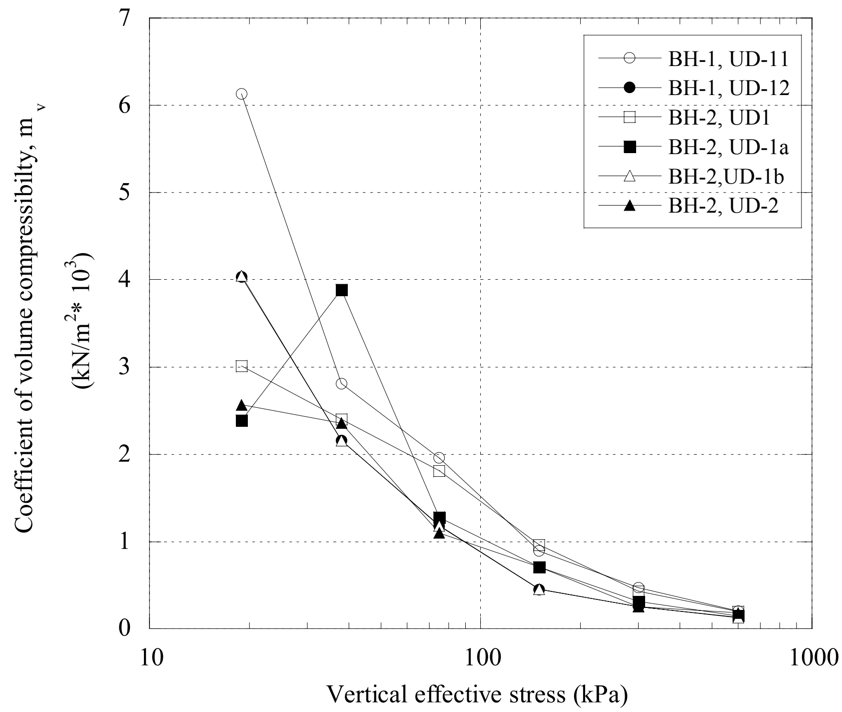


Figure 4.8 Coefficient of volume compressibility vs. vertical effective stress of the samples collected from area S-1

Test results and graph of coefficient of volume compressibility (m_v) has been presented in Table 4.5 and Fig. 4.8, respectively. From the table, it is seen that m_v varies from 0.13×10^{-3} to 6.13×10^{-3} kN/m^2. From the Fig. 4.8, variations of m_v with effective vertical stress have been seen. In addition, except one data of BH-2, almost every cases m_v decreases with the increases of vertical effective stress.

Permeability properties

Coefficient of permeability (k) of the soil samples has been estimated from one-dimensional consolidation results. A summary of the permeability values of the organic clay samples are presented in Table 4.6. From this table, it is seen that values of k vary between 9.48×10^{-12} and 1.31×10^{-9} m/sec. It indicates that permeability is very low, which are typical to soft clay behavior. It has also been found that permeability is less than the permeability of Khulna organic soil (permeability of Khulna soil varies from 5.89×10^{-9}~8.59×10^{-9} m/sec; Ferdous, 2007). Fig. 4.9 presents the relationship between k and vertical effective stress of the study area S-1. From the Fig. 4.9, it is seen that k decreases with the increases of vertical effective stress except two cases (BH-2).

Table 4.6 Summary coefficient of permeability of organic samples collected from study area S-1

BH No./ Sample No./ Depth (m)	k (m/sec)
BH-1/UD-1/5.5	8.46×10^{-11} to 1.88×10^{-09}
BH-2/UD-1/4.0	9.48×10^{-12} to 1.31×10^{-09}
BH-2/UD-2/6.0	1.45×10^{-11} to 2.89×10^{-09}

Note: k = Coefficient of permeability

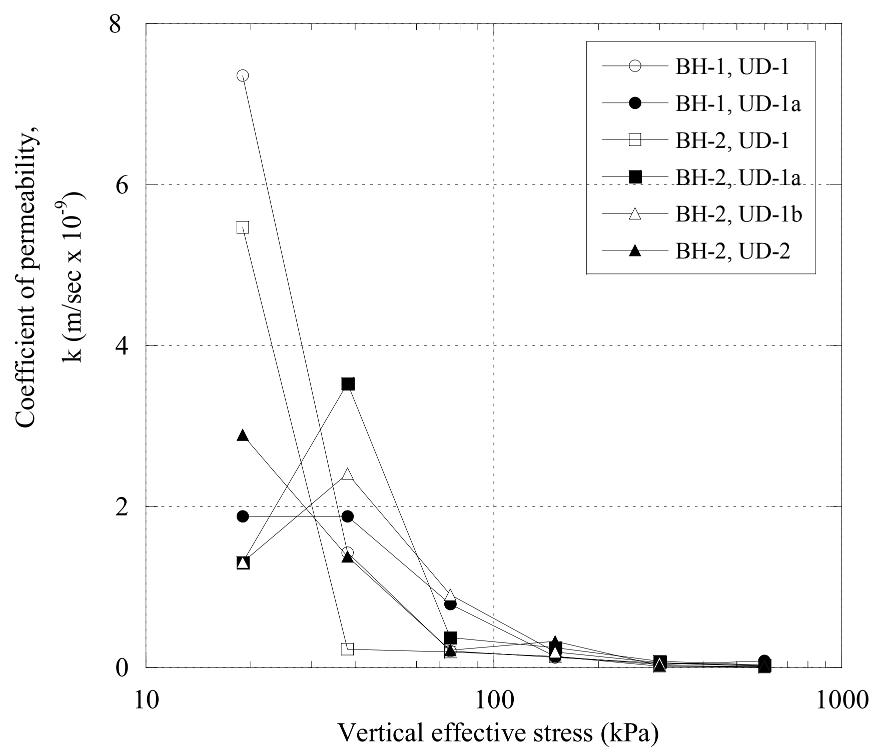


Figure 4.9 Coefficient of permeability (k) vs. vertical effective stress of the samples collected from the area S-1

4.2.2 Sub-soil Characteristics of Bansree (S-2)

The study area S-2 is situated in eastern part of Dhaka city. It is a private land reclaimed by dredge fill materials collected from nearby rivers. Location map of the study area S-2 have been presented in the Fig. 4.10. In area S-2, eight borehole locations have been discussed. In this study area, three boreholes were drilled and data of five boreholes has been collected from different sources (Ahmed, 2005 and Hossain, 2009). Soil profiles of these eight boreholes are presented in Fig. 4.11. The general soil profiles of these areas are filling sand overlaying clay or organic clay layer. Filling sand overlaying soft organic clay layer has been found at five borehole locations. And filling sand overlaying clay layer has been found at other three borehole locations. Detail test results of five boreholes (where soft organic layer has been found) are described and presented below.

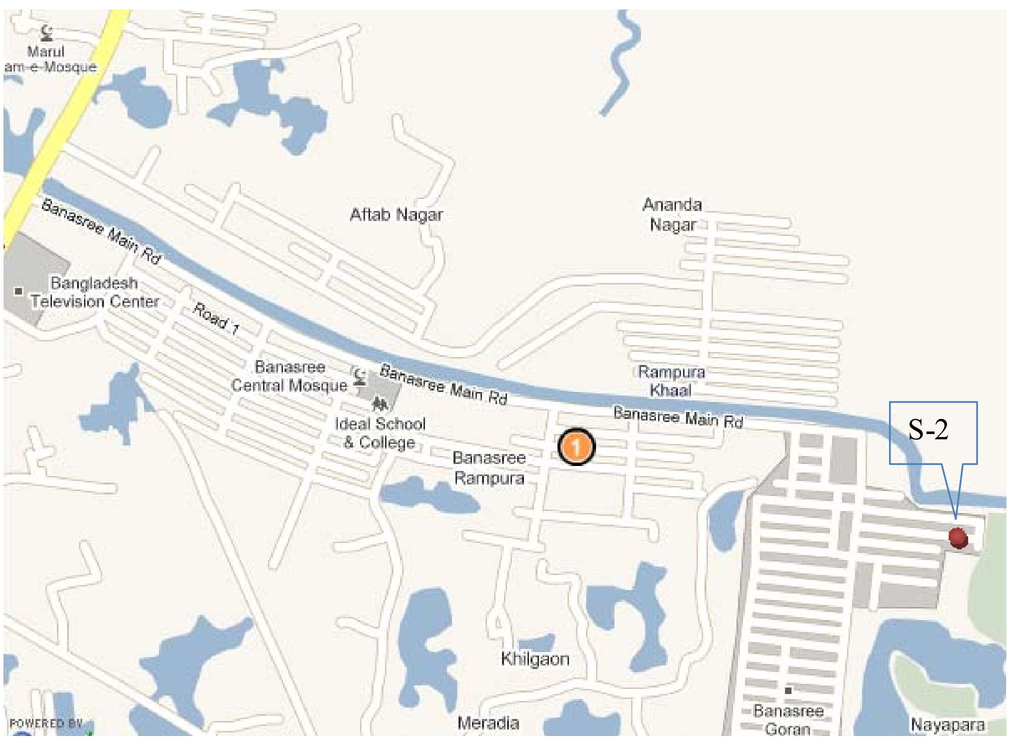


Figure 4.10 Location map of the study area S-2

SPT Results

SPT has been conducted in the area following the procedure described in ASTM D1586. Soil profiles of the boreholes have been shown in Fig. 4.11. Variations of the SPT N-values with depth of all the borehole locations have been presented in Fig. 4.12.

The general soil profile of this area is filling sand overlying soft clay or organic clay layer. The depth of filling sand varies from 1.5 to 6.0 m from Existing Ground Level (EGL).

Soft organic clay layer exists in five boreholes locations (BH-1 to BH-3, BH-5 and BH-6) of study area S-2. Depth of the organic layers varies from 5.5 to 14.0 m from EGL. Out of these five borehole locations, four borehole locations contain grey stiff silt layer beneath the organic layer and one borehole contains sand layer beneath the organic layer. After this organic layer, a grey stiff silt layer exists up to drilled depth (18 m from EGL) at three borehole locations (BH-1 to BH-3) and up to 12 m (from EGL) in one borehole location (BH-6). Beneath this silt layer in borehole location (BH-6), a grey soft silty clay layer has been found up to 18 m depth from EGL. In other borehole (BH-5), sand layer has been found up to 18 m depth from EGL and a grey soft silty clay layer having 1.5 m thickness is present in between this sandy layer.

Among all the borehole locations, a grey soft silty clay layer has been found beneath the filling layer in three borehole locations (BH-4, BH-7 and BH-8). The depth of the grey soft silty clay layer varies from 1.5 to 16.5 m from EGL. Beneath this soft silty clay layer, a sand layer exists in two borehole locations (BH-4 and BH-7) and a silt layer in one borehole location (BH-8). Depth of the sand layer varies from 9 to 18 m from EGL and a silt layer depth varies from 16.5 to 18 m from EGL. Beneath the sand layer, grey soft silty clay layer has been found in one borehole location (BH-4) up to drilled depth (18 m from EGL).

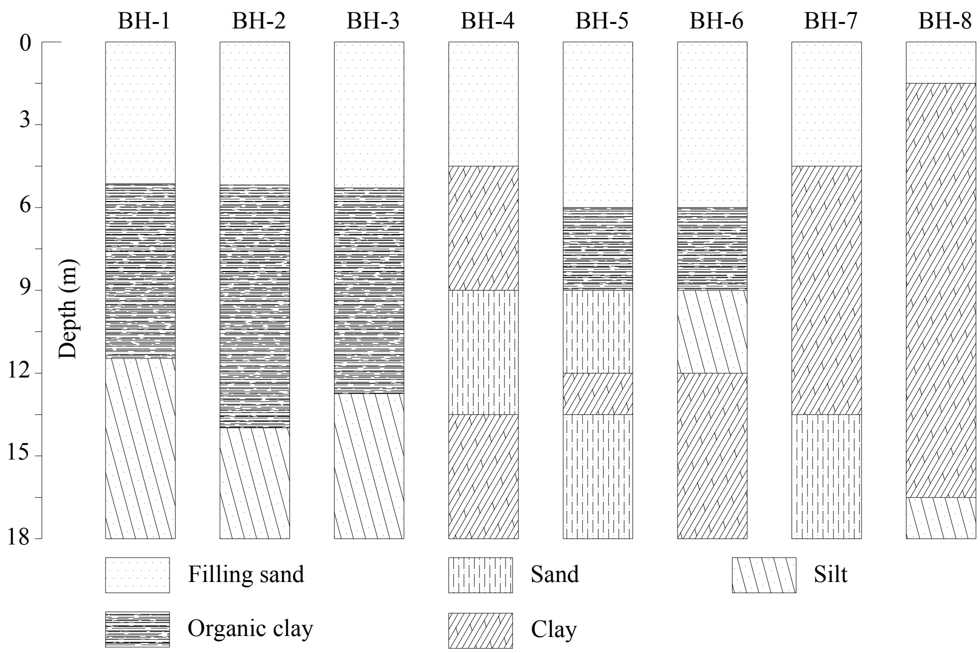


Figure 4.11 Soil profiles of the study area S-2

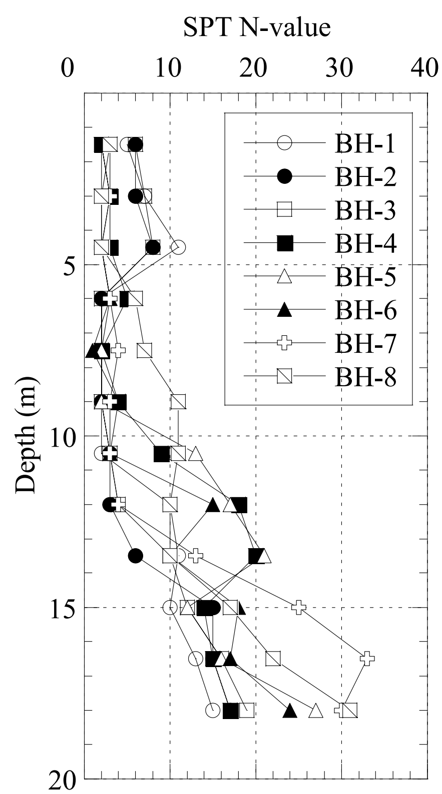


Figure 4.12 Depth vs. SPT N-Value of the study area S-2

The depth of grey stiff silt layers vary from 9.0 to 18.0 m from EGL. Below this silt layer, dense sand layer has been found at 9.0 to 18.0 m from EGL at various locations.

Fig. 4.12 presents the variations of SPT N-values with depth (from EGL) of all the borehole locations (BH-1 to BH-8) that contains filling soil overlaying organic clay and filling soil overlaying grey soft silty clay. It is seen that, the uncorrected SPT N-value of filling sand varies from 2 to 11. The SPT N-value of organic clay varies from 1 to 3. It is seen that organic clay layer is very soft. The SPT N-value of the grey clayey silt layers vary widely from 3 to 31. The SPT N-value of grey clay layer varies from 2 to 24. The SPT N-value of dense sand layer varies from 9 to 30.

Physical and index properties

Typical grain size distributions of sands collected from the various depth of S-2 are shown in Fig. 4.13. Specific gravity of organic clay varies from 2.30 to 2.48. Results of index properties of the soil samples have been presented in Table from 4.7 and Table 4.8, respectively.

The mean grain size (D_{50}) and fines content (F_c) of filling sand vary from 0.148 to 0.190 mm and 20.0 to 28.4%, respectively.

Table 4.7 Summary of physical properties of sand samples collected from the study area S-2

BH No./Sample No./Depth (m)	D_{50} (mm)	F_c (%)
BH-1/D-2/3.0	0.180	28.4
BH-1/D-3/4.5	0.175	20.0
BH-2/D-2/3.0	0.148	26.7
BH-3/D-2/3.0	0.190	21.7

Note: D_{50} = Mean grain size; F_c = Fines content

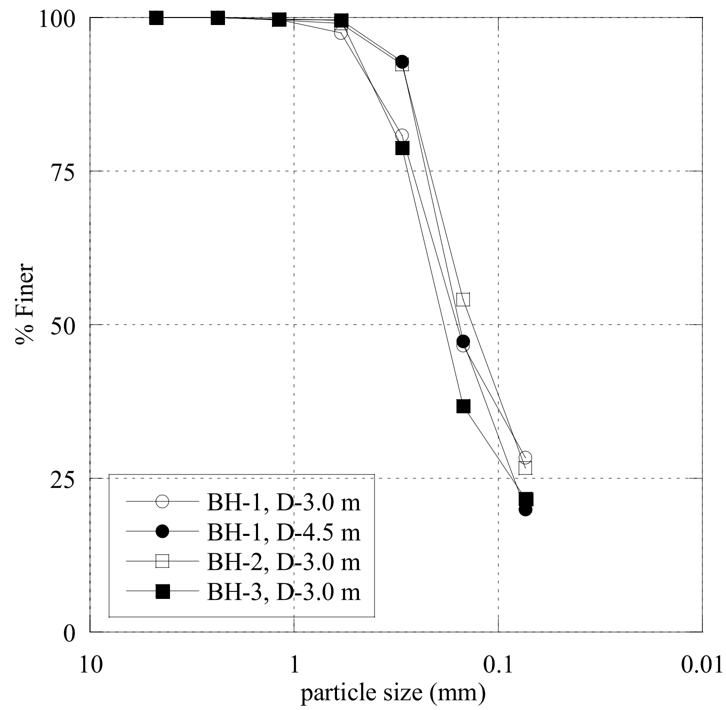


Figure 4.13 Typical grain size distributions of sand samples collected from the study area S-2

Table 4.8 Summary of physical properties of clay samples collected from the study area of S-2

BH No./Sample No./Depth(m)	w_n (%)	e_0	γ_d (kN/m^3)	LL (%)	PL (%)	PI (%)
BH-1/UD-1/5.5	63.4~82.3	1.759	8.8	60	38	22
BH-1/UD-2/7.0	44.1	0.870	12.1	48	28	20
BH-2/UD-1/5.5	83.6~140.0	1.824	8.0	77	55	22
BH-2/UD-2/7.0	49.7	1.822	8.4	68	45	23
BH-3/UD-1/5.5	71.9~80.7	1.634	9.0	62	30	32
BH-3/UD-2/7.0	69.0	1.633	8.8	71	48	23

Note: w_n= Natural moisture content; e_o = Initial void ratio; γ_d = Dry density; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index.

Atterberg's limits test results have been presented in Table 4.8. It has been found that top filling layer is non-plastic sand. Liquid limit (LL) and plasticity index (PI) of organic layer varies from 48 to 60% and 20 to 32%, respectively.

Organic content of the sample from the black very soft clay layer has been determined by loss on ignition method. Organic matter contents of the soils are presented in Table 4.9. It is seen that organic content of the soils varies from 5.0 to 21.8%. The OC range indicates that this soil may be classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils except one case (BH-1/UD-2/7m). This different case (BH-2/UD-2/7m) is considered as inorganic soil. (Table 2.1, chapter 2)

Shear strength properties

Unconfined compression tests have been conducted on organic clay samples collected from the borehole locations (BH-1 to BH-3) of the area S-2. It was observed that organic samples are very soft and sample preparation was very difficult. Special care was taken to trim the samples.

Table 4.9 Organic matter content of clay samples collected from the study area S-2

BH No./Sample No./Depth (m)	Soil description	Organic content (%)
BH-1/UD-1/5.5	Black clay	21.8
BH-1/UD-2/7.0	Grey soft clay	5.0
BH-2/UD-1/5.5	Black clay	14.4
BH-2/UD-2/7.0	Black clay	12.1
BH-3/UD-1/5.5	Black clay	16.4
BH-3/UD-2/7.0	Black clay	15.0

Table 4.10 Summary of unconfined compression test results of the study area S-2

BH No./Sample No./Depth (m)	w_n (%)	γ_{dry} (kN/m^3)	q_u (kPa)	s_u (kPa)	ε_f (%)	Consistency (BS 5930)
BH-1/UD-1/5.5	63.4~82.3	8.8	10~59	5~29	15	Very soft to soft
BH-1/UD-2/7.0	44.1	12.1	42	21	15	Soft
BH-2/UD-1/5.5	83.6~140.0	8.0	13~36	6~18	6~15	Very soft
BH-2/UD-2/7.0	50.0	8.4	39	19	10	Very soft
BH-3/UD-1/5.5	71.7~81.0	9.0	13~63	6~31	15	Soft
BH-3/UD-2/7.0	69.0	8.8	50	25	7	Soft

Note: w_n = Natural moisture content; q_u = Unconfined compressive strength; s_u = Undrained shear strength; ε_f = Failure strain.

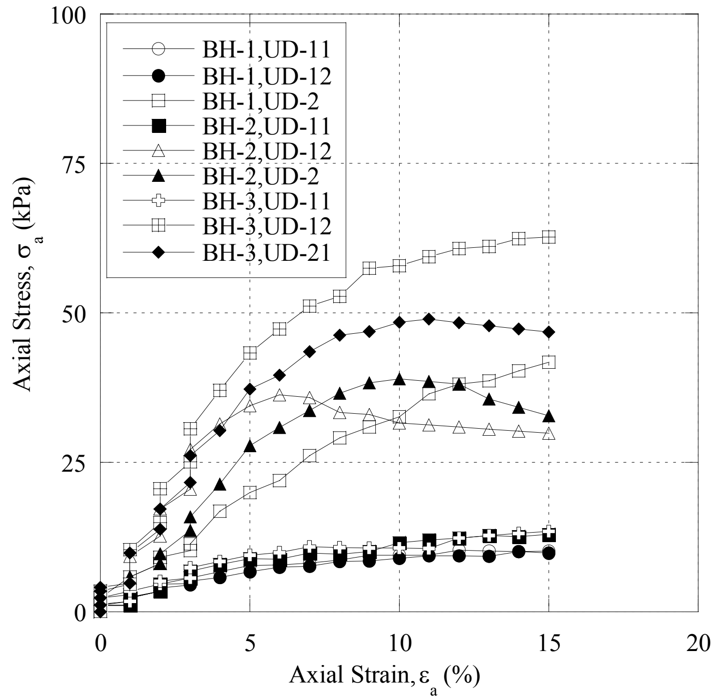


Figure 4.14 Axial stress vs. axial strain graph of organic clay samples collected from the study area S-2

Detail results of samples are presented in Table 4.10. From the table, it is seen that, dry unit weight and moisture content of the samples collected from organic layers vary from 8.0 to 12.1 kN/m^3 and 44 to 140%, respectively. It is seen that, dry unit weight is very low comparison to inorganic clays.

Axial stress versus axial strain graphs of the organic clay samples are presented in Fig. 4.14. From the Table 4.10 and Fig 4.14, it is seen that, undrained shear strength (s_u) and failure strain of the organic samples vary from 5 to 31 kPa and 6 to 15%, respectively. Undrained strength of this soil is very low and its consistency varies from very soft to soft based on BS 5930 (Bowels, 1982). It has been found that s_u of the clay samples of the area S-2 is less than s_u of Dhaka clay (s_u = 82.5~201.5 kPa; Hossain, 2009). In addition, it is similar with the soft organic Dhaka clay (s_u = 13~19 kPa; Islam et al., 2004).

Compressibility properties

One-dimensional consolidation tests have been conducted on undisturbed soil samples collected from organic layers of BH-1 to BH-3 of the study area S-2.

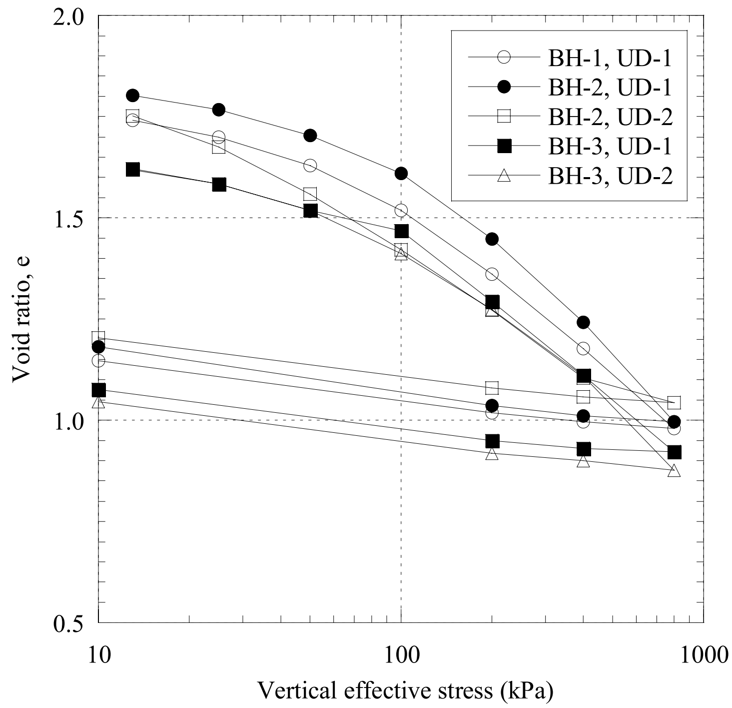


Figure 4.15 Typical e-logP curves of clay samples collected from the study area S-2

Table 4.11 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-2

BH No./Sample No./Depth (m)	e_o	C_c	C_r	c_v (m^2/yr)	m_v (kN/m^2*10^3)
BH-1/UD-1/5.5	1.759	0.66	0.06	0.36~1.63	0.19~1.22
BH-1/UD-2/7.0	0.870	0.33	0.03	3.20~6.88	0.13~1.24
BH-2/UD-1/5.5	1.824	0.81	0.07	1.10~17.25	0.22~1.02
BH-2/UD-2/7.0	1.822	0.53	0.06	0.53~4.14	0.06~2.18
BH-3/UD-1/5.5	1.634	0.63	0.05	0.90~11.08	0.11~1.16
BH-3/UD-2/7.0	1.633	0.78	0.07	0.76~1.41	0.20~1.07

Note: e_o = Initial void ratio; γ_d = Dry density; C_c = Compression index; C_r = Recompression index;

c_v = Coefficient of consolidation; m_v= Coefficient of volume compressibility.

Typical e-logP curves are presented in Fig. 4.15. From the unloading part of the e-logP curves, it is seen that the rebound is small in comparison to the inorganic clays. But the rebound of the other samples is higher than that of organic soil samples of the study area S-1. It means that this type of soil will also undergo large displacement/settlement. This difference may come due to difference of organic matter content (OC varies from 13.2~29.4% for S-1 and 5.0 to 21.8% for S-2).

Table 4.11 presents the one-dimensional consolidation test results. It is found that initial void ratio, compression index and recompression index of soft organic layer varies from 0.87 to 1.82, 0.33 to 0.81 and 0.03 to 0.07, respectively. It is seen that initial void ratio (e_o) of study area S-2 is less than that of Khulna soil (e_o varies from 3.11~6.15 for Khulna soil)

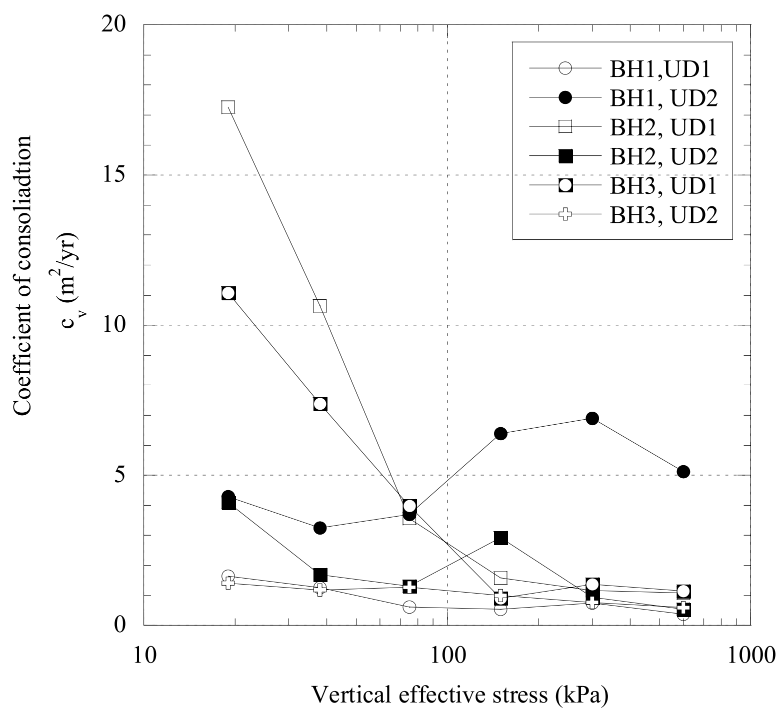


Figure 4.16 Coefficient of consolidation (c_v) vs. vertical effective stress of the samples collected from area S-2

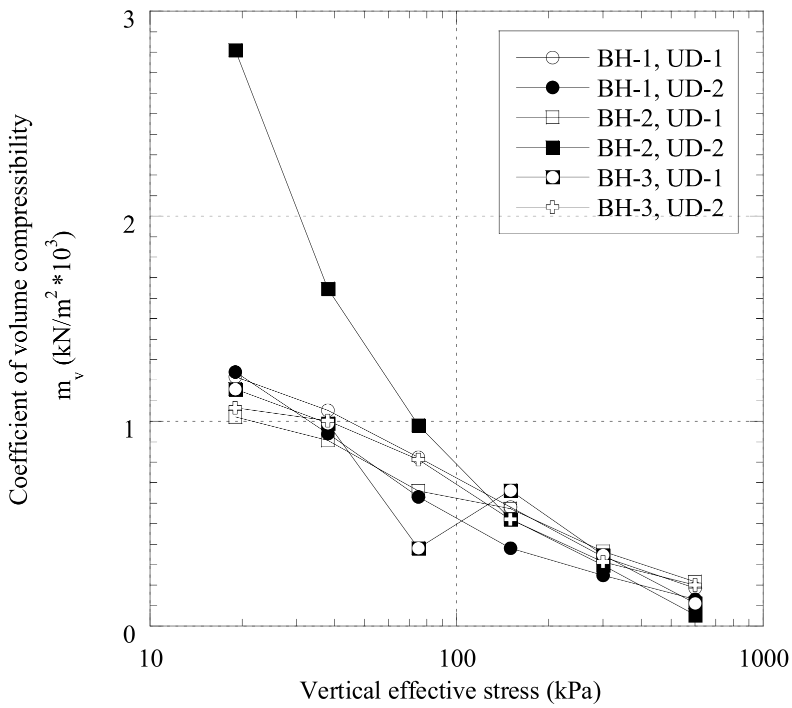


Figure 4.17 Coefficient of volume compressibility vs. vertical effective stress of the samples collected from area S-2

Results and graphs of coefficient of consolidation (c_v) has been presented in the Table 4.11 and Fig. 4.16, respectively. From the table, it is seen that coefficient of consolidation (c_v) widely varies from 0.36 to 17.25 m^2/yr. Fig. 4.16 shows that the values of c_v is variable with effective stress, whereas this variation does not follow any particular trend line.

Test results and graph of coefficient of volume compressibility (m_v) has been presented in Table 4.11 and Fig. 4.17. From the table, it is seen that m_v varies from 0.06×10^{-3} to 2.18×10^{-3}. Variations of m_v with effective vertical stress have been presented in the Fig. 4.17. It has been found that, m_v decreases with the increases of vertical effective stress.

Table 4.12 Summary coefficient of permeability of clay samples collected from the area S-2

BH No./ Sample No./ Depth (m)	k (m/sec)
BH-1/UD-1/5.5	2.09×10^{-11} to 6.17×10^{-10}
BH-1/UD-2/7.0	2.09×10^{-10} to 1.65×10^{-09}
BH-2/UD-1/5.5	7.26×10^{-11} to 5.48×10^{-09}
BH-2/UD-2/7.0	9.06×10^{-12} to 2.81×10^{-09}
BH-3/UD-1/5.5	4.06×10^{-11} to 3.98×10^{-09}
BH-3/UD-2/7.0	3.71×10^{-11} to 3.79×10^{-09}

Note: k = Coefficient of permeability.

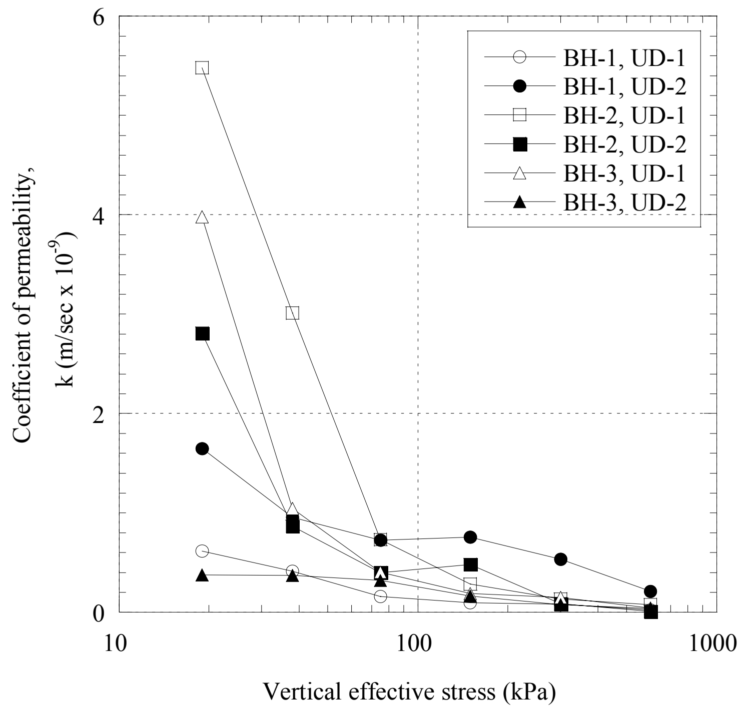


Figure 4.18 Coefficient of permeability (k) vs. vertical effective stress of the samples collected from the area S-2

Permeability properties

Coefficient of permeability (k) of the soil samples has been estimated from one-dimensional consolidation test results. A summary of the permeability values of the organic clay samples are presented in Table 4.12.

From this table 4.12, it is seen that values of k vary between 9.06×10^{-12} and 1.65×10^{-9} m/sec. It indicates that permeability is very low, which are typical to soft clay behavior. It is found that permeability of this soil is less than the permeability of Khulna organic soil (permeability of Khulna soil varies from 5.89×10^{-9}~8.59×10^{-9} m/sec; Ferdous, 2007). Fig. 4.18 presents the relationship between k and vertical effective stress of the study area S-2. From the Fig. 4.18, it is seen that, k decreases with the increases of vertical effective stress.

4.2.3 Sub-soil Characteristics of Pink city (S-3)

Pink city has been named as S-3 which is situated in northeast of Dhaka city. It is also a private housing project. This land area was developed by dredge filled material. Location map of the study area S-3 has been presented in Fig. 4.19. In the area S-3, four borehole locations were drilled from Block C (BH-1 to BH-4) and data of four boreholes has been collected from Block B (BH-5 to BH-8) by pink city authority. Soil profiles of all boreholes locations have been presented in Fig. 4.20.

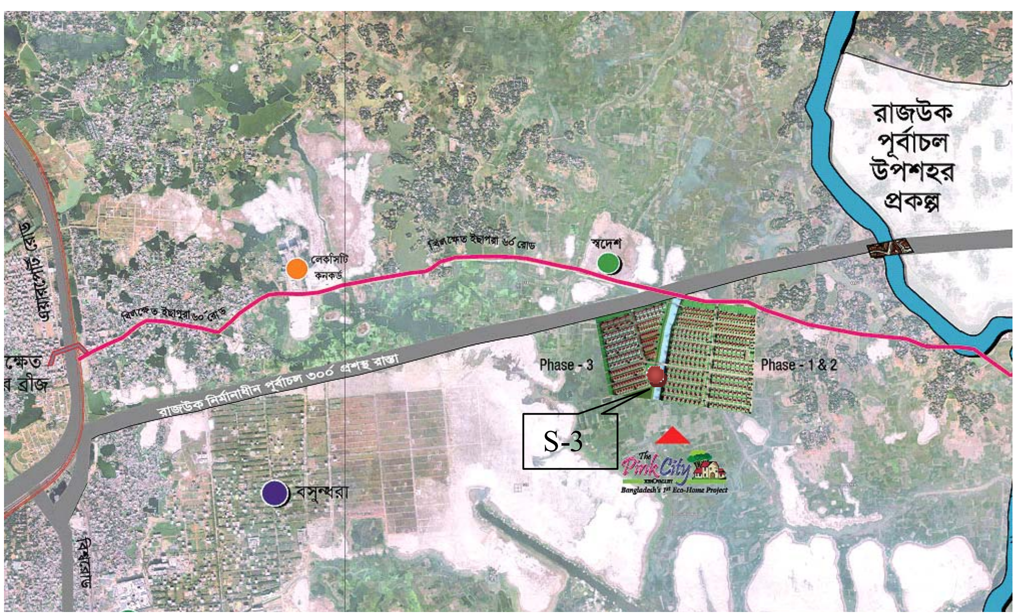


Figure 4.19 Location map of the study area S-3

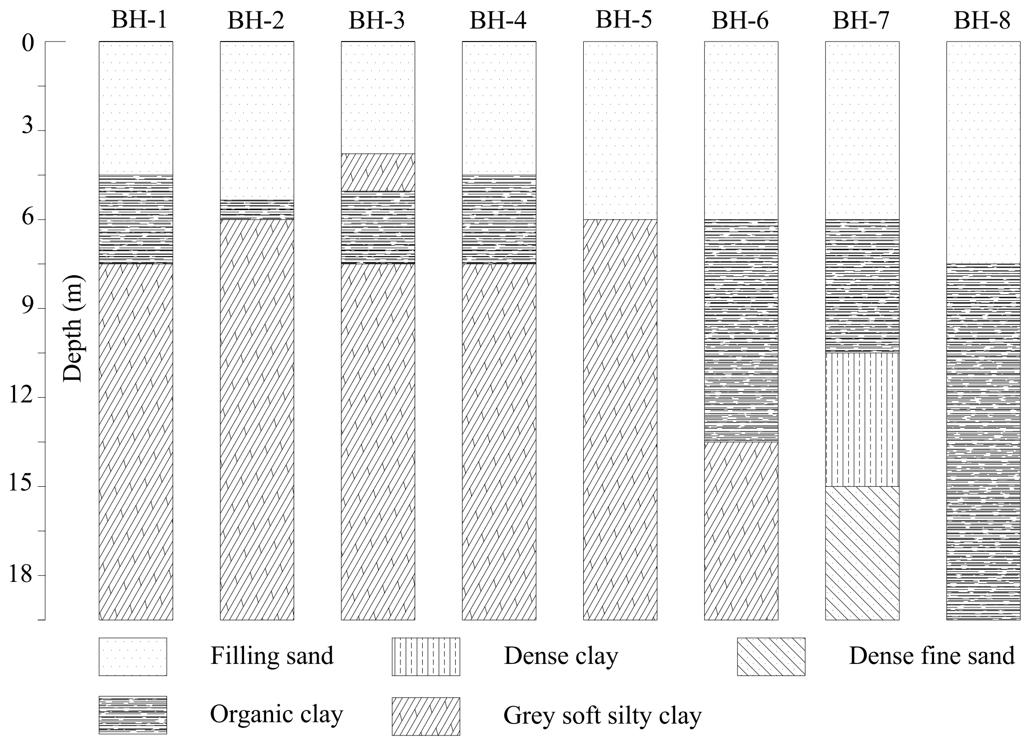


Figure 4.20 Soil profiles of the boreholes of the study area S-3

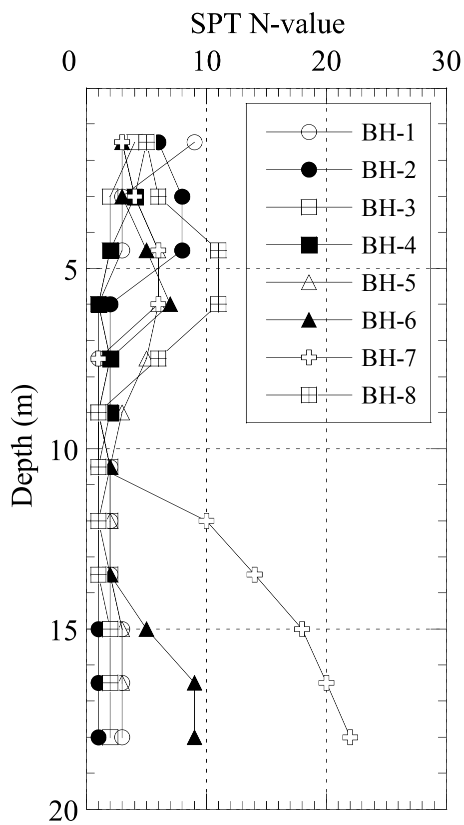


Figure 4.21 Depth vs. SPT N-Value of the study area S-3

The general soil profile of this area is filling sand overlaying clay or overlaying organic clay layer. Filling sand overlaying soft organic clay layer has been found at seven borehole locations of Block B and Block C. Out of eight boreholes locations, at one borehole location of Block B (BH-5) filling sand overlaying grey soft silty clay layer. Detail test results of these boreholes are described and presented below.

SPT Results

SPT has been conducted in the area S-3 following the procedure described in ASTM D1586. The soil profiles of the boreholes of the study area are presented in Fig. 4.21. Variations of SPT N-values with depths have been presented in the Fig. 4.21.

The general soil profile of this area is filling sand overlying soft clay or organic clay layer. The depth of filling sand varies from 4.0 to 7.5 m from EGL.

Soft organic clay layers exist in seven borehole locations (BH-1 to BH-4 and BH-6 to BH-8). Depths of these organic layers vary from 4.5 to 18.0 m from EGL. In area S-3, grey soft silty clay layer exists up to drilled depth (18 m from EGL) beneath the organic layer in five borehole locations (BH-1 to BH-4 and BH-6). In the other Borehole location (BH-7), beneath the organic clay, dense clay overlaying dense sand has been found. Depth of dense clay and dense sand layer has been found 10.5 and 15 m from EGL, respectively.

Fig. 4.21 shows the variations of SPT N-values with depth (from EGL) of all borehole locations (BH-1 to BH-8) of the study area S-3. The SPT N-value of the filling sand varies from 1 to 5. The SPT N-value of organic clay layer varies from 1 to 3. It is seen that this organic clay is very soft. The SPT N-value of soft silty clay layers vary from 10 to 18. The SPT N-value of the dense sand layer varies from 20 to 24.

Physical and index properties

Typical grain size distributions of sands of various filling depth of area S-3 are presented in Fig. 4.22. Specific gravity (G_s) of organic clays varies from 2.41 to 2.50 whereas higher value of specific gravity of the grey clay has been found 2.61.

Results of index properties of the sand and clay samples have been presented in Tables 4.13 and Table 4.14, respectively.

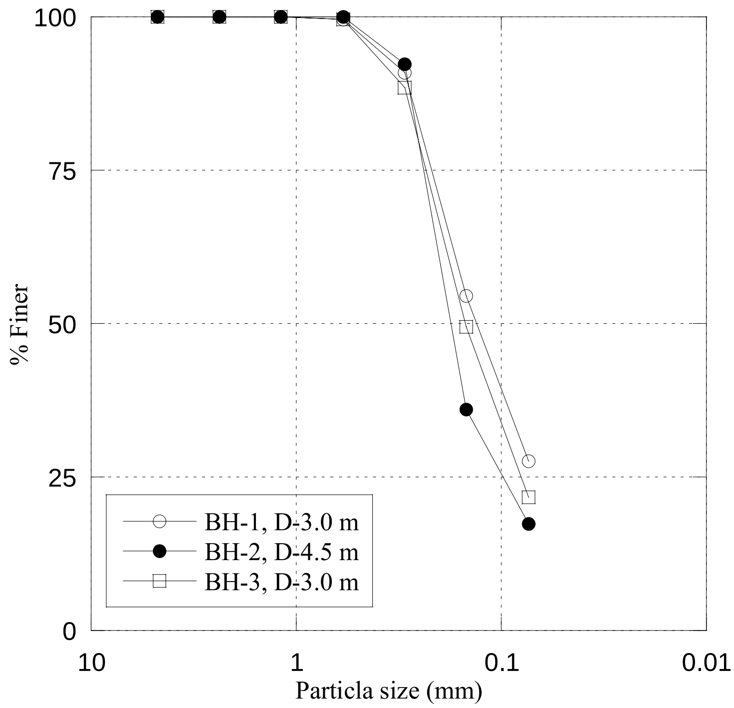


Figure 4.22 Typical grain size distributions of sand samples collected from the study area S-3

Table 4.13 Summary of physical properties of filling sand samples collected from the area S-3

BH No./Sample No./Depth (m)	D_{50} (mm)	F_c (%)
BH-1/D-2/3.0	0.150	27.6
BH-2/D-3/4.5	0.180	17.4
BH-3/D-2/3.0	0.170	21.7

Note: D_{50} = Mean grain size; F_c = Fines content

Table 4.14 Summary of physical properties of clay samples collected from the study area S-3

BH No./Sample No./Depth(m)	w_n (%)	e_0	γ_d (kN/m^3)	LL (%)	PL (%)	PI (%)
BH-1/UD-1/5.5	41.0~66.0	2.92	6.5	48	22	26
BH-1/D-4/6.0	–	–	–	115	59	56
BH-1/UD-2/7.0	66.1	–	–	54	31	23
BH-2/UD-1/5.5	35	–	–	55	26	29
BH-2/UD-2/7.0	37.0~41.0	–	–	40	27	13
BH-3/UD-1/5.5	42.0	1.72	9.4	54	29	25
BH-3/D-4/6.0	–	–	–	80	52	28
BH-3/UD-2/7.0	60.0~71.0	2.82	6.2	63	30	33
BH-4/UD-1/5.0	–	–	–	88	45	43
BH-4UD-2/5.5	–	–	–	42	26	17

Note: w_n= Natural moisture content; e_o = Initial void ratio; γ_d = Dry density; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index

Table 4.15 Organic matter content of the samples collected from the study area S-3

BH No./Sample No./Depth (m)	Soil description	Organic content (%)
BH-1/UD-1/5.5	Black clay	7.9
BH-1/UD-2/7.0	Black clay	13.8
BH-2/UD-1/5.5	Black clay	12.7
BH-2/UD-2/7.0	Grey soft clay	4.4
BH-3/UD-1/5.5	Black clay	8.8
BH-3/UD-2/7.0	Black clay	12.7

From the Fig. 4.22 and Table 4.13, it is seen that, the mean grain size (D_{50}) of the filling sands vary from 0.150 to 0.180 mm. Fines content (F_c) of filling sands have been found to vary from 17.4 to 27.6%.

Atterberg's limits test results have been presented in Table 4.14. It has been found that top filling layer is non-plastic sand. Liquid limit (LL) and plasticity index (PI) of clay layer at different depth varies widely from 22 to 115% and 13 to 56%, respectively.

Organic content of the samples from the very soft clay layer of the area S-3 has been determined by loss on ignition method. Organic matter contents of the soils are presented in Table 4.15. It is seen that organic content of the soils varies from 4.0 to 13.8%. The OC range indicates that this soil may be classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils except one case (BH-2/UD-2/7m). This different case (BH-2/UD-2/7m) is considered as inorganic soil (Table 2.1, chapter 2).

Shear strength properties

Unconfined compression tests have been conducted on organic clay samples collected from the borehole locations (BH-1 to BH-3) of the area S-3. It was observed that organic samples are very soft and sample preparation was very difficult. Special care was taken to trim the samples.

Table 4.16 Summary of unconfined compression test results of the clay samples collected from the study area S-3

BH No./Sample No./Depth (m)	w_n (%)	γ_{dry} (kN/m^3)	q_u (kPa)	s_u (kPa)	ε_f (%)	Consistency (BS 5930)
BH-1/UD-1/5.5	40.8~66.3	6.50	10~33	5~32	13	Very soft to soft
BH-1/UD-2/7.0	66.1	–	6	3	13	Very soft
BH-2/UD-1/5.5	34.9	–	66	33	9	Soft
BH-2/UD-2/7.0	37.4~41.4	–	11~58	5~29	14~15	Very soft to soft
BH-3/UD-1/5.5	42.3	9.4	36	18	12	Soft
BH-3/UD-2/7.0	59.6~70.5	6.2	11	5	9~12	Very soft

Note: w_n = Natural moisture content; q_u = Unconfined compressive strength; s_u = Undrained shear strength; ε_f = Failure strain.

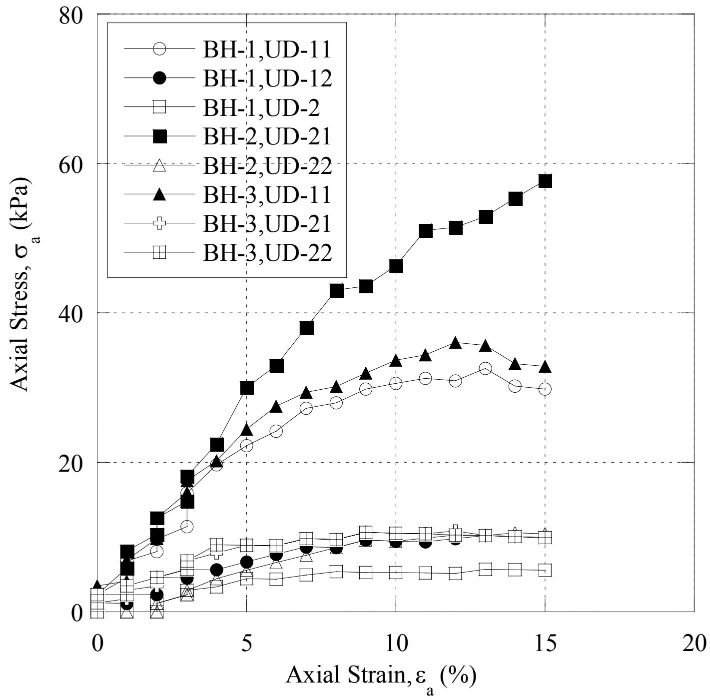


Figure 4.23 Axial stress vs. axial strain graph of organic clay samples collected from the study area S-3

Detail results of samples are presented in Table 4.16. From the table, it is seen that, dry unit weight and moisture content of the samples collected from organic layers vary from 6.2 to 6.5 kN/m^3 and 35 to 71%, respectively. Low dry unit weight has been found for study area S-3 comparison to inorganic clays. Axial stress versus axial strain graphs of the organic clay samples are presented in Fig. 4.23. From the Table 4.16 and Fig 4.23, it is seen that undrained shear strength (s_u) and failure strain of the organic samples vary from 3 to 32 kPa and 9 to 15%, respectively. Undrained strength of such soil is very low and its consistency varies from very soft to soft based on BS 5930 (Bowels, 1982). It has been found that s_u of the clay samples of the area S-3 is less than Dhaka clay (s_u = 82.5~201.5 kPa; Hossain, 2009). In addition, it is similar with the soft organic Dhaka clay (s_u = 13~19 kPa; Islam et al., 2004).

Compressibility properties

One-dimensional consolidation tests have been conducted on undisturbed soil samples collected from organic layers of BH-1 to BH-3 of the study area S-3. Typical e-logP curves are presented in Fig. 4.24. From the unloading part of the e-logP curves, it is seen that the rebound is small in comparison to the inorganic clays. It means that the elastic rebound in such cases is very low and the plastic deformation is high. It is similar with study area S-1. It means that this type of soil will undergo for large displacement/settlement like the soil of the study area S-1.

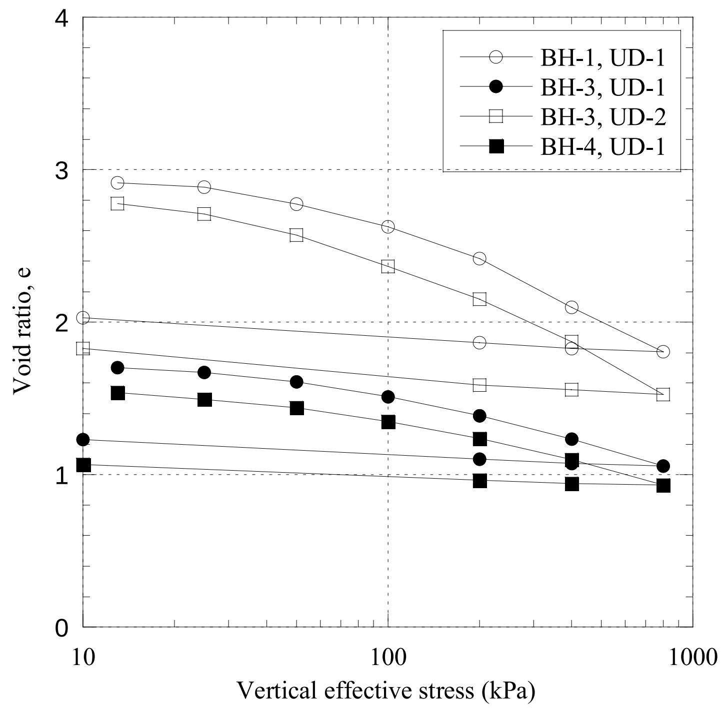


Figure 4.24 Typical e-logP curves of organic samples collected from the study area S-3

Table 4.17 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-3

BH No./Sample No./Depth (m)	e_o	C_c	C_r	c_v (m^2/yr)	m_v ($kN/m^2 * 10^3$)
BH-1/UD-1/5.5	2.92	0.97	0.10	1.20~2.08	0.19~1.13
BH-3/UD-1/5.5	1.72	0.60	0.08	0.57~1.02	0.17~0.97
BH-3/UD-2/7.0	2.82	1.15	0.11	0.20~10.89	0.23~1.47
BH-4/UD-1/5.5	1.57	0.55	0.05	1.05~3.83	0.16~1.40

Note: e_o = Initial void ratio; γ_d = Dry density; C_c = Compression index; C_r = Recompression index; c_v = Coefficient of consolidation; m_v= Coefficient of volume compressibility

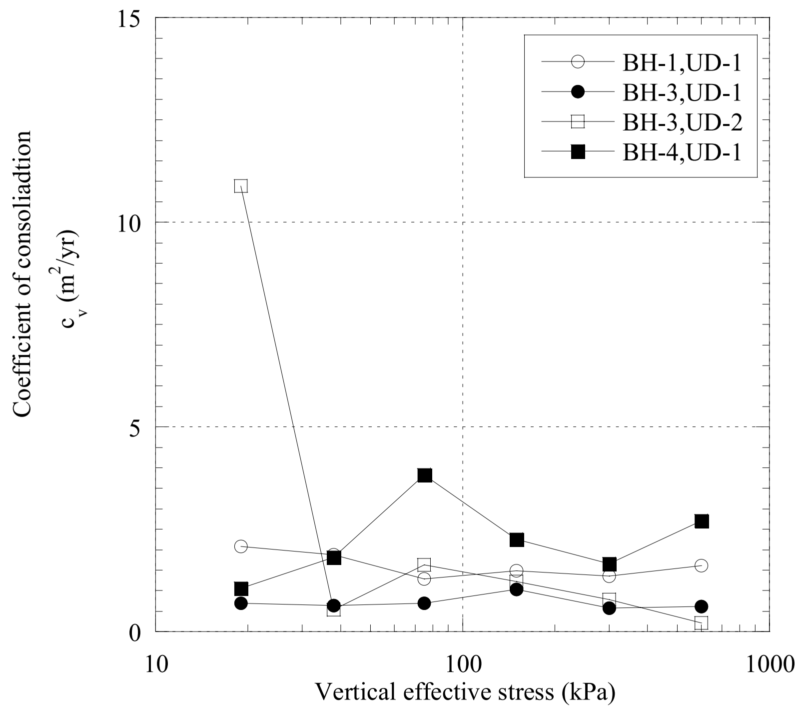


Figure 4.25 Coefficient of consolidation (c_v) vs. vertical effective stress of area the samples collected from area S-3

One-dimensional consolidation results have been presented in Table 4.17. It is found that initial void ratio, compression index and recompression index of soft organic soils vary from 1.57 to 2.92, 0.55 to 1.15 and 0.05 to 0.11, respectively. It is seen that initial void ratio (e_o) of study area S-2 is higher than that of study area S-1(e_o varies from 0.87~1.82) and less than that of Khulna soil (e_o varies from 3.11~6.15 for Khulna soil; Ferdous, 2007)

Results and graphs of coefficient of consolidation (c_v) has been presented in the Table 4.17 and fig. 4.25, respectively. From the table, it is seen that coefficient of consolidation (c_v) widely varies from 0.57 to 10.89 m^2/yr. Fig. 4.25 shows the variation c_v with the vertical effective stress. It is seen that this variation does not follow any particular trend.

Test results and graph of coefficient of volume compressibility (m_v) has been presented in Table 4.17 and Fig. 4.27, respectively. From the table, it is seen that m_v

varies from 0.16×10^{-3} to 1.40×10^{-3} kN/m^2. Fig. 4.27 shows the variation of m_v with effective vertical stress. Generally, m_v decreases with the increases of vertical effective stress except two cases (BH-1 and BH-3).

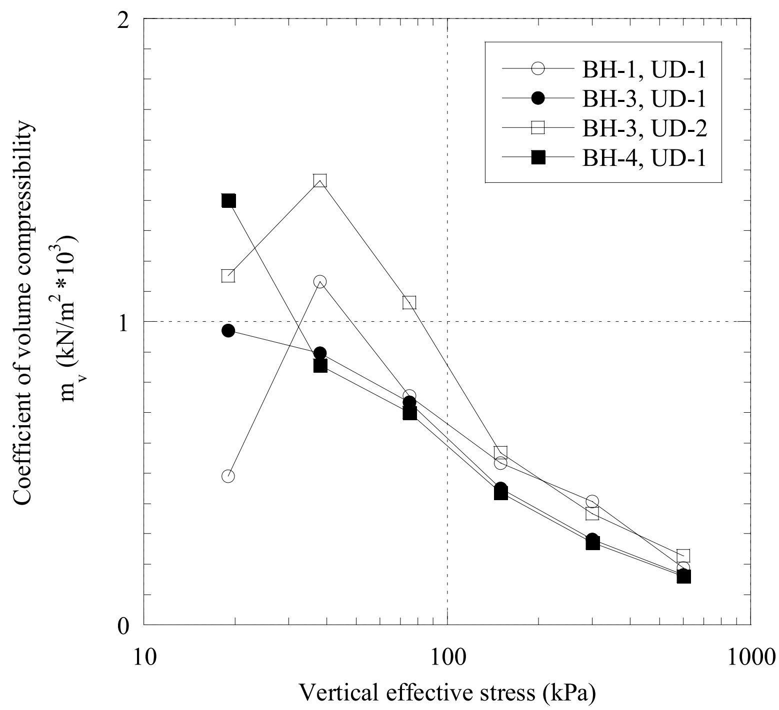


Figure 4.26 Coefficient of volume compressibility vs. vertical effective stress of the samples collected from area S-3

Table 4.18 Summary coefficient of permeability of organic samples collected from the study area S-3

BH No./ Sample No./ Depth (m)	k (m/sec)
BH-1/UD-1/5.5	9.39×10^{-11} to 6.64×10^{-10}
BH-3/UD-1/5.5	3.13×10^{-11} to 2.10×10^{-10}
BH-3/UD-2/7.0	1.47×10^{-11} to 4.97×10^{-09}
BH-4/UD-1/5.5	1.35×10^{-10} to 8.35×10^{-10}

Note: k = Coefficient of permeability

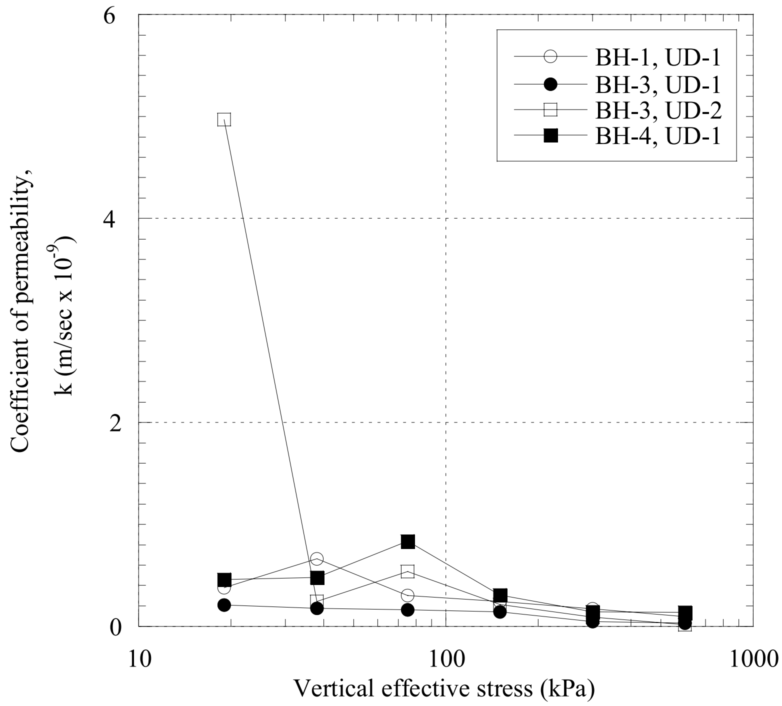


Figure 4.27 Coefficient of permeability (k) vs. vertical effective stress of the samples collected from the area S-3

Permeability properties

Coefficient of permeability (k) of the soil samples has been estimated from one-dimensional consolidation results. A summary of the permeability values of the organic clay samples are presented in Table 4.18.

From the table 4.18, it is seen that values of k vary between 9.39×10^{-12} and 4.97×10^{-9} m/sec. It indicates that permeability is very low, which are typical to soft clay behavior. It has also been found that permeability is less than the permeability of Khulna organic soil (permeability of Khulna soil varies from 5.89×10^{-9}~8.59×10^{-9} m/sec; Ferdous, 2007). Fig. 4.27 presents the relationship between k and vertical effective stress of the study area S-3. From the Fig. 4.27, it is seen that permeability decreases with increase of effective stress except one case (BH-3).

4.2.4 Sub-soil Characteristics of Hatir Jheel (S-4)

The Hatir Jheel is connected to Gulshan-Badda and Gulshan-Banani Lakes through narrow channels. Under the project peripheral roads with two cross bridges, walkway, lake, overpass, bus bay, car park and drain will be created. Total area of the project covers about 300 acre. Total five boreholes have been drilled and data of eleven boreholes were collected from Dhaka soil and BRTC, 2009. The detail soil tests results of this area are given below.

SPT Results

SPT has been conducted in the area following procedure described in ASTM D1586. The soil profiles of the boreholes have been shown in Fig. 4.29. The variations of SPT N-values with depth of typical boreholes have been shown in the Fig. 4.30.

The general soil profile of this area is filling sand overlying soft clay or organic clay layer. The depth of top filling dense sandy layer varies from 2.0 to 4.0 m from EGL.

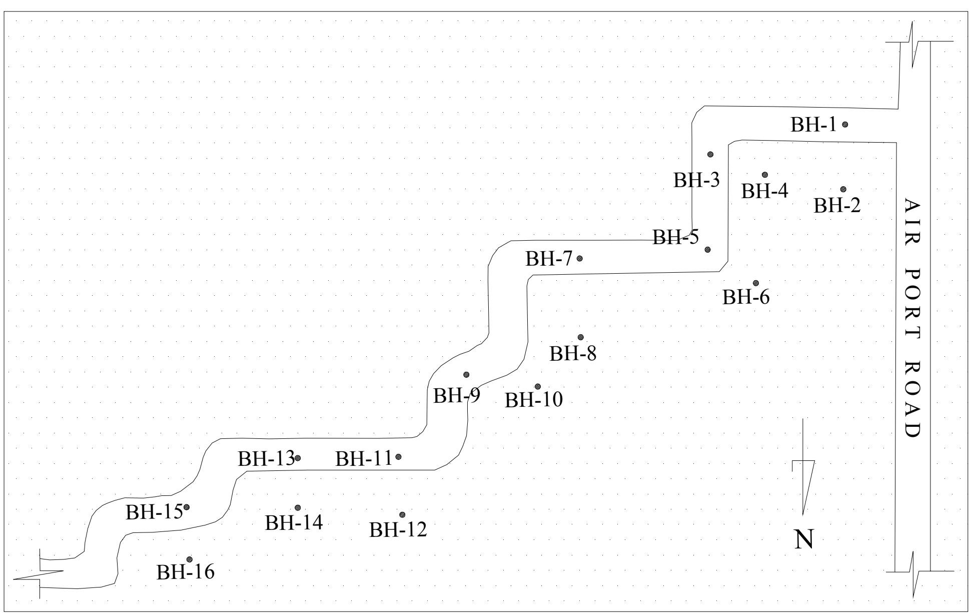


Figure 4.28 Location map of the study area S-4

Soft organic clay layer exists in six borehole locations (BH-1, BH-5, BH-6, BH-10, BH-11, and BH-15) of the study area S-4. Soft organic clay layer exists from 1.5 to 8.5 m from EGL. Out of these six borehole locations, five borehole locations contain grey stiff silty clay layer beneath the organic layer and one borehole contains dense sand layer beneath the organic layer. Depth of the grey soft silty layer varies from 5.0 to 10.5 m from EGL. Beneath this silty clay layer, dense sand has been found up to drilled depth (12 to 16.5 m from EGL) in six borehole locations (BH-1, BH-5, BH-6, BH-10, BH-11, and BH-15).

Among all the borehole locations, a grey soft silty clay layer has been found beneath the filling layer in four borehole locations (BH-3, BH-7, BH-9 and BH-13). The grey soft silty clay layer exists from 1.5 to 11 m from EGL. Dense sand layer has been found at 4.0 to 16.5 m from EGL at various locations. Beneath this silty clay layer, dense sand has been found up to drilled depth (12 to 16.5 m from EGL) in all these four borehole locations.

In the other five borehole locations (BH-2, BH-4, BH-8, BH-12 and BH-14), no organic layer has been found. In these four borehole locations, grey soft silty clay overlaying dense sand. Filling grey clay depth varies from 2.0 to 10.0 m from EGL. Beneath this clay layer, dense sand layer has been found up to drilled depth (12 m from EGL).

Fig. 4.30 presents the typical SPT N-values vs. depth (from EGL) of the five borehole locations (BH-1, BH-5, BH-10, BH-11 and BH-15) that contains filling soil overlaying organic clay. The uncorrected SPT N-value of filling sand varies widely from 12 to over. The SPT N-value of organic clay varies from 1 to 2. The SPT N-value of grey soft silty clay layers varies from 1 to 2. The SPT N-value of dense fine sand layer varies from 14 to 46.

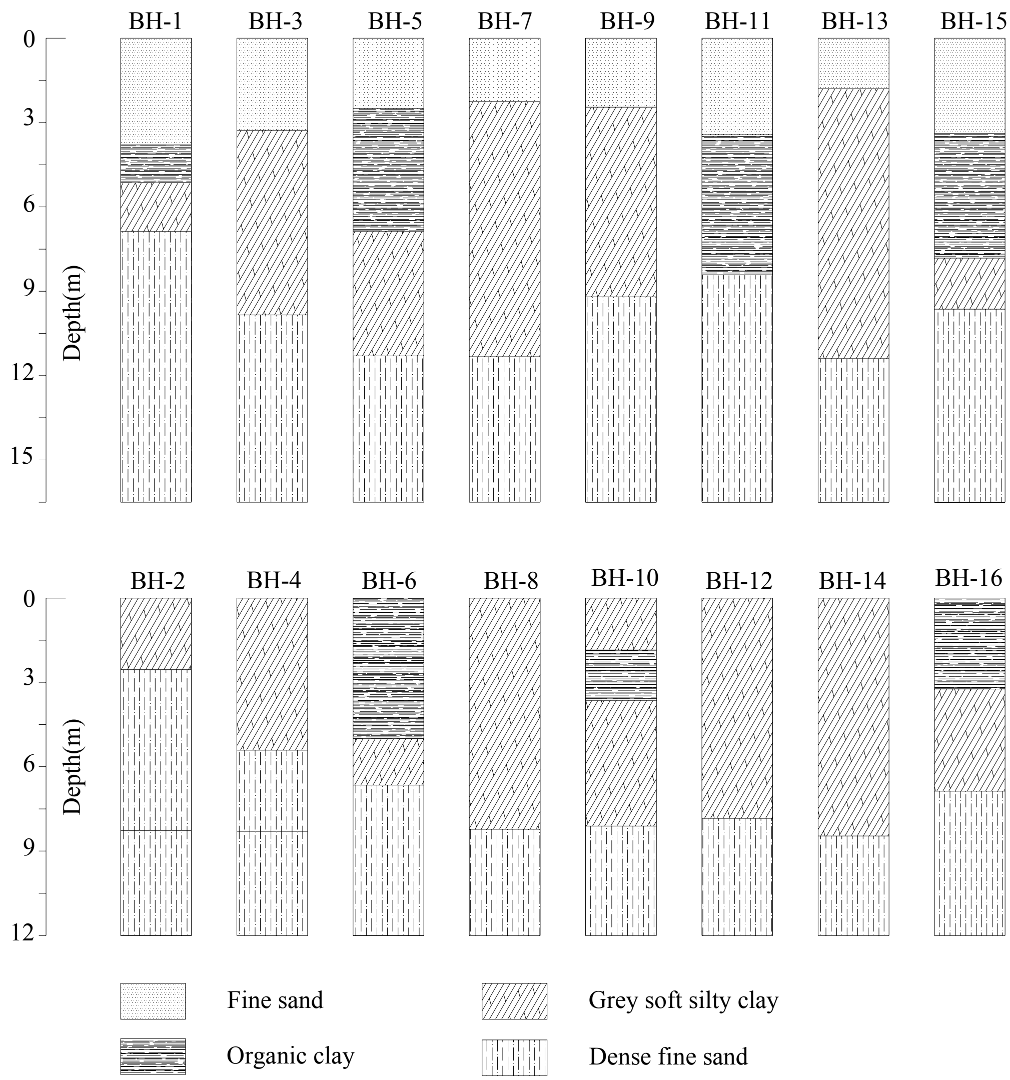


Figure 4.29 Soil profile of the boreholes of the study area S-4

Physical and index properties

Typical grain size distributions of sands of various depth of S-4 are shown in Fig. 4.32. Specific gravity of organic clay and grey clay layer varies from 2.53 to 2.58 and 2.53 to 2.7, respectively. Results of index properties of the sand and clay samples have been presented in Tables 4.19 and Table 4.20, respectively.

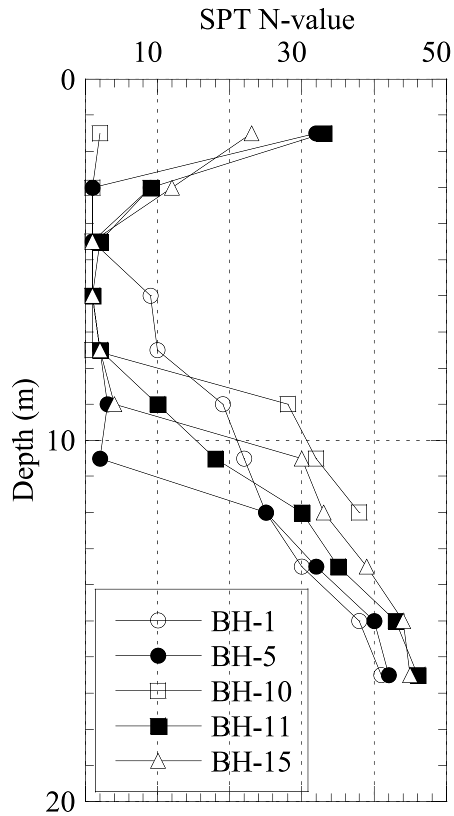


Figure 4.30 Depth vs. SPT N-Value of the study area S-4

Table 4.19 Summary of physical properties of sand samples collected from the study area S-4

BH No./Sample No./Depth (m)	D_{50} (mm)	F_c (%)
BH-10/D-1/1.5	0.210	20.2
BH-16/D-4/6.0	0.180	29.4
BH-16/D-5/7.5	0.175	30.7

Note: D_{50} = Mean grain size; F_c = Fines content.

Table 4.20 Summary of physical properties of clay samples collected from the study area S-4

BH No./Sample No./Depth(m)	w_n (%)	e_0	γ_d (kN/m^3)	LL (%)	PL (%)	PI (%)
BH-10/D-3/3.0	–	–	–	122	72	50
BH-10/D-4/6.0	–	–	–	48	26	22
BH-16/UD-2/3.0	–	–	–	67	42	25

Note: w_n= Natural moisture content; e_o = Initial void ratio; γ_d = Dry density; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index.

Table 4.21 Organic matter content of the samples collected from the study area S-4

BH No./Sample No./Depth (m)	Soil description	Organic content (%)
BH-10/UD-1/2.0	Black clay	14.3
BH-16/UD-2/2.0	Black clay	14.2

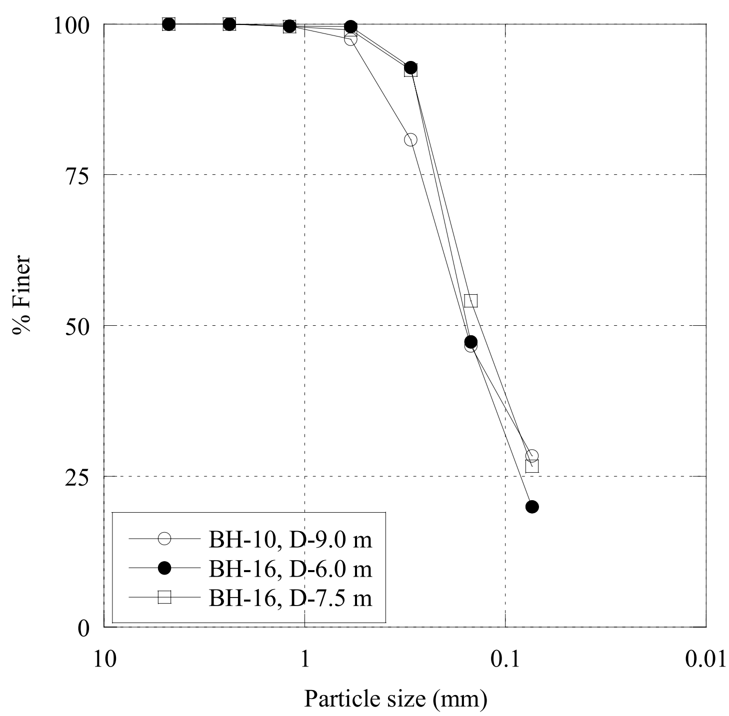


Figure 4.31 Typical grain size distributions of the sand samples collected from the study area S-4

The mean grain size (D_{50}) and fines content (F_c) of sand are 0.175 to 0.210 mm and 20.2 to 30.7%, respectively.

Atterberg's limits test results have been shown in Table 4.20. It has been found that top filling layer is non-plastic sand. Liquid limit (LL) and plasticity index (PI) of organic layer varies from 67 to 122% and 25 to 50%, respectively which are highly plastic. LL and PI of grey clay layer vary 48% and 22%, respectively.

Organic content of the sample from black very soft clay layer has been determined using loss of ignition method. Organic content of the samples are presented in Table 4.21. It is seen that organic content of the soils varies from 14.2 to 14.3%. This soil may be classified according to ASTM (Edil, 1997) as organic silts and clays (Table 2.1, chapter 2).

Compressibility properties

One-dimensional consolidation tests have been conducted on undisturbed soil samples collected from five boreholes locations (BH-3, BH-9, BH-10, BH-15 and BH-16) from organic layer at the study area S-4. Typical e-logP curves are presented in Fig. 4.32. From the unloading part of the e-logP curves, it is seen that the rebound is very small in comparison to the inorganic clay soils. It means that the elastic rebound in such cases is very low and the plastic deformation is high. It indicates that this type of soil will undergo for large displacement/settlement.

One-dimensional consolidation test results have been presented in Table 4.22. It is found that initial void ratio (e_o), compression index (C_c) and recompression index (C_r) of soft organic layer varies from 1.10 to 3.71, 0.28 to 0.94 and 0.03 to 0.09, respectively. It is seen that initial void ratio (e_o) of study area S-4 is less than that of Khulna organic soil (e_o varies from 3.11~6.15 for Khulna soil).

Results and graphs of coefficient of consolidation (c_v) has been presented in the Table 4.22 and Fig. 4.33, respectively. From the table, it is seen that coefficient of consolidation (c_v) widely varies from 0.37 to 13.67 m^2/yr. Fig. 4.33 shows that the values of c_v is variable with effective stress and it does not follow any particular trend line.

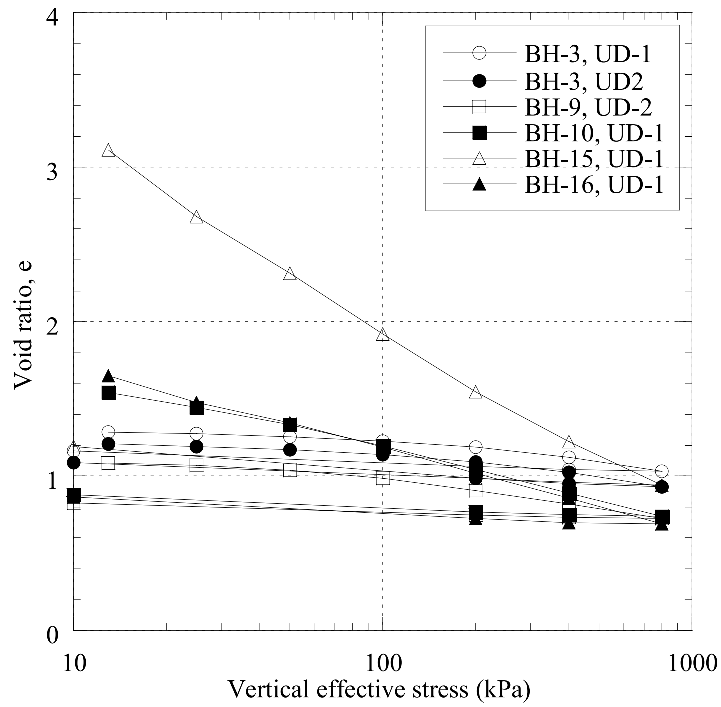


Figure 4.32 Typical e-logP curves of organic samples collected from the area S-4

Table 4.22 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-4

BH No./Sample No./Depth (m)	e_o	C_c	C_r	c_v (m^2/yr)	m_v (kN/m^2*10^3)
BH-3/UD-1/3.6	1.296	0.28	0.05	1.03~11.19	0.09~0.42
BH-3/UD-2/5.2	1.229	0.31	0.09	1.72~11.07	0.10~0.65
BH-9/UD-2/5.2	1.103	0.30	0.03	4.55~13.67	0.11~0.61
BH-10/UD-1/2	1.596	0.50	0.05	0.45~1.82	0.15~3.02
BH-15/UD-1/4	3.706	0.94	0.08	0.37~6.43	0.15~7.30
BH-16/UD-1/4	1.685	0.56	0.06	0.55~7.19	0.16~5.16

Note: e_o = Initial void ratio; C_c = Compression index; C_r = Recompression index; c_v = Coefficient of consolidation; m_v= Coefficient of volume compressibility.

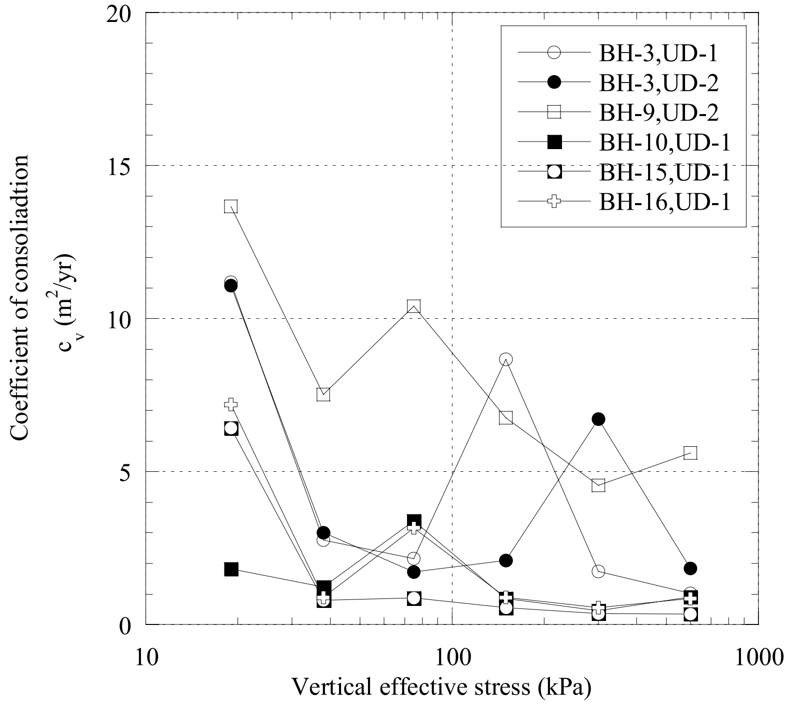


Figure 4.33 Coefficient of consolidation (c_v) vs. vertical effective stress of the samples collected from area S-4

Test results and graph of coefficient of volume compressibility (m_v) has been presented in Table 4.22 and Fig. 4.34. Coefficient of volume compressibility (m_v) varies from 0.09×10^{-3} to 7.30×10^{-3} kN/m^2. Fig. 4.34 shows the variation of m_v with effective vertical stress. It is seen m_v decreases with the increases of vertical effective stress.

Permeability properties

Coefficient of permeability (k) of the soil samples has been estimated from one-dimensional consolidation results. A summary of the permeability values of the organic soil samples are presented in Table 4.23. From the table, it is seen that the coefficient of permeability (k) range varies between 1.68×10^{-11} and 2.59×10^{-9} m/sec. It indicates that permeability is very low. It has also been found that permeability is less than the permeability of Khulna organic soil (permeability of Khulna soil varies from 5.89×10^{-9}~8.59×10^{-9} m/sec; Ferdous, 2007). Fig. 4.35 shows the relationship between permeability and vertical effective stress at selected site S-4. It is seen that permeability decreases with the increases of vertical effective stress.

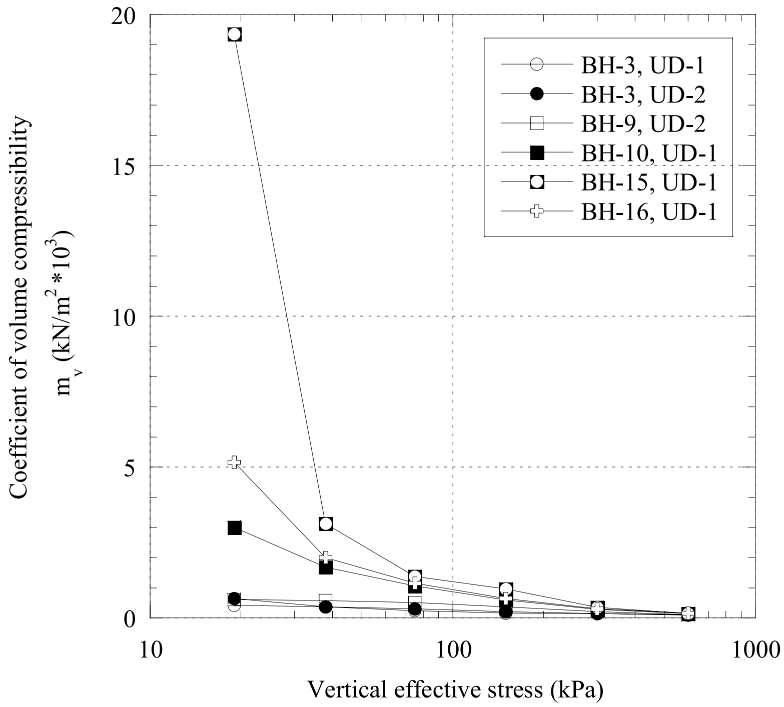


Figure 4.34 Coefficient of volume compressibility vs. vertical effective stress of the samples collected from area S-4

Table 4.23 Summary coefficient of permeability of organic samples collected from study area S-4

BH No./ Sample No./ Depth (m)	k (m/sec)
BH-3/UD-1/3.6	3.00×10^{-11} to 1.45×10^{-09}
BH-3/UD-2/5.2	6.00×10^{-11} to 2.22×10^{-09}
BH-9/UD-2/5.2	1.89×10^{-10} to 2.59×10^{-09}
BH-10/UD-1/2	4.08×10^{-11} to 1.71×10^{-09}
BH-15/UD-1/4	1.68×10^{-11} to 1.46×10^{-08}
BH-16/UD-1/4	4.20×10^{-11} to 1.15×10^{-08}

Note: k = Coefficient of permeability

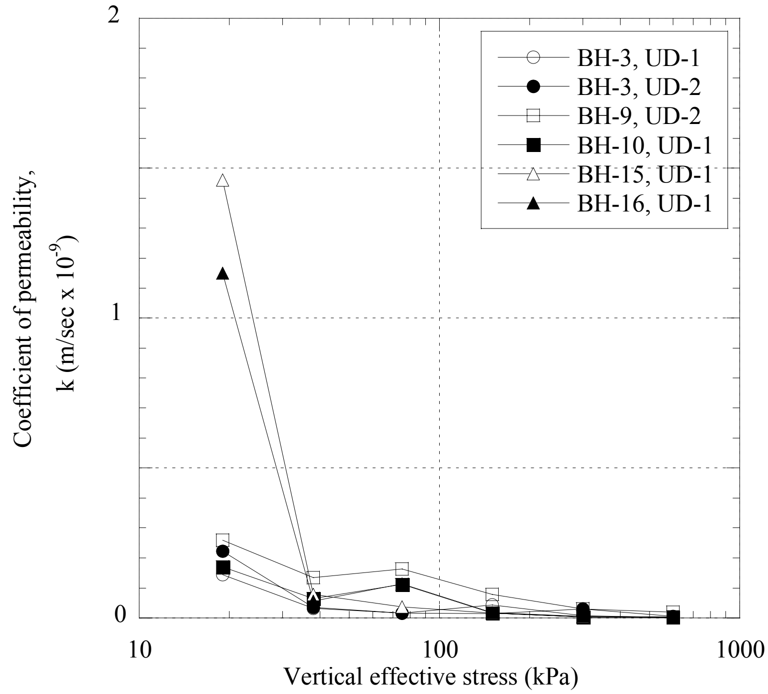
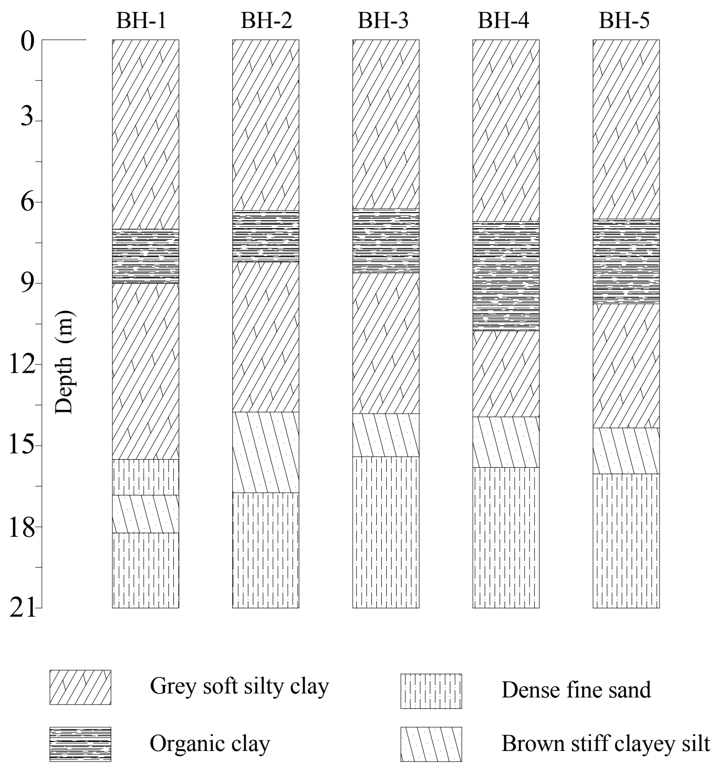


Figure 4.35 Coefficient of permeability (k) vs. vertical effective stress of the samples collected from the area S-4

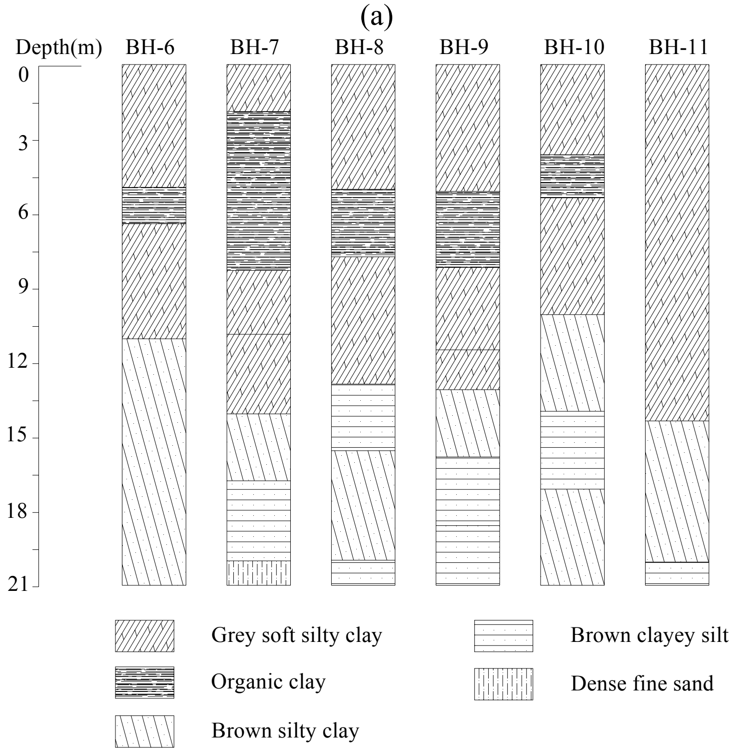
4.2.5 Sub-soil Characteristics of Uttara (S-5)

Uttara model town is situated at the North side of Dhaka city. Data of eleven boreholes have been collected from different others. Among them, data of five boreholes locations (BH-1 to BH-5) were collected from Dhaka Soil and data of six (BH 6 to BH-11) were collected from BRTC, 2010. The detail results are shown in below.

Soil profiles of these boreholes are shown in Fig. 4.36(a) and Fig. 4.36(b). The general soil profile of this area is filling clay overlaying organic clay layer. Filling clay overlaying soft organic clay layer has been found at all the borehole locations (BH-1 to BH-10) except one borehole location (BH-11). Detail test results of these boreholes are described and presented below.


BH-1
BH-2
BH-3
BH-4
BH-5
0
3
6
9
12
15
18
21
Depth (m)
Grey soft silty clay
Dense fine sand
Organic clay
Brown stiff clayey silt

(a)


Depth(m)
BH-6
BH-7
BH-8
BH-9
BH-10
BH-11
0
3
6
9
12
15
18
21
Grey soft silty clay
Brown clayey silt
Organic clay
Dense fine sand
Brown silty clay

(b)

Figure 4.36 Soil profile of the boreholes of the study area S-5: (a) Data collected from Dhaka Soil (BH-1 to BH-5) and (b) Data collected from BRTC, 2010 (BH-6 to BH-11)

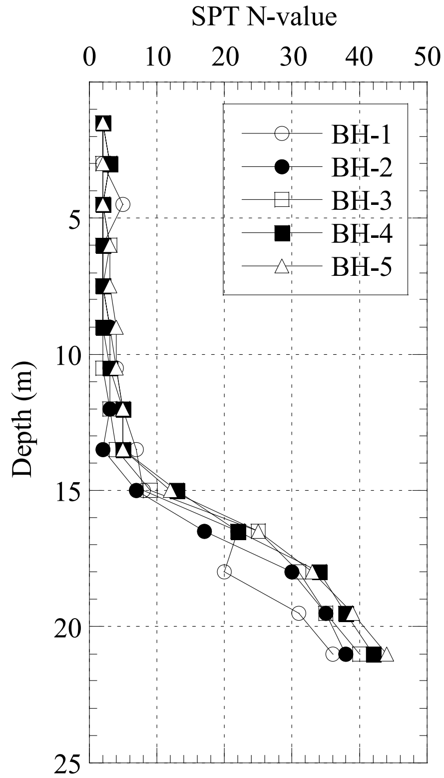


Figure 4.37 Depth vs. SPT N-Value of the study area S-5

SPT Results

The soil profiles of the boreholes have been shown in Fig. 4.36. The variations of SPT N-values with depth of these boreholes have been presented in the Fig. 4.37.

The general soil profile of this area is grey soft silty clay overlying soft organic clay layer. The depth of top soft silty layer varies from 1.5 to 15.0 m from EGL.

Soft organic clay layers exist in ten borehole locations (BH-1 to BH-10). Depth of the organic layer varies from 6.5 to 11.0 m from EGL. In the other borehole location (BH-11), no such layer exists. Beneath the organic layer, a grey soft silty clay layer exists from 8.5 to 14 m (from EGL) in ten borehole locations (BH-1 to BH-10). After this grey soft silty clay, brown clayey silt layer exists in other five borehole locations (BH-2 to BH-5 and BH-8) and brown silty clay exists in five borehole location (BH-6, BH-7, BH-9, BH-10 and BH-11). Stiff clayey silt exists from 14.0 to 18.5 m from EGL. The depth of silty clay layer varies from 11.0 to 21 m from EGL. At the lower end of the borehole, dense sand layer has been found at 15.5 to 21.0 m from EGL at various locations

Fig. 4.21 shows the variations of SPT N-values with depth (from EGL) of typical borehole locations (BH-1 to BH-5) of the study area S-5. The SPT N-value of top soft silty clay layer varies from 2 to 5. The SPT N-value of organic clay varies from 2 to 4. It is seen that this organic clay is very soft. The SPT N-value of grey soft silty clay layers varies from 3 to 8 and dense fine sand layer varies from 22 to 44.

Physical and index properties

The mean grain size (D_{50}) of clay and sand soil samples are ranges in between 0.0075 and 0.22 mm. Specific gravity of top clay layer varies from 2.67 to 2.68. Results of index properties of the soil samples have been presented in Tables 4.24.

Atterberg's limits test results have been shown in Table 4.25. It has been found that top filling layer is grey soft silty clay layer. Liquid limit (LL) and plasticity index (PI) of this layer has been found 53% and 28 %, respectively.

Table 4.24 Summary of physical properties of the samples collected from the study area S-5

BH No./Sample No./Depth (m)	D_{50} (mm)
BH-1/D-1/1.5	0.0075
BH-1/D-11/16.5	0.149
BH-2/D-8/12.0	0.010
BH-2/D-12/18.0	0.190
BH-3/D-10/15.0	0.220
BH-3/D-11/16.5	0.148
BH-4/D-14/21.0	0.160
BH-5/D-10/15.0	0.024

BH-3/D-12/18.0	0.150

Note: D_{50} = Mean grain size.

Table 4.25 Summary of physical properties of clay samples collected from the study area of S-5

BH No./Sample No./Depth(m)	w_n (%)	LL (%)	PL (%)	PI (%)
BH-1/D-1/1.5	31.2	53	25	28
BH-2/D-8/12.0	28.5	52	26	26
BH-3/D-10/15.0	26.5	38	27	11
BH-4/D-8/12.0	26.4	51	24	27
BH-5/D-10/15.0	24.5	36	28	8
BH-7/D-4/6.5	−	136	35	101
BH-9/D-4&5/7.5	−	181	51	130

Note: w_n= Natural moisture content; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index.

Table 4.26 Organic matter content of clay samples collected from the study area S-5

BH No./Sample No./Depth (m)	Soil description	Organic content (%)
BH-6/D-4/6	Black clay	31.8
BH-8/D-4/6	Black clay	30.5
BH-11/UD-1/5.2	−	11.1

From the Table 4.25, it is also seen that, Liquid limit (LL) and plasticity index (PI) of organic clay layer vary from 136 to 181% and 101 to 130%, respectively. It seems that plasticity index of organic clay in this area is very high. Beneath this organic layer, a soft grey clay layer exists. LL and PI of this soft silty clay layer varies from 51 to 52% and 26 to 27 %, respectively.

Organic content of the samples are presented in Table 4.26. It is seen that organic content of the soils varies from 11.1 to 31.8%. This soil may be classified as ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils (Table 2.1, Chapter 2).

Shear strength properties

Unconfined compression test results of the samples collected from the two borehole locations (BH-7 and BH-11) have been presented in Table 4.27.

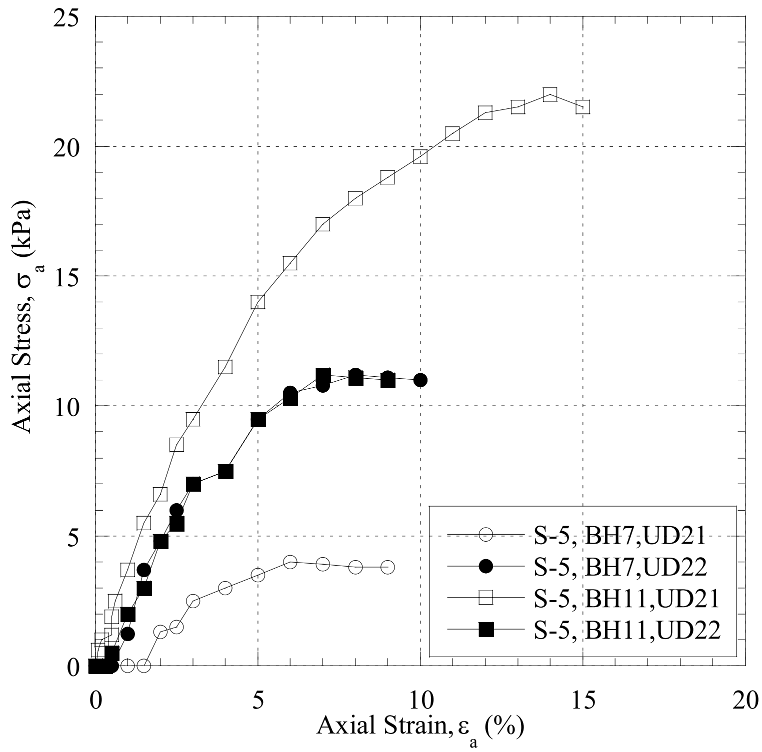


Figure 4.38 Axial stress vs. axial strain graph of organic clay samples collected from the study area S-5

Table 4.27 Summary of unconfined compression test results of the study area S-5

BH No./Sample No./Depth (m)	w_n (%)	γ_{dry} (kN/m^3)	q_u (kPa)	s_u (kPa)	Consistency (BS 5930)
BH-7/UD-21/5.2	104.6	6.64	4	2	Very soft
BH-7/UD-22/5.2	89.9	7.35	7	3	
BH-11/UD21/5.2	78.6	7.98	22	11	
BH-11/UD21/5.2	83.8	7.68	10	5	

Note: w_n = Natural moisture content; γ_{dry} = Dry density; q_u = Unconfined compressive strength; s_u = Undrained shear strength.

From the Table 4.27, it is seen that dry unit weight and moisture content of the organic vary from 6.64 to 7.35 kN/m^3 and 89.9 to 104.6%, respectively. Whereas, dry unit weight and moisture content of the grey soft silty clay vary from 6.68 to 7.98 kN/m^3 and 78.6 to 83.8%, respectively. Low dry unit weight has been found for study area S-5 comparison to inorganic clays. Axial stress versus axial strain graphs of the organic clay samples are presented in Fig. 4.38. From the Table 4.27 and Fig. 4.38, it is seen that undrained shear strength of the organic clay samples varies from 2 to 3 kPa. It is seen that, undrained strength of this soil is very low and its consistency is very soft based on BS 5930 (Bowels, 1982). In addition, it is less then that of soft organic Dhaka clay (s_u = 13~19 kPa; Islam et al., 2004).

Compressibility properties

One-dimensional consolidation test results of four borehole locations (BH-1, BH-5, BH-6 and BH-7) of study area S-5 have been presented in Table 4.28.

Typical e-logP curves are presented in Fig. 4.39. From the unloading part of the e-logP curves, it is seen that the rebound is small in comparison to the inorganic clays. It means that this type of soil will also undergo large displacement/settlement.

From the Table 2.28, it is seen that initial void ratio (e_o) and compression index (C_c) of the organic layer varies from 4.41 to 5.51 and 1.75 to 1.93, respectively. It is seen that initial void ratio and compression index of organic layer in this area is too much high which is similar with e_o of the Khulna organic soil (e_o varies from 3.11~6.15 for Khulna soil).

Table 4.28 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-5

BH No./Sample No./ Depth (m)	w_n (%)	e_o	γ_d (kN/m^3)	G_s	C_c	c_v (m^2/yr)
BH-6/UD-1/5.2	183	4.41	4.21	2.32	1.75	0.5~6.0
BH-7/UD-2/5.2	225	5.51	3.32	2.20	1.93	0.3~7.5

Note: w_n = moisture content; e_o = Initial void ratio; γ_d = dry unit weight; G_s = Specific gravity; C_c = Compression index; c_v = Coefficient of consolidation

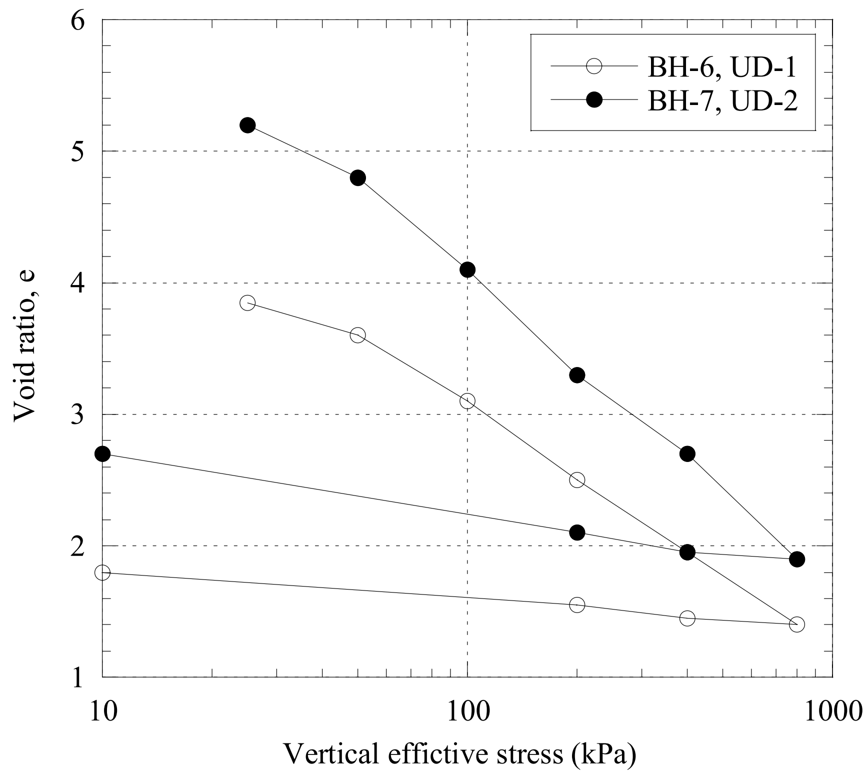


Figure 4.39 Typical e-logP curves of samples collected from the study area S-5

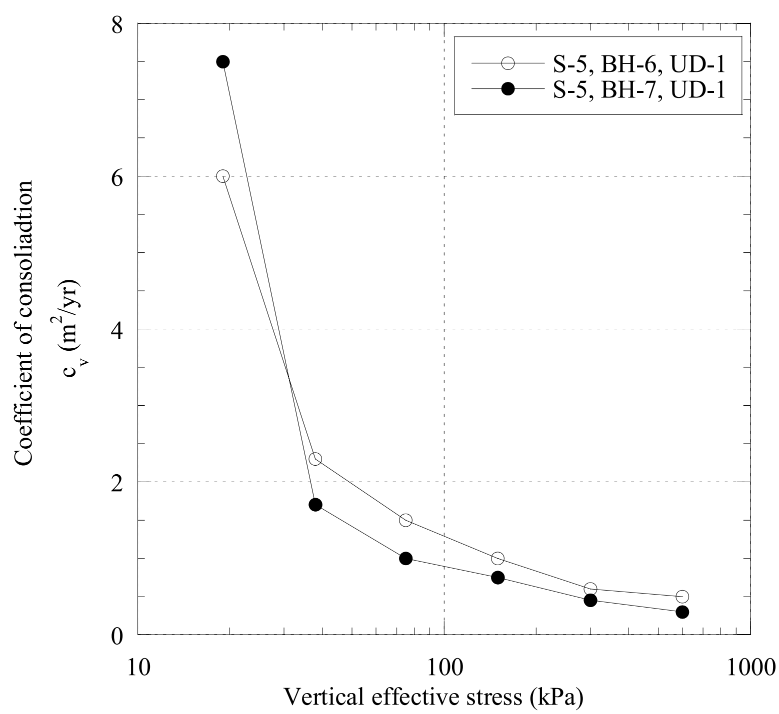


Figure 4.40 Coefficient of consolidation (c_v) vs. vertical effective stress of the samples collected from area S-5

Results and graphs of coefficient of consolidation (c_v) has been presented in the Table 4.18 and Fig. 4.40, respectively. From the table, it is seen that coefficient of consolidation (c_v) varies from 0.3 to 7.5 m^2/yr. Fig. 4.40 shows the variation of c_v with effective stress. It is seen that c_v decreases with the increase of vertical effective stress.

4.2.6 Sub-soil Characteristics of Nikunja-1 (S-6)

Nikunja-1 is North side of Dhaka city. Test results of this area were collected from Dhaka soil. Detail test results have been presented below.

SPT Results

The soil profiles of the boreholes have been presented in Fig. 4.41. The variations of SPT N-values with depth of these boreholes have been shown in the Fig. 4.42.

Three borehole locations (BH-1, BH-2 and BH-3) have been collected from this area. The general soil profile of this area is filling clay overlying soft organic clay layer. The depth of top brown soft silty clay layer varies from 3.65 to 8.20 m from EGL. Among the three locations, organic soil has been found in two borehole locations (BH-1 and BH-2). And in the other borehole location (BH-3) no organic layer has been found. Soft organic clay layer exists from 3.65 to 7.0 m from EGL. A medium stiff clayey silt layer exists from 6.7 to 12.8 m from EGL beneath the organic layer (BH-1 and BH-2) and beneath the filling layer (BH-3). After the silt layer, dense sand layer has been found at 11.6 to 21.0 m from EGL at this area.

The uncorrected SPT N-value of top soft silty clay layer varies from 1 to 2. The SPT N-value of organic clay varies from 2 to 4. The SPT N-value of clayey silty layer varies from 2 to 6. The SPT N-value of dense fine sand layer varies from 14 to 54.

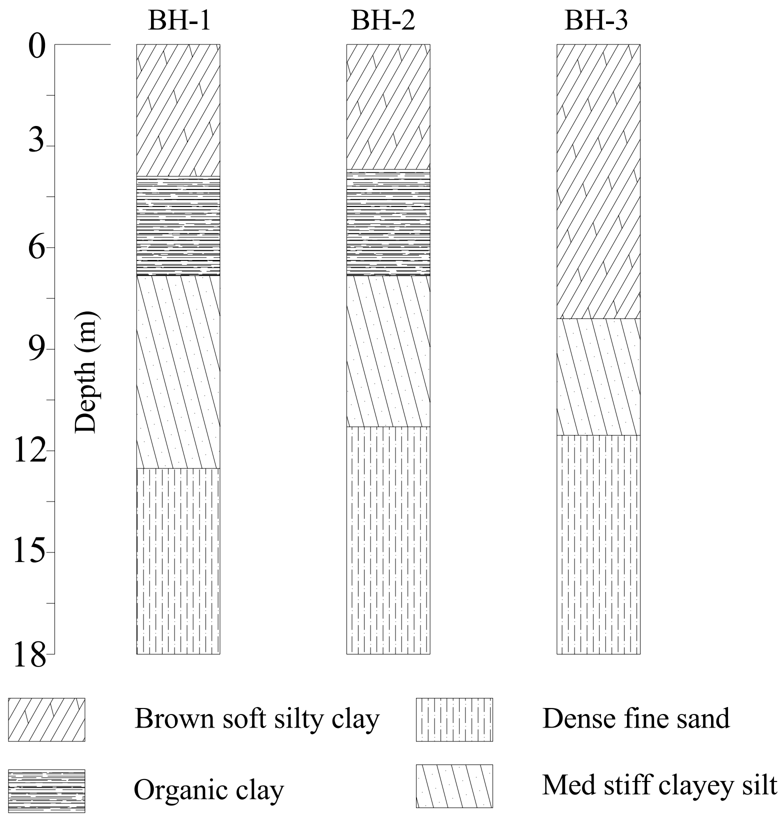


Figure 4.41 Soil profile of the boreholes of the study area S-6

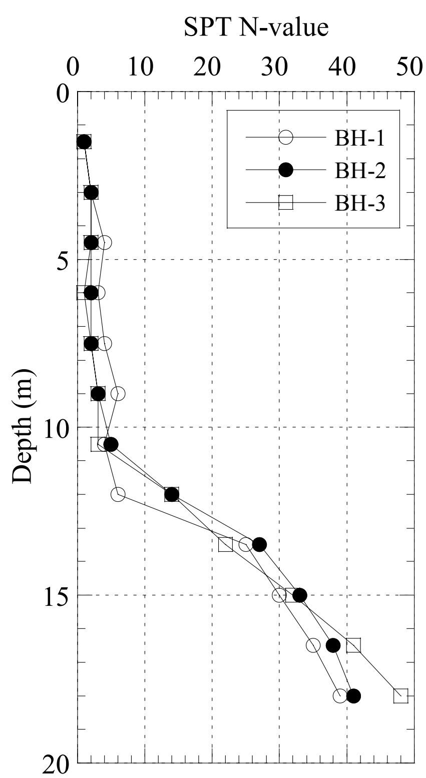


Figure 4.42 Depth vs. SPT N-Value of the study area S-6

Table 4.29 Summary of physical properties of the samples collected from the study area S-6

BH No./Sample No./Depth (m)	D_{50} (mm)
BH-1/D-5/7.5	0.0078
BH-1/D-9/13.5	0.148
BH-2/D-5/7.5	0.0175
BH-2/D-10/15.0	0.0180
BH-3/D-4/6.0	0.008
BH-3/D-12/18.0	0.220

Note: D_{50} = Mean grain size.

Table 4.30 Summary of physical properties of clay samples collected from the study area of S-6

BH No./Sample No./Depth(m)	W_n (%)	LL (%)	PL (%)	PI (%)
BH-1/D-5/7.5	31.4	52	26	26
BH-2/D-5/7.5	29.4	38	27	11
BH-3/D-4/6.0	33.5	55	28	27

Note: w_n= Natural moisture content; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index.

Physical and index properties

The mean grain size (D_{50}) of soil samples are ranges in between 0.008 and 0.148 mm. Specific gravity of top clay layer have been found 2.68. Results of index properties of the soil samples have been presented in Tables 4.29.

Atterberg's limits tests results have been presented in the Table 4.30. Liquid limit (LL) and plasticity index (PI) of silty clay layer varies from 38 to 55% and 11 to 26%, respectively.

Shear strength properties

Unconfined compression tests results of three borehole locations (BH-1, BH-2 and BH-3) have been presented in Table 4.31. Dry unit weight and moisture content of the clay layer varies from 13.3 to 13.6 kN/m^3 and 30.8 to 32.6%, respectively. Dry unit weight of this soil is high with comparison with the organic soil and similar to inorganic soil. From the Table 4.31, it is seen that undrained shear strength of the clay samples varies from 12 to 14 kPa. Although the dry unit weight of this soil is high, it shows low shearing strength. It is seen that, undrained strength of this soil is very low and its consistency is very soft based on BS 5930 (Bowels, 1982). In addition, it is similar with that of soft organic Dhaka clay (s_u = 13~19 kPa; Islam et al., 2004).

Compressibility properties

One-dimensional consolidation test results of three borehole locations (BH-1, BH-2 and BH-3) of study area S-6 have been presented in Table 4.32. From the Table 4.32, it is seen that, initial void ratio (e_o) and compression index (C_c) of clay layer have been found 0.696 and 0.325, respectively.

Table 4.31 Summary of unconfined compression test results of the study area S-6

BH No./Sample No./Depth (m)	w_n (%)	γ_{dry} (kN/m^3)	q_u (kPa)	s_u (kPa)
BH-1/UD-1/2.4	31.4	13.6	28	14
BH-2/UD-1/2.4	30.8	13.6	26	13
BH-3/UD-1/2.4	32.6	13.3	24	12

Note: w_n = Natural moisture content; γ_{dry} = Dry density; q_u = Unconfined compressive strength; s_u = Undrained shear strength;

Table 4.32 Summary of one-dimensional consolidation test results of the clay samples collected from the study area S-6

BH No./Sample No./Depth (m)	e_o	C_c
BH-3/UD-1/2.4	0.696	0.325

Note: e_o = Initial void ratio; C_c = Compression index

4.3 CORRELATIONS

Soft organic layer has been classified by Unified Soil Classification System (USCS). Liquid limit (LL) and plasticity index (PI) data of soft clay have been verified by drawing U-line. Fig. 4.43 is the graphical presentation of the A-chart. It is seen that soils are varying from OL (medium compressibility and organic silt) to OH (highly compressibility and organic clay).

Correlations between s_u/σ_v with others parameters

(i) s_u/σ_v Plasticity Index (PI)

Attempt has been taken to correlate s_u/σ_v with plasticity index (PI). Graphical presentation between s_u/σ_v and PI is presented in Fig. 4.44. Here, s_u = undrained shear strength, σ_v = vertical effective stress and PI = plasticity index.

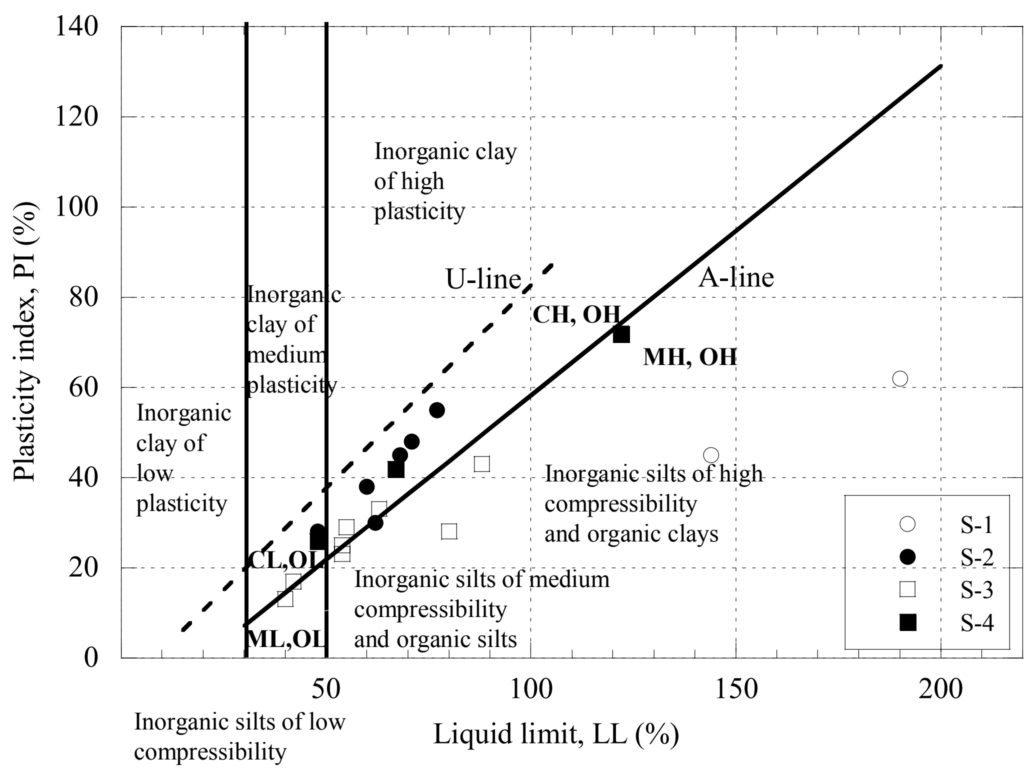


Figure 4.43 Position of the soft soil samples on Casagrande plasticity chart

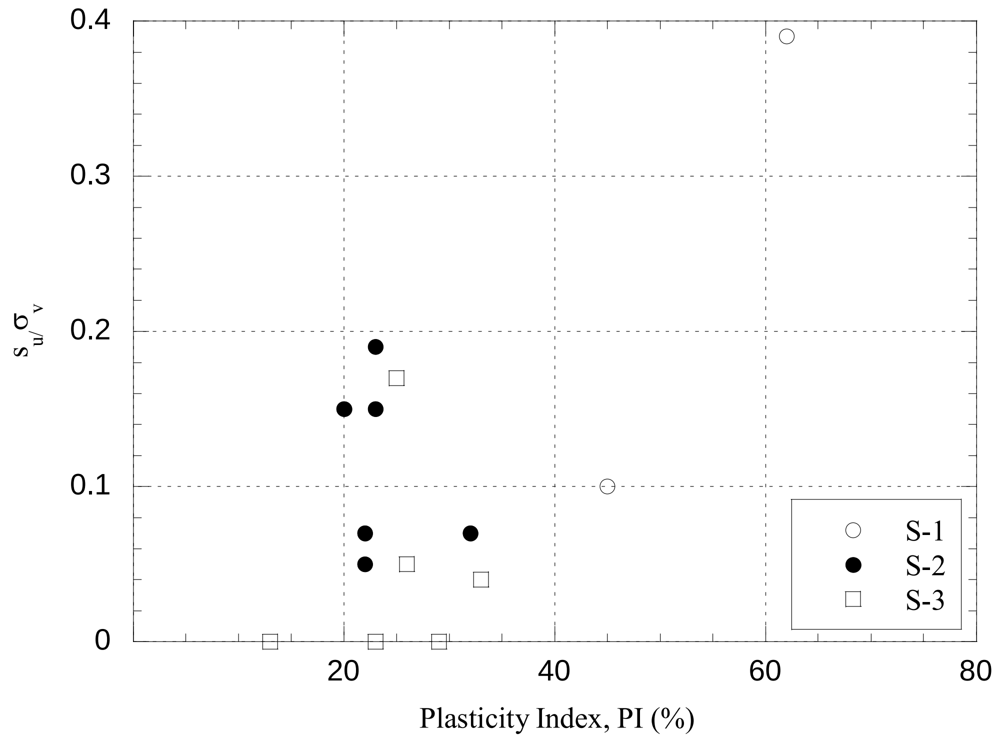


Figure 4.44 Variation of Plasticity index (PI) with s_u/σ_v

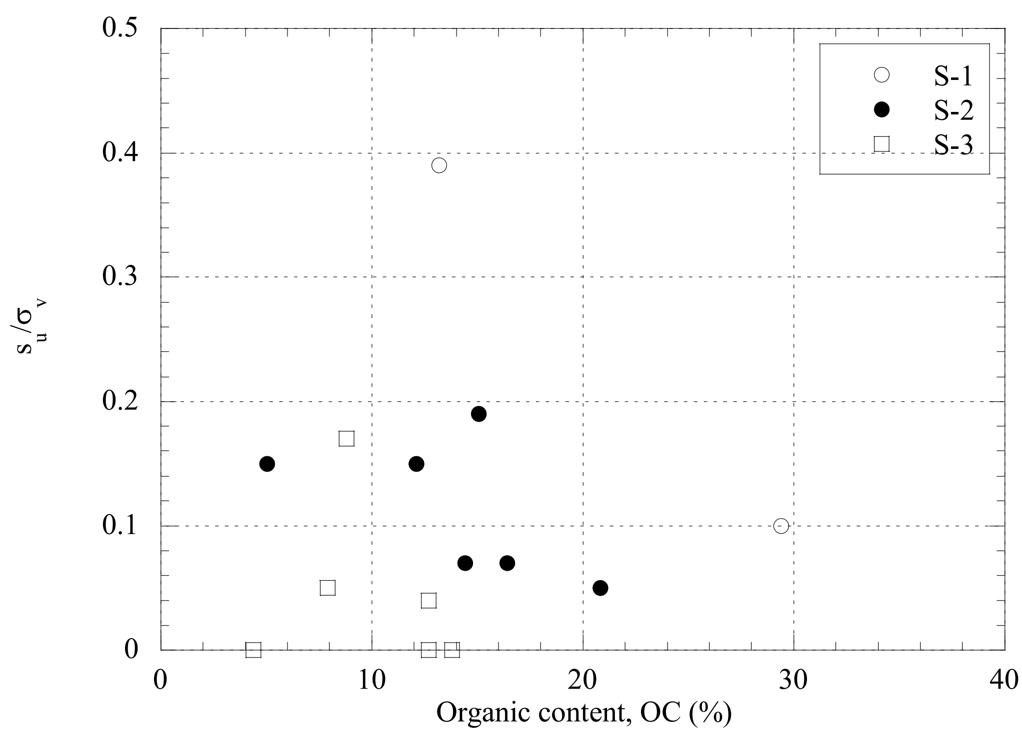


Figure 4.45 Variation of organic content (OC) with s_u/σ_v

From the Fig. 4.44, it is seen that s_u/σ_v and PI range varies from 0.04 to 0.39 and 20 to 40 %, respectively. In this figure, it is seen that no correlation exists between s_u/σ_v and PI. Correlation that available for inorganic soil cannot be used for this organic soil. Test must be conducted to know the shear strength.

(ii) s_u/σ_v Organic Content (OC)

Attempt has been taken to correlate s_u/σ_v with organic content (OC). Graphical presentation between s_u/σ_v and PI is presented in Fig. 4.45. From the figure, it is seen that data is scattered and no correlation exists. It is also seen that s_u/σ_v and OC range varies from 0.04 to 0.39 and 5 to 29 %, respectively.

Correlations between unconfined compressive strength (q_u) with other parameters

(i) q_u and SPT N-value

Attempts have been made to correlate q_u with uncorrected SPT N-value in this study. Relationship between q_u and SPT N-value has been presents in Fig. 4.46. It is observed that the range of the SPT N-value of the soft organic layer remain between 1 and 2. q_u varies from 6 to 66 kPa. Unconfined compressive strength is very low. No correlation exists between q_u and SPT N-Value in this study.

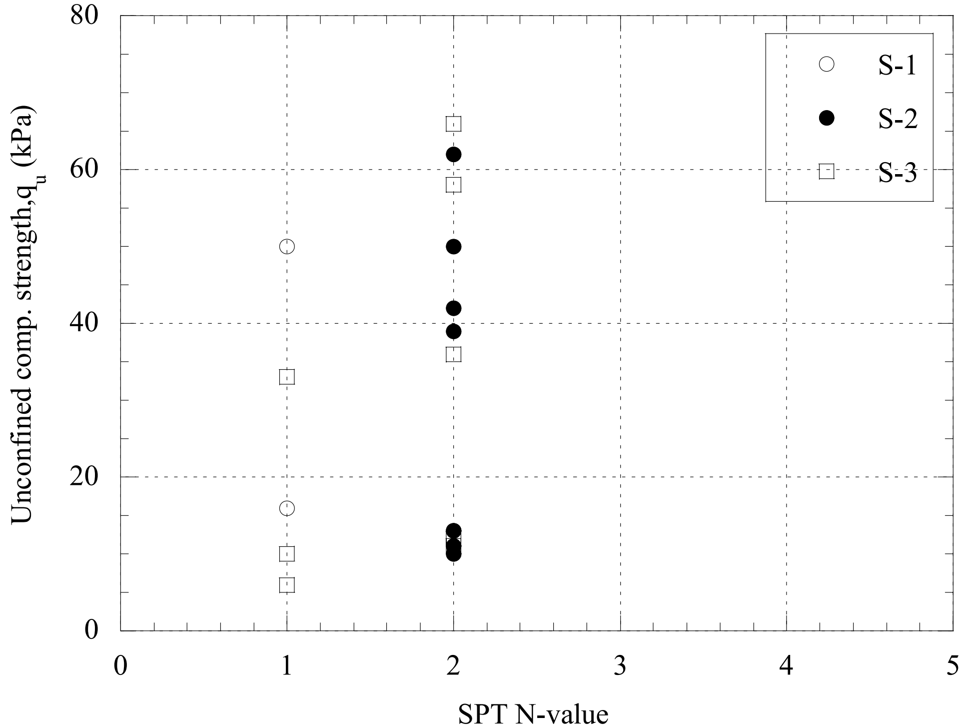


Figure 4.46 Correlation between unconfined compressive strength (q_u) and SPT N-value

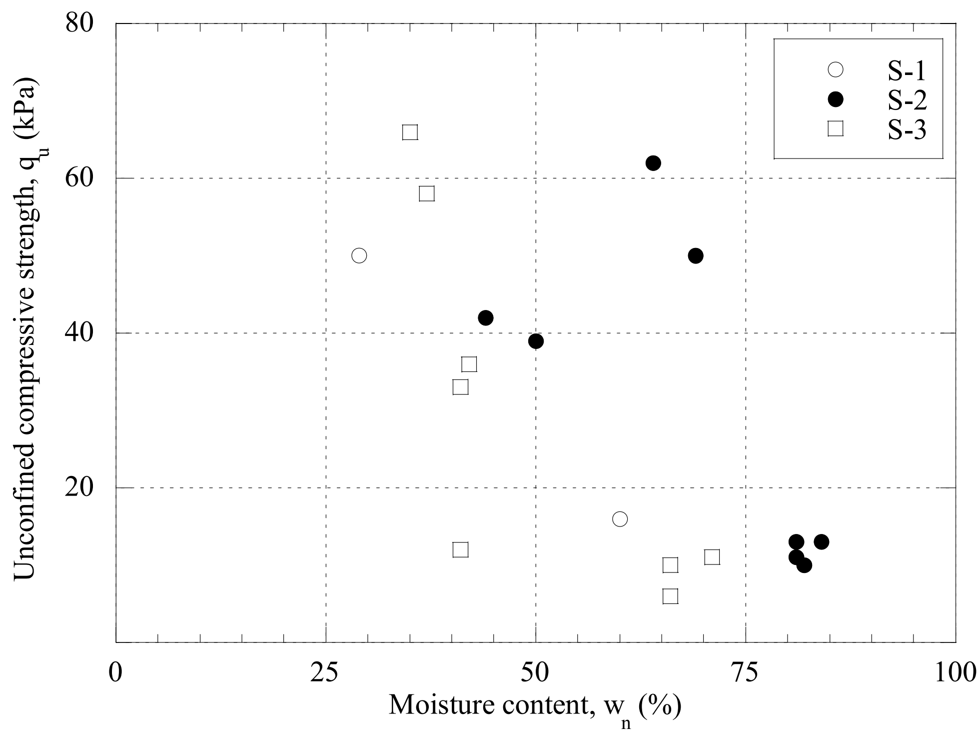


Figure 4.47 Variation of q_u with moisture content (w_n)

(ii) q_u and moisture content (w_n)

Attempts have been taken to correlate q_u with w_n of the soft organic layer. It has been presented in Fig. 4.47. From Fig. 4.47 it is seen that q_u decreases with the increase of w_n. But no correlation exists between unconfined shear strength (q_u) and moisture content (w_n).

(iii) q_u and Plasticity Index (PI)

Attempt has been taken to correlate q_u with PI of the organic soil collected from the study area (S-1, S-2 and S-3). It is presented in Fig. 4.48. From Fig. 4.48 it is seen that q_u increases with the increases of plasticity index. However, but no correlation exists between unconfined shear strength (q_u) and organic content (OC).

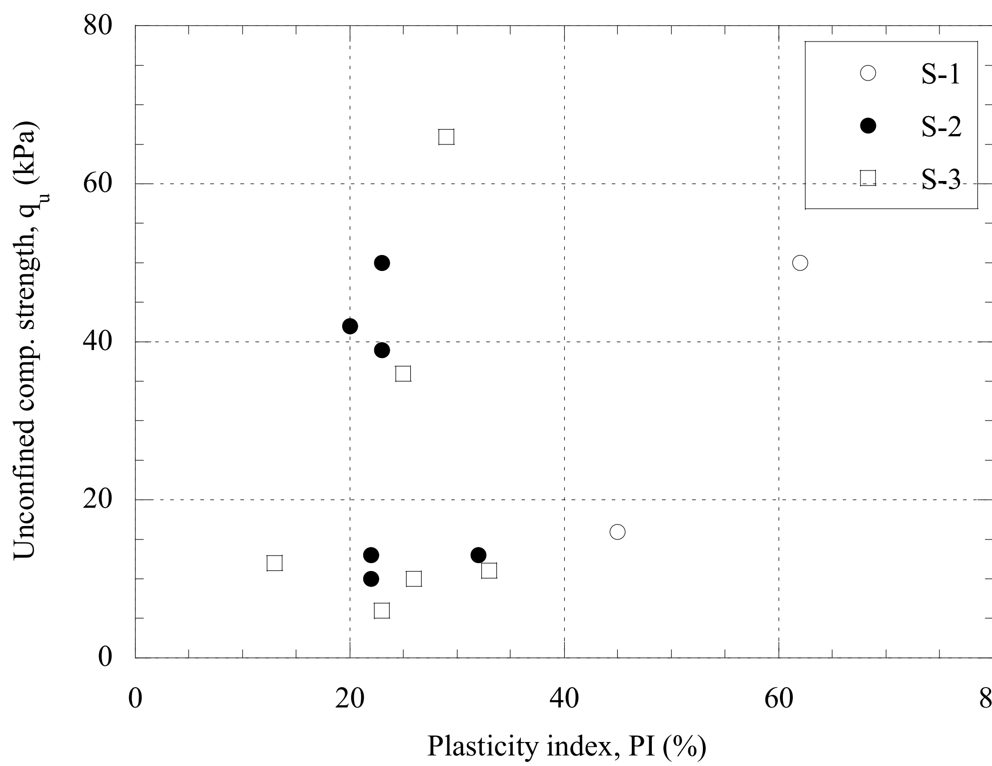


Figure 4.48 Correlation between unconfined compressive strength (q_u) and PI

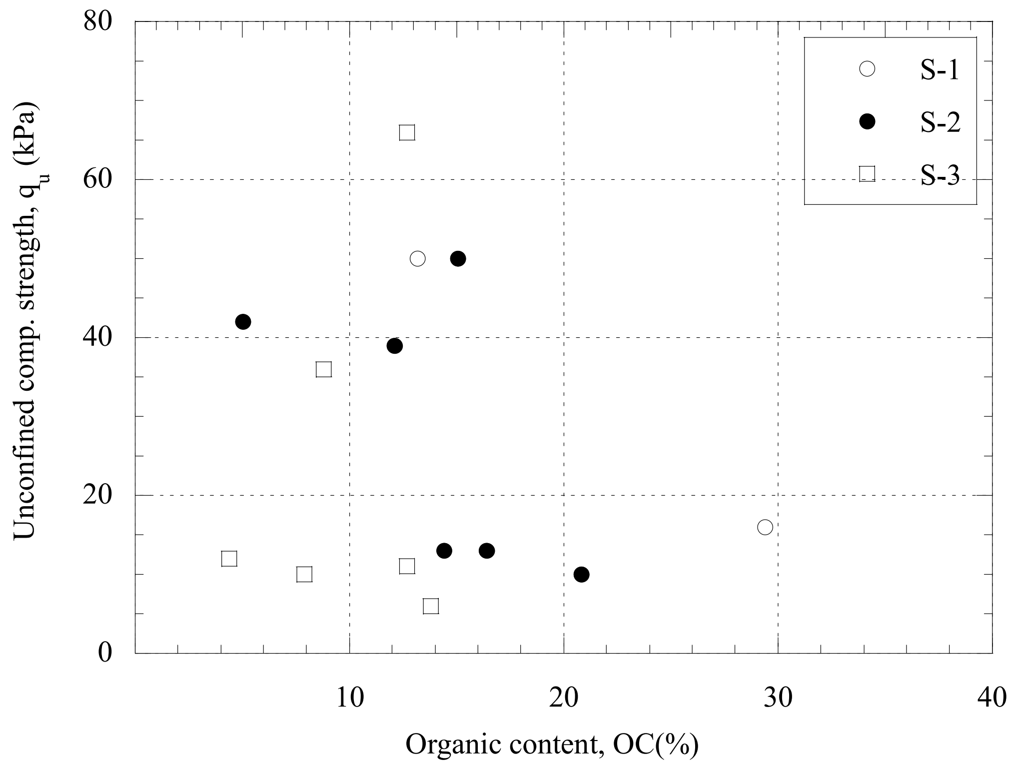


Figure 4.49 Correlation between unconfined compressive strength (q_u) and OC

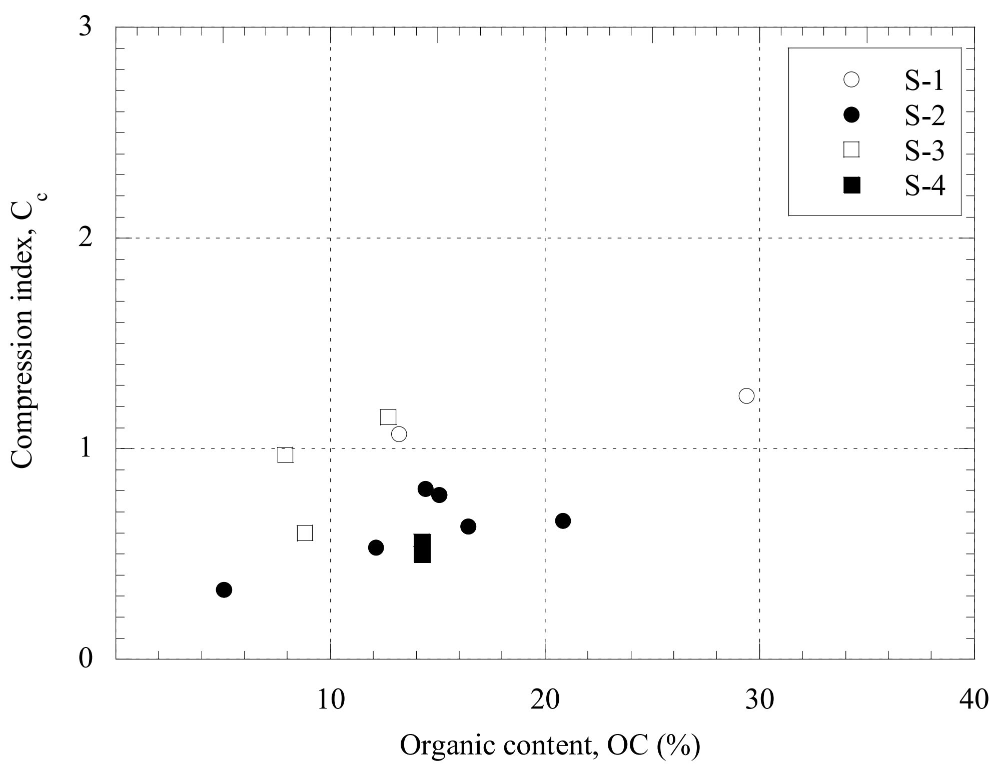


Figure 4.50 Correlation between Compression index (C_c) and OC

(iv) *q_u and Organic content (OC)*

Attempts have also been taken to correlate q_u with OC of this organic layer. It has been presented in Fig. 4.49. From Fig. 4.49 it seems that q_u decreases with the increases of organic content. However, the correlation is not clear.

Correlation with unconfined compression index (C_c) with others

(i) *C_c and Organic content (OC)*

Attempt has been made to correlate C_c with OC. Fig. 4.50 presents the variation of C_c with OC. The organic contents of soil sample collected from the organic layer have been found to vary from 5.0 to 29.4%. It is seen that C_c significantly increases with increases of organic content but no defined correlation exists. More study is needed to find out the relationship between C_c and OC.

(ii) *C_c and initial void ratio (e_o)*

Attempts have been made to correlate C_c with e_o in this study. Relationship between C_c and e_o have been presented in Fig. 4.51. It is seen that C_c increases with increases of e_o. The results obtained from this study have been compared with other relationships for different regional soil in Bangladesh.

Table 4.33 Correlations between C_c and e_o provided by different authors

Correlations	Soil types	References
$C_c = 1.15(e_o-0.35)$	Clays	Nishida (1956)
$C_c = 0.44(e_o-0.36)$	Fine-grained soils	Serajuddin and Ahmed (1967)
$C_c = 0.42(e_o-0.34)$	Costal soils	Amin et al. (1987)
$C_c = 0.25(e_o+0.194)$	Organic clay	Islam et al. (2004)

Note : e_o = Initial void ratio; C_c = Compressive index

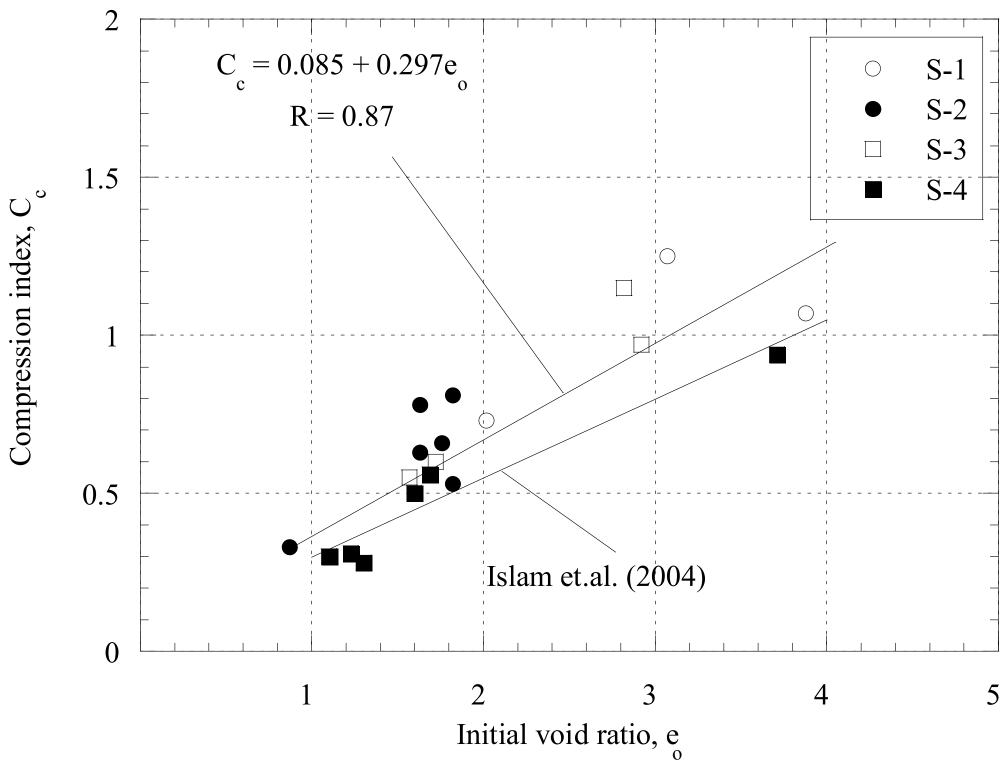


Figure 4.51 Correlation of Compression index (C_c) with Initial void ratio (e_o)

$$C_c = 0.30\ (e_o + 0.28),\ R = 0.87 \qquad (4.3)$$

where, C_c = compression index, e_o=Initial void ratio.

The correlations provided by different authors are listed in Table 4.33. The correlation that has been found from this study can be express by Eq. 4.3.

4.4 SETTLEMENT OF ORGANIC LAYER DUE TO FILLING LAYER

One-dimensional consolidation tests have been conducted on very soft organic clay collected from the layer beneath the filling. Compressibility properties (i. e., compression index, C_c; initial void ratio, e_o; coefficient of consolidation, c_v) have been estimated by using one-dimensional consolidation data. Typical borelog have been taken from areas S-1, S-2 and S-3, respectively. Total consolidation settlement and time dependent consolidation settlement of organic layer due to surcharge of the filling layer have been estimated for the settlement. The coefficient of consolidation (c_v) of samples have been computed using Eq. 4.4 (Das, 1983).

$$c_v = \frac{0.848 H^2}{t_{90}} \tag{4.4}$$

where, c_v = coefficient of consolidation; H = length of the maximum drainage path and t_{90} = time required for 90% consolidation. The estimated settlement has been presented in the followings.

Area S-1

In the study area S-1, consolidation settlement has been calculated at borehole location BH-1 (Fig. 4.52). From the graph, it is observed that depth of filling layer is 2 m. Here, c_v is equals to 0.45 m^2/yr for filling surcharge (Fig 4.7). In addition, depth of organic layer is 7 m. It has been found that 90% consolidation settlement is equals to 1734 mm. However, settlement time for single and double drainage has been found 106.0 and 26.5 years, respectively.

Area S-2

In the study area S-2, consolidation settlement has been calculated at borehole location BH-1 (Fig. 4.53). From the graph, it is observed that depth of filling layer is 5 m. Here, c_v is equals to 1 m^2/yr for filling surcharge (Fig 4.16). In addition, depth of organic layer is 5.5 m. It has been found that 90% consolidation settlement is equals to 668 mm. However, settlement time for single and double drainage has been found 30.5 and 8.0 years, respectively.

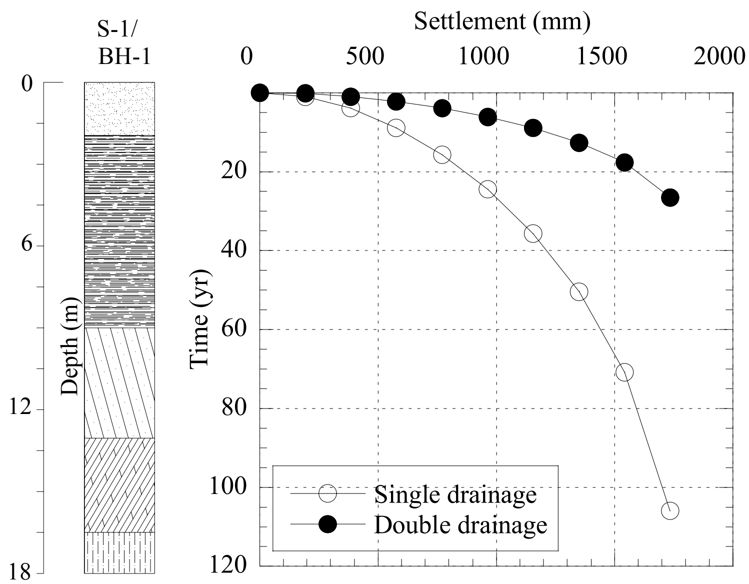


Figure 4.52 Settlement of soft organic layer due to filling layer of S-1

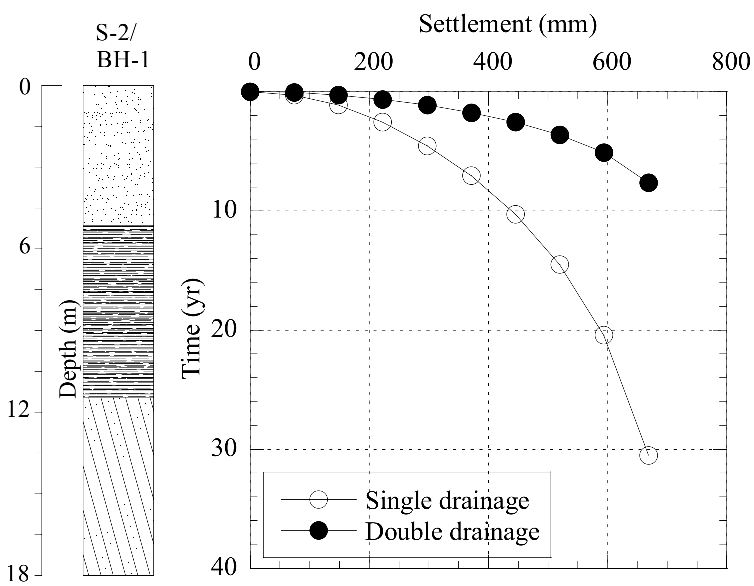


Figure 4.53 Settlement of soft organic layer due to filling layer of S-2

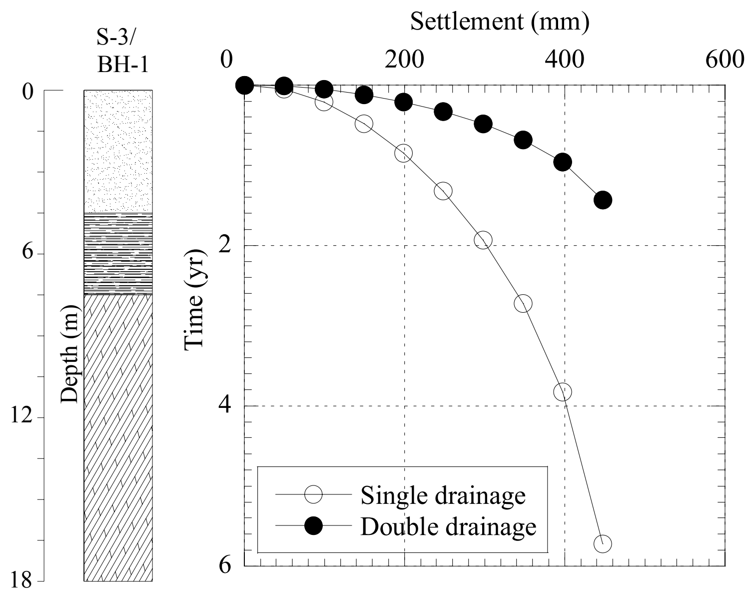


Figure 4.54 Settlement of soft organic layer due to filling layer of S-3

Area S-3

In the study area S-3, consolidation settlement has been calculated at borehole location BH-1 (Fig. 4.54). It is presented in Fig. 4.54. From BH-1, it is observed that depth of filling layer is 4.5 m. Here, c_v is equals to 0.75 m^2/yr for filling surcharge (Fig 4.25). In addition, depth of organic layer is 3 m. 90% consolidation settlement has been found equals to 447 mm. However, settlement time for single and double drainage has been found 6.0 and 1.4 years, respectively.

4.5 SUMMARY

The main objective of this study was to determine the sub-soil characteristics of the soft organic layer underlying the dredge fill layer and to suggest foundation alternatives for the areas having similar sub-soil condition. For this purpose, sub-soil investigations were carried out at six reclaimed areas in Dhaka city (S-1, S-2, S-3, S-4, S-5 and S-6). Among theses six sites, fourteen boreholes have been drilled at four locations (S-1, S-2, S-3 and S-4). Out of these fourteen boreholes, two boreholes were drilled at area S-1, three boreholes were drilled at area S-2, four boreholes were drilled at area S-3 and five boreholes were drilled at area S-4. Again, data of thirty two boreholes were collected for these locations. In these collected data of thirty two boreholes, data of twelve boreholes were for area S-1, five boreholes from area S-2, four boreholes from area S-3 and eleven boreholes from area S-4 were collected. For other two areas (i. e., S-5 and S-6) no borehole was drilled. But data of fourteen boreholes (eleven borehole locations for area S-5 and three borehole locations for area S-6) were collected from different organization (Dhaka Soil, BRTC, 2009 and BRTC, 2010). Based on this field and laboratory tests, the following conclusions/summary are obtained:

Generally, an organic clay or a sity clay or a dense sand layer exists under the filling layer in such areas. However, this study mainly focused on the areas having soft organic layer beneath the filling layer. The depth of filling sand varies from 1.5 to 7.5 m from Existing Ground Level (EGL). The uncorrected SPT N- value of this filling sand varies from 1 to 11. Soft organic clay layer exists from 4.0 to 18.0 m from EGL. The uncorrected SPT N- value of this organic clay varies from 1 to 4. Beneath this soft organic layer, a soft silty clay and dense sand have been found up to 18 m from EGL. The observed groundwater table is 0.6 to 7 m below from EGL. However, this groundwater table is trapped water.

The mean grain size (D_{50}) and fine content (F_c) of filling sand varies from 0.015 to 0.210 mm and 12.0 to 30.7%, respectively. Beneath this filling layer, a very soft organic clay layer has been found with high moisture content (29 to 225%) and low dry unit weight (3.3 to 12.1 kN/m^3). Organic content of this layer found to vary from 5.0 to 31.8%. This soil has been classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils. LL and PI of the soft organic clay vary widely from 48 to 190% and 20 to 130%, respectively. It has been observed that this soft organic layer is highly plastic and similar with that of Khulna organic soil (Ferdous, 2007).

It is found that the shear strength of the soft organic soil is very low (2 to 30.5 kPa). It is seen that consistency of this soil is very soft to soft according to BS 5930 (Bowels, 1982). The values of shear strength properties have been compared with that of Dhaka organic soil and found to be similar (s_u = 13~19 kPa; Islam et al., 2004).

One-dimensional consolidation tests have been conducted on organic clay collected from the layer beneath the filling. Initial void ratio (e_o) and compression index (C_c) of soft organic layer have been found to vary from 0.87 to 5.51 and 0.28 to 1.93, respectively. It have been found that this layer is highly compressible. However, this soil is less compressible in comparison with that of organic soil of Khulna (C_c = 1.95 to 4.01; Ferdous, 2007). But this soil is more compressible than that of organic soil of Dhaka (Islam et al., 2004).

Attempts has been made to correlate unconfined compressive strength (q_u) and SPT N-value, plasticity index (PI), and organic content (OC). Unconfined compressive strength increases with the increase of plasticity index and decreases with the increase of organic content. However, no correlation was found among the properties. It therefore appears that tests must be conducted to know the shear strength of the soft organic soil for each project. Attempts have also been taken to correlate compression index (C_c) with OC and initial void ratio (e_o). C_c increases with the increase of OC and e_o. But in this case also no definite correlation exists. Tests should be conducted to know the compression index of the soil. The relation of the compression index (C_c) and initial void ratio (e_o) that have been found in this study is, $C_c = 0.30\ (e_o + 0.28)$; where R=0.87.

Total consolidation settlement and time dependent consolidation settlement of organic layer due to the surcharge of the filling have been estimated for three different study areas (i. e., S-1, S-2 and S-3). It has been found that total consolidation settlement varies from 447 to 1734 mm. Time for single drainage varies from 6 to 106 years whereas it has been found to vary from 1.4 to 26.5 years for double drainage condition.

Chapter FIVE

SUGGESTED ALTERNATIVE FOUNDATION SYSTEMS

5.1 GENERAL

This chapter includes general sub-soil characteristics of the study areas and different suitable options for relevant foundation systems for the study area. Relevant foundation systems (i. e., spread foundation with Rammed Aggregate Pier, buoyancy raft and pile foundation) are described briefly in this chapter.

5.2 GENERAL SUB-SOIL CHARACTERISTICS OF STUDY AREA

The main objective of this study was to determine the sub-soil characteristics of the soft organic layer underlying the dredge fill layer. And to suggest foundation alternatives for the areas having similar sub-soil condition. For this purpose, sub-soil investigations were carried out at six reclaimed areas in Dhaka city (S-1, S-2, S-3, S-4, S-5 and S-6). Among theses six sites, fourteen boreholes have been drilled at four locations (S-1, S-2, S-3 and S-4). And data of thirty two boreholes were collected for these locations. For other two areas (i. e., S-5 and S-6) no borehole was drilled. But data of fourteen boreholes were collected from different organization (Dhaka Soil, BRTC, 2009 and BRTC, 2010). Based on this field and laboratory tests, the following conclusions/summary are obtained:

Generally, an organic clay or a sity clay or a dense sand layer exists under the filling layer in such areas. However, this study mainly focused on the areas having soft organic layer beneath the filling layer. The depth of filling sand varies from 1.5 to 7.5 m from Existing Ground Level (EGL). The uncorrected SPT N- value of this filling sand varies from 1 to 11. Soft organic clay layer exists from 4.0 to 18.0 m from EGL. The uncorrected SPT N- value of this organic clay varies from 1 to 4. Beneath this soft organic layer, a soft silty clay and dense sand have been found up to 18 m from EGL. The observed groundwater table is 0.6 to 7 m below from EGL. However, this groundwater table is trapped water. Typical borlogs of study areas have been presented in Fig. 5.1.

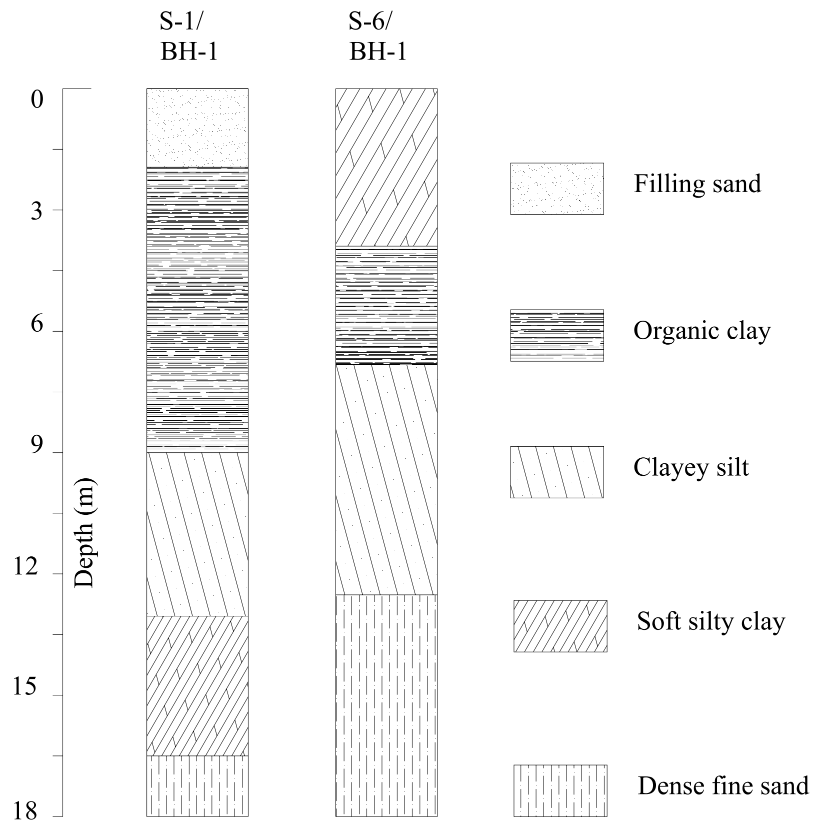


Figure 5.1 Typical borelog of study areas

The mean grain size (D_{50}) and fine content (F_c) of filling sand varies from 0.015 to 0.210 mm and 12.0 to 30.7%, respectively. Beneath this filling layer, a very soft organic clay layer has been found with high moisture content (29 to 225%) and low dry unit weight (3.3 to 12.1 kN/m^3). Organic content of this layer found to vary from 5.0 to 31.8%. This soil has been classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils. LL and PI of the soft organic clay vary widely from 48 to 190% and 20 to 130%, respectively. It is seen that this soft organic layer is highly plastic and similar with that of Khulna organic soil (Ferdous, 2007).

It is found that the shear strength of the soft organic soil is very low (2 to 30.5 kPa). It is seen that consistency of this soil is very soft to soft according to BS 5930 (Bowels, 1982). The values of shear strength properties have been compared with that of Dhaka organic soil and found to be similar (s_u = 13~19 kPa; Islam et al., 2004).

One-dimensional consolidation tests have been conducted on organic clay collected from the layer beneath the filling. Initial void ratio (e_o) and compression index (C_c) of soft organic layer have been found to vary from 0.87 to 5.51 and 0.28 to 1.93, respectively. It is seen that this layer is highly compressible. However, this soil is less compressible in comparison with that of organic soil of Khulna (C_c = 1.95 to 4.01; Ferdous, 2007). But this soil is more compressible than that of organic soil of Dhaka (Islam et al., 2004).

5.3 FOUNDATION ALTERNATIVES

In these sub-soil profiles, it is seen that top filling layer is non-plastic fine sand. Beneath the filling layer, a very soft and highly compressible organic layer has been found. This organic layer may put adverse effect on foundation that having through or on it. The main problem that will take an action is excessive settlement and deferential settlement for shallow foundation as well as negative skin friction for deep foundation. With these considerations, different suitable options for relevant foundation systems for the study area have been described below.

Spread footing with Rammed Aggregate Pier (RAP)

Spread footing with RAP may be one of the best foundation systems for small to medium tall building in these study areas. In this method a great portion below the footing area (35% or more area) of the organic clay is replaced by the crushed stone that provides additional load carrying capacity.

Typical RAP and the surrounding soil properties within the study area have shown in Fig. 5.2. This system is suitable in our study areas because of the following reasons:

a) Organic layer is covered by compacted sand. This filling layer is stiffer than the organic layer.
b) Organic soil is very soft in constancy. Therefore, it will make strong soil matrix bond with RAP.
c) It is found that in most of the cases the depth of organic layer is not greater than 10 m from existing ground level (EGL).

d) Less compressible soil layer exists beneath the organic layer.

e) This method can be used in the construction area without using shore pile. Thus, it is very economical.

A typical settlement calculation for spread footing with Rammed Aggregate Pier footing for the surcharge of the six storied residential building has been shown in appendix. After considering the worst soil profile it is seen that in the study area S-2, RAP foundation will not be applicable (organic layer depth more than 10 m from EGL). Settlement has been found 20 mm for four storied residential building. Whereas, for the same soil condition settlement is 31 mm for six storied residential building. As per BNBC (1993), the allowable range of the soil settlement is 25 mm. It is seen that this foundation system will be more efficient for medium rise building (up to four stories). A typical settlement calculation of this foundation has been shown in Appendix- B.

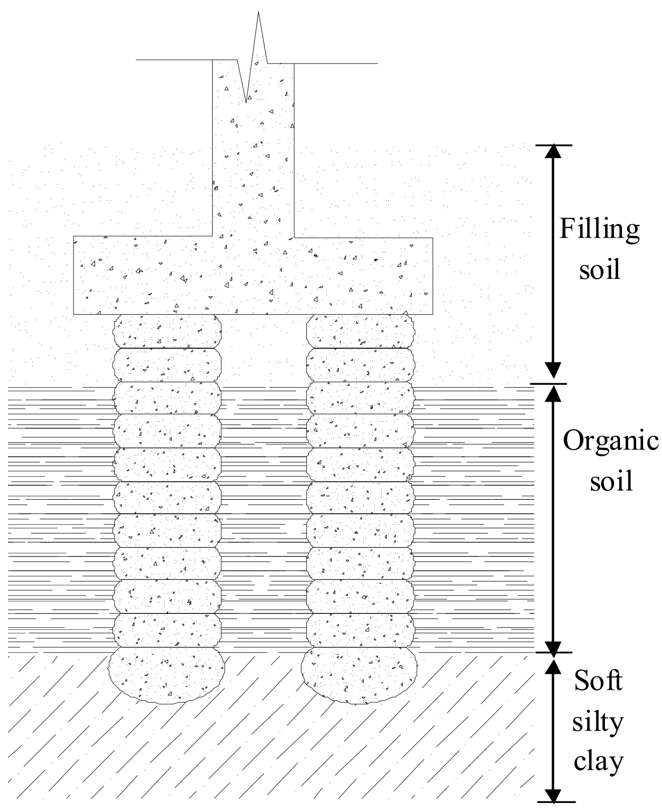


Figure 5.2 Typical Spread footing with Rammed Aggregate Pier (RAP)

Buoyancy Raft Foundation System

Buoyancy Raft Foundation System can be more economical for small to medium rise building for the study areas where basement is required. In general, depth of organic layer of the study is not too large. Therefore, differential settlement can be reduced by providing buoyancy raft foundation.

Total settlement has been estimated for buoyancy raft foundation system in four study areas (S-1, S-2, S-3 and S-4). In every case, a typical six storied residential building has been considered. Within all borehole locations in a single study area, the worst soil condition has been taken to estimate the settlement of the raft foundation. Typical calculation has been shown in appendix and summary is presented in Table 5. 1. It is seen that, for the study area S-2, S-3 and S-4, settlement is negligible with considering one basement. It is also seen that, for the study area S-1, one basement is not enough for balancing the allowable settlement limit (50 mm as per BNBC, 1993). In this case, more than two basements are required for balance.

Table 5.1 Total estimated settlements (for six storied residential building) in the four study areas (S-1, S-2, S-3 and S-4) considering buoyancy raft foundation system.

Study area	Settlement (mm)	Depth of raft (m)	Remarks
S-1	678	3.1	Basement needed more than two
S-2	0	3.1	Single basement is enough
S-3	0	3.1	Single basement is enough
S-4	8	3.1	Single basement is enough

Pile Foundation

Pile foundation may be used to transmit the super structure load to the firm strata. It is most common and widely used all over the world for its availability. Both pre-cast and cast-in-situ piles can be applicable for the study areas. However, in these study area, negative skin friction may be taken an action that can be reduced pile-bearing capacity. Therefore, special attention may be taken for reducing negative skin friction. The suitable attempts that can be reduced negative skin friction are given below.

a) Bituminous coatings can be applied on surface area that may be subjected to negative skin friction. This method can be applied only for precast and end bearing piles.

b) Piles can be designed with additional load (equals to negative skin friction) or higher factor of safety.

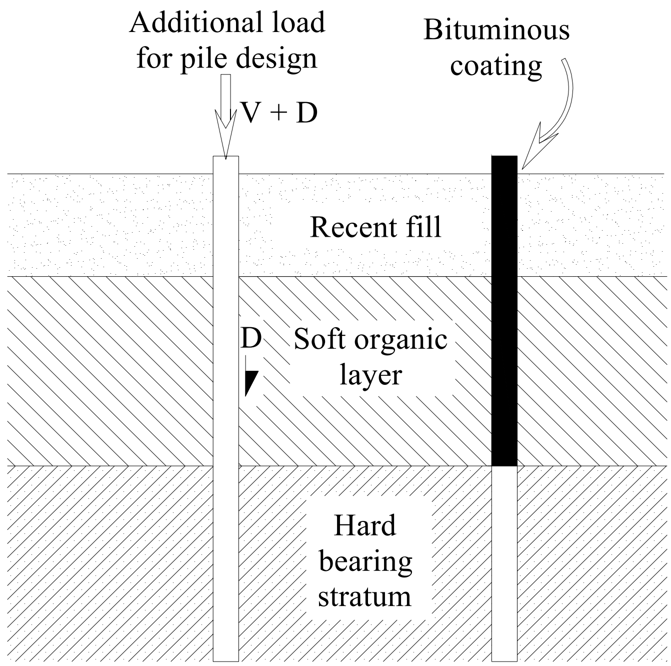


Figure 5.3 Techniques to reduce negative skin friction of pile

Table 5.2 Comparisons of pile length with considering negative skin friction and without considering negative skin friction.

Study area/ BH No.	Pile length (m)	
	Without considering negative skin friction	Considering negative skin friction
S-1/ BH-1	19.7	20.3
S-2/ BH-2	16.2	17.0
S-3/ BH-1	22.3	23.0
S-4/ BH-5	13.7	14.5

Other option for reducing negative skin friction is to decrease the pile diameter. However, for our study areas the slenderness ratio (L/d, L = length of pile and d = diameter of pile) of pile may come high that will cause buckling of the pile. Therefore, this option has dropped from consideration. Figure 5.3 presents two techniques that will reduce negative skin friction.

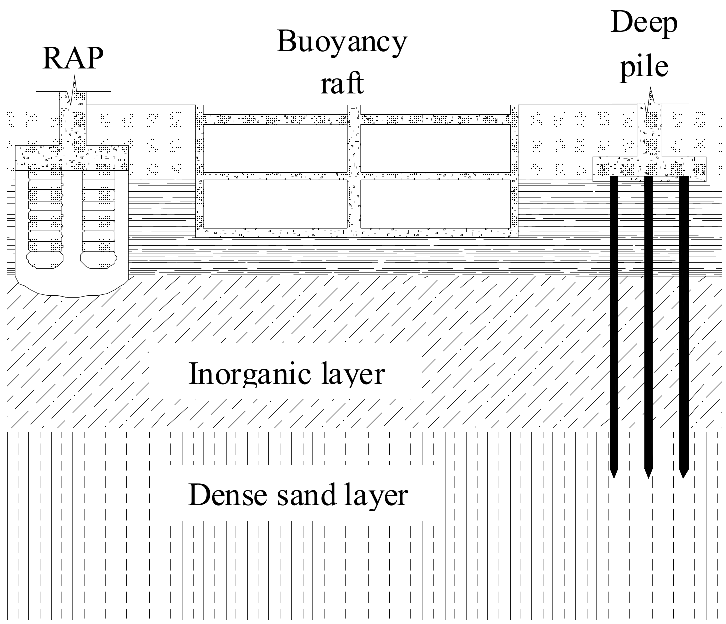


Figure 5.4 Suggested foundation systems for the study area

In this study, attempts have been taken to show the comparisons of pile length with considering negative skin friction (NSF) and without considering NSF (Table 5.2). Within all borehole locations in a single study area, the worst soil condition has been taken to estimate settlement of pile foundation. Pile length has been estimated considering as driven pile. From the Table 5.2, it is seen that, when negative skin friction has been taken into consideration, the pile length increases. Therefore, for safety purpose, extra additional load (equals to negative skin friction) should be included with design load while pile foundations are designed.

5.4 SUMMARY

Total six reclaimed areas within Dhaka city have been considered in this study. The general sub-soil profile of this area is filling sand overlying soft organic clay layer. This filling sand is non-plastic sand. Among them, filling clay layer have also been found in two places.

The underlying organic layer is soft in consistency. Moisture content and organic content of this layer varies from 29 to 225% and 5.0 to 31.8%. The OC range indicates that this soil may be classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils. This layer is highly plastic and highly compressible. LL and PI of organic clay layer vary from 48 to 190% and 20 to 130%, respectively. Unconfined compressive strength of organic clay varies from 2 to 30.5 kPa. It has been found that initial void ratio and compression index of soft organic layer vary from 0.87 to 5.51 and 0.28 to 1.93, respectively. Detail test results are presented in chapter four.

Suggested foundation systems for study areas are shown in Fig. 5.4. Spread footing with RAP may be one of the best foundation system for small to medium rise building in this study area. Buoyancy raft system also can be more economical for small to medium rise for the study areas where basement is required. Pile foundation may be also used for the tall building and in this case negative skin friction should be considered in design.

Chapter SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

The main objective of this study was to determine the sub-soil characteristics of the soft organic layer underlying the dredge fill layer. And to suggest foundation alternatives for the areas having similar sub-soil condition. The general sub-soil characteristics of study areas, correlation, soil settlement due to filling and relevant foundation systems and recommendation for future study have been described in this chapter.

6.2 GENERAL SUB-SOIL CHARACTERISTICS OF STUDY AREA

Sub-soil investigations were carried out at six reclaimed areas in Dhaka city (S-1, S-2, S-3, S-4, S-5 and S-6). Among theses six sites, fourteen boreholes have been drilled at four locations (S-1, S-2, S-3 and S-4). And data of thirty two boreholes were collected for these locations. For other two areas (i. e., S-5 and S-6) no borehole was drilled. But data of fourteen boreholes were collected from different organization (Dhaka Soil, BRTC, 2009 and BRTC, 2010). Based on this field and laboratory tests, the following conclusions/summary are obtained:

Generally, an organic clay or a sity clay or a dense sand layer exists under the filling layer in such areas. However, this study mainly focused on the areas having soft organic layer beneath the filling layer. The depth of filling sand varies from 1.5 to 7.5 m from Existing Ground Level (EGL). The uncorrected SPT N- value of this filling sand varies from 1 to 11. Soft organic clay layer exists from 4.0 to 18.0 m from EGL. The uncorrected SPT N- value of this organic clay varies from 1 to 4. Beneath this soft organic layer, a soft silty clay and dense sand have been found up to 18 m from EGL. The observed groundwater table is 0.6 to 7 m below from EGL. However, this groundwater table is trapped water. Typical borlogs of study areas are presented in Fig. 5.1.

The mean grain size (D_{50}) and fine content (F_c) of filling sand varies from 0.015 to 0.210 mm and 12.0 to 30.7%, respectively. Beneath this filling layer, a very soft organic clay layer has been found with high moisture content (29 to 225%) and low dry unit weight (3.3 to 12.1 kN/m^3). Organic content of this layer found to vary from 5.0 to 31.8%. This soil has been classified according to ASTM (Edil, 1997) as organic silts and clays to silty or clayey organic soils. LL and PI of the soft organic clay vary widely from 48 to 190% and 20 to 130%, respectively. It is seen that this soft organic layer is highly plastic and similar with that of Khulna organic soil (Ferdous, 2007).

It has been found that the shear strength of the soft organic soil is very low (2 to 30.5 kPa). It can be seen that consistency of this soil is very soft to soft according to BS 5930 (Bowels, 1982). The values of shear strength properties were compared with that of Dhaka organic soil and found to be similar (s_u = 13~19 kPa; Islam et al., 2004).

One-dimensional consolidation tests have been conducted on organic clay collected from the layer beneath the filling. Initial void ratio (e_o) and compression index (C_c) of soft organic layer have been found to vary from 0.87 to 5.51 and 0.28 to 1.93, respectively. It has been found that this layer is highly compressible. However, this soil is less compressible in comparison with that of organic soil of Khulna (C_c = 1.95 to 4.01; Ferdous, 2007). But this soil is more compressible than that of organic soil of Dhaka (Islam et al., 2004).

6.3 CORRELATIONS

Attempt has been made to correlate unconfined compressive strength (q_u) with uncorrected SPT N-value in this study. It is observed that the range of the SPT N-value of the soft organic layer remain between 1 and 2. Unconfined compressive strength (q_u) varies from 6 to 66 kPa. Unconfined compressive strength is very low. No correlation exists between q_u and SPT N-Value. It is seen that shear strength could not be estimated by SPT N- value for this soft organic clay. Tests must be conducted to know the shear strength of the soft organic soil for each project.

Similarly, attempt has been taken to correlate q_u with plasticity index (PI) for the organic layer. It is seen that q_u increases with the increase of plasticity index. However, no defined correlation exists.

Again, attempt has also been taken to correlate q_u with OC of the organic layer. It is seen that q_u decreases with the increase of organic content. However, the correlation is not clear.

Similarly, another attempt has also been made to correlate compression index (C_c) with OC. It is found that C_c increases with increase of organic content. But in this case also no defined correlation exists. Therefore, sample must be tested to know the compression index of the soil.

Finally, attempt has been made to correlate C_c with initial void ratio (e_o). It is seen that C_c increases with the increase of e_o. The correlation that has been found from this study can be express by Eq. 6.1.

$$C_c = 0.30\,(e_o + 0.28),\ R = 0.87 \qquad (6.1)$$

where, C_c = compression index, e_o=Initial void ratio.

6.4 SETTLEMENT DUE TO FILLING

One-dimensional consolidation tests have been conducted on samples collected from very soft organic layer. Total consolidation settlements and time dependent settlements of organic layer due to the surcharge from the filling layer have been estimated. Study areas S-1 S-2 and S-3 have been selected for this calculation. Depth of filling layer of study areas have been found to vary from 2 to 5 m. In addition, depth of organic layer varies from 3 to 7m.

Table 6.1 presents the summary of consolidation settlement and settlement time. Settlements have been estimated due to the surcharge of the filling for three study areas (S-1, S-2 and S-3). It is seen that, total consolidation settlement of study area varies from 447 to 1734 mm. This settlement occurs in several years. It has been observed that settlement time for single drainage varies from 6 to 106 years. However, time for double drainage varies from 1.4 to 26.5 years.

Table 6.1 Consolidation settlement and settlement time due to the surcharge of the filling for different study area (S-1, S-2 and S-3)

Location	Total settlement (mm)	Settlement time, year (for 90% consolidation)	
		Single drainage	Double drainage
S-1	1734	106.0	26.5
S-2	668	30.5	8.0
S-3	447	6.0	1.4

6.5 FOUNDATION ALTERNATIVES FOR STUDY AREA

Spread footing with Rammed Aggregate Pier (RAP) may be one of the best foundation system for medium rise building in the study areas. Since, it is seen that in most of the cases, the depth of organic layer is not greater than 10 m from EGL.

Buoyancy Raft Foundation System can be more economical for small to medium rise building for the study areas. It should be in consideration that raft foundation should be deep (generally 3.1 m).

Pile foundation may be employed in every case for tall building for the study area. However, negative skin friction should be in consideration.

6.6 RECOMMENDATIONS FOR FUTURE STUDY

a) Ground improvement techniques were not included in this study. Proper ground improvements technique should be investigated for this type of soil.

b) Actual settlement of this soft layer can be monitored by using settlement plate or other suitable method.

c) Performance of recommended foundation may be studied in the laboratory and/or field.

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Appendix-A

ESTIMATION OF NEGATIVE SKIN FRCITION

Example of negative skin friction from euro code 2007

Pile diameter = 300 mm

Soft layer thickness = 5 m

Negative skin friction = 20 kPa (assumption)

End bearing = 50 kPa (assumption)

Vertical load on pile = 300 KN

Penetration depth = F_d = ?

Characteristics value of force

Characteristics applied load = V_k = 300KN

Characteristics downdrag force = $D_k = \pi D L_D q_{DK}$ (A.1)

$= 3.14 \times 0.3 \times 5 \times 20 = 94.2$ KN

Characteristics shaft resistance = $R_k = \pi \times 0.3 \times 50 \times L_R = 47.1 \times L_R$ kN

Case C1 – Downdrag Force (D) Taken Action

Partial factors for action:

Vertical load, V: $\gamma_G = 1.0$ (Table A.2)

Downdrag, D: $\gamma_G = 1.0$ (Table A.2)

Downdrag is classified as a 'permanent' action because its variation is always in the same direction (monotonic) until the action attentions a certain limit value.

Partial factor for resistances:

Shaft resistance, R: $\gamma_S = 1.3$

Total design load = $F_d = V_d + D_d$ (A.2)

$= V_k \times \gamma_G + D_k \times \gamma_G$ (A.3)

$= 300 \times 1.0 + 94.2 \times 1.0$

$= 394.2$ kN

Design shaft resistance = $R_d = R_k / \gamma_S$ (A.4)

$= 47.1 \times L_R / 1.3$

Require $R_d \geq F_d$

Hence $47.1 \times L_R / 1.3 \geq 394.2$ kN

So, $L_R \geq 10.88$ m

Design force for concrete shaft = $F_d = 394.2$ kN.

Case C2 – Settlement Taken as Action

Partial factors for action:

Vertical load, V: $\gamma_G = 1.0$ (Table A.2)

Settlement: $\gamma_G = 1.0$ (Table A.2)

Partial factor for resistances:

Shaft resistance, R: $\gamma_S = 1.3$

When downdrag force was taken an action, in case C1, a factor of 1.0 was applied to it. However, when the settlement was taken to action its effects to transferred to the pile using the soil strength, which therefore act in an unfavorable manner. EC7, 2.4.2(11) say that a partial factor less than 1 must be applied in such cases, but a more precise value is not given. It could be taken to be $1/\ \gamma_s$ from Table 7.2 or $1/\ \gamma_{cu}$ from Appendix-A. This would give $1/\ 1.3 = 0.769$ or $1/1.4 = 0.714$, respectively. A compromise value of $\gamma_D = 0.75$ will be used here.

Then, $D_d = 94.2/\ \gamma_D$

$= 94.2/\ 0.75$

$= 125.7$ kN.

Total design load = $F_d = V_d + D_d = V_k \times \gamma_G + D_k \times \gamma_G$

$= 300 \times 1.0 + 94.2 \times 125.7$

$= 425.7$ kN

Design shaft resistance = $R_d = R_k / \gamma_S$

$= 47.1 \times L_R / 1.3$

Require $R_d \geq F_d$

Hence $47.1 \times L_R / 1.3 \geq 425.7$ kN

So, $L_R \geq 11.75$ m

Design force for concrete shaft = F_d = 425.7 kN.

Case B1 – Downdrag Force (D) Taken Action

Partial factors for action:

Vertical load, V: $\gamma_G = 1.35$

Downdrag, D: $\gamma_G = 1.35$ (Table A.2)

Partial factor for resistances:

Shaft resistance, R: $\gamma_S = 1.0$ (Table 7.2)

Total design load = $F_d = V_d + D_d$

$= V_k \times \gamma_G + D_k \times \gamma_G$

$= 300 \times 1.35 + 94.2 \times 1.35$

= 532.2 kN

Design shaft resistance = $R_d = R_k / \gamma_S$

$= 47.1 \times L_R / 1.0$

Require $R_d \geq F_d$

Hence $47.1 \times L_R / 1.3 \geq 532.2$ kN

So, $L_R \geq 11.29$ m

Design force for concrete shaft = F_d = 532.2 kN.

Case C2 – Settlement Taken as Action

Partial factors for action:

Vertical load, V: $\gamma_G = 1.35$ (Table A.2)

Any partial factor applied to settlement would have been no effect in this case.

Partial factor for resistances:

Shaft resistance, R: $\gamma_S = 1.0$ (Table 7.2)

Partial factor for unfavorable soil strength transmitting effect to settlement to pile = 1.0.

Hence design downdrag force = D_d =94.2 kN.

Total design load = $F_d = V_d + D_d$

$= V_k \times \gamma_G + D_d$

$= 300 \times 1.35 + 94.2$

$= 499.2$ kN

Design shaft resistance = $R_d = R_k / \gamma_S$

$= 47.1 \times L_R / 1.0$

Require $R_d \geq F_d$

Hence $47.1 \times L_R / 1.0 \geq 499.2$ kN

So, $L_R \geq 10.6$ m

Design force for concrete shaft = $F_d = 499.2$ kN.

Table A.1 Summary of results of negative skin friction

Conclusion	L_R (m)	F_d (kN)
Case C1 – Downdrag force taken as action	10.88	394.2
Case C2 – Settlement taken as action	11.75	425.7
Case B1 – Downdrag force taken as action	11.29	532.2
Case B2 – Settlement taken as action	10.60	499.2

The results of the calculations are summarizes in Table A.1. It is necessary to satisfy both cases B and C, but the choice of the force or displacement as the action is open to the designer. A pile with a penetration into the bearing stratum of L_r=10.88 m structural capacity of F_d=499.2 kN would therefore comply with the code.

The combination of L_r and F_d is derived from calculations C1 and B1, which are not consistence in their use of dragdown force or settlement as the action. However, both calculations C1 and C2, for example, are sufficient but not necessary, and similarly B1 and B2. Hence a design which conforms with C1 and B2 is acceptable.

The Case B2 calculation has a factor of 1.0, in effect, on the negative skin friction effect. The code may be open to criticism at this point.

The results for Case B1 and C1 illustrate a solution which will frequently arise for shaft controlled piles subject to permanent load. Because the factor on the shaft resistance [1.3] is less than the load factor of permanent laods [1.35], Case B requires a longer pile than Case C. The problem does not occirs when variable loads dominate because of the additional factor of 1.3 in Appendix-A for variable loads in Case C, compared with 1.5 in Case B ($\gamma q \times \gamma s = 1.3 \times 1.3$ for Case C>1.5×1.0 for Case B). This inconvenient could be removed from EC7 by small adjustment to the partial factors and ζ values.

Table A.2[Eurocode-2007]

Case 1)	Action	Symbol	Situations	
			P/T	A
Case A Loss of static equilibrium; Strength of structural material or ground insignificant	Permanent actions: self weight of structural and non-structural components, permanent actions caused by ground, groundwater and free water -unfavorable -favorable Variable actions -unfavorable Accidental actions	γ_{Gsup} 4) γ_{Ginf} 4) γ_Q γ_A	(1.10) 2) (0.90) 2) (1.50)	(1.00) (1.00) (1.00) (1.00)
Case B 5) Failure of structure or structural elements, including those of the footing, piles, basement walls, etc., governed by strength of structural material	Permanent actions 6) -unfavorable -favorable Variable actions -unfavorable Accidental actions	γ_{Gsup} 4) γ_{Ginf} 4) γ_Q γ_A	(1.35) 3) (1.00) 3) (1.50)	(1.00) (1.00) (1.00) (1.00)
Case C 5) Failure in the ground	Permanent actions -unfavorable -favorable Variable actions -unfavorable Accidental actions	γ_{Gsup} 4) γ_{Ginf} 4) γ_Q γ_A	(1.00) (1.00) (1.30)	(1.00) (1.00) (1.00) (1.00)

P: Persistent situation T: Transient situation A: Accident situation

1) The design should be varied for each case A, B and C separately as relevant.

2) In this verification the characteristics value of the unfavorable part of the permanent action is multiply by the factor [1.1] and the favorable part by the factor [0.9]. More refined rules are given in ENV 1993 and ENV 1994.

3) In this verification the characteristics value of all permanent actions from one sources are multiplied by [1.35] if the total resulting action effect is unfavorable and by [1.0] if the total resulting actions is favorable.

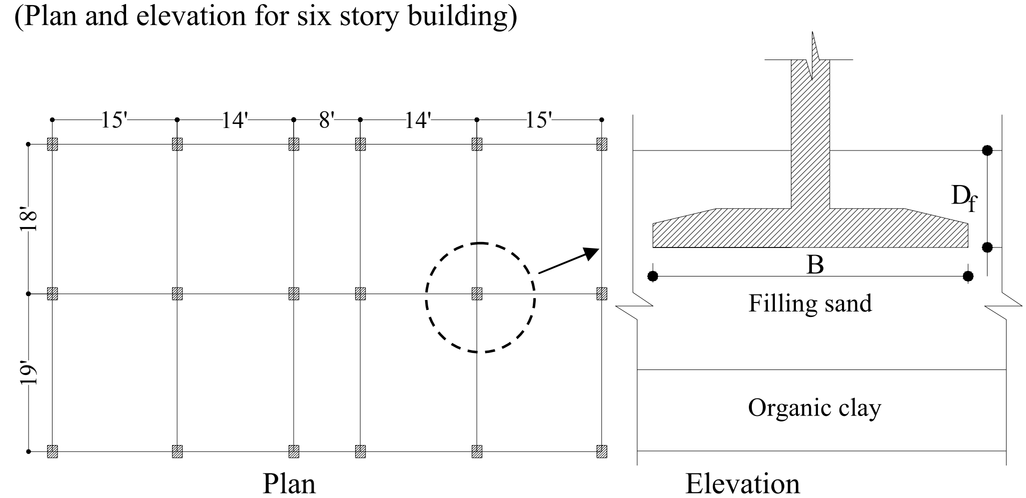
4) In cases when the limit state is very sensitive to variations of permanent actions, the upper and lower characteristics values of these actions should be taken.

5) Foe cases B and C the design ground properties may be different.

6) Instead of using γ_G (1.35) and γ_Q (1.50) for lateral earth pressure actions the design properties may be introduced in accordance with ENV 1997 and a model factor γ_{sd} is applied.

APPENDIX-B
SETTLEMENT CALCULATION FOR DIFFERENT FOUNDATION SYSTEMS

(a) Settlement calculation for spread footing
(Plan and elevation for six story building)



Soil data (taken from S-4, BH-15):

Thickness of filling soil = 3.5 m

$\gamma_{filling}$ = 19 kN/m^3

Thickness of organic clay = 5 m

$\gamma_{organic}$ = 19 kN/m^3

Water table = 3.5 m

C_c (organic clay) = 0.94

e_o = 3.71

Assumed data:

Structural load = 500 kips

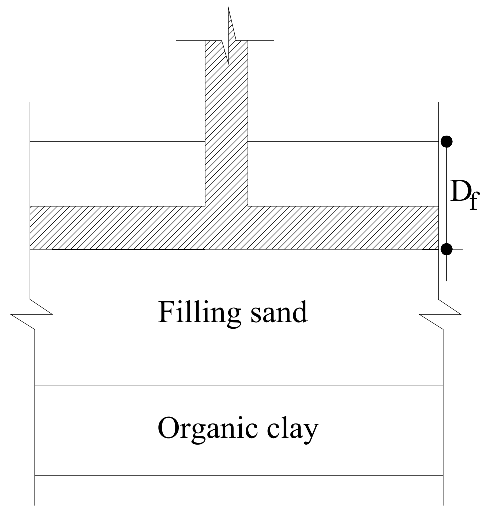
Footing length = 3.084 m

Footing width = 3.084 m

Depth of footing = 1.829 m

Calculated Settlement = 183 mm

(b) Settlement calculation for raft footing
(For six story building)



Elevation of raft

Soil data (taken from S-4, BH-15):

Thickness of filling soil = 3.5 m

$\gamma_{filling}$ = 19 kN/m^3

Thickness of organic clay = 4.5 m

$\gamma_{organic}$ = 19 kN/m^3

Water table = 3.5 m

C_c (organic clay) = 0.94

e_o = 3.71

Assumed data:

Structural load = 5000 kips

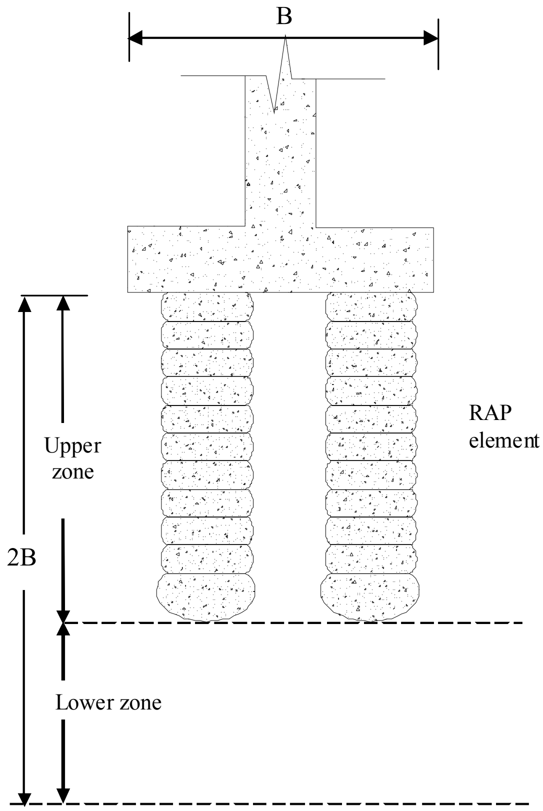
Footing length = 20.12 m

Footing width = 11.28 m

Depth of footing = 3.1 m

Calculated Settlement = 8 mm

(c) Settlement calculation for Rammed Aggregate Pier footing



$$s = \frac{\dfrac{qR_s}{(R_a R_s + 1 - R_a)}}{k_{gp}}$$

Assumed Data:

Structural load = 500 kips

Footing size = 10 (ft) × 10 (ft)

No of geopier = 12

Footing bearing pressure, q = 5 ksf

Stiffness ratio of geopier element to surrounding soil, $R_s = 8$

Ratio of geopier area to footing area, $R_a = 0.377$

Geopier stiffness modulus, $k_{gp} = 108$ k/ft^3

Calculated Settlement:

Settlement of the upper zone = 1.22 in = 30.99 mm