LIQUEFACTION POTENTIAL OF SELECTED RECLAIMED AREAS OF DHAKA CITY BASED ON CONE PENETRATION TEST

BY

RIPON HORE

DEPARTMENT OF CIVIL ENGINEERING

BANGLADESH UNIVERSITY OF ENGINEERING

AND TECHNOLOGY, DHAKA

OCTOBER, 2013

LIQUEFACTIONPOTENTIALOFSELECTEDRECLAIMEDAREASO F DHAKACITYBASEDON CONEPENETRATIONTEST

By

Ripon Hore

Student No: 040804209

In partial fulfillment of the requirements

For the degree of

Masters of Science in Civil & Geotechnical Engineering

Department of Civil Engineering

BangladeshUniversity of Engineering and Technology

Dhaka 1000, Bangladesh

October, 2013

The thesis titled "**LIQUEFACTION POTENTIAL OF SELECTED RECLAIMED AREAS OF DHAKA CITY BASED ON CONE PENETRATION TEST"** submitted by Ripon Hore, Student No. 040804209F, and Session: April, 2008 has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master of Science in Civil & Geotechnical Engineering on 27th October, 2012.

BOARD OF EXAMINERS

Dr. Mehedi Ahmed Ansary, Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.	Chairman (Supervisor)
Dr. A.M.M.Taufiqul Anwar, Professor and Head, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.	Member (Ex-officio)
Dr. Abu Siddique, Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka-1000, Bangladesh.	Member
Dr. Sultan Mohammad Farooq, Associate Professor, Department of Civil Engineering, Chittagong University of Engineering and Technology, Chittagong, Bangladesh.	Member (External)

Certificate

This is to certify that the work presented in this thesis paper is the outcome and investigation carried out by author under the supervision of Dr. Mehedi Ahmed Ansary, Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka. It has been also declared that this thesis embodied own research work and composed by the author. Where appropriate, acknowledgement to the work of other has been made.

Ripon Hore

ACKNOWLEDGEMENT

All praises to God, the merciful and the kind.

The author wishes to express his deepest gratitude to Dr. Mehedi Ahmed Ansary, Professor, Department of Civil Engineering, BUET for his continuous guidance, invaluable suggestions and affectionate encouragement at every stage of this study. The author also wishes to express his deepest gratitude to Dr. A.M.M.Taufiqul Anwar, Professor and Head, Department of Civil Engineering, BUET, for their invaluable suggestions and encouragement of this research. The author wishes to express sincere appreciation to Dr. Abu Siddique, Professor. Department of Civil Engineering, BUET for his valuable guidance and suggestions during the present research. The author express extreme gratitude to Dr. Sultan Mohammad Farooq, Associate Professor, Department of Civil Engineering, CUET for his cooperation & valuable suggestion in this thesis work.

The author wishes to express his gratitude and appreciation to his parents and sister for their inspiration and encouragement.

ABSTRACT

The purpose of this research is to estimate the earthquake induced liquefaction potential of selected reclaimed areas of Dhaka city based on both Cone Penetration Test (CPT) and Standard Penetration Test (SPT). The Cone Penetration Test (CPT) is an in situ testing method used to determine the Geotechnical Engineering properties of soils delineating soil stratigraphy. CPT is a method of assessing subsurface stratigraphy associated with soft materials, discontinuous lenses, organic materials (peat), potentially liquefiable materials (silt, sands and granule gravel) and landslides. Different parameters like cone tip resistance, local friction, friction ratio and pore water pressure have been found directly from CPT. CPT data have been collected from ten selected reclaimed areas of greater Dhaka city. The filling depth of the reclaimed areas varies from 1.5 to 13.5 m from the existing ground level (EGL). For the liquefaction analyses, the values of peak ground acceleration, a_{max} and the magnitude, M have been taken as 0.15g and 7.5, respectively. The range of SPT-N, Cone tip resistance (MPa), local friction (kPa) and friction ratio varies between 1~42, 0.17~18.58, 0~273.2 and 0~9.34 respectively of the ten selected sites. Liquefaction potential has been estimated based on both CPT and SPT data. It has been observed that in reclaimed areas of Dhaka city especially for the locations reclaimed by dredged soil up to a filling depth of 1.5 to 4.5m there is high probability of liquefaction occurrence. In most of the cases, liquefaction zone for CPT have been observed in two different depth zones. Liquefaction zone for CPT of Bramangaon, Ashian City, Badda, Banasree, Gabtoli, Kawran Bazar, Purbachal, United City, Uttara and Kamrangir Char are 1.5 ~10 m and 18~22.5m, 1.8~3.9 m and 12.9~14.4 m, 4.5 ~5.5 m and 8~9.5m, 0.7 ~4.8 m and 14.4~15.3 m, 0.6 ~5.4 m, 11.4 ~12.3 m, 1.5 ~5.0 m and 15~15.6 m, 3.0~4.5 m. and 10.5~12.0 m, 2.7 ~4.8 m and 8.7~12.3 m, and 1.5 ~6 m and 7.5~12 m respectively. The range of obtained values of Overconsolidation ratio (OCR), Preconsolidation stress (kPa)**,** Coefficient of lateral earth pressure (K_0) and Angle of internal friction (ϕ). varies between 0.08~61.33, 2.69~463, 0.107~5.45 and 21~45 respectively for the ten selected sites. Different correlations between CPT, SPT and others parameters have also been developed.

Table of Contents

CHAPTER THREE: EQUIPMENT USES & DATA COLLECTION

LIST OF FIGURES

LIST OF TABLES

.

CHAPTER ONE
INTRODUCTION

1.1 General

Liquefaction problem became important when it started to affect human and social activities by disturbing the function of facilities and also after rapid urbanization by expanding the cities in reclaimed areas. Ground failures generated by liquefaction had been a major cause of damage during past earthquakes e.g., 1964 niigata, Japan and 1964 Alaska,USA, 1971 San Fernando, 1989 Loma prieta,1995 Kobe, Japan and 2004 Chuetsu, Japan earthquakes. Liquefaction affects buildings, bridges, buried pipelines and lifeline facilities etc in many ways.

The historical seismicity data and recent seismic activities in Bangladesh and adjoining areas indicate that Bangladesh is at high seismic risk. As Bangladesh is the world's most densely populated area, any future earthquake shall affect more people per unit area than other seismically active regions of the world. Bangladesh including capital city Dhaka is largely an alluvial plain consisting of fine sand and silt deposits with shallow ground water table in most places. Although the older alluvium is less susceptible to liquefaction, the deposits along the river flood plains may liquefy during a severe earthquake. Human made soil deposits also deserve attention. Loose fills, such as those placed without compaction, are very likely to be susceptible to liquefaction.

Over the past 30~40 years Dhaka city has experienced a rapid growth of urban population and it will continue in the future due to several unavoidable reasons. This high population increase demands rapid expansion of the city. Unfortunately, most parts of Dhaka city has already been occupied. As a result, new areas have been reclaimed by both government and private agencies in and around Dhaka city. In many cases, the practice for developing such new areas is just to fill lowlands of the depth 3~12m with dredged material consisting of silty sand. This causes liquefaction susceptibility for such areas.

After recognizing the liquefaction phenomenon during the 1964 great Nigata and 1964 Alaska earthquakes, many researchers have presented the liquefaction determination procedures like Japanese code of bridge design (1990) including Chinese criterion, Seed-Idriss simplified procedure, which have been updated over the years (e.g., seed et al.,1983). A few researches have conducted liquefaction possibilities at local levels in Bangladesh. Rashid (2000) developed seismic microzonation map of Dhaka city based on site amplification and liquefaction. Rahman (2004) updated the seismic microzonation maps for liquefaction as well as site amplification due to earthquake. Saha (2005) developed liquefaction potential map for Rangpur town. Islam (2005) estimated the seismic losses especially due to liquefaction for Sylhet city. Islam and Ahmed (2005) conducted preliminary evaluation of liquefaction potential of some selected reclaimed area of Dhaka city. Tanvir (2009) estimated earthquake induced liquefaction potential of selected areas of Dhaka city based on shear wave velocity. It was that some parts of the reclaimed areas are susceptible to liquefaction. But those studies were mostly based on SPT N value. The reliability of such SPT data in Bangladesh is questionable. Liquefaction potential estimated using different methods which have been based on SPT data are also different. It has been felt necessary to develop a suitable analysis method to evaluate liquefaction potential for reclaimed areas of Dhaka soil based on Cone Penetration Test (CPT) results.

1.2 Geology of Dhaka city

Dhaka city which is a metropolis as well as the capital city of Bangladesh lies between latitude 23°40’ N to 23°54’ N and longitude from 90°20’ E to 90°30’ E and covers an area of about 470 km^2 having the altitude of 6.5 to 9 m above mean sea level. Geologically, it is an integral part in the southern tip of the Madhupur tract an uplifted block in the Bengal basin, with many depressions of recent origin in it. It is bounded by the Tongi khal (Small River) in the North, the Bariganga river in the south and southeast, the Balu river in the East and Turag river in the West.

The subsurface geology of Dhaka city shows that upper formation is Madhupur clay layer and termed as aquitard and it is 6 to 12 m thick in most parts of the city. The Madhupur clay mainly consists of Kaolinite (27~53%) and Illite (14~33%) with very small amount of Illite smectite (2~13%) down to 5m depth (Zahid et al., 2004). However, below the clay layer, medium to coarse grained formation exist.

Kamal and Midorikawa (2004) delineated the geomorphology of Dhaka city area, differentiating the ground of the city into seventeen geomorphic units using aerial photographs. These geomorphic units represent the soil conditions.surface geology of Dhaka with minor anthropogenic modifications. It has been observed that the city has been expanding rapidly even in the low-lying geomorphic units by fill practices for urban growth since 1960. They also classified the fill-sites into four classes based on the thickness of fills. In order to collect the fill-thickness, the boreholes and old topographic map prepared in 1961 are used. Later on, the classified fills have been integrated with the pre-urban geomorphic-soil units. Figure 1.1 shows the geological map of Bangladesh.

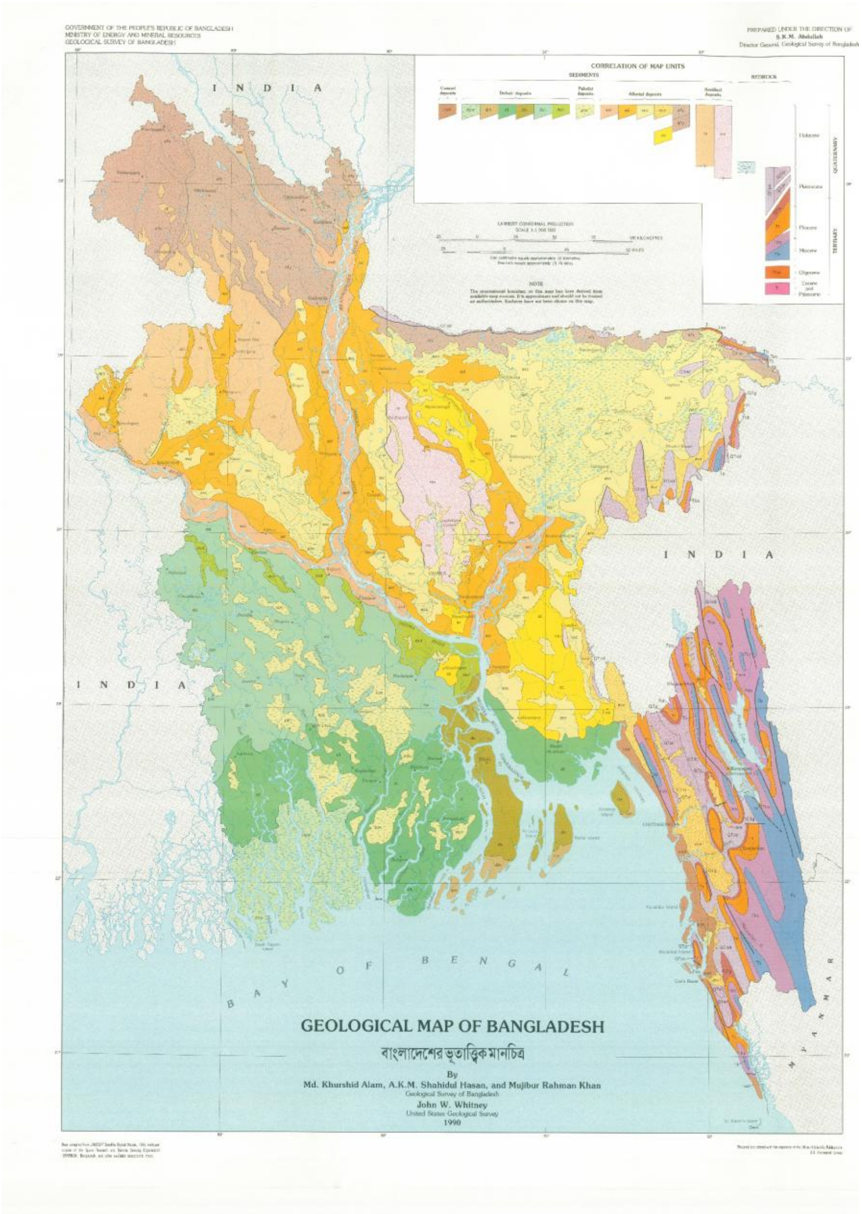


Fig 1.1: Geological map of Bangladesh (Geological survey of Bangladesh)

Alluvial Silt and Clay: Medium to dark grey Silt to Clay; Colour is darker as amount of organic anmaterial increases. Map unit is a combination of alluvial and paludal deposits; includes flood–basin Silt, backs wamp silty clay, and organic rich Clay in sag ponds and large depressions. Some depressions contain peat. Large areas underlain by this unit are dry only a few months of the years the deeper part of depressions and bils contains water throughout the year.

Alluvial Silt: Light to medium grey, Fine sandy to clayey silt. Commonly poorly stratified; average grain size decreases away from main channels. Chiefly deposited in flood basins and interstream areas. Units includes small backswamp deposits and varying episodic or unusually large floods. Illite is the most abundant clay mineral. Most areas have been flooded annually. Included in this unit are thin veneers of sand spread by episodic large floods over flood plain silts. Historic potery, artifact, and charcoal found in upper 4 m.

Madhupur Clay residuum: light yellowish grey, orange, light to brick red and grayish white, amicaceous silty clay to sandy clay; plastic and abundentey mattled in upper 8 m, contains small clusters of organic matter. Sand fraction dominantly quartz; minor feldspar and mica; sand content increases with depth. Dominant clay minerals are kaolinite and Illite. Iron manganese oxide modules rare.

1.3 Objective

The goal of the proposed research is to estimate the earthquake induced liquefaction potential of selected reclaimed areas of Dhaka city based on cone penetration test (CPT). The following are the specific objectives of the research:

1) To estimate liquefaction potential of some selected reclaimed areas of Dhaka City by Cone penetration test results. (Robertson and Wride,1998);

2) To compare the liquefaction potential of these areas estimated by cone penetration test result with SPT-N value.(Seed et al;1983);

3) To develop correlations between CPT, SPT and other parameters.

1.4 **Outline of the Thesis**

The thesis has been comprised of six chapters.

Chapter one provides a brief introduction to the subject and states the major objectives of the research.

Chapter two has been devoted to the review of past researches in Bangladesh and other parts of the world. Details of liquefaction has been discussed in this Chapter. Damages due to liquefaction and countermeasures against liquefaction have also been discussed. Methods for liquefaction potential analysis have been described in this Chapter. Corelation between CPT, SPT and other parameters have been also described in this chapter.

Chapter three describes the CPT and SPT test procedures and results. The variation of different parameters like friction, cone resistance, friction ratio and SPT value (N value) with depth have been described here.

Estimation of Liquefaction potential has been discussed in chapter four. Liquefaction potential have been calculated based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data.

In the chapter five different soil parameters estimated from CPT have been presented and different correlations have been developed.

Chapter six presents the conclusion and recommendation of this research.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

The objective of this chapter is to analyze the past researches related to liquefaction in home and abroad. In addition to that detail theoretical aspects of liquefaction including its estimation procedures and possible mitigation methods have also been discussed.

2.2 Seismicity in Bangladesh and problem hazards

Significant damaging historical earthquakes have occurred in and around Bangladesh and damaging moderate magnitude earthquake occurred every few years. The country's position adjacent to the very active Himalayan front and ongoing deformation in nearby parts of south-east Asia expose it to strong shaking from a variety of earthquake sources that can produce tremors of magnitude 8 or greater. The potential for magnitude 8 or greater earthquake on the nearby Himalayan front is very high, and the effects of strong shaking from such an earthquake directly affect much of the country. In addition, historical seismicity within Bangladesh indicates that potential for damaging moderate to strong earthquake exist throughout most of the country.

Large earthquakes occur less frequently than serious floods, but they can affect much larger areas and can have long lasting economic, social and political effects. Bangladesh covers one of the largest deltas and one of the thickest sedimentary basins in the world. According to the report on time predictable fault modeling CDMP (2009), earthquake and tsunami preparedness component of CDMP have identified five tectonic fault zones which may produce damaging earthquakes in Bangladesh. These are :

a) Madhupur fault zone
b) Dauki fault zone.
c) Plate boundary fault zone-1
d) Plate boundary fault zone-2
e) Plate boundary fault zone-3

Considering fault length, fault characteristics, earthquake records etc, the maximum magnitude of earthquakes that can be produced in different tectonic blocks have been given in Table 2.1.

In the generalized tectonic map of Bangladesh as shown in Figure 2.1 the distribution of epicenters has been found to be linear along the Dauki fault system and random in other regions of Bangladesh. The investigation of the map demonstrates that the epicentres are lying in the weak zones comprising surface or subsurface faults. Most of the events are of moderate rank (magnitude 4~6) and lie at a shallow depth, which suggests that the recent movements occurred in the sediments overlying the basement rocks. In the northeastern region (surma basin), major events have been controlled by the Dauki fault system. The events located in and around the Madhupur tract also indicate shallow displacement in the faults separating the block from the alluvium. Figure 2.2 shows the major fault lines which affect seismicity in Bangladesh.

Information of earthquake in and around Bangladesh is available for the last 250 years . Among these, during the last 150 years, seven major earthquakes have affected Bangladesh. The surface wave magnitude, maximum intensity according to European Macroseismic scale (EMS) and epicentral distance from Dhaka has been presented in Table 2.3. Characteristics of some recent earthquakes have also been shown in Table 2.2.

Table 2.1 Maximum estimated earthquake magnitude in different tectonic faults (CDMP, 2009)

Fault zone	Earthquake events	Estimated magnitude, m_w
Madhupur fault zone	AD 1885	7.5
Dauki fault zone	AD 1897. AD 1500 to 1630 (AD 1548)	8.0
Plate Boundary-1	AD 1762, AD 680 to 980, BC 150 to AD 60, BC 395 to 740	8.5
Plate Boundary-2	Before 16th century	8.0
Plate Boundary-3	Before 16th century	8.3

Table 2.2 Recent earthquakes in Bangladesh

Date	Place of earthquake	magnitude	Destructions
13 november,1997	Chittagong	6.0	It caused minor damage around Chittagong town.
12 july,1999	Maheshkhali Island	5.2	Severely felt around maheshali island and the adjoining sea.
7 july,2003	Kolabunia union of barkal upazila, rangamati district	5.1	Houses cracks and landslides.

Table 2.3 List of major earthquake affecting Bangladesh during last 150 years (Ms>7) (Sabri, 2002)

Date	Name of earthquake	Surface wave magnitude (m_s)	Maximum intensity (EMS)	Epicentral distance from Dhaka (km)	Basis
10 january, 1869	Cachar earthquake	7.5	IX	250	Back calculation from intensity
14 july,1885	Bengal earthquake	7.0	VII to IX	170	Directly from seismograph
12 june, 1897	Great Indian earthquake	8.7	X	230	
8 july,1918	Srimongal earthquake	7.0	VII to IX	150	
2 july,1930	Dhubri earthquake	7.1	IX	250	
15 january,1934	Bihar-nepal earthquake	8.3	X	510	
15 August,1950	Assum earthquake	8.5	X	780	

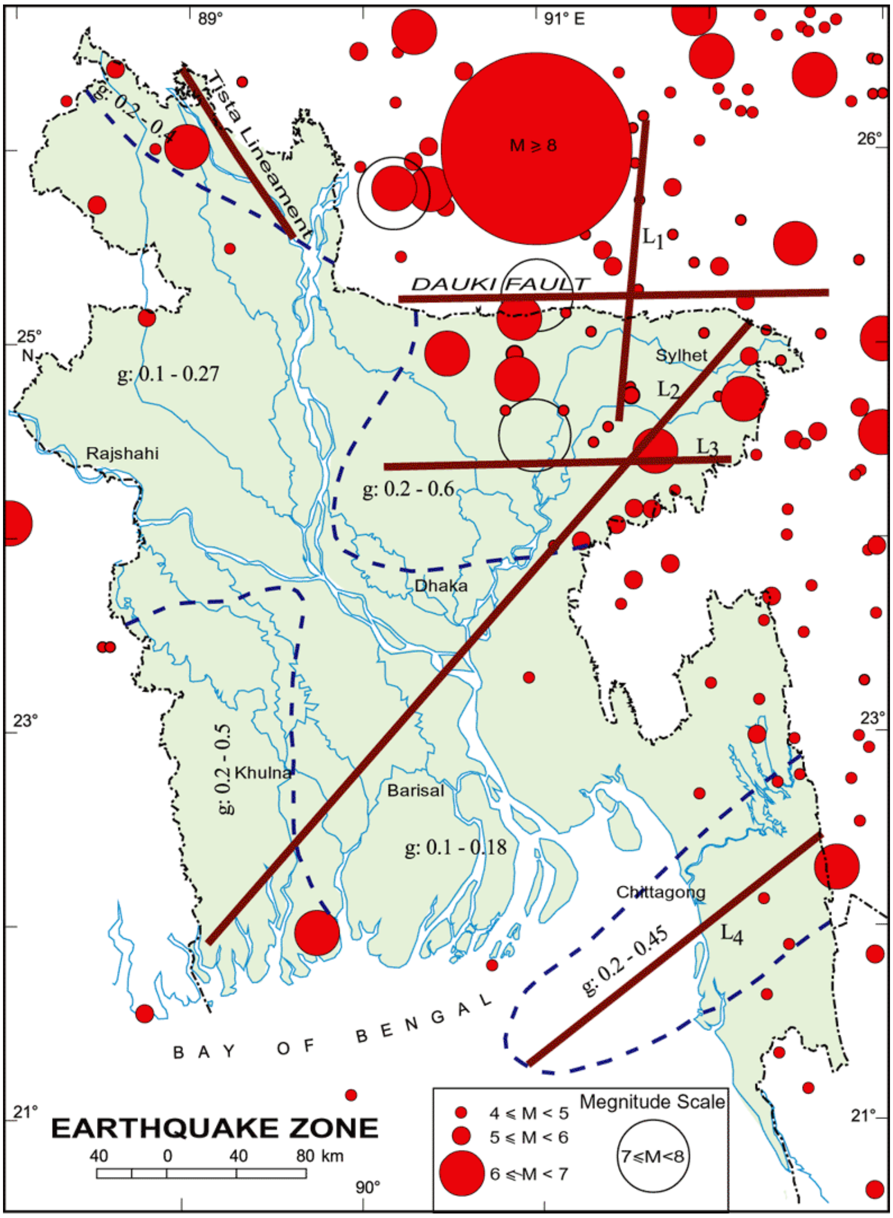


Fig 2.1: Seismo-tectonic lineaments capable of producing damaging earthquakes

(Source: www.banglapedia.com)

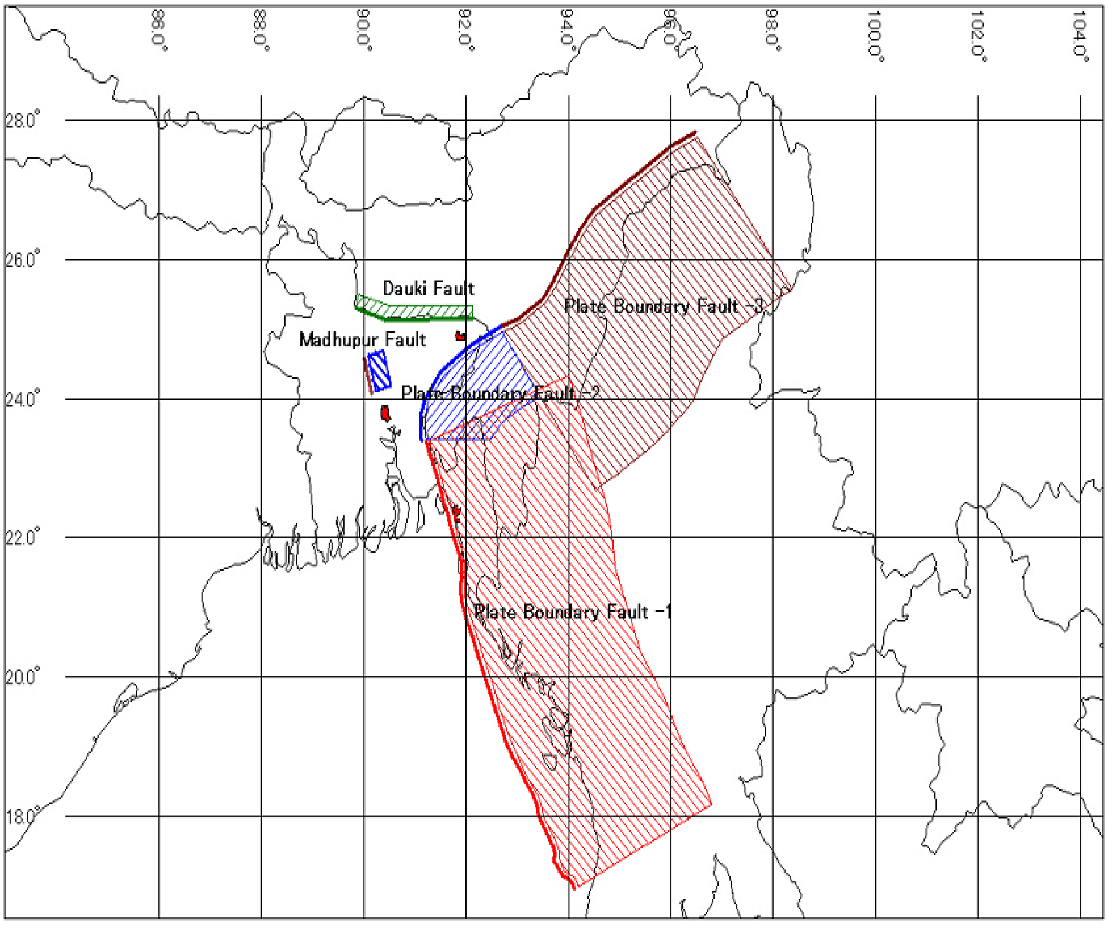


Fig 2.2: The major fault lines which affect seismicity in Bangladesh (CDMP,2009)

Based on the above discussions , the probable hazard scenari for an earthquake to a scale of M_w = 6.5 or above in Dhaka city could cause:

a) Panic among the city dwellers and no knowledge of what is to be done during and immediately after earthquake occurrence.

b) Possible sinking of many of the buildings on filled earth with shallow foundations due to the liquefaction effect.

c) If the earthquake occurs during monsoon time possible damage of the Dhaka flood protection embankment due to liquefaction effect causing sudden submergence of a large area.

d) Large scale damage and some collapse of poorly constructed and /or old building.

e) Possible outbreak of fire in most of the building from the gas lines.

f) Possible damage of power installation and power cut off for identifing period.

g) Water supply failure as almost all the deep tube wells are run by power, and possible water line damage.

h) Damage of roads and blockage of traffic due to falling of debris from collapsed buildings and other installations on or near roads.

i) Some of the hospital buildings may collapse killing a large number of inmates and stopping medical for the disaster victims.

j) Some of the school building may collapse killing and injuring a large number of students.

k) An after shock may cause further collapse of many of the already damaged buildings.

l) A few rescue equipment , whatever is available , can not be operated due to the lack of guidance , availability of operators, some can not find access to resque sports due to road blockage,etc.

m) Limited access from outside as most or the highways/bridges, airport may not be functional.

Although during the last decade much advancement has been achieved in the earthquake engineering, modern science has yet to invent any technology that can predict earthquake. But some earthquake induced damages like liquefaction susceptibility can be evaluated before hand. Therefore, liquefaction susceptibility of Dhaka soil demands extensive research in order to mitigate and reduce earthquake induced hazards to the most populated city in the world.

2.2.1 Seismic zoning map of Bangladesh

The seismicity zones and the zone coefficients may be determined from the earthquake magnitude for various return periods and the acceleration attention relationship. It has been required that for design or ordinary structures, seismic ground motion having 10% probability of being exceeded in design life of a structure (50 years) considered critical. An earthquake having 200 years return period originating in sub-Dauki zone have epicentral acceleration of more than 1.0g but at 50 kilometer the acceleration shall be reduced to as low as 0.3g.

Ali (1998) presented the earthquake base and seismic zoning map of Bangladesh. Tectonic frame work of Bangladesh adjoining areas indicate that Bangladesh has been situated adjacent to the plate margins of India and Eurasia where devastating earthquake have occurred in the past. Non-availability of earthquake, geology and tectonic data posed great problem in earthquake hazard mapping of Bangladesh in the past. The first seismic map which has been prepared in 1979 was developed considering only the epicentral location of past earthquake and isoseismic map of very few of them. During preparation of National Building Code of Bangladesh in 1993, substantial effort was given in revising the existing seismic zoning map using geophysical and tectonic data, earthquake data, ground motion attenuation data and strong motion data available from within as well as outside of the country. Geophysical and tectonic data were available from Geological survey of Bangladesh. Earthquake data have been collected from NOAA data files and geodetic survey, US.Dept. of commerce.

Seismic zoning map for Bangladesh has been presented in Bangladesh National Building code (BNBC) published in 1993. The pattern of ground surface acceleration contours having 200 year return period from the basis of this seismic zoning map. There are three zones in the map zone 1, zone 2, zone 3. The seismic coefficients of the zones are 0.075g, 0.15g and 0.250g for zone 1, zone 2and zone 3 respectively. Bangladesh National building Code (1993) placed Dhaka city area in seismic zone 2 as shown in Fig 2.3. However, the seismic zones in the code have been not based on the analytical assessment of seismic hazard and mainly based on the location of historical data.

The first seismic zoning map of the subcontinent has been compiled by the Geological Survey of India in 1935. The Bangladesh Meteorological Department adopted a seismic zoning map in 1972. In 1977, the Government of Bangladesh constituted a Committee of Experts to examine the seismic problem and make appropriate recommendations. The Committee proposed a zoning map of Bangladesh in the same year. Figure 2.4 shows the proposed Seismic Zoning Map of Bangladesh.

According to Bangladesh National Building Code (BNBC, 1993), Bangladesh has been divided into 3 earthquake zones (Fig 2.3).

Zone-3 comprising the northern and eastern regions of Bangladesh with the presence of the Dauki Fault system of eastern Sylhet and the deep seated Sylhet Fault, and proximity to the highly disturbed southeastern Assam region with the Jaflong thrust, Naga thrust and Disang thrust, is a zone of high seismic risk with a basic seismic zoning co-efficient of 0.25. Northern Bangladesh comprising greater Rangpur and Dinajpur districts is also a region of high seismicity because of the presence of the Jamuna Fault and the proximity to the active east-west running fault and the Main Boundary Fault to the north in India. The Chittagong-Tripura Folded Belt experiences frequent earthquakes, as just to its east is the Burmese Arc where a large number of shallow depth earthquakes originate.

Zone-2 comprising the central part of Bangladesh represents the regions of recent uplifted Pleistocene blocks of the Barind and Madhupur Tracts, and the western extension of the folded belt. The zone extends to the south covering Chittagong and Cox's Bazar. Seismic zoning coefficient for Zone II is 0.15.

Zone-1 comprising the southwestern part of Bangladesh is seismically quiet, with an estimated basic seismic zoning co-efficient of 0.075.

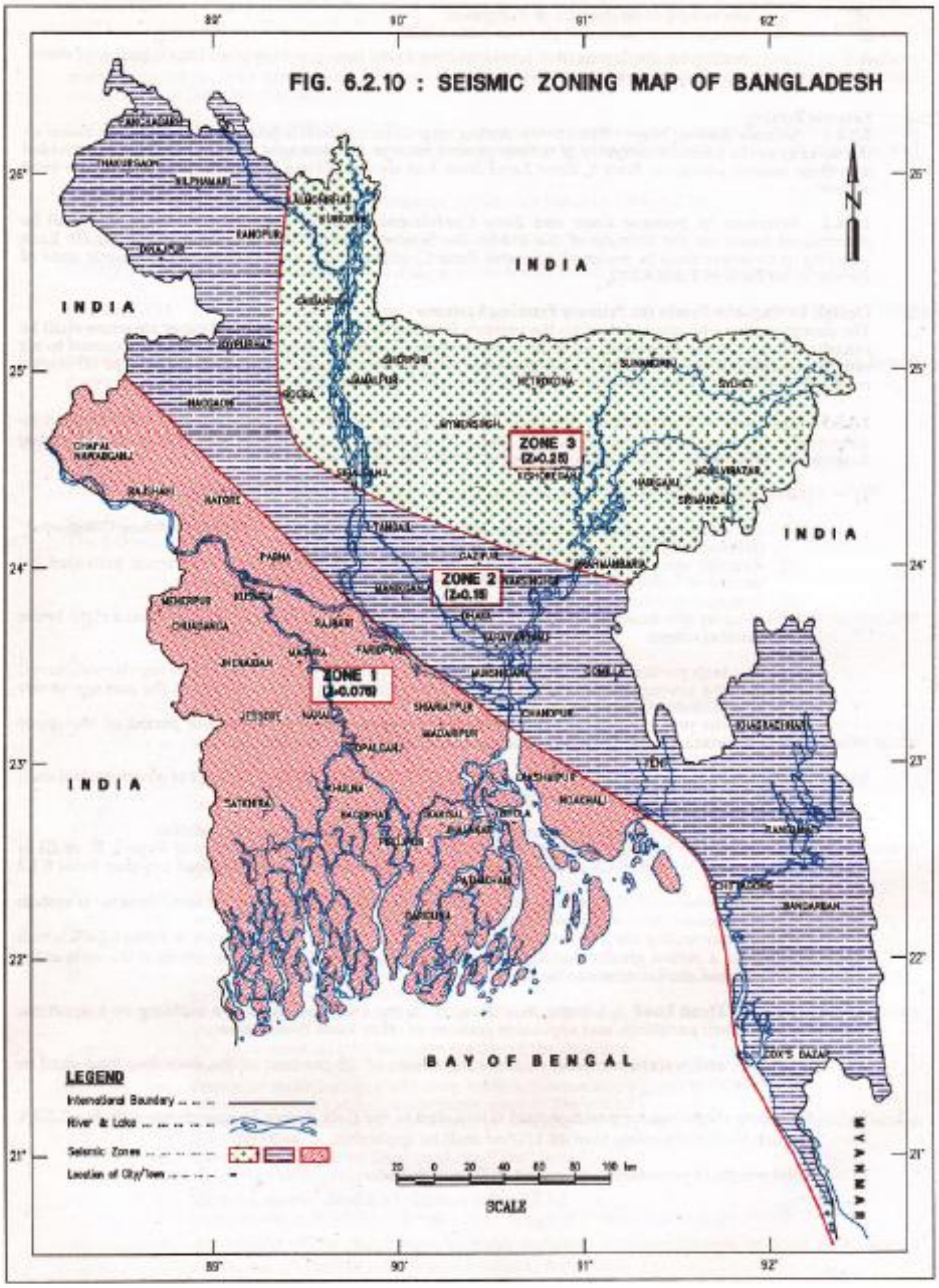


Fig 2.3: Seismic Zoning Map of Bangladesh (After BNBC, 1993)

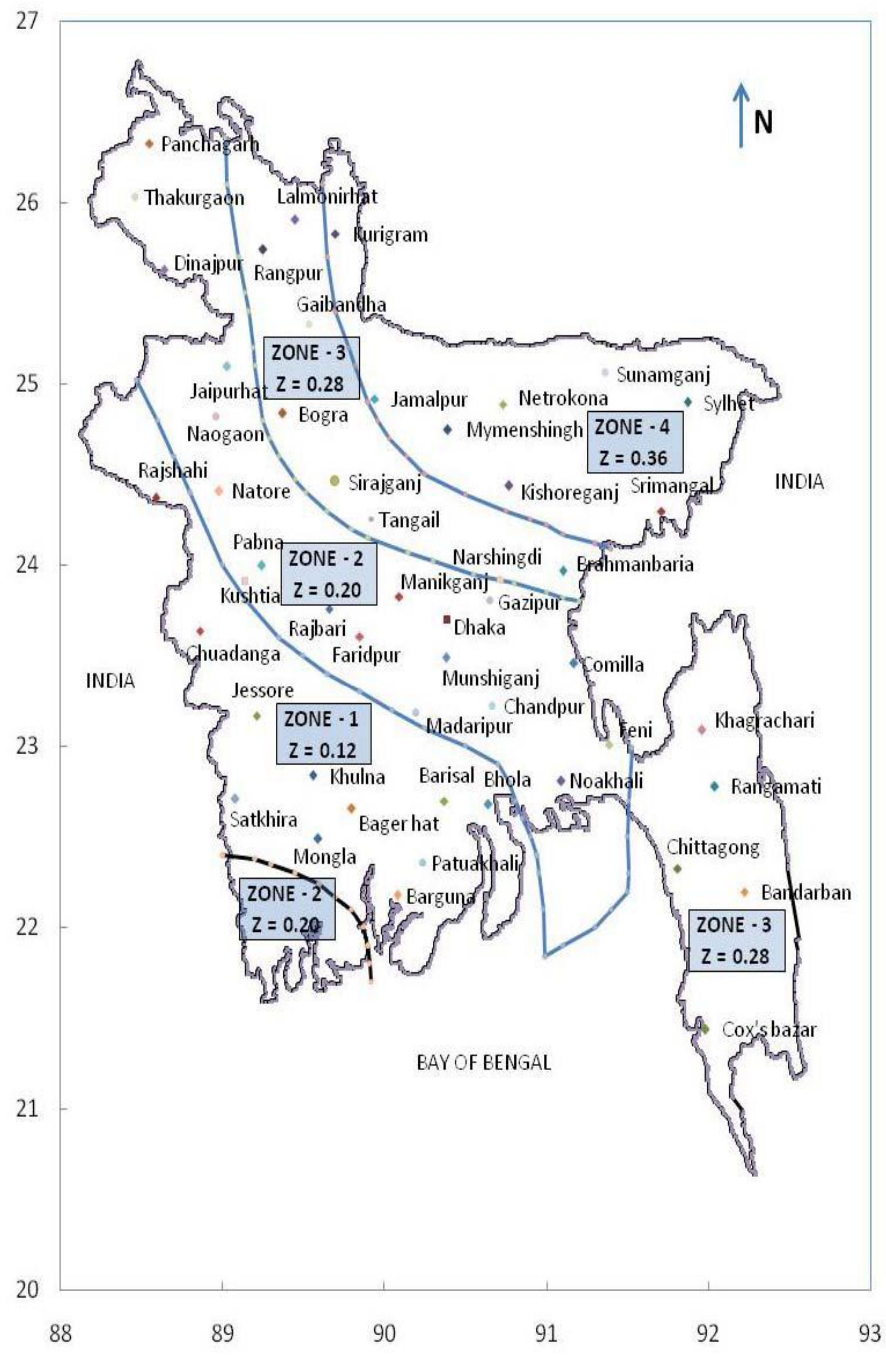


Fig 2.4 : Proposed Seismic Zoning Map of Bangladesh (Proposed BNBC)

2.2.2 Major Source of Earthquake in Bangladesh

Bangladesh is one of the most earthquake prone countries in the world. Specialists are expecting a severe earthquake in this area in near future, which will cause a serious human casualty, damages of infrastructure and other losses.

Although Bangladesh is extremely vulnerable to seismic activity, the nature and the level of this activity is yet to be defined. In Bangladesh complete earthquake monitoring facilities are not available. The Meteorological Department of Bangladesh established a seismic observatory at Chittagong in 1954. This remains the only observatory in the country.

Since the whole Indian subcontinent is situated on the junction of Indo- Australian plate and Eurasian plate, the tectonic evaluation of Bangladesh can be explained as a result of collision of the north moving Indo- Australian plate with the Eurasian plate. Figure 2.5 shows the Tectonic plates.

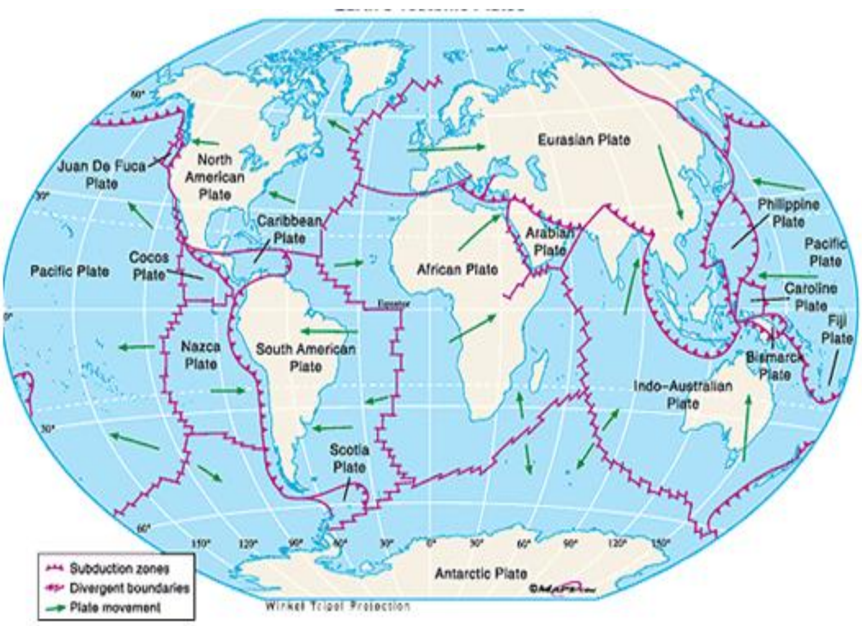


Fig 2.5: Tectonic plates

2.3 Liquefaction and its significant

If saturated sand has been subjected to ground vibrations, it tends to compact and decrease in volume; if drainage is unable to occur/prevented, the tendency to decrease in volume results in an increase in pore water pressure, and if the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress become zero, the sand loses its strength completely, and it develops a liquefied state.

The liquefaction problem has been attracting engineering concern for about past 35 years. It was not considered important before, although large earthquakes had caused liquefaction in loose sand deposits. This seems so because cities in old times were not too large and were confined within areas of state deposits, reclaimed land was rare, and attention was paid mostly to such seismic effects as collapse and burning of buildings. The liquefaction problem became important for the first time when it started to affect human and social activities by disturbing the function of facilities. The loss of function can be follows:

a) Subsidence of road embankments which leads to cracking in surface pavements and block traffic.

b) Building subsidence and tilting to such an extent that its normal use is not possible.

c) Lateral movement of bridge abutments and piers, as well as , in the most extreme cases, collapse of a bridge.

d) Breakage and separation of buried pipes, which take supply of water and gas out of service.

e) Floating of sewerage treatment tanks and buried pipes, which make normal flow of water impossible.

Liquefaction phenomena can affect buildings, bridges, buried pipelines and other constructed facilities in many different ways. Liquefaction can also influence the nature of ground surface motions. Flow liquefaction can produce massive flow slides and contribute to the sinking or tilting of heavy structures, the floating of light buried structures, and to the failure of retaining structures. Cyclic mobility can cause slumping of slopes, settlement of buildings, lateral spreading and retaining wall failure. Substantial ground oscillation, round surface settlement, sand boils and post-earthquake stability failures can develop at level ground sites. Figure 2.6 shows the some effects of liquefaction during the 1964 Niigata, Japan earthquake.

Soil liquefaction describes the behavior of soils that, when loaded, suddenly go from a solid state to a liquefied state, or having the consistency of a heavy liquid. Liquefaction is more likely to occur in loose to moderate saturated granular soils with poor drainage, such as silty sands or sands and gravels capped or containing seams of impermeable sediments. During loading, usually cyclic undrained loading, e.g. earthquake loading, loose sands tend to decrease in volume, which produces an increase in their pore water pressures and consequently a decrease in shear strength, i.e. reduction in effective stress.

Liquefaction can cause damage to structures in several ways. Buildings whose foundations bear directly on sand which liquefies will experience a sudden loss of support, which will result in drastic and irregular settlement of the building. Liquefaction causes irregular settlements in the area liquefied, which can damage buildings and break underground utility lines where the differential settlements are large. Pipelines and ducts may float up through the liquefied sand. Sand boils can erupt into buildings through utility openings, and may allow water to damage the structure or electrical systems. Soil liquefaction can also cause slope failures. Areas of land reclamation are often prone to liquefaction because many are reclaimed with hydraulic fill, and are often underlain by soft soils which can amplify earthquake shaking. Soil liquefaction was a major factor in the destruction in San Francisco's Marina District during the 1989 Loma Prieta earthquake. Mitigating potential damage from liquefaction is part of the field of geotechnical engineering.

If saturation sand has been subjected to ground vibrations, it tends to compact and decrease in volume; if drainage is unable to occur/prevented, the tendency to decrease in volume results in an increase in pore water pressure, and if the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress become zero, the sand loses its strength completely, and it develops a liquefied state.

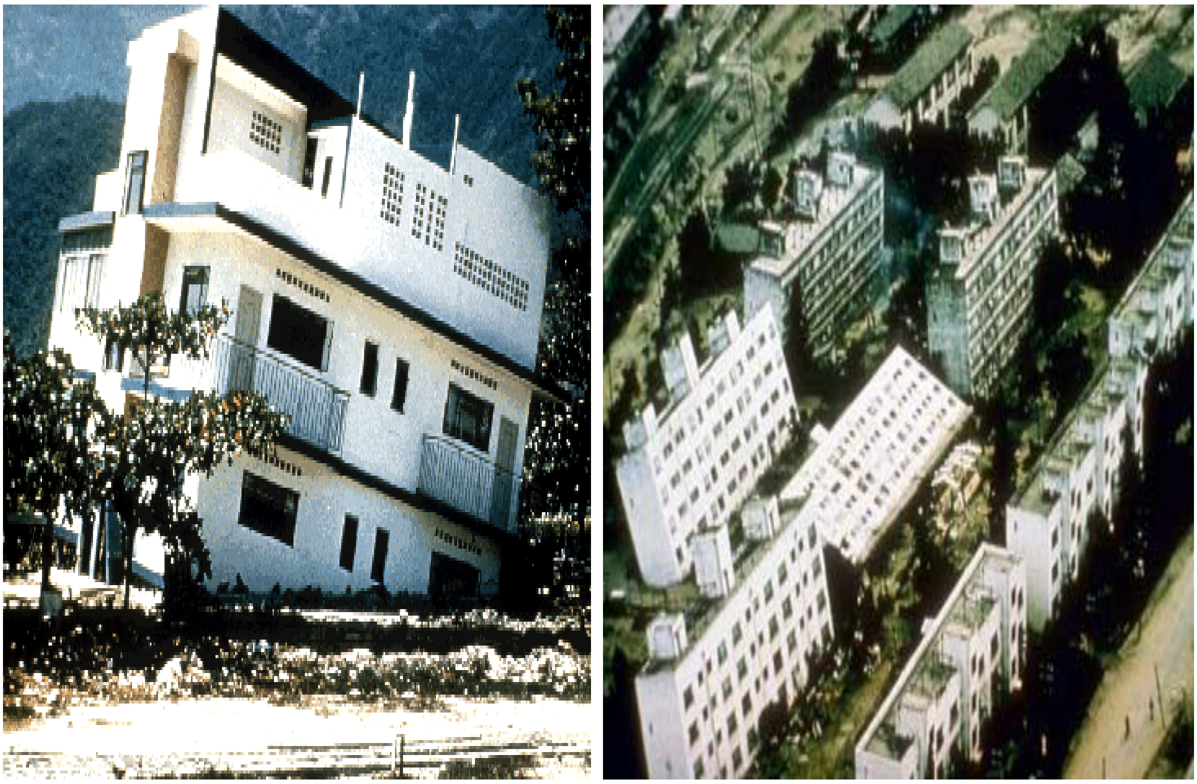


Fig 2.6: Some effects of liquefaction during the 1964 Niigata, Japan earthquake.

2.4 Main Factors that Govern Liquefaction

There are many factors that govern the liquefaction process for in situ soil. Based on results of laboratory tests as well as field observations and studies, the most important factors that govern liquefaction are as follows:

1. Earthquake Intensity and Duration

In order to have earthquake induced liquefaction of soil, there must be ground slinking. The character of the ground motion, such as acceleration and duration of shaking, determines the shear strains that cause the contraction of the soil particles and the development of excess pore water pressures leading to liquefaction. The most common cause of liquefaction is due to the seismic energy released during an earthquake. The potential for liquefaction increases as the earthquake intensity and duration of shaking increase. Those earthquakes that have the highest magnitude will produce both the largest ground acceleration and the longest duration of ground shaking. Although data are sparse, there would appear to be a shaking threshold that has been needed produce liquefaction.

These threshold values are a peak ground acceleration a_{max} of about 0.10_g and local magnitude M_L of about 5 (Ishihara 1985). Thus, a liquefaction analysis would typically not be needed for those sites having a peak ground acceleration a_{max} less than 0.l0g or a local magnitude M_L less than 5. Besides earthquakes, other conditions can cause liquefaction such as subsurface blasting, pile driving and vibrations from train traffic.

2. Groundwater Table

The condition most conducive to liquefaction is a near-surface groundwater table. Unsaturated soil located above the groundwater table will not liquefy. If it can be Unsaturated that the soils are currently above the groundwater table and are highly Unlikely to become saturated for given foreseeable changes in the hydrologic regime, then such soils generally do not need to be evaluated for liquefaction potential. At sites, where the ground water table significantly fluctuates, the liquefaction potential Will also fluctuate. Generally, the historic high groundwater level should be used in the liquefaction analysis unless other information indicates a higher or lower level is appropriate (Division of Mines and Geology, 1997).

Poulos et al. (1985) state that liquefaction can also occur in very large masses of Sands or silts that are dry and loose and loaded so rapidly that the escape of air from the voids is restricted. Such movement of dry and loose sands is often referred to as running soil or running ground Although such soil may flow as liquefied soil does, in this text, such soil deformation will not be termed liquefaction. It is best to *con*sider that liquefaction only occurs for soils that are located below the groundwater table.

3. Soil Type

In terms of the Soil Types most susceptible to liquefaction, Ishihara (1985) states: "The hazard associated with soil liquefaction during earthquakes has been known to be encountered in deposits consisting of fine to medium sand and sands containing low plasticity fines. Occasionally, however, cases are reported where liquefaction apparently occurred in gravelly soils." Thus, the soil types susceptible to liquefaction are non-plastic (cohesionless) soils. An approximate listing of cohesionless soils from least to must resistant to liquefaction is clean sands, non-plastic silty sands, non-plastic silt and gravels.

There could be numerous exceptions to this sequence. For sample Ishihara (1985 and 1993) describes the case of tailings derived from the industry that were essentially composed of ground-up rocks and were clasified as rock flour. Ishihara (1985 and 1993) states that the rock flour in a water saturated slate did not possess significant cohesion and behaved as if it were clean sand. These tailings were shown to exhibit as low a resistance to liquefaction as clean sand.

Seed et al. (1983) stated that based on both laboratories testing and field performance, the great majority of cohesive soils will not liquefy during earthquakes. Using criteria originally staled by Seed and Idriss (1982) and subsequently confirmed by Youd and Oilstrap (1999): in order for a cohesive soil to liquefy, it must meet all the following three criteria:

- The soil must have less than 15 percent of the particles, based on dry weight, that are finer than 0.005 mm (i.e., percent finer at 0.005 mm < 15 percent).
- The soil must have a liquid limit (LL) that is less than 35.
- The water content, w of the soil must be greater than 0.9 of the liquid limit.

If the cohesive soil does not meet all three criteria, hence it is generally considered to be not susceptible to liquefaction. Although the cohesive soil may not liquefy, there could still be a significant undrained shear strength loss due to the seismic shaking.

4. Soil Relative Density, D_r

Based on field studies, cohesionless soils in a loose relative density state are susceptible to liquefaction. Loose non-plastic soils will contract during the seismic shaking which will cause the development of excess pore water pressures. Upon reaching initial liquefaction, there will be a sudden and dramatic increase in shear deplacement for loose sands. For dense sands, the state of initial liquefaction does produce large deformations because of the dilation tendency of the sand upon of the cyclic shear stress. Poulos et al. (1985) state that if the in situ soil can be shown to be dilative, then it need not be evaluated because it will not be susceptible to liquefaction. In essence, dilative soils are not susceptible to liquefaction because undrained shear strength is greater than their drained shear strength.

5. Particle Size Gradation

Uniformly graded non-plastic soils tend to form more unstable particle arrangements and are more susceptible to liquefaction than well-graded soils. Well-graded soils will also have small particles that fill in the void spaces between the large particles. This lends to reduce the potential contraction of the soil, resulting in less excess pore water pressures being generated during the earthquake. Kramer (1996) states that field evidence indicates that most liquefaction failures have involved uniformly graded granular soils.

6. Placement Conditions or Depositional Environment

Hydraulic fills (fill placed under water) tend to be more susceptible to liquefaction because of the loose and segregated soil structure created by the soil particles falling through water. Natural soil deposits formed in lakes, rivers, or the ocean also tend to a loose and segregated soil structure and are more susceptible to liquefaction, Soils that are especially susceptible to liquefaction are formed in lacustrine, alluvial, and marine depositional environments.

7. Drainage. Conditions

If the excess pore water pressure can quickly dissipate, the soil may not liquefy. Thus highly permeable gravel drains or gravel layers can reduce the liquefaction potential of adjacent soil.

8. Confining Pressures

The greater the confining pressure, the less susceptible the soil is to liquefaction. Conditions that can create a higher confining pressure are a deeper groundwater Table, soil that is located at a deeper depth below ground surface, and a surcharge pressure applied at ground surface. Case studies have shown that the possible zone of liquefaction usually extends from the ground surface to a maximum depth of about 50 ft (15 m). Deeper soils generally do not liquefy because of the higher confining pressures. This does not mean that a liquefaction analysis should not be performed for soil that is below depth of 50 ft (15 m). In many cases, it may be appropriate to perform a liquefaction analysis for soil that is deeper than 50 ft (15 m). An example would be sloping ground, such as a sloping berm in front of a waterfront structure or the slopingg shell of an earth dam. In addition, a liquefaction analysis should be performed for any soil deposit that has been loosely dumped in water (i.e., the liquefaction analysis should be performed for the entire thickness of loosely dumped fill in water, even if it exceeds 50 ft in thickness). Likewise, a site where alluvium is being rapidly deposited may also need a liquefaction investigation below a depth of 50 ft (15 m). Considerable experience and judgment are required in the determination of the proper depth to terminate a liquefaction analysis.

9. Particle Shape

The soil particle shape can also influence liquefaction potential. For example, soils having rounded particles tend to densify more easily than angular-shape soil particles. Hence, a soil containing rounded soil particles is more susceptible to liquefaction than a soil containing angular soil particles.

10. Aging and Cementation

Newly deposited soils tend to be more susceptible to liquefaction than older deposits of soil. It has been shown that the longer a soil is subjected to a confining pressure, the greater the liquefaction resistance (Yoshim, M.,1991). The increase in liquefaction resistance with time could be due to the deformation or compression of soil particles into more stable arrangements. With time, there may be the development of bonds due to cementation at particle contacts.

11. Historical Environment

It has also been determined that the historical environment of the soil can affect its liquefaction potential. For example, older soil deposits that have already been seismic shaking have an increased liquefaction resistance compared to specimen of the same soil having an identical density. Liquefaction resistance also increases with an increase in the ratio (OCR) and the coefficient of lateral earth pressure at rest k_o (Ishihara et al., 1978). An example would be the removal of an upper layer of soil due to erosion. Such a soil that has been preloaded will be more resistant to liquefaction than the same soil that has not been preloaded.

12. Building Load

The construction of a heavy building on top of a sand deposit can decrease the liquefaction resistance of the soil. For example, suppose a mat slab at ground surface supports a heavy building. The soil underlying the mat slab will be subjected to shear stresses caused by the building load. These shear stresses induced into the soil by the balding load can make the soil more susceptible to liquefaction. The reason is that a smaller additional shear stress will be required from the earthquake in order to cause liquefaction and hence liquefaction of the soil. For level-ground liquefaction considered in this research, the effect of the building load is ignored. The building loads must be included in all liquefaction- induced settlement and bearing capacity.

2.5 Problems Due to Liquefaction

1. Common Damages

Liquefaction can cause damage to structures in several ways. Buildings whose foundations bear directly on sand which liquefies will experience a sudden loss of support, which will result in drastic and irregular settlement of the building. Liquefaction causes irregular settlements in the area liquefied, which can damage buildings and break underground utility lines where the differential settlements are large. Pipelines and ducts may float up through the liquefied sand. Sand boils can erupt into buildings through utility openings, and may allow water to damage the structure or electrical systems. Soil liquefaction can also cause slope failures. Areas of land reclamation are often prone to liquefaction because many are reclaimed with hydraulic fill, and are often underlain by soft soils which can amplify earthquake shaking.

2. Hazards to Buildings and Bridges

When liquefaction occurs, the strength of the soil decreases and, the ability of a soil deposit to support foundations for buildings and bridges are reduced as seen in the Fig 2.6 of the overturned apartment complex buildings in Niigata in 1964.

3. Hazards to Retaining Walls

Liquefied soil also exerts higher pressure on retaining walls, which can cause them to tilt or slide. This movement can cause settlement of the retained soil and destruction of structures on the ground surface.

4. Hazards to Dam due to Landslide

Increased water pressure can also trigger landslides and cause the collapse of dams. Lower San Fernando dam suffered an underwater slide during the San Fernando earthquake, 1971. Fortunately, the dam barely avoided collapse, thereby preventing a potential disaster of flooding of the heavily populated areas below the dam.

2.6 Past research on liquefaction

After being recognised the liquefaction phenomenon during the 1964 Great Niigata and 1964 Alaska Earthquakes, many researches have been conducted concerning liquefaction in different parts of the world. Some important and relevant researches have been described below.

Seed and Idriss (1971) presented test procedures for measuring soil liquefaction characteristics. They proposed the basic simplified method to evaluate liquefaction potential. In this procedure that was modified and improved by several researchers (e.g., Seed *et al,* 1985; Youd *et al,* 2001; Idriss and Boulanger, 2004 and Cetin et al, 2004) various relationships have been recommended to determine the cyclic resistance ratio (CRR) of clean and silty sand. In order to consider the influence of fines content on CRR, the SPT blow count of silty sands is converted to equivalent clean sand SPT blow count. These relations have been evolved based on the differences between the boundary curves of clean and silty sands when cyclic stress ratio (CSR) is plotted against corrected SPT blow counts.

Seed and Idriss (1982) made an attempt to summarize as simple way as possible, the potential elements of current state art of evaluating liquefaction or cyclic mobility potential of soil deposits on level ground.

Seed et al. (1983) evaluated liquefaction potential using field test data. They used simplified procedure for evaluating the liquefaction potential of sand deposits using data obtained from Standard Penetration Test (SPT). Field data for sites which were known to be liquefied or not liquefied during earthquakes in the United States, Japan, China, Guatemala, Argentina and other courtiers have been presented to establish a criterion for evaluating the liquefaction potential of sands in Magnitude 7.5 earthquakes. The results of this study were then extended to other magnitude earthquakes using a combination of laboratory and field tests data. Available information on the liquefaction resistance of silty sands was also reviewed and a simple procedure for considering the influence of silt content was proposed. A method was presented for using the field tests data to evaluate the possible magnitude of pore water pressure generation in sands and silty sands which remain stable during earthquake shaking. Finally, the applicability of other in situ field tests, such as the static cone penetrometer, shear wave velocity and electrical measurements for evaluating the liquefaction resistance of soils had been examined. Because the empirical approach had been founded on such a large body of field data, it had been believed by the authors to provide the most useful empirical approach available at the present time. However, it had been noted that the standard penetration test cannot be performed conveniently at all depths (say deeper than 30.5 m or through large depths of water) or in all soils (such as those containing a significant proportion of gravel particles). Thus, it was desirable that it was being supplemented by other in situ test methods which can also be correlated with soil liquefaction potential. In many cases, the static cone test, which can be performed more rapidly and more continuously, may provide a good means for evaluating liquefaction potential, especially if it was correlated on a site dependent basis with SPT results. However, this procedure was limited also to sands and silty sands. In dealing with soils containing large particle, or in difficult other in situ characteristics, such as the shear wave velocity or the characteristics of the soil may provide a more suitable means for assessment liquefaction potential. And, in due course, any or all of these in situ test methods have their own detailed correlation with field performance to validate their as meaningful indicators of liquefaction characteristics. It seems likely, that for onshore sites and with deposits of sand up to 30.5 m deep or so, the of liquefaction characteristics with Standard Penetration Test data will the most direct empirical means of evaluating field liquefaction potential for years to come. Other methods, however, have a significant role to play and should be developed to the fullest extent possible to provide information for different and environments.

Seed (1988) described initiation of soil liquefaction under static loading conditions. Liquefaction of loose, saturated sands may be caused by cyclic or static undrained loading. Most previous studies of static liquefaction behavior had emphasized liquefaction susceptibility and the behavior of liquefied soils. An experimental investigation had been undertaken to evaluate the stress conditions required to initiate liquefaction and the influence of various parameters on those stress conditions. The static liquefaction resistance, defined as the shear stress increase under undrained conditions required to initiate liquefaction, had been observed to increase with increasing relative density and confining pressure, and to decrease dramatically with increasing initial shear stress level. At high initial shear stress levels, initiation of liquefaction had been observed to result from increases in shear stress under undrained conditions of only a few percent of the initial shear stress. The distinction between the initiation and the effects of liquefaction was discussed, and an expression for a factor of safety against the initiation of liquefaction had been proposed.

Idriss and Boulanger (1988) have re-examined and revised Semi-empirical procedures for evaluating the liquefaction potential of saturated cohesionless soils during earthquakes. The stress reduction factor (r_d), earthquake magnitude scaling factor for cyclic stress ratios (MSF), overburden correction factor for cyclic stress ratios (K_s), and the overburden normalization factor for penetration resistances (C_N) are discussed and recently modified relations are presented. These modified relations are used in re-evaluations of the SPT and CPT case history databases. In addition, shear wave velocity based procedures are briefly discussed.

Yeamin et al. (2005) have established the correlation of shear wave velocity with liquefaction resistance based on laboratory tests. According to the results of cyclic triaxial tests on Hangzhou sands, a correlation is presented between liquefaction resistance and elastic shear modulus. For its application to different soils, a method proposed by Tokimatsu and Uchida (1990) is utilized the shear modulus with respect to minimum void ratio. A simplified equation has been established to evaluate the liquefaction potential by shear-wave velocity. The critical shear-wave velocity of liquefaction is in linear relation with 1/4 power of depth and the peak horizontal ground surface acceleration during earthquakes. The equation proposed in this paper is compared with previous methods especially the procedure proposed by Andrus & Stokoe (2000). The results show its simplicity and effectiveness when applied to sands, but more validation or modification is needed for its application to sand with higher fines content.

Gratehev et al. (2006) have conducted research to investigate the liquefaction potential of clayey soils under cyclic loading. This research seeks to investigate the liquefaction of clayey soils, a phenomenon that has been the trigger for many natural disasters in the last few decades, including landslides. Research was conducted on artificial clay-sand mixtures and natural clayey soils collected from the sliding surfaces of earthquake-induced landslides. The undrained response of normally consolidated clayey soils to cyclic loading was studied by means of a ring-shear apparatus. For the artificial clay-sand mixtures, it was found that the presence of a small amount of bentonite (≤ 7%) would cause rapid liquefaction, while a further increase in bentonite content (≥ 11%) produced the opposite effect of raising soil resistance to liquefaction by a significant degree. It was demonstrated that the bentonite-sand mixture was considerably more resistant to liquefaction than the kaolin-, and illite-mixtures, given the same clay content. The test results of plastic soils revealed the significant influence of plasticity on the liquefaction resistance of soil. The microfabric of clayey soil was investigated by means of a scanning electron microscope. The analysis showed that the liquefaction potential of soil was related to certain particle arrangements. For example, soil vulnerable to liquefaction had an open microfabric in which clay aggregations generally gathered at the sand particle contact points, forming low-strength “clay bridges” that were destroyed easily during cyclic loading. On the other hand, the microfabric of soil that was resistant to liquefaction appeared to be more compact, with the clay producing a matrix that prevented sand grains from liquefying. In the case of the natural soils, the obtained results indicated that their cyclic behavior was similarly influenced by factors such as clay content, clay mineralogy and plasticity. The relation between the liquefaction potential of natural soil and its microfabric was thus also established. On the basis of the obtained results, the authors posited an explanation on the mechanism of liquefaction for clayey soil.

A few researches have been conducted to estimate liquefaction possibilities at local levels in Bangladesh. These are stated below.

Khan (1988) studied the soils liquefaction possibilities in Bangladesh. From the historical background of earthquake 3 hazards on the Bengal Basin and its surroundings, it is interpreted that the western and south-eastern portions of Bangladesh are not so vulnerable to earthquake effect. So, north-eastern Zone had been identified as earthquake prone area and liquefaction analysis had been conducted to that area based on Seed-Idriss simplified procedure (1971). Studied areas were Sylhet, Sherpur, Maulavibazar, Sarail, Bhairab Bazar, Narsingdi, Muktagacha. Among them Sherpur-Maulavibazar area had been found to be vulnerable to liquefaction.

Rashid (2000) developed seismic microzonation map of Dhaka city based on site amplification and liquefaction. To analyze liquefaction potential two approach had been used e.g. Seed-Idriss simplified procedure (1971) and method based on topography information by Iwaaki et al. (1982). By using topography information method a liquefaction potential map were produced where information had been taken from engineering geological map of Dhaka city (EPC/MMP, 1991). Then Dhaka city had been divided by grids. Two borehole data had been collected. Then analysis had been done by soil Idriss simplified procedure. Some borehole's soil had been liquefied. There were two recommendations. Firstly, soil parameters like main grain size, D_{50} and fine content, F_c should be tested in laboratory. Secondly, SPT data had been converted correlation equation to shear wave velocity which should have been collected by field test. However, liquefaction analysis method used in this research is updated now and any reclaimed land of Dhaka city was not focused.

Ansary and Rashid (2000) generated liquefaction potential map for Dhaka city. In this research the susceptibility of liquefaction within an area of approximately 200 square feet in the Greater Dhaka Metropolitan area, Bangladesh has been assessed based on standard Penetration Test data from 190 boreholes. The liquefaction potential has been evaluated using two simplified procedures, proposed by Seed & Idriss (1971) and twnsnki et. al. (1982). The analysis results have been classified into two groups to the extent of liquefaction observed, namely, as to whether the site is liquefiable or non-liquefiable. These results have been transformed into a map, which will serve as a general guide to ground-failure susceptibility, effective land use, and efficient town-planning and disaster mitigation. This was the first study initiated in Bangladesh as part of the microzonation investigations.

Hossain et. at. (2003) evaluated sub-soil characteristics and liquefaction potential of Mirpur DOHS area. To characterize soil deposit eight bore holes were drilled at the project site. Moisture content, specific gravity, Atterberg limits, grain size distribution, unconfined compressive strength, density and shear strength parameters of the collected samples were determined in the laboratory. It was observed that geotechnical properties of the soil in the study area varied with depth and location. It was observed that loose soil exists from 4.7 to 14.0 m depth below existing ground level (EGL). From the study, the possibility of the liquefaction had been found to be zero. However, more detailed study has been conducted in this study of this area based on these data and other collected data from different agencies.

Rahman (2004) updated the seismic microzonation maps for liquefaction as well as site amplification due to earthquake. To analyze potential in this research, method described by Seed et al. (1983) have been used.

Islam (2005) estimated the seismic losses scenario of Sylhet city for earthquakes. Liquefaction potential index (P_L) had been determined for the city. It was found that, 115 Km^2 areas has low and very low liquefaction potential, 13.70 Km^2 areas have high liquefaction potential. More than one thirds (37%) area will be affected severely due to liquefaction if Srimongal earthquake (1918) occurs again with same magnitude and cone epicentral distance.

Ahmed (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 includes survey of development procedure of reclaimed areas and laboratory and field tests to determine the sub-soil characteristics of such areas. The study was extended up to determination of liquefaction potential of the selected areas. It has been observed that some parts of the reclaimed areas are susceptible to liquefaction and some parts are not liquefiable.

2.7 Evaluation of Liquefaction Susceptibility

Liquefaction causes a lot of damages to the lifeline structures. Hence, before constructing any structure the soil beneath it should be evaluated properly, Many researchers have presented the liquefaction determination procedures like Japanese code of bridge design (1990) including Chinese criterion (Ishihara, 1990), Seed-Idriss amplified procedure (1971) which have been updated over the years (e.g., Seed et al., 1985; Youd and Idriss, 2001; Cetin et al., 2004 and Idriss & Boulanger, 2004). Liquefaction can be evaluated by various soil criteria, SPT N-value, CPT, BPT and shear wave velocity. Those evaluation procedures have been discussed in brief below.

2.7.1 Evaluation Based on Soil Properties

There are a number of different ways to evaluate the liquefaction susceptibility of a soil deposit based on different criterion. Here they are organized as follows (adopted Kramer, 1996).

1. Historical Criteria

Observations from earlier earthquakes provide a great deal of information about the liquefaction susceptibility of certain types of soils and sites. Soils that have liquefied of the past can liquefy again in future earthquakes. Before constructing a building previous earthquakes can be investigated to see if they caused liquefaction at the site.

2. Geological Criteria

The type geologic process that created a soil deposit has a strong influence on its liquefaction susceptibility. Saturated soil deposits that have been created by alimentation in rivers and lakes (fluvial or alluvial deposits), deposition of debris or eroded material (colluvial deposits), or deposits formed by wind action (aeolian deposits) can be very liquefaction susceptible. These processes sort particles into uniform grain sizes and deposit them in loose state which tends to densify when shaken by earthquakes. The tendency for densification leads to increasing pore water pressure and decreasing strength. Man-made soil deposits, particularly those created by the process of hydraulic filling, may also susceptible to liquefaction.

3. Compositional Criteria

Liquefaction susceptibility depends on the soil type. Clayey soil, particularly sensitive soils, may exhibit strain-softening behavior similar to that of liquefied soil, but do not liquefy in the same manner as sandy soils are. Soils composed of particles that are all about the same size are more susceptible to liquefaction than soils with a wide range of particle sizes. In a soil with many different size particles, the small particles tend to fill in the voids between the bigger particles thereby reducing the tendency for densification and pore water pressure development when shaken. Table 2.4 shows the liquefaction susceptibility based on geomorphologic units (Yasuda, 1988). Table 2.5 shows the characteristics of liquefable soils.

Table 2.4: Liquefaction susceptibility based on geomorphologic units (Yasuda,1988)

Liquefaction Potential	Geomorphologic Unit
High	Reclaimed fill, present and old river beds, young natural levee, interdune lowland
Moderate	Lowlands other than above, fan, nature levee, sand dune, flood plain, beach
Low	Terrace, hill, mountain

The friction between angular particles are higher than between rounded particles, hence a soil deposit with angular particles is normally stronger and less susceptible to liquefaction. So, a uniformly graded soil is more susceptible to soil liquefaction than a well-graded soil because the reduced tendency for volumetric strain of a well-graded soil decreases the amount of excess pore pressure that can develop under untrained conditions. Historically, sands were considered to be the only type of soil susceptible to liquefaction, but liquefaction has also been observed in gravel and silt. Strain-softening of fine grained soils can produce effects similar to those of liquefaction. Fine-grained soils are susceptible to this type of behavior if they satisfy the criteria (Wang, 1979) shown below.

- Fraction finer 4.005 mm ≤ 15%
- Liquid Limit, LL ≤ 35%
- Natural water content ≥ 0.9 LL
- Liquidity Index ≤ 0.7

Liquefaction susceptibility also depends on particle shape. Soil deposits with rounded particles, usually found in the types of deposits described in geological criteria, are more susceptible to liquefaction than soils with angular particles.

State Criteria :

The initial "state" of a soil is defined by its density and effective stress at the time it is subjected to rapid loading. At a given effective stress level, loose soils are more susceptible to liquefaction than dense soils Here have been described that are of importance to the liquefaction susceptibility. At constant pressure, the liquefaction resistance increases with the relative density, D_r, and, at constant relative density, the liquefaction resistance increases with increasing pressure. Various investigations (Castro, 1969 and Kramer & Seed, 1988) have shown that pre-existing shear static stress in a soil deposit is critical to a soil's susceptibility to static liquefaction. The higher the initial shear stresses, the greater are the liquefaction potential and the smaller disturbance is needed to liquefy the soil.

2.7.2 Evaluation Based on Laboratory Tests

Several laboratory tests are in practice to estimate the liquefaction potential of soils such as cyclic triaxial test, cyclic simple shear test and shaking table test etc. Among those cyclic triaxial test is more popular. The cyclic triaxial test has been used extensively in the study of soil subjected to simulated earthquake loading. Laboratory test procedures are described in ASTMD 5311-96 (2000).

Table 2.5 Characteristics of liquefiable soil.

Characteristics	Value
Mean size, D_{50} (mm)	0.02 to 1.00
Fines content (d 0.005 mm)	< 10%
Uniformity coefficient, C_u	< 10
Relative density, D_r	< 75%
Plasticity index, I_p	< 10
Earthquake Intensity	> VI
Depth	< 20 m

2.7.3 Evaluation of Liquefaction Based on In Situ Tests

The empirical criteria have been developed over the years based on standard penetration test (SPT), cone penetration test (CPT) and shear wave velocity for the assessment of liquefaction potential (Seed and Idriss, 1971; Seed and Peacock, 1971 et al., 1982; Seed et al., Robertson and Campanclla, 1985; Youd). These procedures are briefly discussed below.

1. Evaluation Based on SPT N-Value

The most common index properties for estimating liquefaction strength is the N-value obtained from the standard penetration test. The standard penetration test (SPT) consists of driving a thick-walled sampler into the granular soil deposit. The measured SPT N-value (blows per foot) is defined as the penetration resistanceof the soil, which equals the sum of the number of bows required to drive the SPT sampler over the depth interval of 15 to 45 cm (6 to 18 in).

Liquefaction analysis, the corrected standard penetration test N-value is used. with the limitations and all the corrections that must be applied to the measured value, the standard penetration test is probably the most widely used field test in the Bangladesh as well as in the world. This is because it is relatively easy to use, the test is ecomonical compared to other types of field testing, and the SPT equipment can be quickly adapted and included as part of almost any type of drilling rig.

The most comprehensive liquefaction data catalogs are based on Standard Penetration test (SPT) blow counts (SPT N). Starting in the 1970's H.B. Seed and his colleagues worked to develop a reliable method for assessing liquefaction potential based on SPT data. Their framework for SPT-based assessments of liquefaction potential was developed in a series of papers that includes & Idriss (1971), Seed & Idriss (1982) and Seed et al. (1983) significant contributions were also suggested in the of Tokimatsu & Yoshimi (1983). This research culminated in the liquefaction criteria published by Seed et al. (1985).

The empirical chart published by Seed et al. (1985) is based on a standardized SPT blow count, $(N_1)_{60}$ and the cyclic stress ratio (CSR).To get $(N_1)_{60}$, the measured N_{SPT} crotected for the energy delivered by different hammer systems and normalized with respect to overburden stress. Boundary curves separating liquefied from liquefied soils, in terms of CSR and $(N_1)_{60}$ were conservatively drawn to nearly al observed cases of liquefaction in the data catalog. Three separate boundary curves were presented for clean to silty sands. To consider the effects of earthquake magnitude on the duration of strong shaking, magnitude scaling factors were pecified. Over the last decade, the empirical method given by Seed et al. (1985), sometimes referred to as the "simplified procedure", has been widely used for valuating soil liquefaction potential in all around the world.

Finally, liquefaction assessments in Japan are often performed using the SPT-based, empirical method specified in the Japanese code of bridge design (1990). This method was developed in Japan from a large number of cyclic triaxial tests on soil samples with a known SPT penetration resistance (Ishihara, 1985 and 1993). Hence, this empirical method is based largely on laboratory tests, where sample disturbance is a potential issue, in contrast to the direct correlation with field behavior used in the other methods described here.

2. Evaluation Based on CPT

For estimating liquefaction potential by in-situ test other than the SPT, the most advanced ones are those using cone penetration resistances. The idea for the cone penetration test is similar to the standard penetration test except that instead of driving a thick-walled sampler into the soil, a steel cone is pushed into the soil. The force required to move the cone into the extended position divided by the horizontally projected area of the cone is defined as the cone resistance, q_c. For liquefaction analysis, the cone penetration test value q_c is corrected for the vertical effective stress. A major advantage of the cone penetration test is that by using the electric cone, a continuous subsurface record of the cone resistance q_c can be obtained. This is in contrast to the standard penetration test, which obtains data at intervals in the soil deposit. Disadvantage of the cone penetration lest are that soil samples cannot be recovered and special equipment is required to produce a steady and slow penetration of the cone. Unlike the SPT, the ability to obtain a steady and slow penetration of the cone is not included as part of conventional drilling rigs. Because of these factors, in Bangladesh, the CPT is used less frequently than the SPT.

The Cone Penetration test (CPT) yields a continuous profile of penetration resistance and is thus well-equipped for detecting thin, liquefiable layers within a larger, stable soil deposit. Early CPT-based empirical methods for liquefaction evaluations were developed by converting SPT blow counts in the available liquefaction case studies to equivalent CPT tip resistances. In a different approach, Mitchell and Tseng (1990) used a model of cone penetration together with laboratory test data to suggest a liquefaction criteria using the CPT. However, these methods suffer from a lack of direct correlation between the measured CPT tip resistance and observed field performance of liquefiable soils in earthquakes. Using data from sites mostly in China, Shibata and Teparaksa (1988) developed a CPT-based database, Stark and Olson (1995) also developed an empirical method based on measured CPT tip resistances. Stark and Olson used a normalized tip resistance and drew bounding curves between liquefied and unliquefied states for clean sand, silty sand, and sandy silt.

3. Evaluation Based on BPT

In the late 1950's, the Becker Penetration Test (BPT) was developed in Canada as a penetration type test to provide index properties similar to the SPT N-Value, but in soils with significant gravel content. The BPT is performed using a 6-5/8 inch drill rod, which consists of a double-walled system arranged such that the driving forces are carried by the outer pipe, while the inner pipe floats independently. The Becker drill string is often advanced with a close end. To advance the larger diameter rod, the BPT has traditionally used a double acting diesel hammer. Similar to SPT, the data from a BPT is logged as the number of lows required to advance the rod one foot, which yields the BPT blow count, N_b. Similar to the SPT, N_b is corrected to a constant transferred energy, as measured by either direct stress wave measurements or by monitoring the hammer's bounce chamber pressure.

In the last two decades, the BPT has been used, via correlation to SPT N_{60}, as a way estimatethe liquefaction potential of gravel deposits. This method has been used by the bureau of Reclamation to account for the seismic risk of earth fill dams. Harder and seed (1986) initially developed a correlation between N_b and N_{60}; Sy and Campanella (1993) extended the number of data points in the correlation, while simultaneously investigating the effects of the resistance developed along the side of the casing on the value of N_b. Sy and Campanella (1993) suggested the following tips for correcting the BPT data and correlating it to N_{60} from an SPT:

- Monitor BPT test during driving to determine energy transfer
- Correct the recorded blow count to N_{B30}.
- Select stress wave date from blows obtained at critical depths to determine shaft resistance with CAPWAP (PDI, 2006) or using static methods.
- Using N_{B30} and the measured shaft resistance, determine equivalent SPT N_{60}.

Once N_{60} has been determined, existing methods for evaluating the liquefaction potential through correlation to cyclic stress ratio (for example, Youd and Idriss, 2001) can be used.

4. Evaluation Based on Shear Wave Velocity

The earthquake ground shaking consists of body waves (P-wave and S-wave) and surface waves (Rayleigh and love waves). Among these, the shear deformation due to S-waves traveling vertically upward has a significant influence on the occurrence of soil liquefaction.

Many of the same factors that contribute to the liquefaction resistance of a soil deposit (density, confinement, stress history, geologic age, etc.) also influence the velocity of traveling shear waves (Robertson et al., 1992). Moreover, the shear wave velocity of a soil deposit can be measured economically with surface geophysics; this is particularly advantageous in evaluating gravelly soils that are difficult to penetrate or simple, Hence, several researchers have attempted to correlate liquefaction potential with in situ shear wave velocity. Robertson et al. (1992) present correlations, in terms of a normalized shear wave velocity and cyclic stress ratio that were developed directly from a limited number are reviewed by Andrus & Stokoe (1997). The method for evaluation liquefaction potential based on shear wave velocities is described by Andrus & Stokoe (2000) which is used in this study.

2.8 Cone Penetration Test

The cone penetration test (CPT) is an in situ testing method used to determine the geotechnical engineering properties of soils and delineating soil stratigraphy. It has been initially developed in the 1950s at the Dutch Laboratory for Soil Mechanics in Delft to investigate soft soils. Based on this history it has also been called the "Dutch cone test". Today, the CPT is one of the most used and accepted in situ test methods for soil investigation worldwide.

The standardized cone-penetrometer test (CPT) involves pushing a 1.41-inch diameter 55° to 60° cone through the underlying ground at a rate of 1 to 2 cm/sec. CPT soundings can be very effective in site characterization, especially sites with discrete stratigraphic horizons or discontinuous lenses. Cone penetrometer testing or CPT (ASTM D-3441, adopted in 1974) is a valuable method of assessing subsurface stratigraphy associated with soft materials, discontinuous lenses, organic materials (peat), potentially liquefiable materials (silt, sands and granular gravel) and landslides. Cone rigs can usually penetrate normally consolidated soils and colluvium, but have also been employed to characterize weathered Quaternary and Tertiary-age strata. Cemented or unweathered horizons, such as sandstone, conglomerate or massive volcanic rock can impede advancement of the probe, but it has always been able to advance CPT cones in materials of Tertiary-age sedimentary rocks. The cone is able to delineate even the smallest (0.64 mm/1/4-inch thick) low strength horizons, easily missed in conventional (small-diameter) sampling programs. Some examples of CPT electronic logs are attached, along with hand-drawn lithologic interpretations. Most of the commercially-available CPT rigs operate electronic friction cone and piezocone penetrometers, whose testing procedures are outlined in ASTM D-5778, adopted in 1995. These devices produce a computerized log of tip and sleeve resistance, the ratio between the two, induced pore pressure just behind the cone tip, pore pressure ratio (change in pore pressure divided by measured pressure) and lithologic interpretation of each 2 cm interval have been continuously logged and printed out.

The early applications of CPT mainly determined the soil geotechnical property of bearing capacity. The original cone penetrometers involved simple mechanical measurements of the total penetration resistance to pushing a tool with a conical tip into the soil. Different methods were employed to separate the total measured resistance into components generated by the conical tip (the "tip friction") and friction generated by the rod string. A friction sleeve was added to quantify this component of the friction and aid in determining soil cohesive strength in the 1960s (Begemann, 1965). Electronic measurements began in 1948 and improved further in the early 1970s (de Reister, 1971). Most modern electronic CPT cones now also employ a pressure transducer with a filter to gather pore water pressure data. The filter has been usually located either on the cone tip (the so-called U1 position), immediately behind the cone tip (the most common U2 position) or behind the friction sleeve (U3 position). Pore water pressure data aids determining stratigraphy and is primarily used to correct tip friction values for those effects. CPT testing which also gathers this piezometer data has been called CPTU testing. CPT and CPTU testing equipment generally advances the cone using hydraulic rams mounted on either a heavily ballasted vehicle or using screwed-in anchors as a counter-force. One advantage of CPT over the Standard Penetration Test (SPT) is a more continuous profile of soil parameters, with CPTU data recorded typically at 2 cm intervals.

TYPES OF CONE PENETRATION TEST

A. Mechanical the earliest type, often called the Dutch cone since it has been first developed and used in the Netherlands.

B. Electric friction-first modification using strain gauges to measure q_c (Point Resistance) and q_s (Side friction).

C. Electric piezo-a modification of the electric friction cone to allow measuring the pore water pressure during the test at the cone tip.

D. Electric piezo/friction-a further modification to measure point resistance, sleeve friction, and pore pressure.

E. Seismic cone-a further modification to include a vibration sensor to obtain data to compute the shear wave velocity from a surface hammer impact so that the dynamic shear modulus can be computed [Robertson et al. (1985)].

2.8.1 Component of Cone Penetration Test

TIP RESISTANCE:

The tip resistance has been measured by load cells located just behind the tapered cone .The tip resistance has been theoretically related to undrained shear strength of a saturated cohesive material, while the sleeve friction is theoretically related to the friction of the horizon being penetrated (Robertson and Campanella, 1986, Guidelines for Use and Interpretation of the Electric Cone Penetration Test, 3rd Ed.: Hogentogler & Co., Gaithersburg, MD, 196 p.). The tapered cone head forces failure of the soil about 15 inches ahead of the tip and the resistance has been measured with an embedded load cell in tons/ft^2 (tsf).

LOCAL FRICTION:

The local friction has been measured by tension load cells embedded in the sleeve for a distance of 4 inches behind the tip. They measure the average skin friction as the probe has been advanced through the soil. If cohesive soils have been partially saturated, they may exert appreciable skin friction, negating the interpretive program.

FRICTION RATIO:

The friction ratio has been given in percent. It is the ratio of skin friction divided by the tip resistance (both in tsf). It has been used to classify the soil, by its behavior, or reaction to the cone being forced through the soil. High ratios generally indicate clayey materials while lower ratios are typical of sandy materials (or dry desiccated clays). Typical skin friction to tip friction ratios are 1 % to 10%. The ratio seldom, if ever, exceeds 15%. Sands are generally identified by exhibiting a ratio < 1%.

PORE PRESSURE:

Piezocones also measure insitu pore pressure (in psi), in either dynamic (while advancing the cone) or static (holding the cone stationary) modes. Piezocones employ a porous plastic insert just behind the tapered head that is made of hydrophilic polypropylene, with a nominal particle size of 120 microns. The piezocell must be saturated with glycerin prior to its employment. The filter permeability is about 0.01 cm/sec (1 x 10-2 cm/sec). When using the cone to penetrate dense layers, such as cemented siltstone, sandstone or conglomerate, the piezo filter element can become compressed, thereby inducing high positive pore pressures. But, the plastic filters do not exhibit this tendency, though they do become brittle with time and may need to be replaced periodically. In stiff over-consolidated clays the pore pressure gradient around the cone may be quite high. This pore pressure gradient often results in dissipations recorded behind the CPT tip that initially increase before decreasing to the equilibrium value.

2.9 Standard Penetration Test (SPT)

The standard penetration test, developed in 1927, is currently the most popular and economical means to obtain subsurface information. It has been estimated that 85 to 90 percent of conventional foundation design in North and South America is made using the SPT. This test is also widely used in other geographic regions. The method has been standardized as ASTM D 1586 since 1958 with periodic revisions to date. The test consists of:

1. Driving the standard split-barrel sampler of dimensions. a distance of 460 mm (18 in) into the soil at the bottom of the boring.

2. Counting the number of blows to drive the sampler the last 305 mm (12 in) to obtain the N number. Table 2.6 shows the penetration resistance and Soil Properties on Basis of the Standard Penetration Test (SPT).

3. Using a 63.5 kg (140 lb) driving mass (or hammer) falling "free" from a height of 760 mm (30 in). Several hammer configurations have been shown in Figure 2.7.

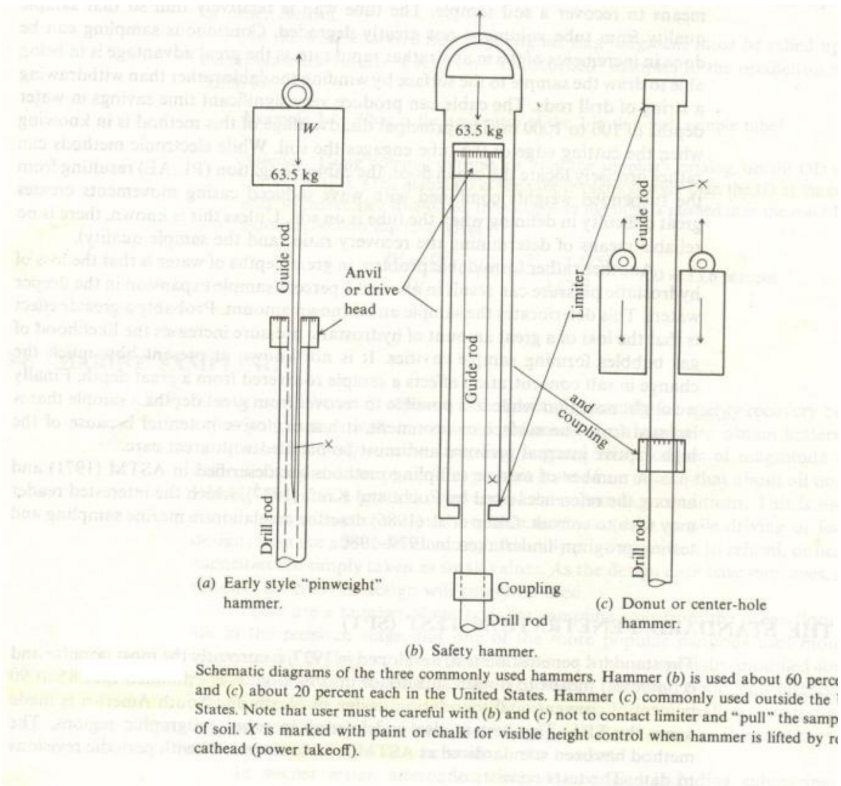


Fig 2.7: Several hammer configurations.

Table 2.6: Penetration Resistance and Soil Properties on Basis of the Standard Penetration Test (SPT).

Sands		Clays	
Number of blows per ft, N	**Relative Density**	**Number of blows per ft, N**	**Consistency**
		Below 2	Very soft
0-4	Very loose	2-4	Soft
4-10	Loose	4-8	Medium
10-30	Medium	8-15	Stiff
30-50	Dense	15-30	Very stiff
Over 50	Very dense	Over 30	Hard

2.10 Liquefaction analysis using CPT

Liquefaction potential based on CPT have been determined by Robertson and Wride, 1998. Flow Chart 2.1 shows the liquefaction potential analysis using Robertson method. This method has been described step by step:

Step 1:

From the CPT test the available data are tip resistance (q_c), sleeve friction (f_s). Then we can find out in situ stress (σ_{v0}, $\sigma_{v0'}$). Where σ_{v0} and $\sigma_{v0'}$ are the overburden pressure and effective overburden pressure respectively.

Step 2:

Correction factor for grain characteristics:

$Q= (q_c- \sigma_{v0})\ \sigma_{v0'}$ $F= f_s/(q_c- \sigma_{v0})*100\%$

$I_c=[(3.47-\log Q)^2 +(\log F+1.22)^2]^{0.5}$

Here Q is Normalized cone resistance.

F is Normalized friction ratio.

I_c is soil behavior type index.

When I_c <2.6

$q_{c1N} =(q_c /P_{a2})(p_a/\sigma_{v0'})^{0.5}$ here p_a=100 kPa if $\sigma_{v0'}$ is in kPa

P_{a2} =0.1 MPa if q_c is in MPa.

Here q_{c1N} is corrected cone penetration resistance for overburden stress.

p_a is a reference pressure in same unit $\sigma_{v0'}$

P_{a2} is a reference pressure in same unit q_c.

When I_c >2.6

q_{c1N} =Q

step 3:

if I_c <1.64 k_c=1.0

if I_c >1.64 k_c=-0.403I_c^4 + 5.58I_c^3 -21.63I_c^2 +33.75I_c -17.88

if I_c >2.6 evalute using other criteria; likely non-liquefiable if F>1% as well. BUT if , 1.64<I_c >2.36 and F<0.5% , set k=1.0

$$(q_{c1N})_{cs} = (k_c)(q_{c1N})$$

Here k_c is CPT grain characteristic correction factor.

Step 4:

If 50<$(q_{c1N})_{cs}$<160 $CRR_{7.5}$ = 93*(($q_{c1N})_{cs}$ /1000) +0.08

If $(q_{c1N})_{cs}$<50 $CRR_{7.5}$ = 0.833**(($q_{c1N})_{cs}$ /1000) +0.05

Here Figure 2.8 shows the relation between CRR and corrected CPT Tip Resistance.

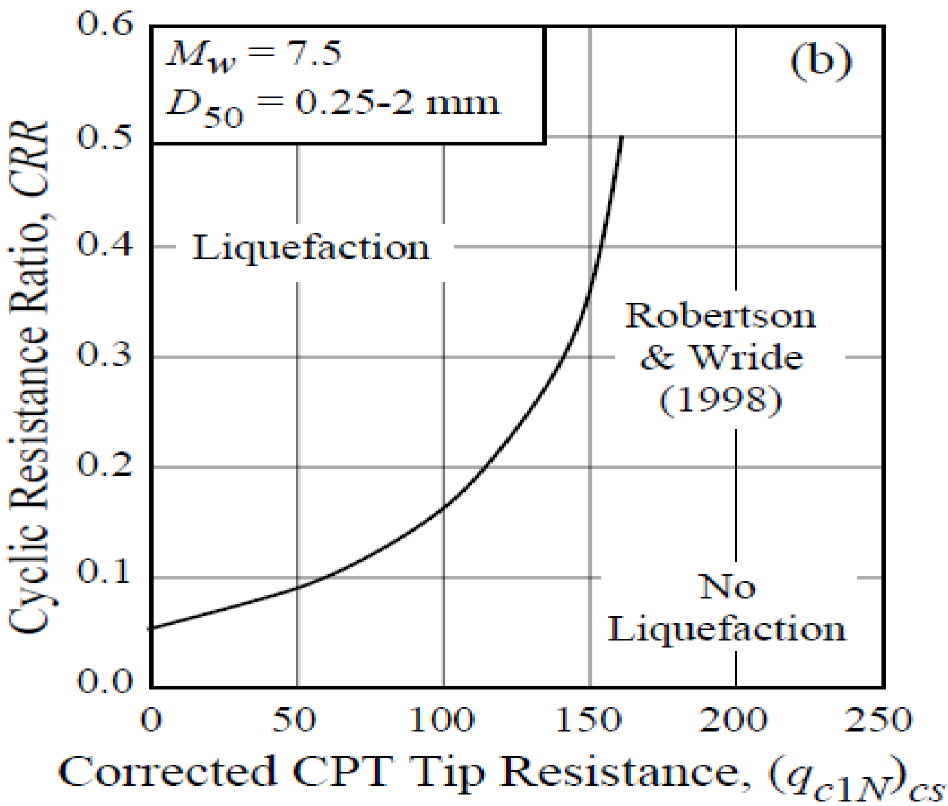


Fig 2.8 : CRR vs corrected CPT Tip Resistance

Step 5:

Stress reduction factor:

$$r_d = \begin{cases} (130 - z)/131 ; & \text{if } z<9.15 \text{ m} \\ (44 - z)/37 ; & \text{if } 9.15\text{m} \leq z \leq 23\text{m} \\ (93 - z)/125 ; & \text{if } 23\text{m} \leq z \leq 30\text{m} \\ 0.5 ; & \text{if } r_d>30\text{m} \end{cases}$$

Critical stress ratio induced by earthquake:

$$CSR_{eq} = 0.65 \text{ x } (a_{max}/g) \text{ x } r_d \text{ x } (\sigma_v/\sigma'_v)$$

Corrected Critical Stress Ratio Resisting Liquefaction:

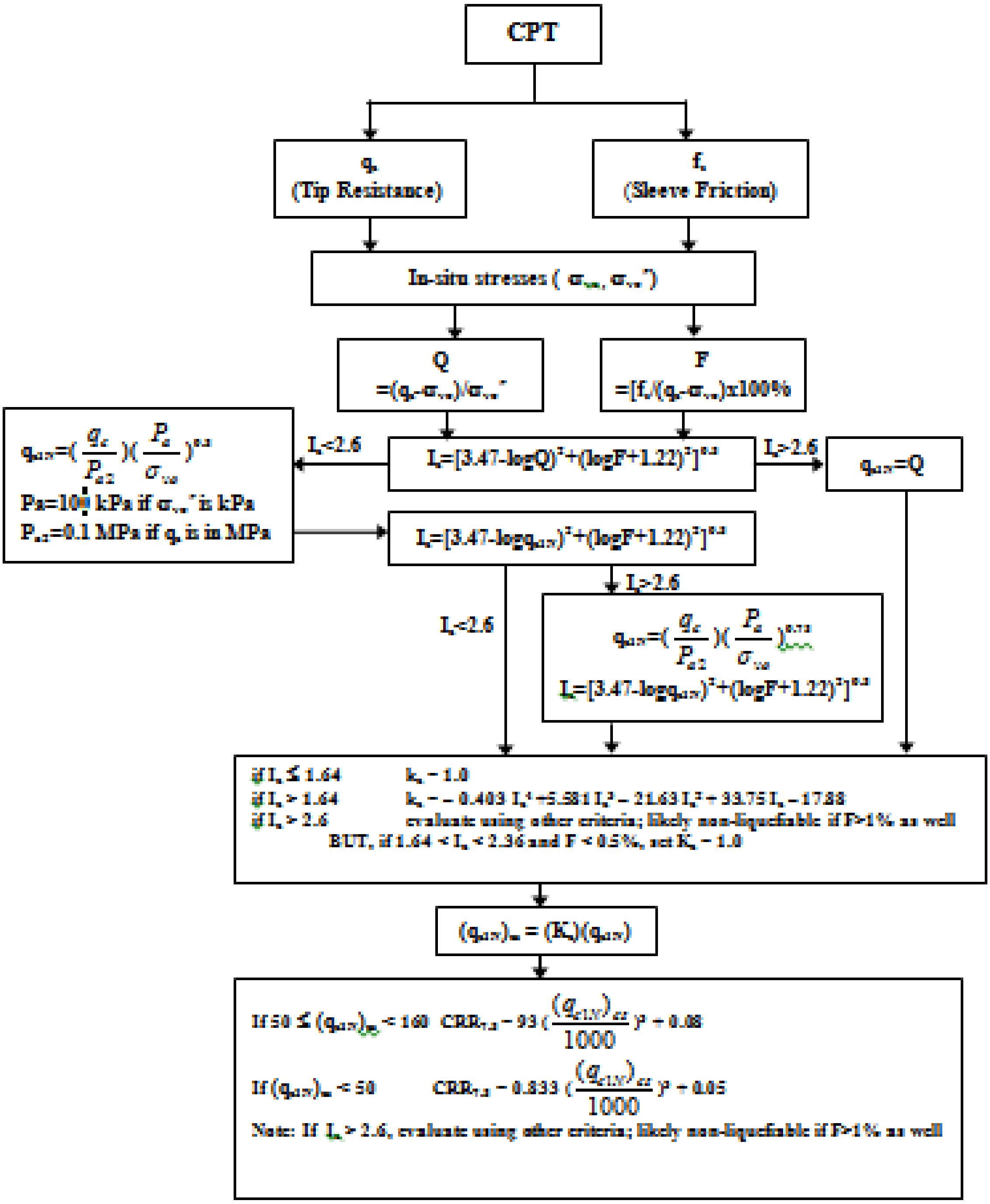
$CSR_l = CSR_{eq}$ x k_m x k_α x k_s

k_m=	Correction factor for earthquake magnitude other than 7.5
	1.00
k_α=	Correction factor for initial driving static shear
	1.00
k_s=	Correction factor for stress level larger than 96 KPa
	1.00

STEP 6:

Factor of safety against liquefaction:

$FS = CRR/CSR_l$



Flow Chart 2.1: Liquefaction potential analysis (Robertson,1998).

2.11 Liquefaction analysis using SPT

Liquefaction potential based on SPT have been determined by Seed et al;1983. This method has been described step by step:

Step 1:

From the SPT test the available data are SPT value (N). Then we can find out in situ stress (σ_v, $\sigma_{v'}$). Where σ_v and $\sigma_{v'}$ are the overburden pressure and effective overburden pressure respectively

Step 2:

Stress reduction factor:

$r = 1 - 0.015z$ here z= depth from ground level.

Critical stress ratio induced by earthquake:

$$\mathbf{CSR_{eq} = 0.65 \times (a_{max}/g) \times r_d \times (\sigma_v/\sigma'_v)}$$

Correction for SPT (N) value for overburden pressure:

$(N)_{60} = C_N * N_{60}$

$C_N = 9.79 * (1/\sigma_{v'})^{1/2}$

Critical stress ratio resisting liquefaction:

$CSR_{7.5}$ has been found from the following graph (Fig 2.9). Figure 2.10 shows the relation between cyclic Stress Ratio and modified penetration Resistance

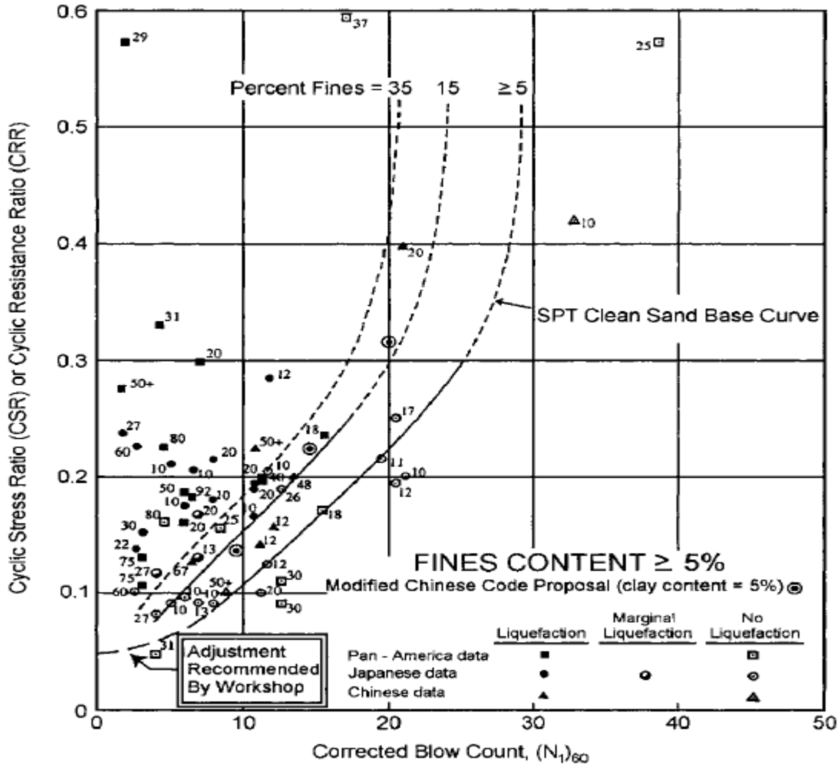


Fig 2.9: CSR vs Corrected blow count (Seed et al;1983).

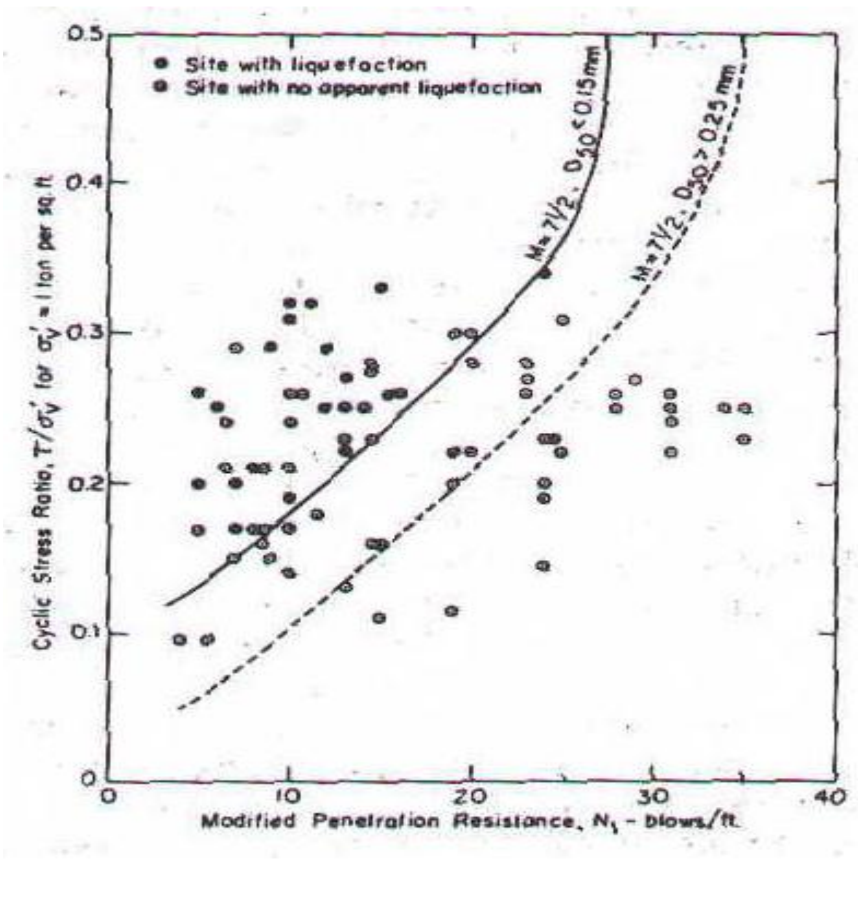


Fig 2.10: Cyclic Stress Ratio vs Modified penetration Resistance (Seed et al;1983)

Corrected Critical Stress Ratio Resisting
Liquefaction:

$CSR_l = CSR_{7.5} \times k_m \times k_\alpha \times k_s$

k_m=	Correction factor for earthquake magnitude other than 7.5
	1.00
k_α=	Correction factor for initial driving static shear
	1.00
k_s=	Correction factor for stress level larger than 96 KPa
	1.00

STEP 3:

Factor of safety against liquefaction:

$FS = CSR_l / CSR_{eq}$

2.12 Soil Parameter evaluation based on CPT

Soils are very complex materials because they can comprised of a wide and diverse assemblage of different particles sizes, mineralogies, packing arrangements, and fabric. Moreover, they can be created from various geologic origins (marine, lacustrine, glacial,residual, Aeolian, deltaic,alluvial, estuarine,fluvial, biochemical, etc) that have undergone long periods of environmental, seasonal, hydrological and thermal processes. These facets have imparted complexities of soil behavior that relate to their initial geostatic strss state, natural prestressing, nonlinear stress-strain-strength response, and drainage and flow characteristics, as well as theological and time rate effects. As such, a rather large number of different geotechnical parameters have been identified to quantify soil behavior in engineering terms. These include state parameters such as void ratio (e_0), unit weight (γ), porosity (n), relative density (D_R), overconsolidation ratio (OCR), strength parameters (c, ϕ), permeability (k), lateral stess parameters (k) and more. In this section, the evaluation of select geotechnical parameters from CPT data has been addressed including various post processing approaches based on theoretical, numerical, analytical and empirical methods.

The stress history of clay soils is classically determined from one dimensional oedometer tests on high quality undisturbed samples. The yield point in one dimensional loading (i.e., consolidation test) denotes the preconsolidation stress (σ_p'), formerly designated σ_{vmax}' or p_c'. In normalized form, the degree of preconsolidation has been termed the over consolidation ratio (OCR) from equation 2.1 below.

$$OCR = (\sigma_p'/ \sigma_{vo}') \qquad 2.1$$

For intact clays , a first order estimated of the preconsolidation stress can be obtained from net cone tip resistance (Mayne 1995) by equation 2.2 below.

$$\sigma_p' = 0.33(q_t - \sigma_{vo}) \qquad 2.2$$

For purposes here in , the sands are primarily siliceous (quartz and feldspar) with applied stress histories ranging from NC to overconsolidated states (1<=OCR<=15). Multiple regression analyses of the chamber test data (n=636) from anisotropically consolidated sands indicate that the induced OCR is a function of the applied effective vertical stress (σ_{vo}'), effective horizontal stress [$\sigma_{ho}' = k_0 \cdot \sigma_{vo}'$] and measured cone tip resistance (q_t). Here, the OCR has been shown normalized by $Q= (q_t- \sigma_{vo})/ \sigma_{vo}'$. The results can be presented by the following closed form expression (equation 2.3).

$$OCR = [\{0.192(q_t/ \sigma_{atm})\text{^}0.22\}/\{(1-\sin\phi').(\sigma_{vo'}/ \sigma_{atm})\text{^}0.31\}]\text{^}(1/(\sin\phi' -0.27) \qquad 2.3$$

Where ϕ' = effective stress friction angle of the sand, σ_{vo}'= effective overburden stress, and σ_{atm} = a reference stress equal to one atmosphere = 1 bar = 100 KPa .

From the OCR the apparent preconsolidation stress of the sand can be calculated from equation 2.4 below.

$$\sigma_p' = OCR \cdot \sigma_{vo}' \qquad 2.4$$

The strength of soils is controlled by the effective stress frictional envelope, often represented in terms of the Mohr Coulomb parameters: ϕ' = effective stress friction angle of the sand and C' = effective cohesion intercept. Effective stress friction angle have been found from equation 2.5 below.

$$\phi' = 17.6^{\circ} + 11.0^{\circ} * \log(q_{r1}) \quad 2.5$$

where $q_{r1} = (q_t / \sigma_{atm})/(\sigma_{vo}' / \sigma_{atm})^{0.5}$ is a more appropriate form for stress normalization of CPT results in sands (e.g., Jamiolkowski et al. 2001)

The geostatic horizontal stress has been represented by the K_0 coeficient, where $k_0 = \sigma_{ho}' / \sigma_{vo}'$. In general, laboratory data on small triaxial specimens and instrumented oedometer tests indicate that the following relationship have been adopted in uncemented sands and well behaved clays of low to medium sensitivity. K_0 have been found from equation 2.6 below.

$$K_0 = (1 - \sin \phi') \, OCR^{\sin \phi'} \quad 2.6$$

For the clean sands, data from large calibration chamber tests and small laboratory triaxial and oedometer test series show the K_0 – OCR trends . Related to the previous OCR, the derived formulation for the lateral stress from chamber tests and expressed by equation 2.7 below.

$$K_0 = 0.192 * (q_t / \sigma_{atm})^{\wedge 0.22} . (\sigma_{atm} / \sigma_{v0'})^{\wedge 0.31} . (OCR)^{\wedge 0.27} \quad 2.7$$

2.13 Correlation between CPT and SPT

The Standard Penetration Test (SPT) is the most common in situ test for site investigations and most of foundation designs have been based on SPT-N values and physical properties of soils recovered in the SPT sampler. The SPT has some disadvantages such as potential variability of measured resistances depending on operator variability and possibility of missing delicate changes of soil properties owing to the inevitable discrete record. Around the world the Cone Penetration Test (CPT) is becoming increasingly popular as an in situ test for site investigation and geotechnical design especially in deltaic areas since it provides a continuous record which is free from operator variability (Suzuki et al. 1998). Thus there is a need for reliable CPT-SPT correlations so that CPT data can be used.

2.13.1 Previous Works

It is very valuable to correlate the static cone tip resistance q_c, to SPT N-value so that the available database of the field performances and property correlations with N value could be effectively utilized. Hence many empirical relations have been established between the SPT N-values and CPT cone bearing resistances, q_c (Robertson et all. 1983). Table 2.7 summarizes most of the works. Table 2.8 shows the previous works for CPT and SPT correlations.

Table 2.7 Summary of previous SPT-CPT correlations (Salehzadeh et al, 2011)

Researcher(s)	**Relationship**
Robertson et al. (1983)	$q_c/N= f\ (d_{50})$
Seed & DeAlba(1983)	
Kulhawy & Mayne(1990)	
Stark & Olson(1995)	
Emrem et al. (2000)	
Muromachi (1981)	$q_c/N= f\ (F_C)$
Jamiolkowski et al. (1985)	
Kasim et al. (1986)	
Chin et al.(1988)	
Kulhawy & Mayne(1990)	
Jefferies & Davies (1993)	
Robertson (1986),	Soil classification chart with suggested q_c/N ratio for each soil behavior type
Lunne et al.(1997)	
De Alencar Velloso(1959)	Suggested constant values for q_c/N or $(q_c+f_s)/N$ in different soil types
Meigh & Nixon(1961)	
Franki Piles(1960)	
Schmertmann(1970)	
Barata et al.(1978)	
Ajayi & Balogun(1988)	
Chang(1988)	
Danziger & de Valleso(1995)	
Danziger et al.(1998)	
Akca(2003)	

Table 2.8 Previous works for CPT and SPT correlations (Kara, O. and Gunduz, Z)

Author(s)	Soil Types	Relationship
(De Alencar Velloso 1959)	Clay and silty clay	$n = q_c / N = 0.35$
	Sandy clay and silty sand	$n = q_c / N = 0.2$
	Sandy silt	$n = q_c / N = 0.35$
	Fine sand	$n = q_c / N = 0.6$
	Sand	$n = q_c / N = 1.00$
(Meigh & Nixon 1961)	Coarse sand	n = qc / N = 0.2
	Gravelly sand	$n = q_c / N = 0.3\text{-}0.4$
(Franki Piles 1960) from (Akca 2003)	Sand	$n = q_c / N = 1.00$
	Clayey sand	$n = q_c / N = 0.6$
	Silty sand	$n = q_c / N = 0.5$
	Sandy clay	$n = q_c / N = 0.4$
	Silty clay	$n = q_c / N = 0.3$
	Clays	$n = q_c / N = 0.2$
(Schmertmann 1970)	Silt, sandy silt and silt-sand mix	. $n = (q_c+f_s)/N= 0.2$
	Fine to medium sand, silty sand	$n = (q_c+f_s)/N= 0.3\text{-}0.4$
	Coarse sand, sand with gravel	$n = (q_c+f_s)/N= 0.5\text{-}0.6$
	Sandy gravel and gravel	$n = (q_c+f_s)/N= 0.8\text{-}1.0$
(Barata et al. 1978)	Sandy silty clay	$n = q_c / N *= 1.5\text{-}2.5$
	Clayey silty sand	$n = q_c / N * = 2.0\text{-}3.5$
(Ajayi & Balogun 1988)	Lateritic sandy clay	$n = q_c / N * = 3.2$

Authors	Soil types	Relationship
(Chang 1988)	Sandy clayey silt	$n = q_c$ / N *= 2.1
	Clayey silt, sandy clayey silt	$n = q_c$ / N *= 1.8
(Danziger & de Valleso 1995)	Silt, sandy silt and silt-sand	$n = (q_c+f_s)/N = 0.2$
	Fine to medium sand, silty sand	$n = (q_c+f_s)/N = 0.3\text{-}0.4$
	Coarse sand, sand with gravel	$n = (q_c+f_s)/N = 0.5\text{-}0.6$
	Sandy gravel and gravel	$n = (q_c+f_s)/N = 0.8\text{-}1.0$
	Silt, sandy silt and silt-sand	$n = (q_c+f_s)/N = 0.2$
	Silty sand	$n = q_c$ / N * = 7.0
(Danziger et al. 1998)	Sand	$n = q_c$ / N * = 5.7
	Silty sand, Silty clay	$n = q_c$ / N * = 5.0-6.4
	Clayey silt	$n = q_c$ / N * = 3.1
	Clay, silt and sand mixtures	$n = q_c$ / N * = 1.0-3.5
	Clayey sand and silty clay	$n = q_c$ / N * = 4.6-5.3
	Sandy clay	$n = q_c$ / N * =1.8-3.5
	Clay	$n = q_c$ / N * =4.5
(Emrem et al. 2000) (Akca 2003)	Turkey soils	$n = q_c/N = f(d_{50})$
	Sand	$n = q_c / N = 0.77$
	Silty sand	$n = q_c / N = 0.70$

2.13.2 CPT/SPT Correlations

CPT/SPT Correlations depends on several factors**.** Figure 2.11 shows the correlation between CPT and SPT data with grain size data.

- Energy level delivered to SPT - use N_{60}
- Grain size distribution (d_{50})
- Fines content (FC)
- Overburden stress + other factors

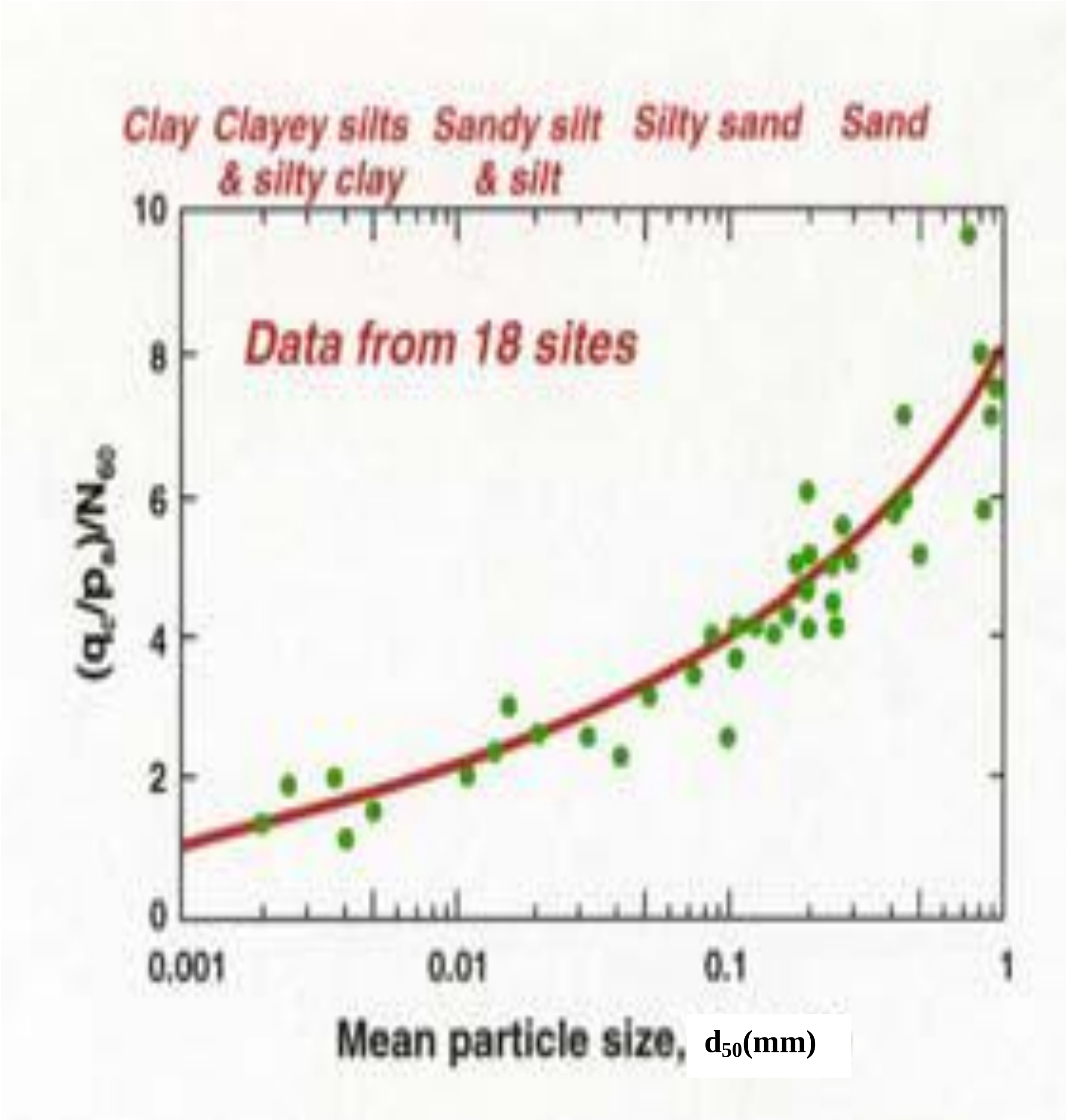


Fig 2.11: Correlation between CPT and SPT data with grain size data (Robertson and Campanella,1983).

Due to lack of soil grain size data, use Robertson (1990) soil classification chart to define soil behaviour type index.

$Q= (q_c- \sigma_{v0})\ \sigma_{v0'}$ $F= f_s/(q_c- \sigma_{v0})*100\%$

$I_c=[(3.47\text{-}\log Q)^2 +(\log F+1.22)^2]^{0.5}$

p_a= atm. Press. = 100 kPa

N_{60}: SPT value corresponding to energy ratio of 60

CPT Software:

A software is used named CPeT-IT (version: v.1.7.6.3) finding probable soil class at the different layers of the soil profile. Table 2.9 shows the boundaries of soil behavior type.

Table 2.9 : Boundaries of soil behavior type (After Robertson 1990)

Soil behaviour type index I_c	Soil behaviour type
$I_c < 1.31$	Gravelly sand to dense sand
$1.31<I_c<2.05$	Sands : clean sand to silty sand
$2.05<I_c<2.60$	Sand mixture: silty sand to sandy silt
$2.60<I_c<2.95$	Silt mixture: clayey silt to silty clay
$2.95<I_c<3.60$	Clays: silty clay to clay
$I_c>3.60$	Organic soils: peats

2.14 Methods to Mitigate Liquefaction

There are basically three possibilities to reduce liquefaction hazards when designing and constructing new buildings or other structures as bridges, tunnels, and roads.

1. Avoiding Liquefaction Susceptible Soils

The first possibility is to avoid construction on liquefaction susceptible soils. There are various criteria to determine the liquefaction susceptibility of a soil. By characterizing the soil at a particular building site according to these criteria one can decide if the site is susceptible to liquefaction and therefore unsuitable for the desired structure.

2. Building Liquefaction Resistant Structures

If it is necessary to construct on liquefaction susceptible soil because of space restrictions, favorable location, or other reasons, it may be possible to make the structure liquefaction resistant by designing the foundation elements to resist the effects of liquefaction. A structure that possesses ductility, has the ability to accommodate large deformations, adjustable supports for correction of differential settlements, and having foundation design that can span soft spots can decrease the amount of damage a structure may suffer in case of liquefaction (Committee on Earthquake Engineering, NRC, 1985). To achieve these features in a building there are various aspects to consider.

a. Shallow Foundation Aspects

Its is important that all foundation elements in a shallow foundation has been tied together to make the foundation move or settle uniformly, thus decreasing the amount of shear forces induced in the structural elements resting upon the foundation. The well-reinforced perimeter and interior wall footings are tied together to enable them to bridge over areas of local settlement and provide better resistance against soil movements.

b. Buried Utilities Aspects

Buried utilities, such as sewage and water pipes, should have ductile connections to the structure to accommodate the large movements and settlements that can occur due to liquefaction.

c. Deep Foundation Aspects

Liquefaction can cause large lateral loads on pile foundations. Piles driven through a weak, potentially liquefiable, soil layer to a stronger layer not only have to carry vertical loads from the superstructure, but must also be able to resist horizontal loads and bending moments induced by lateral movements if the weak layer liquefies. Sufficient resistance can be achieved by piles of larger dimensions and/or more reinforcement.

It is important that the piles are connected to the cap in a ductile manner that allows some rotation to occur without a failure of the connection. If the pile connections fail, the cap cannot resist overturning moments from the superstructure by developing vertical loads in the piles .

3. Ground Improvement

The third option involves mitigation of the liquefaction hazards by improving the strength, density, and/or drainage characteristics of the soil. This can be done using a variety of soil improvement techniques.

The main goal of most soil improvement techniques used for reducing liquefaction hazards is to avoid large increases in pore water pressure during earthquake shaking. This can be achieved by densification of the soil and/or improvement of is drainage capacity.

a. Vibroflotation

Vibroflotation involves the use of a vibrating probe that can penetrate granular soil to depths of over 100 feet. The vibrations of the probe cause the grain structure to collapse thereby densifying the soil surrounding the probe. To treat an area of potentially liquefiable soil, the vibroflot is raised and lowered in a grid pattern. Vibro Replacement (Fig 2.12) is a combination of vibroflotation with a gravel backfill faulting in stone columns, which not only increases the amount of densification, but provides a degree of reinforcement and a potentially effective means of drainage.

Fig 2.12: Photograph showing Vibroflotation technique

b. Dynamic Compaction

Densification by dynamic compaction has been performed by dropping a heavy weight of steel or concrete in a grid pattern from heights of 30 to 100 ft. It provides an economical way of improving soil for mitigation of liquefaction hazards. Local liquefaction can be initiated beneath the drop point making it easier for the sand grains to density. When the excess pore water pressure from the dynamic loading dissipates, additional densification occurs. As illustrated in the Fig 2.13 however, the process is somewhat invasive; the surface of the soil may require shallow compaction with possible addition of granular fill following dynamic compaction.

Fig 2.13: Photograph showing dynamic compaction technique

c. Stone Columns

As described above, stone columns are columns of gravel constructed in the ground. Stone columns can be constructed by the vibroflotation method. They can also be installed in other ways, for example, with help of a steel casing and a drop hammer as in the Franki Method. In this approach the steel casing has been driven in to the soil and gravel filled in from the top and tamped with a drop hammer as the steel casing successively withdrawn.

d. Compaction Piles

Installing compaction piles is a very effective way of improving soil. Compaction piles are usually made of prestressed concrete or timber. Installation of compaction piles both densifies and reinforces the soil. The piles are generally installed in a grid pattern and are generally driven to depth of up to 60 ft.

e. Compaction Grouting

Compaction grouting is a technique whereby a slow-flowing water/sand/cement mix is injected under pressure into a granular soil. The grout forms a bulb that displaces and hence densifies, the surrounding soil (Fig 2.14). Compaction grouting is a good option if the foundation of an existing building requires improvement, since it is possible to inject the grout from the side or at an inclined angel to reach beneath the building.

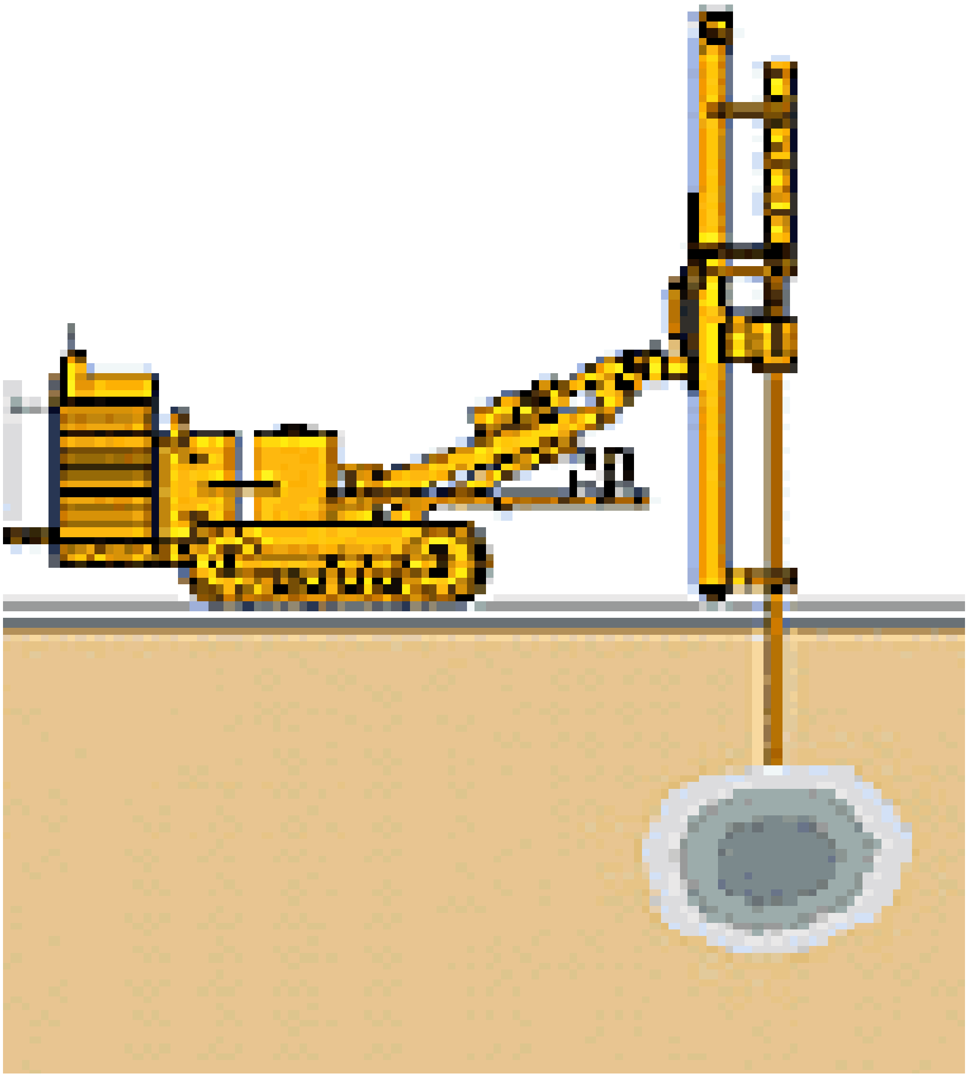


Fig 2.14: Photograph showing Compaction Grouting technique

f. Drainage technique

Liquefaction hazards can be reduced by increasing the drainage ability of the soil. If the pore water within the soil can drain freely, the build-up of excess pore water pressure will be reduced. Drainage techniques include installation of drains of gravel, sand or synthetic material. Synthetic wick drains can be installed at various angles, in contrast to gravel or sand drains that have been usually installed vertically. Drainage techniques have been often used in combination with other types of soil improvement techniques for more effective liquefaction hazard reduction.

g. Verification of Improvement

A number of methods can be used to verify the effectiveness of soil improvement. In-situ techniques are popular because of the limitations of many laboratory techniques. Usually, in-situ test have been performed to evaluate the liquefaction potential of a soil deposit before the improvement was attempted. With the knowledge of the existing ground characteristics, one can then specify a necessary level of improvement in terms of in situ test parameters. Performing in-situ tests after improvement has been completed allows one to decide if the degree of improvement was satisfactory. In some cases, the extent of the improvement has been not reflected in in-situ test results until some time after the improvement has been completed.

2.15 Concluding Remarks

In this chapter past researches related to liquefaction have been discussed. Different methods estimation of Liquefaction based on CPT and SPT have also been discussed. The different mitigatigation procedure have also been explained in this chapter.

CHAPTER THREE: EQUIPMENT USES & DATA COLLECTION

3.1 General

The objective of this chapter is to describe the different parts of the cone penetration test mechine. It also describes the variation of the different parametes like friction, cone resistance, friction ratio and SPT value (N value) with depth. Here the locations of the tests for the research have been described.

3.2 General Specifications of CPT

Pushes down the CPT cone at a nominal rate of 2 cm per second. Pull up rate is 5 cm per second. Handled by two men, the complete equipment can be transported on a pick-up van. After that the soil anchors have been installed, the machine has been positioned on the test site, the wheels have been removed and the reaction beams installed and secured. The anchors can give between 8 and 16 tons reaction force. The Machine can also be used as a separate stand alone unit with the wheels arrangement removed. Figure 3.1 shows a typical CPT machine.

Fig 3.1 : CPT Machine (Ref: Envi 200 kN CPT Pusher / Puller Operating Manual)

In the following articles different preparatory steps of CPT have been described. Figure 3.2 shows these steps.

3.2.1 Soil Anchors

Start to install the 4 SOIL ANCHORS in a square configuration with the size 1.3 x 1.6 m. Install the MANUAL CROSS HEAD or the MOTOR DRIVE on top of the SOIL ANCHOR using two 12 mm BOLTS. Use 2 or 3 CPT RODS to turn the SOIL ANCHOR into the ground. Screw down the auger until the top of the rod is 0.6 m above the ground. If the soil is hard, it can be enough with the soil anchor rod, but if the upper soil is soft, use the extension rods. Use wood pieces to support the pusher and erect it horizontally. The SUPPORTS shall be high enough so that the wheels are free from the ground. Remove the bolts that are locking the wheel and pull out the wheels. Insert the two SHORT BEAMS inside the machine like the picture shows.

3.2.2 Hydraulic pump

Before connecting the **HYDRAULIC HOSES**, clean the quick coupling with a rag. Before starting the engine, the **PUMP VALVE** shall be in **OPEN** position. Check the **OIL LEVEL** in the engine.

3.2.3 Start Engine

Turn the FUEL VALVE to OPEN. To apply CHOKE on a cold engine, turn the choke lever to the left. Put the INGITION SWITCH to ON. Start the engine by pulling the starter line.

3.2.4 Operation

Firstly Close the PUMP VALVE . Then run the CYLINDERS UP. With no load or light load on the machine, Both VALVES can be used to increase the speed going upwards. This does not function going down. Pull the right lever for push dawn. The speed has been regulated by changing the engine r.p.m. For CPT, the standard RATE OF PENETRATION shall be 1.2 meter/minute + / - 25 %.

3.2.5 Prepare for CPT

Before the penetration can be started, the Memocone must have been prepared. This incudes filling the filter point and connecting to the Datalogger for start up and zero readings. Ref. to PC-mon manual. When the MEMOCONE has been prepared and started up together with the datalogger, put it inside the machine. Adjust the DEPTH SENSOR WHEEL. Turn the LEVER to the right. Adjust if necessary on the screw so that the wheel is turning when the memocone is moving up and down. Do not press the wheel too hard against the memocone, only so much that it turns the wheel safely. Depth sensor. Connect the depth sensor cable with the datalogger. Insert the PUSHING HEAD or MICROPHONE into the ANVIL It will stay in position by the means of magnets. Move the head down and guide the Memocone into the center. When the head makes contactact with the Memocone, press the + button on the datalogger and start the penetration. When the resistance is getting higher, check that the automatic locks are gripping OK. PUSH about 0.5 meter.

3.2.6 PRESSURE READING

It is possible to know the pushing force by checking the hydraulic pressure.

50 Bar = 6 ton

100 Bar = 12 ton

150 Bar = 18 ton

200 Bar = 24 ton

3.2.7 Maintenance:

Every 1 year should be executed the following scheme:

1. Change engine oil.
2. Lubricate the depth sensor wheel with oil.

Every 2 year:

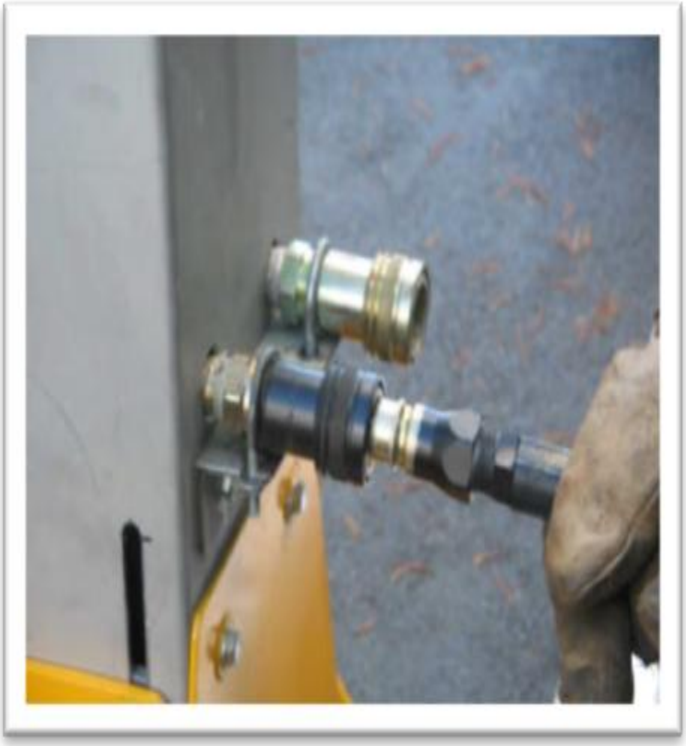
1. Change the hydraulic oil.

Use wood pieces to support the pusher and erect it horisontally.

Insert the two SHORT BEAMS inside the machine

Place the two LONG BEAMS on top of the short ones.

AUTOMATIC LOCKS on the auger rods.



Hydraulic pump

Close the PUMP VALVE

DEPTH SENSOR WHEEL

LEVER to the right

Insert the PUSHING HEAD

Start the pull up.

Fig 3.2: Cone Penetration Test Procedure.

3.2.8 PC-Mon.

PC-mon stands for PC Interface monitor. This unit is the link between the CPTU probes Memocone type II and type III and a portable PC. The portable PC is NOT INCLUDED in the delivery. The software PC-mon v 1.0 or later has to be installed in the portable PC. The handling of the PC-Mon is totally menu operated. All possibilities at upstart, operation, registration of data and collection of data is clearly described on the screen. All input has been done by the keyboard and arrow-up and arrow-down buttons.

Technical specification:
Size: 420 x 300 x 55 mm
Weight: 5 Kg
Cabinet: Machined Aluminium
Power requirement: 12 V DC (Car battery)
Consumption: 1 A , FUSE : 6 A
Inputs: 12 Volts power, CPT Probe, Depth transducer encoder, Microphone,
Pressure sensor (Bosch type)
Outputs: 12 Volts for PC, USB port

3.3 Selected areas for the Research

Total ten areas of the Dhaka city have been selected for this research. The main targeted areas have been reclaimed lands since some of these lands found susceptible to liquefaction (Ahmed, 2005). Total ten areas have been selected which almost surrounding the Dhaka city. The reclaimed areas are **Bramangaon**, **Ashian City**, **Badda**, **Banasree**, **Gabtoli**, **Kawran Bazar**, **Purbachal**, **United City**, **Uttara** and **Kamrangirchar**. Figure 3.3 shows the selected study area.

Fig 3.3: Map Showing the selected areas of Dhaka City for the Research

3.4 Standard Penetration Test (SPT)

It has been conducted at all ten areas. SPT has been used for the determination of liquefaction potential. To utilize SPT results in measuring liquefaction potential, SPT have been conducted according to ASTM D1586 (ASTM,2000). The main objective of SPT are as follows:

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

3.5 Cone Penetration Test (CPT)

It has been conducted at all ten areas. CPT is being used for the determination of liquefaction potential. Tip resistance (q), sleeve friction (f) have been found from this test.

3.6 Laboratory Tests

Disturb and undisturb samples have been collected during SPT. Collected samples have been tested at geotechnical laboratory of BUET. Grain size analysis have been performed according to the procedure specified by ASTM D 422. These test results have used in the corelation between CPT and SPT.

3.7 Sub-Soil characteristics

Total ten locations have been selected for investigating liquefaction potential in Dhaka city in this research. SPT has been conducted and disturbed as well as undisturbed samples collected from all these locations. Sub-soil characteristics ,SPT and laboratory test results of these samples have been presented in this section. Besides this CPT test results have also been described.

3.7.1 Sub- Soil characteristics of BRAMANGAON

This site has been situated in southest part of Dhaka city. It is a private land development project where main filing has been done by dredged river sand. The depth of filling of fine sand is 5.0 m from existing ground level. The clayey silt layer exists from 5.0 m to 6.5 m from EGL. After that 1.5 m is sandy silt layer. Then 12.0 m is clayey silt. The uncorrected SPT N value of filling fine sand varies from 4 to 5. The SPT N value of clayey silt layers varies from 5 to 8. The maximum value of SPT N is 8. The minimum value of SPT N is 4. From the CPT test, the cone resistance varies from 0.51 to 16.78 MPa. The average value of cone resistance is 3.88 MPa. Friction varies from 0 to 120.2 kPa. The average value of Friction is 30.79 kPa. Friction ratio varies from 0 to 4.13 kPa. The average value of Friction ratio is 1.11 kPa. Figure 3.4 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at BRAMANGAON.

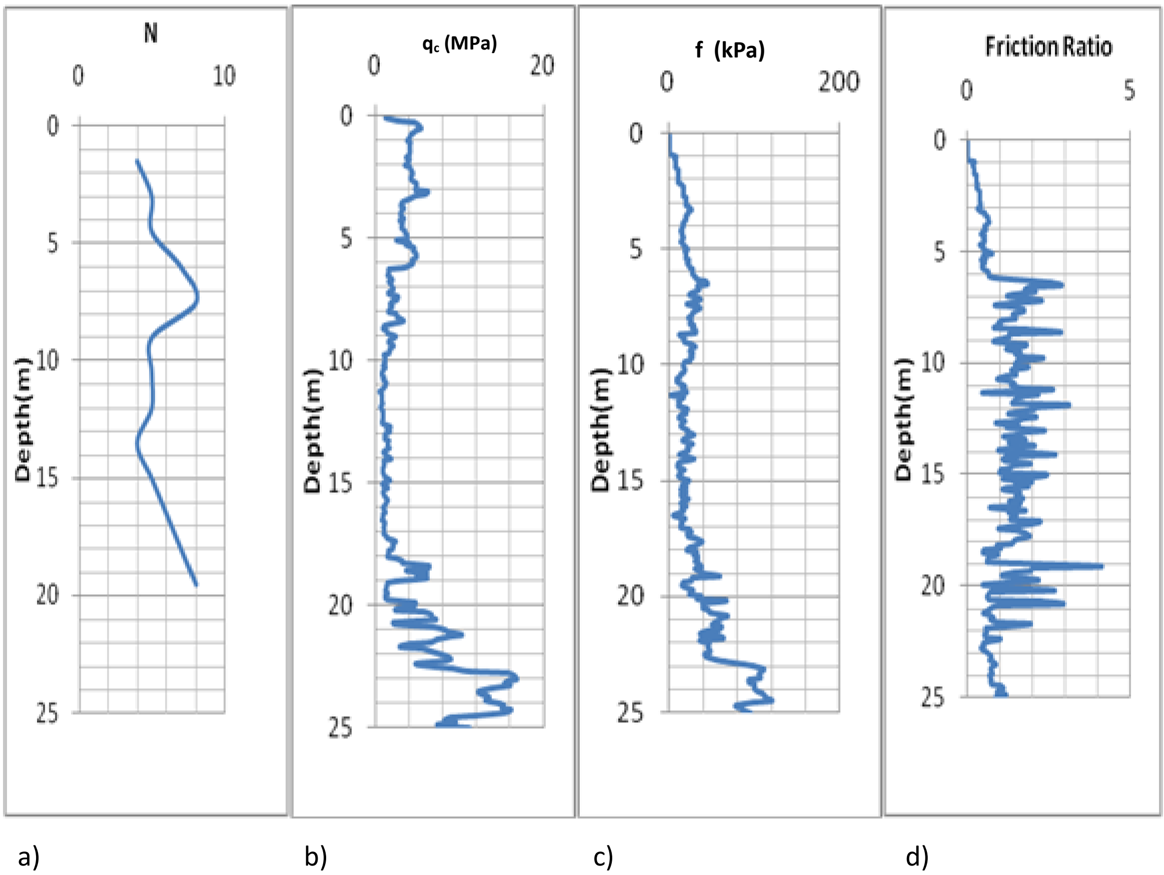


Fig 3.4: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at BRAMANGAON

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.1(a). The graph between depth vs N value has been shown in Fig 3.4 (a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.4(c) and 3.4(b). From the CPT test, the cone resistance varies from 0.51 to 16.78 MPa. The maximum value of cone resistance is 16.78 MPa. The minimum value of cone resistance is 0.51 MPa. The average value of cone resistance is 3.88 MPa. Friction varies from 0 to 120.2 kPa. The maximum value of Friction is 120.2 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 30.79 kPa. Friction ratio varies from 0 to 4.13 kPa. The maximum value of Friction ratio is 4.13 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.11 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.1(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm, 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of silt are 0.046 mm, 71% respectively. Table 3.1 b shows Probable soil classification using CPT data (Robertson, 1990) at BRAMANGAON.

Table 3.1(a) : Grain size analysis at BRAMANGAON

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	4	20	0.16
3	Filling Sand	5	21	0.17
4.5	Filling Sand	5	20	0.17
6	Clayey Silt	7	71	0.046
7.5	Sandy silt	8	51	0.076
9	Clayey Silt	5	71	0.046
10.5	Clayey Silt	5	71	0.046
12	Clayey Silt	5	72	0.046
13.5	Clayey Silt	4	71	0.046
15	Clayey Silt	5	71	0.046
16.5	Clayey Silt	6	71	0.046
18	Clayey Silt	7	71	0.046
19.5	Clayey Silt	8	71	0.046

Table 3.1(b): Probable soil classification using CPT data (Robertson, 1990) at BRAMANGAON

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	0.82-1.52	Sand
3-6	1.22-1.92	Sand
6-9	1.77-2.82	Sand/Silt
9-12	2.34-3.24	Silty Sand/Siltly Clay
12-15	2.48-3.1	Silty Sand/Siltly Clay
15-18	2.51-3.11	Silty Sand/Siltly Clay
18-21	1.99-3.15	Silty Sand/Siltly Clay
21-24	1.66-2.67	Sand

3.7.2 Sub-Soil Characteristics of ASHIANCITY

This site has been situated in Northern part of Dhaka city. The depth of filling of fine sand is 3.5 m from existing ground level. The silty clay layer exists from 3.5 m to 12.5 m from EGL. After that 4.5 m is fine sand layer. Then 3.0 m is silty clay. The uncorrected SPT N value of filling fine sand varies from 4 to 5. The SPT N value of silty clay layers varies from 3 to 7. The maximum value of SPT N is 42. The minimun value of SPT N is 3. From the CPT test, the cone resistance varies from 0.195 to 7.805 MPa. The average value of cone resistance is 1.78 MPa. Friction varies from 0 to 233.9 kPa. The average value of Friction is 31.21 kPa. Friction ratio varies from 0 to 6.35 kPa. The average value of Friction ratio is 1.29 kPa. Figure 3.5 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at ASHIANCITY.

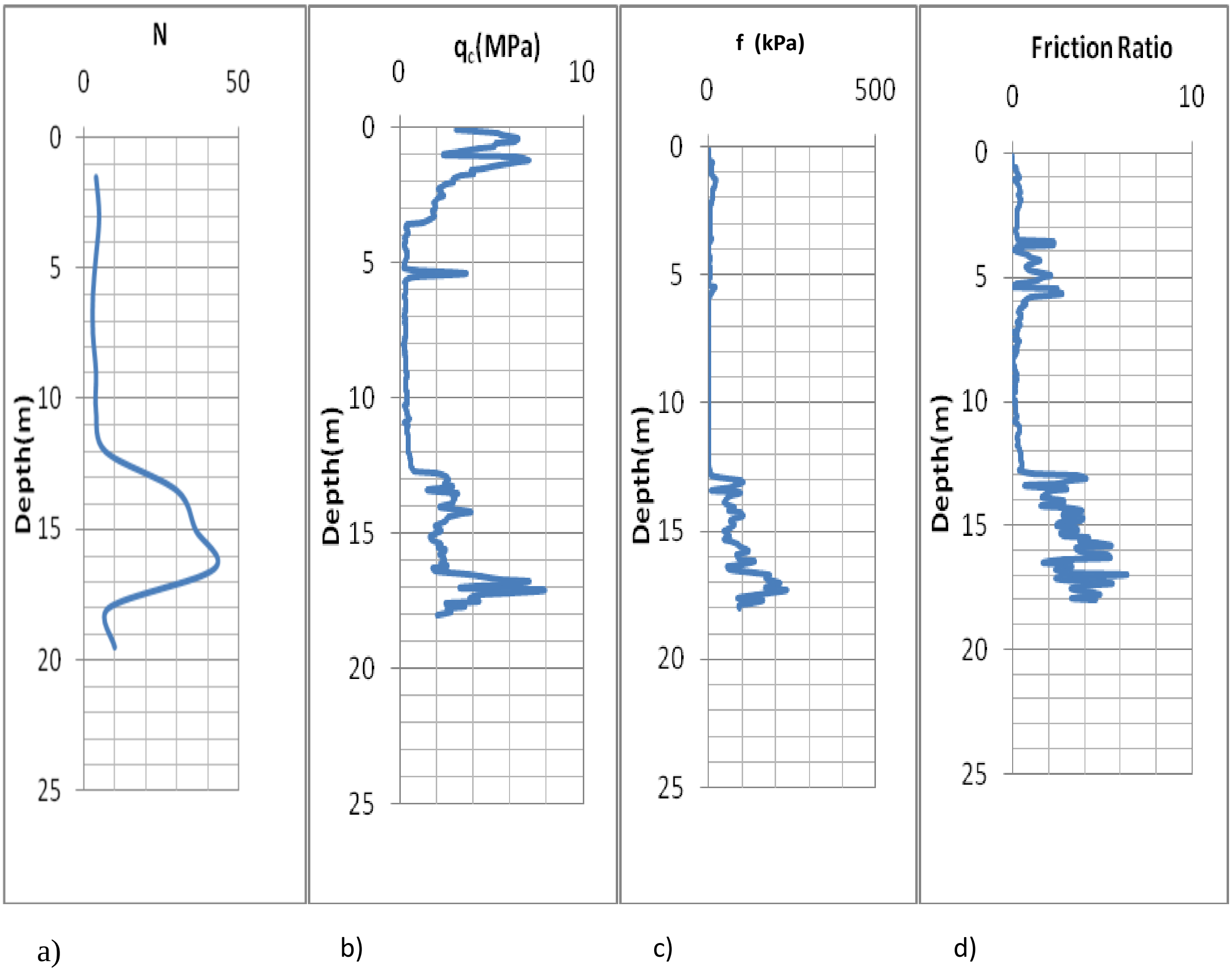


Fig 3.5: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at ASHIANCITY.

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.2(a). The graph between depth vs N value has been shown in Fig 3.5 (a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.5(c) and 3.5(b). From the CPT test, the cone resistance varies from 0.195 to 7.805 MPa. The maximum value of cone resistance is 7.805 MPa. The minimum value of cone resistance is 0.195 MPa. The average value of cone resistance is 1.78 MPa. Friction varies from 0 to 233.9 kPa. The maximum value of Friction is 233.9 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 31.21 kPa. Friction ratio varies from 0 to 6.35 kPa. The maximum value of Friction ratio is 6.35 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.29 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.2(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm, 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.002 to 0.003 mm, 94 to 96% respectively. The mean grain size (d_{50}), fine content (F_c) of sand are 0.16 mm, 21% respectively. Table 3.2(b) shows the probable soil classification using CPT data (Robertson, 1990) at ASHIANCITY.

Table 3.2(a) : Grain size data at ASHIANCITY

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	4	20	0.16
3	Filling Sand	5	21	0.17
4.5	Silty Clay	4	94	0.002
6	Silty Clay	3	95	0.0025
7.5	Silty Clay	3	96	0.003
9	Silty Clay	4	96	0.003
10.5	Silty Clay	4	96	0.003
12	Silty Clay	7	96	0.003
13.5	Sand	30	21	0.16
15	Sand	36	21	0.16
16.5	Sand	42	21	0.16
18	Silty Clay	8	95	0.0025
19.5	Silty Clay	10	95	0.0025

Table 3.2 (b): Probable soil classification using CPT data (Robertson, 1990) at ASHIANCITY

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	0.64-1.61	Sand
3-6	1.44-3.25	Sand/Silty clay
6-9	2.84-3.51	Silty Clay
9-12	2.81-3.45	Sandy silt/Silty Clay
12-15	2.28-3.05	Silty sand/Silty Clay
15-18	2.29-3.04	Silty sand/Silty Clay

3.7.3 Sub-Soil Characteristics of BADDA

This site has been situated in Eastern part of Dhaka city. The depth of filling of fine sand is 5.0 m from existing ground level. The organic clay layer exists from 5.0 m to 8.0 m from EGL. After that 6.5 m is silty clay layer. Then 4.0 m is sandy silt. Then 1.5 m is fine sand .The uncorrected SPT N value of filling fine sand varies from 5 to 8. The SPT N value of silty clay layers varies from 5 to 12. The maximum value of SPT N is 38. The minimun value of SPT N is 1. From the CPT test, the cone resistance varies from 0.17 to 4.465 MPa. The average value of cone resistance is 1.39 MPa. Friction varies from 0 to 145 kPa. The average value of Friction is 34.1 kPa. Friction ratio varies from 0 to 6.24 kPa. The average value of Friction ratio is 2.0 kPa. Figure 3.6 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth(m) vs Friction (kPa) d) Depth(m) vs Friction Ratio at BADDA.

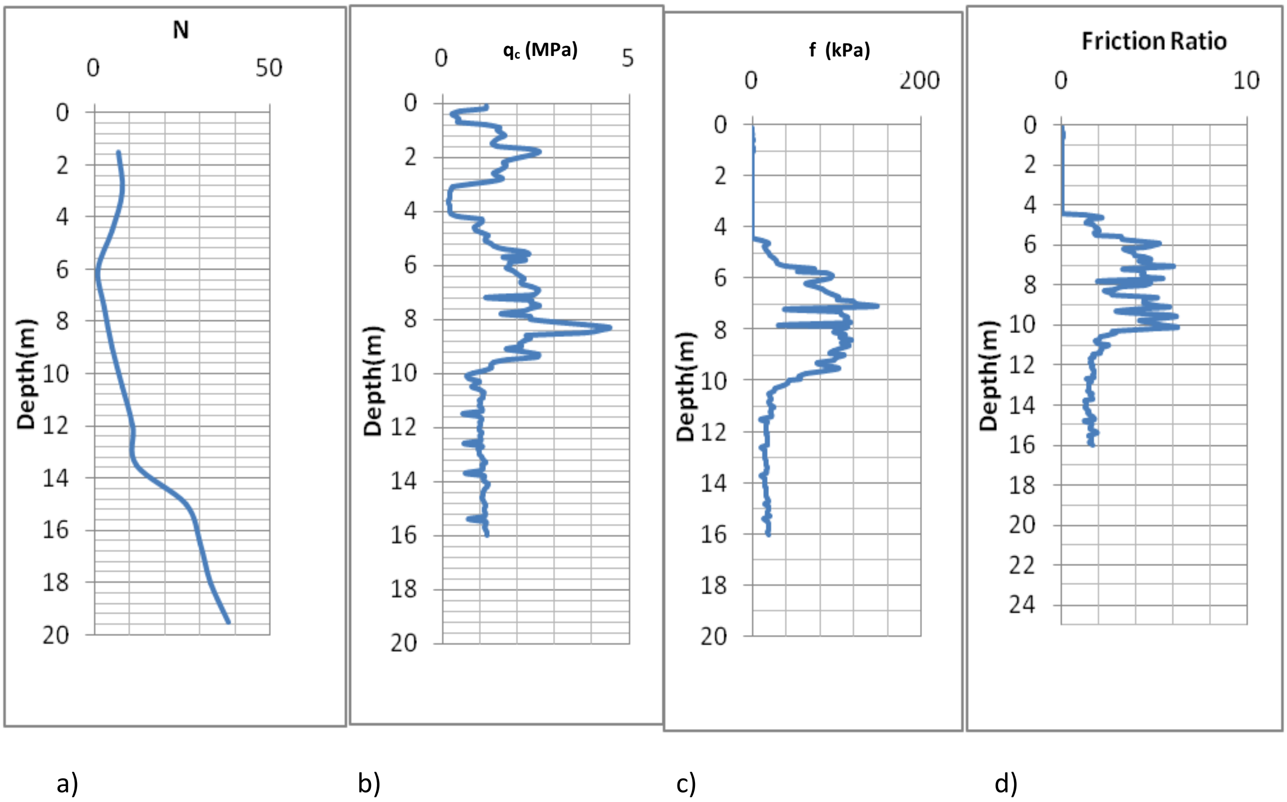


Fig 3.6: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth(m) vs Friction (kPa) d) Depth(m) vs Friction Ratio at BADDA

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.3(a). The graph between depth vs N value has been shown in Fig 3.6 (a).

CPT results:

The graph between Depth vs friction and cone resistance have been shown in Fig 3.6(c) and 3.6(b). From the CPT test, the cone resistance varies from 0.195 to 7.805 MPa. The maximum value of cone resistance is 7.805 MPa. The minimum value of cone resistance is 0.195 MPa. The average value of cone resistance is 1.78 MPa. Friction varies from 0 to 233.9 kPa. The maximum value of Friction is 233.9 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 31.21 kPa. Friction ratio varies from 0 to 6.35 kPa. The maximum value of Friction ratio is 6.35 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.29 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.3(a). The mean grain size (d_{50}), fine content (Fc) of filling sand varies 0.17 to 0.19 mm, 20 to 23% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.005 mm , 91 to 92% respectively. The mean grain size (d_{50}), fine content (F_c) of sand 0.165 mm , 30% respectively. The mean grain size (d_{50}), fine content (F_c) of organic clay varies 0.009 mm, 87 to 98% respectively. The mean grain size (d_{50}), fine content (F_c) of silt varies 0.025 to 0.04 mm, 70 to 78% respectively. Table 3.3(b) shows the probable soil classification using CPT data (Robertson, 1990) at BADDA.

Table 3.3 (a): Grain size data at BADDA

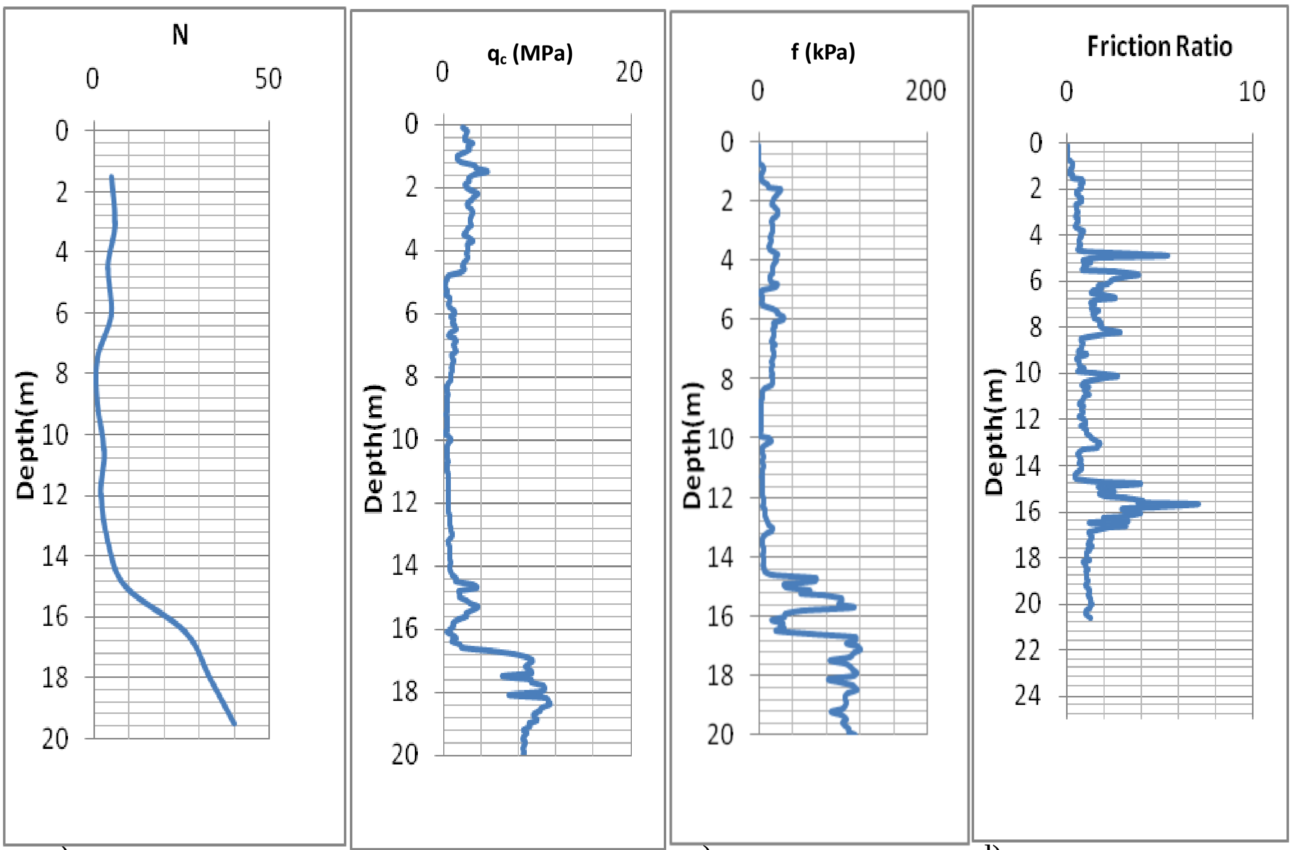
Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	7	23	0.17
3	Filling Sand	8	20	0.19
4.5	Filling Sand	5	20	0.19
6	Organic Clay	1	87	0.009
7.5	Organic Clay	3	98	0.009
9	Silty Clay	5	91	0.0055
10.5	Silty Clay	8	92	0.005
12	Silty Clay	11	92	0.005
13.5	Silty Clay	12	96	0.005
15	Sandy Silt	26	70	0.046
16.5	Sandy Silt	30	72	0.04
18	Sandy Silt	33	78	0.025
19.5	Sand	38	30	0.165

Table 3.3 (b): Probable soil classification using CPT data (Robertson, 1990) at BADDA

Depth Range(m)	I_C Range	Probable Soil Classification
0-3	1.06-1.50	Sand
3-6	2.25-2.66	Silty Sand/Silty Clay
6-9	2.22-2.75	Silty Sand/ silty Clay
9-12	2.52-3.33	Silty Sand/ silty Clay
12-15	2.78-3.24	Silty Clay
15-16	2.89-3.25	Silty Clay

3.7.4 Sub–Soil Characteristics of BANASREE

This site has been situated in eastern part of Dhaka city. It is a private land development project where main filing has been done by dredged river sand. The depth of filling of fine sand is 4.0 m from existing ground level. The clay layer exists from 4.0 m to 14.0m from EGL. After that 2.0 m is clay layer. Then 4.0 m is silty fine sand. The uncorrected SPT N value of filling fine sand varies from 5 to 6. The SPT N value of clay layers varies from 1 to 9. The maximum value of SPT N is 40. The minimum value of SPT N is 1. From the CPT test, the cone resistance varies from 0.175 to 11.37 MPa. The average value of cone resistance is 2.91 MPa. Friction varies from 0 to 130.3 kPa. The average value of Friction is 32.96 kPa. Friction ratio varies from 0 to 7.04 kPa. The average value of Friction ratio is 1.26 kPa. Figure 3.7 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction ratio at BANASREE.



a) b) c) d)

Fig 3.7: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction ratio at BANASREE.

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.4(a). The graph between depth vs N value have been shown in Fig 3.7 (a).

CPT results:

The graph between Depth vs friction and Cone resistance have been shown in Fig 3.7 (c) and 3.7 (b). From the CPT test, the cone resistance varies from 0.175 to 11.37 MPa. The maximum value of cone resistance is 11.37 MPa. The minimum value of cone resistance is 0.175 MPa. The average value of cone resistance is 2.91 MPa. Friction varies from 0 to 130.3 kPa. The maximum value of Friction is 130.3 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 32.96 kPa. Friction ratio varies from 0 to 7.04 kPa. The maximum value of Friction ratio is 7.04 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.26 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.4(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.17 mm, 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.003 to 0.004 mm, 90 to 96% respectively.The mean grain size (d_{50}), fine content (F_c) of silt varies 0.046 mm, 70 to 71% respectively. Table 3.4(b) shows the probable soil classification using CPT data (Robertson,1990) at BANASREE.

Table 3.4 (a): Grain size analysis at BANASREE

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	5	20	0.17
3	Filling Sand	6	21	0.17
4.5	Silty Clay	4	94	0.003
6	Silty Clay	5	95	0.003
7.5	Silty Clay	1	96	0.003
9	Silty Clay	1	90	0.004
10.5	Silty Clay	3	90	0.004
12	Silty Clay	2	90	0.004
13.5	Silty Clay	4	90	0.004
15	Silty Clay	9	90	0.004
16.5	Silt	26	71	0.046
18	Silt	33	71	0.046
19.5	Silt	40	70	0.046

Table 4.4 (b): Probable soil classification using CPT data (Robertson, 1990) at BANASREE

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	0.80-1.67	Sand
3-6	1.58-3.27	Sand/Silty Clay
6-9	2.41-3.33	Silty Clay
9-12	2.87-4.54	Silty clay
12-15	2.19-3.21	Silty Clay
15-18	1.94-4.03	Silty Clay
18-21	1.89-2.14	Sand

3.7.5 Sub-Soil Characteristics of GABTOLI

This site has been situated in north-west part of Dhaka city. It is a private land development project where main filing is done by dredged river sand. The depth of filling of fine sand is 5.0 m from existing ground level. The clay layer exists from 5.0 m to 11.0m from EGL. After that 7.5 m is sandy silt layer. Then 1.5 m is silty fine sand. The uncorrected SPT N value of filling fine sand varies from 3 to 6. The SPT N value of clay layers varies from 4 to 7. The maximum value of SPT N is 15. The minimun value of SPT N is 3. From the CPT test, the cone resistance varies from 0.52 to 7.82 MPa The average value of cone resistance is 2.93 MPa. Friction varies from 0 to 70.1 kPa. The average value of Friction is 24.75 kPa. Friction ratio varies from 0 to 9.34 kPa. The average value of Friction ratio is 1.32 kPa. Figure 3.8 shows the a) Depth(m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at GABTOLI.

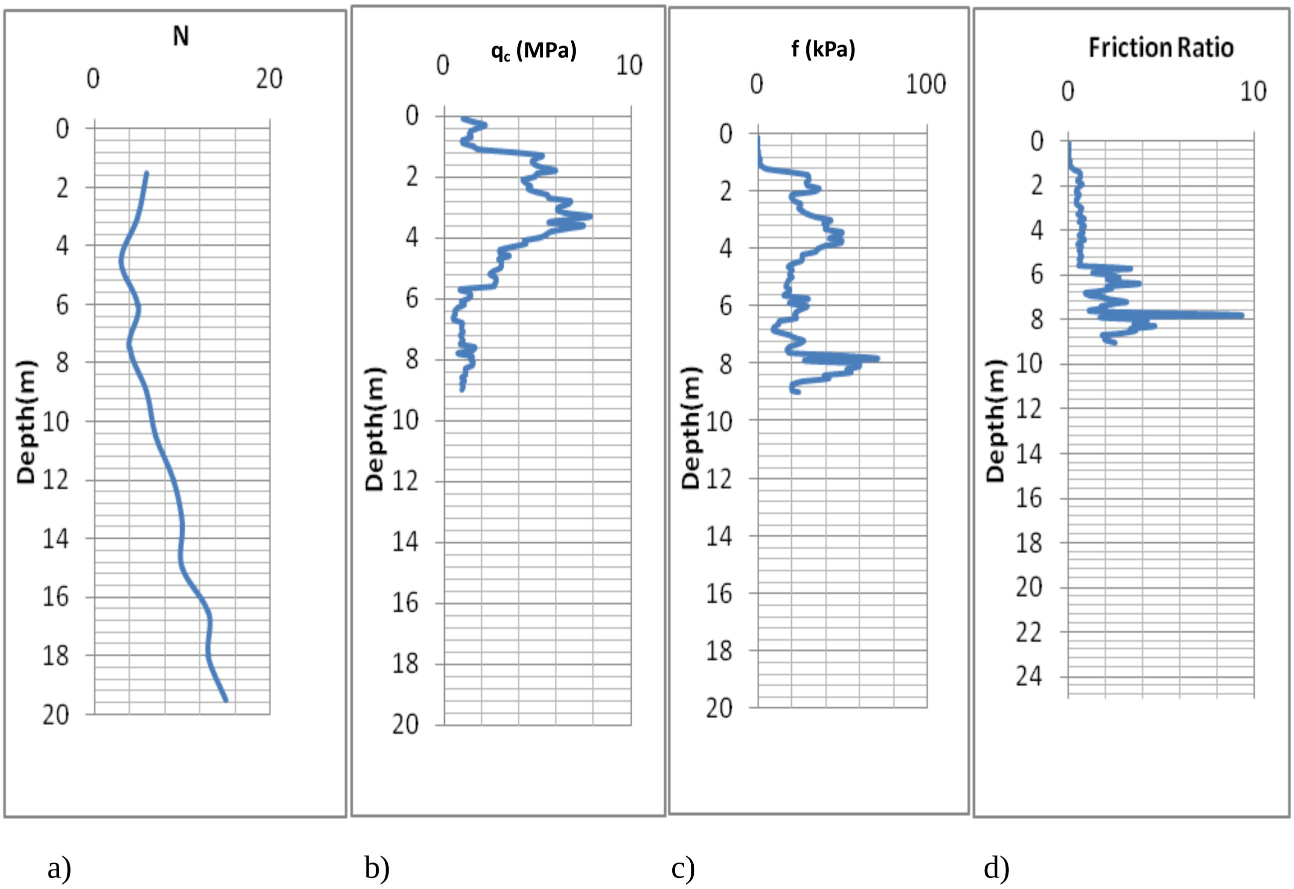


Fig 3.8: a) Depth(m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at GABTOLI.

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.5(a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.8(c) and 3.8(b). From the CPT test, the cone resistance varies from 0.52 to 7.82 MPa. The maximum value of cone resistance is 7.82 MPa. The minimum value of cone resistance is 0.52 MPa. The average value of cone resistance is 2.93 MPa. Friction varies from 0 to 70.1 kPa. The maximum value of Friction is 70.1 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 24.75 kPa. Friction ratio varies from 0 to 9.34 kPa. The maximum value of Friction ratio is 9.34 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.32 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.5(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm, 18.7 to 20% respectively. The mean grain size (d_{50}), fine content (Fc) of clay varies 0.003 mm 90 % respectively. The mean grain size (d_{50}), fine content (F_c) of silt 0.046 mm, 70 to 71% respectively. Figure 3.5(b) shows the probable soil classification using CPT data (Robertson, 1990) at GABTOLI.

Table 3.5(a): Grain size Data at GABTOLI

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	6	18.7	0.17
3	Filling Sand	5	20	0.17
4.5	Filling Sand	3	20	0.16
6	Silty Clay	5	90	0.003
7.5	Silty Clay	4	90	0.003
9	Clayey Silt	6	70	0.046
10.5	Clayey Silt	7	71	0.046
12	Sandy Silt	9	61	0.076
13.5	Sandy Silt	10	61	0.076
15	Sandy Silt	10	61	0.076
16.5	Sandy Silt	13	61	0.076
18	Sandy Silt	13	61	0.076
19.5	Sand	15	33	0.14

Table 3.5(b): Probable soil classification using CPT data (Robertson, 1990) at GABTOLI

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	0.96-1.51	Sand
3-6	1.37-2.87	Sand/Silty Clay
6-9	2.46-3.37	Clay/Silty Clay

3.7.6 Sub-Soil Characteristics of KAWRAN BAZAR

This site has been situated in north-east part of Dhaka city. The depth of filling of fine sand is 3.5 m from existing ground level. The clay layer exists from 3.5 m to 11.0 m from EGL. After that 9.0 m is silt layer. The SPT N value of clay layers varies from 2 to 15. The maximum value of SPT N is 36. The minimun value of SPT N is 1. From the CPT test, the cone resistance varies from 0 to 18.575 MPa. The average value of cone resistance is 3.77 MPa. Friction varies from 0 to 226.7 kPa. The average value of Friction is 47.71 kPa. Friction ratio varies from 0 to 7.72 kPa. The average value of Friction ratio is 1.72 kPa. Figure 3.9 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth(m) vs Friction Ratio at KAWRAN BAZAR.

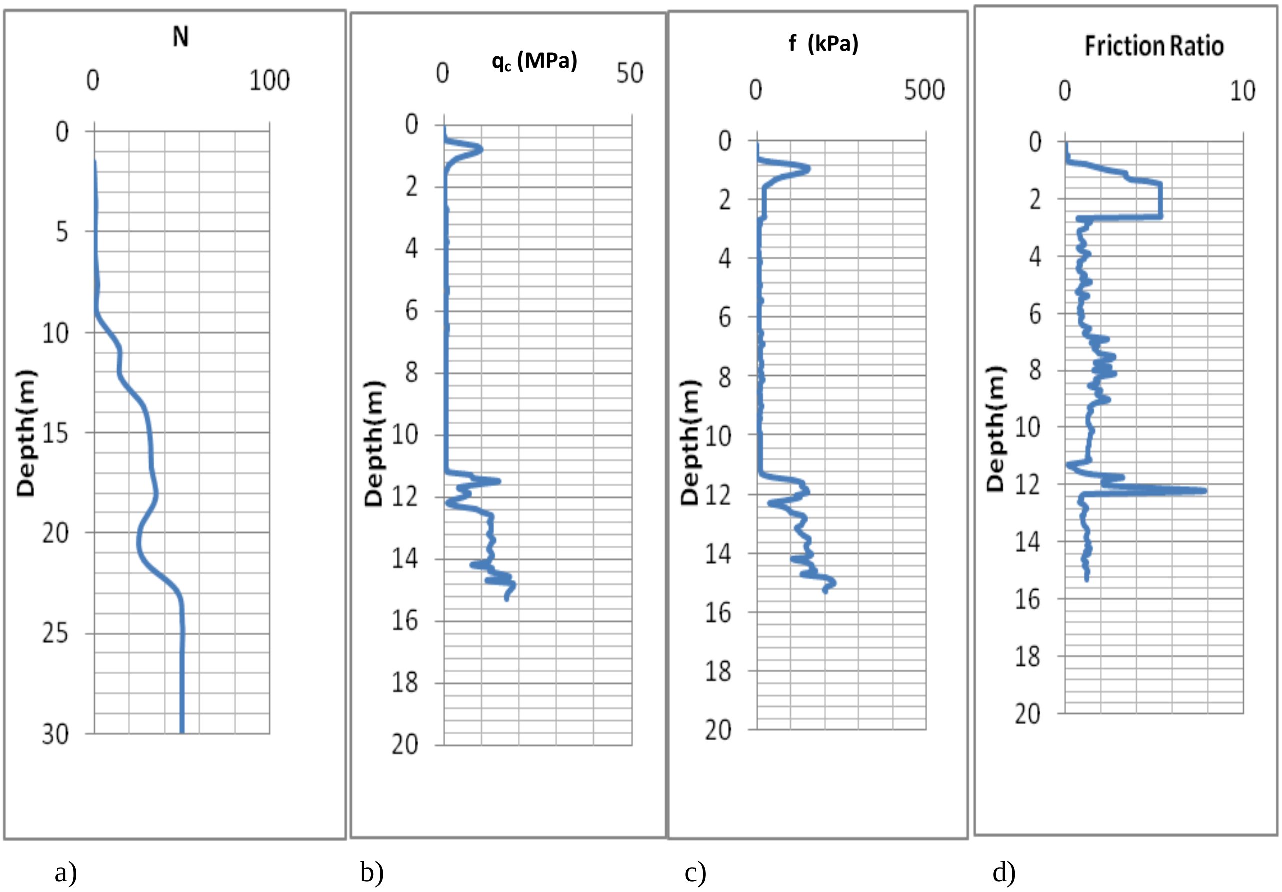


Fig 3.9: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth(m) vs Friction Ratio at KAWRAN BAZAR.

SPT Results :

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.6(a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.9(c) and 3.9(b). From the CPT test, the cone resistance varies from 0 to 18.575 MPa. The maximum value of cone resistance is 18.575 MPa. The minimum value of cone resistance is 0 MPa. The average value of cone resistance is 3.77 MPa. Friction varies from 0 to 226.7 kPa. The maximum value of Friction is 226.7 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 47.71 kPa. Friction ratio varies from 0 to 7.72 kPa. The maximum value of Friction ratio is 7.72 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.72 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.6(a). The mean grain size (d_{50}), fine content (F_c) of fine sand varies 0.14 to 0.16 mm, 30 to 33% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.002 to 0.003 mm, 94 to 96% respectively. The mean grain size (d_{50}), fine content(F_c) of silt 0.046 mm, 71% respectively. The mean grain size (d_{50}), fine content (F_c) of organic clay varies 0.001 mm, 90% respectively. Figure 3.6(b) shows the Probable soil classification using CPT data (Robertson, 1990) at KAWRAN BAZAR.

Table 3.6(a): Grain size Data at KAWRAN BAZAR

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	clay	1	90	0.003
3	organic clay	1	90	0.001
4.5	Clay	1	94	0.002
6	Clay	2	95	0.002
7.5	Clay	2	96	0.002
9	Silt	14	71	0.002
10.5	Silt	15	71	0.16
12	Fine Sand	28	30	0.16
13.5	Fine Sand	32	30	0.16
15	Fine Sand	33	33	0.16
16.5	Fine Sand	35	33	0.16
18	Fine Sand	26	30	0.16
19.5	Fine Sand	28	30	0.14

Table 3.6(b): Probable soil classification using CPT data (Robertson 1990) at KAWRAN BAZAR

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	0.80-2.88	Clay/Silty sand
3-6	2.27-2.68	Silty Clay
6-9	2.53-3.15	Clay
9-12	1.55-3.05	Clay/ Silty Sand
12-15	1.69-3.23	Silty Sand/Silty Clay

3.7.7 Sub-Soil Characteristics of PURBACHAL

This site has been situated in north-west part of Dhaka city. It is a Government land development project where main filing is done by dredged river sand. The depth of filling of fine sand is 6.5 m from existing ground level. The clay layer exists from 6.5 m to 13.5 m from EGL. The uncorrected SPT N value of filling fine sand varies from 1 to 3. The SPT N value of clay layers varies from 1 to 8. The maximum value of SPT N is 8. The minimun value of SPT N is 1. From the CPT test, the cone resistance varies from 0.355 to 7.06 MPa. The average value of cone resistance is 1.57 MPa. Friction varies from 0 to 75.6 kPa. The average value of Friction is 16.41 kPa. Friction ratio varies from 0 to 5.87 kPa. The average value of Friction ratio is 1.37 kPa. Figure 3.10 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at PURBACHAL.

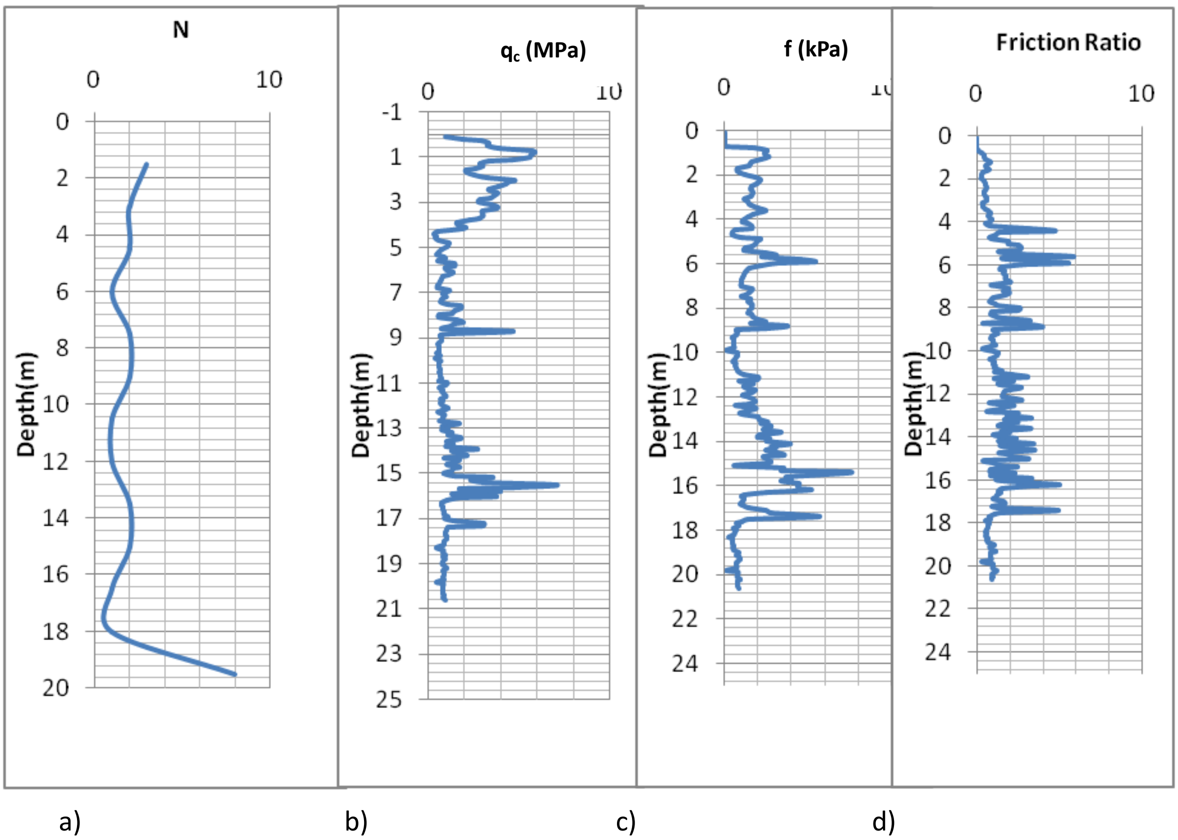


Fig 3.10: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at PURBACHAL.

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.7(a).

CPT results:

The graph between Depth vs friction and Cone resistance have been shown in Fig 3.10(c) and 3.10(b). From the CPT test, the Cone resistance varies from 0.355 to 7.06 MPa. The maximum value of cone resistance is 7.06 MPa. The minimum value of Cone resistance is 0.355 MPa. The average value of Cone resistance is 1.57 MPa. Friction varies from 0 to 75.6 kPa. The maximum value of Friction is 75.6 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 16.41 kPa. Friction ratio varies from 0 to 5.87 kPa. The maximum value of Friction ratio is 5.87 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.37 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.7(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm , 20 to 23% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.003 to 0.0055 mm, 88 to 96% respectively. Figure 3.7(b) shows the probable soil classification using CPT data (Robertson, 1990) at PURBACHAL.

Table 3.7(a): Grain size Data at PURBACHAL

DEPTH(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	3	20	0.17
3	Filling Sand	2	21	0.17
4.5	Filling Sand	2	22	0.16
6	Filling Sand	1	23	0.16
7.5	Clay	2	96	0.003
9	Clay	2	92	0.005
10.5	Clay	1	91	0.005
12	Clay	1	89	0.0055
13.5	Clay	2	88	0.0055
15	Clay	2	90	0.005
16.5	Clay	1	91	0.005
18	Clay	1	91	0.005
19.5	Clay	8	90	0.005

Table 3.7(b): Probable soil classification using CPT data (Robertson, 1990) at PURBACHAL

Depth Range (m)	I_c Range	Probable Soil Classification
0-3	0.60-1.62	Sand
3-6	1.42-3.15	Clay/sandy Silt/silty sand
6-9	1.72-3.15	Clay
9-12	2.70-3.24	Clay
12-15	2.37-3.34	Clay
15-18	1.99-3.34	Clay/Silty sand
18-21	2.94-3.50	Clay

3.7.8 Sub-Soil Characteristics of UNITED CITY

This site has been situated in Eastern part of Dhaka city. The depth of filling fine sand is 5.0 m from existing ground level. The clay layer exists from 5.0 m to 11.0m from EGL. After that 7.5 m is sandy silt layer. Then 1.5 m is silty fine sand. The uncorrected SPT N value of filling fine sand varies from 7 to 8. The SPT N value of clay layers varies from 4 to 7. The maximum value of SPT N is 15. The minimun value of SPT N is 3. From the CPT test, the cone resistance varies from 0.55 to 7.17MPa. The average value of cone resistance is 2.33 MPa. Friction varies from 0 to 273.2 kPa. The average value of Friction is 57.6 kPa. Friction ratio varies from 0 to 6.39 kPa. The average value of Friction ratio is 2.25 kPa. Figure 3.11 shows Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at UNITED CITY.

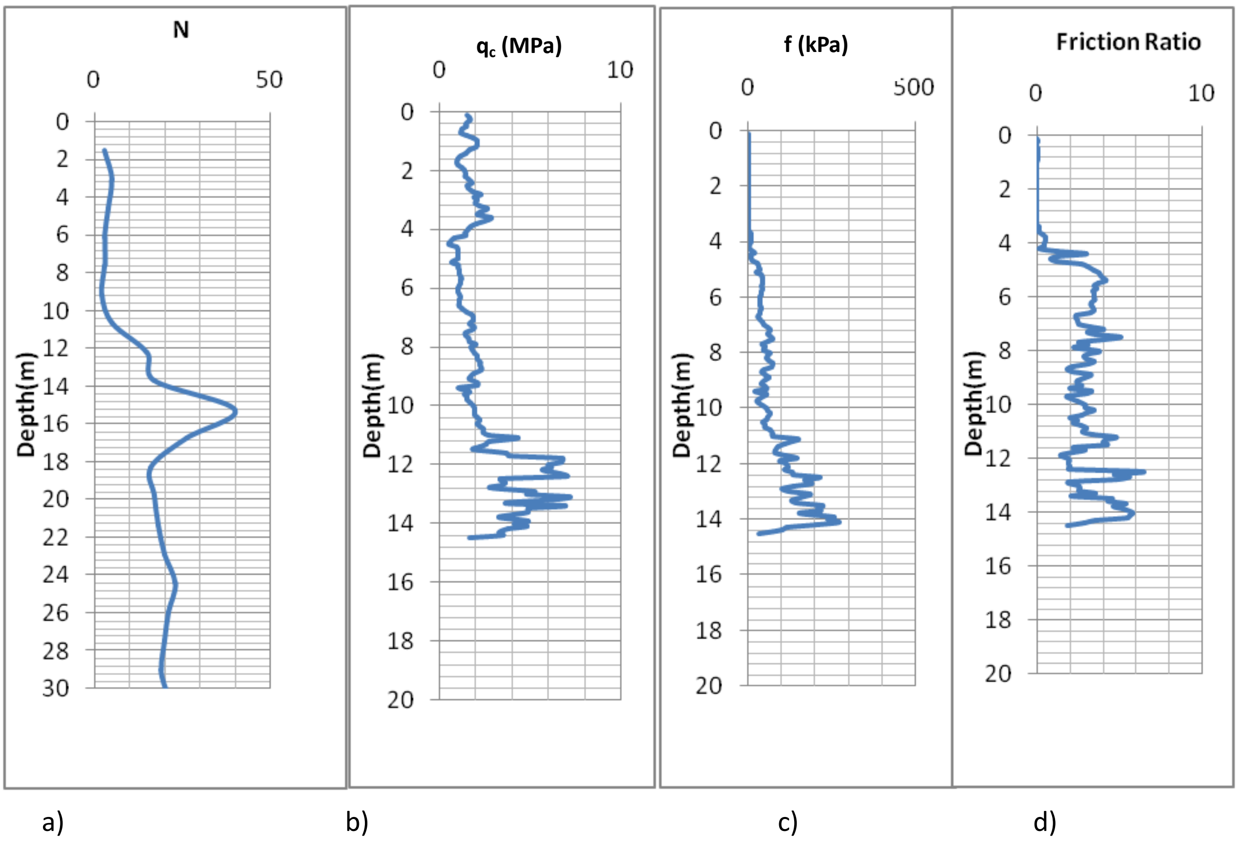


Fig 3.11: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at UNITED CITY

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.8(a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.11(c) and 3.11(b). From the CPT test, the cone resistance varies from 0.55 to 7.17 MPa. The maximum value of cone resistance is 7.17 MPa. The minimum value of cone resistance is 0.55 MPa. The average value of cone resistance is 2.33 MPa. Friction varies from 0 to 273.2 kPa. The maximum value of Friction is 273.2 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 57.6 kPa. Friction ratio varies from 0 to 6.39 kPa. The maximum value of Friction ratio is 6.39 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 2.25 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.8(a). The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm, 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.002 to 0.003 mm, 90 to 95 % respectively. The mean grain size (d_{50}), fine content (F_c) of sand varies 0.14 to 0.15 mm, 31 to 33% respectively. Table 3.8(b) shows the Probable soil classification using CPT data (Robertson, 1990) at UNITED CITY.

Table 3.8(a): Grain size analysis at UNITED CITY

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	8	20	0.16
3	Filling Sand	7	21	0.17
4.5	Clay	8	94	0.002
6	Clay	3	95	0.002
7.5	Clay	5	90	0.002
9	Clay	14	90	0.002
10.5	Clay	22	90	0.002
12	sand	32	33	0.14
13.5	sand	38	31	0.14
15	sand	45	31	0.15
16.5	sand	21	31	0.14
18	clay	10	91	0.003
19.5	clay	11	91	0.003

Table 3.8(b): Probable soil classification using CPT data (Robertson, 1990) at UNITED CITY

Depth Range (m)	I_c Range	Probable Soil Classification
0-3	0.77-1.44	Sand
3-6	1.40-2.87	Clay/sand
6-9	2.30-2.81	Clay/Silty sand
9-12	2.05-2.81	Clay/Silty Sand
12-15	2.15-2.76	Clay/Silty Sand

3.7.9 Sub-Soil Characteristics of UTTARA

This site has been situated in north of Dhaka city. It is a Government land development project. The depth of filling fine sand is 3.5 m from existing ground level. The clay layer exists from 3.5 m to 8.0m from EGL. After that 12 m is silt layer. The uncorrected SPT N value of filling fine sand varies from 2 to 3. The SPT N value of clay layers varies from 9 to 13. From the CPT test, the cone resistance varies from 0.36 to 4.85 MPa. The average value of cone resistance is 1.42 MPa. Friction varies from 0 to 69.1 kPa. The average value of Friction is 12.28 kPa. Friction ratio varies from 0 to 3.1 kPa. The average value of Friction ratio is 1.1 kPa. Figure 3.12 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at UTTARA.

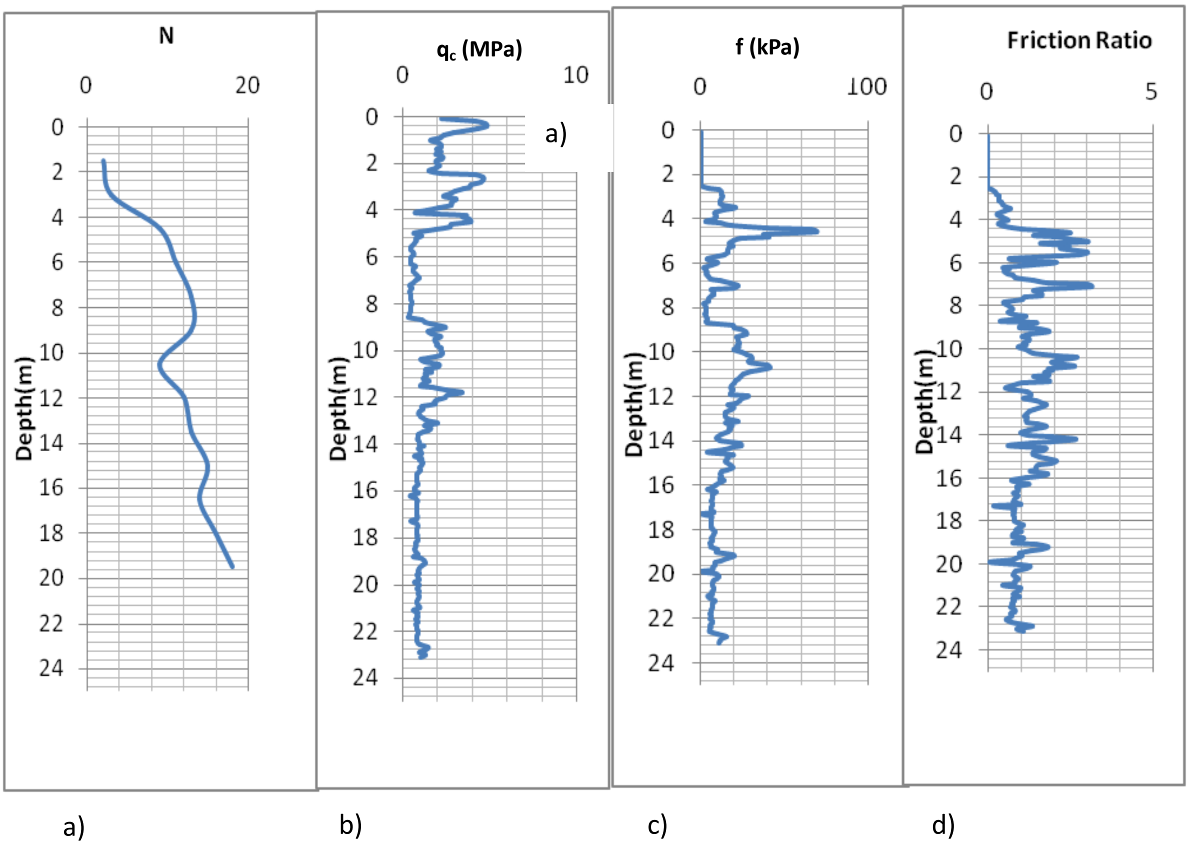


Fig 3.12: a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at UTTARA

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes have been shown in the Table 3.9(a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.12(c) and 3.12(b). From the CPT test, the cone resistance varies from 0.36 to 4.85 MPa. The maximum value of cone resistance is 4.85 MPa. The minimum value of cone resistance is 0.36 MPa. The average value of cone resistance is 1.42 MPa. Friction varies from 0 to 69.1 kPa. The maximum value of Friction is 69.1 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 12.28 kPa. Friction ratio varies from 0 to 3.1 kPa. The maximum value of Friction ratio is 3.1 kPa. The minimum value of Friction ratio is 0 kPa. The average value of Friction ratio is 1.1 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.9(a). The mean grain size (d_{50}) , fine content (F_c) of filling sand varies 0.16 to 0.17 mm , 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.002 to 0.003 mm, 94 to 96% respectively. The mean grain size (d_{50}), fine content (F_c) of silt varies 0.046 mm, 71 to 72% respectively. Table 3.9(b) shows the probable soil classification using CPT data (Robertson, 1990) at UTTARA.

Table 3.9(a): Grain size analysis at UTTARA

Depth(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	2	20	0.17
3	Filling Sand	3	21	0.16
4.5	Silty Clay	9	94	0.003
6	Silty Clay	11	95	0.003
7.5	Silty Clay	13	96	0.002
9	Sandy Silt	13	71	0.056
10.5	Sandy Silt	9	71	0.056
12	Sandy Silt	12	72	0.056
13.5	Sandy Silt	13	71	0.056
15	Sandy Silt	15	71	0.066
16.5	Sandy Silt	14	71	0.066
18	Clayey Silt	16	71	0.046
19.5	Cayey Silt	18	71	0.046

Table 3.9(b): Probable soil classification using CPT data (Robertson, 1990) at UTTARA

Depth Range(m)	I_c Range	Probable Soil Classification
0-3	1.04-1.38	Sand
3-6	1.44-2.96	Sand/Silty clay
6-9	2.22-3.17	Clay/Silty clay
9-12	2.06-2.89	Sandy Silt
12-15	2.49-3.12	Clay/Sandy Silt
15-18	2.91-3.57	Clay
18-23	2.84-3.50	Clay

3.7.10 Sub-soil characteristics of KAMRANGICHAR

This site has been situated in south-east part of Dhaka city. It is an island surrounded by river Buriganga. It is connected to main land by two bailey bridges and there is quite dense population in the area. The depth of filling fine sand is 3.5 m from existing ground level. The clay layer exists from 8.0 m to 9.5 m from EGL. After that 10.5 m is fine sand layer. The uncorrected SPT N value of filling fine sand varies from 3 to 7. The maximum value of SPT N is 28. The minimun value of SPT N is 3. From the CPT test, the cone resistance varies from 0.38 to 9.85 MPa. The average value of cone resistance is 4.21 MPa. Friction varies from 0 to 96.5 kPa. The average value of Friction is 29 kPa. Friction ratio varies from 0 to 5.40 kPa. The average value of Friction ratio is 0.88 kPa. Figure 3.13 shows a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at KAMRANGICHAR.

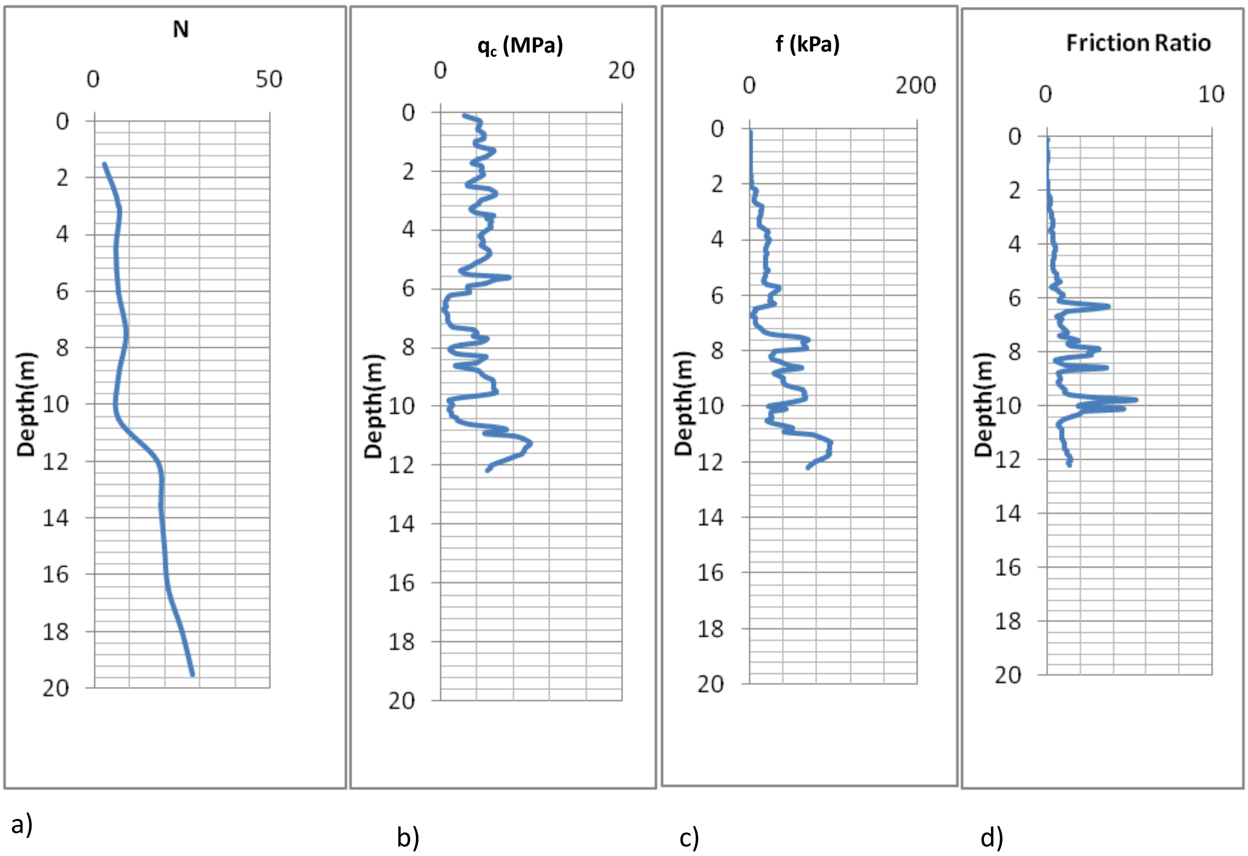


Fig 3.13:a) Depth (m) vs N b) Depth (m) vs Cone Resistance (MPa) c) Depth (m) vs Friction (kPa) d) Depth (m) vs Friction Ratio at KAMRANGICHAR.

SPT Results:

SPT has been conducted in the area following procedure described in ASTM D1586. The SPT N values of the boreholes are shown in the Table 3.10(a).

CPT results:

The graph between depth vs friction and cone resistance have been shown in Fig 3.10(c) and 3.10(b). From the CPT test, the cone resistance varies from 0.38 to 9.85 MPa. The maximum value of cone resistance is 9.85 MPa. The minimum value of cone resistance is 0.38 MPa. The average value of cone resistance is 4.21 MPa. Friction varies from 0 to 96.5 kPa. The maximum value of Friction is 96.5 kPa. The minimum value of Friction is 0 kPa. The average value of Friction is 29 kPa. Friction ratio varies from 0 to 5.40 kPa. The maximum value of Friction ratio is 5.40 kPa. The minimum value of Friction ratio is 0 KPa. The average value of Friction ratio is 0.88 kPa.

Grain size analysis results:

Results from grain size analysis of the soil samples have been presented in Table 3.10. The mean grain size (d_{50}), fine content (F_c) of filling sand varies 0.16 to 0.17 mm, 20 to 21% respectively. The mean grain size (d_{50}), fine content (F_c) of sand varies 0.14 to 0.17 mm , 30 to 33% respectively. The mean grain size (d_{50}), fine content (F_c) of clay varies 0.003 mm, 90% respectively. Table 3.10 shows the Probable soil classification using CPT data (Robertson, 1990) at KAMRANGICHAR.

Table 3.10(a) Grain size analysis at KAMRANGICHAR

DEPTH(m)	Description of Soil	SPT N Value	F_c(%)	d_{50}(mm)
1.5	Filling Sand	3	20	0.17
3	Filling Sand	7	21	0.16
4.5	Sand	6	33	0.16
6	Sand	7	32	0.16
7.5	Sand	9	33	0.18
9	Silty Clay	7	90	0.003
10.5	Sand	7	35	0.17
12	Sand	18	32	0.17
13.5	Sand	19	31	0.16
15	Sand	20	31	0.16
16.5	Sand	21	31	0.18
18	Sand	25	30	0.17
19.5	Sand	28	33	0.14

Table 3.10(b): Probable soil classification using CPT data (Robertson, 1990) at KAMRANGICHAR

Depth Range (m)	I_c Range	Probable Soil Classification
0-3	0.73-1.52	Sand
3-6	1.16-2	Sand
6-9	1.78-2.93	Silty sand/Sandy sily
9-12	1.78-3.08	Sand/silty Clay

3.8 Conclusion Remarks

The maximum value of N is 42 from the ten selected sites. The maximum value of q_c is 16.78 MPa from the ten selected sites. The variation of others different parameters like friction and friction ratio with depth has been described in this chapter. The equipments which have been used in CPT test also have been described in this chapter. Probable soil classification using CPT data have been established by Robertson 1990 method.

CHAPTER FOUR
LIQUEFACTION POTENTIAL ANALYSIS BASED ON SPT AND CPT

4.1 General

The main objective of this chapter is to present the liquefaction potential of the selected reclaimed areas. Soil characteristics of the selected reclaimed areas have been determined by field and labortatory tests. Test results have been presented in Chapter Three. Liquefaction potential have been estimated using two methods based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data. The results of these estimations have been presented in this chapter. Results obtained from different methods have been compared and suitable methods for estimating liquefaction potential has been presented.

4.2 Liquefaction Potential

In this research liquefaction potential has been estimated by procedures based on both SPT and CPT test results. Detail procedures have been discussed in chapter Three. Parameter of analysis and details results of estimations based on these procedures has been discussed below.

Parametes for analysis

Liquefaction analysis based on SPT and CPT need ground motion characteristics and moment magnitude due to earthquake and depth of analysis. In this study, following values of these parameters have been considered.

1) Ground motion and Moment Magnitude:

In this research , the value of a_{max} has been taken as 0.15g as Dhaka city exist in the zone 2 of seismic zonation map of Bangladesh (BNBC, 1993). Other researchers (Ansary and Rashid, 2000) also used similar values of a_{max} for the similar purpose. Though at present the value of a_{max} is being update by various researchers and agencies from 0.15 to 0.2. But it has not been taken to the consideration for this study since it has not been incorporated to BNBC yet. Earthquake ground motion has been influenced by a number of factors. Most important factors are moment magnitude, epicenter distances, local soil conditions, earthquake sources, etc. In seed-Idriss simplified procedure moment magnitude (M_w) input parameter is also important correction factor. From Table 2.1, it is seen that ranges of M_w at nearby faults from Dhaka varies 7.5~8.5. However, this value can not be considered directly for Dhaka since those faults are at quite distant places from Dhaka. Due to non-availability of attenuation law and suitable correlaions between distance and ground motion characteristics for Dhaka, the design moment magnitude has been taken 7.0 for this study, which is the lowest value in Table 2.1.

2) Sources of Damaging Earthquake near Dhaka city:

Bangladesh covers one of the largest deltas and one of the thickest sedimentary basins in the world. According to the report on time predictable fault modeling (2009), earthquake and tsunami preparedness component of CDMP have identified five tectonic fault zones which may produce damaging earthquakes in Bangladesh. These are Madhupur fault zone, dauki fault zone, plate boundary fault zone -1, plate boundary fault zone -2, and plate boundary fault zone -3. Among these, Madhupur fault zone has been considered as a source of damaging earthquake near Dhaka in this study.

4.2.1 Liquefaction Potential of BRAMANGAON

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al;1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.1. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~4.5 m. (Fig 4.1).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.1, liquefaction zones vary between 1.5 ~10 m and from 18~22.5m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~10 m and from 18~22.5 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

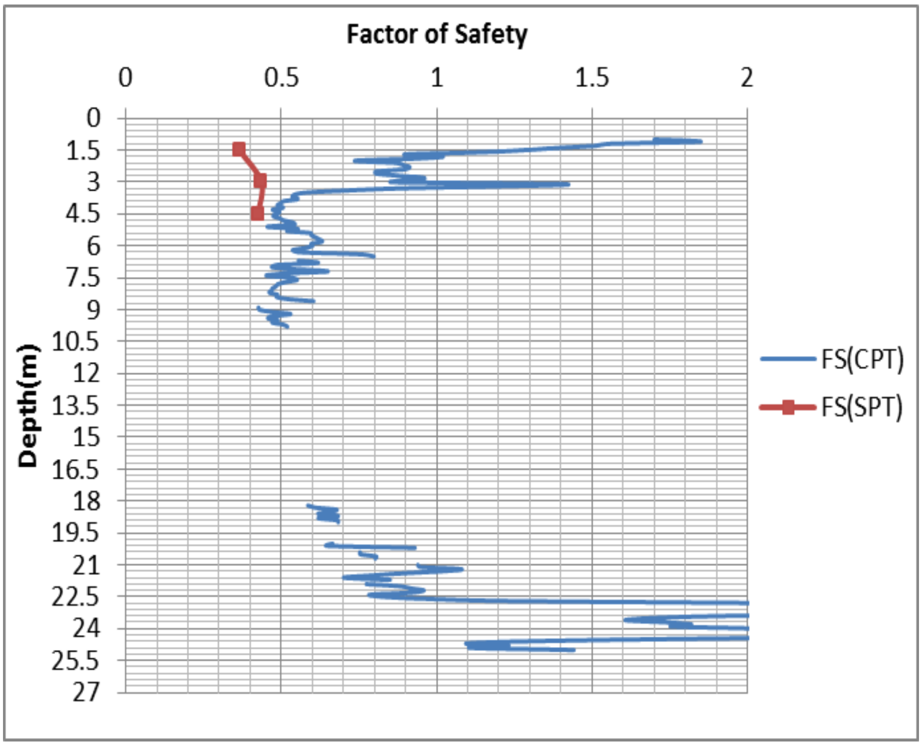


Fig 4.1: Depth(m) vs FS at BRAMANGAON

4.2.2 Liquefaction Potential of ASHIAN CITY

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.2. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~3 m. (Fig 4.2).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.2, liquefaction zones vary between 1.8~3.9 m and from 12.9~14.4 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~3.9 m and from 12.9~14.4 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

.

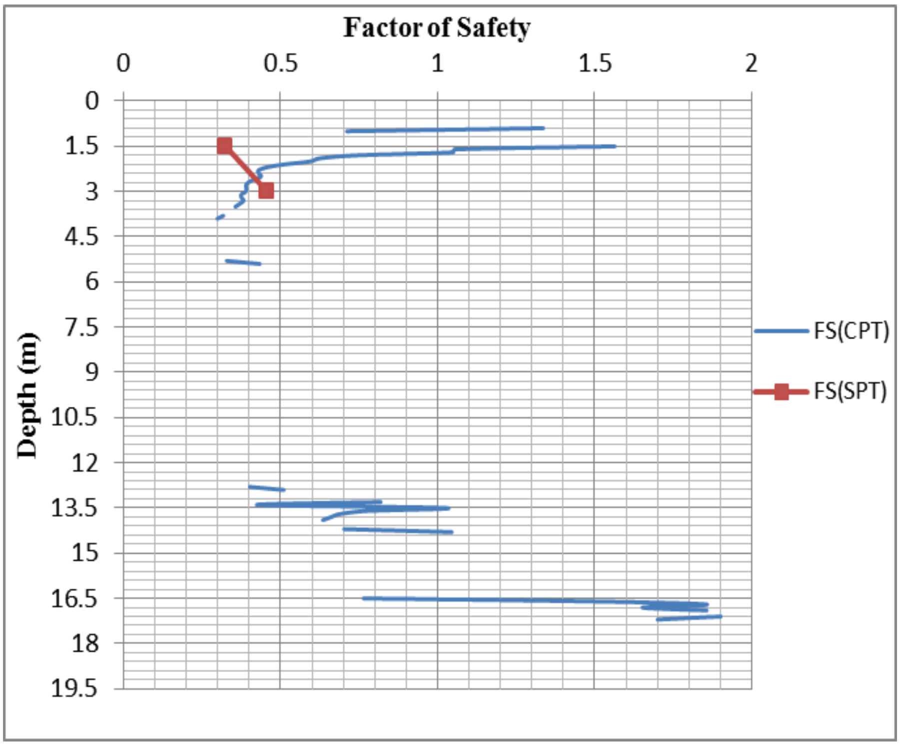


Fig 4.2: Depth(m) vs FS at ASHIAN

4.2.3 Liquefaction Potential of BADDA

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data were have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.3. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~4.5 m. (Fig 4.3).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.3, liquefaction zones vary between 4.5 ~5.5 m and from 8~9.5m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~5.5 m and from 8~9.5 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

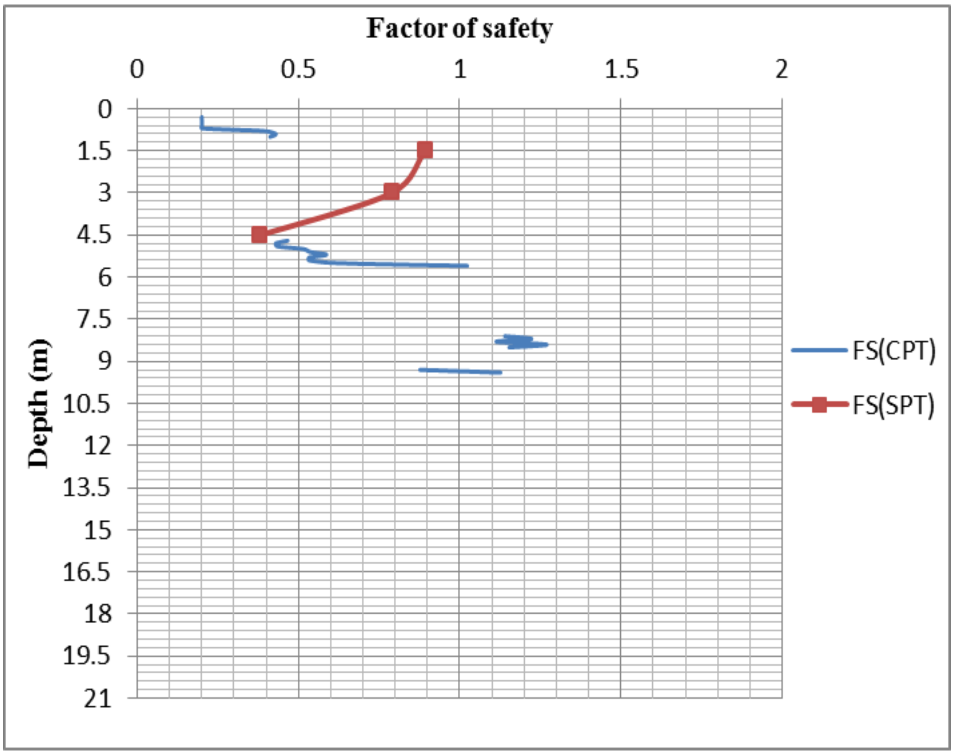


Fig 4.3: Depth (m) vs FS at BADDA

4.2.4 Liquefaction Potential of BANASREE

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.1. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~4.5 m. (Fig 4.1).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.1, liquefaction zones vary between 0.7 ~4.8 m and 14.4~15.3 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 0.7~4.8 m and from 14.4~15.3 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1 m depth.

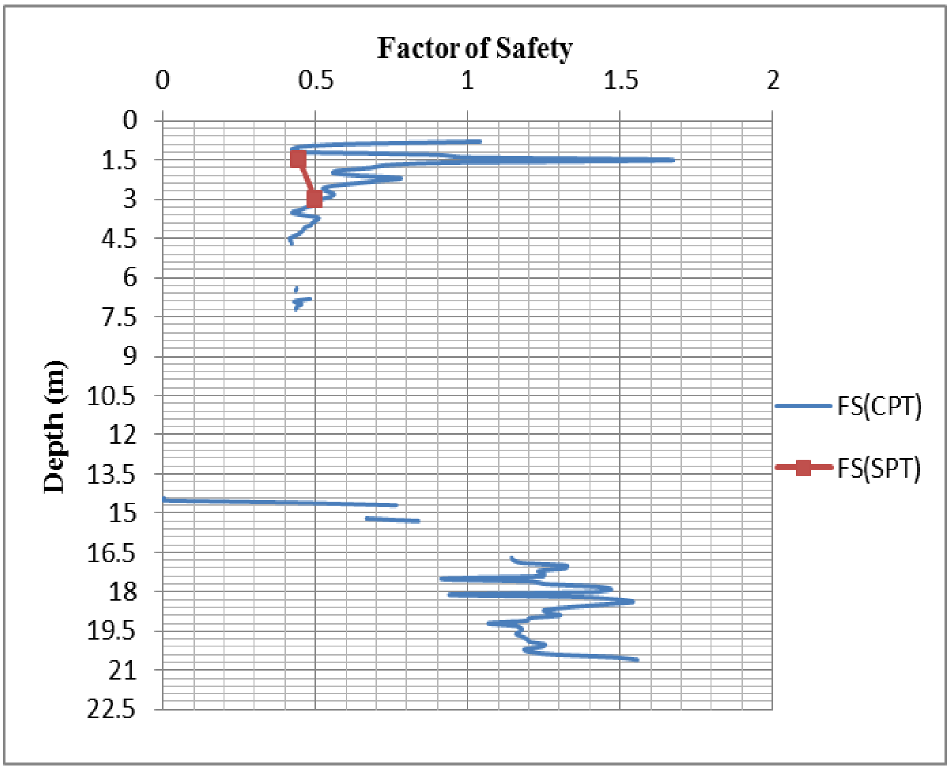


Fig 4.4: Depth(m) vs FS at BANASREE

4.2.5 Liquefaction Potential of GABTOLI

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.5. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~4.5 m. (Fig 4.5).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.5, liquefaction zones vary between 0.6 ~5.4 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 0.6~5.4 m depth if an Earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

.

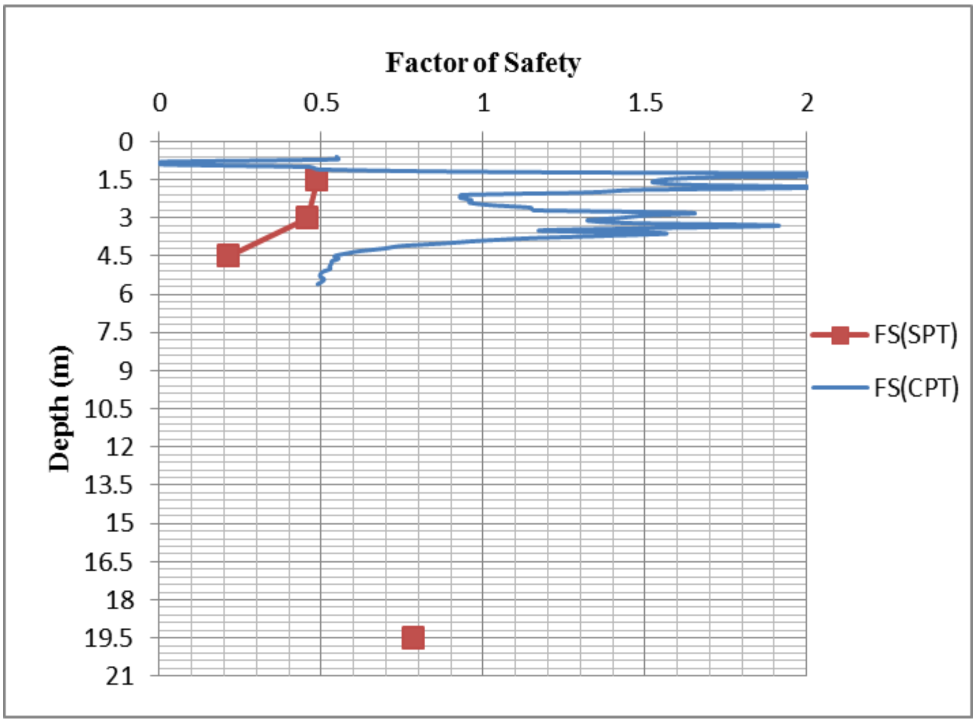


Fig 4.5: Depth (m) vs FS at GABTOLI

4.2.6 Liquefaction Potential of KAWRAN BAZAR

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.6. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction have been occurred at 12 m depth. (Fig 4.6).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.6, liquefaction zones vary between 11.4 ~12.3 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 11.4~12.3 m depth if an Earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

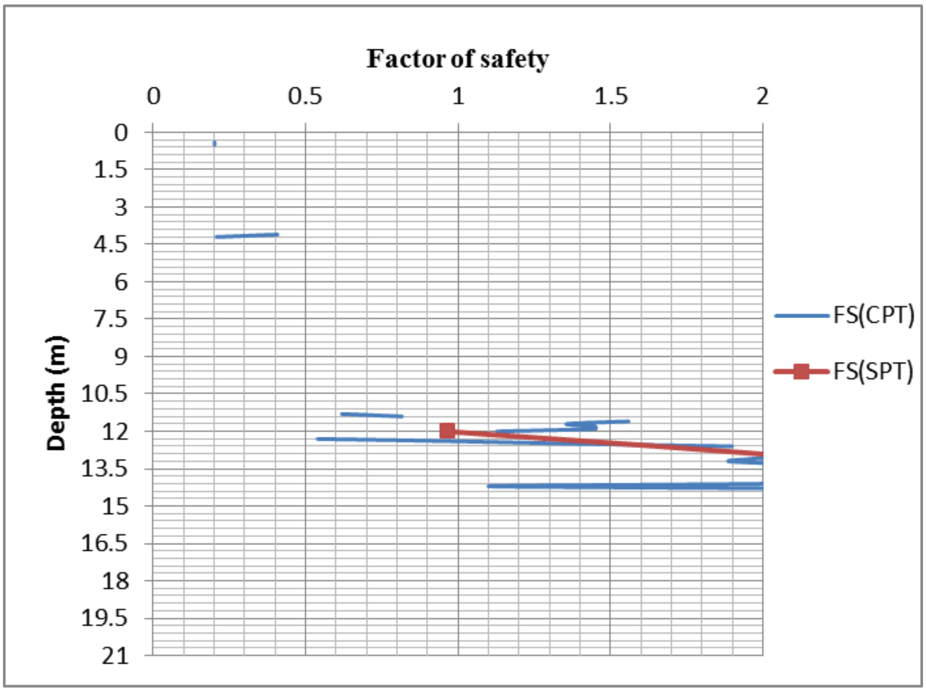


Fig 4.6: Depth(m) vs FS at KAWRAN BAZAR

4.2.7 Liquefaction Potential of PURBACHAL

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.7. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~6 m. (Fig 4.7).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.7, liquefaction zones vary between 1.5 ~5.0 m and from 15~15.6 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~6 m and from 18~22.5 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

.

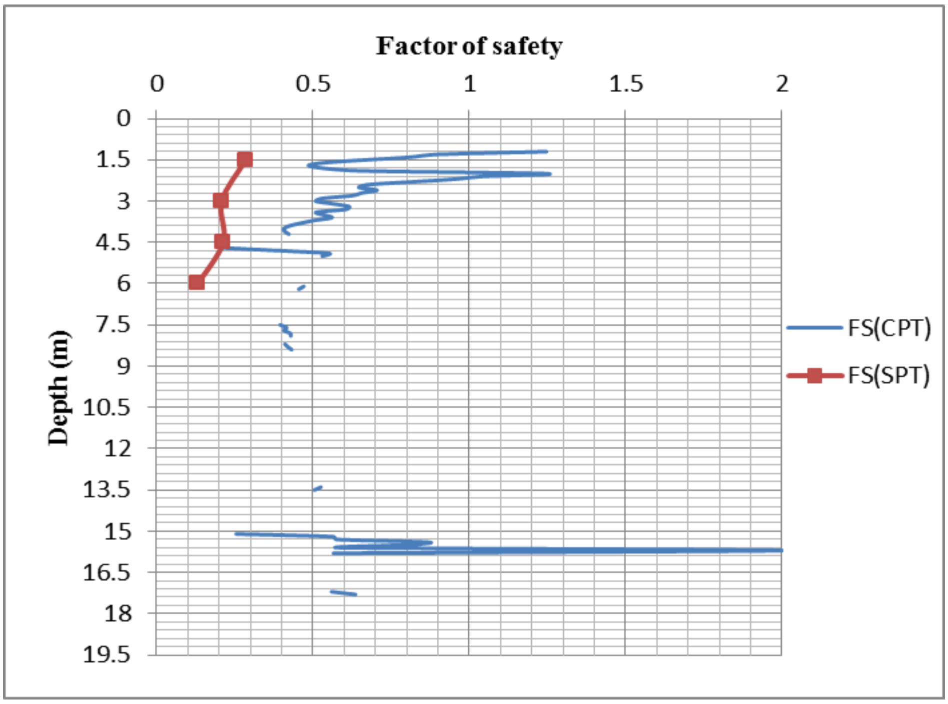


Fig 4.7: Depth(m) vs FS at PURBACHAL

4.2.8 Liquefaction Potential of UNITED CITY

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.8. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~3 m (Fig 4.8).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.8, liquefaction zones vary between 3.0~4.5 m. and 10.5~12.0 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~4.5 m and from 10.5~12 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

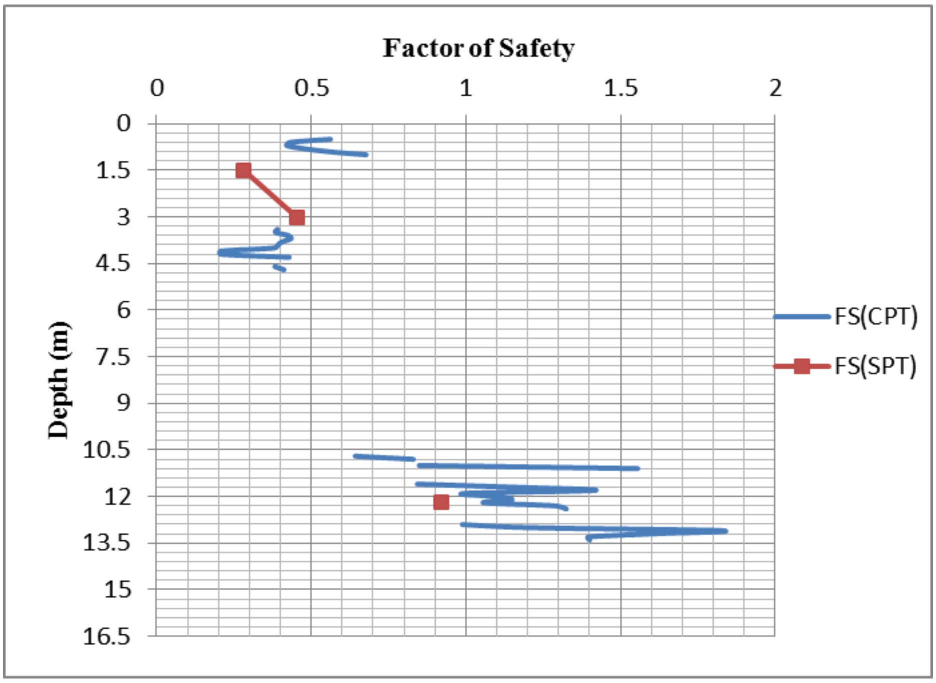


Fig 4.8: Depth(m) vs FS at UNITED CITY

4.2.9 Liquefaction Potential of UTTARA

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.9. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~4.5 m. (Fig 4.9).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.9, liquefaction zones vary between 2.7 ~4.8 m and from 8.7~12.3 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~4.8 m and from 8.7~12.3 m depth if an earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

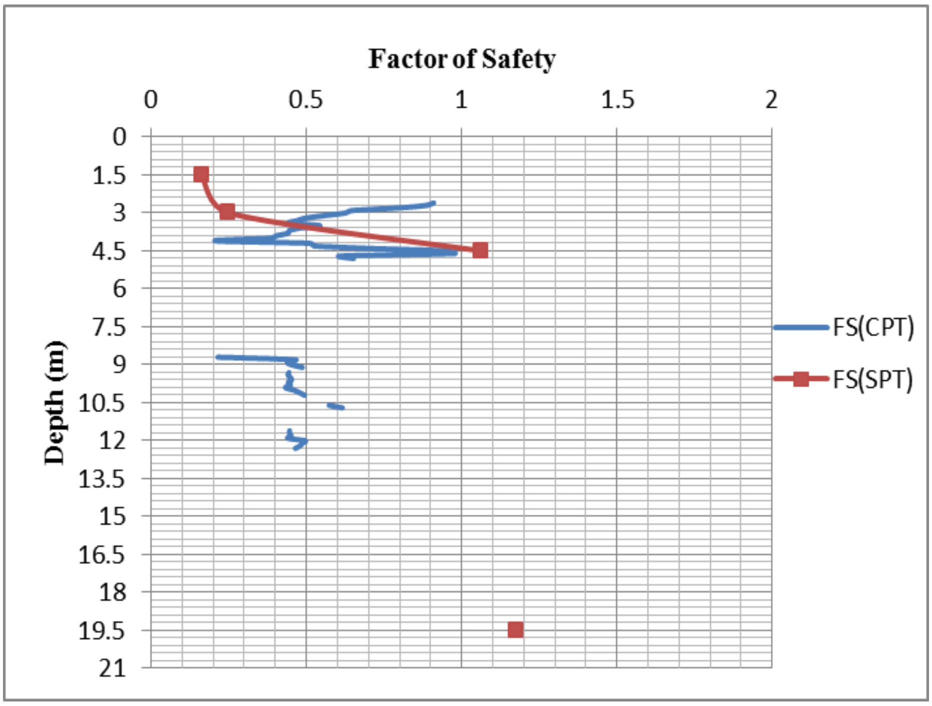


Fig 4.9: Depth(m) vs FS at UTTARA

4.2.10 Liquefaction Potential of KAMRANGICHAR

On the basis of soil characteristics of this locations that have been presented in chapter 3. Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated. A typical liquefaction potential analysis has been shown in Fig 4.10. Liquefiable zone is where F_l <1, on the other hand Non liquefiable zone is where F_l >1. The liquefaction analyses results by different procedures have been presented below:

- Liquefaction susceptibility has been estimated based on the method proposed by Seed et al;1983 at different depths. The liquefaction zones vary between 1.5~7.5 m and from 10.5~11.5 m (Fig 4.10).
- Liquefaction susceptibility has been estimated based on the method proposed by Robertson and Wride, 1998. From the Fig 4.10, liquefaction zones vary between 1.5 ~6 m and from 7.5~12 m.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~12 m depth if an Earthquake of sufficient energy occurs. CPT is more reliable than SPT as it is performed at each 0.1m depth.

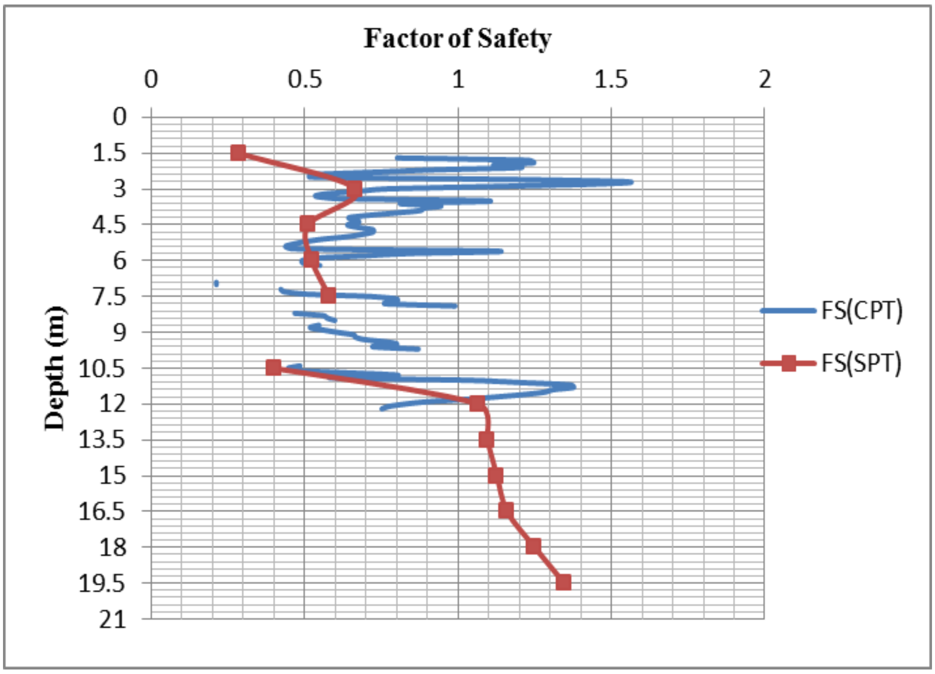


Fig 4.10 Depth(m) vs FS at KAMRANGICHAR

4.3 Summary

Liquefaction potential from SPT and CPT data have been obtained in this chapter. Liquefaction potential vs depth have been plotted in this chapter. Most of the case liquefaction susceptible soil have been found upper 1.5 to 4.5 m depth of the filling sand. Liquefaction zone for CPT of Bramangaon, Ashian City, Badda, Banasree, Gabtoli, Kawran Bazar, Purbachal, United City, Uttara AND Kamrangirchar are 1.5 ~10 m and 18~22.5m, 1.8~3.9 m and 12.9~14.4 m, 4.5 ~5.5 m and 8~9.5m, 0.7 ~4.8 m and 14.4~15.3 m, 0.6 ~5.4 m, 11.4 ~12.3 m, 1.5 ~5.0 m and 15~15.6 m, 3.0~4.5 m. and 10.5~12.0 m, 2.7 ~4.8 m and 8.7~12.3 m, and 1.5 ~6 m and 7.5~12 m respectively. Table 4.1 shows the liquefaction zone of two different method.

Table 4.1: Liquefaction zone of two different method.

Site Name	Liquefaction zone (Seed et al;1983)	Liquefaction Zone (Robertson and Wride, 1998)
BRAMANGAON	1.5~4.5 m	1.5 ~10 m and 18~22.5m
ASHIAN CITY	1.5~3 m	1.8~3.9 m and 12.9~14.4 m
BADDA	1.5~4.5 m	4.5 ~5.5 m and 8~9.5m
BANASREE	1.5~4.5 m	0.7 ~4.8 m and 14.4~15.3 m
GABTOLI	1.5~4.5 m	0.6 ~5.4 m
KAWRAN BAZAR	at 12 m depth	11.4 ~12.3 m
PURBACHAL	1.5~6 m	1.5 ~5.0 m and 15~15.6 m
UNITED CITY	1.5~3 m	3.0~4.5 m. and 10.5~12.0 m
UTTARA	1.5~4.5 m	2.7 ~4.8 m and 8.7~12.3 m
KAMRANGI CHAR	1.5~7.5 m and from 10.5~11.5 m	1.5 ~6 m and 7.5~12 m

CHAPTER FIVE

ESTIMATION OF SOIL PARAMETER FROM CPT

5.1 General

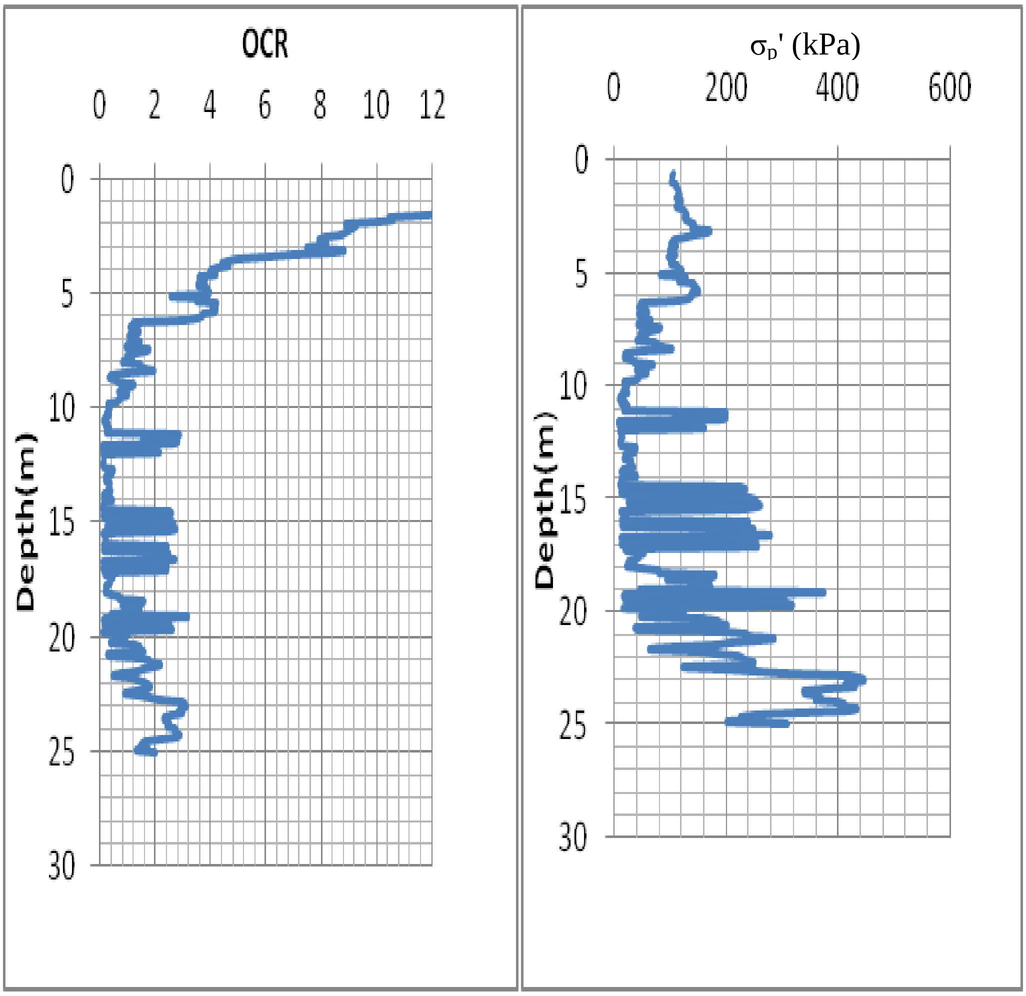
In this chapter, it has been known that there are different correlations between Cone Penetration Test (CPT) and Standard Penetration Test (SPT). The SPT has some disadvantages such as potential variability of measured resistances depending on operator variability and possibility of missing delicate changes of soil properties owing to the inevitable discrete record. Around the world the Cone Penetration Test (CPT) is becoming increasingly popular as an in situ test for site investigation and geotechnical design especially in deltaic areas since it provides a continuous record which is free from operator variability (Suzuki et al. 1998). Geotechnical engineers have gained considerable experience in design based on local SPT correlations. Thus there is a need for reliable CPT-SPT correlations so that CPT data can be used.

5.2 Estimation of Various Parameters

Soils are very complex materials because they can comprised of a wide and diverse assemblage of different particles sizes, mineralogies, packing arrangements, and fabric. Moreover, they can be created from various geologic origins (marine, lacustrine, glacial,residual, Aeolian, deltaic,alluvial, estuarine,fluvial, biochemical, etc) that have undergone long periods of environmental, seasonal, hydrological and thermal processes. These facets have imparted complexities of soil behavior that relate to their initial geostatic stress state, natural prestressing, nonlinear stress-strain-strenth response, and drainage and flow characteristics, as well as rheological and time rate effects. As such, a rather large number of different geotechnical parameters have been identified to quantify soil behavior in engineering terms. These include state parameters such as void ratio (e_0), porosity (n), relative density (D_R), overconsolidation ratio (OCR), strength parameters (c, ϕ), permeability (k), lateral stess parameters (k) and more. In this section, the evaluation of selected geotechnical parameters from CPT data has been adressed including various post processing approaches based on theoretical, numerical, analytical and empirical methods.

5.2.1 Soil Parameter of BRAMANGAON

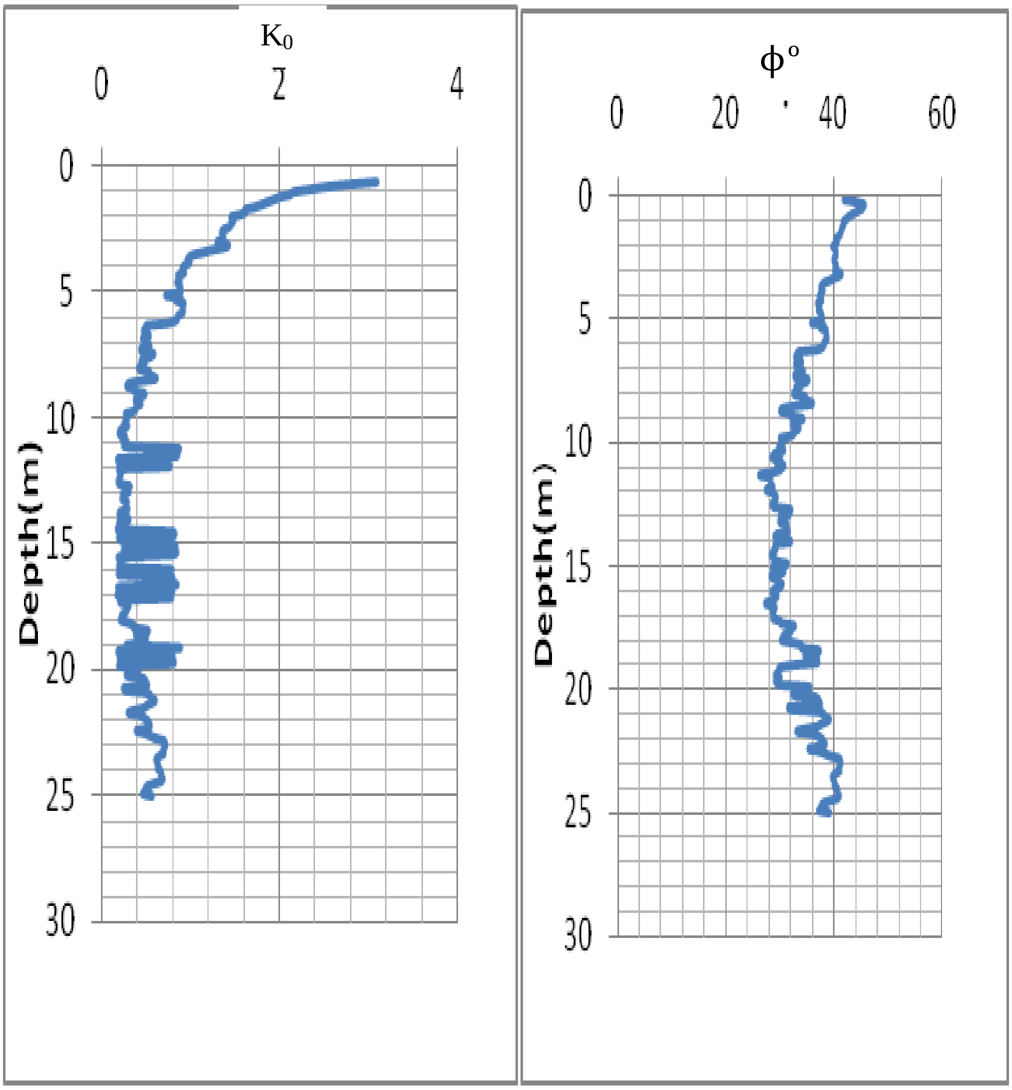
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.1 a). The maximum value of OCR is 28.13.The minimum value of OCR is 0.107. The average value of OCR is 2.73. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.1 b). The maximum value of σ_p' is 444 kPa. The minimum value of σ_p' is 7.69 kPa. The average value of σ_p ' is 122 kPa.



(a) (b)

Fig 5.1:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at BRAMANGAON

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.2 a). The maximum value of K_0 is 3.10. The minimum value of K_0 is 0.175. The average value of K_0 is 0.64. Again from the equation (2.5), the $\phi°$ value have been found. Then depth vs ϕ graph has been drawn (Fig 5.2 b). The maximum value of ϕ is 45.49°. The minimum value of ϕ is 26.23°. The average value of ϕ is 34.59°.



(a) (b)

Fig 5.2: a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at BRAMANGAON

5.2.2 Soil Parameter of ASHIAN CITY

From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.3 a). The maximum value of OCR is 61.33. The minimum value of OCR is 0.08. The average value of OCR is 3.60. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.3 b). The maximum value of σ_p' is 220 kPa. The minimum value of σ_p' is 2.18 kPa. The average value of σ_p' is 72.11 kPa.

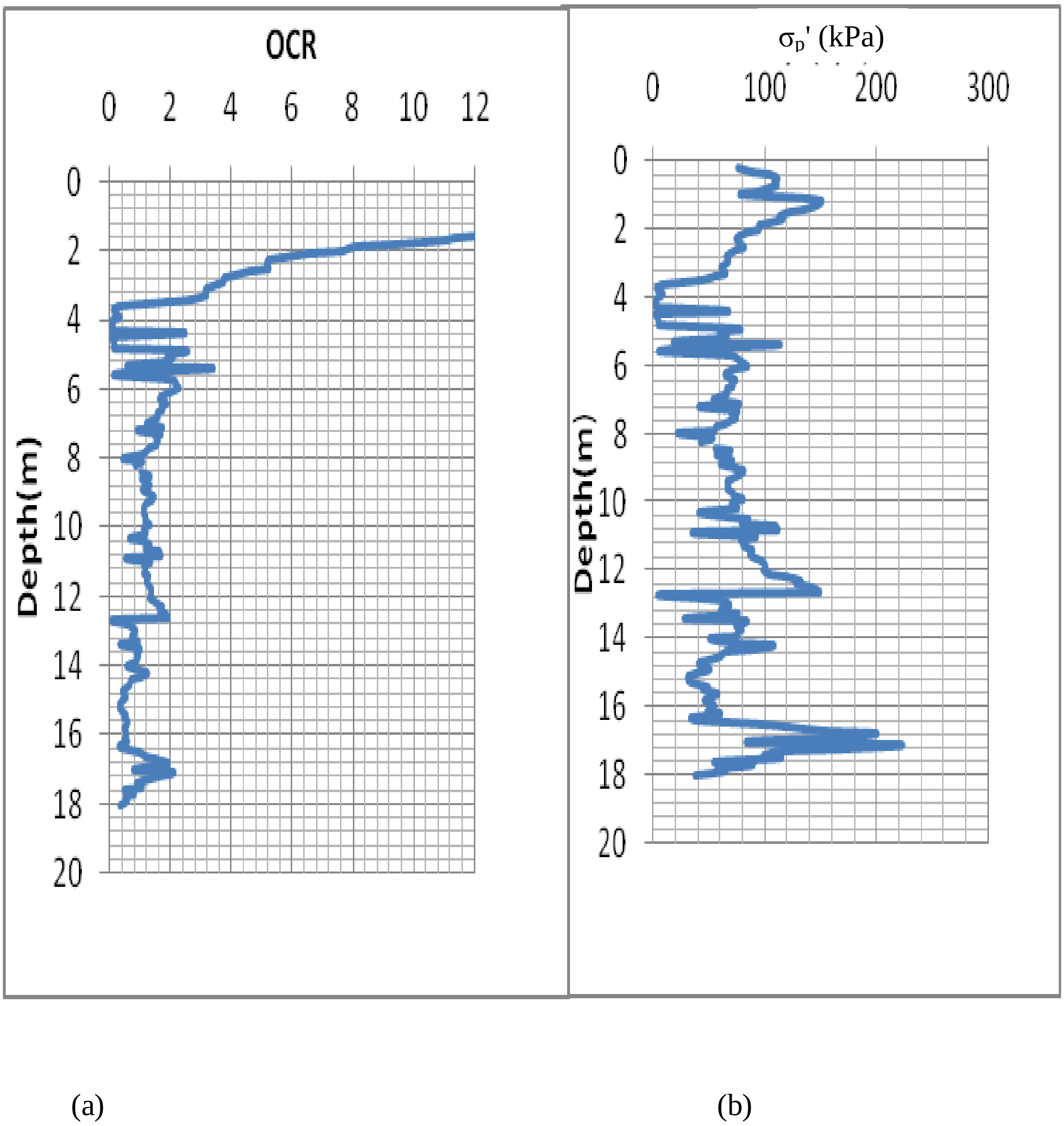
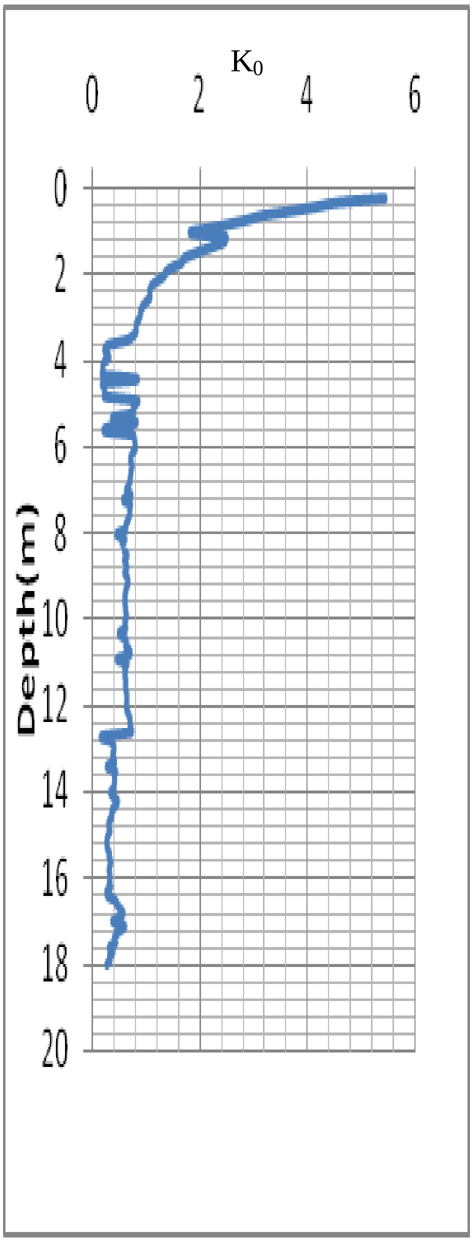
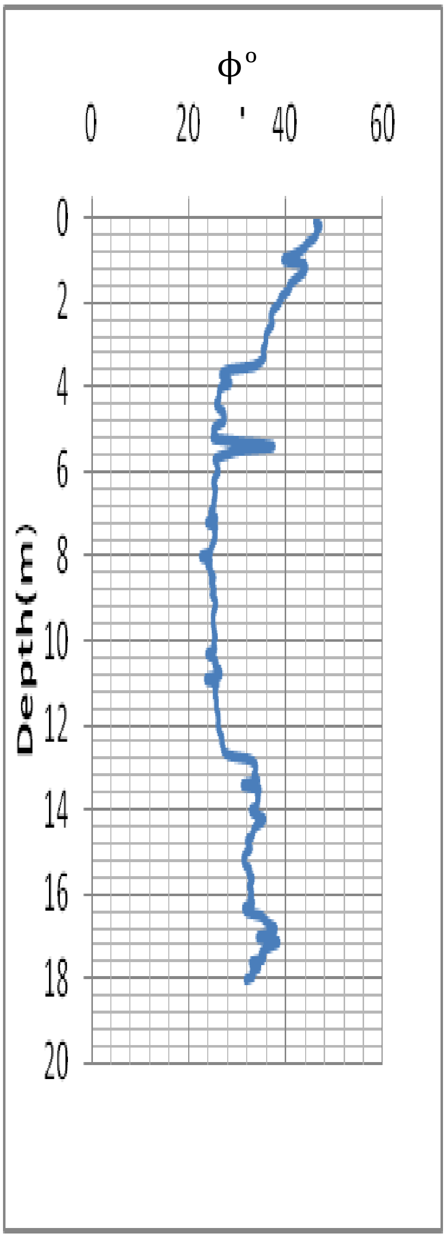


Fig 5.3:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at ASHIAN CITY

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.4 a). The maximum value of K_0 is 5.45. The minimum value of K_0 is 0.174. The average value of K_0 is 0.76. Again from the equation (2.5), the ϕ value have been found. Then depth vs ϕ^o graph has been drawn (Fig 5.4 b). The maximum value of ϕ is 47°. The minimum value of ϕ is 22°. The average value of ϕ is 30°.



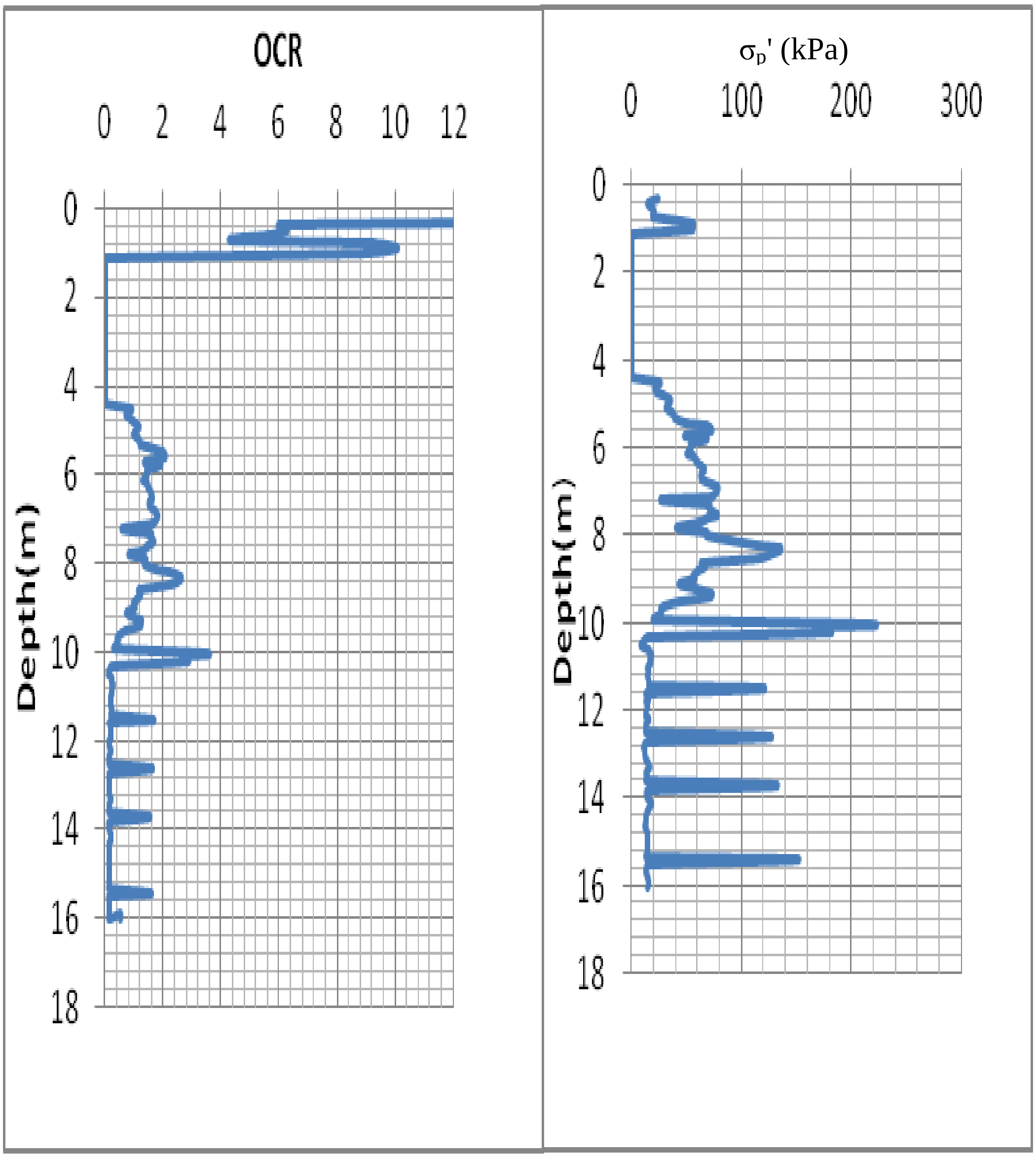


(a) (b)

Fig 5.4:a) Depth (m) vs K_0 b) Depth (m) vs ϕ^o at ASHIAN CITY

5.2.3 Soil Parameter of BADDA

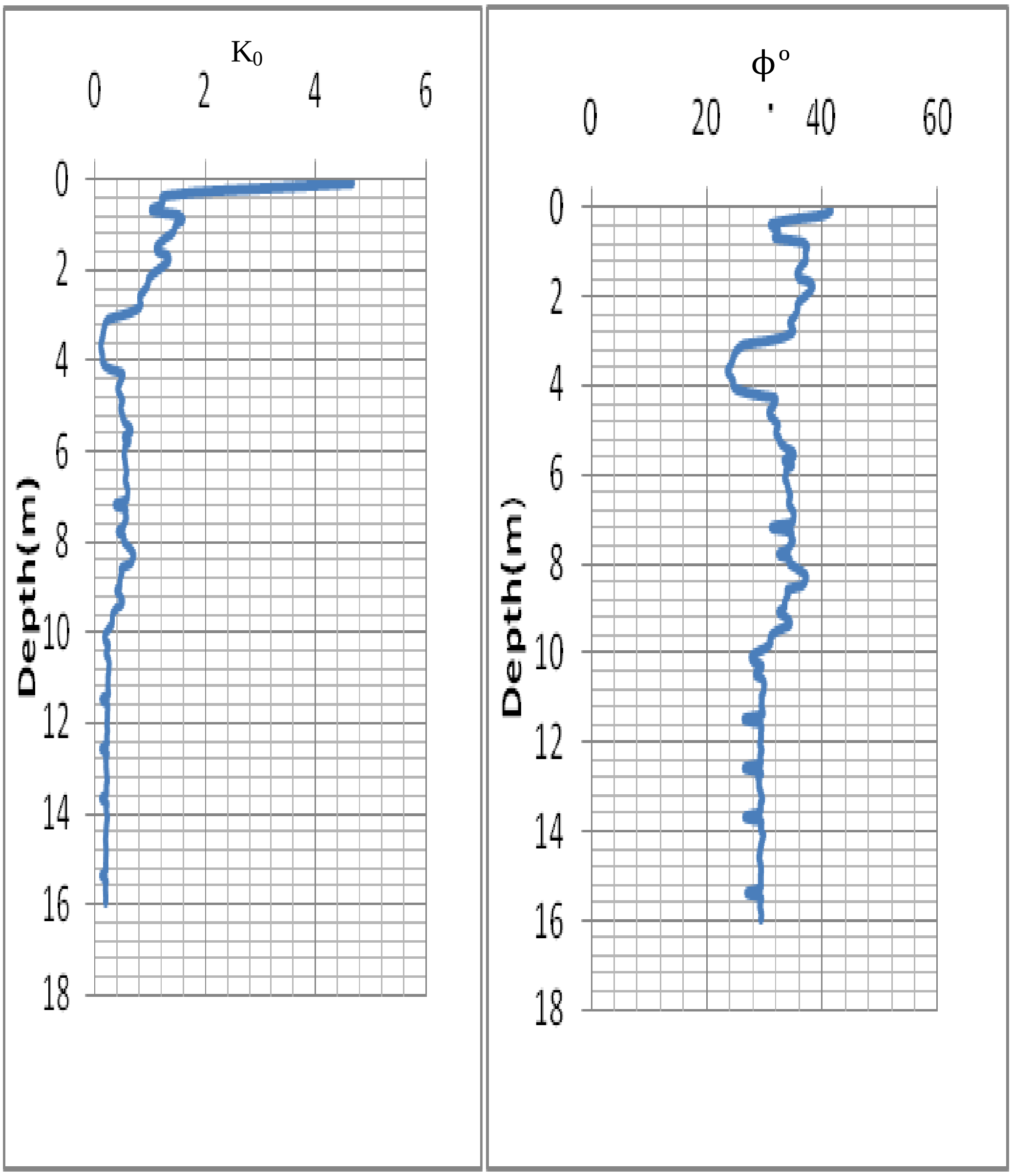
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.5 a). The maximum value of OCR is 13.24. The minimum value of OCR is 0.13. The average value of OCR is 1.31. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.5 b). The maximum value of σ_p' is 217 kPa. The minimum value of σ_p' is 9 kPa. The average value of σ_p' is 43 kPa.



(a) (b)

Fig 5.5:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at BADDA

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K graph has been drawn (Fig 5.6 a). The maximum value of K_0 is 4.67. The minimum value of K_0 is 0.107. The average value of K is 0.52. Again from the equation, the ϕ value have been found. Then depth vs ϕ^o graph has been drawn (Fig 5.6 b). The maximum value of ϕ is 42^o. The minimum value of ϕ is 24^o.The average value of ϕ is 41^o.

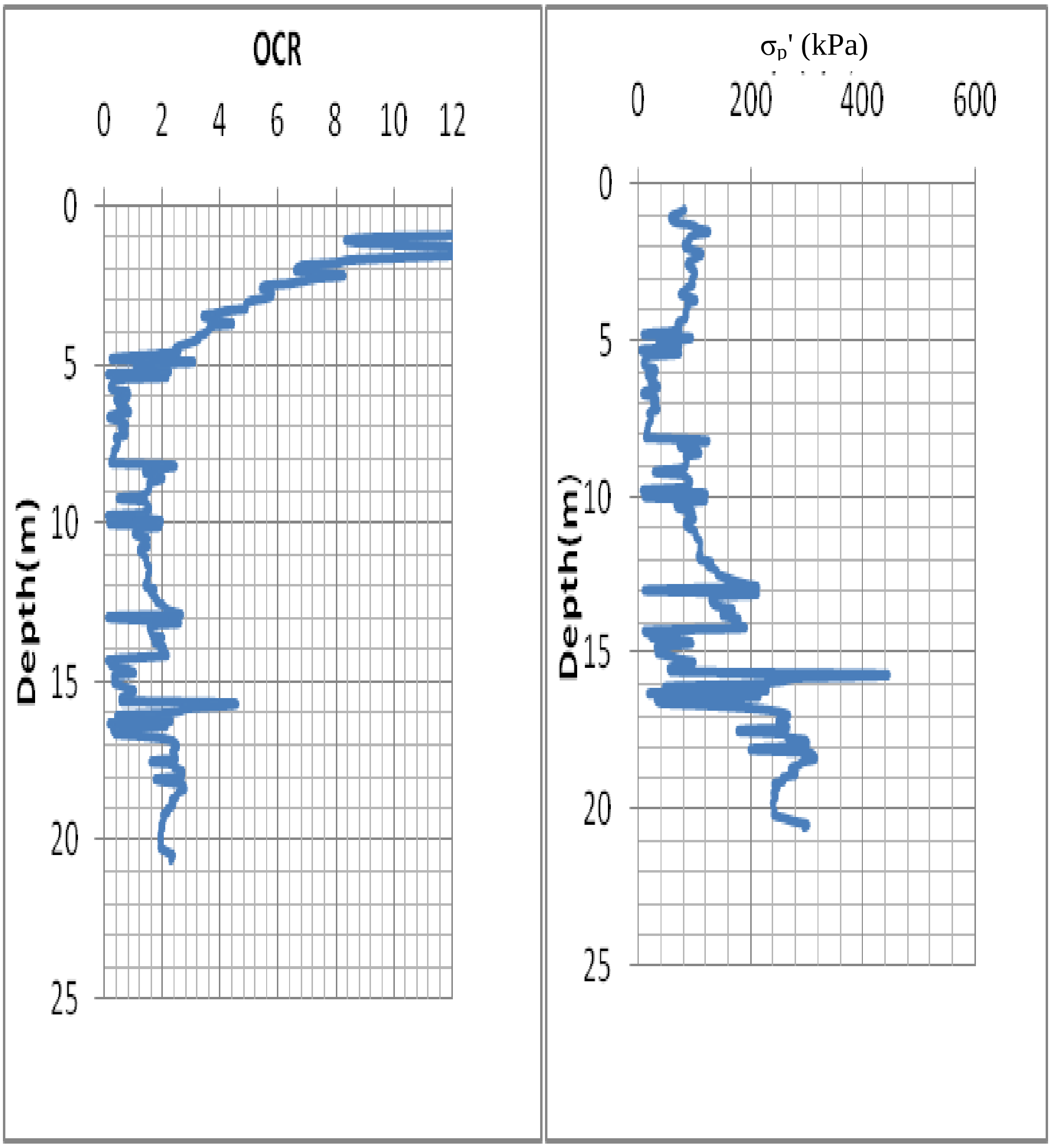


(a) (b)

Fig 5.6:a) Depth (m) vs K_0 b) Depth (m) vs ϕ^o at BADDA

5.2.4 Soil Parameter of BANASREE

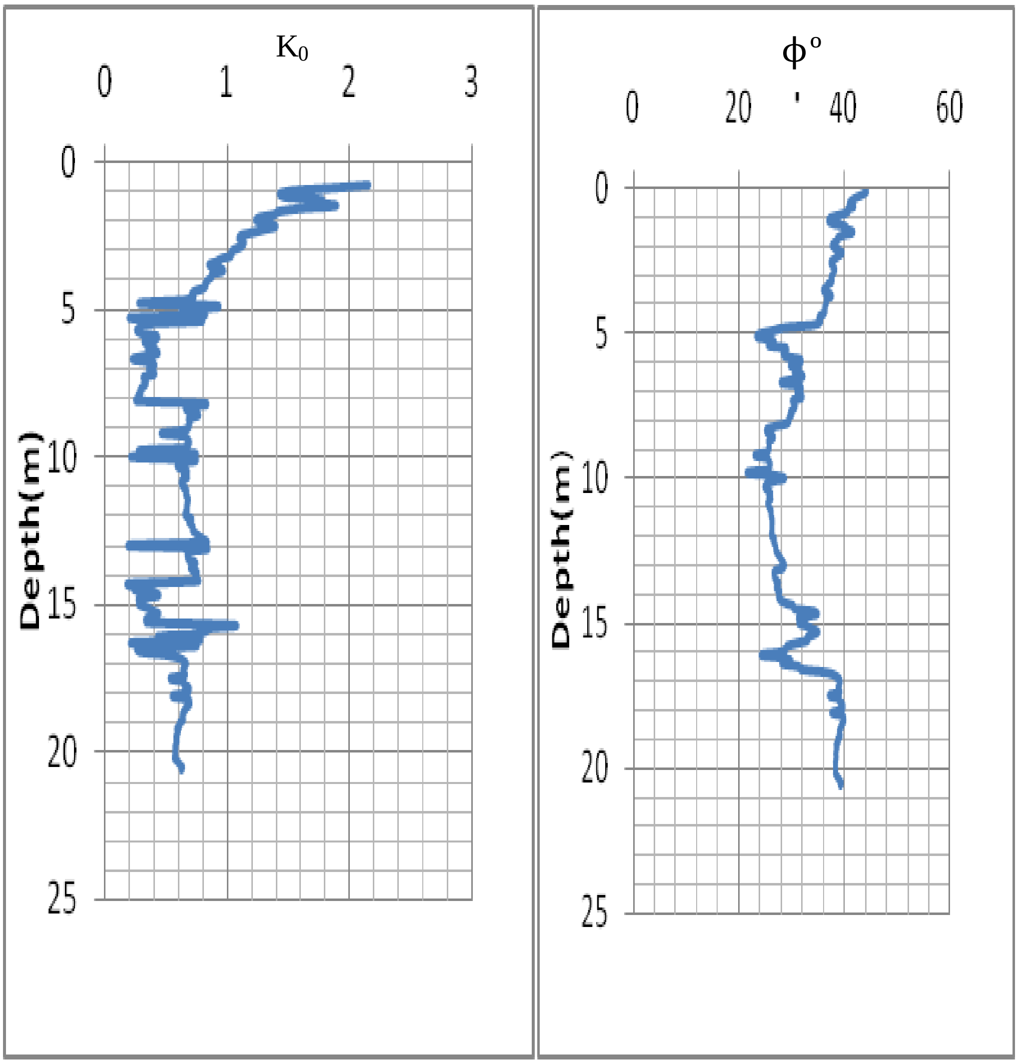
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.7 a). The maximum value of OCR is 16. The minimum value of OCR is 0.099. The average value of OCR is 2.45. Again from the equation, the σ_p' value have been found (Fig 5.7 b). Then depth vs σ_p' graph has been drawn. The maximum value of σ_p' is 438 kPa. The minimum value of σ_p' is 3.30 kPa. The average value of σ_p' is 119 kPa.



(a) (b)

Fig 5.7:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at BANASREE

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.8 a). The maximum value of K_0 is 2.1. The minimum value of K_0 is 0.18. The average value of K_0 is 0.68. Again From the equation (2.5), the ϕ value have been found. Then depth vs $\phi°$ graph has been drawn (Fig 5.8 b). The maximum value of ϕ is 44°. The minimum value of ϕ is 21°. The average value of ϕ is 33°.

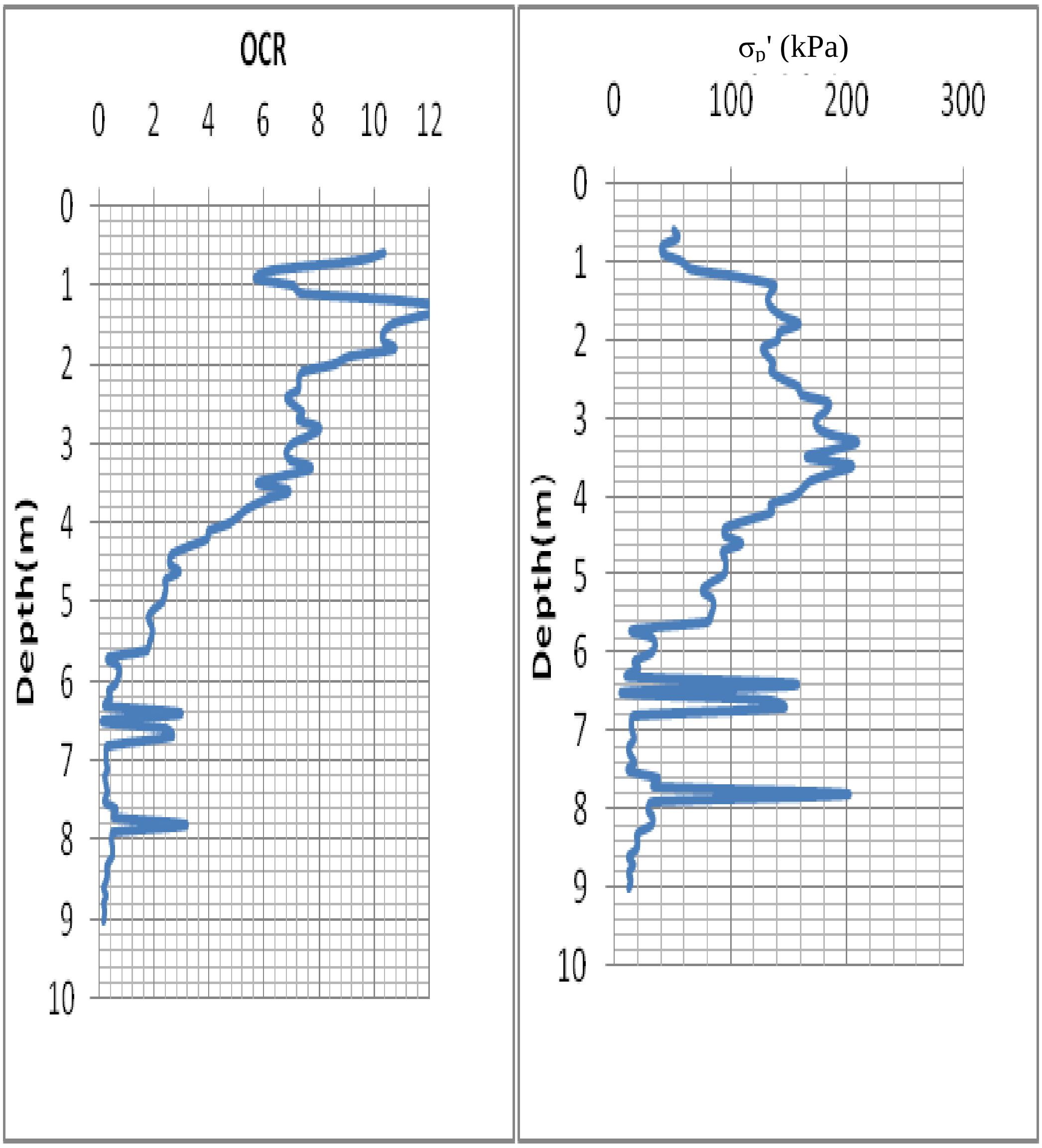


(a) (b)

Fig 5.8:a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at BANASREE

5.2.5 Soil Parameter of GABTOLI

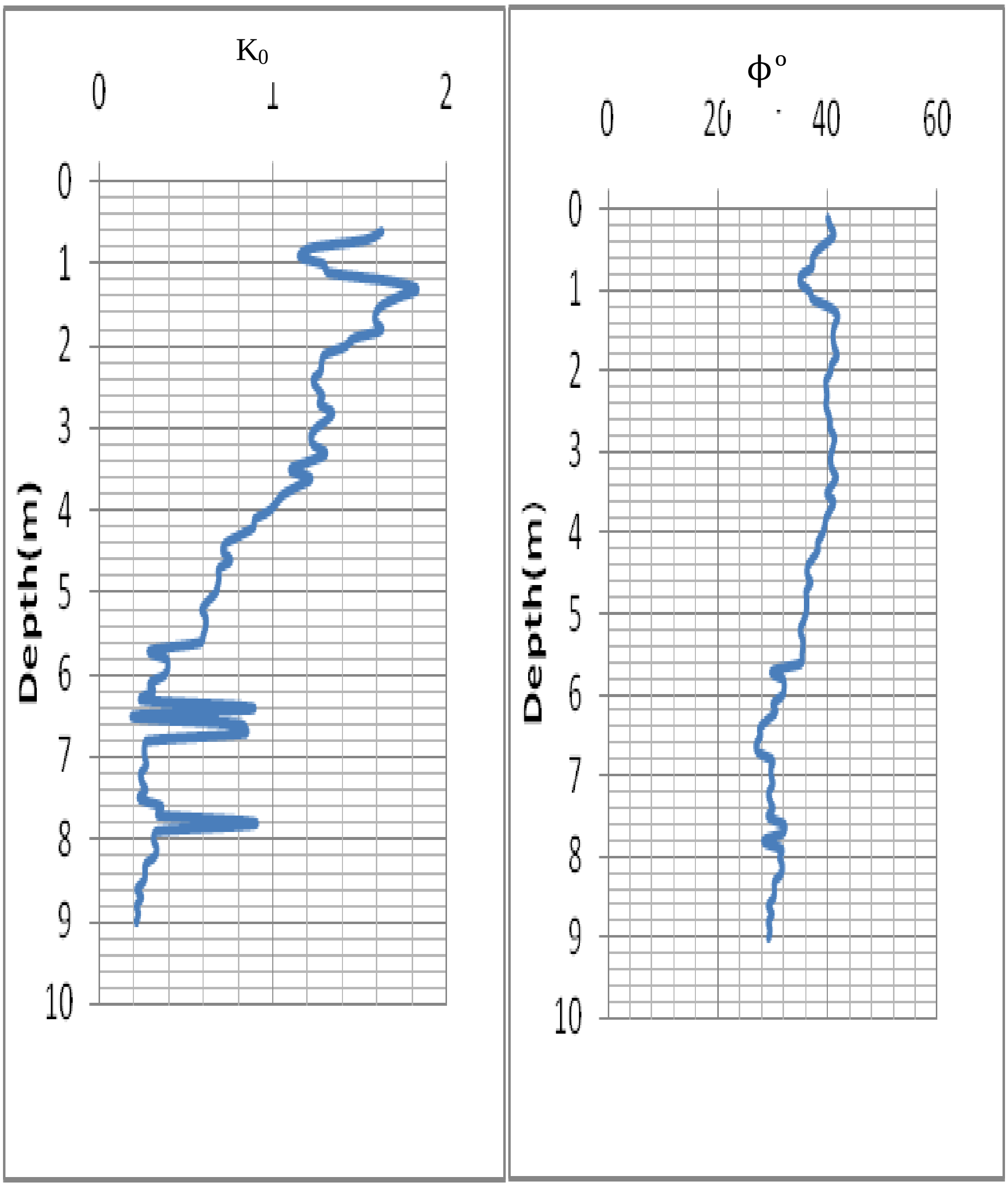
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.9 a). The maximum value of OCR is 12.8. The minimum value of OCR is 0.108. The average value of OCR is 3.97. Again From the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.9 b). The maximum value of σ_p' is 208 kPa. The minimum value of σ_p' is 5.78 kPa. The average value of σ_p' is 90 kPa .



(a) (b)

Fig 5.9:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at GABTOLI

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.10 a). The maximum value of K_0 is 1.8. The minimum value of K_0 is 0.19. The average value of K_0 is 0.82. Again from the equation, the ϕ value have been found. Then depth vs ϕ^o graph has been drawn (Fig 5.10 b). The maximum value of ϕ is 41º. The minimum value of ϕ is 26º. The average value of ϕ is 35º.

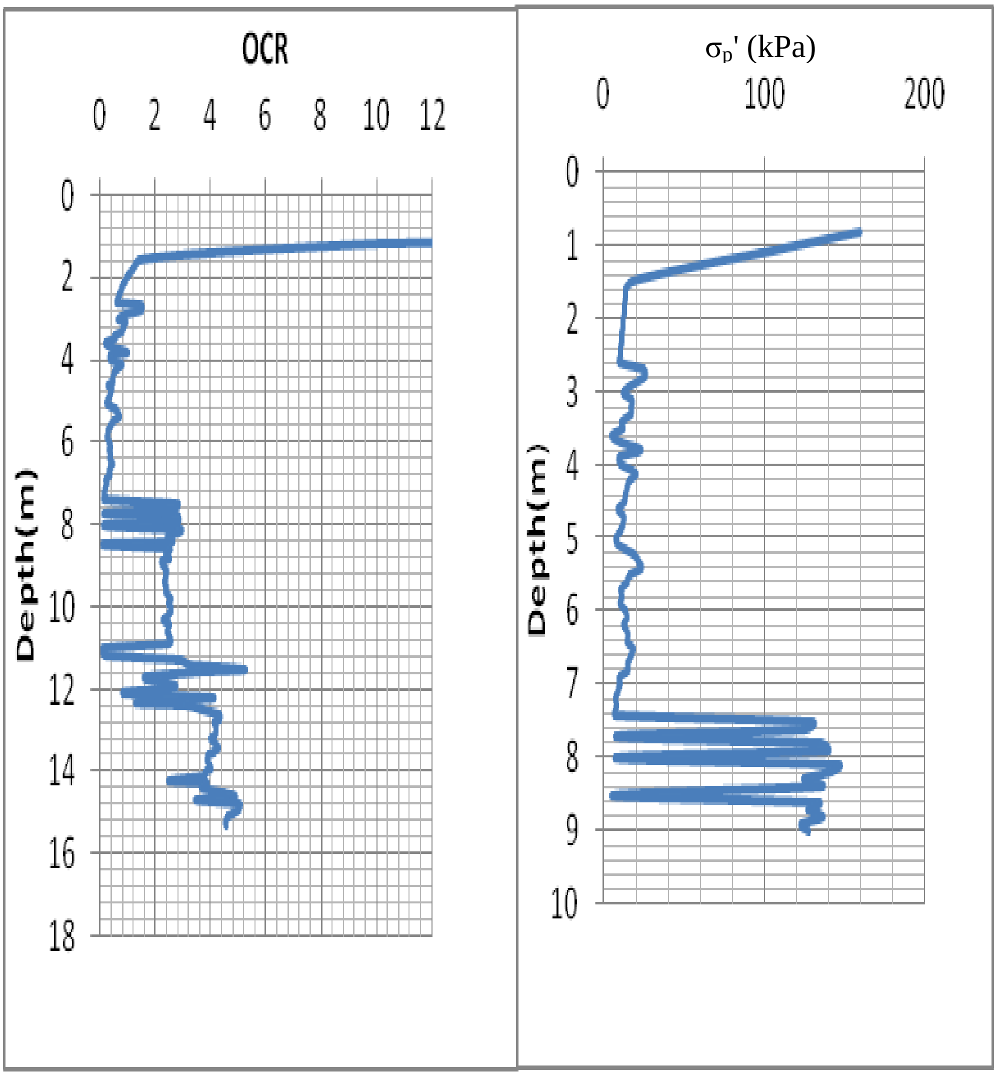


(a) (b)

Fig 5.10:a) Depth (m) vs K_0 b) Depth (m) vs ϕ^o at GABTOLI

5.2.6 Soil Parameter of KACHABAZAR

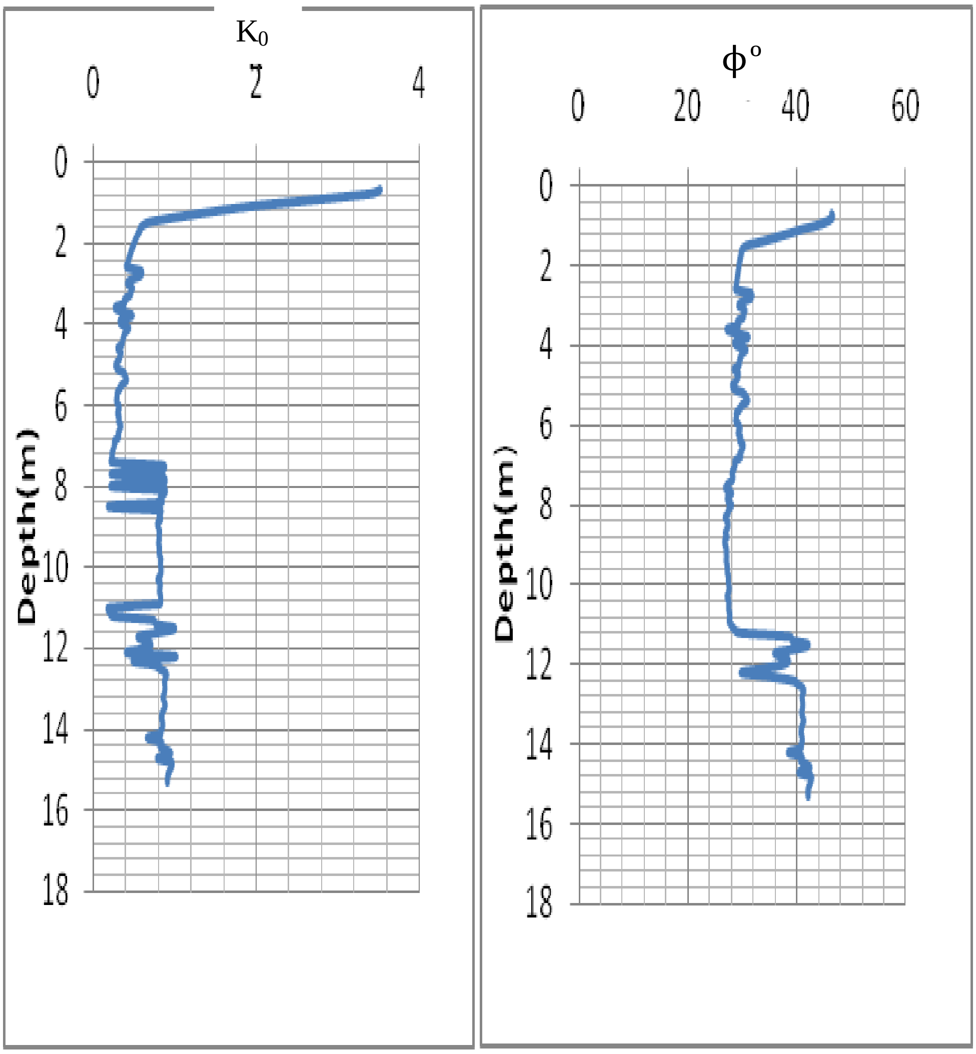
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.11 a). The maximum value of OCR is 14.39. The minimum value of OCR is 0.09. The average value of OCR is 2.18. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.11 b). The maximum value of σ_p' is 463 kPa. The minimum value of σ_p' is 5.00 kPa. The average value of σ_p' is 128 kPa.



(a) (b)

Fig 5.11:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at KACHABAZAR

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.12 a). The maximum value of K_0 is 3.52. The minimum value of K_0 is 0.17. The average value of K_0 is 0.68. Again from the equation, the ϕ value have been found. Then depth vs $\phi°$ graph has been drawn (Fig 5.12 b). The maximum value of ϕ is 46°. The minimum value of ϕ is 26°. The average value of ϕ is 32°.

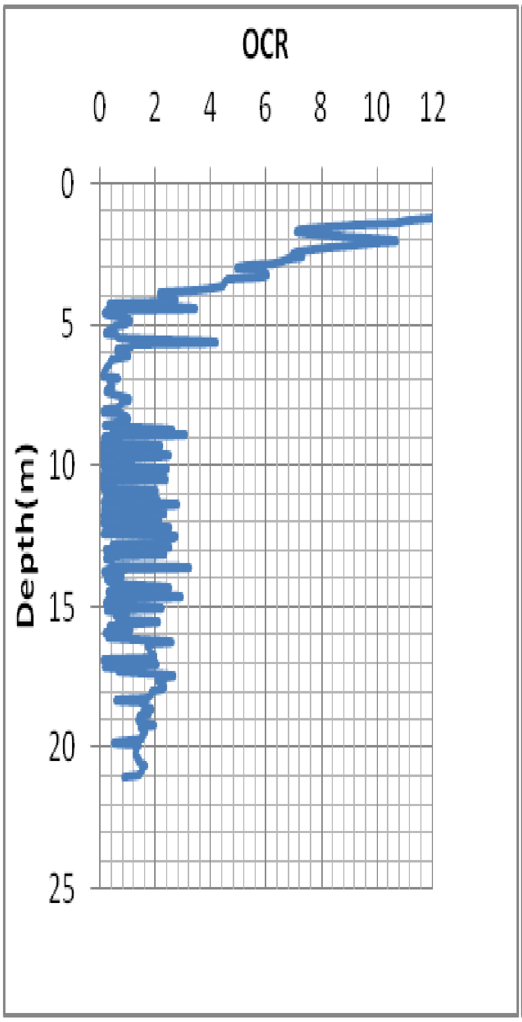


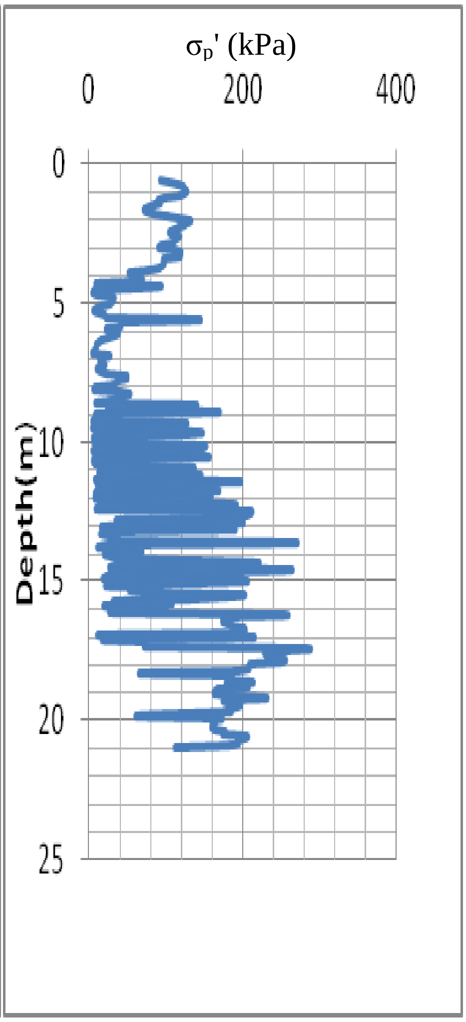
(a) (b)

Fig 5.12:a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at KACHABAZAR

5.2.7 Soil Parameter of PURBACHAL

From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.13 a). The maximum value of OCR is 27. The minimum value of OCR is 0.08. The average value of OCR is 2.56. Again from the equation (2.2 and 2.4) , the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.13 b). The maximum value of σ_p 'is 284 kPa. The minimum value of σ_p' is 4.88 kPa. The average value of σ_p' is 94 kPa.

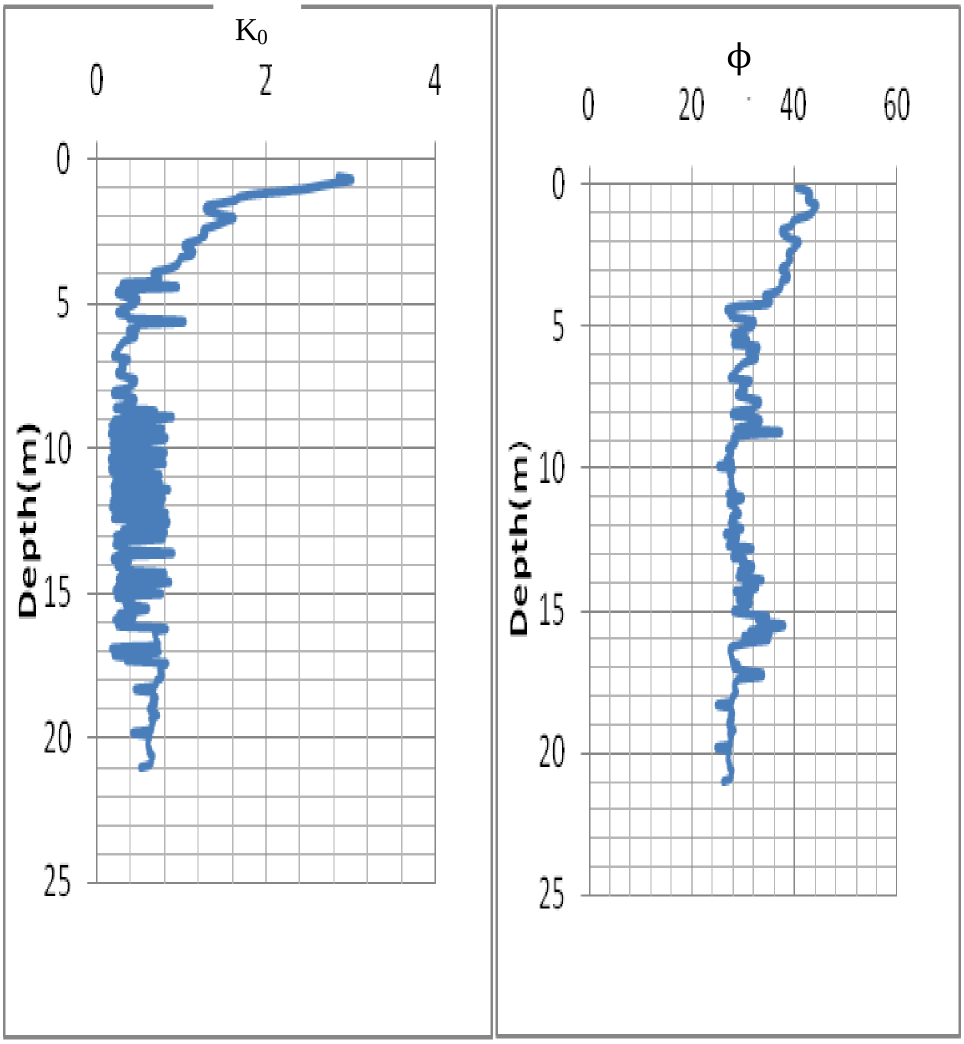




(a) (b)

Fig 5.13:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at PURBACHAL

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn. The maximum value of K_0 is 3.0 . The minimum value of K_0 is 0.17 . The average value (Fig 5.14 a) of K_0 is 0.65. Again from the equation (2.5), the ϕ value have been found. Then depth vs $\phi°$ graph has been drawn (Fig 5.14 b). The maximum value of ϕ is 44°. The minimum value of ϕ is 24°. The average value of ϕ is 31°.

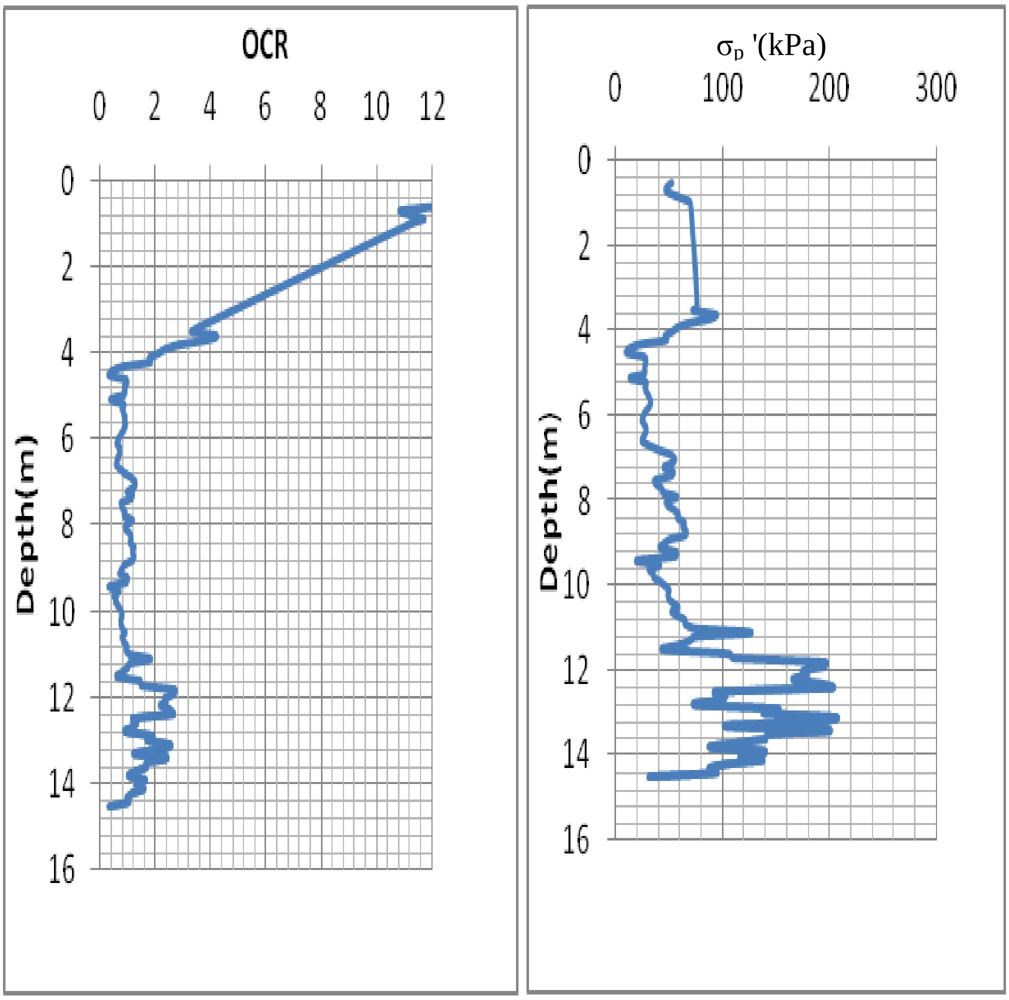


(a) (b)

Fig 5.14:a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at PURBACHAL

5.2.8 Soil Parameter of UNITED CITY

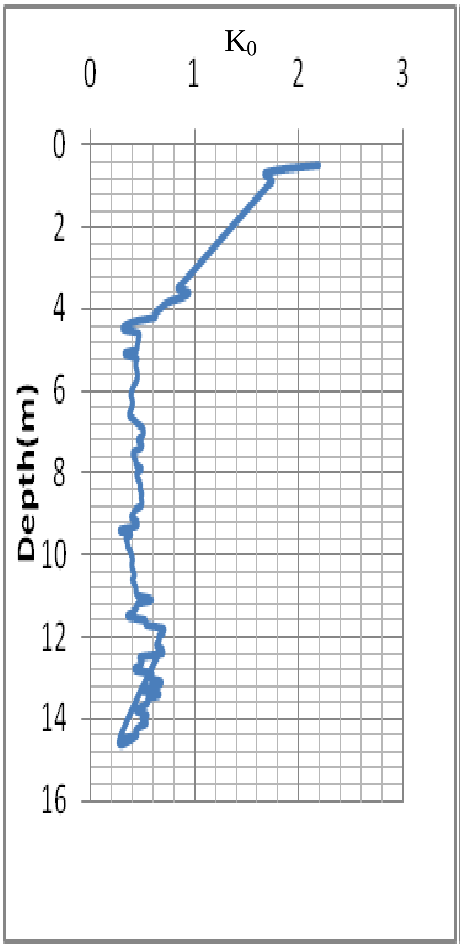
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.15 a). The maximum value of OCR is 16 . The minimum value of OCR is 0.32. The average value of OCR is 1.8. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.15 b). The maximum value of σ_p' is 206 kPa. The minimum value of σ_p' is 9.77 kPa. The average value of σ_p ' is 69 kPa.

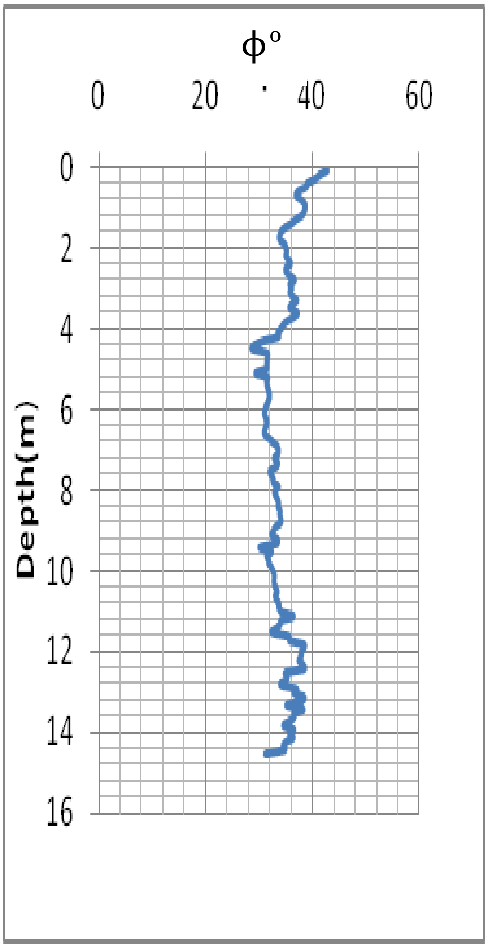


(a) (b)

Fig 5.15:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at UNITED CITY

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.16 a). The maximum value of K_0 is 2.18. The minimum value of K_0 is 0.27. The average value of K_0 is 0.55. Again from the equation (2.5), the ϕ value have been found. Then depth vs $\phi°$ graph has been drawn (Fig 5.16 b). The maximum value of ϕ is 42°. The minimum value of ϕ is 28°. The average value of ϕ is 34°.



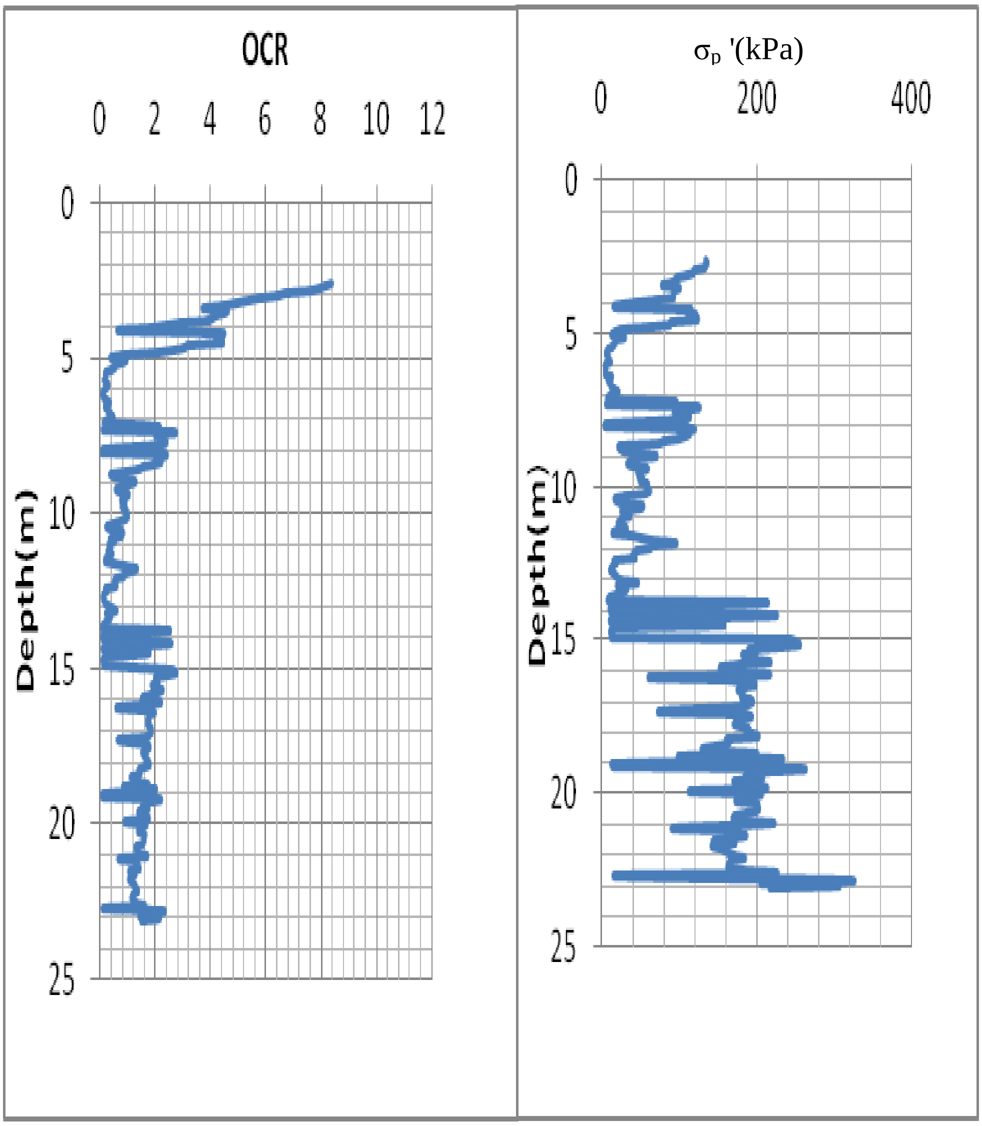


(a) (b)

Fig 5.16:a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at UNITED CITY

5.2.9 Soil Parameter of UTTARA

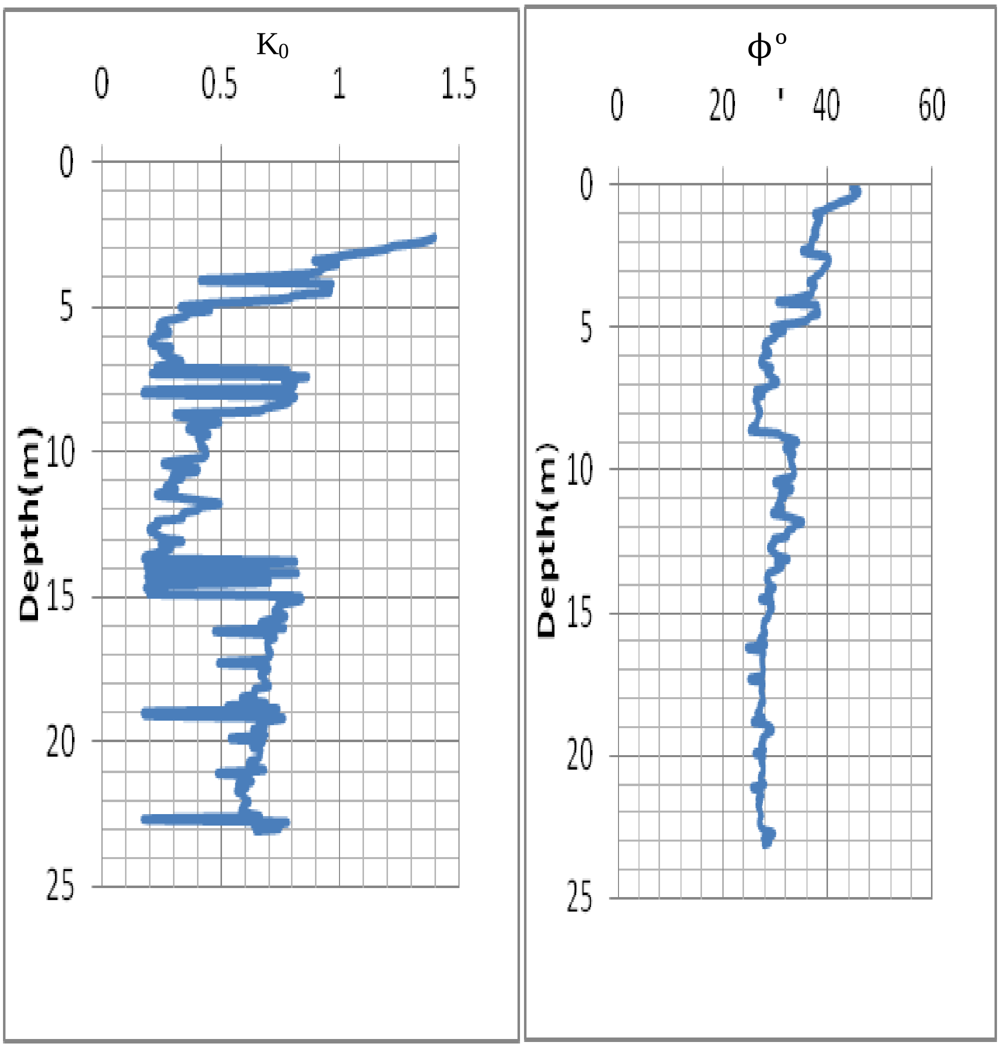
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.17 a). The maximum value of OCR is 8.3. The minimum value of OCR is 0.08. The average value of OCR is 1.32. Again from the equation (2.2 and 2.4), the σ_p' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.17 b). The maximum value of σ_p' is 321 kPa. The minimum value of σ_p' is 4.19 kPa. The average value of σ_p ' is 101 kPa.



(a) (b)

Fig 5.17:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at UTTARA

From the equation (2.6 and 2.7), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.18 a). The maximum value of K_0 is 1.39. The minimum value of K_0 is 0.17. The average value of K_0 is 0.54. Again from the equation (2.5), the ϕ value have been found. Then depth vs $\phi°$ graph has been drawn (Fig 5.18 b). The maximum value of ϕ is 45°. The minimum value of ϕ is 24°. The average value of ϕ is 30°.

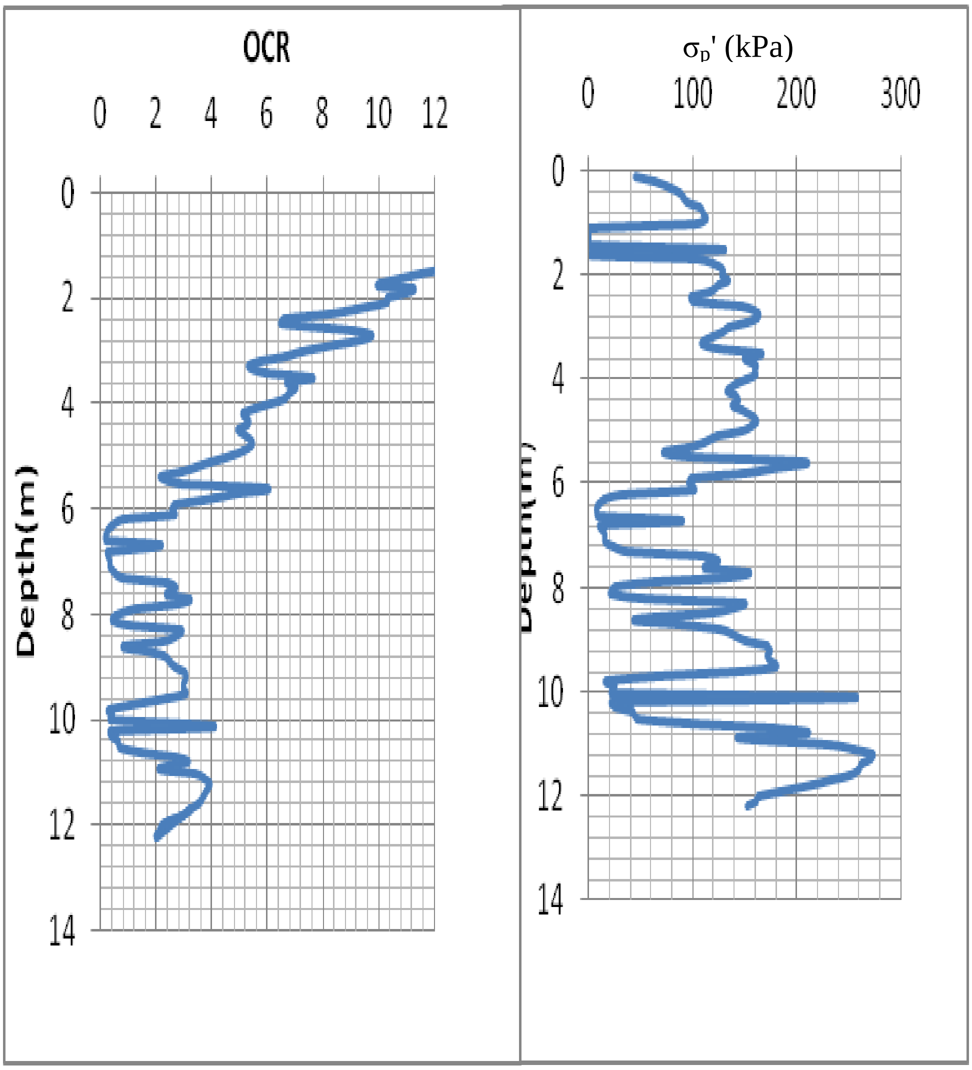


(a) (b)

Fig 5.18: a) Depth (m) vs K_0 b) Depth (m) vs $\phi°$ at UTTARA

5.2.10 Soil Parameter of KAMRANGICHAR

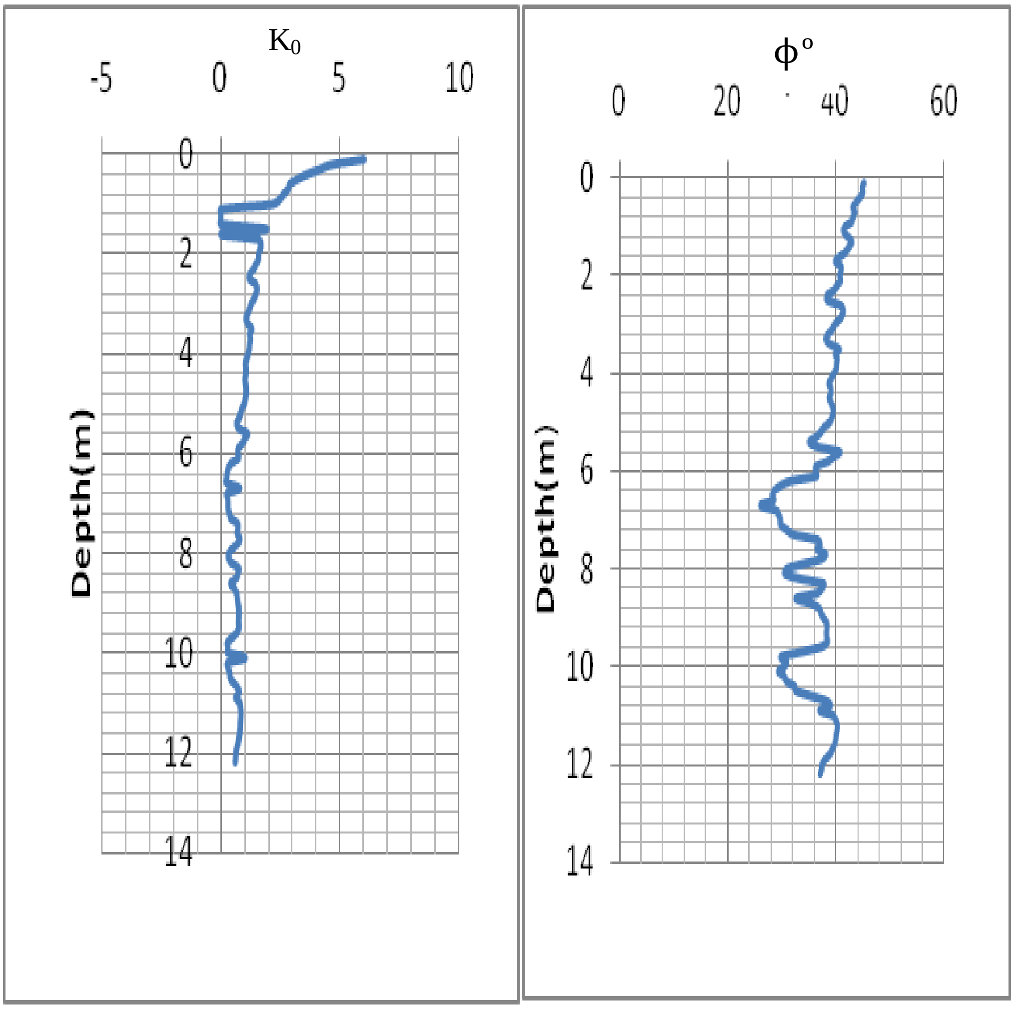
From the equation (2.1 and 2.3), the OCR value have been found. Then depth vs OCR graph has been drawn (Fig 5.19 a). The maximum value of OCR is 71. The minimum value of OCR is 0.19. The average value of OCR is 6.43. Again from the equation, the σ_p ' value have been found. Then depth vs σ_p' graph has been drawn (Fig 5.19 b). The maximum value of σ_p' is 272 kPa. The minimum value of σ_p' is 8.0 kPa. The average value of σ_p' is 132 kPa.



(a) (b)

Fig 5.19:a) Depth (m) vs OCR b) Depth (m) vs σ_p' (kPa) at KAMRANGICHAR

From the equation (2.5 and 2.6), the K_0 value have been found. Then depth vs K_0 graph has been drawn (Fig 5.20 a). The maximum value of K_0 is 6.0. The minimum value of K_0 is 0.24. The average value of K_0 is 1.0. Again from the equation, the ϕ value have been found. Then depth vs ϕ^o graph has been drawn (Fig 5.20 b). The maximum value of ϕ is 45º. The minimum value of ϕ is 26º. The average value of ϕ is 37º.



(a) (b)

Fig 5.20:a) Depth (m) vs K_0 b) Depth (m) vs ϕ^o at KAMRANGICHAR

5.3 CPT/SPT Correlations

The Standard Penetration Test (SPT) is one of the most commonly used in-situ tests for site investigation and foundation designs. Many empirical correlations have been created between the SPT N-value, and other engineering properties of soils. Although this test has disadvantages such as discrete strength measurement and dependence on operator and apparatus, but it is still the most popular and economic mean for subsurface investigation. On the other hand the Cone Penetration Test (CPT) is becoming progressively popular for its high ability to delineate stratigraphy of soil and assess soil properties rapidly and continuously. For many geotechnical projects, it is common to determine that the preliminary design based on soil parameters obtained from standard penetration tests, whereas the final design is based on cone penetration test results, or vice versa. Thus it is very valuable to find reliable CPT-SPT correlations so that available database of the test sites performances and property correlation with SPT N-value could be effectively used.A correlation between q_c vs N_{60} has been proposed by Robertson et all, 1983. The data compiled by the current research also shown similar trend as shown in Fig 5.21.

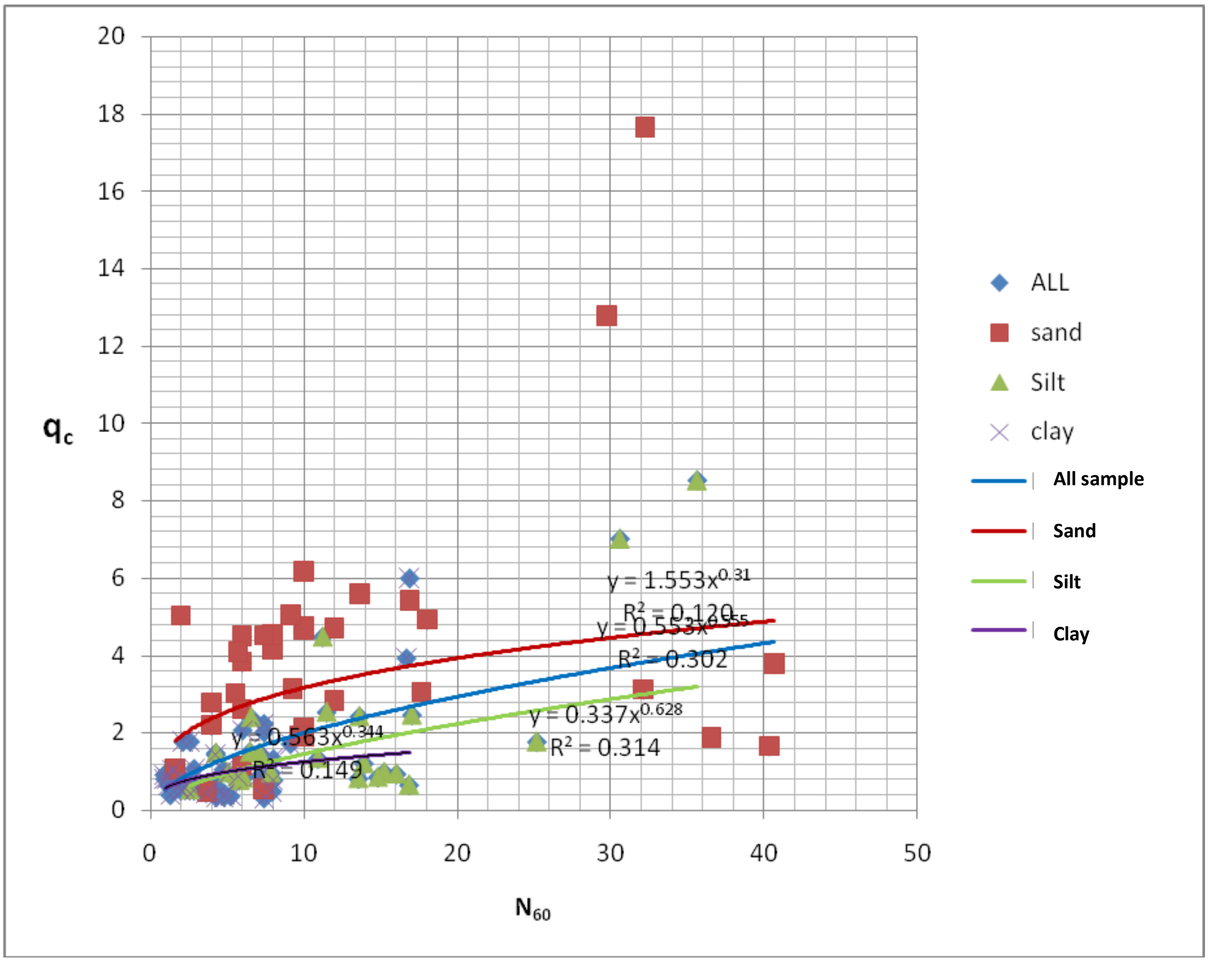


Fig 5.21: Cone resistance q_c vs N_{60} curve.

Due to lack of adequate soil grain size data, use Robertson (1990) soil classification chart to define soil behaviour type index. Here build up a corelaton between $(q_c/p_a)/N_{(60)}$ and I_c .For doing this job, firstly find out I_c value. Table 5.1 describes the procedure to find out I_c value. Then a graph have been drawn between $(q_c/p_a)/N_{(60)}$ and I_c curve (Fig 5.22). From this graph the correlation equation 5.1.

$$(q_c/p_a)/N_{(60)} = -3.095 * I_c + 9.898 \qquad 5.1$$

Table 5.1: Estimation of Soil behavior type Index for BRAMANGAON site.

Depth (m)	q (MPa)	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	4.16	8	5.2	1.443145
3	4.75	10	4.75	1.648143
4.5	3.14	9.274727	3.385545	1.743598
6	4.48	11.24501	3.98399	1.781953
7.5	2.535	11.49468	2.205369	2.071447
9	2.4	6.558222	3.659528	2.008121
10.5	0.83	6.071734	1.36699	2.51169
12	0.865	5.679587	1.522998	2.579403
13.5	1.48	4.283813	3.454866	2.381299
15	0.97	5.079977	1.909457	2.648399
16.5	0.775	5.812282	1.333383	2.771514
18	1.51	6.492309	2.325829	2.481499
19.5	1.48	7.128695	2.076116	2.506657

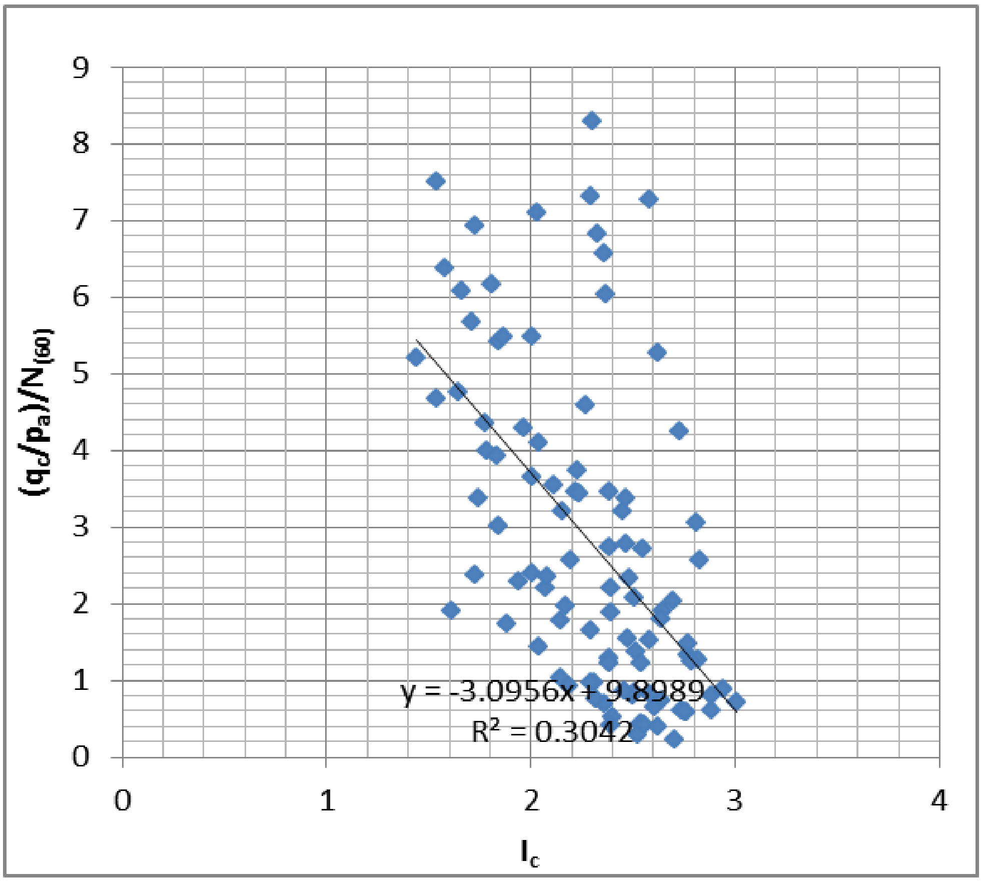


Fig 5.22 : $(q_c/p_a)/N_{(60)}$ vs I_c

A considerable number of studies have taken place over the years to quantify the relationship between SPT N values and CPT cone bearing resistance, q_c. A wide range of q_c/N ratios have been published leading to much confusion. The variations in published q_c/N ratio can be rationalized somewhat by reviewing the derived q_c/N ratios as a function of mean grain size (d_{50}). It is clear that the q_c/N ratio increases with increasing grain size. The scatter in results appears to increase with increasing grain size. This is not surprising since penetration in gravelly sand (d_{50}=1.0 mm) has been significantly influenced by the larger gravel sized particles, not to mention the variability of delivered energy in the SPT data. Also sand deposits in general have been usually stratified or non-homogeneous causing rapid variations in CPT tip resistance. A correlation between $(q_c/p_a)/N_{(60)}$ vs $d_{(50)}$ have been proposed by Roberatson and campanella (1983). The data compiled by the current research also shown similar trend as shown in Fig 5.23.

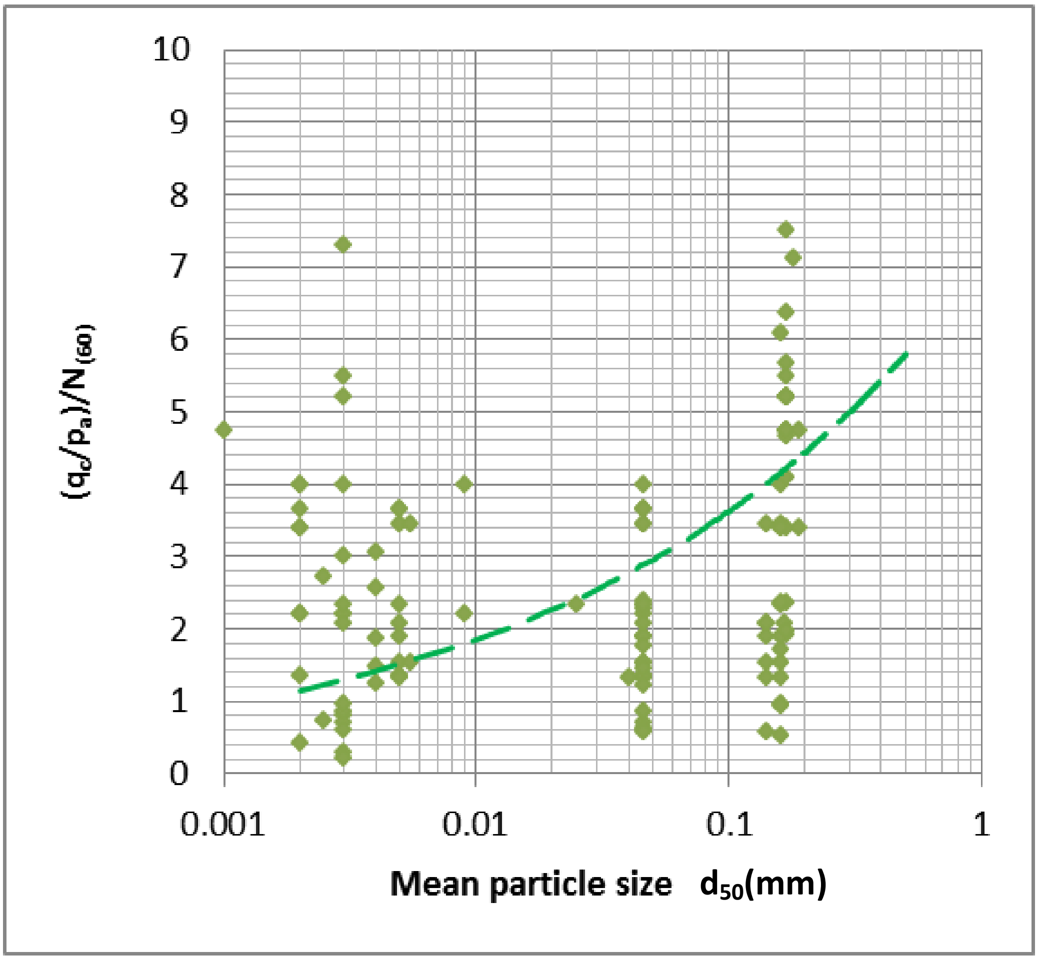


Fig 5.23 : Robertson and Campanella (1983) $(q_c/p_a)/N_{(60)}$ vs d_{50}

5.4 Summary

Based on CPT data, OCR, σ_p', K_0 and ϕ values for ten sites have been estimated and presented in this chapter. Also correlation between $(q_c/p_a)/N_{(60)}$ vs I_c and $(q_c/p_a)/N_{(60)}$ vs d_{50} has been obtained for those data. These correlations are similar to other existing correlations developed for soils deposits of other countries.

CHAPTER SIX
CONCLUSIONS AND RECOMMENDATIONS

6.1 General

The purpose of this research is to estimate the liquefaction potential of soil collected from some selected reclaimed areas of Dhaka city based on both standard penetration test (SPT) and cone penetration test (CPT). This research includes field and laboratory investigations to determine the sub-soil characteristics of such areas in order to estimate the liquefaction potential. Field investigations that includes approximately 10 (Ten) boreholes up to 20 m depth at ten locations of Dhaka city have been conducted. CPT data will be collected with cone penetration test apparatus. Different parameters like cone resistance, local friction, and friction ratio and pore water pressure have been found directly from the cone penetration test (CPT). During drilling, SPT N values have been taken at 1.5 m intervals as well as disturbed and undisturbed samples have been collected. Laboratory tests like grain size analysis have been conducted on the collected samples from various depths of each boring. Sub soil profiles of the areas have been determined based on the test results. The filling depth of the reclaimed areas varies from 3.5 to 5.0 m from EGL. The SPT N value of the filling depth varies from 1 to 8. Mean grain size and fines content of the soil in the filling depth varies from 0.10 to 0.32 mm and 6 to 48% respectively. Liquefaction potential of those ten sites have been done based on CPT using Robertson and Wride method and based on SPT using Seed et al method. In this research, the value of peak ground acceleration, a_{max} has been taken as 0.15g as Dhaka city exist in the zone 2 of seismic zonation map of Bangladesh (BNBC, 1993). Due to non-availability of attenuation law and suitable correlations between distance and ground motion characteristics for Dhaka, the design moment magnitude M_w has been taken as 7.5 for this study based on some historical earthquake data. Liquefaction as well as different correlation have been determined for Dhaka soil. This chapter presents the summary and salient conclusions derived from this study.

6.2 Liquefaction Potential of the Studied Areas

Liquefaction potential result of the ten sites slightly varies in the two method. Area wise summary of results of these ten reclaimed areas are stated below.

6.2.1 Liquefaction potential of BRAMANGAON

Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al,1983) data have been estimated.

- Liquefaction zones vary between 1.5~4.5 m according to Seed-Idriss's method.
- Liquefaction zones vary between 1.5 ~10 m and from 18~22.5 m according to Robertson and Wride's method.

From the above results, it has been seen that liquefaction potential slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~10 m and from 18~22.5 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.2 Liquefaction potential of ASHIAN CITY

Liquefaction potential based on CPT (Robertson and Wride, 1998) and SPT (Seed et al; 1983) data have been estimated.

- Liquefaction zones vary between 1.5~3 m according to Seed-Idriss's method.
- Liquefaction zones vary between 1.8~3.9 m and from 12.9~14.4 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~3.9 m and from 12.9~14.4 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.3 Liquefaction potential of BADDA

Liquefaction potential based on CPT(Robertson and Wride,1998) and SPT(Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~4.5 m according to Seed-Idriss's method.

- Liquefaction zones vary between 4.5 ~5.5 m and from 8~9.5 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~5.5 m and from 8~9.5 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.4 Liquefaction potential of BANASREE

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~4.5 m according to Seed-Idriss's method.

- Liquefaction zones vary between 0.7 ~4.8 m and 14.4~15.3 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 0.7~4.8 m and from 14.4~15.3 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.5 Liquefaction potential of GABTOLI

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~4.5 m according to Seed-Idriss's method.
- Liquefaction zones vary between 0.6 ~5.4 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 0.6~5.4 m depth if an Earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.6 Liquefaction potential of KAWRAN BAZAR

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction have been occurred at 12 m depth according to Seed-Idriss's method.
- Liquefaction zones vary between 11.4 ~12.3 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 11.4~12.3 m depth if an Earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.7 Liquefaction potential of PURBACHAL

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~6 m according to Seed-Idriss's method.

- Liquefaction zones vary between 1.5 ~5.0 m and from 15~15.6 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~6 m and from 18~22.5 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.8 Liquefaction potential of UNITED CITY

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~3 m according to Seed-Idriss's method.

- Liquefaction zones vary between 3.0~4.5 m. and 10.5~12.0 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~4.5 m and from 10.5~12 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.9 Liquefaction potential of UTTARA

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~4.5 m according to Seed-Idriss's method.
- Liquefaction zones vary between 2.7 ~4.8 m and from 8.7~12.3 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~4.8 m and from 8.7~12.3 m depth if an earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.10 Liquefaction potential of KAMRANGICHAR

Liquefaction potential based on CPT (Robertson and Wride,1998) and SPT (Seed et al;1983) data have been estimated.

- Liquefaction zones vary between 1.5~7.5 m and from 10.5~11.5 m according to Seed-Idriss's method.
- Liquefaction zones vary between 1.5 ~6 m and from 7.5~12 m according to Robertson and Wride's method.

From the above discussion, it has been seen that liquefaction potential result slightly varies in the two methods. It may be concluded that the soil may liquefy from 1.5~12 m depth if an Earthquake of magnitude 7.5 with acceleration value of 0.15g occurs.

6.2.11 Summary

In this research liquefaction potential have been evaluated using procedures based on both SPT and CPT. SPT based evaluation has been carried out by Seed-Idriss simplified procedure (1971). Cone Penetration Test (CPT) based evaluation has been carried out by the methods proposed by Robertson and Wride,1998. The results have been found different in different methods. One of the reasons of this variation may be due to process of data collection. SPT N value had been found 1.5 m interval. On the other hand CPT value were found 0.01 m interval. In fact CPT is more realiable than SPT as it is performed at each 0.01 m depth. Liquefaction zone for CPT of Bramangaon, Ashian City, Badda, Banasree, Gabtoli, Kawran Bazar, Purbachal, United City, Uttara and Kamrangirchar are 1.5 ~10 m and 18~22.5m, 1.8~3.9 m and 12.9~14.4 m, 4.5 ~5.5 m and 8~9.5m, 0.7 ~4.8 m and 14.4~15.3 m, 0.6 ~5.4 m, 11.4 ~12.3 m, 1.5 ~5.0 m and 15~15.6 m, 3.0~4.5 m. and 10.5~12.0 m, 2.7 ~4.8 m and 8.7~12.3 m, and 1.5 ~6 m and 7.5~12 m respectively.

6.3 C**orrelation between** C**PT and SPT N value and others parameters for DHAKA**

In this research cone resistance and friction of soil up to about 20 m depth have been determined by CPT test in ten selected areas of Dhaka city. These cone resistances and frictions data have been obtained at an interval of 0.01 m by CPT software. Again at the same ten locations SPT has been conducted up to 25 m depth. The data compiled by the current research for q_c and N_{60} shows similar trend as presented by Robertson et al. Correlation between the two parameters for different soil types have been shown in Table 6.1.

Table 6.1: Correlation between q_c and N_{60}.

Soil type	**Equation**
Sand	$q_c = 1.5538*N_{60}^{0.31}$ $R^2 = 0.1207$
Silt	$q_c = 0.3373*N_{60}^{0.6284}$ $R^2 = 0.3141$
Clay	$q_c = 0.5637*N_{60}^{0.3447}$ $R^2 = 0.1492$
All sample	$q_c = 0.553*N_{60}^{0.5556}$ $R^2 = 0.3023$

Robertson's soil classification chart to define soil behaviour type index has been used. A correlaton between $(q_c/p_a)/N_{(60)}$ and I_c have been developed and shown below.

$$\mathbf{(q_c/p_a)/N_{60} = -3.095 * I_c + 9.898}$$

Where, q_c = cone resistance (MPa); N_{60}= corrected N value;

I_c= soil classification index

6.4 Scopes for Future Research

The research conducted in testing program and empirical analysis has led to many questions and subsequent future research interests. The areas of future research have been listed below followed by brief comments:

a) Study may be conducted to prepare guidelines for reclamation procedure to avoid liquefaction of reclaimed areas.

b) Study may be conducted to determine the suitable ground improvement techniques for such areas.

c) It is observed that the liquefaction potential determined by various methods varies significantly. Laboratory test such as cyclic loading test or shaking table test may be conducted to determine the cyclic strength of the soils and further validating the results obtained by empirical methods in this studies.

d) Liquefaction potential analysis may be performed for soils of selected reclaimed Areas of Bangladesh based on Cone Penetration Test.

e) Study may be conducted to make a GIS Map of Bangladesh based on CPT.

f) Correlations between CPT and SPT may be developed for whole Bangladesh .

References:

Ahamed, S.(2005). Soil Characteristics and Liquefaction Potential of Selected Reclaimed Areas of Dhaka City. M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Alam, M.K, Hasan, A.K.M. S, Khan,M.R and Whithe,J.W.(1990). Geological Map of Bangladesh. Geologycal survey of Bangladesh.

Ali, M. H. (1998). Earthquake database and seismic zoning of Bangladesh, proceedings of International Symposium-Information Technology tools for Natural Disaster Risk Management, INCEDE report 11, pp. 59-73.

Andrus R.D. and Stokoe, K.H., (1997). Liquefaction resistance based on shear wave velocity, Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soil, National Center for Earthquake Engineering Research, State University of New York at Buffalo, pp.89-128.

Andrus R.D. and Stokoe, K.H., (2000). Liquefaction resistance of soils from shear wave velocity. J Geotech Geoenviron.Eng.2000; 126(11):1015-25.

Ansary, M.A. and Rashid,M.A.(2000). Generation of liquefaction potential map for Dhaka, Bangladesh, proceeding of 8th ASCE speciality conference on probabilistic Mechanics and Structural and Reliability, paper no. PMC 200-061.

BNBC (1993). Bangladesh National Building Code, HBRI-BSTI.

Castro, G. (1969). Liquefaction of sands, Harvard soil Mechanics Series, No.81, pierce Hall.

CDMP (2009). Report on time predictable fault modeling.

Cetin et al., (2004). "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential." J. Geotech. Geoenviron. Eng., 130(12), 1314–1340.

Chang, M.F. (1988). In-situ testing of residual soils in Singapore. Proceedings 2nd International Conference Geomechanics in Tropical Soils. V1 97-108, Singapore.

Chin, C., S. Duann, et al. (1988). SPT-CPT correlations for granular soils. International Symposium on Penetration Testing.

Demers, D. and Leroueil S.(2002). "Evaluation of Preconsolidation Pressure and the Overconsolidation Ratio from Piezocone Tests of Clay Deposits in Quebec," Canadian Geotechnical Journal, Vol. 39, No. 1, pp. 174–192.

Division of Mines and Geology (1997). Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117. Department of Conservation, Division of Mines and Geology, California.

Gratehev et al. (2006). Investigation the liquefaction potential of clayey soils under cyclic loading.

Hamouche, K.K., Leroueil S. and Roy M.(1995), and A.J. Lutenegger, "In-Situ Evaluation of K_0 in Eastern Canada Clays," Canadian Geotechnical Journal, Vol. 32, No. 4, pp. 677–688

Hossain, T (2009). "Estimation of Earthquake Induced Liquefaction Potential of Selected Areas of Dhaka City Based on Shear Wave Velocity", MSc. Engg. Thesis, Department of Civil Engineering, BUET,Dhaka ,Bangladesh.

Hossain, S.M.Z., Alam, M.S. and Islam, M.S., (2003). Geotechnical characterization and liquefaction potential of sub-soil of Mirpur DOHS, Undergraduate thesis, Department of Civil Engineering, Military Institute of Science and Technology, Dhaka, Bangladesh.

Idriss and Boulanger (1988). Semi-empirical procedures for evaluating the liquefaction potential of saturated cohesionless soils during earthquakes.

Ishihara,K. (1985). "Stability of natural deposits during earthquake." Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, vol. 1, San Francisco, pp.321-376.

Ishihara, K.(1993). Liquefaction and flow failure duing earthquakes, 33rd Rankine Lecture of the British Geotechnical Society, Geotechnique 43,, No. 3 The Institute of Civil Engineers, pp.349-415.

Ishihara et al.,(1978). Liquefaction susceptibility map of Downtown Tokyo, Proceedings of 2nd International Conference on microzonation, San Francisco, Vol. 2, pp.897-910.

Islam, M.R. (2005). "Seismic Loss Estimation for Sylhet City", MSc. Engg. Thesis, Department of Civil Engineering, BUET,Dhaka ,Bangladesh.

Islam, M.S. and Ahamed, S.(2005). "Liquefaction Potential of Selected Reclaimed areas of Dhaka City", Proc. Of third APM, IEB, Dhaka, Bangladesh, pp.341-349.

Japanese Road Association (1990), Specification for Highway Bridges, Part V, Seismic Design Tokyo Japan.

Jamiolkowski, M.(2003). "Soil Parameters Relevant to Bored Pile Design from Laboratory and In-Situ Tests," Deep Foundations on Bored and Auger Piles, Millpress, Rotterdam, The Netherlands, pp. 83–100.

Jamiolkowski, M., Ladd C.C., Germaine J.T., and Lancellotta R.(1985). "New Developments in Field and Lab Testing of Soils," Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, San Francisco, Calif., Aug. 12–16, pp. 57–154.

Jamiolkowski, M., Lopresti D.C.F., and Manassero M.(2001). "Evaluation of Relative Density and Shear Strength of Sands from Cone Penetration Test and Flat Dilatometer Test," Soil Behavior and Soft Ground Construction(GSP 119), American Society of Civil Engineers, Reston, Va., 2001, pp. 201–238.

Kamal and Midorikawa (2004). GIS-based geomorphological mapping using remote sensing data and supplementary geoinformation: A case study of the Dhaka city area, Bangladesh. International Journal of Applied Earth Observations and Geoinformation, 6/2, 111-125.

Kara, O. and Gunduz, Z., " Correlation between CPT and SPT in Adapzari, Turkey.

Khan, A.A.(2004). Earthquake scenario of Bangladesh in reference to Kolabunia earthquake of July 27, 2003 vulnerability assessment of Megacity, Dhaka, Bangladesh, Seminar on 2003 Rangamati Earthquake.

Khan (1988). Soils liquefaction possibilities in Bangladesh. From the historical background of earthquake 3 hazards on the Bengal Basin and its surroundings.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering, Prentice Hall, Inc., Upper Saddle River, New Jersey.

Kramer, S.L and Seed, H.B. (1998). Initiation of soil liquefaction under static loading conditions, Journal of Geotechnical Engineering. ASCE, vol. 114, No.4, pp.412-430, No.22334.

Kulhawy, F.H. and Mayne P.W.(1990). Manual on Estimating Soil Properties for Foundation Design, Report EPRI EL-6800, Electric Power Research Institute, Palo Alto, Calif., 306 pp.

Mayne, P.W.(2001). "Stress-Strain-Strength-Flow Parameters from Enhanced In-Situ Tests," Proceedings, International Conference on In-Situ Measurement of Soil Properties and Case Histories, Bali, Indonesia, pp. 27–48.

Mayne, P.W.(2005). "Integrated Ground Behavior: In-Situ and Lab Tests," Deformation Characteristics of Geo materials, Vol. 2 (Proc. Lyon, France), Taylor & Francis, London, United Kingdom, pp. 155–177.

Mayne, P.W. and Stewart H.E.(1988). "Pore Pressure Response of K_0 Consolidated Clays," Journal of Geotechnical Engineering, Vol. 114, No. 11, pp. 1340–1346.

Mayne, P.W., Kulhawy F.H., and Kay J.N.(1990). “Observations on the Development of Pore water Pressures During Piezocone Testing in Clays,” Canadian Geotechnical Journal, Vol. 27, No. 4, pp. 418–428.

Mayne, P.W. and Rix G.J.(1995). “Correlations Between Shear Wave Velocity and Cone Tip Resistance in Clays,” Soils & Foundations, Vol. 35, No. 2, pp. 107–110.

Mitchell, J.K. and Lunne T.(1978). “Cone Resistance as a Measure of Sand Strength,” Journal of Geotechnical Engineering, Vol. 104 (GT7), pp. 995–1012.

Mitchell, J.K. and. Solymar Z.V.(1984). “Time-Dependent Strength Gain in Freshly Deposited or Densified Sand,” Journal of Geotechnical Engineering, Vol. 110, No. 11, pp.1559–1576.

Mitchell and Tseng (1990). Model of cone penetration together with laboratory test data to suggest a liquefaction criteria using the CPT.

Mlynarek, Z., Welling E. and Tschuschke W.(1995). “Conductivity Piezocone Penetration Test for Evaluation of Soil Contamination,” Proceedings, International Symposium on Cone Penetration Testing, Vol. 2, Swedish Geotechnical Society Report 3:95, Linköping, Sweden, pp. 233–237.

Mohammad, L.N., Titi H.H. and Herath A.(2002), “Effect of Moisture Content and Dry Unit Weight on the Resilient Modulus of Subgrade Soils Predicted by Cone Penetration Test,” Report FHWA/LA.00/355, Louisiana Transportation Research Center, Baton Rouge, 86 pp.

National Cooperative Highway Research Program, SYNTHESIS 368, Cone penetration Testing, Chapter six, Soil Parameter Evaluation.

Poulos, S.1., Castro, G. and France, W. (1985). "Liquefaction evaluation procedure." Of Geotech Eng, ASCE, J 11(6), 772-792.

Rahman, M.G.(2004).“Seismic Damage Scenario for Dhaka City”, M.Sc. Engg. Thesis, Department of Civil Engineering, BUET,Dhaka ,Bangladesh.

Rashid, A.(2000). “Seismic Microzonation of Dhaka City based on site amplification and liquefaction”, M. Engg. Thesis, Department of Civil Engineering, BUET,Dhaka ,Bangladesh.

Robertson, P.K.(1990). “Soil Classification Using the Cone Penetration Test,” Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151–158.

Robertson, P.K.(2001). “Sixty Years of the CPT—How Far Have We Come?” Proceedings, International Conference on In-Situ Measurement of Soil Properties and Case Histories, Bali, Indonesia, May 21–24, pp. 1– 16.

Robertson, P.K. and Campanella R.G.(1983), “Interpretation of Cone Penetration Tests: Sands,” Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 719–733.

Robertson, P.K and Campanella R.G.(1985), “Liquefaction Potential of Sands Using the CPT”, Journal of Geotechnical Engineering,ASCE, Vol.111(3),pp.384-403.

Robertson and Campanella (1986), Guidelines for Use and Interpretation of the Electric Cone Penetration Test, 3rd Ed.: Hogentogler & Co., Gaithersburg, MD, 196 p.

Robertson, P.K., Campanella R.G. and Wightman A.(1983), “SPT–CPT Correlations,” Journal of the Geotechnical Engineering Division, Vol. 108, No. GT 11, pp. 1449–1459.

Robertson, P.K. and Campanella R.G.(1984), Guidelines for Use and Interpretation of the Electronic Cone Penetration Test, Hogentogler & Company, Inc., Gaithersburg, Md., 154 pp.

Robertson, P.K and Wride C.E.(1998), “Evaluating Cyclic Liquefaction Potential using the Cone Penetration Test”, Canadian Geotechnical Journal, Vol.35,pp.442-459.

Saha, A.(2005), “Development of Liquefaction Potential Map of Rangpur Town”, M.Engg. Thesis, Department of Civil Engineering, BUET, Dhaka, Bangladesh.

Salehzadeh, H., Hozouri, A., and Golestani, A. R.(2011), Correlation between Cone and Standard penetration tests.

Schmertmann, J. (1978). "Use of SPT to Measure Dynamic Soil Properties?... Yes But!" Dynamic Geotechnical Testing-STP 654: 341.

Seed, H.B. and Idriss, I.M.(1971), Simplified procedure for evaluating soil liquefaction potential, Journal of Soil Mechanics and Foundation Divisions, ASCE, vol.97, No. SM9, proceedings paper 8371, pp.1249-1473.

Seed, H.B. and Idriss, I.M.(1982), Ground motions and soil liquefaction during earthquake, Earthquake Engineering Research Institute, Oakland, California, Monograph series, pp.13.

Seed, H.B, Idriss, I.M. and Iganacio,A.(1983), “Evaluation of Liquefaction Potencial using Field Performance Data”, Journal of Geotechnical Engineering, ASCE, Vol.109(3), pp.458-482.

Shibata and Teparaksa (1988), Evaluation of liquefaction potentials of soil using cone penetration tests, Soils and Foundations. 28:49-60.

Skempton, A. (1986). "Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation." Geotechnique 36(3): 425-447

Stark and Olson (1995), An empirical method based on measured CPT tip resistances.

Suzuki, Y. et al. (1998). Correlation between SPT and seismic CPT. In Robertson&Mayne (eds), Geotechnical Site Characterization, p.1375-1380. Rotterdam: Balkema.

Tanvir (2009), Liquefaction Potential of selected areas of Dhaka city based on Shear Wave Velocity.

Tokimatsu and Uchida (1990), A simplified equation has been established to evaluate the liquefaction potential by shear-wave velocity.

Yasuda et al (1994), "A study on appropriate number of cyclic shear tests for seismic response analyses", 1st IS-Hokkaido'94, 1, pp197-202.

Yeamin et al. (2005), Correlation of shear wave velocity with liquefaction resistance based on laboratory tests.

Yoshim, M.,(1991). Liquefaction of sand deposit,2nd edition,Giho-do (in Japanese).

Youd, T.L., et al.(2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, pp. 817–833.

www.banglapedia.com

Zahid et al (2004). Groundwater Level Declining Trend in Dhaka City Aquifer. In: Water Resource Management in Dhaka City, Hassan, M.Q. (Ed.). Goethe-Institute, Dhaka, pp: 17-32.

Appendix A

Estimation of Soil behavior type Index:

Site: Ashian City

Depth(m)	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	4.535	8	5.66875	1.713664
3	1.915	10	1.915	1.608729
4.5	0.315	7.419781	0.424541	2.539043
6	0.35	4.819289	0.726248	2.635153
7.5	0.345	4.310503	0.800371	2.883228
9	0.37	5.246578	0.705222	3.007255
10.5	0.425	4.857387	0.874956	2.943073
12	0.49	7.951422	0.616242	2.887924
13.5	3.12	32.1286	0.971098	2.309399
15	1.885	36.57583	0.515368	2.403375
16.5	3.78	40.68597	0.929067	2.186528
18	2.02	7.419781	2.722452	2.548171

Site: BADDA

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	1.33	22.49002	0.591374	
3	0.76	18.17468	0.418164	
4.5	0.955	9.274727	1.02968	2.146282
6	1.82	1.60643	11.32947	2.419122
7.5	2.6	4.310503	6.031778	2.368221
9	2.105	6.558222	3.209711	2.445542
10.5	0.795	9.714774	0.818341	2.579719
12	1.005	12.49509	0.804316	2.497734
13.5	1.08	12.85144	0.840373	2.51011
15	1.135	26.41588	0.429666	2.539637
16.5	1.18	29.06141	0.406037	2.552922

Site: BANASREE

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	4.665	10	4.665	1.541732
3	2.84	12	2.366667	1.725404
4.5	2.235	7.419781	3.012218	1.838736
6	0.78	8.032149	0.971098	2.290781
7.5	1.05	1.436834	7.307731	2.292226
9	0.4	1.311644	3.049607	2.810609
10.5	0.46	3.64304	1.262682	2.817735
12	0.585	2.271835	2.575011	2.825272
13.5	0.635	4.283813	1.482324	2.772507
15	1.72	9.143959	1.881023	2.39535
16.5	1.77	25.18655	0.702756	2.362276
18	7.015	30.6066	2.291989	1.94258
19.5	8.525	35.64348	2.391742	2.002358

Site: GABTOLI

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N(60)$	I_c
1.5	4.705	12	3.920833	1.836047
3	6.165	10	6.165	1.81185
4.5	3.015	5.564836	5.417949	1.838825
6	1.33	8.032149	1.655846	2.296968
7.5	0.885	5.747338	1.539843	2.476267
9	0.97	7.869866	1.23255	2.539209

Site: KAWRAN BAZAR

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	0.495	0		2.431162
3	0.515	2	2.575	2.194253
4.5	0.635	1.840046	3.451001	2.21756
6	0.73	1.593527	4.581035	2.271573
7.5	0.515	2.850587	1.806646	2.634675
9	0.53	2.602218	2.036724	2.692832
10.5	0.655	16.8643	0.388394	2.619059
12	5.42	16.9019	3.20674	2.15488
13.5	12.77	29.74583	4.293039	1.967215
15	17.65	32.25071	5.472748	2.004338

Site: PURBACHAL

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	2.61	6	4.35	1.774256
3	2.775	4	6.9375	1.730548
4.5	0.475	3.709891	1.280361	2.381623
6	1.055	1.60643	6.567358	2.356786
7.5	1.075	2.873669	3.740862	2.229802
9	0.73	2.623289	2.782766	2.468754
10.5	0.64	1.214347	5.270323	2.61992
12	0.825	1.135917	7.262852	2.581343
13.5	1.775	2.141906	8.28701	2.298158
15	0.865	2.031991	4.256909	2.727695
16.5	0.815	0.968714	8.413219	2.748186
18	0.92	0.927473	9.91943	2.716272
19.5	0.89	7.128695	1.248475	2.789801

Site: UNITED CITY

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	2.61	6	4.35	1.774256
3	2.775	4	6.9375	1.730548
4.5	0.475	3.709891	1.280361	2.381623
6	1.055	1.60643	6.567358	2.356786
7.5	1.075	2.873669	3.740862	2.229802
9	0.73	2.623289	2.782766	2.468754
10.5	0.64	1.214347	5.270323	2.61992
12	0.825	1.135917	7.262852	2.581343
13.5	1.775	2.141906	8.28701	2.298158
15	0.865	2.031991	4.256909	2.727695
16.5	0.815	0.968714	8.413219	2.748186
18	0.92	0.927473	9.91943	2.716272
19.5	0.89	7.128695	1.248475	2.789801

Site: UTTARA

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	2.195	4	5.4875	
3	3.83	6	6.383333	1.575638
4.5	3.93	16.69451	2.354068	2.077565
6	0.5	17.67073	0.282954	2.52216
7.5	0.405	18.67885	0.216823	2.707693
9	2.465	17.05138	1.445631	2.035667
10.5	1.34	10.92912	1.226082	2.382387
12	2.425	13.63101	1.779032	2.148678
13.5	1.21	13.92239	0.869104	2.45826
15	0.98	15.23993	0.643048	2.60592
16.5	0.82	13.56199	0.604631	2.735272
18	0.855	14.83956	0.576163	2.763406
19.5	0.94	16.03956	0.586051	2.755161

Site: KAMRANGICHAR

Depth	q_c	$N_{(60)}$	$(q_c/p_a/)N_{(60)}$	I_c
1.5	5.03	2	25.15	0.862937
3	4.5	6	7.5	1.537951
4.5	4.515	7.419781	6.085085	1.663905
6	3.05	17.67073	1.726018	1.885292
7.5	4.085	5.747338	7.107639	2.027914
9	5.04	9.181511	5.489293	1.86547
10.5	1.915	9.714774	1.971224	2.166313
12	5.58	13.63101	4.093608	2.037535

Appendix B

Calculation for soil Liquefaction Potential (SPT)

Site: BRAMANGAON

Depth(m)	N_{60}	U0 (kPa)	SigV0 (kPa)	SigV0" (kPa)	r_d	CSR_{eq}	CSR_L
1.5	4	14.715	24	9.285	0.9775	0.246349	0.246349
3	5	29.43	48	18.57	0.955	0.240679	0.240679
4.5	5	44.145	72	27.855	0.9325	0.235008	0.235008
6	7	58.86	96	37.14	0.91	0.229338	0.229338
7.5	8	73.575	120	46.425	0.8875	0.223667	0.223667
9	5	88.29	144	55.71	0.865	0.217997	0.217997
10.5	5	103.005	168	64.995	0.8425	0.212326	0.212326
12	5	117.72	192	74.28	0.82	0.206656	0.206656
13.5	4	132.435	216	83.565	0.7975	0.200985	0.200985
15	5	147.15	240	92.85	0.775	0.195315	0.195315
16.5	6	161.865	264	102.135	0.7525	0.189645	0.189645
18	7	176.58	288	111.42	0.73	0.183974	0.183974
19.5	8	191.295	312	120.705	0.7075	0.178304	0.178304

C_N	N_{60}	$CSR_{7.5}$	CSR_L	FS(SPT)
2	8	0.09	0.09	0.365335
2	10	0.105	0.105	0.436267
1.854945	9.274727	0.1	0.1	0.425517
1.60643	11.24501	0.13	0.16	0.697661
1.436834	11.49468	0.12	0.165	0.737703
1.311644	6.558222	0.09	0.11	0.504595
1.214347	6.071734	0.08	0.105	0.494522
1.135917	5.679587	0.08	0.105	0.508091
1.070953	4.283813	0.06	0.09	0.447794
1.015995	5.079977	0.07	0.095	0.486394
0.968714	5.812282	0.08	0.105	0.553667
0.927473	6.492309	0.09	0.105	0.570732
0.891087	7.128695	0.09	0.11	0.616925

Site: BADDA

DEPTH	N_{60}	U0 (kPa)	SigV0 (kPa)	SigV0" (kPa)	r_d	CSR_{eq}	CSR_L
1.5	7	14.715	24	9.285	0.9775	0.246349	0.246349
3	8	29.43	48	18.57	0.955	0.240679	0.240679
4.5	5	44.145	72	27.855	0.9325	0.235008	0.235008
6	1	58.86	96	37.14	0.91	0.229338	0.229338
7.5	3	73.575	120	46.425	0.8875	0.223667	0.223667
9	5	88.29	144	55.71	0.865	0.217997	0.217997
10.5	8	103.005	168	64.995	0.8425	0.212326	0.212326
12	11	117.72	192	74.28	0.82	0.206656	0.206656
13.5	12	132.435	216	83.565	0.7975	0.200985	0.200985
15	26	147.15	240	92.85	0.775	0.195315	0.195315
16.5	30	161.865	264	102.135	0.7525	0.189645	0.189645
18	33	176.58	288	111.42	0.73	0.183974	0.183974
19.5	38	191.295	312	120.705	0.7075	0.178304	0.178304

C_N	(N_{60})	$CSR_{7.5}$	CSR_L	FS
3.21286	22.49002	0.22	0.22	0.893042
2.271835	18.17468	0.19	0.19	0.789435
1.854945	9.274727	0.09	0.09	0.382966
1.60643	1.60643	0.1	0.1	0.436038
1.436834	4.310503	0.12	0.12	0.536511
1.311644	6.558222	0.14	0.14	0.642211
1.214347	9.714774	0.19	0.19	0.894849
1.135917	12.49509	0.21	0.21	1.016182
1.070953	12.85144	0.21	0.21	1.044852
1.015995	26.41588	0.6	0.6	3.07196
0.968714	29.06141	0.6	0.6	3.163813
0.927473	30.6066	0.6	0.6	3.261328
0.891087	33.8613	0.6	0.6	3.365045

Site: BANASREE

DEPTH	N_{60}	U0 (kPa)	SigV0 (kPa)	SigV0" (kPa)	r_d	CSR_{eq}	CSR_L
1.5	5	14.715	24	9.285	0.9775	0.246349	0.246349
3	6	29.43	48	18.57	0.955	0.240679	0.240679
4.5	4	44.145	72	27.855	0.9325	0.235008	0.235008
6	5	58.86	96	37.14	0.91	0.229338	0.229338
7.5	1	73.575	120	46.425	0.8875	0.223667	0.223667
9	1	88.29	144	55.71	0.865	0.217997	0.217997
10.5	3	103.005	168	64.995	0.8425	0.212326	0.212326
12	2	117.72	192	74.28	0.82	0.206656	0.206656
13.5	4	132.435	216	83.565	0.7975	0.200985	0.200985
15	9	147.15	240	92.85	0.775	0.195315	0.195315
16.5	26	161.865	264	102.135	0.7525	0.189645	0.189645
18	33	176.58	288	111.42	0.73	0.183974	0.183974
19.5	40	191.295	312	120.705	0.7075	0.178304	0.178304

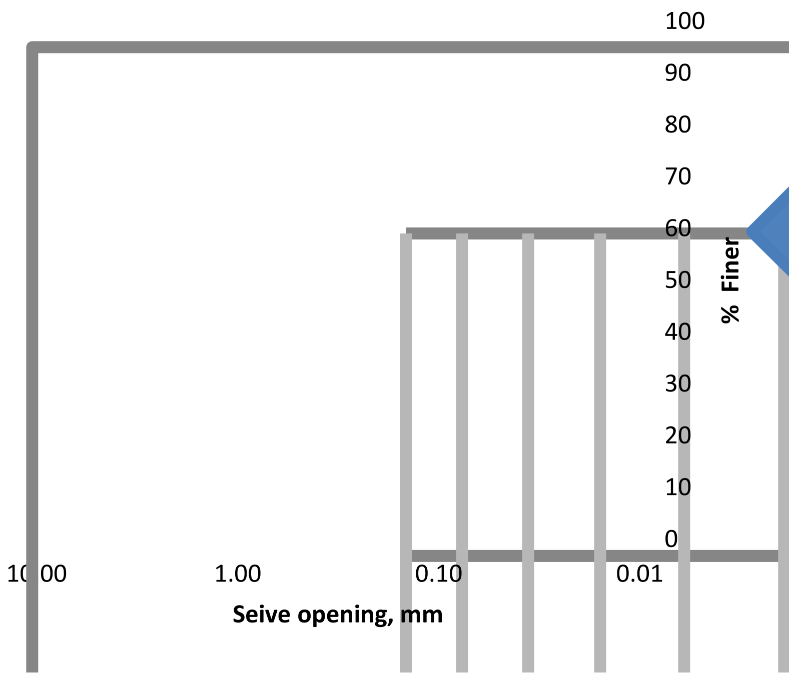
C_N	(N_{60})	$CSR_{7.5}$	CSR_L	FS
2	10	0.11	0.11	0.446521
2	12	0.12	0.12	0.49859
1.854945	7.419781	0.11	0.11	0.468069
1.60643	8.032149	0.115	0.115	0.501444
1.436834	1.436834	0.07	0.07	0.312965
1.311644	1.311644	0.07	0.07	0.321106
1.214347	3.64304	0.09	0.09	0.423876
1.135917	2.271835	0.08	0.08	0.387117
1.070953	4.283813	0.09	0.09	0.447794
1.015995	9.143959	0.18	0.18	0.921588
0.968714	25.18655	0.27	0.27	1.423716
0.927473	30.6066	0.6	0.6	3.261328
0.891087	35.64348	0.6	0.6	3.365045

Appendix C

BH-29, D3

Grain Size Analysis						
Sieve No.	Sieve opening (mm)	Wt. of Sample (gm)	Wt. of soil retained (gm)	% of soil retained	Cumulative % Retained	% Finer
4	4.75	100	0	0	0	100
8	2.38	100	0	0	0	100
16	1.2	100	0	0	0	100
30	0.599	100	0	0	0	100
50	0.297	100	11.9	11.9	11.9	88.1
100	0.15	100	47.1	47.1	59	41
200	0.075	100	9.2	9.2	68.2	31.8
	pan		31.8			

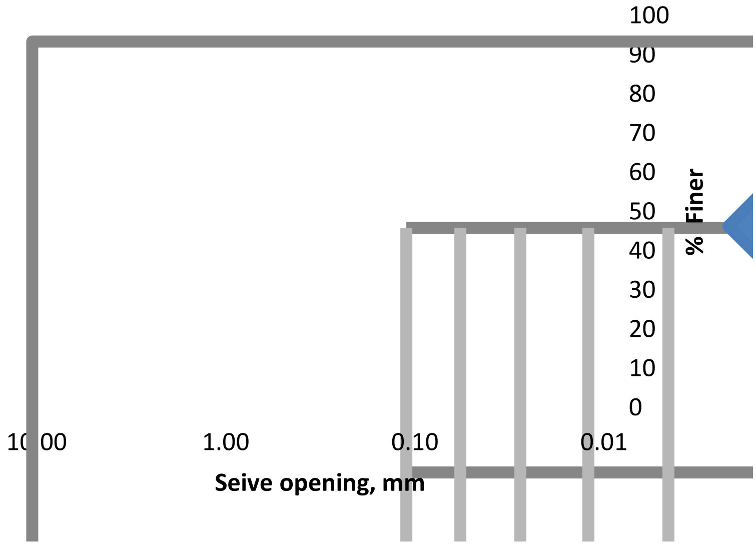
D_{60}	D_{30}	D_{10}	Cu	Cz	FM
0.200					0.71



BH-17, D3

Grain Size Analysis						
Sieve No.	Sieve opening (mm)	Wt. of Sample (gm)	Wt. of soil retained (gm)	% of soil retained	Cumulative % Retained	% Finer
4	4.75	100	0	0	0	100
8	2.38	100	0	0	0	100
16	1.2	100	0.2	0.2	0.2	99.8
30	0.599	100	0.2	0.2	0.4	99.6
50	0.297	100	12.3	12.3	12.7	87.3
100	0.15	100	52.3	52.3	65	35
200	0.075	100	12.9	12.9	77.9	22.1
	pan		22.1			

D_{60}	D_{30}	D_{10}	Cu	Cz	FM
0.220	0.120	0.000			0.78



BH-17, D6

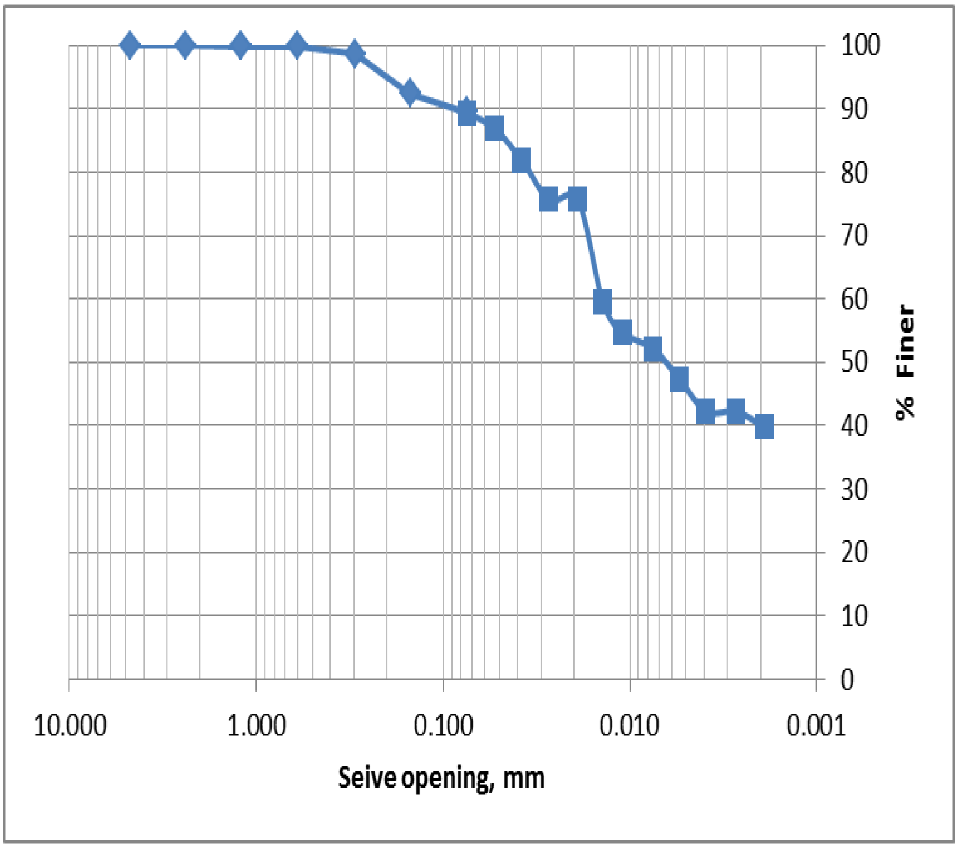
Grain Size Analysis

Sieve No.	Sieve opening (mm)	Wt. of Sample (gm)	Wt. of soil retained (gm)	% of soil retained	Cumulative % Retained	% Finer
4	4.75	100	0	0	0	100
8	2.38	100	0	0	0	100
16	1.2	100	0.1	0.1	0.1	99.9
30	0.599	100	0.1	0.1	0.2	99.8
50	0.297	100	1.1	1.1	1.3	98.7
100	0.15	100	6.3	6.3	7.6	92.4
200	0.075	100	2.9	2.9	10.5	89.5
	pan		89.5			

Hydrometer Analysis

Particle size (mm)	% finer	Corrected % Finer
0.074	71.31	89.35
0.053	69.33	86.87
0.038	65.37	81.91
0.027	60.33	75.59
0.019	60.33	75.59
0.014	47.57	59.61
0.011	43.61	54.64
0.0075	41.64	52.17
0.0054	37.68	47.21
0.0039	33.72	42.25
0.0027	33.72	42.25
0.0019	31.75	39.78

D_{60}	D_{30}	D_{10}	Cu	Cz	FM
0.015	0.000	0.000			0.09


100
90
80
70
60
50
40
30
20
10
0
% Finer
10.000
1.000
0.100
0.010
0.001
Seive opening, mm