EVALUATION OF LIQUEFACTION RESISTANCE USING DIFFERENT METHODS

A DISSERTATION

Submitted in partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Soil Dynamics)

By

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CANDIDATE'S DECLARATION

I, Mukesh Petshali, hereby declare that the work which is being presented in this dissertation entitled, **"EVALUATION OF LIQUEFACTION RESISTANCE USING DIFFERENT METHODS"**, in the partial fulfilment of the requirements for the award of the degree of **MASTER OF TECHNOLOGY** in **EARTHQUAKE ENGINEERING**, with specialization in **SOIL DYNAMICS**, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out for a period from July 2015 to May 2016, under the supervision of **Dr. B. K. Maheshwari**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee.

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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Liquefaction is serious threat both to life and property and need to be prepared for in advance. This makes liquefaction assessment compulsory. SPT has been in use traditionally but slow speed and inaccuracy are some of its serious limitations.

Keeping in mind the importance of both accuracy and speed in actual field projects, CPTu has been used for liquefaction assessment and soil profiling at four different sites in IIT Roorkee. Attempt has been made to assess the credibility of CPTu for rapid and more detailed analysis as compared to SPT which can ensure extensive use of CPTu as fair replacement of SPT for soil profiling and liquefaction analysis.

Also, in case of limited time and unavailability of actual fines content data, Robertson's formula for determination of fines content can be very useful. Fines content obtained from the above formula has been compared with the actual fines content and depth wise comparison of the results has been done graphically.

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Date: 31-05-2016 --------------------------------

-**Place:** Roorkee **(Mukesh Petshali)**

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Chapter 1

Introduction

1.1 General

Liquefaction has been a major problem associated with earthquake for many years. It is in-fact one of the deadliest after effect of earthquake in which the soil loses its shear strength due to rapid loading generated by earthquake. The shaking due to earthquake increases pore-pressure which leads to reduction in contact stresses between the soil-grains resulting in loss of shear strength of soil.

The most dangerous consequence of liquefaction is settlement. Liquefaction leads to sinking of heavy structures and floating of light structure (Bray J.D. 2013). Construction of structures over such foundation is not possible without ground improvement or considering liquefaction in design.

Therefore it is necessary to find out the liquefaction potential of the site before the construction. There are many field methods available to evaluate liquefaction potential like CPTu and SPT methods. A comparison of above methods need to be made for suitability purposes

Moreover depth wise soil profiling is required in every geotechnical project. SPT is traditionally used for the same. CPTu is fast and gives a continuous profile and hence can be used as a better alternative. Reliability of CPTu for soil profiling has been assessed by comparing the results with those obtained from SPT.

1.2 Scope of Dissertation

Liquefaction is serious threat both to life and property and need to be prepared for in advance. This makes liquefaction assessment compulsory. SPT has been in use traditionally but slow speed and inaccuracy are some of its serious limitations.

Keeping in mind the importance of both accuracy and speed in actual field projects, CPTu has been used for liquefaction assessment and soil profiling at four different sites in IIT Roorkee.

Attempt has been made to assess the credibility of CPTu for rapid and more detailed analysis as compared to SPT which can ensure extensive use of CPTu as fair replacement of SPT for soil profiling and liquefaction analysis.

Also, in case of limited time and unavailability of actual fines content data, Robertson's formula for determination of fines content can be very useful. Fines content obtained from the above formula has been compared with the actual fines content and depth wise comparison of the results has been done graphically.

1.3 Objective of the Present Study

1. To delineate soil stratigraphy of four sites Convocation Hall, Hospital ground and Earthquake Engineering department campus and Solani kunj (IIT Roorkee) using CPTu
2. Write a program in MATLAB for soil profiling and to determine the variation of liquefaction potential with depth and soil profiling using Robertson and Wride (1998) method.
3. To review different field methods available to determine the liquefaction potential of soil using SPT and CPTu and estimate the reliability of CPTu for the determination of liquefaction potential by comparing its results with those obtained by SPT.
4. To ensure the reliability of Robertson's formula for evaluation of fines percent by comparing it against the actual fine content determined by lab analysis of SPT samples.

1.4 Organization of Dissertation

The dissertation is divided into 7 chapters.

Chapter 2 is literature review. In this chapter basic information about liquefaction and different field methods of determining liquefaction potential are discussed. In chapter 3 experimental set up and testing program is discussed.

In chapter 4 soil profiling in done for four sites inside IIT Roorkee campus using CPTu method and the result are compared with actual soil profile obtained from samples procured from SPT.In chapter 5 liquefaction analysis is done for the same four sites both using CPTu and SPT methods and the results were compared for reliability of CPTu to assess liquefaction potential of soil.

In chapter 6 fines content determined from Robertson's formula is compared with actual fines content and validity of the formula in absence of actual fines content is assessed. Chapter 7 contains all major conclusions from previous chapters.

Chapter 2

Literature Review

2.1 Liquefaction

Liquefaction is the phenomena when there is loss of strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. It mainly occurs in silty and sandy soils of low plasticity.

2.2 Mechanism of Liquefaction

Occurrence of liquefaction is the result of rapid load application and break down of the loose and saturated sand because of which the loosely-packed individual soil particles tries to move into a denser configuration. However, there is not enough time for the pore-water of the soil to be squeezed out in case of earthquake. Instead, the water is trapped and prevents the soil particles from moving closer together. Thus, there is an increase in water pressure which reduces the contact forces between the individual soil particles causing softening and weakening of soil deposit. In extreme conditions, the soil particles may lose contact with each other due to the increased pore-water pressure. In such cases, the soil will have very little strength, and will behave more like a liquid than a solid - hence, the name "liquefaction".

Liquefaction may be of two types i.e. flow liquefaction and cyclic softening (Kramer, 1996). When dense sands are sheared monotonically, the soil gets compressed first, and then it gets dilated as sand particles move up and over one another. When dense saturated sands are sheared impeding the pore water drainage, their tendency of volume increase results in a decrease in pore water pressure and an increase in the effective stress and shear strength. When dense sand is subjected to cyclic small shear strains under undrained pore water conditions, excess pore water pressure may be generated in each load cycle leading to softening and the accumulation of deformations. However, at lager shear strains, increase in volume relieves the excess pore water pressure resulting in an increased shear resistance of the soil.

After initial liquefaction if large deformations are prevented because of increased undrained shear strength then it is termed," limited liquefaction" When dense saturated sands are subjected to static loading they have the tendency to progressively soften in undrained cyclic shear achieving limiting strains which is known as cyclic mobility. Cyclic mobility is different from liquefaction based on the fact that in liquefaction there is no appreciable increase in shear strength of liquefied soil however large the straining is.

Liquefaction is most commonly observed in shallow, loose, saturated cohesion-less soils subjected to strong ground motions in earthquakes. Unsaturated soils are not subject to liquefaction because volume compression does not generate excess pore water pressure. Liquefaction and large deformations are more associated with contractive soils while cyclic softening and limited deformations are more likely with expansive soils. In practice, the liquefaction potential in a given soil deposit during an earthquake is often evaluated using in-situ penetration tests and empirical procedures.

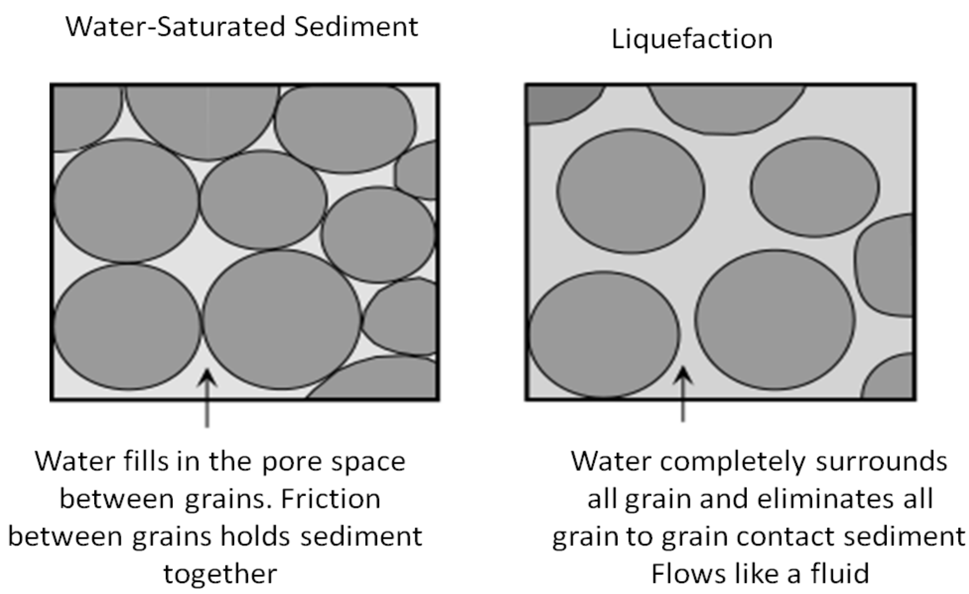


Fig. 2.1 Mechanism of liquefaction

2.3 Evaluation of Liquefaction Resistance

It is basically the quantification of resistance of the soil to liquefaction. Summarized as Cyclic Resistance Ratio (CRR).

Two widely used approaches are:

1. Lab methods
2. Field methods

Lab methods include

a. Cyclic tri axial test
b. Shake table test

Field methods are:

a. SPT
b. CPTu
c. Shear wave velocity method

SPT and CPTu are used for analysis in this thesis.

2.3.1 Overview of Liquefaction evaluation methods

The Seed-Idriss (1971) simplified procedure for evaluating liquefaction resistance basically involves the calculation of two parameters: 1) the level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio; and 2) the resistance of the soil to liquefaction, expressed as a cyclic resistance ratio.

The cyclic stress ratio, CSR, at a particular depth in a level soil deposit is calculated by expression given by Seed and Idriss (1971) discussed in chapter 5.

r_d is a function of depth and earthquake magnitude describing the ratio of cyclic stresses for a flexible soil column to the cyclic stresses for a rigid soil column and is calculated as in chapter 5.

Shear stress needs to be adjusted so that the new values correspond to equivalent uniform shear stress induced by the earthquake having a moment magnitude of 7.5.

MSF is magnitude scaling factor which is a function of magnitude of the earthquake and is evaluated as discussed in chapter 5.

2.4 Liquefaction resistance using SPT

In Figure 2, the curve for determining CRR from energy and overburden stress corrected SPT blow count, $(N1)_{60}$ by Seed et al. and modified by Youd et al.(2001) is shown. This curve is for earthquakes with moment magnitude, M_w of 7.5 and sands with fines content, FC < 5 %. To apply the curve to soils with FC > 5 %, I. Seed and Idriss (1971) developed the empirical relation for estimate CRR corresponding values of $(N_1)_{60}$ for clean sand value as discussed in chapter 5.

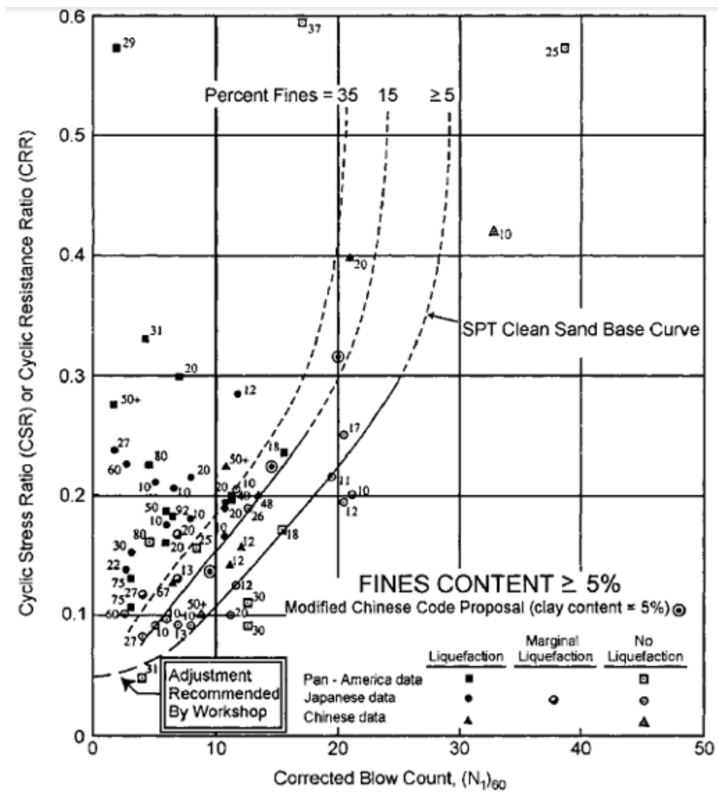


Fig. 2.2 SPT clean sand base curve for magnitude 7.5 earthquakes. Seed and Idriss (1971)

2.5 Liquefaction resistance using CPTu

Cone penetration test 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. Hence many empirical relations have been established between the SPT N- values and CPT cone bearing resistance.

Robertson and Wride (1998), Olsen and Jaung (2003) methods are there to evaluate CRR from CPT test data. Robertson and Wride method is discussed here.

Curves prepared by Robertson and Wride (1998) (fig. 2.3) for direct determination of CRR for clean sands (FC <5%) from CPT data valid for magnitude 7.5 earthquakes only, shows calculated cyclic resistance ratio plotted as a function of dimensionless, corrected, and normalized CPT resistance.

q_{c1N} from sites where surface effects of liquefaction were or were not observed following past earthquakes.

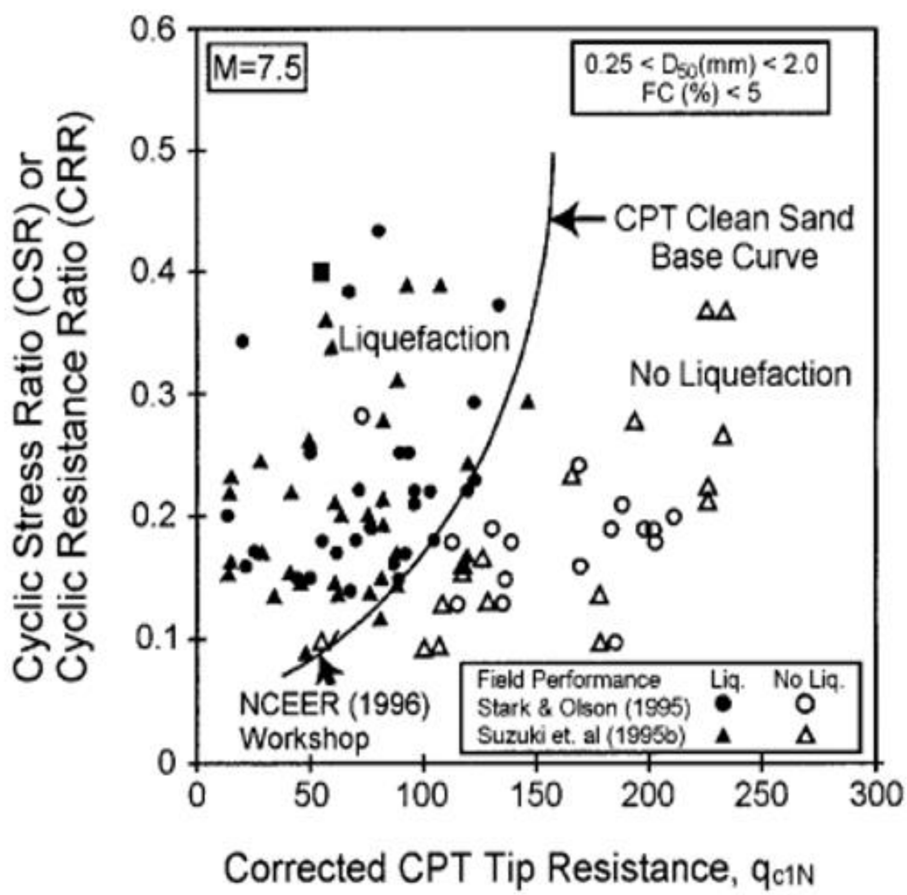


Fig. 2.3 Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data (Robertson and Wride 1998)

Empirical equations were given to approximate the clean sand base curve discussed in section 5.1.2(a)

2.5.1 Normalization of Cone Penetration Resistance

Normalization of tip resistance and friction ratio is done to take into account the effect of overburden stresses with depth. So tip resistance and friction ratio are normalized to a reference pressure of 100kPa. For shallow depths values of normalizing factor should be limited to 1.7. Equation for the same are given in chapter 4.

2.5.2 Soil Behavior Type Index

The CPT friction ratio (sleeve resistance f_s divided by cone tip resistance q_c) generally increases with increasing fines content and soil plasticity, allowing rough estimates of soil type and fines content to be determined from CPT data. Robertson and Wride (1998) constructed the chart shown in fig. 2.4.

Soil behavior type index (SBT) is used to differentiate sands and silts from clays. Soil behavior index is defined as function of normalized tip resistance and normalized friction ratio as given in section 4.2. For classification the first step is to differentiate soil types characterized as clays from soil types characterized as sands and silts. This differentiation is performed by assuming an exponent *n* of 1.0 (characteristic of clays) and calculating the dimensionless CPT tip resistance. If the recalculated I_c<2.6, the soil is classed as non-plastic and granular. This I_c is used to estimate liquefaction resistance. However, if the recalculated I_c >2.6, the soil is likely to be very silty and possibly plastic. In this instance, q_{c1N} should be recalculated from using an intermediate exponent *n* of 0.7.

The normalized penetration resistance (q_{c1N}) for silty sands is corrected to an equivalent clean sand value $(q_{c1N})_{cs}$ as discussed in section chapter 5. The correction factor depends on grain characteristics and depends on Soil Behavior Index for fine soils.

2.5.3 Apparent Fines Content

In absence of actual fines content, apparent fines content may be calculated using soil behavior index by equations as given in chapter 6.

2.5.4 Liquefaction Potential using Piezocone

It is similar to CPT except that pore pressure is also taken into account. In soft clays and silts and in overwater works q_c must be corrected for pore water pressure. It improves soil classification.

2.5.5 Soil Unit Weight

When it is not possible to collect the soil sample for unit weight determination it can be approximately calculated using the CPT data following the relationship as given in chapter 4.

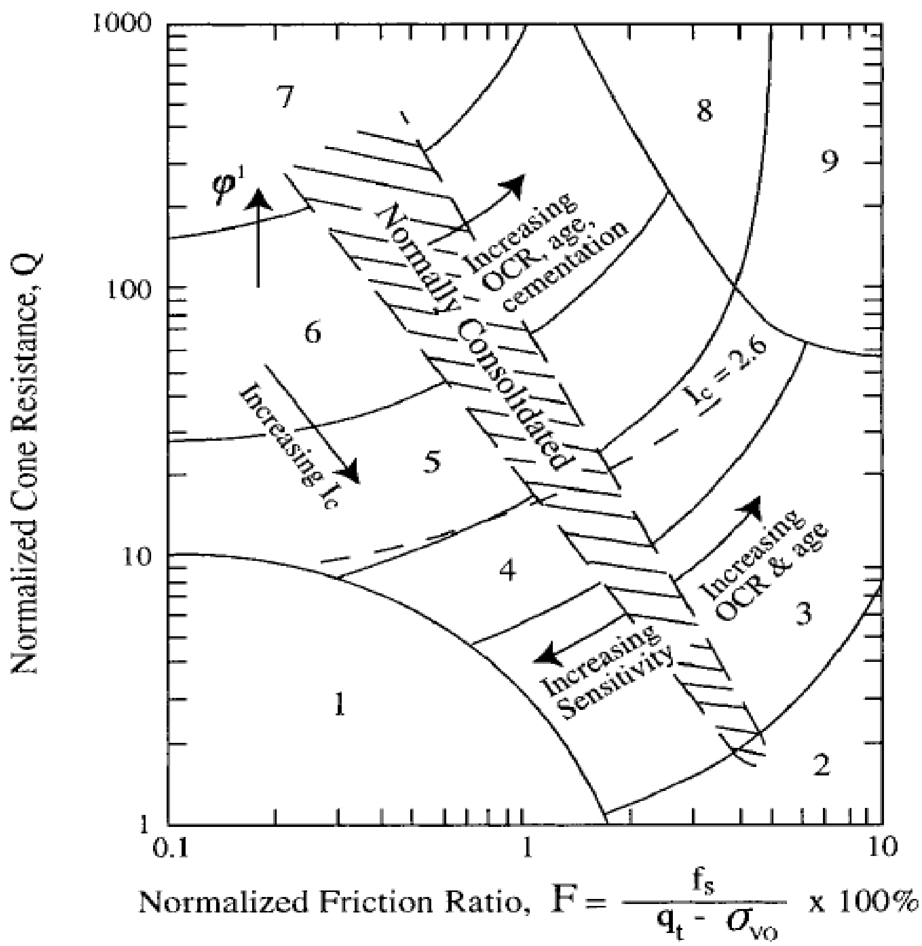


Fig. 2.4 CPT Soil Behavior Type chart proposed by Robertson (1990)

1. **Sensitive fine grained**
2. **Organic soil- peats**
3. **Silty clay to clay**
4. **Silt mixtures - clayey silt to silty clay**
5. **Sand mixtures - silty sand to sandy silt**
6. **Clean sand to silty sand**
7. **Gravelly sand to dense sand**
8. **Very stiff sand to clayey sand***
9. **Very stiff, fine grained***

***(heavily over consolidated or cemented)**

Chapter 3

Experimental Set-Up and Testing

In this chapter parts of Cone Penetration testing machine and experimental setup of the CPT machine in field is discussed. Testing program for all four sites is presented. Basic layout for the MATLAB code developed for data analysis is also discussed in this chapter.

3.1 Cone Penetration Test

In the static cone penetration test a truncated 60^o cone of 10 cm^2 base area is pushed vertically into the ground by static thrust required to cause a bearing capacity failure of the soil immediately surrounding the point where measurements are required to be made. The cone advances with a two-point system. The outer casing provides structural strength and protects the inner rod from soil friction and buckling. The protected inner rod advances the point during the thrust measurement, which is indicated by pressure gauges.

The mantle tube of diameter 36 mm and area 150 cm^2 with a uniform diameter enables the determination of total cumulative skin friction on the soil in addition to the cone resistance which are used for predetermining the load carrying capacity of pile to be casted in the site. Various parts of CPT are shown in fig. 3.1(a). Typical Set up of CPT machine in field is shown in fig. 3.1(b). Masses of different parts of CPT machine used in calculations are as follows:

Mass of cone used =1.1 kg

Mass of friction jacket used = 1.5 kg

Mass of sounding rod = 1.5 kg

3.2 CPT with Piezocone (CPTu)

Using piezocone, pore water pressure can also be measured in addition to tip resistance and skin friction.

Piezocone consists of the following parts:

1. Piezocone
2. Sounding rods
3. Connecting cables
4. Computer system and transducers

Cone is 36 mm diameter with apex angle of 60^o, area of the friction sleeve is 150 cm^2 pore-water pressure is measured by porous element just above the tip which is first saturated in glycerin solution to remove entrapped in pores fig. 3.2(a). There are three sensors attached to it to read depth, pore water pressure and tip and friction resistance respectively. The piezocone is connected to a computer directly by a cable which gives data record. An amplifier is also provided for data record. Seventeen sounding rods were used and data was taken up to 15 meters. The arrangement is shown in fig. 3.2(b).

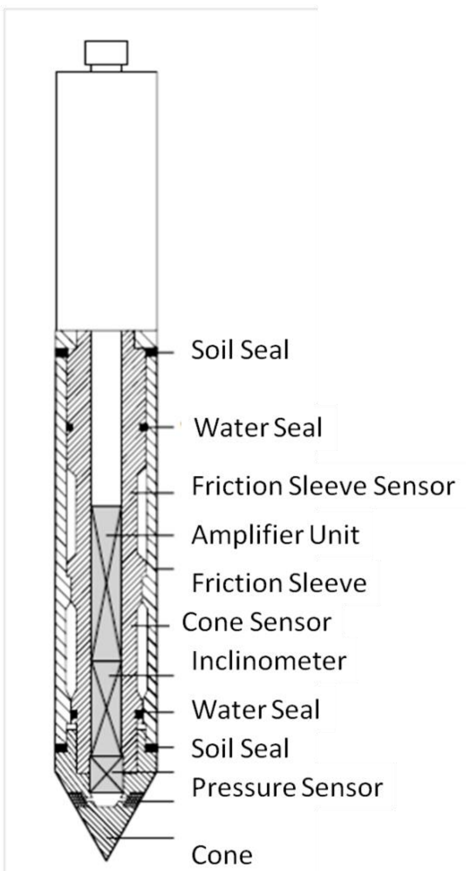


Fig 3.1(a) Parts of CPT **3.1(b) Set up of CPT in field**

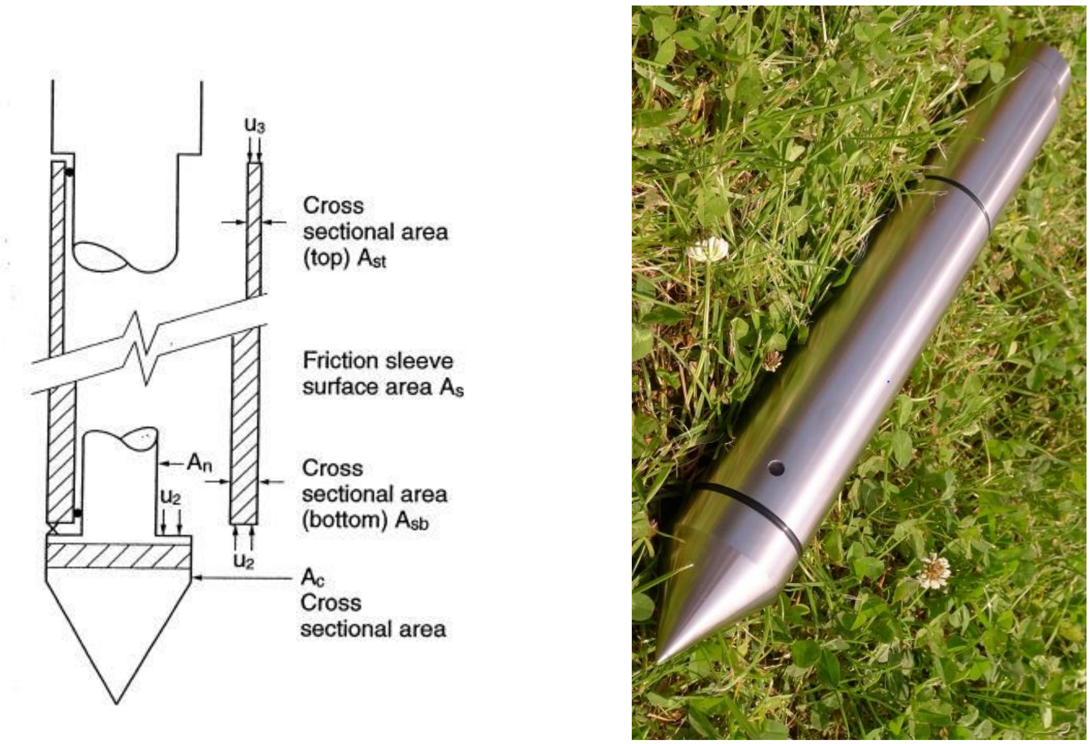


Fig. 3.2(a) Seismic piezocone

Fig. 3.2(b) sounding rods arrangement for CPTu test

3.3 Data Acquisition System

This is a PC based equipment (fig. 3.3) which displays the recorded data on the screen and saves in the hard-disk for measuring:

a. Depth (m)
b. Cone tip resistance (kN)
c. Friction resistance (kN)
d. Pore-water pressure with depth (MPa)
e. Inclination of the cone in degrees

When rods are added and the cone is pushed by hydraulic pressure the software automatically give these readings on screen of the computer and display continuous variation of the above said parameters with depth.

Fig. 3.3 Data acquisition system used in CPTu test

3.4 Testing Program

CPTu and SPT data for following four sites were obtained:

Site 1: Convocation Hall site	N 29^{o}52.046’ E 77^{o}53.535’
Site 2: Hospital ground site	N 29^{o}51.693’ E 77^{o}53.591’
Site 3: DEQ site (Earthquake Engineering Department)	N 29^{o}51.949’ E 77^{o}54.050’
Site 4: Solani kunj site (IIT Roorkee campus)	N 29^{o}52.173’ E 77^{o}54.085’

CPTu Tests were conducted at DEQ and Solani kunj sites. For the other two sites i.e. Convocation hall and Hospital ground sites the data available from previously conducted tests by Mr. P. Muley, Research Scholar at DEQ was used. A code was written in MATLAB for data analysis of CPTu data.

For all the four sites, results obtained from lab testing of samples were used. Samples for lab tests were procured by SPT tests conducted by Mr. P. Muley. Lab tests were performed to obtain index properties, grain size distribution and fines content with depth. Lab tests of samples were conducted to determine variation of soil type with depth to set up a bench mark to assess the results from CPTu. N values after corrections were used for determining the liquefaction resistance at all sites.

3.5 Development of MATLAB Code for Data Analysis

Instead of analyzing every data separately in excel, a code was written in MATLAB to perform entire analysis and comparison. Robertson and Wride (1998) method has been used finding the liquefaction resistance using CPTu data in the code. Seed and Idriss (1971) method has been used to analyze SPT data. The methods have been discussed in chapter 5. Flowchart used in the development of MATLAB code is shown in fig. 3.4.

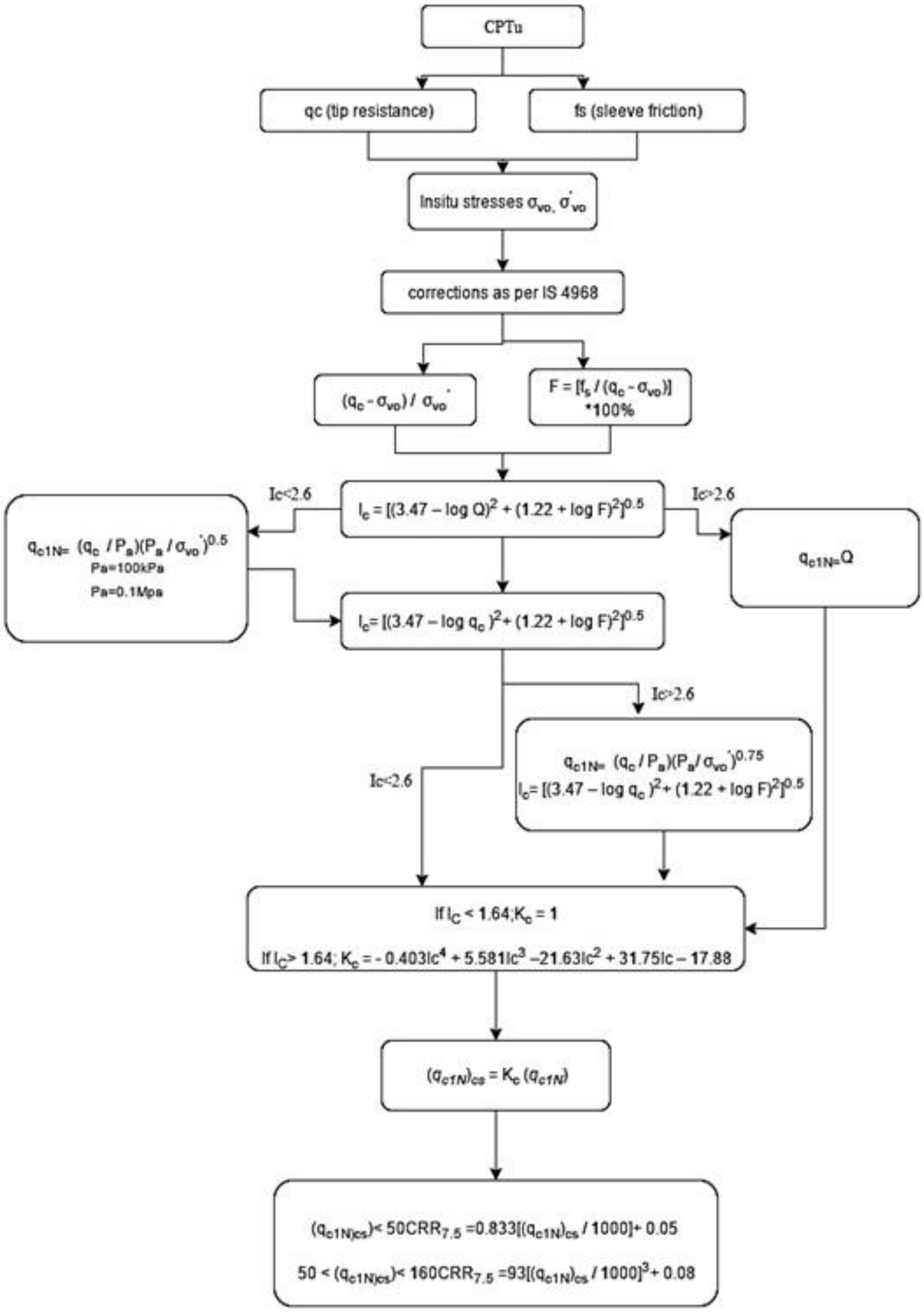


Fig. 3.4 Flowchart for determining cyclic resistance ratio using CPT data (Robertson and Wride 1998)

Chapter 4

Soil Profiling

To study liquefaction potential and for other geotechnical purposes, knowledge of depth wise soil type variation is required. Soil profiling is basically a vertical section which shows variation of soil type at all depths. For this purpose, usually SPT test is used to procure samples in field which are then tested in laboratory to determine their index properties and grain size characteristics.

For all the four sites mentioned in section 3.4, data obtained from CPTu and SPT were analyzed using different methods to get soil profiles. Soil profiles from both the approaches were compared.

The basic idea used in soil profiling was that CPTu parameters like tip resistance, sleeve friction and pore water pressure can reveal the physical properties of soil. For stratigraphic profiling from CPTu data Robertson method (1989) was used using normalized tip resistance and friction ratio as in section 4.2. An additional parameter Soil Behavior Type (SBT) Index as defined by Robertson and Wride (1998) is used to find stratigraphic variation in soil type as discussed in section 4.2.

Value of density used for calculation of overburden stress was calculated from Robertson's formula (section 4.2).

4.1 Correction and Smoothing

Corrections to the raw values obtained from CPTu were applied as per IS 4968 as follows:

Correction for tip resistance

Mass of cone (m) = 1.1 kg

Mass of each sounding rod (m_1) = 1.5 kg

Cone area at base = 10 cm^2

Correction = $\frac{m+n*m1}{10}$, where n is the number of sounding rods.

Correction for friction resistance

Mass of friction jacket = m_f kg

Area of surface friction jacket = a cm^2

Correction factor to be added to frictional resistance = $\frac{mf}{a}$ kg/cm^2

The raw data from CPTu was too inconsistent and zig-zag with depth for practical interpretation and analysis. So it was necessary to smoothen the data for more practical interpretation of the results.

Data was smoothened out for more practical representation of the actual field conditions. For smoothing, inbuilt function in MATLAB was used.

Site wise soil profiling using CPTu and its comparison with corresponding results from lab tests are as below.

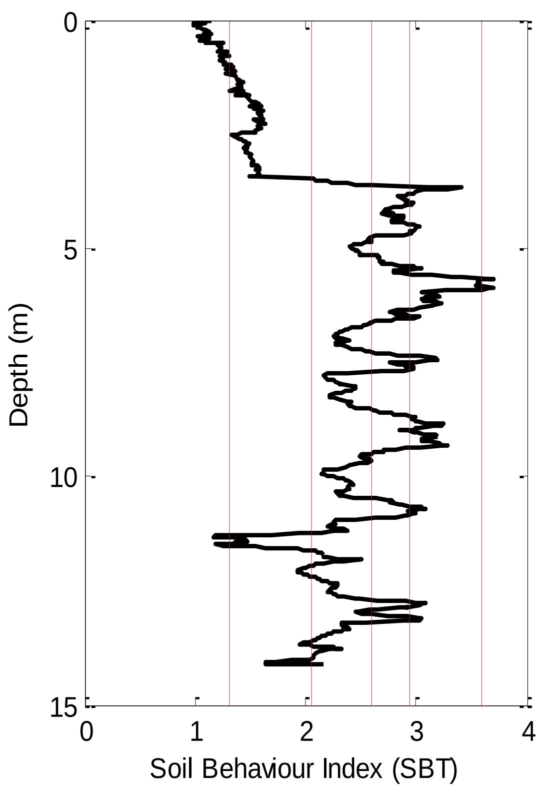


Fig. 4.2(a) Un-smoothed data

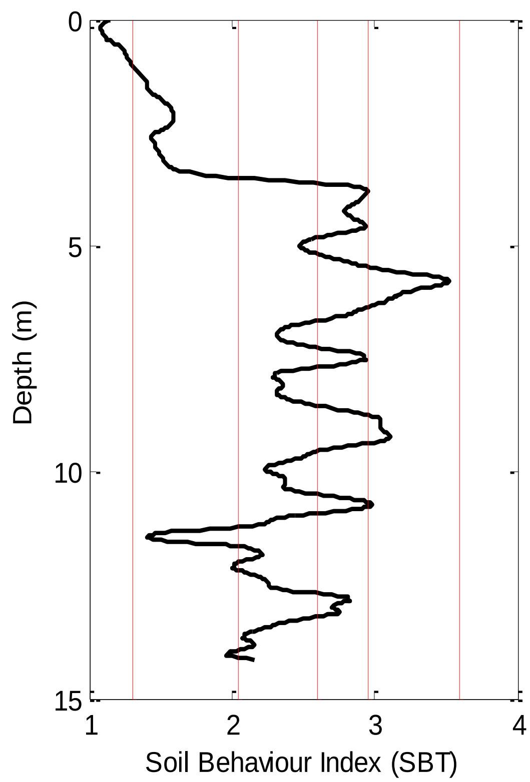


Fig. 4.2(b) Smoothed data

MATLAB Function "smooth (parameter)" used for smoothing of results was. It takes the average of n consecutive values at a time. The average value replaces the previous value of the first entry and then the averaging is done for next 2^{nd} to $n+1^{th}$ entry.

As it can be seen in above figures fig. 4.2 (a) which is un-smoothed is zig-zag and difficult to interpret against fig. 4.2 (b) which is after the data has been smoothed out shows a more realistic variation of the plotted parameter.

4.2 Soil Classification based on CPT Data

Normalization of CPTu tip resistance and friction ratio as given by Robertson (1989) was done using equations 4.1, 4.2, and 4.3.

$$Q = [(q_c - \sigma_{vo}) / P_a] [(P_a / \sigma'_{vo})^n] \tag{4.1}$$

where, P_a = 1 atm. of pressure in the same units used for σ'_{vo} ;

n = exponent that varies with soil type; and q_c = field cone penetration resistance measured at the tip. At shallow depths C_Q becomes large because of low overburden pressure; however, values >1.7 should not be applied.

$$\text{Friction ratio (F)} = [f_s / (q_c - \sigma_{vo})] \times 100\% \tag{4.2}$$

Where f_s = skin friction.

Soil behavior type index as defined by Robertson and Wride (1998) for soil classification. It basically gives an idea of transition from silts and sands to clays. It is defined as a function of normalized tip resistance and friction ratio as in the following equation.

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \tag{4.3}$$

The values of I_c gives an indication of soil type as I_c increases as clay fraction in soil increases. If $I_c<2.6$, the soil is classed as non-plastic and granular. However if $I_c>2.6$ soil is most likely to be silty and possibly plastic.

Soil behavior based on soil behavior type index as given by Robertson (1990) is given in Table 4.1.

In absence of actual field data unit weight of soil can be determined from equation 4.4 (Robertson and Cabal (2010).

$$\gamma / \gamma_w = 0.27[\log R_f] + 0.36[\log(q_t / p_a)] + 1.236 \quad (4.4)$$

q_t is corrected tip resistance

R_f is friction ratio

γ_w is unit weight if water in same units as γ

p_a is atmospheric pressure in same units as q_t

Table 4.1 Boundaries of Soil behavior type (Robertson 1990)

Soil Behaviour type Index (Ic)	Zone	Soil Behaviour Type
$I_C < 1.31$	VII	Gravelly sand to dense sand
$1.31 < I_C < 2.05$	VI	Sands: clean sand to silty sand
$2.05 < I_C < 2.60$	V	Sand mixtures: silty sand to sandy silt
$2.60 < I_C < 2.95$	IV	Silt mixtures: clayey silt to silty clay
$2.95 < I_C < 3.60$	III	Clays: silty clay to clay
$I_C > 3.60$	II	Organic soils: peats

4.3 Site 1: Convocation hall

4.3.1 Soil Profiling using CPTu

Depth wise variation of different parameters and soil type prediction has been done in Table 4.1. Based on range of values of various parameters as in table 4.1, soil at convocation hall site showed soil to exhibit sand like behavior up to 6m. Clay layer was indicated at 7m and 9m and soil was marked with high silt content up to 17m. 17m-19m soil showed dense sand like behavior.

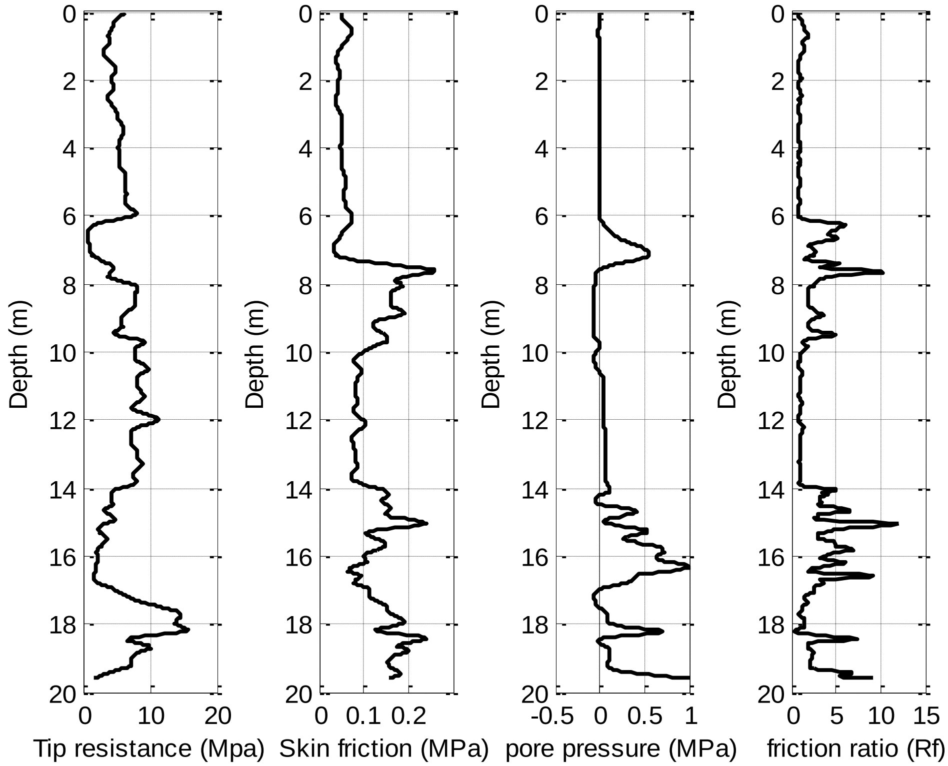
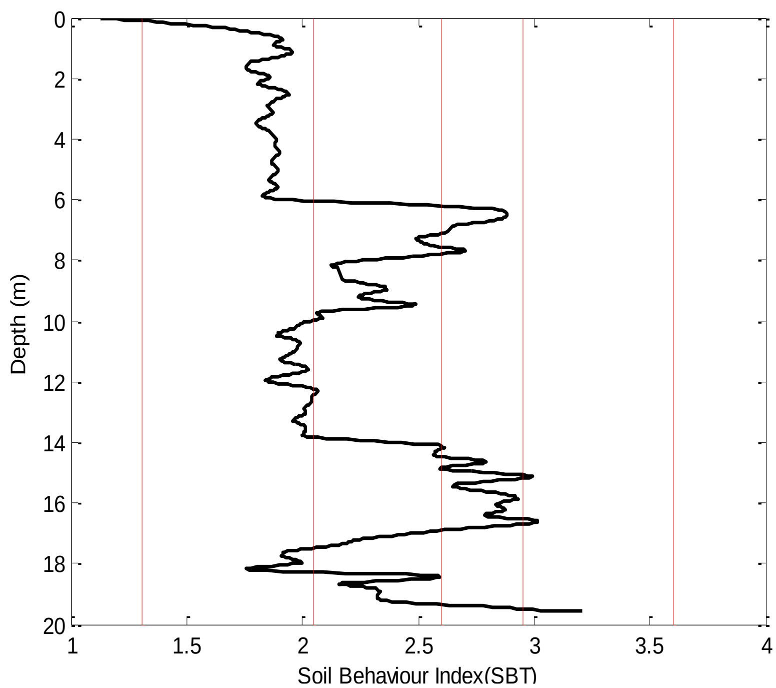


Fig. 4.3 Tip resistance, skin friction, pore-water pressure and friction ratio variation with depth (Convocation hall site)



zone VII zone VI zone V zone IV zone III zone II

Fig. 4.4 Soil Behavior Type Index (SBT) variation with depth (convocation hall site)

Grain size distribution curve of the soil was prepared as per section 2.4.1 using Normalization of tip resistance and overall soil type was assessed as explained in fig. 2.4. In Normalization, value of tip resistance is brought to a common reference overburden pressure. Normalization of tip resistance is done in order to avoid confusion arising out of different use of unit systems.

Soil behavior type classification curve (fig. 4.5) shows soil to be ranging from mostly fine grained contractive to fine grained dilative.

Table 4.2 Soil profiling using CPTu (Convocation hall site)

Depth (m)	Tip resistance (MPa)	Skin Friction (MPa)	Friction ratio (%)	Pore-water pressure (MPa)	Soil behavior Index	Remarks
0-6	2.5-7.5	0.025-0.075 (low)	1 Indicating sand	0	Zone VI indicating sand	The values of all parameters indicates sand like behavior.
6-8	Reduces to 0.5 (loose deposits)	Abruptly increases to 0.200 at 7.5m	> 4	Abruptly rises to 0.5	Zone IV-V	Clay presence at around 7 m.
8-10	7.5-5 (stiff deposit)	Reduces from 200 to 0.1	1 average Increases to 4 at about 9 m	Negative (increase in fines)	Zone V	Dilative silt-sand mixtures with clay layer at 9m
10-14	Increases to 9 (dense)	0.1	1 (reduction in fines)	0.1	Zone VI	Dense deposits of clean sand with lesser fines
14-17	Drops to 2.5	>0.1	5 average >10 at15m (cohesive fines)	>0.5	Zone IV-V Silty clay	Silty clay
17-19	15 (very high)	0.1-0.2	Around 2	Low except for 18 m where it increases to 0.5	Zone V-VI	Dense sand mixtures

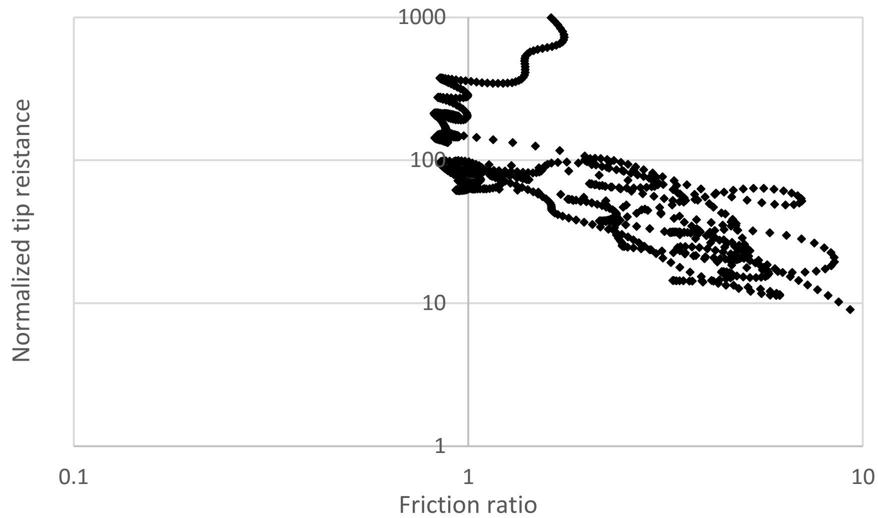


Fig. 4.5 Soil behavior type classification curve (Convocation hall site)

4.3.2 Soil Profiling using SPT samples

The depth wise lab analysis of the soil was done by Mr. P. Muley. Sieve analysis was done for various depths and fines content were determined. For depths where fine soil was present consistency limits were determined at various depths. The results are as shown:

Table 4.3(a) Index properties of sand samples (Convocation hall site)

Depth(m)	**Soil type**	**Fines Content (%)**
0.75	SP	32.97
1.50	SP	7.27
2.25	SP	5.15
3.00	SP	4.31
4.50	SP	3.11
6.00	SP	4.16
10.50	SP	5.4
12.50	SP	2.44

Table 4.3(b) Index properties of clay samples (Convocation hall site)

Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Type
7.50	36	15	21	CI
9.00	37	17	20	CI
15.00	38	19	19	CI

4.3.3 Comparison of SPT and CPTu Data

Table 4.4 Comparison of SPT and CPTu results (Convocation hall site)

Depth (m)	Lab analysis using SPT	Field analysis using CPTu	Remarks
0-6	Poorly graded sand	Sandy soil	Results matching
6-7.5	Clay	Clay at around 7m	Results matching
7.5-9	Clay	Thin clay layer	Results matching
9-14	Poorly graded sand	Dense sand	Almost same
14-15	Clay	Silty clay	Results matching

Comparison of results from SPT and CPTu are seen to be matching at all depths and gave similar profiles.

4.4 Site 2: Hospital Ground

4.4.1 Soil Profiling using CPTu

Variation of tip resistance, skin friction, pore-water pressure and friction ratio with depth were plotted in MATLAB.

Soil profiling based on table 4.4 shows soil to be sandy up to 11m. Silt mixtures up to 17m with clay deposits 12m, 15m and 17m. After 17m soil displays dense sand like behavior.

Soil behavior type classification curve (fig. 4.8) shows soil to be mostly coarse grained contractive (loose sand).

Low value of friction ratio (< 0.1) Normalized tip resistance between 10 and 100.

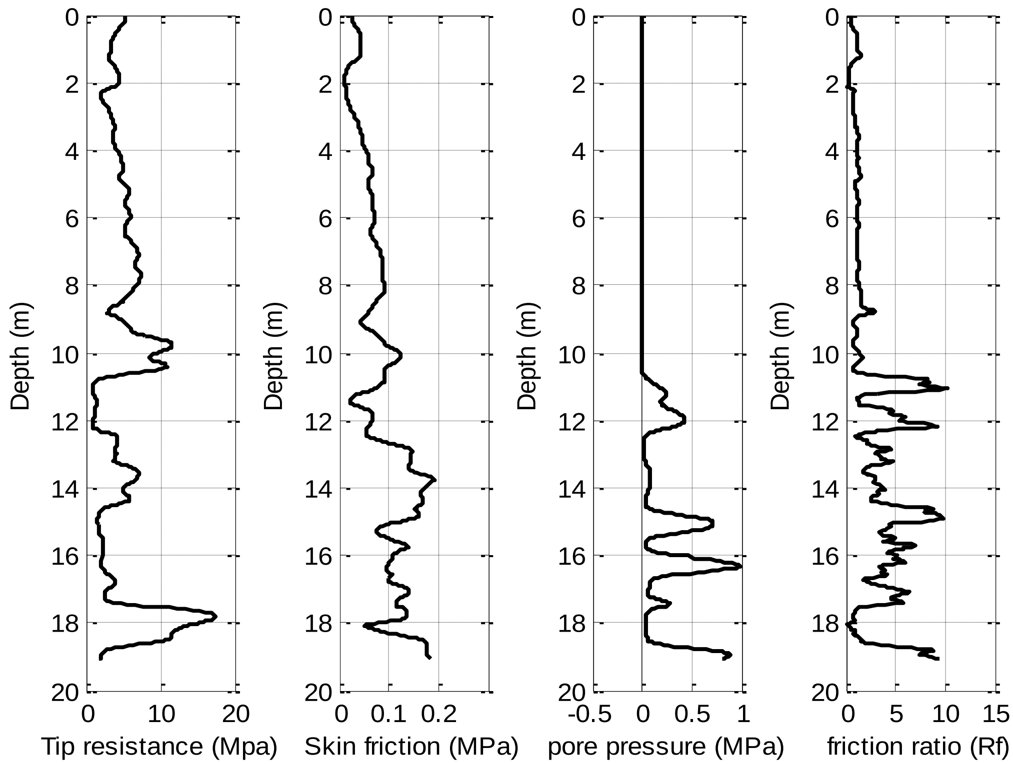


Fig. 4.6 Tip resistance, skin friction and pore pressure variation with depth (Hospital site)

Table 4.5 Soil profiling using CPTu (Hospital site)

Depth (m)	Tip resistance (MPa)	Skin Friction (MPa)	Friction ratio (%)	Pore-water pressure (MPa)	Soil behavior Index	Remarks
0-9	2.5-7.5	0.05 (low)	1 (indicating sand)	0	Zone V-VI Silty sand	Sand like behavior
9-11	Increases to 10	0.1	Low (increases at 11 m)	0	Zone VI Sand	Dense sand deposit
11-13	Decreases to 2	0.05-0.1	Rises to >5 at 12 m	Increases to 0.4 at 12m	Zone II Silty clay	Clayey deposit
13-17	Becomes too low at 15m	0.1-0.15	Average 5 increases to 10 at 15m.	Increases >0.7 at 15m and 17m	Silty clay to clay	Clayey soil
17-19	>15 (very high)	0.1	1 (low)	<0.1 (reduction in fines)	Zone V-VI Sand mixtures	Dense sand behavior

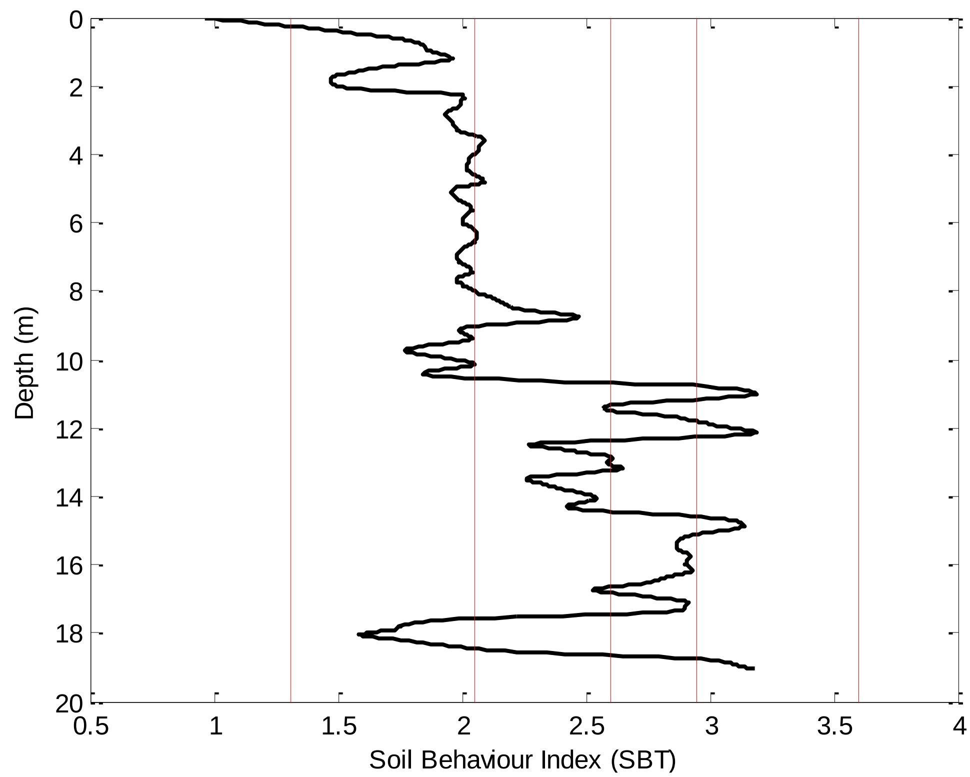


Fig. 4.7 Soil Behavior Type Index (SBT) variation with depth (Hospital site)

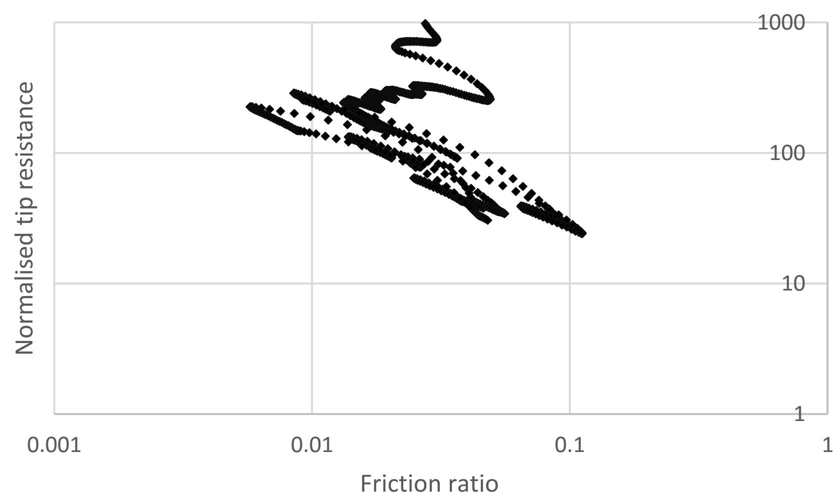


Fig. 4.8 Soil behavior type classification curve (Convocation Hall Site)

4.4.2 Soil Profiling using SPT samples

The lab analysis of samples procured from SPT test for hospital site:

Table 4.6(a) Index properties of sand samples (Hospital ground site)

Depth (m)	Soil type	Fines Content (%)
0.75	SP	13.45
1.50	SP	11.27
2.25	SP	11.85
3.00	SP	7.53
4.50	SP	5.85
6.00	SP	5.61
7.50	SP	4.52
9.00	SP	4.54
10.50	SP	4.18

Table 4.6(b) Index properties of clay samples (Hospital ground site)

Depth(m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Type
12.00	35	16	19	CL

4.4.3 Comparison of SPT and CPTu Results

Table 4.7 Comparison of SPT and CPTu results (Hospital ground site)

Depth (m)	Lab analysis using SPT	Field analysis using CPTu	Remarks
0-10	Poorly graded sand	Sand	Results similar
10-12	Clay	Clay	Results matching

It can be seen from table 4.9 that the soil profiles from both SPT and CPTu match.

4.5 Site 3: DEQ campus

4.5.1 Soil Profiling using CPTu

As seen in table 4.7, up to 3.5m soil indicated sand like behavior. From 3.5 m to 6.5 m silty clay behavior was observed. Clay layers were predicted at about 7m and 9m.

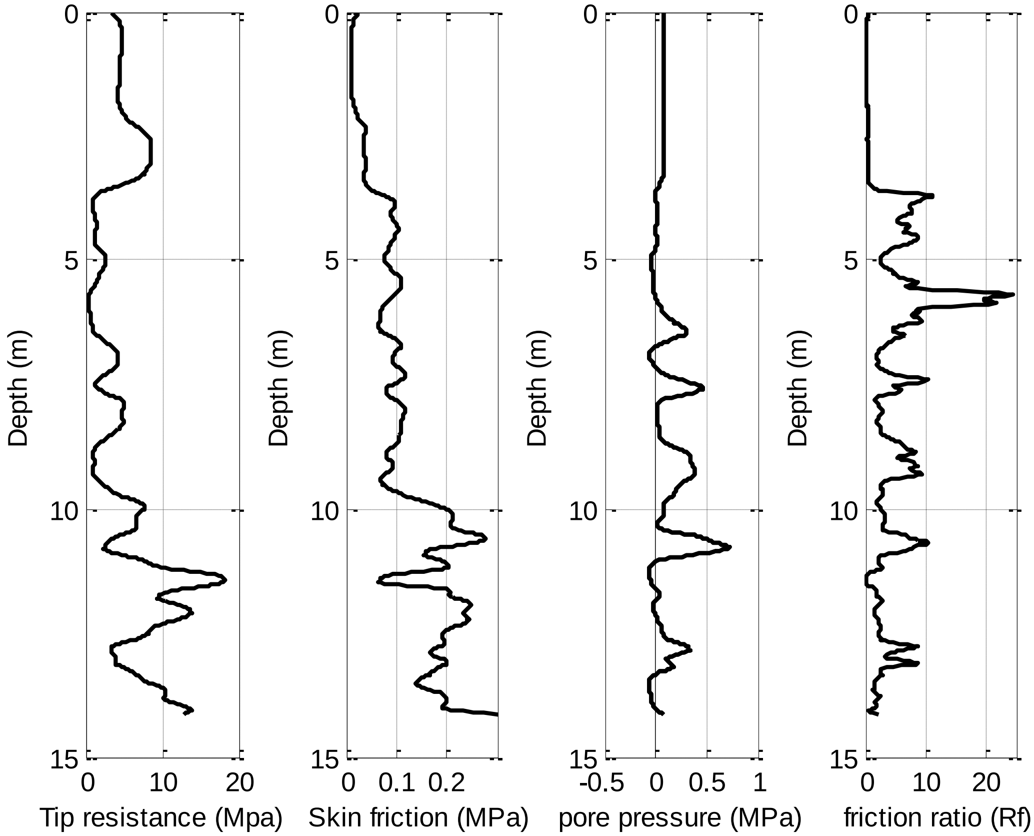


Fig. 4.9 Tip resistance, skin friction and pore pressure variation with depth (DEQ site)

10m-15m showed properties of dense sand mixtures except for presence of clay layers at 10.5m and 13m.

Soil behavior type classification curve (fig. 4.11) indicates soil to be mostly fine grained dilative.

Friction ratio ranges from 1 to 10

Normalized tip resistance ranges from 10 to 100.

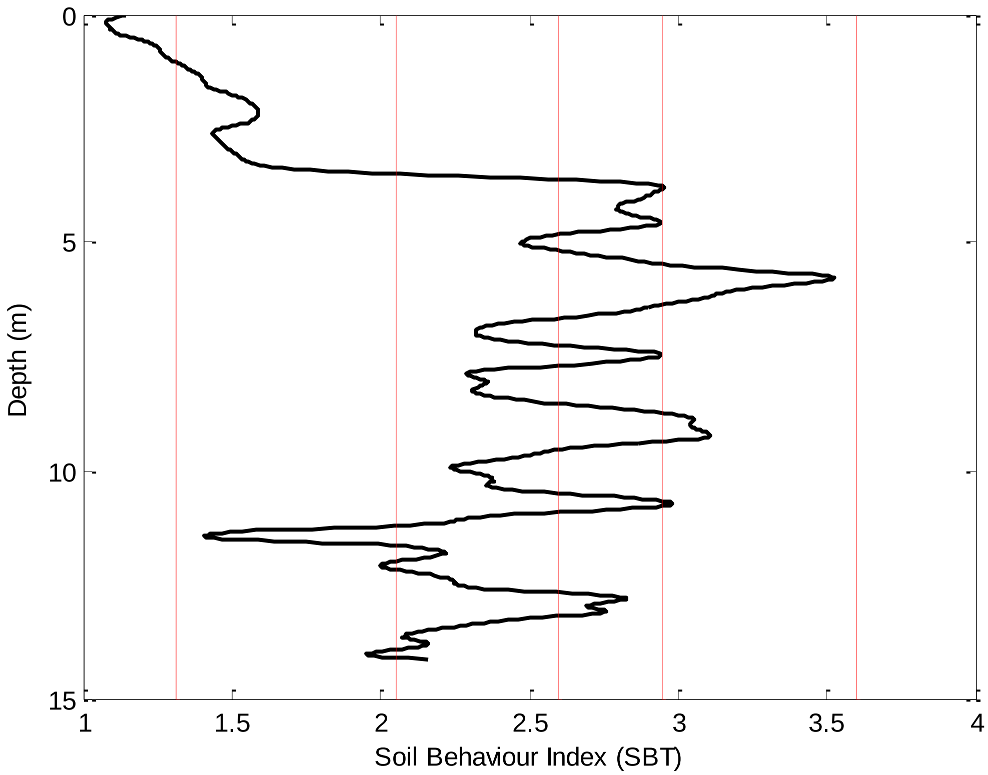


Fig. 4.10 Soil Behavior Type Index (SBT) variation with depth (DEQ site)

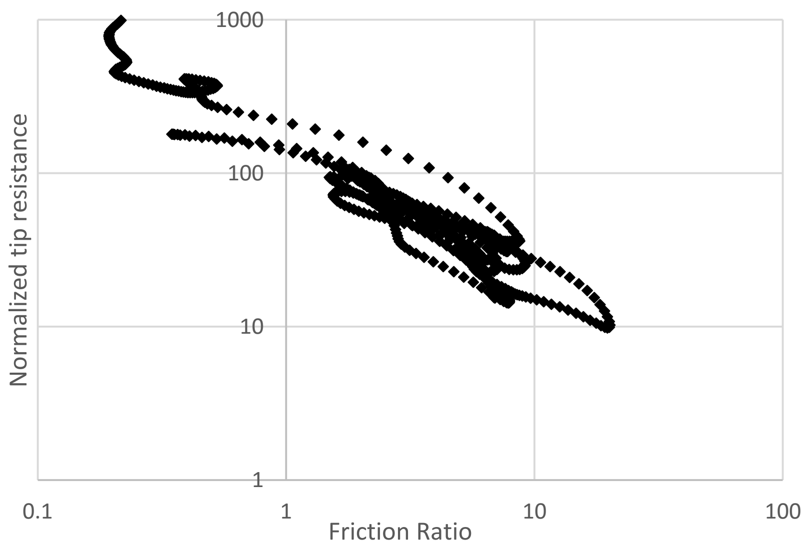


Fig. 4.11 Soil behavior type classification curve (DEQ site)

Table 4.8 Soil profiling using CPTu (DEQ)

Depth (m)	Tip resistance (MPa)	Skin Friction (MPa)	Friction ratio (%)	Pore-water pressure (MPa)	Soil behavior Index	Remarks
0-3.5	5-7.5	<0.05 (low)	0 (indicates sand like behavior)	0.1	Zone VI	Sand
3.5-6.5	<2.5	0.1	>5 goes to >20 at 6m (increase in fines)	0	Zone III-IV Silty clay-clay	Clay layer at 6m
6.5-10	<5 significant reduction at 7m and 9m	0.1	2 (increases to 8 at 7.5m and 9m)	<0.1 increases to 0.4 at 7.5m and 9m	Zone IV-V Soil enters zone III at 7m and 9m	Silt clay mixtures Clay layers at 7m and 9m
10-15	>10 from 11m to 13m(dense)	0.1-0.2	3 (Increases to >7 at 10.5m and 13m)	0 reaches 0.6 at 10.5m and 0.3 at 13m	Zone IV-V	Dense sand mixtures. Clay layer at 10.5m and 13m

4.5.2 Soil Profiling using SPT Samples

Table 4.9(a) Index properties of sand samples (DEQ site)

Depth(m)	Soil Type	Fines Content (%)
0.75	SP	3.6
1.50	SP	3.2
7.50	SP	3.8
9.00	SP	2.8

Table 4.9(b) Index properties of clay samples (DEQ site)

Depth(m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Type
3.00	33	13	19	CL
4.50	28	14	17	CL
6.00	37	17	20	CI

4.5.3 Comparison of SPT and CPTu Results

Table 4.10 Comparison of SPT and CPTu results (DEQ site)

Depth (m)	Lab analysis using SPT	Field analysis using CPTu	Remarks
0-1.5m	Poorly graded sand	Sand	Results similar
1.5m-6.5m	Clay	Silty clay-clay	Results matching
6.5m-10m	Poorly graded sand	Silty clay	Results not matching

Table 4.13 reflects the results from the two methods to be in agreement up to 6.5m after which the results from the two methods do not match.

4.6 Site 4: Solani Kunj (IIT Roorkee campus)

4.6.1 Soil Profiling using CPTu

Up to 7m soil showed to exhibit silt clay mixtures. 7m-10m is marked by dense sandy silt. Again up to 14m soil was observed to be silt clay mixture. 14m-17m was marked by dense sand.

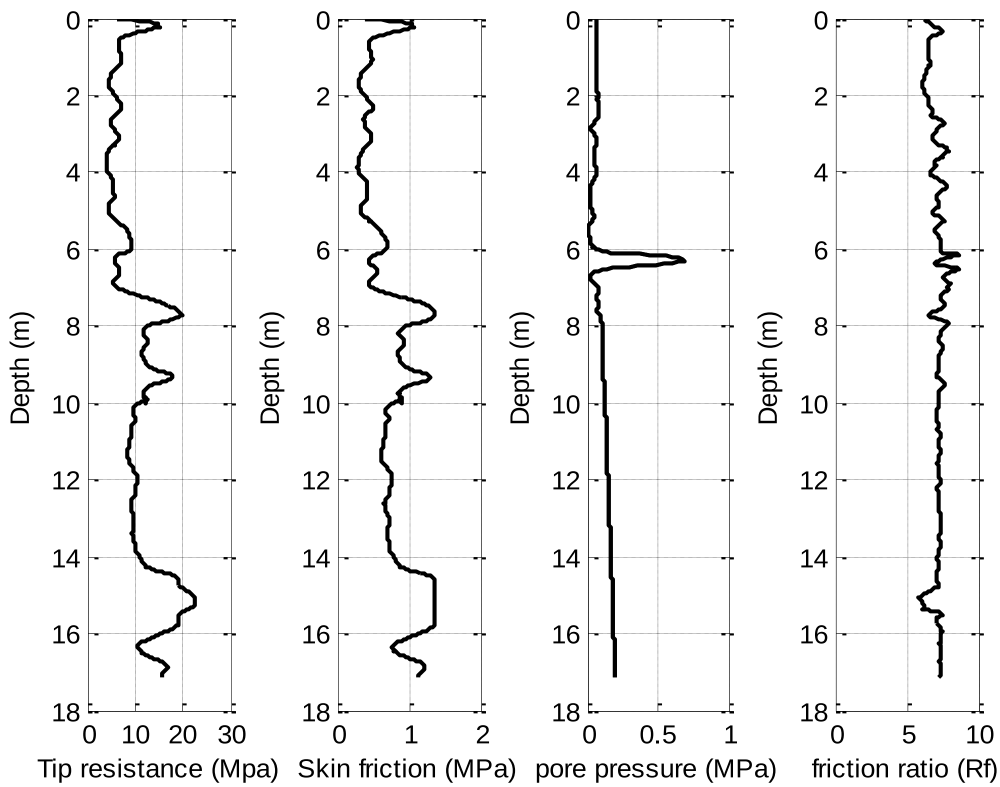


Fig. 4.12 Tip resistance, skin friction and pore pressure variation with depth (Solani kunj site)

Table 4.11 Soil profiling using CPTu (Solani kunj site)

Depth (m)	Tip resistance (MPa)	Skin Friction (MPa)	Friction ratio (%)	Pore-water pressure (MPa)	Soil behavior Index	Remarks
0-7	5	0.5	7-8	<100 abruptly increases to 6 at 6m	Zone IV-V	Sandy silt to clayey silt
7-10	>15	1	7-8	100	Zone V	Dense sandy silt
10-14	10	0.7	7-8	<200	Zone IV-V	Silt-clay mixtures
14-17	Reaches 20	1.3	Drops to 6	200	Zone V	Highly dense sand

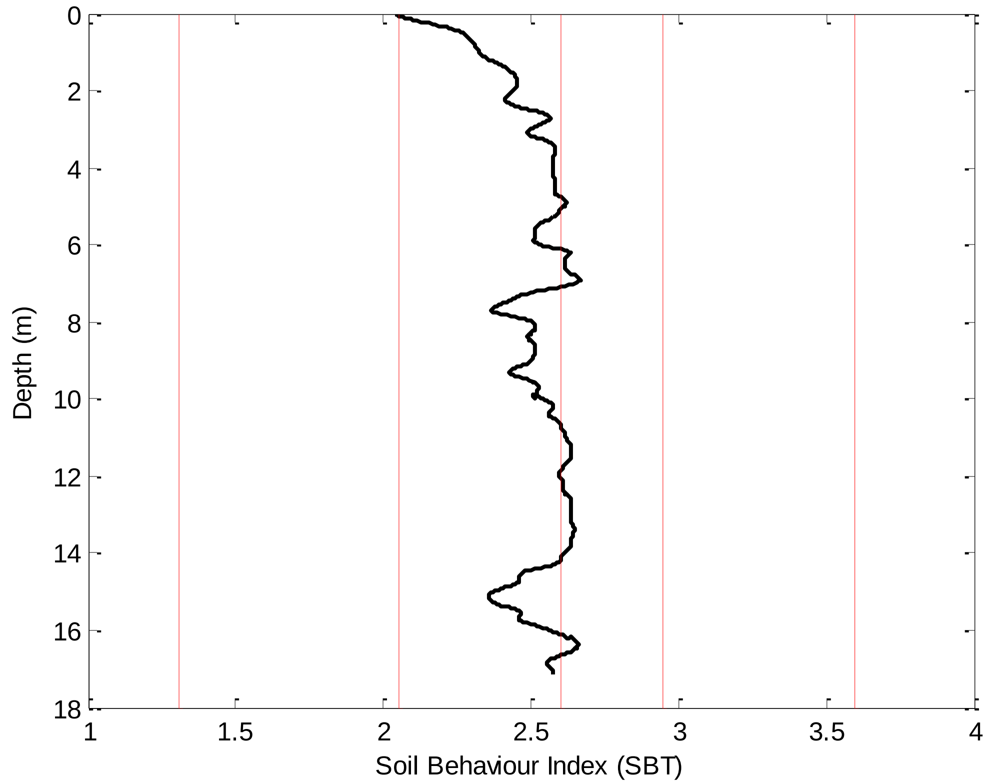


Fig. 4.13 Soil Behavior Type Index (SBT) variation with depth (Solani kunj site)

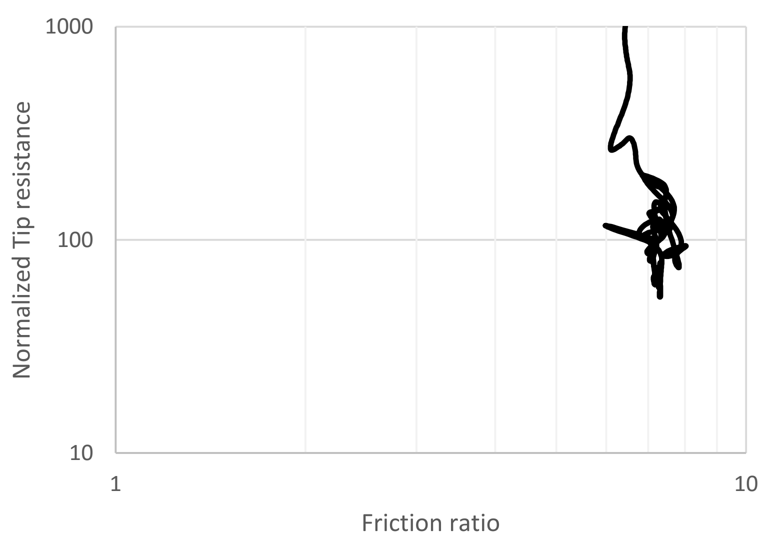


Fig. 4.14 Soil behavior type classification curve (Solani kunj Site)

Soil behavior type classification curve indicated most of the soil to be fine grained dilative

friction ratio close about 10%

normalized tip resistance varying from 100 to 1000).

4.6.2 Soil Profiling using SPT

Soil after lab analysis showed sand like behavior up to 9.5 m except for a clay zone from 4.5 m to 6 m. data was available only up to 9.5 m.

4.6.3 Comparison of SPT and CPTu Results

Table 12 comparison of SPT and CPTu results

Depth (m)	Lab analysis using SPT	Field analysis using CPTu	Remarks
0-7	Sand	Sandy silt	Results similar
7-10	Clay	Silty clay-clay	Results matching

4.8 Summary and Concluding Remarks

Results for the four sites can be summarized as follows:

At site 1 soil is fine grained at most of the depths with clay layer at 7m and 9m. denseness of soil increases with depth.

At site 2 soil is loose sand at initial depths up-to 11m and clay layer at 12m, 15m and 17m.

At site 3 soil is mostly fine grained comprising silts and is dilative in nature. Clay layers are encountered at 3m, 4.5m and 6m.

At site 4 soil is mostly silty and dilative. Up to 7m it is silty clay followed by dense sand upto 10m.

Following conclusions were obtained from this chapter:

- For all four sites soil profiling using CPTu and SPT were quite similar.

- Tip resistance gave a fair indication of denseness or looseness of the soil. A value of 5 MPa can be used as a marginal value to distinguish stiff and soft soils.
- Friction ratio gave a pretty good indication of increase in fines. Friction ratio >5% can be taken as indication of silt and still higher value (up to 10%) may mean clay.
- Pore water pressure showed rapid rise whereever fine silt or clay was encountered. A value of 0.5 MPa was established to differentiate between fine (silts) and coarse soils (sands).
- After comparing soil profiling from both the methods it can be concluded that CPTu can be reliably used for soil profiling as a quick and convenient method.

Chapter 5

Liquefaction Analysis

5.1 Evaluation of Liquefaction Potential

In this section liquefaction analysis is carried out for the four sites mentioned earlier in section 3.4 using CPTu and SPT test data. For liquefaction analysis using CPTu data, Robertson and Wride (1998) method was used while for SPT test data, Seed and Idriss method (1971) was used. Attempt has been made to assess the credibility of CPTu for liquefaction analysis by comparing the results with those obtained from SPT.

5.1.1 Cyclic Stress Ratio (CSR)

CSR, which is the seismic demand on soil due to earthquake was calculated using expression 5.1 as given by Seed and Idriss (1971) for different depths.

$$CSR = 0.65(a_{max}/g)(\sigma_v/\sigma_v')r_d \tag{5.1}$$

where, a_{max} = peak horizontal ground surface acceleration, g = acceleration of gravity, σ_v = total vertical (overburden) stress at the depth in question, σ_v' = effective overburden stress at the same depth, and r_d=shear stress reduction coefficient.

$$\ln(r_d)=\alpha(z)+\beta(z)M \tag{5.2a}$$

$$\alpha(z)= -1.012-1.126\sin(z/11.73+5.133) \tag{5.2b}$$

$$\beta(z)=0.106+0.118\sin(z/11.28+5.142) \tag{5.2c}$$

Above equations are for z<=34 m. for z >34 m:

$$r_d = 0.12\exp(0.22M) \tag{5.2d}$$

z= depth

M= magnitude of earthquake (7.5 is taken in this study)

For Roorkee (zone IV), a_{max} = 0.24

5.1.2 Cyclic Resistance Ration (CRR)

In this subsection cyclic resistance ratio of all four sites were evaluated using two methods as given below.

(a) Robertson and Wride Method:

Liquefaction resistance using CPTu data was evaluated using Robertson and Wride method (1998). Empirical equations were given (equation 5.2 and 5.3) to evaluate liquefaction resistance as an approximation of curve between corrected CPT tip resistance and Cyclic Resistance Ratio given by Robertson and Wride (fig. 2.3)

$$\text{If } (q_{c1N)cs}) < 50 \quad CRR_{7.5} = 0.833[(q_{c1N})_{cs} / 1000] + 0.05 \tag{5.3a}$$

$$50 < (q_{c1N)cs}) < 160 \quad CRR_{7.5} = 93[(q_{c1N})_{cs} / 1000]^3 + 0.08 \tag{5.3b}$$

where $(q_{c1N})cs$ = clean-sand cone penetration resistance normalized to approximately 100 kPa.

$$(q_{c1N})_{cs} = K_c\,(q_{c1N}) \tag{5.3c}$$

Where, K_c is correction factor for grain size. K_c as given by Robertson and Wride as

$$\text{For } I_C < 1.64;\ K_c = 1 \tag{5.4a}$$

$$\text{For } I_C > 1.64;\ K_c = -\,0.403Ic^4 + 5.581Ic^3 – 21.63Ic^2 + 31.75Ic – 17.88 \tag{5.4b}$$

Ic is calculated as per equation 4.3.

Corrections as per IS: 4968(part III) were applied to tip resistance and sleeve friction section 4.1. After applying pore pressure corrections tip resistance and friction ratio (sleeve friction/tip resistance) were normalized to 100 kPa. Then CRR was evaluated.

CRR evaluated is for 7.5 magnitude earthquake. For an earthquake of different magnitude M, CRR is multiplied with Magnitude scaling factor (Idriss 1999) defined as

$$\text{Magnitude scaling factor} = 6.9 \exp(-M/4)-0.058 \tag{5.5}$$

(b) Seed and Idriss Method:

Cyclic resistance ratio (CRR), cyclic stress ratio (CSR) and Factor of safety against liquefaction at different depths were calculated using Seed-Idriss (1971) method.

SPT N values used were taken from the test already conducted Kirar et al. (2016) and Kant (2014). Corresponding dry densities were determined using lab testing and corrected N values (N_1) were obtained after applying overburden corrections to them. The corrected N values were converted to equivalent clean sand values after applying corrections given in equation 5.6.

$$(N_1)_{60cs} = \alpha + \beta(N_{160}) \quad (5.6a)$$

Where $(N_1)_{60cs}$ = equivalent clean sand value of $(N_1)_{60}$ and α and β are coefficients determined using the following relationships:

$$\alpha = 0 \text{ and } \beta = 1 \quad \text{for FC} <= 5\% \quad (5.6b)$$

$$\alpha = \exp(1.76 - 190/FC^2) \text{ and } \beta = 0.99 + FC^{1.5}/1000 \quad \text{for } 5\% < FC < 35\% \quad (5.6c)$$

$$\alpha = 5 \text{ and } \beta = 1.2 \quad \text{for FC}>=35\% \quad (5.6d)$$

CRR was calculated using empirical equation 5.7 for each depth and the results obtained were compared against those of CPTu using the MATLAB code.

$$CRR_{7.5}=1/(34-(N_1)_{60cs})+((N_1)_{60cs})/135+50/[10.(N_1)_{60cs}+45]^2-1/200 \quad (5.7)$$

This equation is valid for $(N1)_{60cs} < 30$. For $(N1)_{60cs} > 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Factor of safety was evaluated as the ratio of CRR to CSR.

5.2 Site 1: Convocation Hall

5.2.1 Liquefaction Analysis using CPTu

0-6 m FOS<1; liquefaction occurred.

6- 8 m clay

8-10 m FOS>1; no liquefaction.

10-14 m FOS<1; liquefaction occurred

14- 17 m clay

17-19 m FOS>1 no liquefaction.

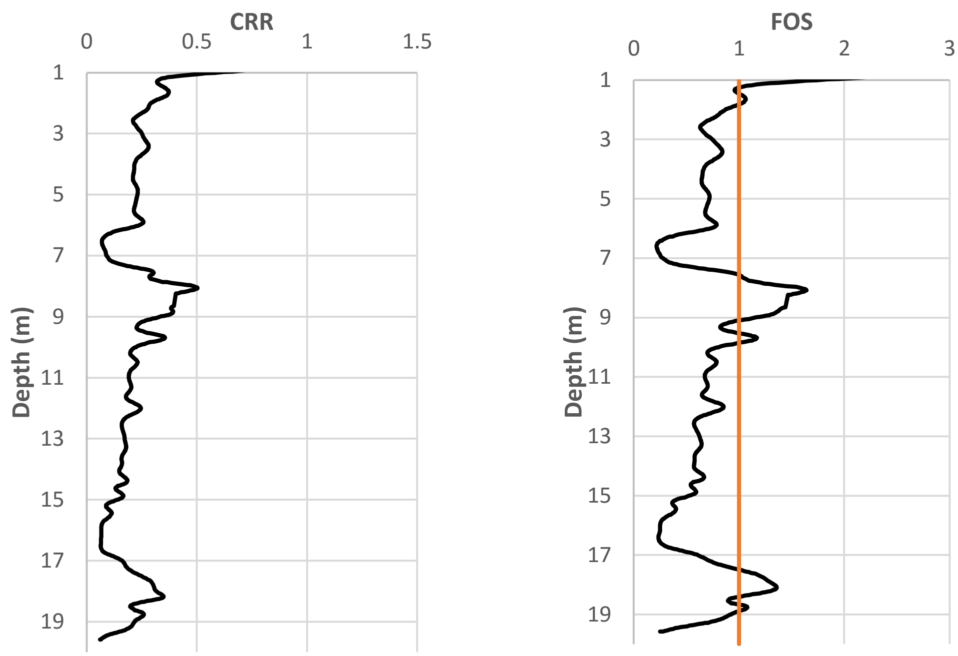


Fig. 5.1 CRR, FOS variation with depth using CPTu (Convocation Hall Site)

Table 5.1 CSR, CRR and FOS using CPTu (Convocation hall site)

Depth (m)	q_c (MPa)	fs (kPa)	σ_{vo} (kPa)	σ'_{vo} (kPa)	Q	F	I_c	CSR	CRR	FOS
0.75	3.764	69	14	7	571.82	1.84	1.98	0.33	1.00	2.86
1.50	4.697	40	27	12	380.55	0.85	1.77	0.35	0.37	1.04
2.25	4.997	48	41	19	261.57	0.96	1.85	0.34	0.29	0.80
3.00	5.347	51	56	26	206.87	0.96	1.88	0.33	0.26	0.76
4.50	5.677	50	83	38	145.65	0.90	1.91	0.33	0.20	0.64
5.99	8.977	74	115	55	161.34	0.84	1.80	0.31	0.30	0.80
7.50	8.758	158	150	75	114.24	1.84	2.09	0.29	0.36	1.05
8.99	5.713	153	178	88	62.56	2.76	2.37	0.29	0.30	1.18
10.50	11.388	100	206	101	110.76	0.90	1.84	0.28	0.28	0.82
12.51	7.557	70	238	113	64.81	0.95	2.01	0.28	0.16	0.57
15.00	3.483	346	309	159	19.96	10.89	3.07	0.25	0.07	0.55

5.2.2 Liquefaction analysis using SPT data

Liquefaction analysis was done using SPT data. Seed and Idriss method was used to determine the liquefaction potential. The overburden corrected N value was multiplied by equivalent grain size factor to convert it to clean sand equivalent, calculated as given in the Literature review.

Table. 5.2 CSR, CRR and FOS using SPT (Convocation Hall Site)

Depth (m)	N	σ_{vo} (kPa)	σ'_{vo} (kPa)	(C_N)	$(N_1)_{60}$	(N_{1cs})	CSR	CRR	FOS
0.75	6.00	14	10	1.70	10.20	12.20	0.33	0.13	0.39
1.50	7.00	27	12	1.70	11.90	12.23	0.35	0.13	0.37
2.25	12.00	41	19	1.70	20.40	21.46	0.34	0.23	0.68
3.00	12.00	56	26	1.70	20.40	21.73	0.33	0.24	0.73
4.50	11.00	83	38	1.61	17.75	19.32	0.33	0.21	0.64
6.00	16.00	115	55	1.35	21.59	22.20	0.31	0.24	0.77
7.50	14.00	150	75	1.15	16.13	19.58	0.29	0.21	0.72
9.00	16.00	178	88	1.06	17.01	23.32	0.29	0.26	0.90
10.50	15.00	206	101	1.00	14.93	15.74	0.28	0.17	0.61
12.50	13.00	238	113	0.94	12.23	14.73	0.28	0.16	0.57

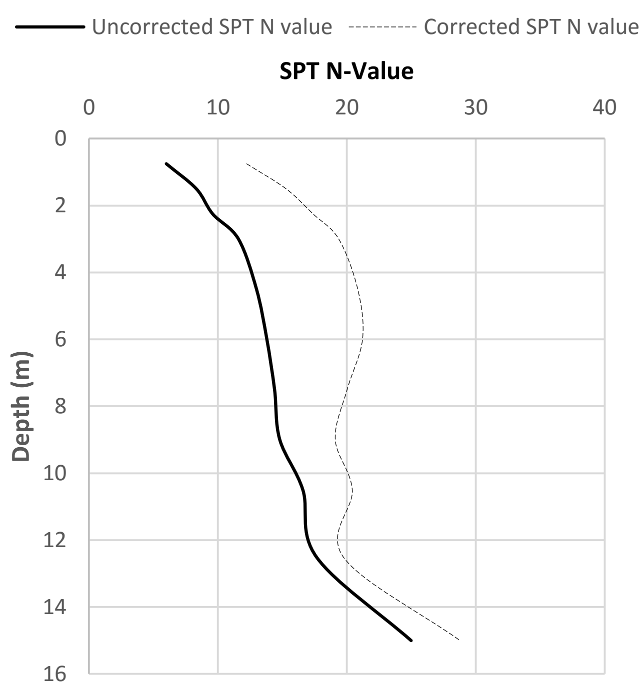
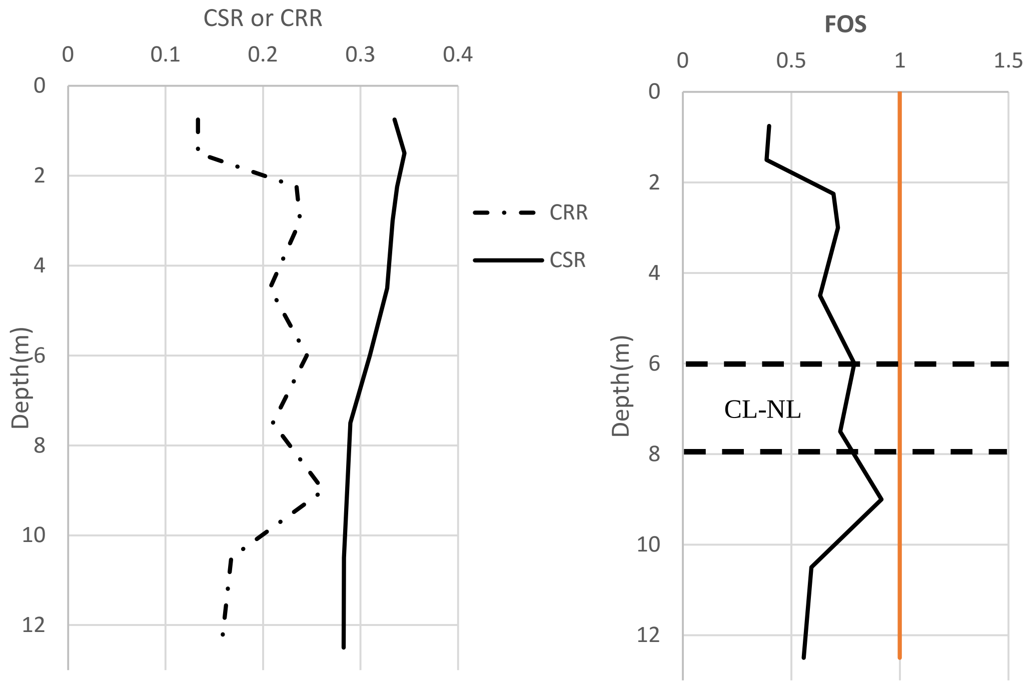


Fig. 5.2 Uncorrected & Corrected N Values with the Depth (Convocation Hall Site)

Uncorrected and corrected SPT N value against depth were plotted.

The CSR, CRR and FOS were plotted against depth as shown in fig. 5.3



CL-NL = Clay--No Liquefaction

Fig. 5.3 CSR, CRR and FOS variation with depth using SPT (Convocation hall site)

Liquefaction occurred at 0-6 m and 10 m -14 m same as predicted by CPTu.

5.2.3 Comparison of Liquefaction Resistance from CPTu and SPT

Liquefaction resistance and Factor of Safety using both the methods were plotted against depth in MATLAB. It can be seen from the figure that the results were fairly in agreement at an appreciable number of depths. The CRR and FOS values from both approaches are close enough to be said in agreement and follow quite similar trend except at initial depths. SPT typically gives low values of CRR and FOS initially while CPTu initially gives comparatively higher values of CRR and FOS (which aren't practically significant). For this reason initial 1 m depth from the analysis has been ignored.

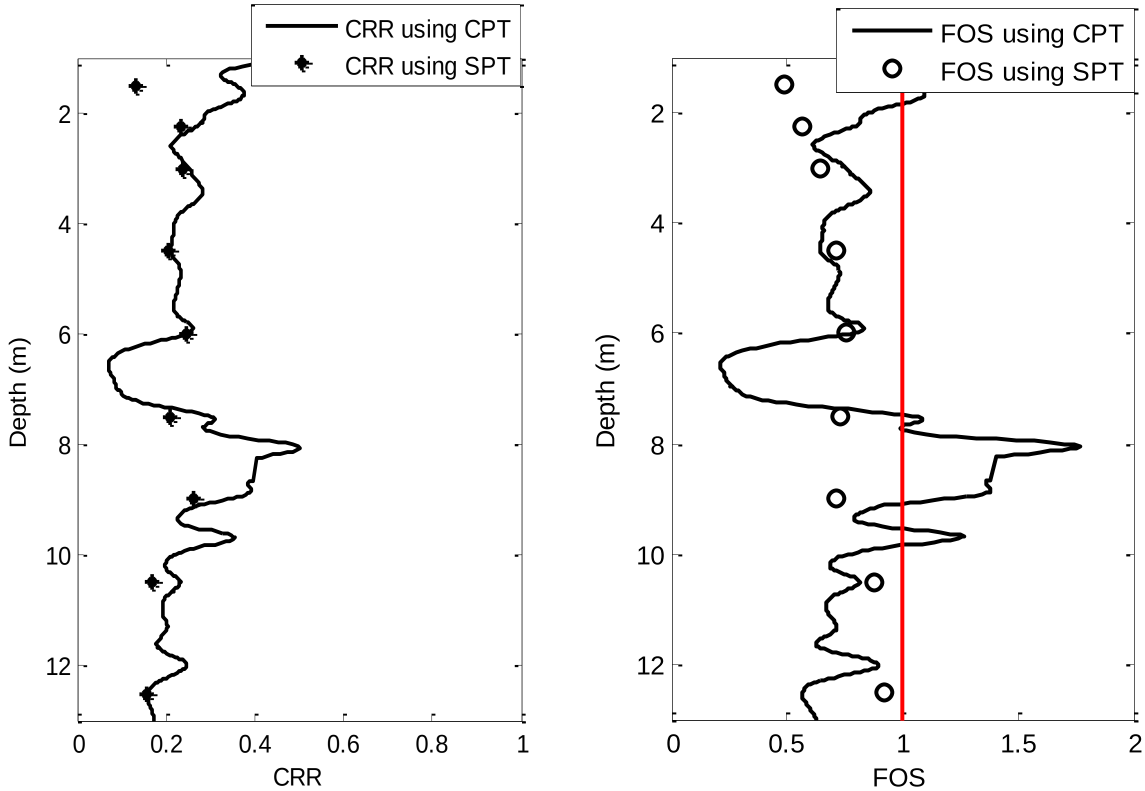


Fig. 5.4 Comparison of CRR and FOS obtained by using CPTu and SPT (Convocation hall site)

5.3 Site 2: Hospital Ground

5.3.1 Liquefaction Analysis using CPTu Data

Up-to 9 m FOS<1; liquefaction occurred

9-11 m FOS>1; No liquefaction

11-13 m clay

13-15 m FOS<1; liquefaction occurred

15-17 m clay

17-19 m FOS>1; No liquefaction (too dense too liquefy).

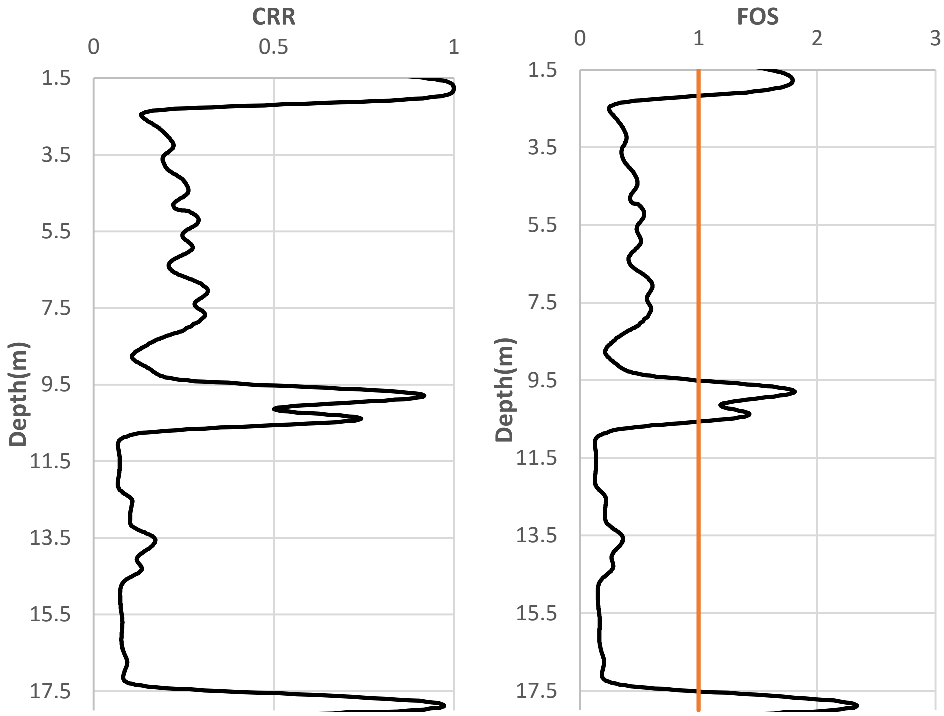


Fig. 5.5 CRR, FOS variation with Depth using CPTu at Hospital ground site

Table 5.3 CSR, CRR and FOS using CPTu (Hospital ground site)

Depth(m)	**q_c (Mpa)**	**f_s (kPa)**	**σ_{vo} (kPa)**	**σ'_{vo} (kPa)**	**Q**	**F**	**I_c**	**CSR**	**CRR**	**FOS**
0.76	37.36	45	14	6	594.51	1.21	1.84	0.35	0.82	2.37
1.51	41.35	19	26	11	374.43	0.46	1.63	0.37	0.28	0.75
2.24	18.66	17	38	15	119.57	0.93	2.15	0.38	0.16	0.42
2.99	36.46	31	53	23	154.20	0.86	1.97	0.35	0.16	0.46
4.50	52.26	62	84	39	130.55	1.21	2.02	0.32	0.21	0.65
6.00	64.47	68	114	54	118.04	1.08	1.97	0.31	0.21	0.65
7.50	69.27	87	144	69	97.73	1.28	2.04	0.30	0.22	0.73
9.00	53.97	45	165	75	69.34	0.86	2.04	0.31	0.13	0.43
10.50	109.96	83	203	98	109.60	0.77	1.80	0.29	0.24	0.82
12.00	14.23	63	219	99	12.16	5.23	3.02	0.30	0.06	0.21

5.3.2 Liquefaction Analysis using SPT Data

Table 5.4 Depth wise Calculation of CSR, CRR (Hospital ground Site)

Depth (m)	N	σ_{vo} (kPa)	σ'_{vo} (kPa)	(C_N)	$(N_1)_{60}$	(N_{1cs})	CSR	CRR	FOS
0.76	2.00	14	3	1.70	3.40	5.13	0.57	0.07	0.12
1.51	3.00	26	6	1.70	5.10	5.23	0.56	0.07	0.13
2.24	5.00	38	8	1.70	8.50	8.64	0.59	0.10	0.17
2.99	9.00	53	11	1.70	15.30	16.82	0.56	0.18	0.32
4.50	11.00	84	18	1.70	18.70	19.02	0.54	0.20	0.37
6.00	15.00	114	24	1.70	25.50	27.78	0.52	0.36	0.69
7.50	17.00	144	30	1.70	28.90	37.46	0.51	0.02	0.04
9.00	16.00	165	35	1.68	26.94	35.23	0.50	0.55	1.10
10.50	18.00	203	44	1.51	27.11	30.08	0.47	0.47	1.00
12.00	14.00	219	41	1.57	21.92	25.13	0.53	0.29	0.55

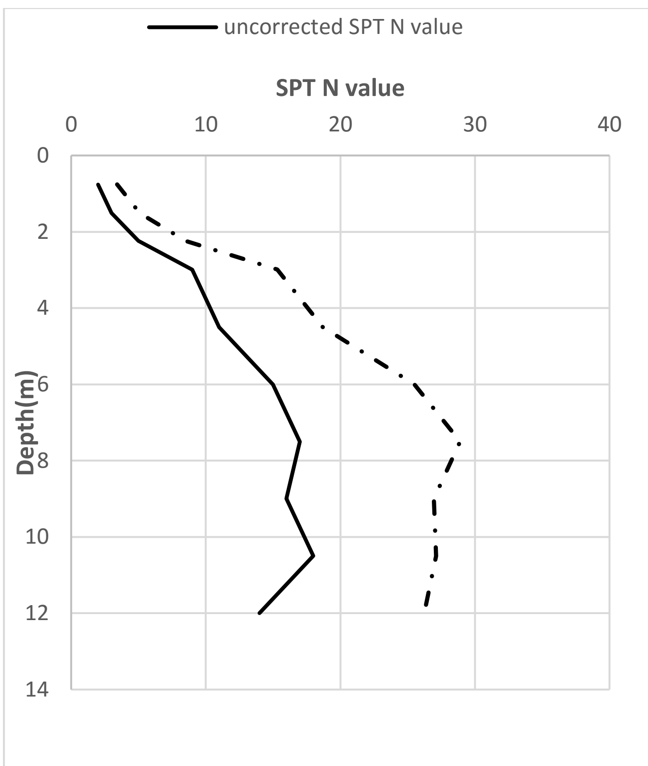


Fig. 5.6 Uncorrected & Corrected N Values with the Depth (Hospital Site)

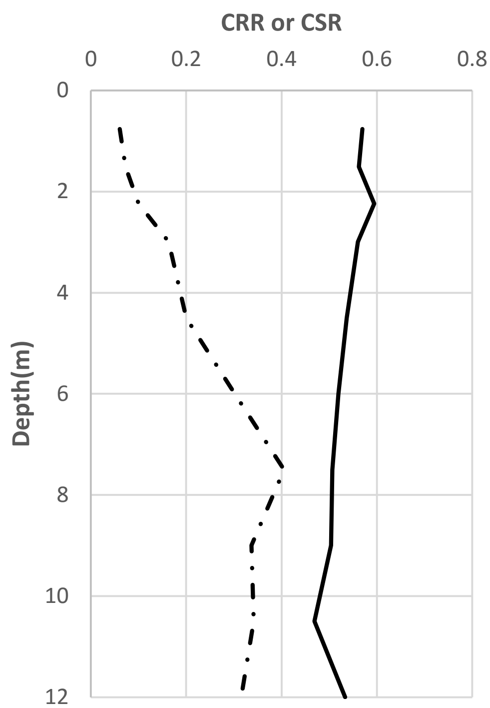


Fig. 5.7 CSR and CRR variation with depth using SPT (hospital site)

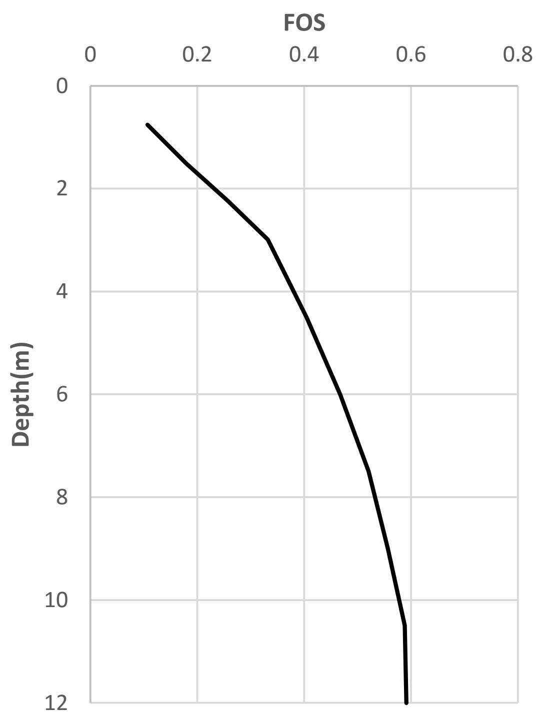


Fig. 5.8 FOS variation with depth (Hospital site)

Liquefaction occurred at 0-10 m. After 13 m SPT data was not available.

5.3.3 Comparison of Liquefaction Resistance from CPTu and SPT

Comparison of CRR and FOS evaluated from the two methods showed good agreement among each other. The values obtained from both the methods were fairly close to each other.

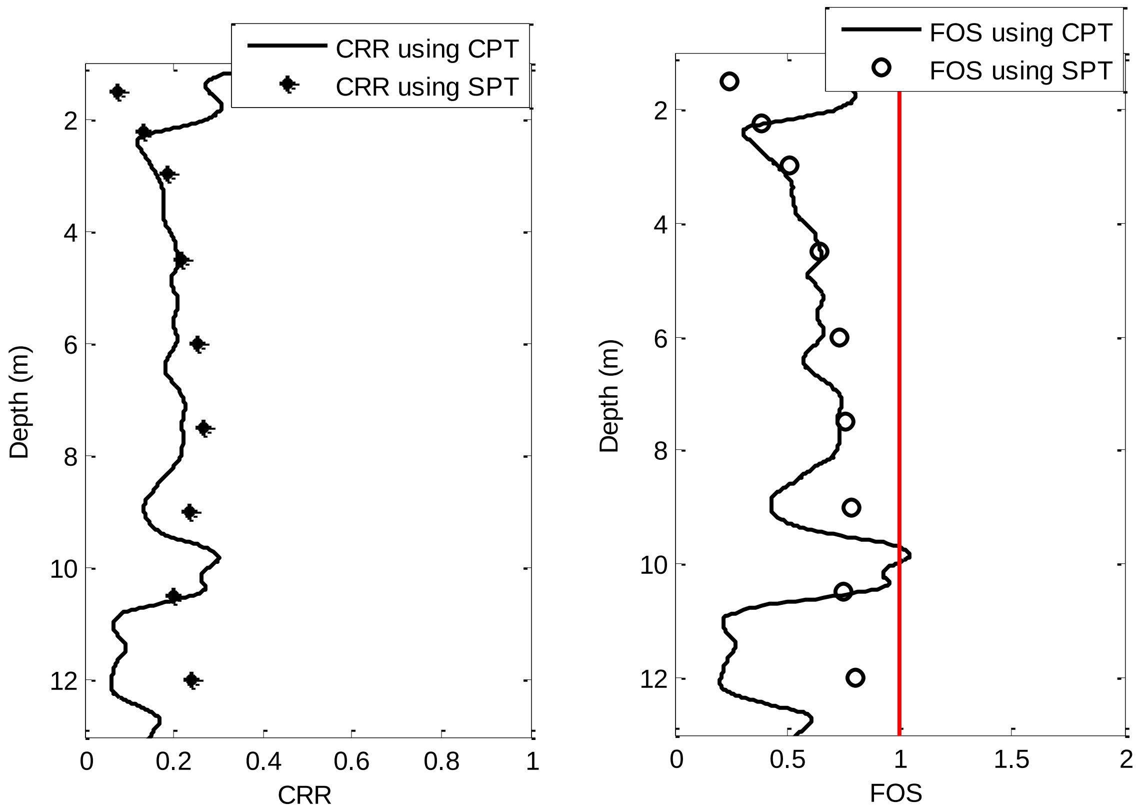


Fig. 5.9 Comparison of CRR and FOS obtained by using CPTu and SPT (Hospital site)

5.4 Site 3: DEQ

5.4.1 Liquefaction Analysis using CPTu Data

Up to 3 m FOS>1; no liquefaction

3 m-6 m; clay

6 m -8.5 m FOS<1; liquefaction occurred

9 m onwards; no liquefaction

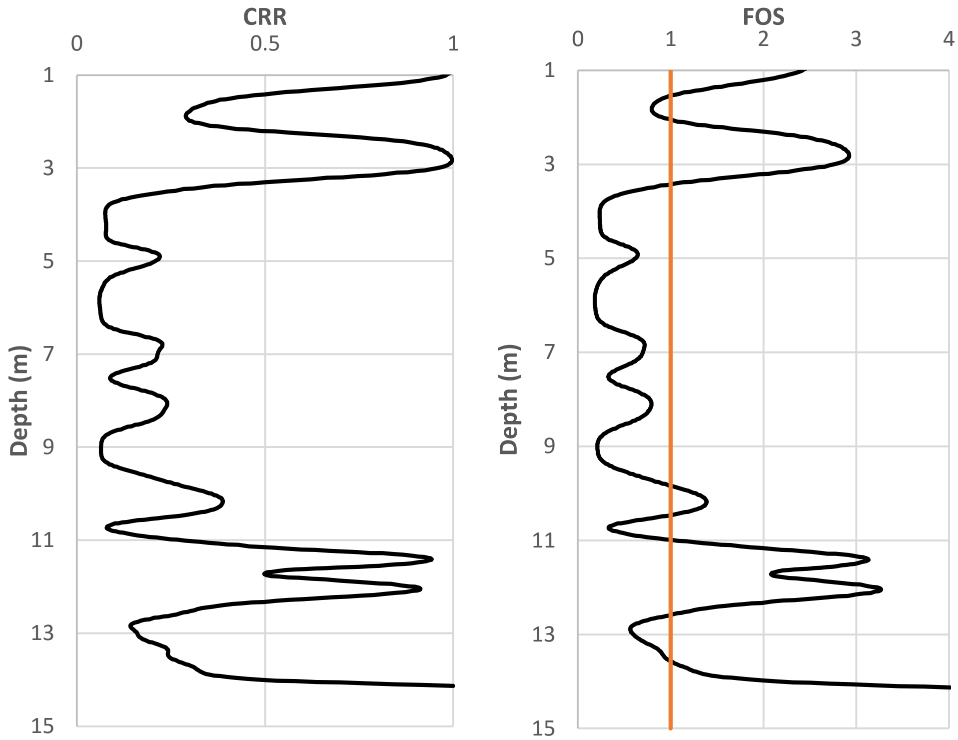


Fig. 5.10 CRR, FOS variation with Depth using CPTu at DEQ Site

Table 5.5 CSR, CRR and FOS using CPTu (DEQ site)

Depth(m)	q_c (MPa)	f_s (kPa)	σ_{vo} (kPa)	σ'_{vo} (kPa)	Q	F	I_c	CSR	CRR	FOS
0.51	5.00	8	8	3	1559.34	0.15	1.10	0.41	1.00	2.52
1.01	4.70	8	17	6	730.24	0.18	1.29	0.40	0.98	2.47
1.50	4.80	10	25	10	487.36	0.20	1.39	0.39	0.41	1.04
1.99	4.93	22	35	15	328.39	0.44	1.62	0.36	0.30	0.84
2.49	6.35	15	43	18	353.92	0.24	1.42	0.37	0.92	2.66
3.00	8.88	38	55	25	350.67	0.43	1.49	0.34	0.96	2.85
3.50	3.74	44	64	29	127.62	1.21	2.08	0.34	0.24	0.71
3.99	1.20	86	74	34	32.97	7.62	2.93	0.33	0.08	0.23
4.49	1.30	102	84	39	30.77	8.38	2.97	0.32	0.08	0.24
4.99	2.96	68	93	43	66.54	2.36	2.42	0.32	0.21	0.65
5.49	1.76	124	105	50	33.05	7.47	2.89	0.31	0.07	0.23

5.4.2 Liquefaction Analysis using SPT Data

Table 5.6 Depth wise calculation of Liquefaction potential (DEQ site)

Depth	N	σ_{vo} (kPa)	σ'_{vo} (kPa)	C_N	$(N_1)_{60}$	(N_{1cs})	CSR	CRR	FOS
0.50	4.00	8	3	1.70	6.80	6.80	0.41	0.09	0.22
1.00	5.00	17	6	1.70	8.50	8.50	0.40	0.10	0.25
1.50	4.00	25	10	1.70	6.80	6.80	0.39	0.09	0.23
2.00	10.00	35	15	1.70	17.00	17.00	0.36	0.18	0.50
2.50	6.00	43	18	1.70	10.20	10.20	0.37	0.11	0.30
3.00	7.00	55	25	1.70	11.90	11.90	0.34	0.13	0.38
3.50	8.00	64	29	1.70	13.60	16.87	0.34	0.18	0.53
4.00	9.00	74	34	1.70	15.30	23.36	0.33	0.26	0.79
4.50	7.00	84	39	1.59	11.15	18.38	0.32	0.20	0.63
5.00	5.00	93	43	1.52	7.62	13.11	0.32	0.14	0.44
5.50	7.00	105	50	1.41	9.89	16.86	0.31	0.18	0.58
6.00	1.00	8	48	1.44	1.44	6.72	0.33	0.09	0.27

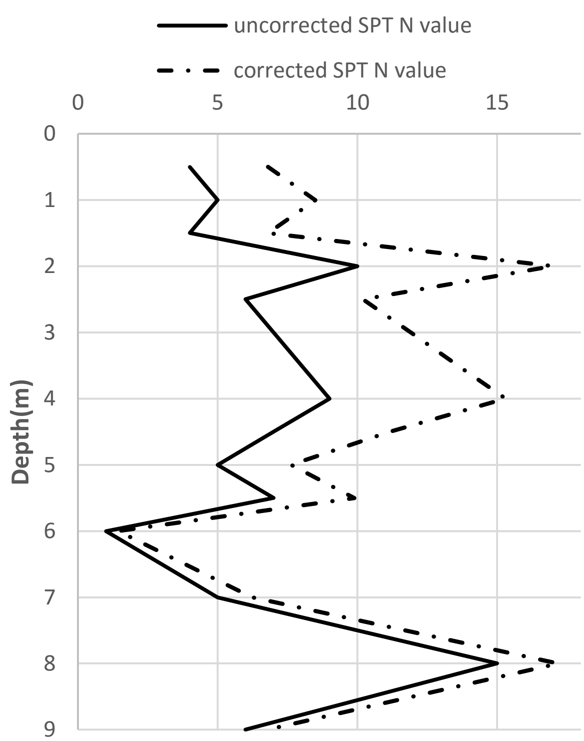


Fig. 5.11 Uncorrected & Corrected N Values with the Depth (DEQ Site)

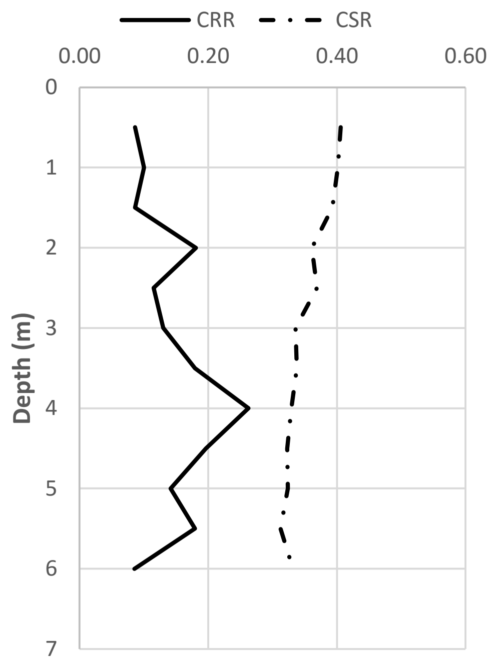
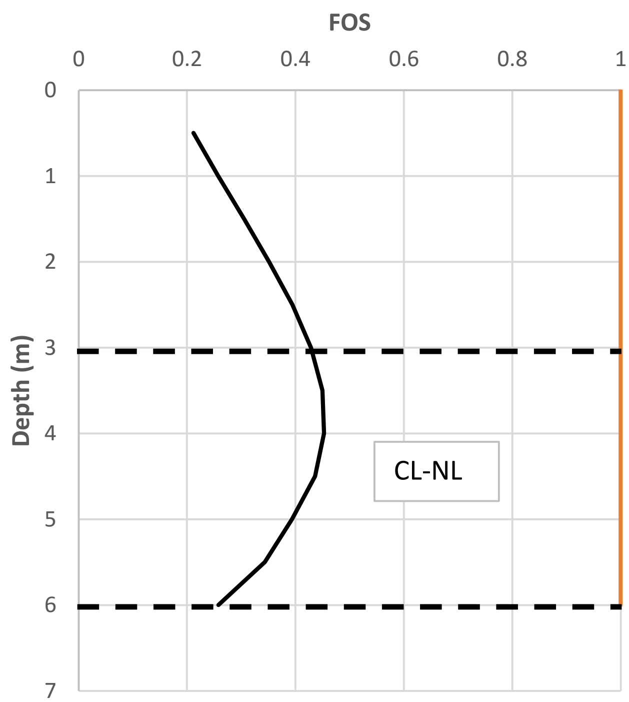


Fig. 5.12 CSR and CRR variation with depth using SPT (DEQ site)



CL-NL = Clay--No Liquefaction

Fig. 5.13 FOS variation with depth using SPT (DEQ site)

Liquefaction occurred up to 2 m after which clay was established up to a depth of 6 m. Data available only up to 6 m.

5.4.3 Comparison of Liquefaction Resistance fron CPTu and SPT

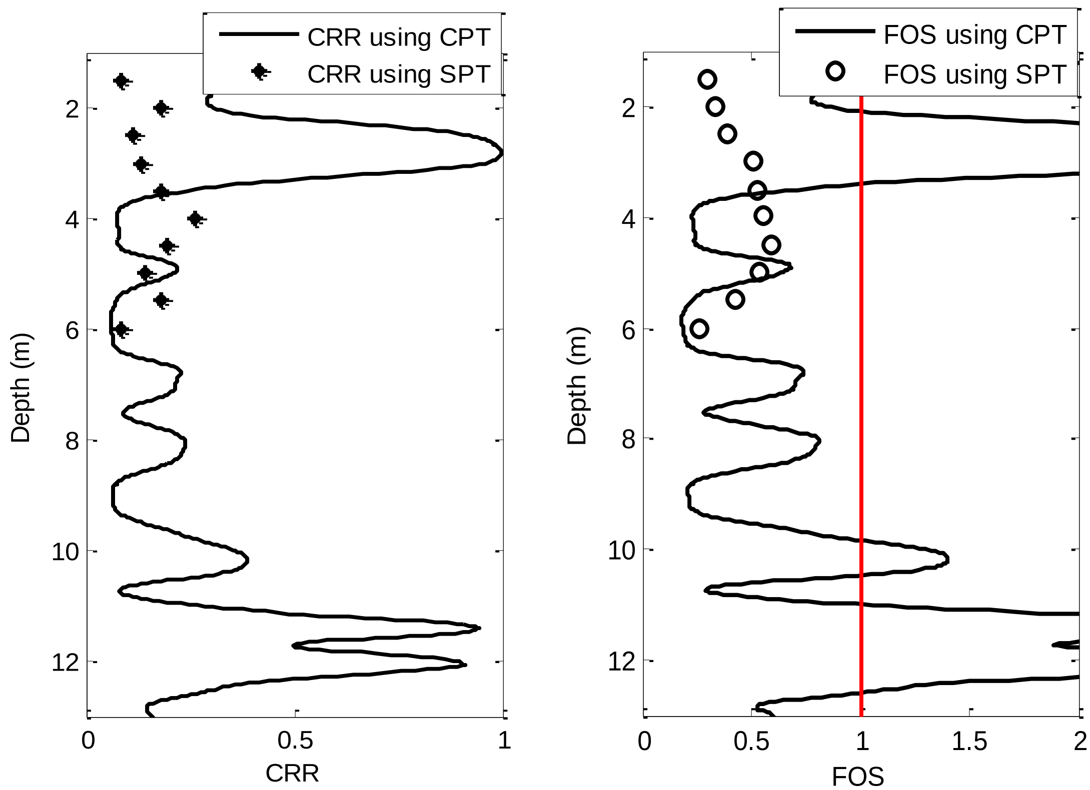


Fig. 5.14 Comparison of CRR and FOS obtained by using CPTu and SPT (DEQ site)

As per liquefaction is considered there is no liquefaction at most of the depths as established by soil profiling and both the methods predict practically similar results.

The values of CRR and FOS from the two approaches have significant difference in 2 m to 3.5 m. While CPTu gives a FOS of greater than 2 and predicts no liquefaction, SPT gives pretty low value and predicts occurrence of liquefaction.

5.5 Site 4: Solani Kunj

5.5.1 Liquefacation Analysis using CPTu

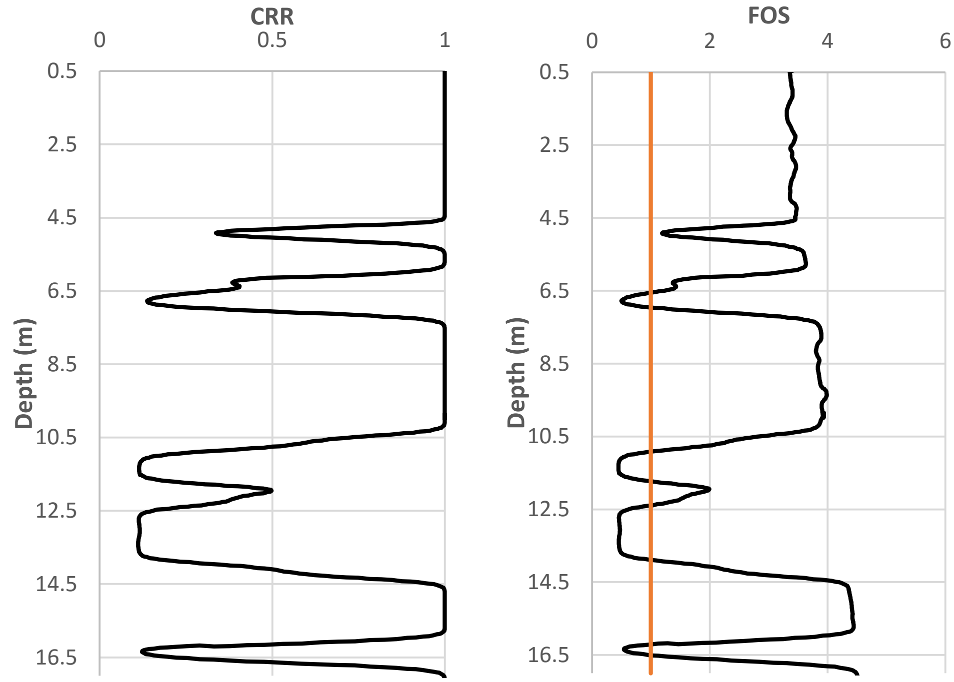


Fig. 5.15 CRR, FOS variation with the Depth using CPTu (Solani kunj site)

Liquefaction analysis by CPTu showed soil to be liquefiable up to 4 m and after that it is non-liqefiable at all depths.

Table 5.7 CSR, CRR and FOS using CPTu (Solani kunj site)

Depth(m)	q_c(Mpa)	f_s(kPa)	σ_{vo} (kPa)	σ'_{vo} (kPa)	Q	F	I_c	CSR	CRR	FOS
0.74	6.87	445	16	8	829.70	6.49	2.31	0.30	1.00	3.38
1.24	6.80	445	26	14	489.50	6.57	2.37	0.29	1.00	3.39
2.25	7.92	520	48	26	306.88	6.60	2.41	0.29	1.00	3.45
2.50	5.92	394	52	27	214.10	6.72	2.49	0.29	1.00	3.40
4.25	6.01	438	90	47	125.63	7.39	2.59	0.29	1.00	3.48
6.00	10.07	729	131	71	139.22	7.34	2.51	0.27	0.86	3.14
8.00	12.41	948	178	98	124.45	7.76	2.52	0.26	1.00	3.83

5.5.2 Liquefaction Analysis using SPT

Table 5.8 Depth wise Calculation of CSR, CRR (Solani kunj Site)

Depth(m)	N	σ_{vo} (kPa)	σ'_{vo} (kPa)	C_N	$(N_1)_{60}$	(N_{1cs})	CSR	CRR	FOS
0.75	5.00	16	8	1.70	8.50	13.34	0.30	0.14	0.47
1.25	8.00	26	14	1.70	13.60	19.45	0.30	0.21	0.70
2.25	8.00	48	26	1.70	13.60	19.76	0.29	0.21	0.72
2.50	5.00	52	27	1.70	8.50	14.52	0.29	0.16	0.55
4.25	14.00	90	47	1.46	20.39	29.27	0.29	0.42	1.45
6.00	10.00	131	71	1.18	11.84	18.53	0.27	0.20	0.74
8.00	17.00	178	98	1.01	17.15	24.78	0.26	0.29	1.12

CRR and FOS obtained from SPT values were low and gradually increasing with depth SPT data was available only up to 8 m. CPTu on the other hand gave significantly high values of CRR and FOS (no liquefaction) up to 4 m and 7 m to 11 m. this discrepency can be attributed to the fact that N values predict soil to be medium dense while tip resistance indicate dense silt in the corrosponding zone.

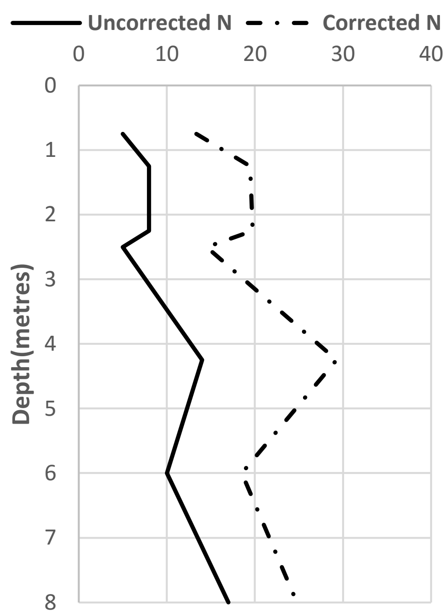
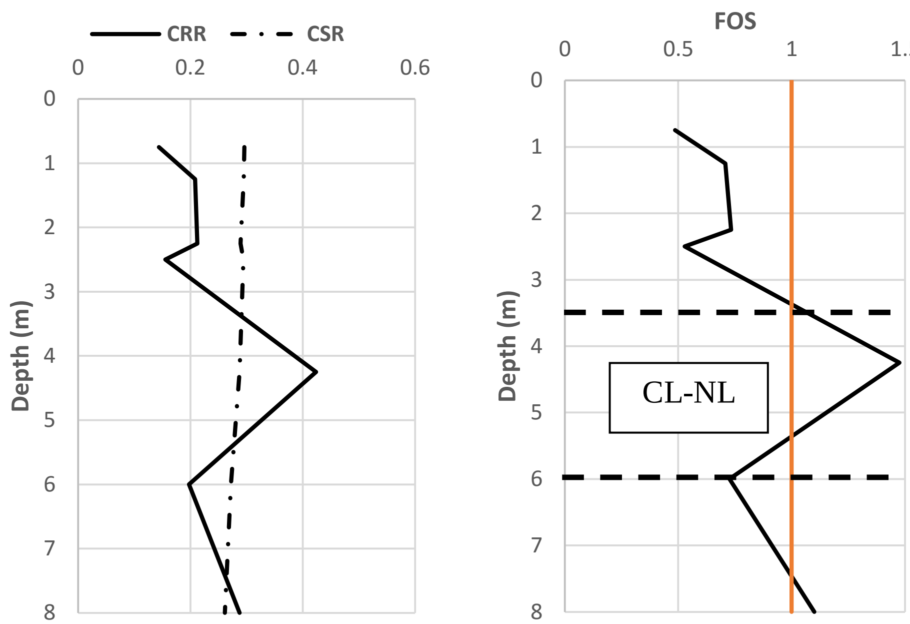


Fig. 5.16 Uncorrected & Corrected N Values with Depth (Solani kunj site)



CL-NL = Clay--No Liquefaction

Fig. 5.17 CSR,CRR and FOS variation with Depth using SPT (Solani kunj site)

Liquefation occurred up to 3.5 m. After that clay layer was encountered up to 6 m.

5.5.3 Comparison of liquefaction resistance fron CPTu and SPT

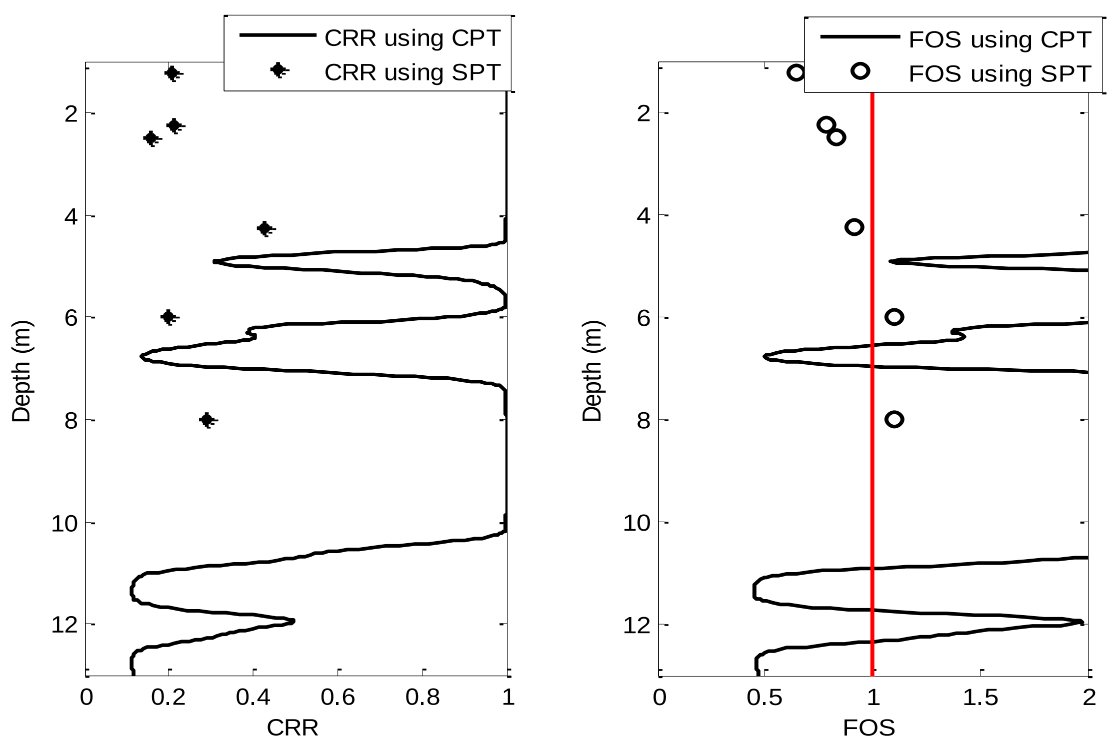


Fig. 5.18 Comparison of CRR and FOS obtained by using CPTu and SPT (Solani kunj)

5.6 Summary and Concluding Remarks

Out of all the sites, site 1 and site 2 was found to be more liquefiable than the rest of the sites.

Site 1 has two 4 m of liquefiable sand layers.

Site 2 consists mostly loose, compressible sand up to 9m which makes it highly liquefiable.

Site 3 and 4 were fine grained soils with high plasticity and had a very few zones of liquefiable layers.

Site 3 has silty clay at most of the depths which is non-liquefiable

Site 4 except for the initial 4m depth was non-liquefiable at all depth due to the presence of alternate clay and dense sand layers.

Following conclusions were drawn from this chapter:

- Liquefaction potential evaluated from both the approaches showed good resemblance for the three out of four sites.
- SPT gave lower liquefaction potential at initial depths while CPTu stated off with higher values. It can be attributed to different methodologies used in the two approaches.
- Comparison of liquefaction evaluation from the two approaches reveals that CPTu can be reliably used for liquefaction analysis.

Chapter 6

Comparison of Fines Content using Field and Lab Tests

Many times the exact data obtained by lab analysis of samples procured from the site is not available and conducting such a test is not possible due for some reason like shortage of time. In that case dependence of fines content on Soil Behavior Index can be used to estimate fines percent and its variation with depth. The basic concept used here is that value of soil behavior index used for fines content determination increases with fines. So dependability of fines content on soil behavior index can be used as a tool for estimation of fines with depth.

6.1 Fines Content from CPTu Data

Fines content used in the liquefaction analysis using CPTu was determined from Robertson and Wride (1998) formula (equation 6). To check the validity of the formula and reliability of our liquefaction analysis, fines content were compared against the actual fines content determined from lab analysis of procured samples using SPT for all sites.

If $I_C < 1.26$, $FC\ (\%) = 0$ (6a)

If $1.26 < I_C < 3.5$, $FC\ (\%) = 1.75\ I_C^{3.25} - 3.7$ (6b)

If $I_C > 3.5$, $FC\ (\%) = 100$ (6c)

6.2 Site 1: Convocation Hall

Fines content determined from CPTu and those obtained from the lab tests were plotted in the same figure for comparison. As can be seen in fig. 6.1 up to 6m, fines content from CPTu are more or less constant (about 10%) after which it suddenly increases to >40% which is because of clay presence at this depth (section 4.1). Fines content again decreases after 8m indicating reduction in fines content.

Data points from the lab tests can be seen to follow similar pattern. Low fines content up to 6m and then increasing between 6m to 8m after which it again decreases up to 12m.

So, it can be concluded for this site that the fines content from CPTu and lab tests are in good agreement.

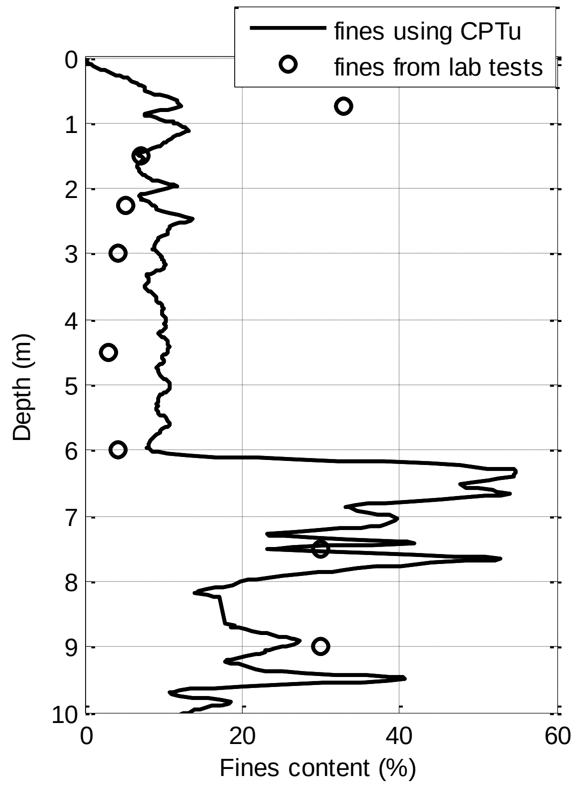


Fig. 6.1 Comparison between fines from CPTu and fines obtained from lab test (Convocation hall site)

6.3 Site 2: Hospital Ground

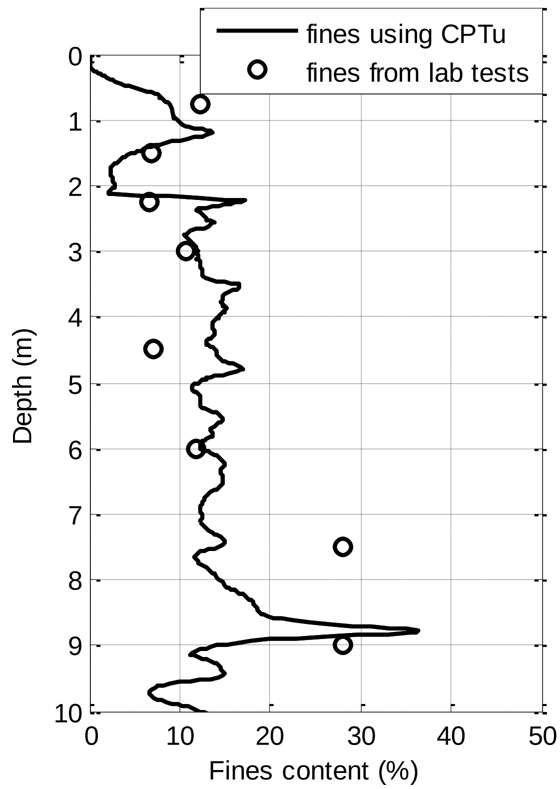


Fig. 6.2 Comparison between fines from CPTu and fines obtained from lab test (Hospital ground site)

The fines content determined from CPTu is consistently less up to 10m (<15%) after which it increases abruptly to 70% which is because of clay inclusion at that depth (section 4.2).

Data points from lab tests follow similar pattern as shown in fig. 6.2

The values of the fines content from both the approaches are close enough at large number of points.

6.4 Site 3: DEQ

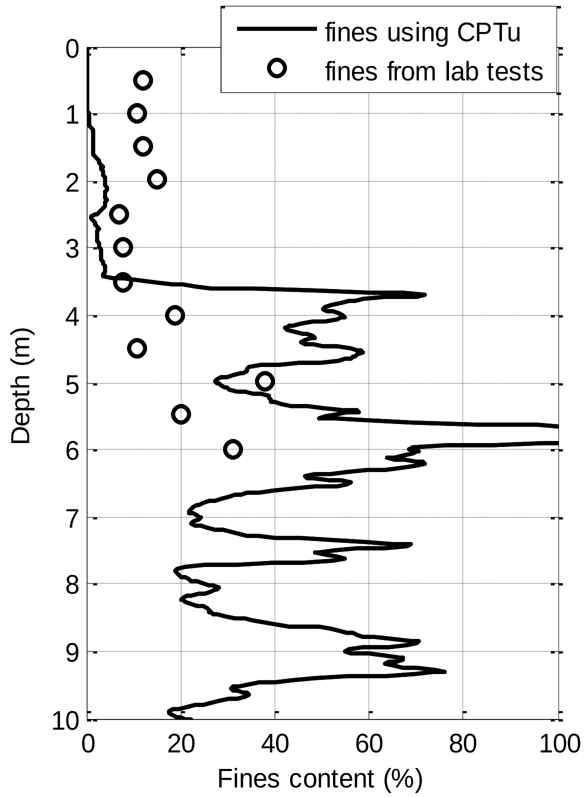


Fig. 6.3 Comparison between fines from CPTu and fines obtained from lab test (DEQ site)

Fines determined from CPTu were less than 15% up to 3.5m. After 3.5m value of fines content increased to 50% up to 4m. The value again reduces to 25% 4m to 6m and then increases abruptly at 6m.

Data points from lab tests show fines content less than 20% up to 4.5m. The values then shows increment to a maximum of 40% at 5m.

It can be concluded for this site that the agreement between both the results is good at initial depths i.e. up to 3.5m. After 3.5m, as finer soil is encountered, even though the difference between the two values increases, pattern followed by both fines content is similar.

6.5 Site 4: Solani Kunj

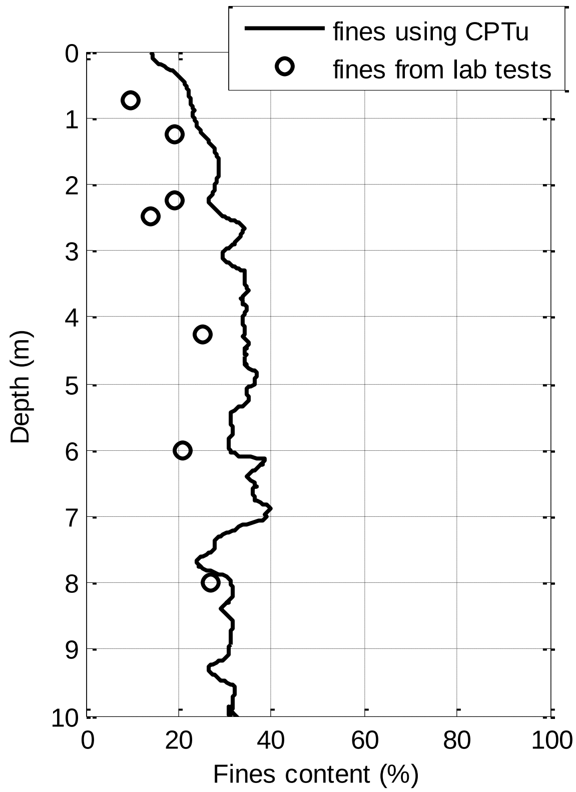


Fig. 6.4 Comparison between fines from CPTu and fines obtained from lab test (Solani kunj Site)

Fines content from CPTu show a gradual increase up to a maximum of 35% at 7m. After 7m the value is fluctuates about 35%.

Data points from lab tests increases gradually with depth reaching to 25% at 8m.

Fines percentage from both approaches show a similar variation with depth. The values from the two approaches are however not matching.

6.5 Summary and Concluding Remarks

On comparing the fines content variation of all the sites it was found that site 1 and site 2 showed comparatively lower percentage of fines than site 3 and site 4. Also for site 1 and site 2 the comparison of fines estimated by Robertson and Wride's (1998) formula against actual fines content was found to be in better agreement than site 3 and site 4.

Site 3 and site 4 has greater percentage of fines which gradually increases with depth.

Of all the four sites, site 2 has minimum fines content while site 3 has maximum fines content.

Following conclusions were drawn from this chapter:

- Fines content determined using Robertson and Wride's (1998) formula were in close agreement with actual fines content.
- The agreement was more precise for sandy soils as compared to silt clay mixtures.
- In the absence of actual data, Robertson's formula can be dependably used for the estimation of fines content.

Chapter 7

Conclusions

A summary of the present research work on liquefaction potential of four different locations in IIT Roorkee Campus is presented. The major conclusions of the study are discussed in this Chapter.

1. In this study soil profiling and liquefaction analysis has been done by code developed in MATLAB based on Seed and Idriss (1971) approach for SPT data and Robertson and Wride (1998) for CPT data. This code can further be used for others sites.
2. CPTu proved to be successful for soil profiling at all the four sites. The results from CPTu and SPT were in good agreement. Being faster and more convenient as compared to SPT it can be reliably used for soil profiling. CPTu was able to detect variations in soil over small depths which were passed unnoticed by SPT giving a more gradual but misleading profile.
3. Tip resistance can be used to differentiate dense from loose soils. Average tip resistance of 5 MPa was observed as a separating value to distinguish stiff and loose soils.
4. It was observed that friction ratio increases with increase in percentage of fines. Friction ratio of 5% and 10 % were established as boundaries for silty and clayey soils.
5. There is no clear cut boundary line to delineate plastic and non plastic fines in SPT while in CPTu; friction ratio, pore water pressure and Soil behavior type index gives a more clear picture of soil.
6. The results of liquefaction analysis from CPTu were in fair agreement with the results from SPT. For rapid study, CPTu can be successfully used for liquefaction analysis of soil.
7. Fines content from Robertson's formula was in close agreement with actual fines content. The formula gave better results for coarse grained soil as compared to fine grained soils. In absence of actual fines content, Robertson's formula for evaluating fines content can be reliably used for determining depth wise fines content of the soil.

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