University of Anbar
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CHAPTER TWO

SUBSOIL EXPLORATION (SITE INVESTIGATION)

LECTURE
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2.1 Introduction

To design a foundation that will support adequate structural load, one must understand the natural of the soils that will support the foundation. The process of determining the layers of natural soil deposits that will underlie a proposed structure and their physical properties is generally referred to as **subsurface exploration.** We will divide these techniques into three categories:

1- **Site investigation-** includes methods of defining the soil profile and other relevant data and recovering soil samples.

2- **Laboratory testing** – includes testing the soil samples in order to determine relevant engineering properties.

3- **In-site testing** – includes methods of testing the soil-in-place, thus avoiding the difficulties associated with recovering samples.

2.2 Purpose of Subsurface Exploration

The purpose of subsurface exploration is to obtain information that will aid the geotechnical engineer in following:

1. Selecting the type and depth of foundation suitable for a given structure.

2. Evaluating the load-bearing capacity of the foundation.

3. Estimating the probable settlement of a structure.

4. Determining the nature of soil at the site and its stratification.

5. Determining potential foundation problems (for example, expansive soil, collapsible soil, sanitary landfill, etc…).

6. Determining the location of the water table.

7. Determining the depth and nature of bedrock, if and when encountered.

8. Performing some in situ field tests, such as permeability tests, van shear test, and standard penetration test.

9. Predicting the lateral earth pressure for structures such as retaining walls, sheet pile bulkheads, and braced cuts.

10. Establishing construction methods for changing subsoil conditions.

2.3 Subsurface Exploration Program

Subsurface exploration comprises several steps, including the collection of preliminary information, reconnaissance, and site investigation.

2.3.1 Collection of Preliminary Information

This step involves obtaining information regarding the type of structure to be built and its general use. For the construction of buildings, the approximate column loads and their spacing and the local building-code and basement requirements should be known. The construction of bridges requires determining the lengths of their spans and the loading on piers and abutments.

A general idea of the topography and the type of soil to be encountered near and around the proposed site can be obtained from the following sources:

1. Geological Survey maps.

2. State government geological survey maps.

3. Agriculture's Soil Conservation Service county soil reports.

4. Agronomy maps published by the agriculture departments of various states.

5. Hydrological information published, including records of stream flow, information on high flood levels, tidal records, and so on.

6. Highway department soil manuals published.

The information collected from these sources can be extremely helpful in planning a site investigation. In some cases, substantial savings may be realized by anticipating problems that may be encountered later in the exploration program.

2.3.2 Reconnaissance

The engineer should always make a visual inspection (field trip) of the site to obtain information about:

1. The general topography of the site, the possible existence of drainage ditches, and other materials present at the site.
2. Evidence of creep of slopes and deep, wide shrinkage cracks at regularly spaced intervals may be indicative of **expansive soil**.

3. Soil stratification from deep cuts, such as those made for the construction of nearby highways and railroads.
4. The type of vegetation at the site, which may indicate the nature of the soil.
5. Groundwater levels, which can be determined by checking nearby wells.
6. The type of construction nearby and the existence of any cracks in walls (**indication for settlement**) or other problems.
7. The nature of the stratification and physical properties of the soil nearby also can be obtained from any available soil-exploration reports on existing structures.
8. High-water marks on nearby buildings and bridge abutments.

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2.3.3 Site Investigation:

This phase consists of:

1. Planning (adopting steps for site investigation, and future vision for the site)
2. Making test boreholes.
3. Collecting soil samples at desired intervals for visual observation and laboratory tests.

Determining the number of boring:

There is no hard-and-fast rule exists for determining the number of borings are to be advanced. For most buildings**, at least one boring at each corner and one at the center** should provide a start. Spacing can be increased or decreased, depending on the condition of the subsoil. If various soil strata are more or less uniform and predictable, fewer boreholes are needed than in nonhomogeneous soil strata.

The following **Table 2.1** gives some guidelines for borehole spacing between for different types of structures:

Table 2.1 Approximate Spacing of Boreholes

Type of project	Spacing (m)
Multistory building	10–30
One-story industrial plants	20–60
Highways	250–500
Residential subdivision	250–500
Dams and dikes	40–80

Determining the depth of boring:

The approximate required minimum depth of the borings should be predetermined. The estimated depths can be changed during the drilling operation, depending on the subsoil encountered (e.g., **Rock**).

To determine the approximate required minimum depth of boring, engineers may use the rules established by the American Society of Civil Engineers (ASCE 1972):

1. Determine the net increase in effective stress ($\Delta\sigma'$) under a foundation with depth as shown in **Fig. 2.1**.
2. Estimate the variation of the vertical effective stress (σ_o') with depth.
3. Determine the depth ($D = D_1$) at which the effective stress increase **($\Delta\sigma'$) is equal to** $(1/10)$ **q** (q = estimated net stress on the foundation).
4. Determine the depth ($D = D_2$) at which **($\Delta\sigma'/\sigma_o'$)** $=\mathbf{0.05}$.
5. Determine the depth ($D = D_3$) which is the distance from the **lower face** of the foundation to **bedrock** (if encountered).
6. Choose the **smaller** of the three depths, (D_1, D_2, and D_3), just determined is the approximate required minimum depth of boring.

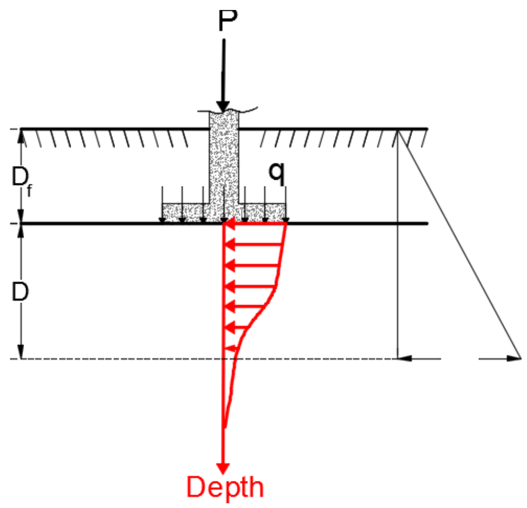

Fig. 2.1 **Foundation with depth**

After determining the value of (D) as explained above the final depth of boring (from the ground surface to the calculated depth) is:

$$D_{boring} = D_f + D$$

Because the Drilling will starts from the ground surface.

If the preceding rules are used, the depths of boring for a building with a width of 30 m (100 ft) will be approximately the following, according to Sowers and Sowers (1970):

No. of stories	Boring depth
1	3.5 m (11 ft)
2	6 m (20 ft)
3	10 m (33 ft)
4	16 m (53 ft)
5	24 m (79 ft)

To determine the boring depth for hospitals and office buildings, Sowers and Sowers (1970) also used the following rules.

$$D_b = 3S^{0.7} \quad \text{(For light steel or narrow concrete buildings)} \qquad (2.1)$$

and

$$D_b = 6S^{0.7} \quad \text{(For heavy steel or wide concrete buildings)} \qquad (2.2)$$

where

D_b= depth of boring

S = number of stories

Notes:

- When deep excavations are anticipated, the depth of boring should be at least 1.5 times the depth of excavation.
- Sometimes, subsoil conditions require that the foundation load be transmitted to bedrock. The minimum depth of core boring into the bedrock is about 3 m (10 ft). If the bedrock is irregular or weathered, the core borings may have to be deeper.

- The engineer should also take into account the ultimate cost of the structure when making decisions regarding the extent of field exploration. The exploration cost generally should be 0.1 to 0.5% of the cost of the structure.

2.4 Exploratory Borings in the Field

Auger boring - is the simplest method of making exploratory boreholes. **Fig. 2.2** shows two types of hand auger: the *posthole auger* and the *helical auger*. Hand augers cannot be used for advancing holes to depths exceeding 3 to 5 m (10 to 16 ft). However, they can be used for soil exploration work on some highways and small structures.

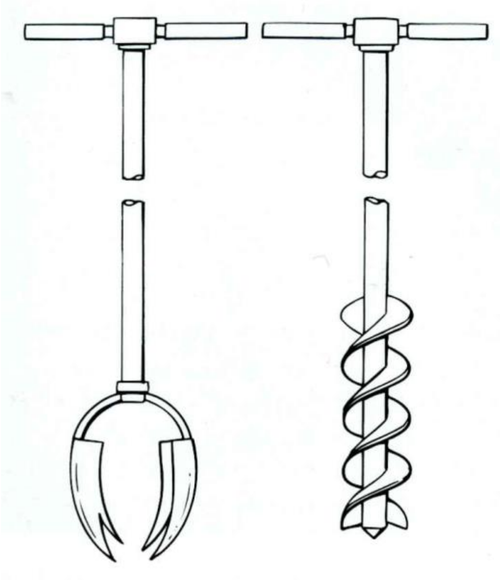

Fig. 2.2 **Hand tools: (a) posthole auger; (b) helical auger**

Portable power- driven helical augers (76 mm to 305 mm in diameter) are available for making deeper boreholes. The soil samples obtained from such borings are highly disturbed. In some non-cohesive soils or soils having low cohesion, the walls of the boreholes will not stand unsupported. In such circumstances, a metal pipe is used as a *casing* to prevent the soil from caving in.

Continuous –flight augers (when power is available), are probably the most common method used for advancing a borehole. The power for drilling is delivered by truck- or tractor-mounted drilling rigs. Boreholes up to about 60 to 70 m can easily be made by this method. Continuous-flight augers are available in sections of about 1 to 2 m with either a solid or hollow stem. Some of the commonly used solid-stem augers have outside diameters of 66.68 mm d, 82.55 mm, 101.6 mm, and 114.3 mm. Common commercially available hollow-stem augers have dimensions of 63.5 mm ID and 158.75 mm OD, 69.85 mm ID and 177.8 OD, 76.2 mm ID and 203.2 OD, and 82.55 mm ID and 228.6 mm OD. The tip of the auger is attached to a cutter head (Fig. 2.3). During the drilling operation (Fig. 2.4), section after section of auger can be added and the hole extended downward. The flights of the augers bring the loose soil from the bottom of the hole to the surface. The driller can detect changes in the type of soil by noting changes in the speed and sound of drilling. When solid-stem augers are used, the auger must be withdrawn at regular intervals to obtain soil samples and also to conduct other operations such as standard penetration tests. Hollow-stem augers have a distinct advantage over solid-stem augers in that they do not have to be removed frequently for sampling or other tests. As shown schematically in Fig. 2.5, the outside of the hollow-stem auger acts as a casing.

The hollow-stem auger system includes the following components:

Outer component: (a) hollow auger sections, (b) hollow auger cap, and (c) drive cap

Inner component: (a) pilot assembly, (b) center rod column, and (c) rod-to-cap adapter

The auger head contains replaceable carbide teeth. During drilling, if soil samples are to be collected at a certain depth, the pilot assembly and the center rod are removed. The soil sampler is then inserted through the hollow stem of the auger column.

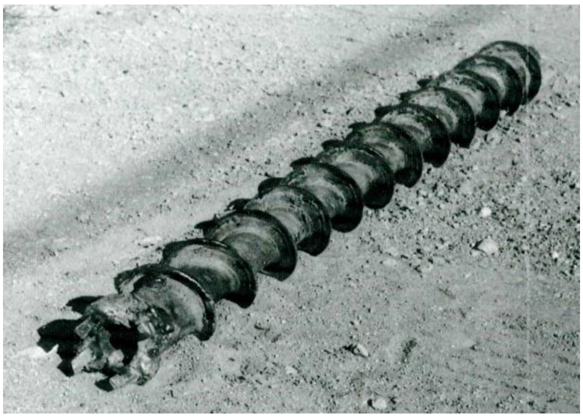

Fig. 2.3 **Carbide-tipped cutting head on auger flight (*Courtesy of Braja M. Das, Henderson, Nevada*)**

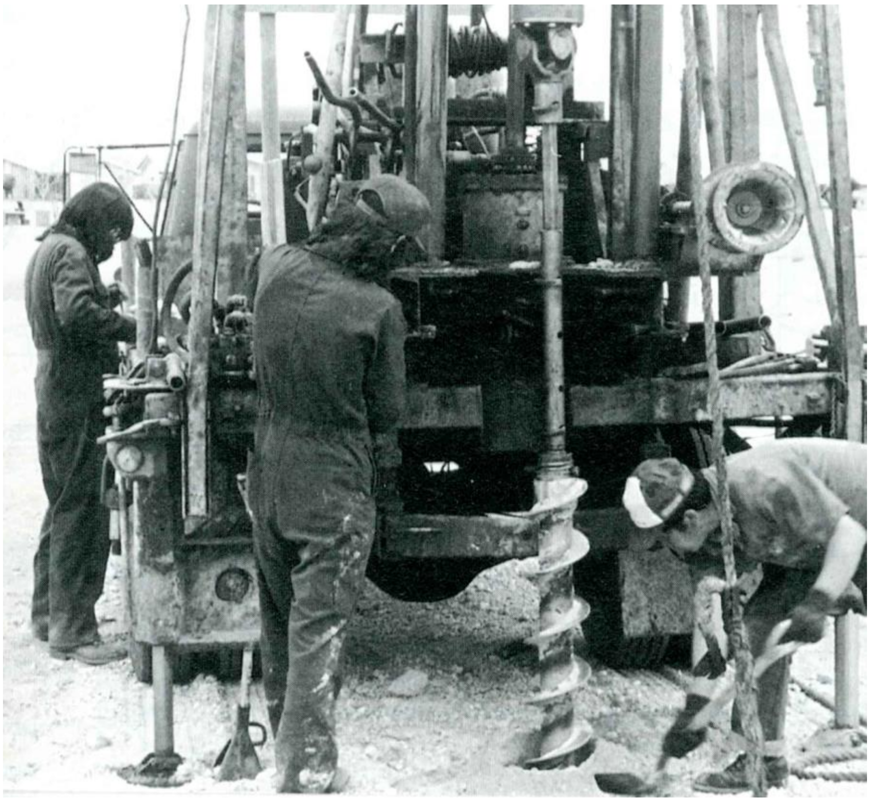

Fig. 2.4 Drilling wit continuous-flight augers

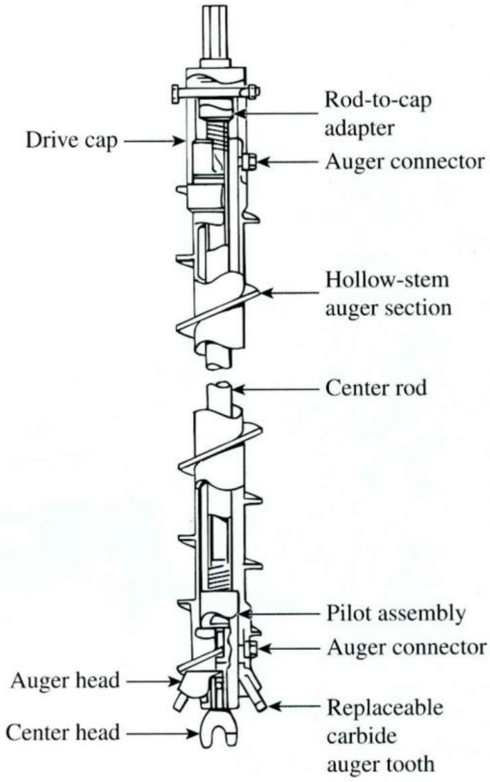

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Fig 2.5 **Hollow-stem auger components (After ASTM, 2001) (Based on ASTM D4700-91: Standard Guide for Soil Sampling from the Vadose Zone.)**

Wash boring - is another method of advancing boreholes. In this method, a casing about 2 to 3 m long is driven into the ground. The soil inside the casing is then removed by means of a chopping bit attached to a drilling rod. Water is forced through the drilling rod and exits at a very high velocity through the holes at the bottom of the chopping bit (Fig. 2.6). The water and the chopped soil particles rise in the drill hole and overflow at the top of the casing through a T connection. The washwater is collected in a container. The casing can be extended with additional pieces as the borehole progresses; however, that is not required if the borehole will stay open and not cave in. Wash borings are rarely used now in the United States and other developed countries.

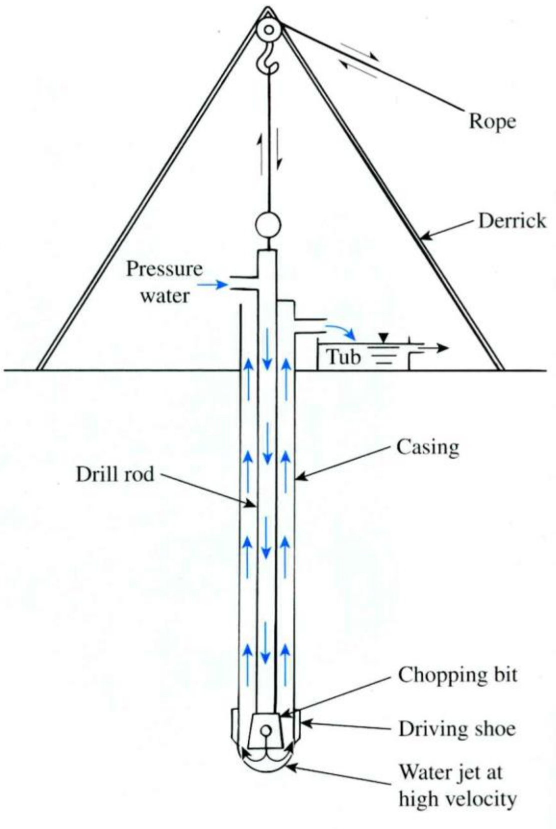

Fig 2.6 **Wash boring**

Rotary drilling is a procedure by which rapidly rotating drilling bits attached to the bottom of drilling rods cut and grind the soil and advance the borehole. There are several types of drilling bit. Rotary drilling can be used in sand, clay, and rocks (unless they are badly fissured). Water or *drilling mud* is forced down the drilling rods to the bits, and the return flow forces the cuttings to the surface. Boreholes with diameters of 50 to 203 mm can easily be made by this technique. The drilling mud is a slurry of water and bentonite. Generally, it is used when the soil that is encountered is likely to cave in. When soil samples are needed, the drilling rod is raised and the drilling bit is replaced by a sampler. With the environmental drilling applications, rotary drilling with air is becoming more common.

Percussion drilling is an alternative method of advancing a borehole, particularly through hard soil and rock. A heavy drilling bit is raised and lowered to chop the hard soil. The chopped soil particles are brought up by the circulation of water. Percussion drilling may require casing.

2.5 Procedures for Sampling Soil

There are two types of samples:

Disturbed Samples - These types of samples are disturbed but representative, and may be used for the following types of laboratory soil tests:

- Grain size analysis.
- Determination of liquid and plastic limits.
- Specific gravity of soil solids.
- Determination of organic content.
- Classification of soil.
- But disturbed soil samples cannot be used for consolidation, hydraulic conductivity, or shear tests, because these tests must be performed on the same soil of the field without any disturbance (to be representative).

The major equipment used to obtain disturbed sample is (Split Spoon) which is a steel tube has inner diameter of 34.93 mm and outer diameter of 50.8mm.

Undisturbed Samples - These types of samples are used for the following types of laboratory soil tests:

- Consolidation test.
- Hydraulic Conductivity test.
- Shear Strength tests.

These samples are more complex and expensive, and it's suitable for **clay**, however in **sand** is very difficult to obtain undisturbed samples. The major equipment used to obtain undisturbed sample is (Thin-Walled Tube).

2.6 Type of Samplers

2.6.1 Split –Spoon Sampling

Split-spoon samplers can be used in the field to obtain soil samples that are generally disturbed, but still representative. A section of a *standard split-spoon sampler* is shown in Fig. 2.7a. The tool consists of a steel driving shoe, a steel tube that is split longitudinally in half, and a coupling at the top. The coupling connects the sampler to the drill rod. The standard split tube has an inside diameter of 34.93 mm and an outside diameter of 50.8 mm ; however, samplers having inside and outside diameters up to 63.5 mm and 76.2 mm , respectively, are also available. When a borehole is extended to a predetermined depth, the drill tools are removed and the sampler is lowered to the bottom of the hole. The sampler is driven into the soil by hammer blows to the top of the drill rod. The standard weight of the hammer is 622.72 N, and for each blow, the hammer drops a distance of 0.762 m. The number of blows required for a spoon penetration of three 152.4-mm (6-in.) intervals are recorded. The number of blows required for the last two intervals are added to give the *standard penetration number, N,* at that depth. This number is generally referred to as the *N value* (American Society for Testing and Materials, 2014, Designation D-1586-11). The sampler is then withdrawn, and the shoe and coupling are removed. Finally, the soil sample recovered from the tube is placed in a glass bottle and transported to the laboratory. This field test is called the standard penetration test (SPT). Fig. 2.8a and b show a split-spoon sampler unassembled before and after sampling.

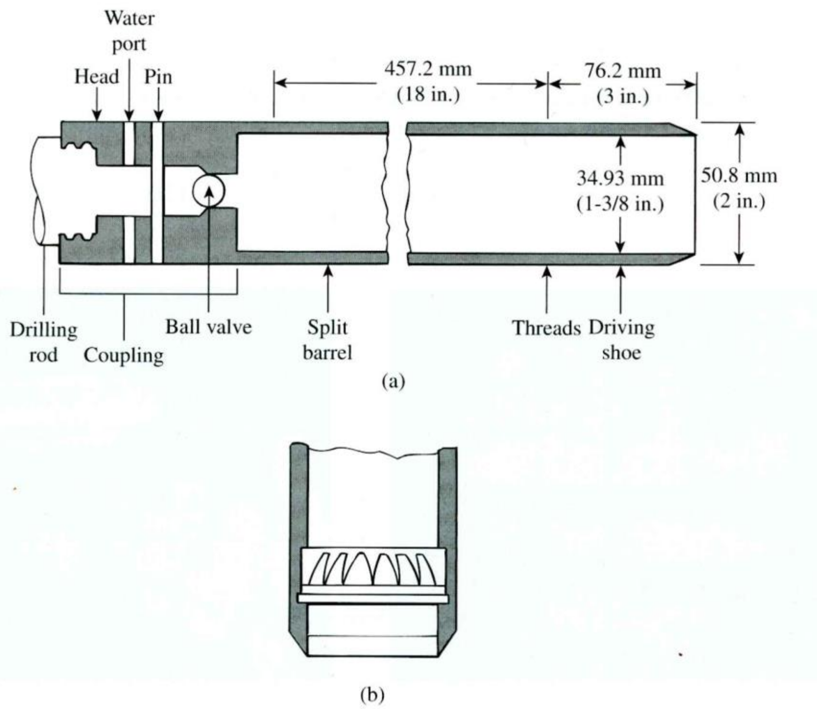

Fig. 2.7 **(a) Standard split-spoon sampler; (b) spring core catcher**

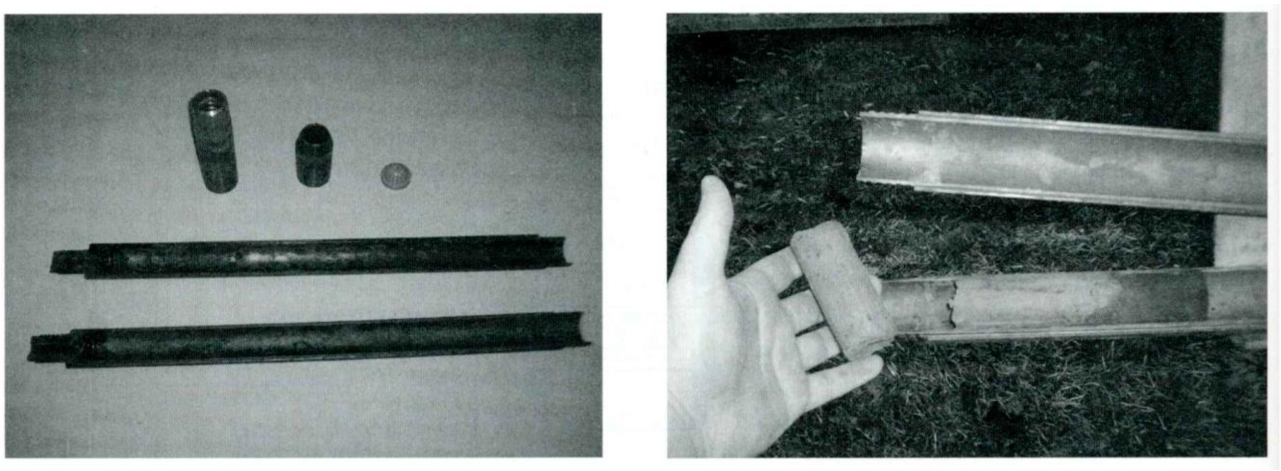

Fig. 2.8 **(a) Unassembled split-spoon sampler; (b) after sampling (*Courtesy of Professional Service Industries, Inc. (PSI), Waukesha, Wisconsin*)**

The degree of disturbance for a soil sample is usually expressed as

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2}(100) \tag{2.3}$$

where

A_R = area ratio (ratio of disturbed area to total area of soil)

D_o = outside diameter of the sampling tube

D_i = inside diameter of the sampling tube

When the area ratio is 10% or less, the sample generally is considered to be undisturbed. For a standard split-spoon sampler,

$$A_R(\%) = \frac{(50.8)^2 - (34.93)^2}{(34.93)^2}(100) = 111.5\%$$

Hence, these samples are highly disturbed. Split-spoon samples generally are taken at intervals of about 1.5 m. When the material encountered in the field is sand (particularly fine sand below the water table), recovery of the sample by a split-spoon sampler may be difficult. In that case, a device such as a *spring core catcher* may have to be placed inside the split spoon (Fig. 2.7b).

At this juncture, it is important to point out that several factors contribute to the variation of the standard penetration number N at a given depth for similar soil profiles. Among these factors are the SPT hammer efficiency, borehole diameter, sampling method, and rod length (Skempton, 1986; Seed, et al., 1985). The two most common types of SPT hammers used in the field are the *safety hammer and donut hammer*. They are commonly dropped by a rope with *two wraps around a pully*.

on the basis of field observations, it appears reasonable to standardize the field penetration number as a function of the input driving energy and its dissipation around the sampler into the surrounding soil, or

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60} \quad (2.4)$$

where

N_{60} = standard penetration number, corrected for field conditions

N = measured penetration number

η_H = hammer efficiency (%)

η_B = correction for borehole diameter

η_S = sampler correction

η_R = correction for rod length

Variations of η_H, η_B, η_S, and η_R, based on recommendations by Seed et al. (1985) and Skempton (1986), are summarized in Table 2.5.

Table 2.5 Variations of η_H, η_B, η_S, and η_R

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter		
mm	in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length		
m	ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

Correlations for N_{60} in Cohesive Soil

Besides compelling the geotechnical engineer to obtain soil samples, standard penetration tests provide several useful correlations. For example, the consistency of clay soils can be estimated from the standard penetration number, N_{60}. In order to achieve that, Szechy and Vargi (1978) calculated the ***consistency index*** **(CI)** as

$$\mathrm{CI} = \frac{\mathrm{LL} - w}{\mathrm{LL} - \mathrm{PL}} \tag{2.5}$$

where

w = natural moisture content (%)

LL = liquid limit

PL = plastic limit

The approximate correlation between CI, N_{60}, and the unconfined compression strength (q_u) is given in Table 2.6.

Table 2.6 Approximate Correlation between CI, N_{60}, and q_u

Standard penetration number, N_{60}	Consistency	CI	Unconfined compression strength, q_u (kN/m²)	Unconfined compression strength, q_u (lb/ft²)
<2	Very soft	<0.5	<25	500
2–8	Soft to medium	0.5–0.75	25–80	500–1700
8–15	Stiff	0.75–1.0	80–150	1700–3100
15–30	Very stiff	1.0–1.5	150–400	3100–8400
>30	Hard	>1.5	>400	8400

Hara, et al. (1971) also suggested the following correlation between the undrained shear strength of clay (c_u) and N_{60}.

$$\frac{c_u}{p_a} = 0.29 N_{60}^{0.72} \qquad (2.6)$$

where p_a = atmospheric pressure (≈< 100 kN/m²; ≈ < 2000 lb/in²).

The overconsolidation ratio, OCR, of a natural clay deposit can also be correlated with the standard penetration number. On the basis of the regression analysis of 110 data points, Mayne and Kemper (1988) obtained the relationship

$$\text{OCR} = 0.193\left(\frac{N_{60}}{\sigma'_o}\right)^{0.689} \qquad (2.7)$$

where σ'_o = effective vertical stress in MN/m2.

It is important to point out that any correlation between cu, OCR, and N60 is only approximate.

Stroud (1974) suggested that

$$c_u = K\,N_{60} \qquad (2.8)$$

K= constant = 3.5 – 6.5 kN/m²

N_{60} = standard penetration number obtained from the field

Correction for N_{60} in Granular Soil

In granular soils, the value of N_{60} is affected by the effective overburden pressure, σ'o. For that reason, the value of N_{60} obtained from field exploration under different effective overburden pressures should be changed to correspond to a standard value of σ'_o. That is,

$$(N_1)_{60} = C_N N_{60} \tag{2.9}$$

where

$(N_1)_{60}$ = value of N_{60} corrected to a standard value of σ'_o = *pa* [<100 kN/m^2 *CN* 5 correction factor

N_{60} = value of N obtained from field exploration

In the past, a number of empirical relations were proposed for C_N. Some of the relationships are given next. The most commonly cited relationships are those of Liao and Whitman (1986) and Skempton (1986).

In the following relationships for C_N, note that σ'_o is the effective overburden pressure and p_a = atmospheric pressure (≈ < 100 kN/m^2)

Liao and Whitman's relationship (1986):

$$C_N = \left[\frac{1}{\left(\frac{\sigma'_o}{p_a}\right)}\right]^{0.5} \tag{2.10}$$

Skempton's relationship (1986):

$$C_N = \frac{2}{1 + \left(\frac{\sigma'_o}{p_a}\right)} \tag{2.11}$$

Correlation between *N*60 and Relative Density of Granular Soil

Kulhawy and Mayne (1990) modified an empirical relationship for relative density that was given by Marcuson and Bieganousky (1977), which can be expressed as

$$D_r(\%) = 12.2 + 0.75\left[222N_{60} + 2311 - 711OCR - 779\left(\frac{\sigma'_o}{p_a}\right) - 50C_u^2\right]^{0.5} \quad (2.12)$$

where

D_r = relative density

σ'_o = effective overburden pressure

C_u = uniformity coefficient of sand

OCR = preconsolidation pressure, σ'_c / effective overburden pressure, σ'_o

p_a = atmospheric pressure

Cubrinovski and Ishihara (1999) also proposed a correlation between N_{60} and the relative density of sand D_r that can be expressed as

$$D_r(\%) = \left[\frac{N_{60}\left(0.23 + \frac{0.06}{D_{50}}\right)^{1.7}}{9}\left(\frac{1}{\frac{\sigma'_o}{p_a}}\right)\right]^{0.5}(100) \quad (2.13)$$

An approximate relationship between the corrected standard penetration number and the relative density of sand is given in **Table 2.8**.

Table 2.8 Relation between the Corrected $(N_1)_{60}$ Values and the Relative Density in Sands

Standard penetration number, $(N_1)_{60}$	Approximate relative density, D_r (%)
0–5	0–5
5–10	5–30
10–30	30–60
30–50	60–95

Correlation between Angle of Friction and Standard Penetration Number

The peak friction angle, ϕ', of granular soil has also been correlated with N_{60} or $(N_1)_{60}$ by several investigators. Some of these correlations are as follows:

1. Peck, Hanson, and Thornburn (1974) give a correlation between N_{60} and ϕ' in a graphical form, which can be approximated as (Wolff, 1989)

$$\phi'(\text{deg}) = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2 \qquad (2.14)$$

2. Schmertmann (1975) provided the correlation between N_{60}, σ'_o, and ϕ'. Mathematically, the correlation can be approximated as (Kulhawy and Mayne, 1990)

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{p_a}\right)}\right]^{0.34} \qquad (2.15)$$

where

N_{60} = field standard penetration number

σ'_o = effective overburden pressure

p_a = atmospheric pressure in the same unit as *s*9*o*

ϕ' = soil friction angle

3. Hatanaka and Uchida (1996) provided a simple correlation between *f*9 and s*N*1d60 that can be expressed as

$$\phi' = \sqrt{20(N_1)_{60}} + 20 \qquad (2.16)$$

The following qualifications should be noted when standard penetration resistance values are used in the preceding correlations to estimate soil parameters:

1. The equations are approximate.

2. Because the soil is not homogeneous, the values of *N*60 obtained from a given borehole vary widely.

3. In soil deposits that contain large boulders and gravel, standard penetration numbers may be erratic and unreliable.

Although approximate, with correct interpretation the standard penetration test provides a good evaluation of soil properties. The primary sources of error in standard penetration tests are inadequate cleaning of the borehole, careless measurement of the blow count, eccentric hammer strikes on the drill rod, and inadequate maintenance of water head in the borehole. Fig. 2.9 shows approximate borderline values for D_r, N_{60}, $(N_1)_{60}$, ϕ' and $(N_1)_{60} / D_r^2$.

	*Very loose	Loose	Medium dense	Dense	Very dense	
#D_r (%)	0	15	35	65	85	100
*N_{60}		4	10	30	50	
##$(N_1)_{60}$		3	8	25	42	
**ϕ'(deg)		28	30	36	41	
##$(N_1)_{60}/D_r^2$			65	59	58	

*Terzaghi & Peck (1948); #Gibb & Holtz (1957); ##Skempton (1986); **Peck et al. (1974)

Fig. 2.9 Approximate borderline values

Correlation between Modulus of Elasticity and Standard Penetration Number

The modulus of elasticity of granular soils (E_s) is an important parameter in estimating the elastic settlement of foundations. A first-order estimation for E_s was given by Kulhawy and Mayne (1990) as

$$\frac{E_s}{p_a} = \alpha N_{60} \tag{2.17}$$

where

p_a = atmospheric pressure (same unit as E_s)

$$\alpha = \begin{cases} 5 \text{ for sands with fines} \\ 10 \text{ for clean normally consolidated sand} \\ 15 \text{ for clean overconsolidated sand} \end{cases}$$

2.6.2 Sampling with a Scraper Bucket

When the soil deposits are sand mixed with pebbles, obtaining samples by split spoon with a spring core catcher may not be possible because the pebbles may prevent the springs from closing. In such cases, a scraper bucket may be used to obtain disturbed representative samples (Fig. 2.10). The scraper bucket has a driving point and can be attached to a drilling rod. The sampler is driven down into the soil and rotated, and the scrapings from the side fall into the bucket.

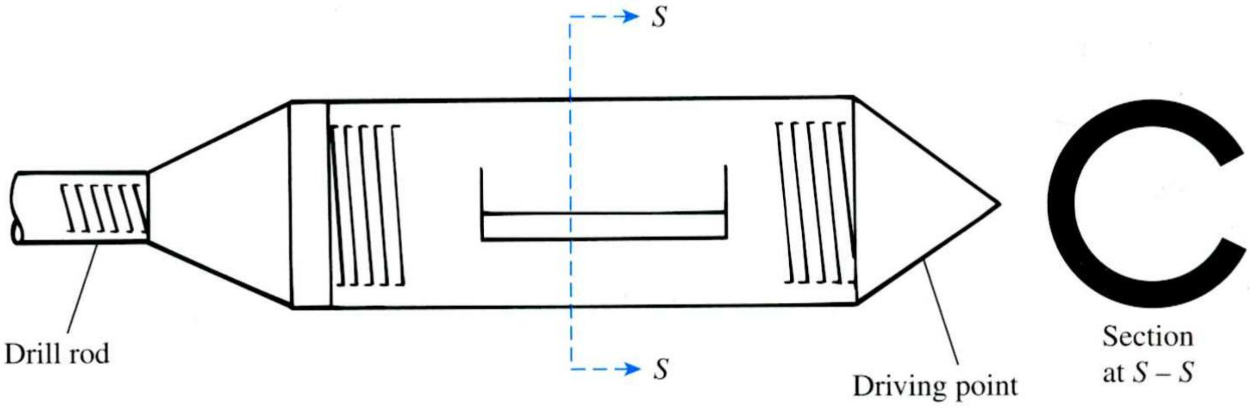

Fig 2.10 Scraper bucket

2.6.3 Sampling with a Thin-Walled Tube

Thin-walled tubes are sometimes referred to as *Shelby tubes*. They are made of seamless steel and are frequently used to obtain undisturbed clayey soils. The most common thin-walled tube samplers have outside diameters of 50.8 mm and 76.2 mm. The bottom end of the tube is sharpened. The tubes can be attached to drill rods (Fig. 2.11). The drill rod with the sampler attached is lowered to the bottom of the borehole, and the sampler is pushed into the soil. The soil sample inside the tube is then pulled out. The two ends are sealed, and the sampler is sent to the laboratory for testing.

Samples obtained in this manner may be used for consolidation or shear tests. A thin-walled tube with a 50.8-mm outside diameter has an inside diameter of about 47.63 mm. The area ratio is

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2}(100) = \frac{(50.8)^2 - (47.63)^2}{(47.63)^2}(100) = 13.75\%$$

Increasing the diameters of samples increases the cost of obtaining them.

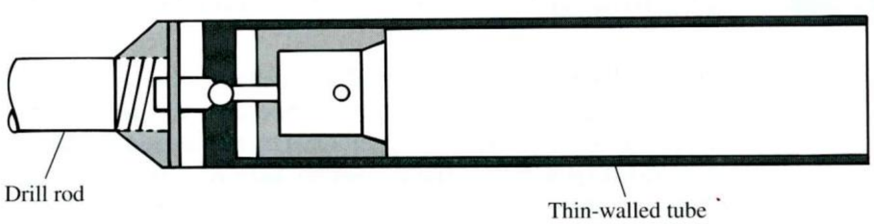

Fig. 2.11 **Thin-walled tube**

2.6.4 Sampling with a Piston Sampler

When undisturbed soil samples are very soft or larger than 76.2 mm in diameter, they tend to fall out of the sampler. Piston samplers are particularly useful under such conditions. There are several types of piston sampler; however, the sampler proposed by Osterberg (1952) is the most useful (see Fig. 2.12a and 2.12b). It consists of a thin walled tube with a piston. Initially, the piston closes the end of the tube. The sampler is lowered to the bottom of the borehole (Fig. 2.12a), and the tube is pushed into the soil hydraulically, past the piston. Then the pressure is released through a hole in the piston rod (Fig. 2.12b). To a large extent, the presence of the piston prevents distortion in the sample by not letting the soil squeeze into the sampling tube very fast and by not admitting excess soil. Consequently, samples obtained in this manner are less disturbed than those obtained by Shelby tubes.

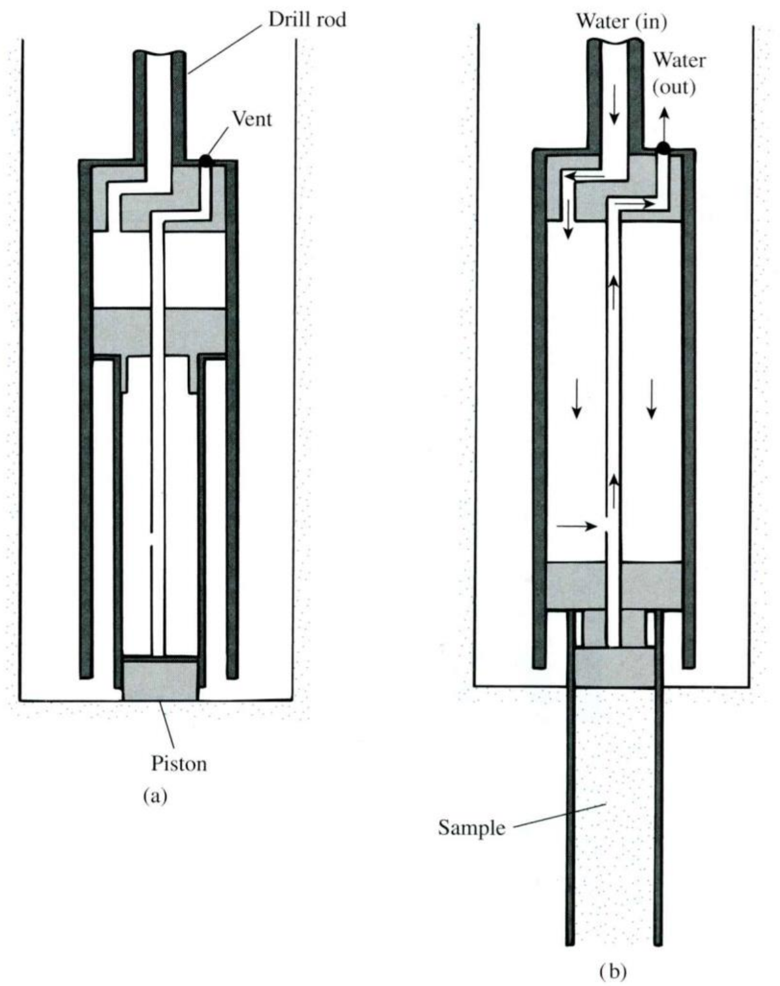

Fig. 2.12 **Piston sampler: (a) sampler at the bottom of borehole; (b) tube pushed into the soil hydraulically**

2.6.5 Coring of Rocks

When a rock layer is encountered during a drilling operation, rock coring may be necessary. To core rocks, a *core barrel* is attached to a drilling rod. A *coring bit* is attached to the bottom of the barrel (Fig. 2.13). The cutting elements may be diamond, tungsten, carbide, and so on. The coring is advanced by rotary drilling. Water is circulated through the drilling rod during coring, and the cutting is washed out.

Two types of core barrel are available: the *single-tube core barrel* (Fig. 2.13a) and the *double-tube core barrel* (Figure 2.13b). Rock cores obtained by single- tube core barrels can be highly disturbed and fractured because of torsion.

When the core samples are recovered, the depth of recovery should be properly recorded for further evaluation in the laboratory. Based on the length of the rock core recovered from each run, the following quantities may be calculated for a general evaluation of the rock quality encountered:

$$\text{Recovery ratio} = \frac{\text{length of core recovered}}{\text{theoretical length of rock cored}} \tag{2.18}$$

$$\text{Rock quality designation (RQD)} = \frac{\Sigma \text{ length of recovered pieces equal to or larger than 101.6 mm (4 in.)}}{\text{theoretical length of rock cored}} \tag{2.19}$$

A recovery ratio of unity indicates the presence of intact rock; for highly fractured rocks, the recovery ratio may be 0.5 or smaller. Table 2.11 presents the general relationship (Deere, 1963) between the RQD and the *in situ* rock quality.

Table 2.11 Relation between *in situ* Rock Quality and RQD

RQD	Rock quality
0–0.25	Very poor
0.25–0.5	Poor
0.5–0.75	Fair
0.75–0.9	Good
0.9–1	Excellent

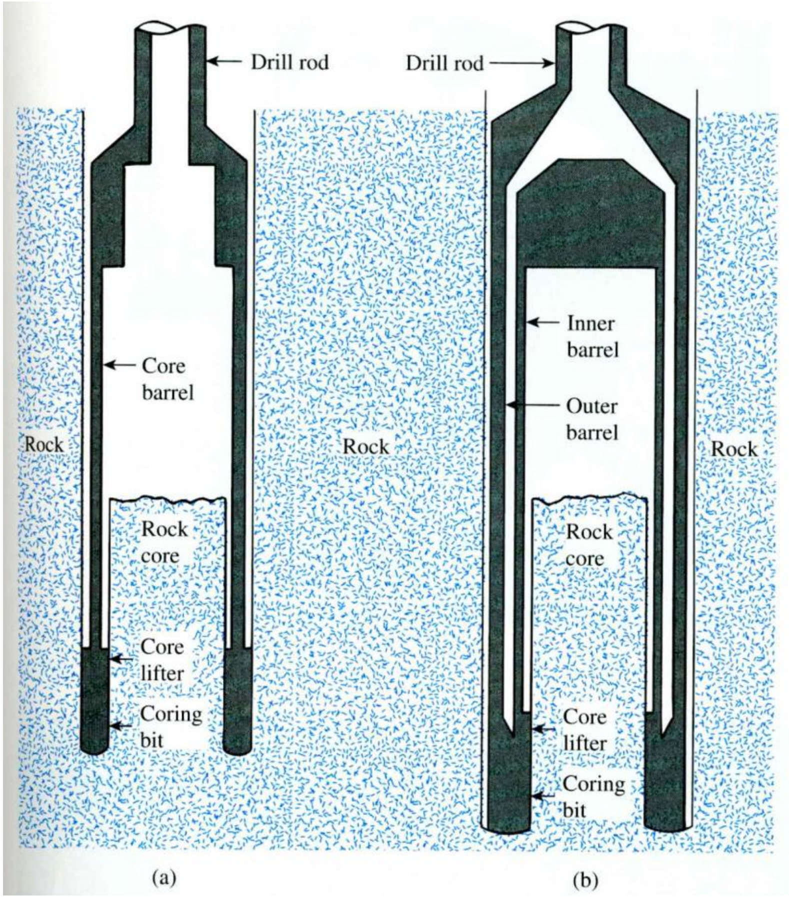

Fig. 2.13 **Rock coring: (a) single-tube core barrel; (b) double-tube core barrel**

2.7 Observation of Water Tables

The presence of a water table near a foundation significantly affects the foundation's load bearing capacity and settlement, among other things. The water level will change seasonally. In many cases, establishing the highest and lowest possible levels of water during the life of a project may become necessary.

In highly impermeable layers, the water level in a borehole may not stabilize for several weeks. In such cases, if accurate water-level measurements are required, a *piezometer* can be used. A piezometer basically consists of a porous stone or a perforated pipe with a plastic standpipe attached to it. Figure 2.14 shows the general placement of a piezometer in a borehole. This procedure will allow periodic checking until the water level stabilizes.

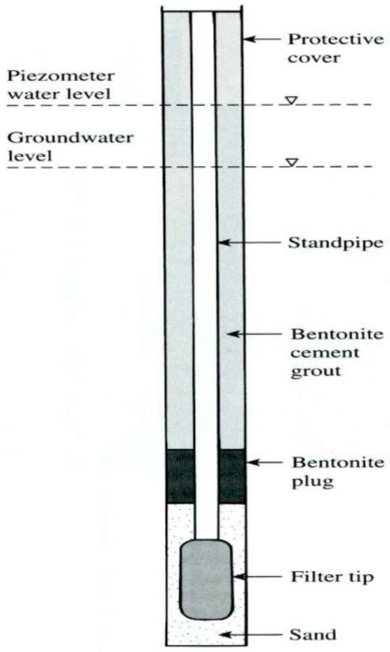

Fig. 2.14 **Casagrande-type piezometer (*Courtesy of N. Sivakugan, James Cook University, Australia.*)**

2.8 Types of in situ or Field Tests

2.8.1 Vane Shear Test

The *vane shear test* (ASTM D-2573) may be used during the drilling operation to determine the *in situ* undrained shear strength s*cu*d of clay soils—particularly soft clays. The vane shear apparatus consists of four blades on the end of a rod, as shown in Fig. 2.15. The height, *H*, of the vane is twice the diameter, *D*. The vane can be either rectangular or tapered (see Figure 2.15). The dimensions of vanes used in the field are given in Table 2.9. The vanes of the apparatus are pushed into the soil at the bottom of a borehole without disturbing the soil appreciably. Torque is applied at the top of the rod to rotate the vanes at a standard rate of 0.18/sec. This rotation will induce failure in a soil of cylindrical shape surrounding the vanes. The maximum torque, *T*, applied to cause failure is measured. Note that

$$T = f(c_u, H, \text{and } D) \qquad (2.\ \ 20)$$

$$c_u = \frac{T}{K} \qquad (2.21)$$

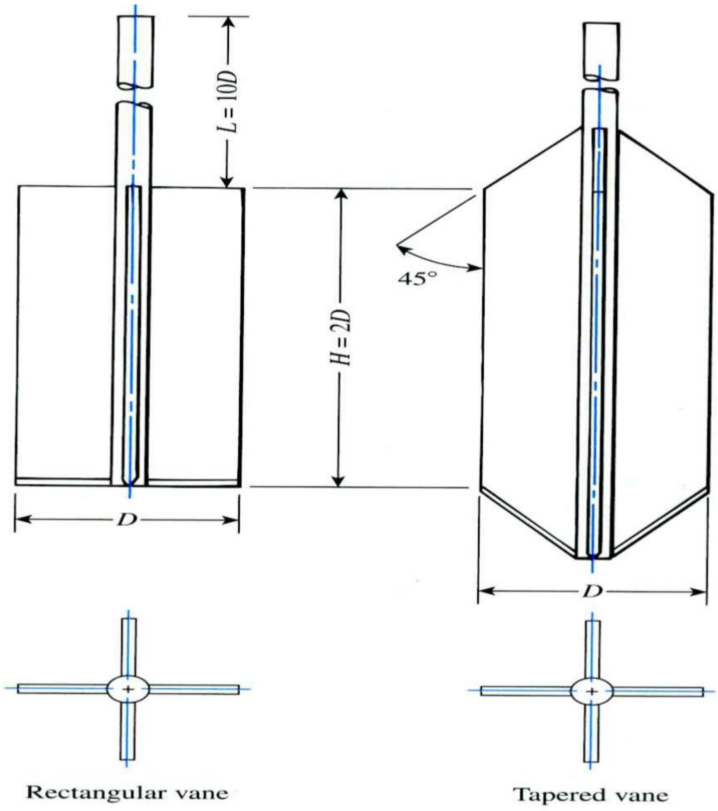

Fig. 2.15 Geometry of field vane (After ASTM, 2014) (Based on *Annual Book of ASTM Standards, Vol. 04.08.*)

Table 2.9 ASTM Recommended Dimensions of Field Vanes[a] (Annual Book of ASTM Standards, Vol. 04.08. Copyright ASTM INTERNATIONAL. Reprinted with permission.)

Casing size	Diameter, *D* mm (in.)	Height, *H* mm (in.)	Thickness of blade mm (in.)	Diameter of rod mm (in.)
AX	38.1 ($1\frac{1}{2}$)	76.2 (3)	1.6 ($\frac{1}{16}$)	12.7 ($\frac{1}{2}$)
BX	50.8 (2)	101.6 (4)	1.6 ($\frac{1}{16}$)	12.7 ($\frac{1}{2}$)
NX	63.5 ($2\frac{1}{2}$)	127.0 (5)	3.2 ($\frac{1}{8}$)	12.7 ($\frac{1}{2}$)
101.6 mm (4 in.)[b]	92.1 ($3\frac{5}{8}$)	184.1 ($7\frac{1}{4}$)	3.2 ($\frac{1}{8}$)	12.7 ($\frac{1}{2}$)

[a]The selection of a vane size is directly related to the consistency of the soil being tested; that is, the softer the soil, the larger the vane diameter should be.
[b]Inside diameter.

Where

T is in N.m, C_u is in kN/m^2, and

K = a constant with a magnitude depending on the dimension and vane

The constant

$$K = \left(\frac{\pi}{10^6}\right)\left(\frac{D^2 H}{2}\right)\left(1 + \frac{D}{3H}\right) \quad (2.22)$$

Where

D = diameter of vane in cm

H = measured height of vane in cm

If H/D = 2, Eq.(2.22) yields

$$K = 366 \times 10^{-8} D^3 \quad (D = cm) \qquad (2.23)$$

For actual design purposes, the undrained shear strength values obtained from field vane shear tests [$c_{u(VST)}$] are too high, and it is recommended that they be corrected according to the equation

$$c_{u(\text{corrected})} = \lambda c_{u(\text{VST})} \qquad (2.24)$$

where

λ = correction factor

Several correlations have been given previously for the correction factor λ. Some more are as follows:

Bjerrum (1972):

$$\lambda = 1.7 - 0.54 \log[\text{PI}(\%)] \qquad (2.25)$$

Morris and Williams (1994) provided the following correlations:

$$\lambda = 1.18e^{-0.08(\text{PI})} + 0.57 \ (\text{for PI} > 5) \qquad (2.26)$$

$$\lambda = 7.01e^{-0.08(\text{LL})} + 0.57 \ (\text{where LL is in } \%) \qquad (2.27)$$

The field vane shear strength can be correlated with the preconsolidation pressure and the overconsolidation ratio of the clay. Using 343 data points, Mayne and Mitchell (1988) derived the following empirical relationship for estimating the preconsolidation pressure of a natural clay deposit:

$$\sigma'_c = 7.04[c_{u(\text{field})}]^{0.83} \qquad (2.28)$$

Here,

σ'_c = preconsolidation pressure (kN/m^2)

$c_{u(\text{field})}$ = field vane shear strength (kN/m^2)

The overconsolidation ratio, OCR, also can be correlated to $c_{u(\text{field})}$ according to the Equation

$$\text{OCR} = \beta \frac{c_{u(\text{field})}}{\sigma'_o} \qquad (2.29)$$

where

σ'_o = effective overburden pressure.

The magnitudes of *b* developed by various investigators are given below.

- **Mayne and Mitchell (1988):**

$$\beta = 22[\mathrm{PI}(\%)]^{-0.48} \tag{2.30}$$

Fig. 2.16 shows the general test procedures for the field vane in fine-grained soils.

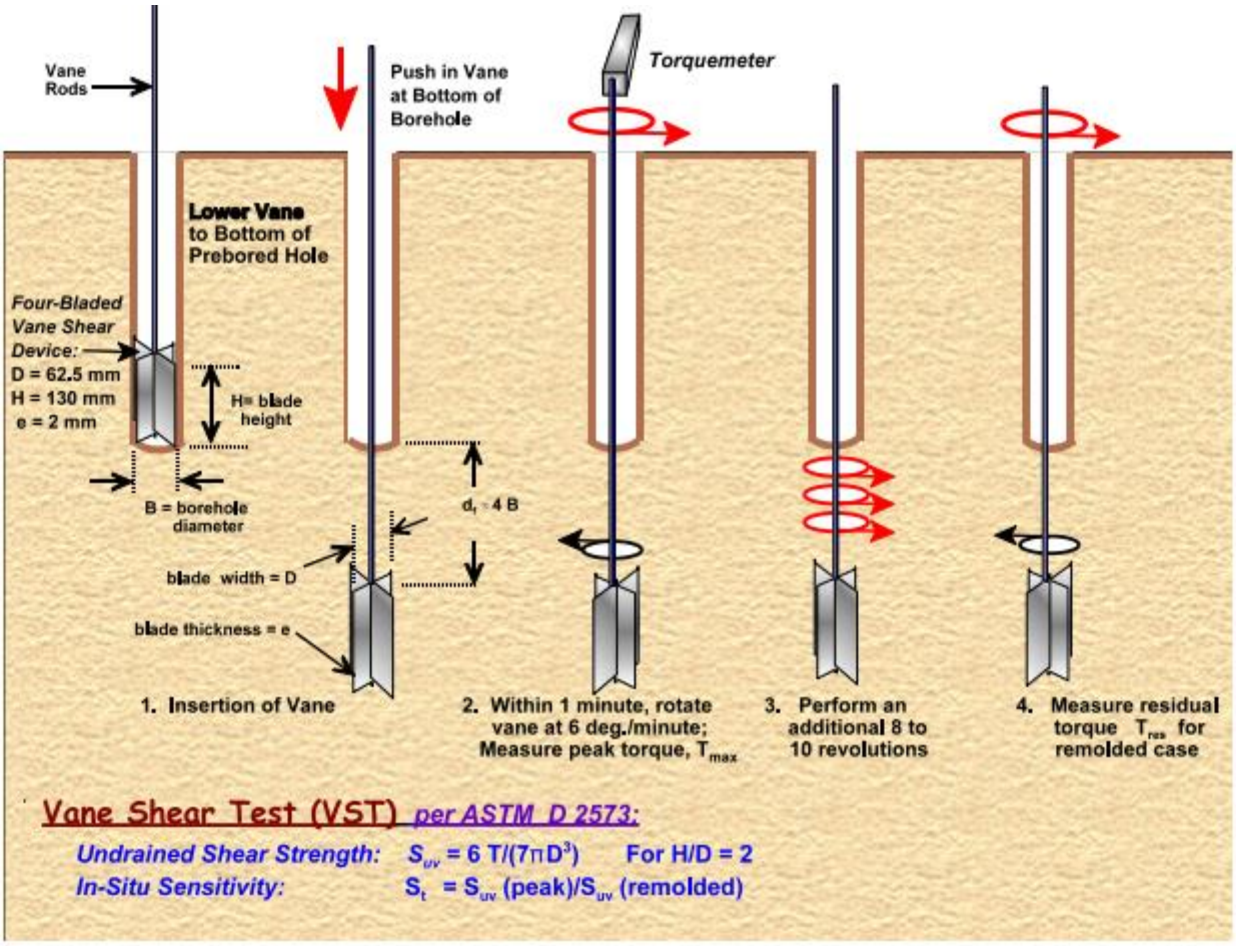

Fig. 2.16 General Test Procedures for the Field Vane in Fine-Grained Soils.

2.8.2 Standard Penetration Test (SPT)

This test is one of the most important soil tests for geotechnical engineers because it's widely used in calculating different factors as will explained later. This test is performed according the following procedures:

1. Determining the required number and depth of boreholes in the site.
2. The **sampler** used in SPT test is (Standard Split Spoon) which has an inside diameter of 34.39 mm and an outside diameter of 50.8 mm.
3. Using drilling machine, **1.5m** are drilled.
4. The drilling machine is removed and the sampler will lowered to the bottom of the hole.
5. The sampler is driven into the soil by hammer blows to the top of the drill rod, the standard weight of the hammer is 622.72 N (63.48 Kg), and for each blow, the hammer drops a distance of 76.2 cm.
6. The number of blows required for a spoon penetration of three 15 cm intervals are recorded.
7. The first 15 cm drive is considered as seating load and is ignored.
8. The number of blows required for the **last two intervals** are added to give the **Standard Penetration Number (N)** at that depth.
9. The sampler is then withdrawn and the soil sample recovered from the tube is placed in a glass bottle and transported to laboratory.
10. Using the drilling machine to drill another 1.5m and then repeat the above steps for each 1.5 m till reaching the specified depth of borehole.
11. Take the average for (N) value from each 1.5 m to obtain the final Standard Penetration Number.
12. Split Spoon samples are taken at intervals (1.5m) because theses samples are highly disturbed.

Fig. 2.17 shows the sequence of driving split-barrel sampler during the standard penetration test.

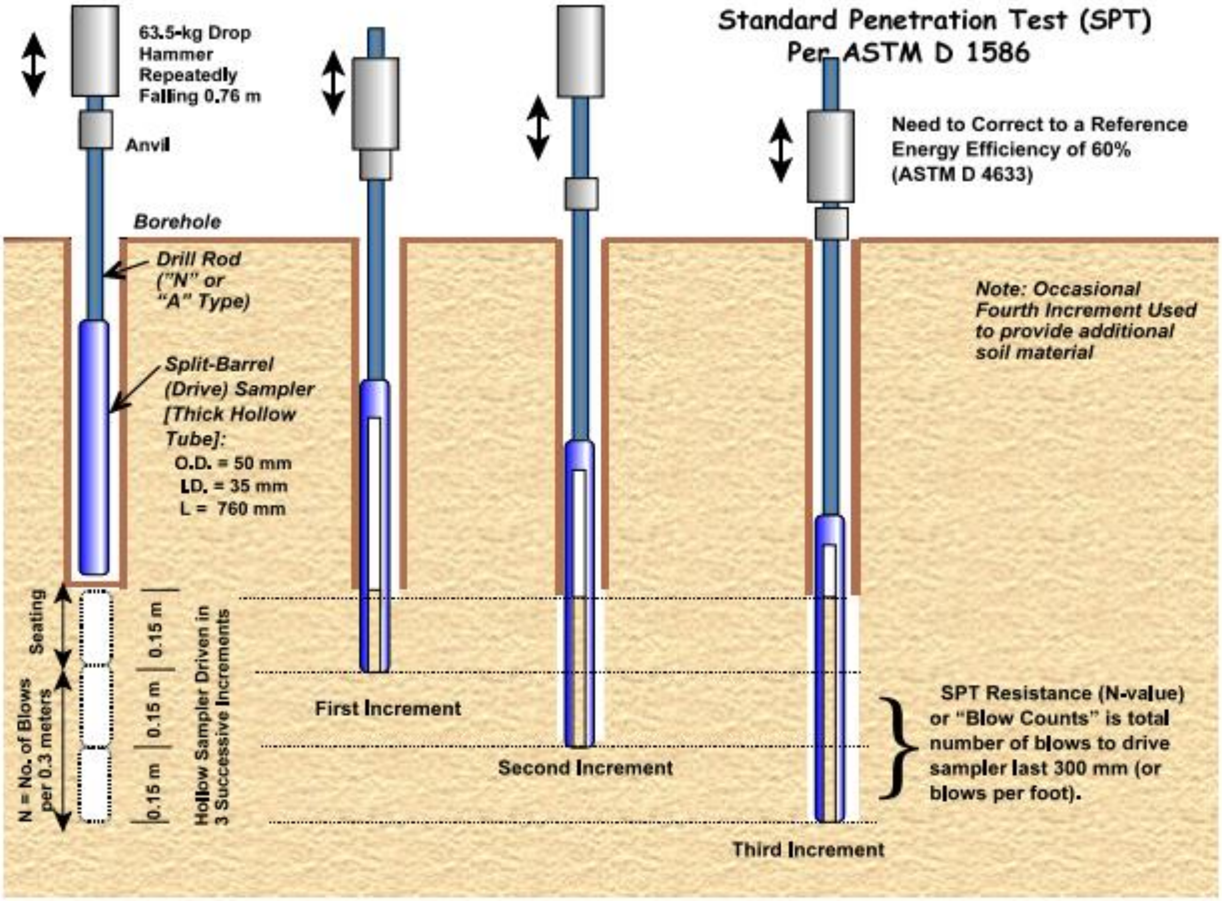

Fig. 2.17 Sequence of Driving Split-Barrel Sampler During the Standard Penetration Test.

2.8.3 Cone Penetration Test

The cone penetration test (CPT), originally known as the Dutch cone penetration test, is a versatile sounding method that can be used to determine the materials in a soil profile and estimate their engineering properties. The test is also called the *static penetration test*, and no boreholes are necessary to perform it. In the original version, a 60^o cone with a base area of 10 cm^2 s1.55 in.2d was pushed into the ground at a steady rate of about 20 mm/sec, and the resistance to penetration (called the point resistance) was measured.

The cone penetrometers in use at present measure (a) the *cone resistance* (q_c) to penetration developed by the cone, which is equal to the vertical force applied to the cone, divided by its horizontally projected area; and (b) the *frictional resistance* (q_c), which is the resistance measured by a sleeve located above the cone with the local soil surrounding it. The frictional resistance is equal to the vertical force applied to the sleeve, divided by its surface area—actually, the sum of friction and adhesion.

Generally, two types of penetrometers are used to measure q_c and f_c:

1. *Mechanical friction-cone penetrometer* (Fig. 2.18). The tip of this penetrometer is connected to an inner set of rods. The tip is first advanced about 40 mm, giving the cone resistance. With further thrusting, the tip engages the friction sleeve. As the inner rod advances, the rod force is equal to the sum of the vertical force on the cone and sleeve. Subtracting the force on the cone gives the side resistance.

2. *Electric friction-cone penetrometer* (Fig. 2.19). The tip of this penetrometer is attached to a string of steel rods. The tip is pushed into the ground at the rate of 20 mm/sec. Wires from the transducers are threaded through the center of the rods and continuously measure the cone and side resistances.

Fig. 2.20 shows the results of penetrometer test in a soil profile with friction measurement by an electric friction-cone penetrometer.

Several correlations that are useful in estimating the properties of soils encountered during an exploration program have been developed for the point resistance (q_c) and the friction ratio (F_r) obtained from the cone penetration tests. The friction ratio is defined as

$$F_r = \frac{\text{frictional resistance}}{\text{cone resistance}} = \frac{f_c}{q_c} \quad (2.31)$$

As in the case of standard penetration tests, several correlations have been developed between q_c and other soil properties. Some of these correlations are presented next.

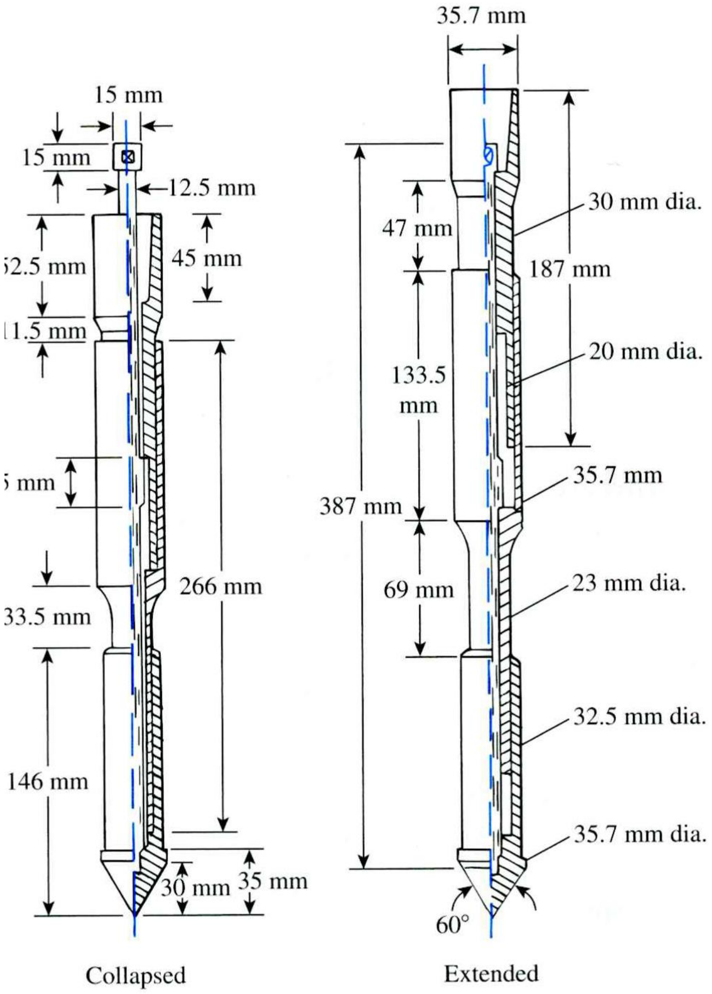

Fig. 2.18 Mechanical friction-cone penetrometer (After ASTM, 2001) (Based on *Annual Book of ASTM Standards, Vol. 04.08.*)

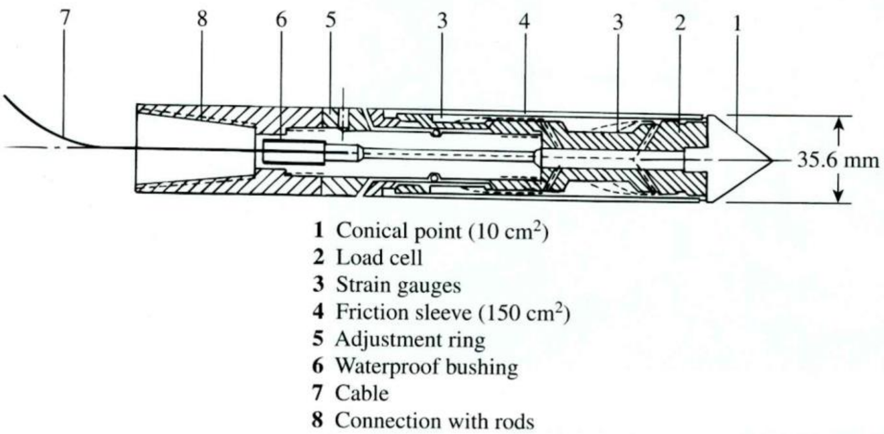

Fig. 2.19 Electric friction-cone penetrometer (After ASTM, 2001) (Based on *Annual Book of ASTM Standards, Vol. 04.08.*)

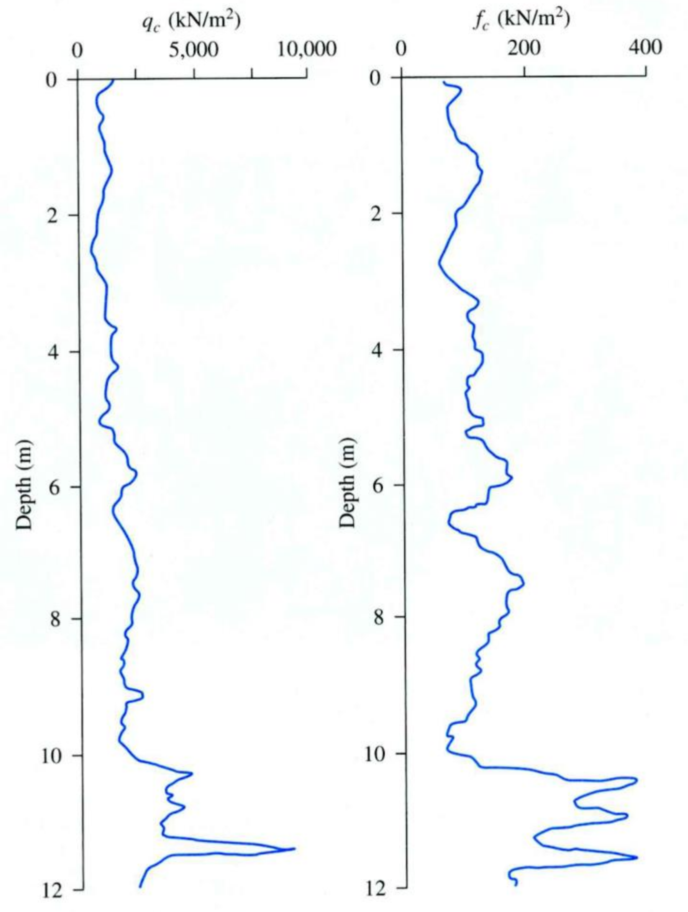

Fig. 2.21 Cone penetrometer test with friction measurement

Correlation between Relative Density (D_r) and q_c for Sand

Kulhawy and Mayne, 1990 proposed the following relationship to correlate D_r, q_c and the vertical effective stress σ'_o :

$$D_r(\%) = 68\left[\log\left(\frac{q_c}{\sqrt{p_a \cdot \sigma'_0}}\right) - 1\right] \qquad (2.32)$$

In this equation,

OCR = overconsolidation ratio

p_a = atmospheric pressure

Q_c = compressibility factor

The recommended values of Q_c are as follows:

Highly compressible sand = 0.91

Moderately compressible sand = 1.0

Low compressible sand = 1.09

Correlation between q_c and Drained Friction Angle (ϕ') for Sand

On the basis of experimental results, Robertson and Campanella (1983) suggested the variation of D_r, σ'_o, and ϕ' for normally consolidated quartz sand. This relationship can be expressed as (Kulhawy and Mayne, 1990)

$$\phi' = \tan^{-1}\left[0.1 + 0.38 \log\left(\frac{q_c}{\sigma'_o}\right)\right] \tag{2.33}$$

Correlations for Undrained Shear Strength (c_u), Preconsolidation Pressure (σ'_c), and Overconsolidation Ratio (OCR) for Clays

The undrained shear strength, c_u , can be expressed as

$$c_u = \frac{q_c - \sigma_o}{N_K} \tag{2.34}$$

where

σ_o = total vertical stress

N_K = bearing capacity factor

The bearing capacity factor, N_K, may vary from 11 to 19 for normally consolidated clays and may approach 25 for overconsolidated clay. According to Mayne and Kemper (1988)

N_K= 15 (for electric cone)

and

N_K = 20 (for mechanical cone)

Mayne and Kemper (1988) provided correlations for preconsolidation pressure (σ'_c) and overconsolidation ratio (OCR) as

$$\underset{\text{MN/m}^2}{\underset{\uparrow}{\sigma'_c}} = 0.243(\underset{\text{MN/m}^2}{\underset{\uparrow}{q_c}})^{0.96} \tag{2.35}$$

and

$$\text{OCR} = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01} \tag{2.36}$$

where σ_o and σ'_o = total and effective stress, respectively.

Fig. 2.22 shows the geometry and measurements taken by cone and piezocone penetrometers.

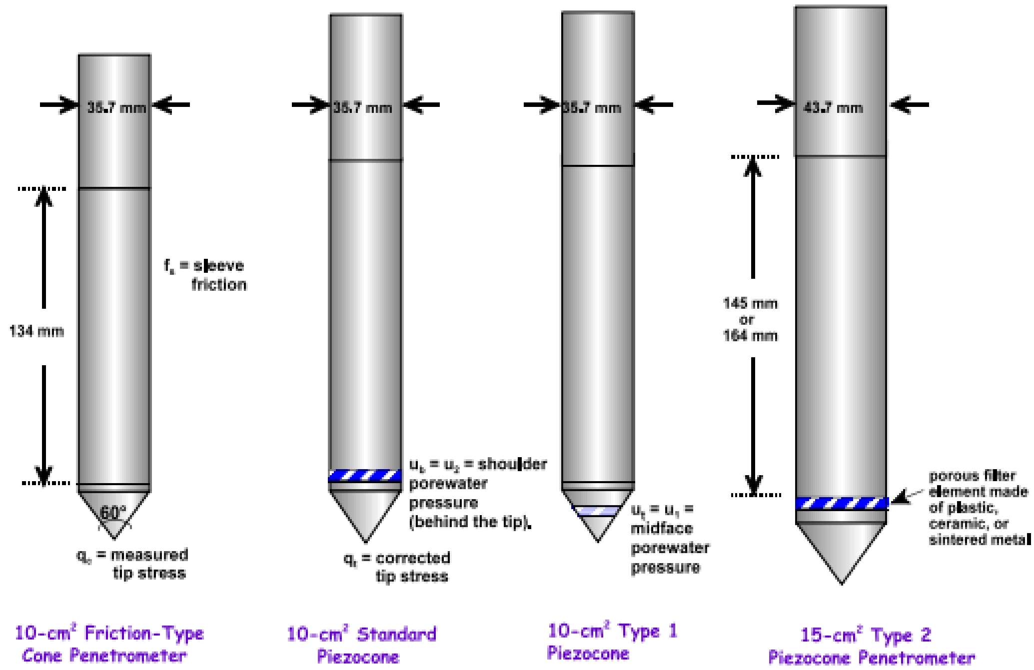

Fig. 2.22 Geometry and Measurements Taken by Cone and Piezocone Penetrometers.

2.8.4 Pressuremeter Test (PMT)

2.8.5 Dilatometer Test

2.9 Preparation of Boring Logs

The detailed information gathered from each borehole is presented in a graphical form called the *boring log*. As a borehole is advanced downward, the driller generally should record the following information in a standard log:

1. Name and address of the drilling company
2. Driller's name
3. Job description and number
4. Number, type, and location of boring
5. Date of boring
6. Subsurface stratification, which can be obtained by visual observation of the soil brought out by auger, split-spoon sampler, and thin-walled Shelby tube sampler

7. Elevation of water table and date observed, use of casing and mud losses, and so on
8. Standard penetration resistance and the depth of SPT
9. Number, type, and depth of soil sample collected
10. In case of rock coring, type of core barrel used and, for each run, the actual length of coring, length of core recovery, and RQD

This information should never be left to memory, because doing so often results in erroneous boring logs.

After completion of the necessary laboratory tests, the geotechnical engineer prepares a finished log that includes notes from the driller's field log and the results of tests conducted in the laboratory. Fig. 2.23 shows a typical boring log. These logs have to be attached to the final soil-exploration report submitted to the client. The figure also lists the classifications of the soils in the left-hand column, along with the description of each soil (based on the Unified Soil Classification System).

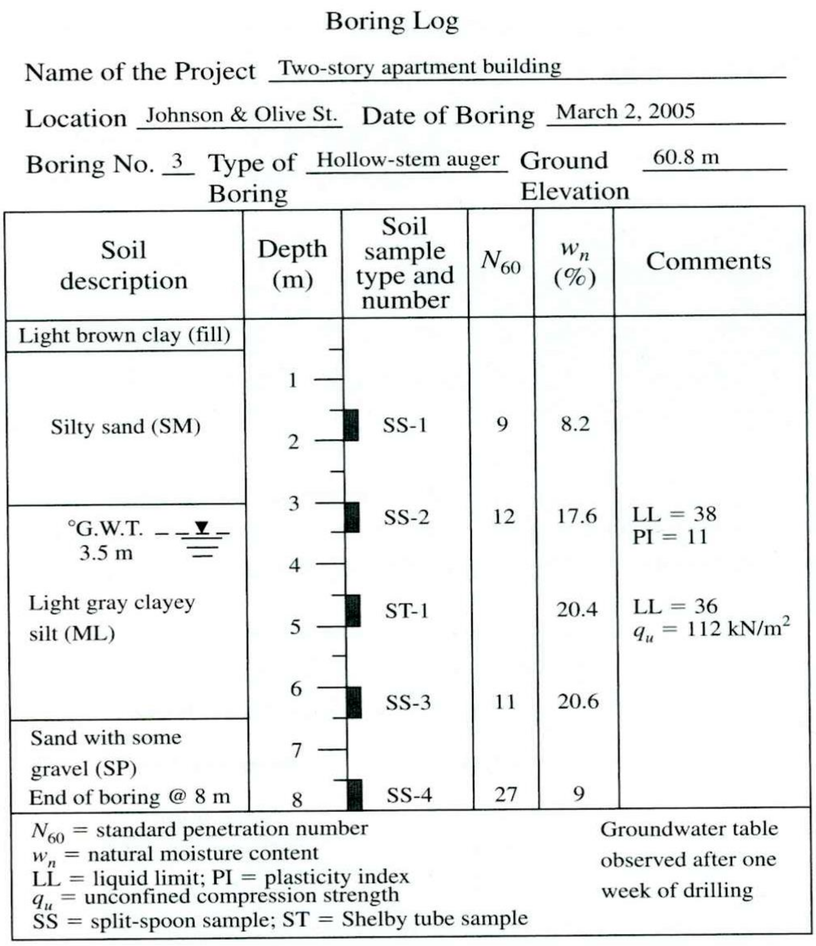

Fig. 2.23 **A typical boring log**

2.10 Subsoil Exploration Report

At the end of all soil exploration programs, the soil and rock specimens collected in the field are subject to visual observation and appropriate laboratory testing. After all the required information has been compiled, a soil exploration report is prepared for use by the design office and for reference during future construction work. Although the details and sequence of information in such reports may vary to some degree, depending on the structure under consideration and the person compiling the report, each report should include the following items:

1. A description of the scope of the investigation

2. A description of the proposed structure for which the subsoil exploration has been conducted

3. A description of the location of the site, including any structures nearby, drainage conditions, the nature of vegetation on the site and surrounding it, and any other features unique to the site

4. A description of the geological setting of the site

5. Details of the field exploration—that is, number of borings, depths of borings, types of borings involved, and so on

6. A general description of the subsoil conditions, as determined from soil specimens and from related laboratory tests, standard penetration resistance and cone penetration resistance, and so on

7. A description of the water-table conditions

8. Recommendations regarding the foundation, including the type of foundation recommended, the allowable bearing pressure, and any special construction procedure that may be needed; alternative foundation design procedures should also be discussed in this portion of the report

9. Conclusions and limitations of the investigations

The following graphical presentations should be attached to the report:

1. A site location map

2. A plan view of the location of the borings with respect to the proposed structures and those nearby

3. Boring logs

4. Laboratory test results

5. Other special graphical presentations

The exploration reports should be well planned and documented, as they will help in answering questions and solving foundation problems that may arise later during design and construction.