# CHAPTER 3

# SUBSURFACE EXPLORATION

Sections : 3.11 to 3.25

# GENERAL OBSERVATION

- Soil does not posses a unique or linear stress-strain relationship.
- Soil behavior depends upon the pressure, time and environment.
- Soil at every location is essentially different.
- Nearly in all the cases, the mass of soil involved is underground and cannot be seen entirely, but must be evaluated on the basis of small size samples, obtained from isolated locations.
- Most soils are very sensitive to disturbance from sampling and thus the behavior measured by a lab test may be unlike that of in situ soil.

# SUBSOIL EXPLORATION

- **Natural soil deposits are not homogeneous, elastic, or isotropic. In some places, the stratification of soil deposits may change greatly within a short horizontal distance.**

- **For foundation design and construction work, one must know the actual soil stratification at a given site, the laboratory test results of the soil samples obtained from various depths, and the observations made during the construction of other structures built under similar conditions.**

- **For most major structures, adequate subsoil exploration at the construction site must be conducted.**

# DEFINITION OF SUBSOIL EXPLORATION

**The process of determining the layers of natural soil deposits that will underlie a proposed structure and their physical properties**

# PURPOSE OF SUBSOIL EXPLORATION

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

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 potential foundation problems (e.g., expansive soil, collapsible soil, sanitary landfill, and so on).
5. Determining the location of the water table.
6. Predicting the lateral earth pressure for structures such as retaining walls, sheet pile bulkheads, and braced cuts.
7. Establishing construction methods for changing subsoil conditions.

# SUBSURFACE EXPLORATION PROGRAM

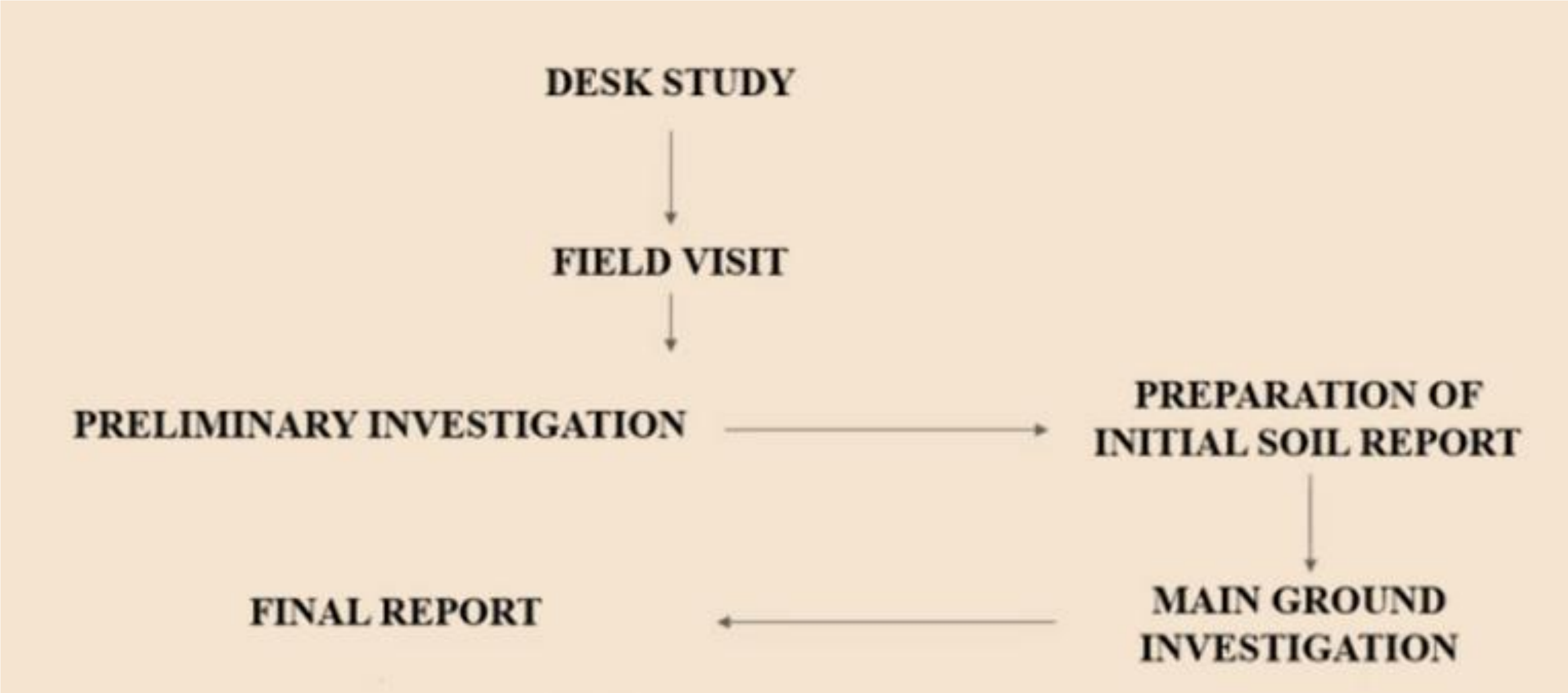
A soil exploration program for a given structure can be divided broadly into three phases:

I. Collection of Preliminary Information (Desk Study)

II. Reconnaissance (Field Trip)

III. Site Investigation

# SUBSURFACE EXPLORATION PROGRAM



# I. Collection of Preliminary Information

**This step includes obtaining information regarding the type of structure to be built and its general use.**

**For the construction of building:**

- **The approximate column loads and their spacing.**
- **Local building-codes.**
- **Basement requirement.**

**For the construction of bridge:**

- **The length of their spans.**
- **The loading on piers and abutments.**

**It also includes obtaining information regarding the general topography and type of soil to be encountered near and around the proposed site which can be obtained from Saudi Geological Survey and other sources.**

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# II. RECONNAISSANCE (FIELD TRIP)

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

- **The general topography of the site, the possible existence of drainage ditches,  and other materials present at the site.**
- **Evidence of creep of slopes and deep, wide shrinkage cracks at regularly spaced intervals may be indicative of expansive soil.**
- **Soil stratification from deep cuts, such as those made for the construction of nearby highways and railroads.**
- **The type of vegetation at the site, which may indicate the nature of the soil.**
- **Groundwater levels, which can be determined by checking nearby wells.**
- **The type of construction nearby and the existence of any cracks in walls (indication for settlement) or other problems.**
- **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.**

# II. RECONNAISSANCE (FIELD TRIP)

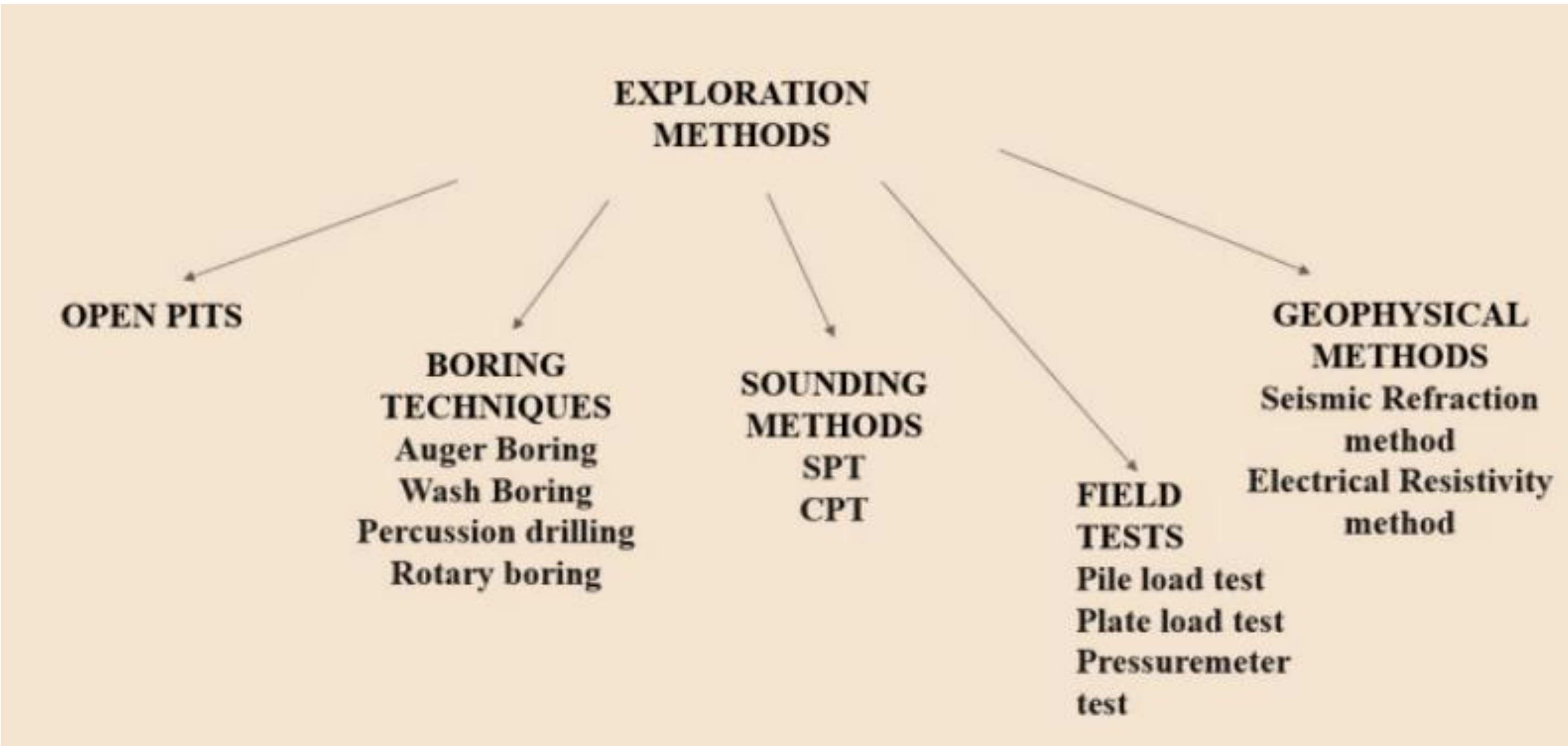
- *Talk to local residents*
- *Idea about existing structures*
- *Ground features to be noted*
- *Existing water bodies*
- *Work out the transportation routes*

# III. SITE INVESTIGATION

**This phase consists of:**

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

# SUBSURFACE EXPLORATION PROGRAM



# NUMBER OF BORING

## 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.

TABLE 3.4 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 |

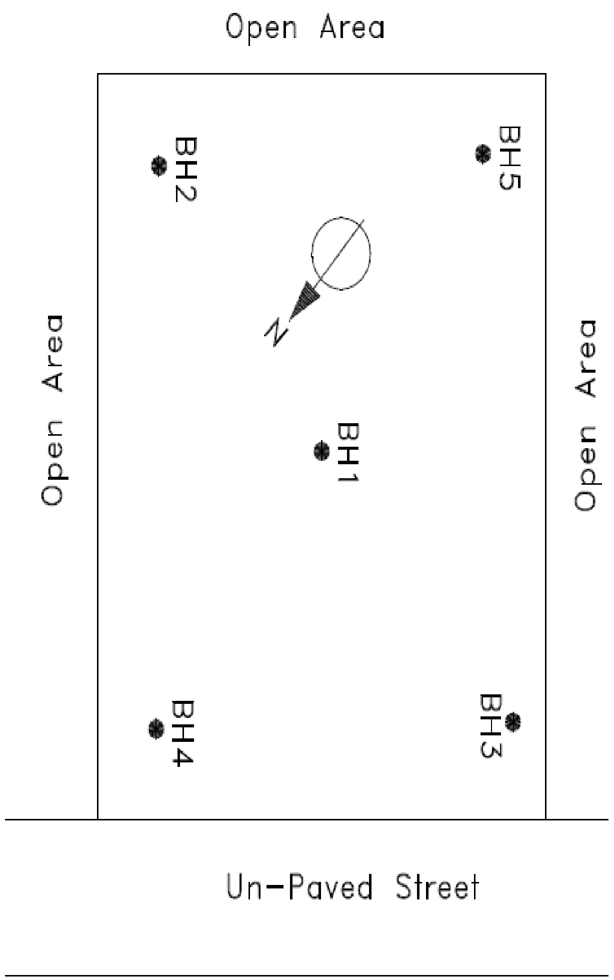
# EXAMPLE

**In practice:**
**number of boreholes and the depth of each borehole will be identified according to the type of project and the subsoil on site.**

**Example for a 5 story residential building with dimensions of (40 x 70) m:**

**The required number of boreholes = 5 boreholes (one at each corner and one at the center) as mentioned previously.**

**The figure shows the distribution of boreholes on the land**



# DEPTH OF BORING

## 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.

2. Estimate the variation of the vertical effective stress ($\sigma o'$) with depth.

3. Determine the depth ($D = D_1$) at which the effective increase $\Delta\sigma' = (1/10)\,q$ (q = estimated net stress on the foundation).

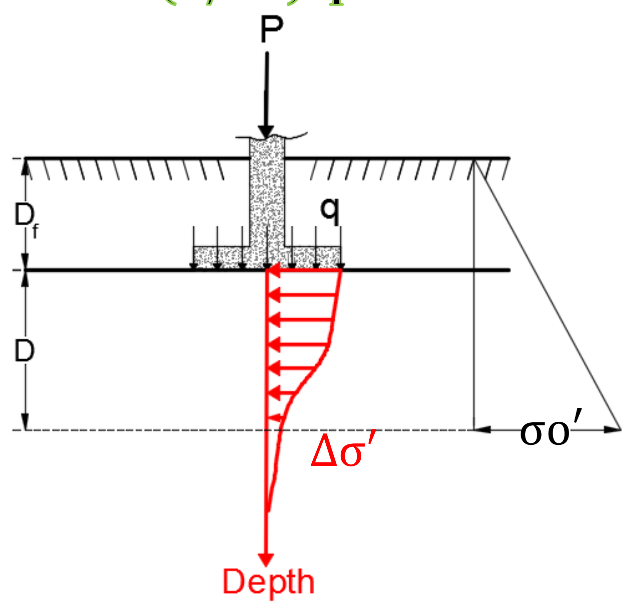
4. Determine the depth ($D = D_2$) at which $\Delta\sigma'/\sigma o' = 0.05$

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 $D_3$) is the approximate required minimum depth of boring.

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 start from the ground surface.



# DEPTH OF BORING

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

| No. of stories | Boring depth |
|---|---|
| 1 | 3.5 m |
| 2 | 6 m |
| 3 | 10 m |
| 4 | 16 m |
| 5 | 24 m |

# DEPTH OF BORING

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

- For light steel or narrow concrete buildings,

$$\frac{D_b}{S^{0.7}} = a$$

where

$D_b$ = depth of boring

$S$ = number of stories

$$a = \begin{cases} \approx 3 \text{ if } D_b \text{ is in meters} \\ \approx 10 \text{ if } D_b \text{ is in feet} \end{cases}$$

- For heavy steel or wide concrete buildings,

$$\frac{D_b}{S^{0.7}} = b$$

where

$$b = \begin{cases} \approx 6 \text{ if } D_b \text{ is in meters} \\ \approx 20 \text{ if } D_b \text{ is in feet} \end{cases}$$

# DEPTH OF BORING

**Determining the value of vertical effective stress ($\sigma o'$):**
The value of ($\sigma o'$) always calculated from the **ground surface** to the required depth, as previously discussed in **(CE382-CHAPTER 9).**

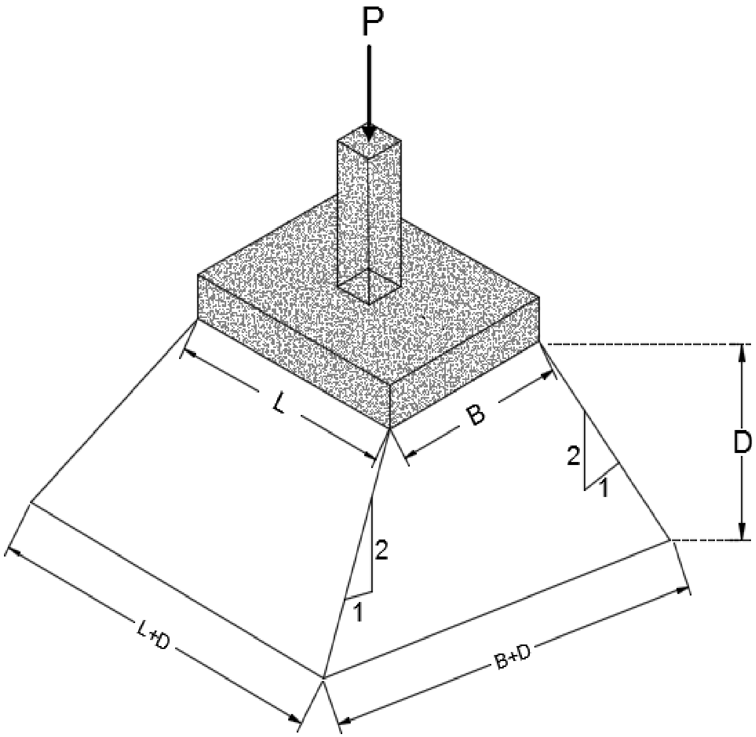
**Determining the increase in vertical effective stress($\Delta\sigma'$):**
The value of ($\Delta\sigma'$) always calculated from the **lower face of the foundation** as discussed previously in **(CE382-CHAPTER 10).**

An alternative approximate method can be used **(2:1 Method)**.
According to this method, the value of ($\Delta\sigma'$) at depth (D) is:

$$\Delta\sigma_D = \frac{P}{A} = \frac{P}{(B+D)(L+D)}$$

P=the load applied on the foundation (KN).
A=the area of the stress distribution at **depth (D)**.

**Note that** the above equation is based on the assumption that the stress from the foundation **spreads out** with a **vertical**-to-**horizontal** slope of **2:1.**



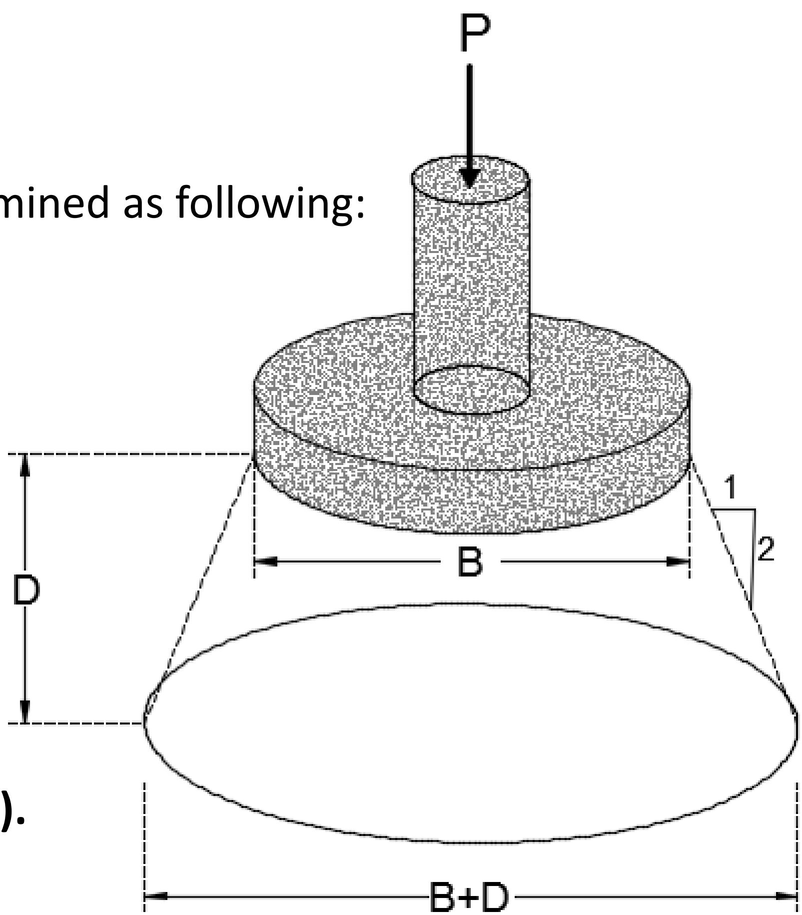
# DEPTH OF BORING

If the foundation is **circular,**

the value of ($\Delta\sigma'$) at depth (D) can be determined as following:

$$\Delta\sigma_D = \frac{P}{\text{Area at depth (D)}}$$

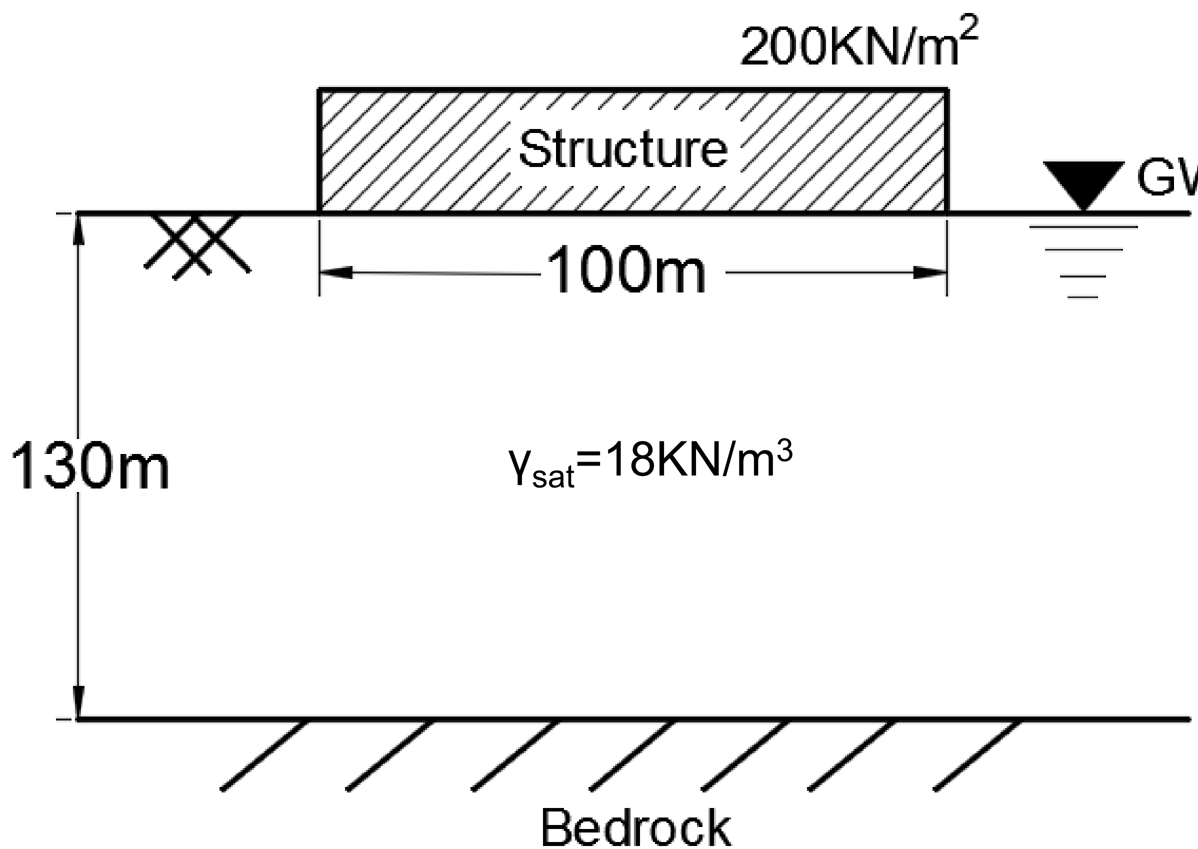
$$\Delta\sigma_D = \frac{P}{\frac{\pi}{4}(B+D)^2}$$




**P=the load applied on the foundation (KN).**
**B=diameter of the foundation(m).**

# EXAMPLE

Site investigation is to be made for a structure of **100m** length and **70m** width. The soil profile is shown below, if the structure is subjected to **200 KN/m²** What is the approximate **depth of borehole**. (Assume $\gamma_w$ **=10KN/m³**).



# SOLUTION

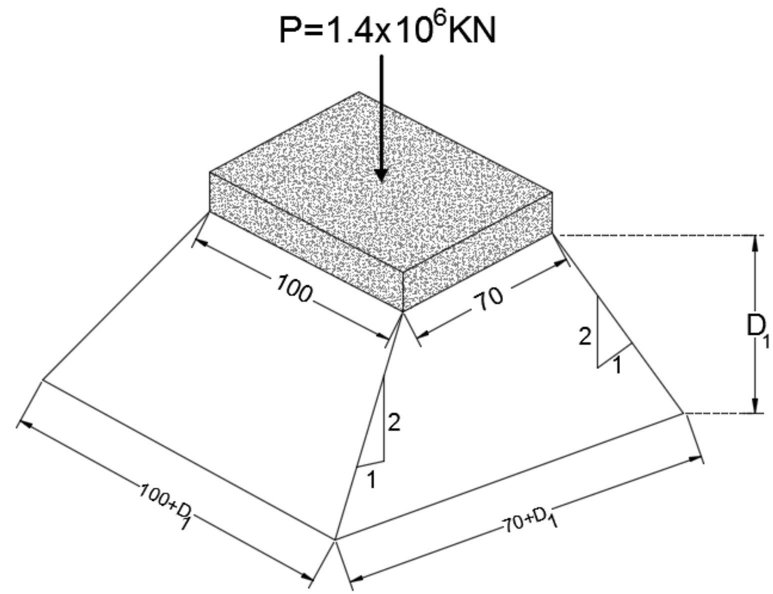
$P = 200 \times (100 \times 70) = 1.4 \times 10^6$ KN

1. Determination of the depth $D_1$) at which the effective increase **$\Delta\sigma' = (1/10)\,q$**

$\Delta\sigma' = (1/10)\,200 = 20$ KN/m$^2$

$$\Delta\sigma_D = \frac{P}{A} = \frac{1.4 * 10^6}{(70 + D_1)(100 + D_1)} = 20$$

$$D_1 = 180 \text{ m}$$




2. Determination of the depth ($D = D_2$) at which

**$\Delta\sigma'/\sigma o' = 0.05$**

$$\Delta\sigma_o = (\gamma_{sat} - \gamma_w) D_2 = (18 - 10) D_2 = 8 * D_2$$

$$\Delta\sigma_D = 0.05(8 * D_2) = 0.40 * D_2$$

$$\Delta\sigma_D = \frac{P}{A} = \frac{1.4 * 10^6}{(70 + D_2)(100 + D_2)}$$

$$\frac{1.4 * 10^6}{(70 + D_2)(100 + D_2)} = 0.40 * D_2$$

$$D_2 = 101.4 \text{ m}$$

$D_1 = 180$ m & $D_2 = 101.4$ m & $D_3 = 130$ m

$D = 101.4$ m (the smallest)

# METHODS OF BORING

# METHODS OF BORING

The boring methods are used for exploration at greater depths where direct methods fail. They provide both disturbed as well as undisturbed samples depending upon the method of boring.

In selecting the boring method for a particular job, consideration should be made for the following:

- The materials to be encountered and the relative efficiency of the various boring methods in such materials
- The available facility and accuracy with which changes in the soil and ground water conditions can be determined
- Possible disturbance of the material to be sampled

# METHODS OF BORING

## METHODS OF EXPLORATION

- **Open excavation**
- **Boring methods**
- **Subsurface Sounding**

**DIRECT METHODS**

- **Geophysical Methods**

**INDIRECT METHODS**

**<u>OPEN METHODS</u>**

- **Test/Trial pits**
- **Trenches**

# METHODS OF BORING

## Test Pits:

- Open excavation (1.5-2.5 deep & approximate 1 m wide)
- Suitable for near surface evaluation, sampling and testing
- Visual inspection
- Excavated by hand or machine
- For small projects where foundation level < 2 m
- Block samples
- For preliminary investigation
- It is relatively fast and inexpensive

# METHODS OF BORING

## Trenches

- Long shallow pits.
- Trenches provide a continuous exposure of the continuity and character of the subsurface material along a given line or section.
- Excavated with ditching machines, backhoes, bulldozers, or pans depending upon the required size and depth of the trench.
- Minimum bottom width of a trench is about 0.6 to 0.9 m

# METHODS OF BORING

**The different types of boring methods are:**

**1. Auger boring**

**2. Continuous sampling**

**3.Wash boring**

**4. Rotary drilling**

**5. Percussion drilling**

# METHODS OF BORING

# AUGER BORING

# METHODS OF BORING

## AUGER BORING

- Hand Augers (upto 6 m)
- Mechanical Augers
- Advancement is made by drilling the auger by simultaneous rotating and pressing into soil
- Dry and unsupported boreholes
- Casing provided in the case of collapsible soils

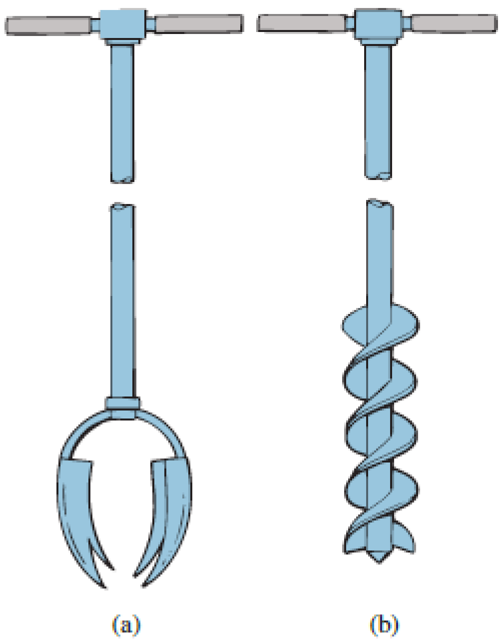
# AUGER BORING

1. Hand auger, two types:

➢ Posthole Auger
➢ Helical Auger

- Depth 3-6 m
- Disturbed samples
- Small structures, highways,…

2. Deeper boreholes:
Portable power-driven helical augers




Hand tools:
(a) Posthole auger
(b) Helical auger

# AUGER BORING

- This method is fast and economical, using simple, light, flexible and inexpensive instruments for large to small holes.
- It is very suitable for soft to stiff cohesive soils and also can be used to determine ground water table.
- Soil removed by this method is disturbed but it is better than wash boring, percussion or rotary drilling.
- This method of boring is not suitable for:
  - Very hard or cemented soils
  - Very soft soils
  - Fully saturated cohesionless soils

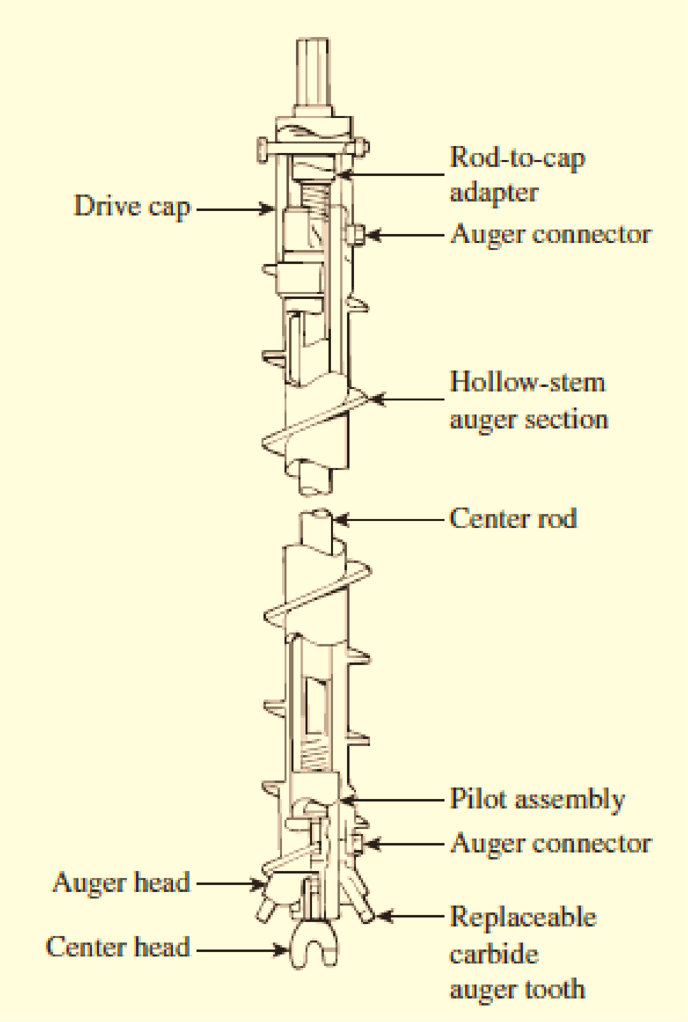
# CONTINUOUS-FLIGHT AUGERS

# CONTINUOUS-FLIGHT AUGERS

- The sampling operation advances the borehole and the boring is accomplished entirely by taking samples continuously.
- Boreholes up to a depth of 60-70 m. They are available in sections of about 1-2 m with either a solid or hollow stem with different diameters.
- 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.
- The tip of the auger is attached to a cutter head. →

- The casing is used to prevent the caving in soils.
- 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.

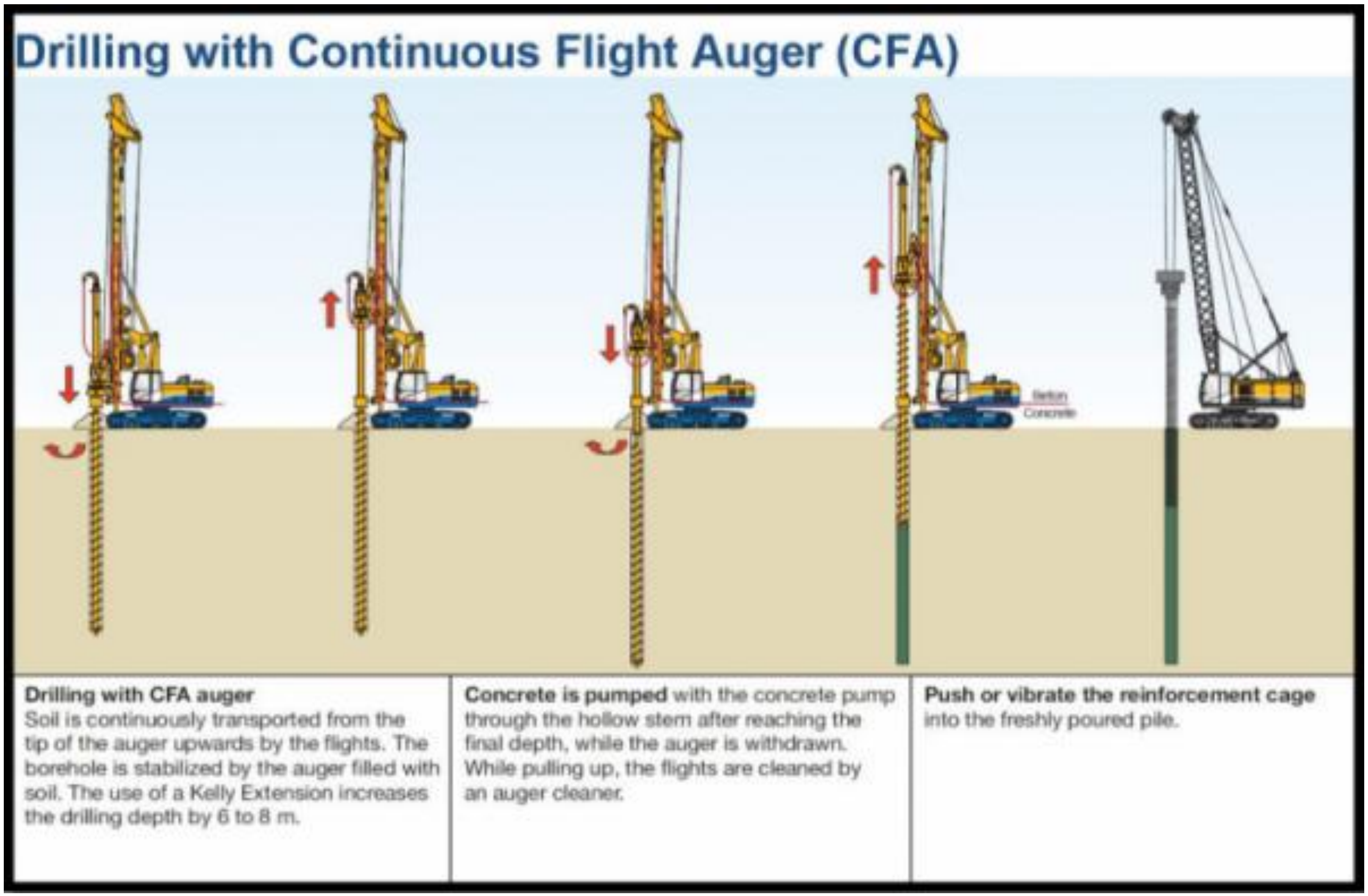
# CONTINUOUS-FLIGHT AUGERS




Hollow-stem auger components

# CONTINUOUS-FLIGHT AUGERS


# CONTINUOUS-FLIGHT AUGERS



# WASH BORING

# WASH BORING

- ✓ It is a popular method due to the use of limited equipment.
- ✓ The advantage of this method is the use of inexpensive and easily portable handling and drilling equipment.
- ✓ First an open hole is formed on the ground so that the soil sampling or rock drilling operation can be done below the hole.
- ✓ The hole is advanced by chopping and twisting action of the light bit. Cutting is done by forced water and water jet under pressure through the rods operated inside the hole.

# WASH BORING

- ✓ A pipe of 5 cm diameter is held vertically and filled with water using horizontal lever arrangement and by the process of suction and application of pressure, soil slurry comes out of the tube and pipe goes down. This can be done up to a depth of 8m –10m.

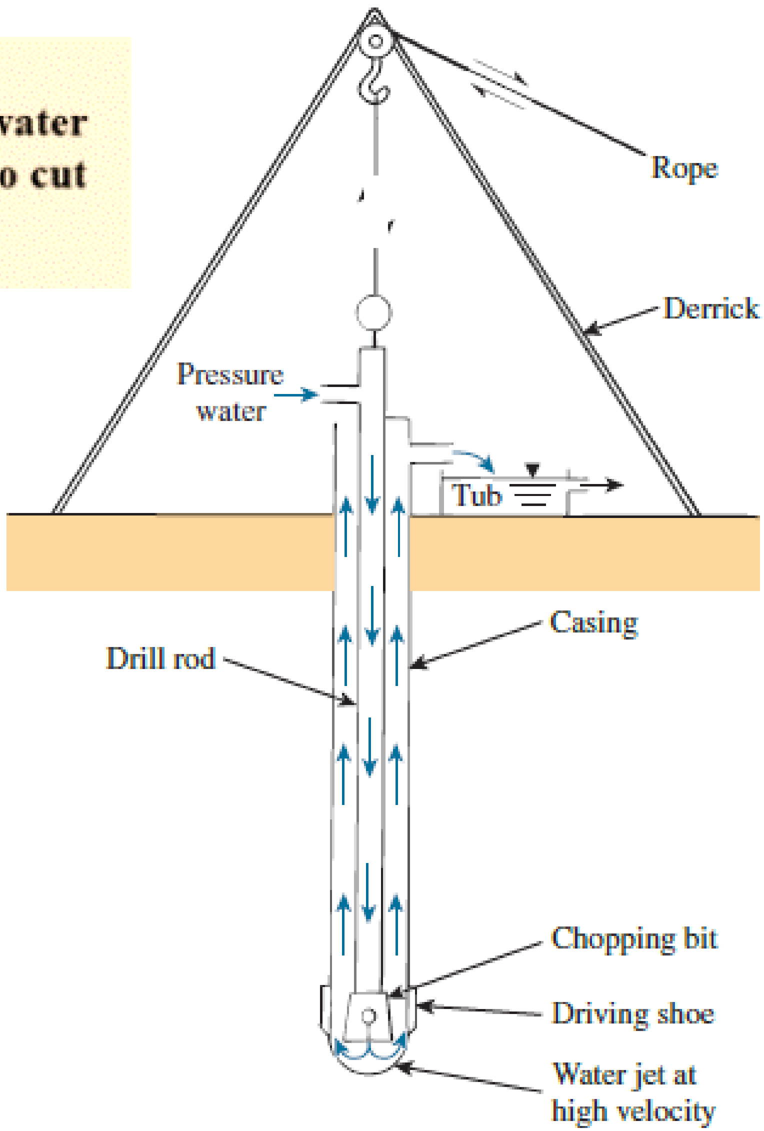
- ✓ Just by noting the change of color of soil coming out with the change of soil character can be identified by any experienced person.

- ✓ It gives completely disturbed sample and is not suitable for very soft soil, fine to medium grained cohesionless soil and in cemented soil.

# WASH BORING

**MECHANISM OF BORING**

- Uses the combined action of water jetting and chopping action of bit to cut through the hole.



# WASH BORING

Soft to stiff cohesive soils are suitable for this method.

**Advantages**

- Can be used in inaccessible locations such as on water, in swamps, or in between buildings.
- Easily portable drilling and handling equipment.

**Disadvantages**

- Disturbed sample due to chopping action.
- Not suitable for stiffer and coarse-grained soils and hard or cemented soils, rock, and soils that contain boulders.

# ROTARY DRILLING

# ROTARY DRILLING

- It is useful in case of highly resistant strata.
- It is related to finding out the rock strata and also to access the quality of rocks from cracks, fissures and joints. It can be used also in sands and silts.
- The bore holes are advanced in depth by rotary method which is similar to wash boring technique. A heavy string of the drill rod is used for choking action.
- The broken rock or soil fragments are removed by circulating water or drilling mud pumped through the drill rods and bit up through the bore hole from which it is collected in a settling tank for recirculation.
- If the depth is small and the soil stable, water alone can be used. However, drilling fluids are useful as they serve to stabilize the bore hole.

# ROTARY DRILLING

- Drilling mud is slurry of bentonite in water. The drilling fluid causes stabilizing effect to the bore hole partly due to higher specific gravity as compared with water and partly due to formation of mud cake on the sides of the hole. As the stabilizing effect is imparted by these drilling fluids no casing is required if drilling fluid is used.

- This method is suitable for boring holes of diameter 10 cm, or more preferably 15 to 20 cm in most of the rocks. It is uneconomical for holes less than 10 cm diameter. The depth of various strata can be detected by inspection of cuttings.

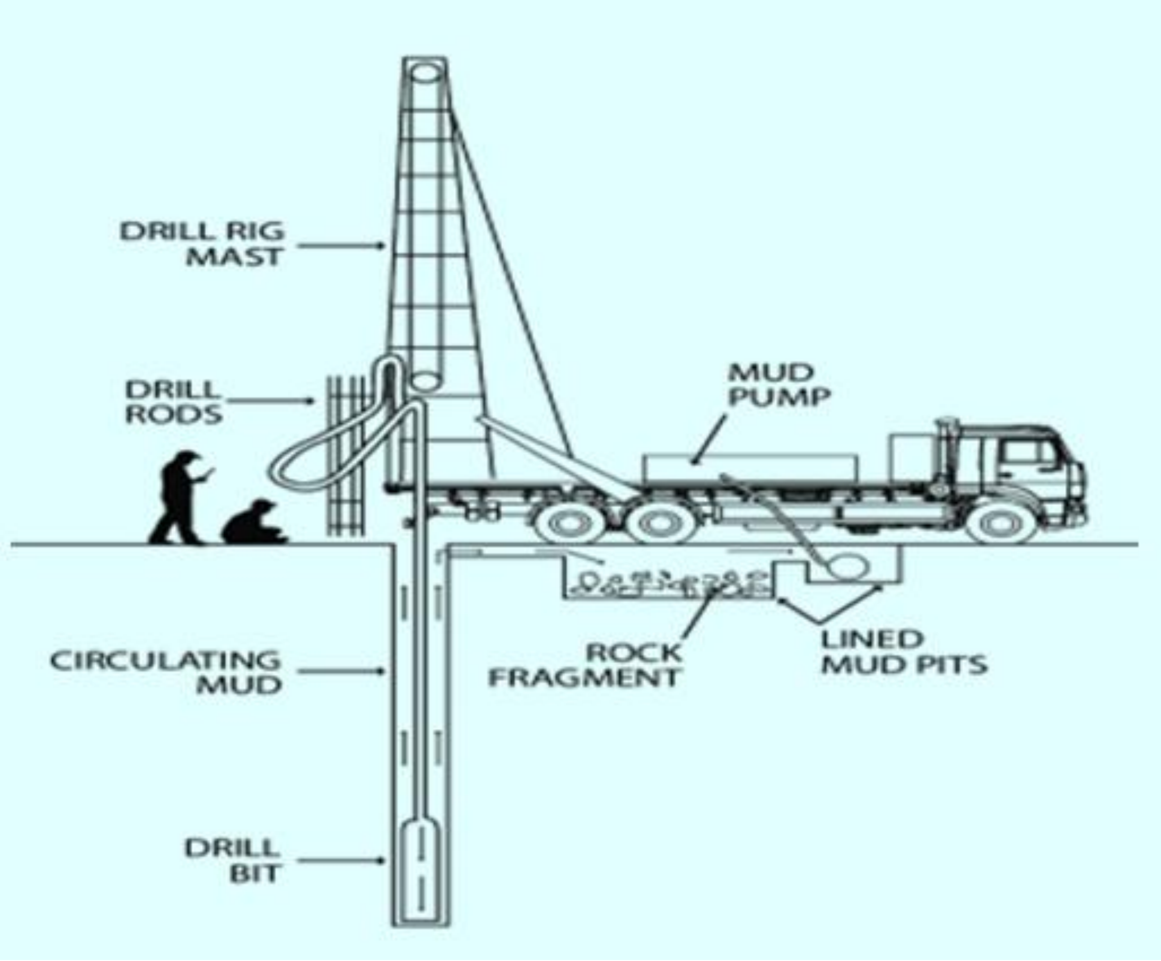
# ROTARY DRILLING

- Rotary drilling is the most common method and is used to drill both exploratory and production wells at depths over 7,000 m.
- Lightweight drills, mounted on trucks, are used to drill low-depth seismic wells on land.
- Medium and heavy rotary mobile and floating drills are used for drilling exploration and production wells.
- Rotary drilling equipment is mounted on a drilling platform with a 30- to 40-m-high derrick and includes a rotary table, engine, mud mixer, and injector pump, a wire-line drum hoist or winch, and many sections of pipe, each approximately 27 m long.
- The rotary table turns a square kelly connected to the drilling pipe.
- The square kelly has a mud swivel on the top which is connected to blowout preventers.
- The drill pipe rotates at a speed of from 40 to 250 rpm, turning either a drill which has drag bits with fixed chisel-like cutting edges or a drill whose bit has rolling cutters with hardened teeth.

# ROTARY DRILLING

**MECHANISM OF BORING**

- Uses a heavy chopping bit to cut through rocks and soils.



# ROTARY DRILLING

All types of soil are suitable for this method.

<u>Advantages</u>

- Fast method of advancing holes in rocks and soils.

- <u>Disadvantages</u>
- Equipment is bulky and expensive.
- The method is not suitable for inaccessible locations.

# PERCUSSION DRILLING

# PERCUSSION DRILLING

- In case of hard soils or soft rock, auger boring or wash boring cannot be employed. For such strata, percussion drilling is usually adopted.

- Advancement of hole is done by alternatively lifting and dropping a heavy drilling bit which is attached to the lower end of the drilling bit which is attached to the cable.

- Addition of sand increases the cutting action of the drilling bit in clays. whereas, when coarse cohesionless soil is encountered, clay might have to be added to increase the carrying capacity of slurry.

- After the carrying capacity of the soil is reached, churn bit is removed and the slurry is removed using bailers and sand pumps. Change in soil character is identified by the composition of the outgoing slurry.

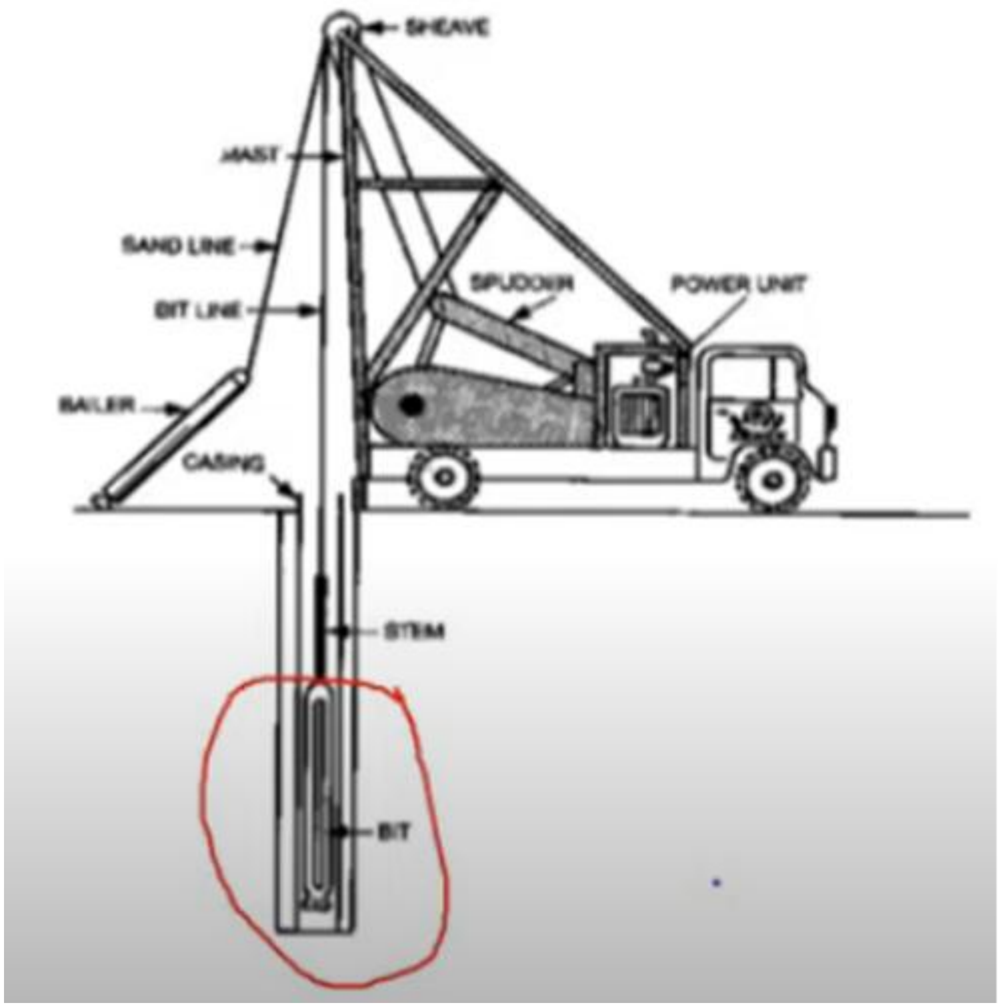
# PERCUSSION DRILLING

- The stroke of bit varies according to the ground condition. Generally, it is 45-100 cm in depth with rate of 35-60 drops/min.
- It is not economical for hole of diameter less than 10cm.
- It can be used in most of the soils and rocks and can drill any material.
- One main disadvantage of this process is that the material at the bottom of the hole is disturbed by heavy blows of the chisel and hence it is not possible to get good quality undisturbed samples. It cannot detect thin strata as well.

# PERCUSSION DRILLING

**MECHANISM OF BORING**

- Uses a heavy bit to break the soil.
- 45-100cm in depth with rate of 35-60 drops/min



# PERCUSSION DRILLING

All types of soil are suitable for this method.

**Advantages**

- Useful to probe cavities and weakness in rock, by observing changes in the drill rate

**Disadvantages**

- Material at the bottom of the hole is disturbed by heavy blows of the chisel.
- It cannot detect thin strata as well.

# SOIL SAMPLING

# SOIL SAMPLING

## Need for Soil Sampling

- A satisfactory design of a foundation depends upon the accuracy with which the various soil parameters required for the design are obtained.
- The accuracy of the soil parameters depends upon the accuracy with which representative soil samples are obtained from the field.
- Sampling is carried out in order that soil and rock description, and laboratory testing can be carried out.
- Laboratory tests typically consist of:
  - Index tests (for example, specific gravity, water content)
  - Classification tests (for example, Atterberg's limit tests on clayey soil)
  - Tests to determine engineering design parameters (for example strength, compressibility, and permeability).

# SOIL SAMPLING

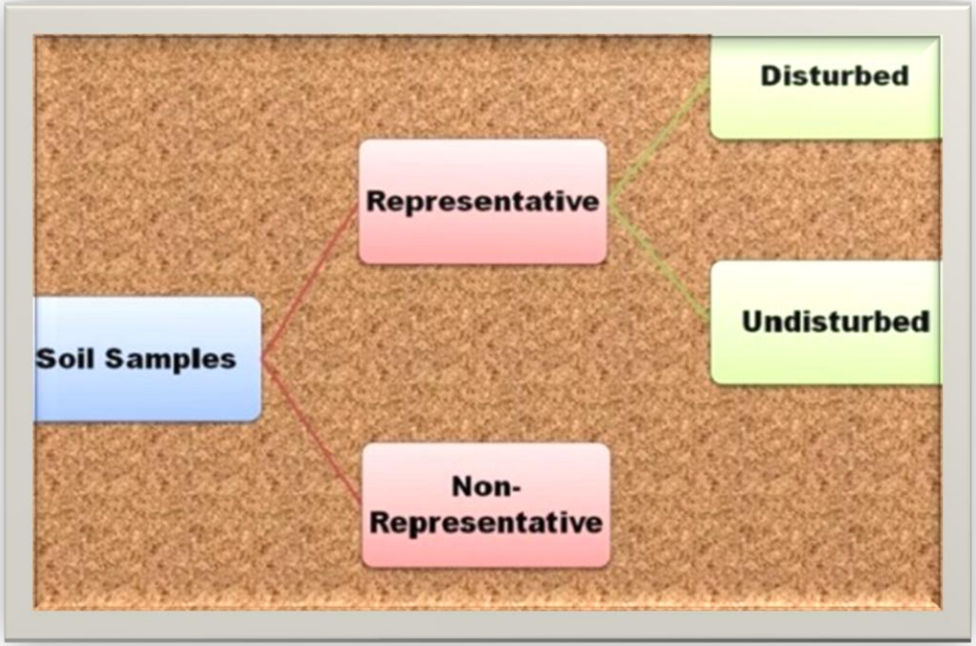
## Factors to be considered while sampling soil

- Samples should be representative of the ground from which they are taken.
- They should be large enough to contain representative particles sizes, fabric, and fissuring and fracturing.
- They should be taken in such a way that they have not lost fractions of the in situ soil (for example, coarse or fine particles).
- Where strength and compressibility tests are planned, they should be subject to as little disturbance as possible.

# SOIL SAMPLING

## Non-Representative Soil Samples

- Non-representative soil samples are those in which neither the in-situ soil structure, moisture content nor the soil particles are preserved.
- They cannot be used for any tests as the soil particles either gets mixed up or some particles may be lost.
- Samples that are obtained through wash boring or percussion drilling are examples of non-representative samples



# SOIL SAMPLING

## Representative Soil Samples

**There are two types of samples:**

- ❑ **Disturbed Soil Samples**
- ❑ **Undisturbed Soil Samples**

# DISTURBED SOIL SAMPLES

- Disturbed soil samples are those in which the in-situ soil structure and moisture content are lost, but the soil particles are intact.

- They are representative.

- They can be used for the following types of laboratory soil tests:
  - grain size analysis
  - liquid and plastic limits
  - specific gravity
  - compaction tests
  - moisture content
  - organic content determination

- The major equipment used to obtain disturbed samples is

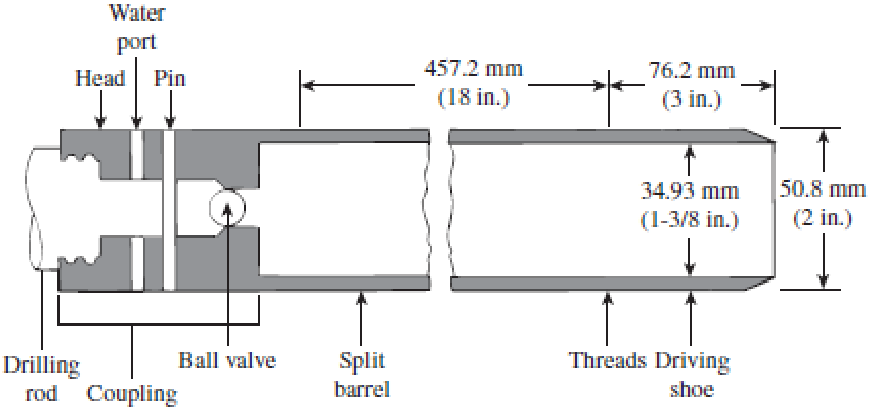
  Split Spoon a steel tube with

  $D_i$ = 34.93 mm

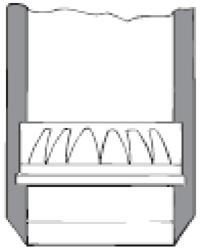
  $D_o$ = 50.8mm

# SPLIT SPOON SAMPLING

$D_i$ = 34.93 mm
$D_o$ = 50.80 mm



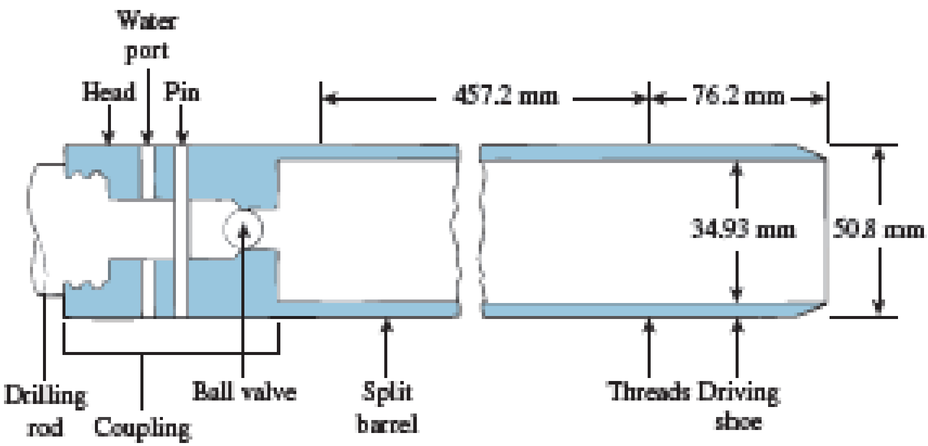

(a)



(b)

*Figure 3.15* (a) Standard split-spoon sampler; (b) spring core catcher

# SPLIT SPOON SAMPLING




$D_i$ = 34.93 mm
$D_o$ = 50.80 mm

Unassembled split-spoon sampler

After sampling

# SCRAPER BUCKET

- If soil deposits are sand mixed with pebbles (split spoon with a spring core catcher may not be possible because pebbles may prevent the springs from closing.
- A scraper bucket is used to obtain disturbed representative samples.
- The scraper bucket is driven in the soil and rotated, the scrapings from the side fall into the bucket.

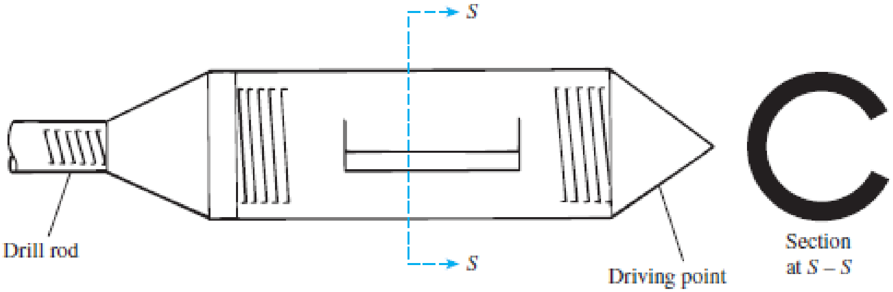



*Figure 3.18* Scraper bucket

# UNDISTURBED SOIL SAMPLES

- Undisturbed soil samples are those in which the in-situ soil structure and moisture content are preserved.

- They are representative and also intact.

- These are used for the following types of laboratory soil tests:
  - Consolidation tests.
  - Hydraulic Conductivity tests.
  - Shear Strength tests.

- These samples are more complex and expensive, and they are suitable for clays, however in sand, it is very difficult to obtain undisturbed samples.

- The major equipment used to obtain undisturbed sample is Shelby tube (thin-walled tube) and piston sampler.

# THIN-WALLED TUBE (SHELBY TUBE)

$D_i$ = 47.63 mm
$D_o$ = 50.80 mm

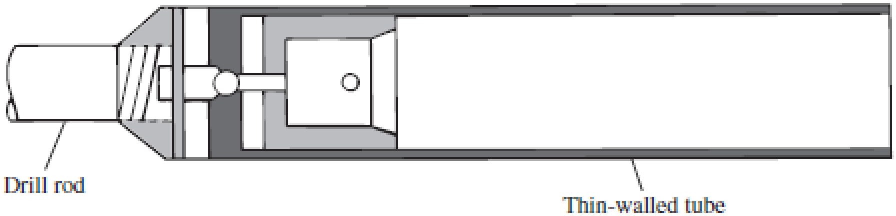



*Figure 3.19* Thin-walled tube

# PISTON SAMPLER

- When undisturbed samples are very soft or larger than 76.2 mm in diameter, they tend to fall out of the sampler
- Piston samplers are used in such conditions
- 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, and the tube is pushed into the soil hydraulically, past the piston. Then the pressure is released through a hole in the piston rod.
- Samples obtained using this sampler are less disturbed than those obtained by Shelby tubes.

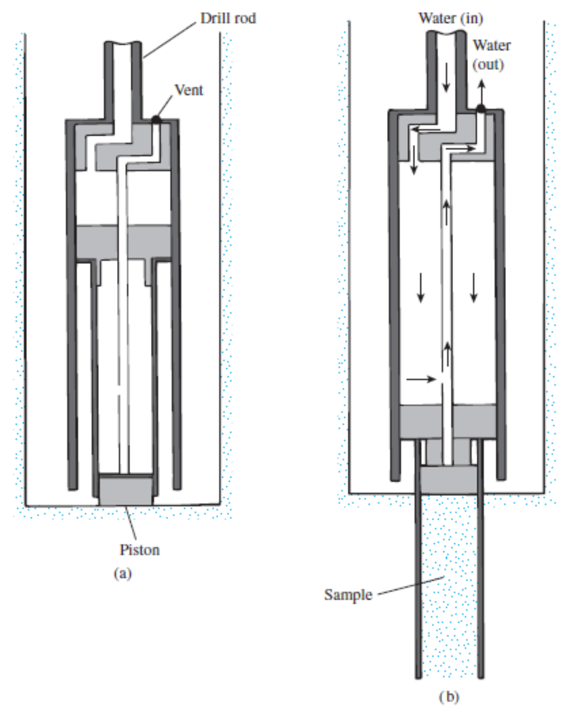



*Figure 3.21* Piston sampler: (a) sampler at the bottom of borehole; (b) tube pushed into the soil hydraulically

# DEGREE OF DISTURBANCE

**If we want to obtain a soil sample from any site, the degree of disturbance for a soil sample is usually expressed as:**

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

**$D_o$=outside diameter of the sampling tube.**
**$D_i$ =inside diameter of the sampling tube.**
**If $(A_R) \leq 10\% \rightarrow$ the sample is undisturbed**
**If $(A_R) > 10\% \rightarrow$ the sample is disturbed**

**For a standard split-spoon sampler (which sampler for disturbed samples):**

$$A_R = \frac{(50.8)^2 - (34.93)^2}{(34.93)^2}(100) = 111.5\% > 10\% \text{ disturbed}$$

**For a Shelby tube (thin-walled tube) -- sampler for undisturbed samples**

$$A_R = \frac{(50.8)^2 - (47.63)^2}{(47.63)^2}(100) = 13.75\% \;\; = 10\% \text{ undisturbed}$$

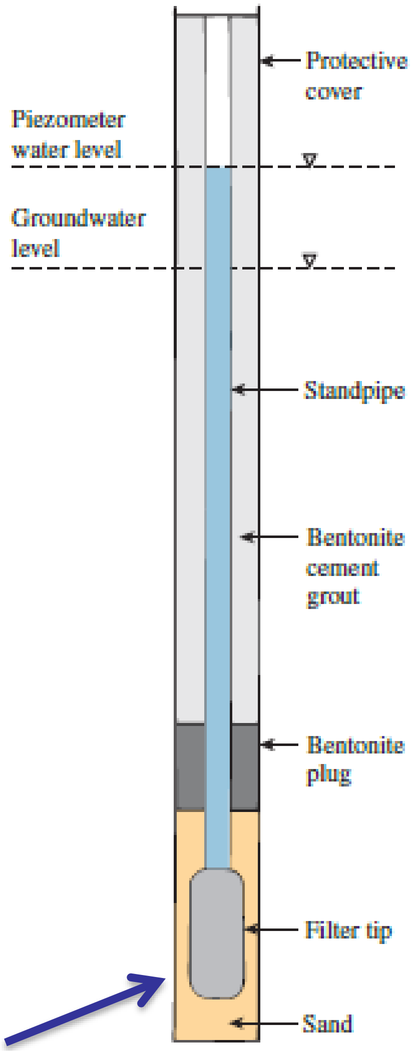
# GROUNDWATER

**Why do you always measure groundwater?**

- Calculation of effective stress
- Can impact the bearing capacity of shallow foundations
- Can impact the pressures against retaining walls
- Impacts the capacity of pile foundations
- Impacts the in-situ permeability
- Impacts construction that may be below groundwater table

**How do you measure groundwater levels?**

- In the borehole immediately after and 24 hours
- In a piezometer (simple well)
- Pore water pressure transducers (data over time)




piezometer

# IN-SITU (FIELD) TESTS

# IN-SITU TESTS

- The ground is tested in-place by instruments that are inserted in or penetrate the ground.
- In-situ tests are normally associated with tests for which a borehole either is unnecessary or is only an incidental part of the overall test procedure, required only to permit insertion of the testing tool or equipment.
- Improvements in apparatus, instrumentation, and technique of deployment, data acquisition and analysis procedure have been significant.

# IN-SITU TESTS

## Advantages

- Tests are carried out in place in the natural environment without sampling disturbance, which can cause detrimental effects and modifications to stresses, strains, drainage, fabric and particle arrangement.
- Continuous profiles of stratigraphy and engineering properties/ characteristics can be obtained.
- Detection of planes of weakness and defects are more likely and practical.
- Methods are usually fast, repeatable, produce large amounts of information and are cost effective.
- Tests can be carried out in soils that are either impossible or difficult to sample without the use of expensive specialized methods.
- A large volume of soil may be tested than is normally practicable for laboratory testing. This may be more representative of the soil mass.

# IN-SITU TESTS

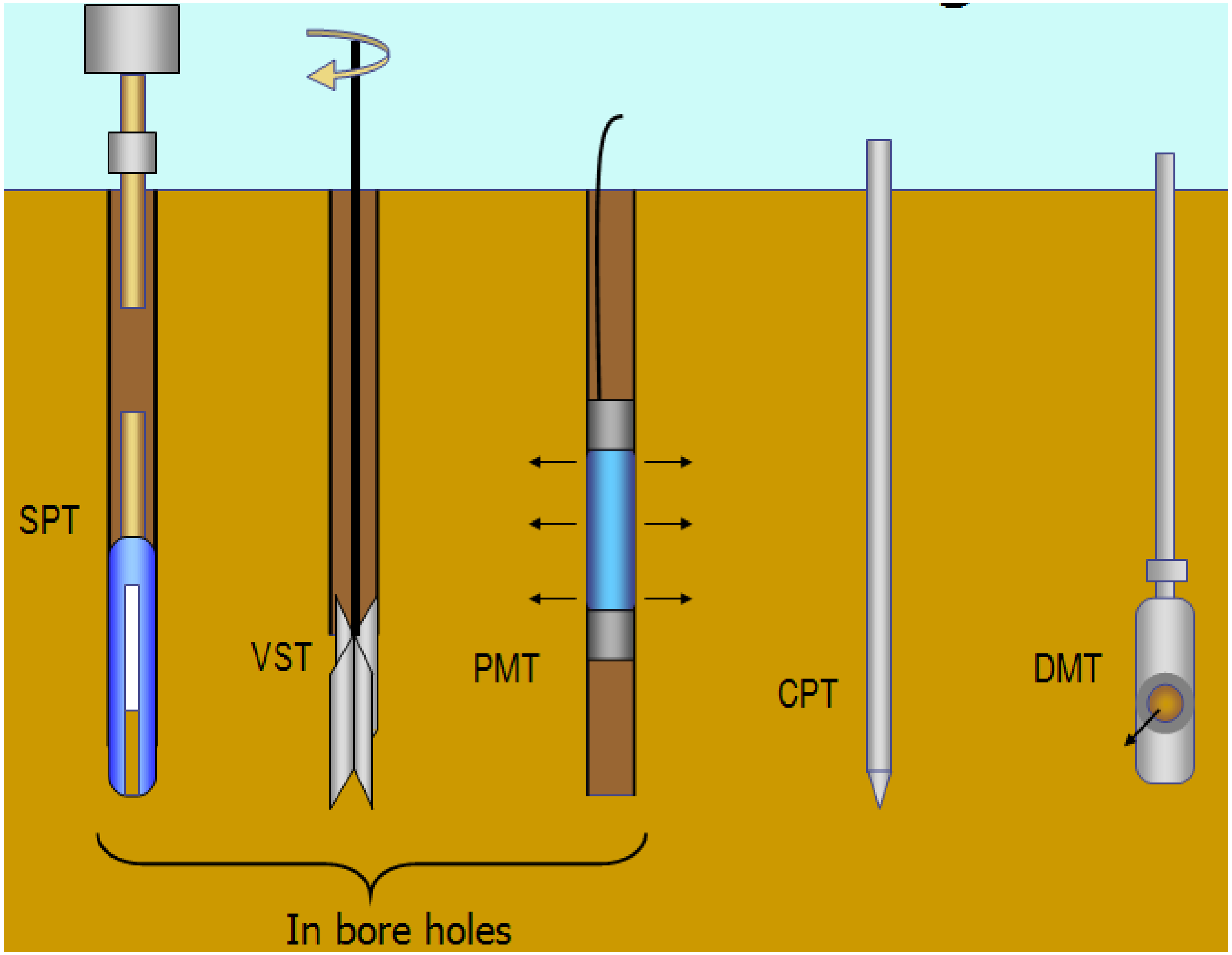
## Disadvantages

- Samples are not obtained; the soil tested cannot be positively identified. The exception to this is the SPT in which a sample, although disturbed, is obtained.
- The fundamental behavior of soils during testing is not well understood.
- Drainage conditions during testing are not known.
- Consistent, rational interpretation is often difficult and uncertain.
- The stress path imposed during testing may bear no resemblance to the stress path induced by full-scale engineering structure.
- Most push-in devices are not suitable for a wide range of ground conditions.
- Some disturbance is imparted to the ground by the insertion or installation of the instrument.
- There is usually no direct measurement of engineering properties. Empirical correlations usually have to be applied to interpret and obtain engineering properties and designs

# IN-SITU TESTS

- Standard Penetration Test (SPT)
- Vane shear test (VST)
- Cone Penetration Test (CPT)
- The Pressuremeter Test (PMT)
- The Flat Dilatometer Test (DMT)
- The Plate Load Test (PLT) → Later

# IN-SITU TESTS



# STANDARD PENETRATION TEST (SPT)

# 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.
- It is used as an indicator of relative density and stiffness of granular soils as well as an indicator of consistency in a wide range of other ground.
- Methods have been developed to apply SPT results to a wide range of geotechnical applications including shallow and deep foundations.
- The main standard for the SPT is the American Society for Testing and Materials (ASTM D-1586-99).

***Aim:*** **To perform standard penetration to obtain the penetration resistance (N-value) along the depth at a given site.**

# STANDARD PENETRATION TEST (SPT)

## *Advantages of SPT*:

- Simple and rugged
- Low cost
- Obtain a sample
- Can be performed in most soil types

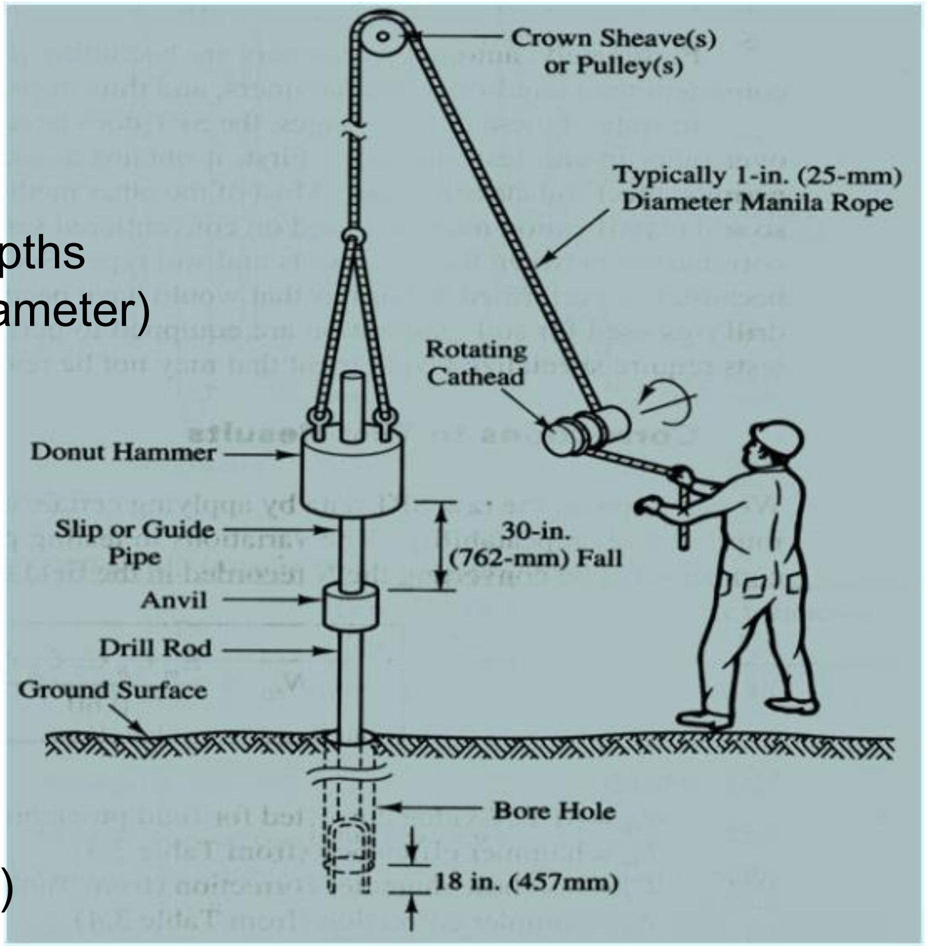
## *Disadvantages of SPT:*

- Disturbed sample (index tests only)
- Crude number (*N* value)
- Not applicable in soft clays and silts
- High variability and uncertainty.

# STANDARD PENETRATION TEST (SPT)

## Equipment & Apparatus

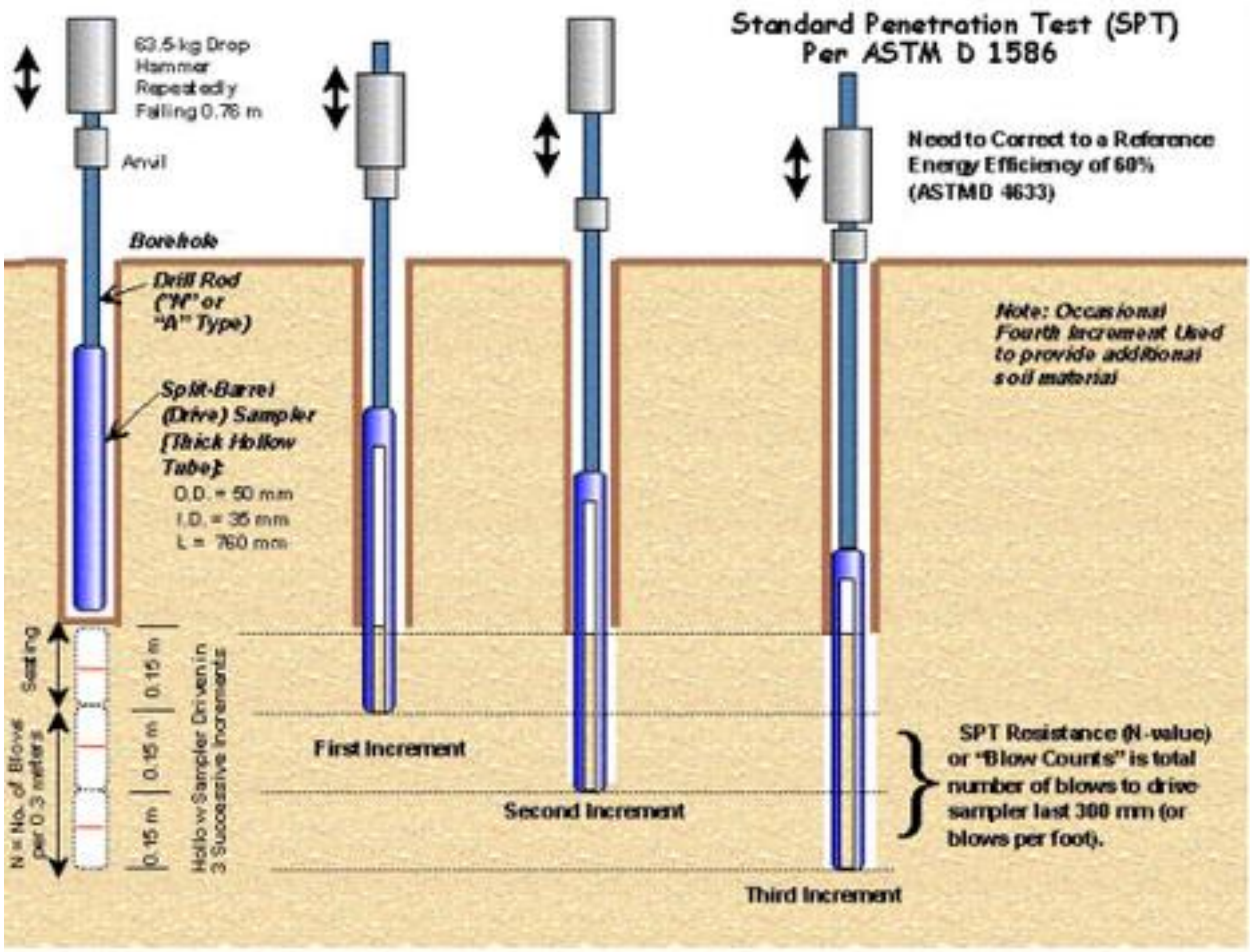
- Tripod (to give a clear height of about 4 m; one of the legs of the tripod should have ladder to facilitate a person to reach tripod head.)
- Tripod head with hook
- Pulley
- Guide pipe assembly
- Standard split spoon sampler
- A drill rod for extending the test to deeper depths
- Heavy duty post hole auger (100-150 mm diameter)
- Heavy duty helical auger
- Heavy duty auger extension rods
- Sand bailer
- Rope (about 15 m long & strong enough to lift 63.5 kg load repeatedly)
- A light duty rope to operate sand bailer
- Chain pulley block
- Casing pipes
- Casing couplings
- Casing clamps *A straight edge (50 cm)
- Measuring tapes *Tool box



# SPT (PROCEDURE)

1. Determine the required number and depth of boreholes in the site.
2. The sampler used in SPT test is (Standard Split Spoon).
3. Using drilling machine, 1.5m are drilled.
4. The drilling machine is removed and the sampler is 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.

# STANDARD PENETRATION TEST (SPT)



# STANDARD PENETRATION TEST (SPT)

## PRECAUTIONS

1.Results of standard penetration test are not reproducible in cohesionless soil below water level unless care is taken to maintain the water level inside the borehole always slightly above the natural groundwater level. If the water level in the borehole is lower than natural groundwater level, quick conditions develop and soil becomes loose.

2.The split spoon sampler must be in good condition with no excessive damage or wear and tear to the cutting shoe.

3.The drill rods should be the right size and not too heavy or too light. The drill rods also should not be bent.

4.The fall of the weight should be free. Friction in the pulley or guide rod, or braking action by crew, or interference due to hoist rope can result in higher than actual blow count.

# STANDARD PENETRATION TEST (SPT)

## PRECAUTIONS

5. The height of free fall of weight must be 750 mm. It is obvious that the change in the height of fall will result in a value different from the actual value for N.

6. The bottom of borehole must be properly cleaned before seating the split spoon sampler. Otherwise the test will be carried out in the loose and disturbed soil at the bottom of the bore hole.

7. If casing is used in borehole it must not be driven ahead of the level at which SPT is being carried out. Otherwise the SPT will be carried out in a soil plug enclosed at the bottom of the casing.

8.The rate of delivery of the blows should not be too fast.

9. Careless work on the part of drilling crew, improper and incorrect counting of blows and recording must be avoided.

# SPT (CORRECTION TO N VALUE)

There are several factors contribute to the variation of the standard penetration number (N) at a given depth for similar profiles. Among these factors are the SPT hammer efficiency, borehole diameter, sampling method, and rod length.

In the field, the magnitude of hammer efficiency can vary from 30 to 90%, the standard practice now is to express the N-value to an average energy ratio of 60% ($N_{60}$), so correcting for field procedures is required as following:

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60}$$

N=measured penetration number.

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

$\eta_H$=hammer efficiency (%).

$\eta_B$=correction for borehole diameter.

$\eta_S$=sampler correction.

$\eta_R$=correction for rod length.

# SPT (CORRECTION TO N VALUE)

$$N_{60} = \frac{N\eta_H \eta_B \eta_S \eta_R}{60}$$

**TABLE 3.5** Variations of $\eta_H$, $\eta_B$, $\eta_S$, and $\eta_R$

**1. Variation of $\eta_H$**

| Country | Hammer type | Hammer release | $\eta_H$ (%) |
|---|---|---|---|
| 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 $\eta_B$**

| Diameter mm | $\eta_B$ |
|---|---|
| 60–120 | 1 |
| 150 | 1.05 |
| 200 | 1.15 |

**3. Variation of $\eta_S$**

| Variable | $\eta_S$ |
|---|---|
| Standard sampler | 1.0 |
| With liner for dense sand and clay | 0.8 |
| With liner for loose sand | 0.9 |

**4. Variation of $\eta_R$**

| Rod length m | $\eta_R$ |
|---|---|
| >10 | 1.0 |
| 6–10 | 0.95 |
| 4–6 | 0.85 |
| 0–4 | 0.75 |

# SPT ($N_{60}$ CORRELATIONS)

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

- ❑ The equations are approximate.
- ❑ Because the soil is not homogeneous, the values of $N_{60}$ obtained from a given borehole vary widely.
- ❑ In soil deposits that contain large boulders and gravel, standard penetration numbers may be erratic and unreliable.

# Correction for Effective Overburden Pressure

$$(N_1)_{60} = C_N N_{60}$$

where

$(N_1)_{60}$ = value of $N_{60}$ corrected to a standard value of $\sigma'_a = p_a$ ($\approx$ 100 kN/m$^2$)

$C_N$ = correction factor

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

Liao and Whitman's relationship (1986):

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

Skempton's relationship (1986):

$$C_N = \frac{2}{1 + \left(\frac{\sigma'_o}{p_a}\right)} \quad \text{(for normally consolidated fine sand)}$$

$$C_N = \frac{3}{2 + \left(\frac{\sigma'_o}{p_a}\right)} \quad \text{(for normally consolidated coarse sand)}$$

$$C_N = \frac{1.7}{0.7 + \left(\frac{\sigma'_o}{p_a}\right)} \quad \text{(for overconsolidated sand)}$$

Seed et al.'s relationship (1975):

$$C_N = 1 - 1.25 \log\left(\frac{\sigma'_o}{p_a}\right)$$

Peck et al.'s relationship (1974):

$$C_N = 0.77 \log\left[\frac{20}{\left(\frac{\sigma'_o}{p_a}\right)}\right] \left(\text{for } \frac{\sigma'_o}{p_a} \geq 0.25\right)$$

Bazaraa (1967):

$$C_N = \frac{4}{1 + 4\left(\frac{\sigma'_o}{p_a}\right)} \left(\text{for } \frac{\sigma'_o}{p_a} \leq 0.75\right)$$

$$C_N = \frac{4}{3.25 + \left(\frac{\sigma'_o}{p_a}\right)} \left(\text{for } \frac{\sigma'_o}{p_a} > 0.75\right)$$

# EXAMPLE 3.1

## EXAMPLE 3.1

A standard penetration test is carried out in sand where the efficiency of the hammer $\eta_H = 70\%$. If the measured $N$-value at 9.15 m depth is 24, find $N_{60}$ and $(N_1)_{60}$. The unit weight of the sand is 18.08 kN/m$^3$. Assume $\eta_B = \eta_S = \eta_R = 1$.

### SOLUTION

From Eq. (3.6),

$$N_{60} = \frac{(N)(\eta_H)(1)(1)(1)}{60} = \frac{(24)(70)}{60} = 28$$

From Eq. (3.13),

$$C_N = \left[\frac{1}{\sigma'_o/p_a}\right]^{0.5} = \left[\frac{1}{(9.15 \times 18.08)/100}\right]^{0.5} = 0.78$$

From Eq. (3.12),

$$(N_1)_{60} = C_N N_{60} = 0.76 \times 28 \approx 22$$

# SPT ($N_{60}$ CORRELATIONS)

**$N_{60}$ can be used for calculating some important parameters such as:**

## Cohesive soils

- **Consistency Index (CI)**
- **Undrained shear strength ($C_u$)**
- **Overconsolidation ratio (OCR)**

# SPT ($N_{60}$ CORRELATIONS)

## Consistency Index (CI)

$$CI = \frac{LL - w}{LL - PL}$$

where

$w$ = natural moisture content (%)

LL = liquid limit

PL = plastic limit

**TABLE 3.6** Approximate Correlation Among CI, $N_{60}$, and $q_u$

| Standard penetration number, $N_{60}$ | Consistency | CI | Unconfined compression strength, $q_u$ (kN/m$^2$) |
|---|---|---|---|
| <2 | Very soft | <0.5 | <25 |
| 2–8 | Soft to medium | 0.5–0.75 | 25–100 |
| 8–15 | Stiff | 0.75–1.0 | 100–200 |
| 15–30 | Very stiff | 1.0–1.5 | 200–400 |
| >30 | Hard | >1.5 | >400 |

# SPT ($N_{60}$ CORRELATIONS)

## Undrained shear strength ($C_u$)

Hara et al. (1974) also suggested the following correlation between the undrained shear strength of clay ($c_u$) and $N_{60}$ for clays from Japan with OCR = 1–3.

$$\frac{c_u}{p_a} = 0.29 N_{78}^{0.72}$$

where $p_a$ = atmospheric pressure ($\approx$ 100 kN/m$^2$). Since $N_{78} = 0.77 N_{60}$, in terms of $N_{60}$,

$$\frac{c_u}{p_a} = 0.24 N_{60}^{0.72}$$

# SPT ($N_{60}$ CORRELATIONS)

## Overconsolidation ratio (OCR)

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

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

where $\sigma'_o$ = effective vertical stress in MN/m$^2$.

It is important to point out that any correlation among $c_u$, OCR, and $N_{60}$ is only approximate.

Using the field test results of Mayne and Kemper (1988) and others (112 data points), Kulhawy and Mayne (1990) suggested the approximate correlation

$$OCR = 0.58\frac{N_{60}p_a}{\sigma'_o}$$

# SPT ($N_{60}$ CORRELATIONS)

## Preconsolidation Pressure

Kulhawy and Mayne (1990) have also provided an approximate correlation for the preconsolidation pressure ($\sigma'_c$) of clay as

$$\sigma'_c = 0.47 N_{60} p_a$$

# SPT ($N_{60}$ CORRELATIONS)

$N_{60}$ can be used for calculating some important parameters such as:

## Granular soils

- Relative Density ($D_r$)
- Angle of internal friction ($\phi$)

# SPT ($N_{60}$ CORRELATIONS)

## Relative Density ($D_r$)

Meyerhof (1957) developed a correlation between $D_r$ and $N_{60}$ as

$$N_{60} = \left[17 + 24\left(\frac{\sigma'_o}{p_a}\right)\right]D_r^2$$

$$D_r = \left\{\frac{N_{60}}{\left[17 + 24\left(\frac{\sigma'_o}{p_a}\right)\right]}\right\}^{0.5}$$

for clean, medium-fine sand.

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

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

where

$p_a$ = atmospheric pressure ($\approx$ 100 kN/m$^2$)

$D_{50}$ = sieve size through which 50% of the soil will pass (mm)

# SPT ($N_{60}$ CORRELATIONS)

## Relative Density ($D_r$)

Kulhawy and Mayne (1990) correlated the corrected standard penetration number and the relative density of sand in the form

$$D_r(\%) = \left[\frac{(N_1)_{60}}{C_p C_A C_{OCR}}\right]^{0.5} (100)$$

where

$C_P$ = grain-size correlations factor $= 60 + 25 \log D_{50}$

$C_A$ = correlation factor for aging $= 1.2 + 0.05 \log\left(\frac{t}{100}\right)$

$C_{OCR}$ = correlation factor for overconsolidation $= \text{OCR}^{0.18}$

$D_{50}$ = diameter through which 50% soil will pass through (mm)

$t$ = age of soil since deposition (years)

OCR = overconsolidation ratio

It is difficult to estimate the geologic age of a granular soil deposit. In the absence of any reliable data, it can be assumed to be 1000–5000 years with negligible error in the estimate of $D_r$.

Skempton (1986) suggested that, for sands with a relative density greater than 35%,

$$\frac{(N_1)_{60}}{D_r^2} \approx 60$$

where $(N_1)_{60}$ should be multiplied by 0.92 for coarse sands and 1.08 for fine sands.

# SPT ($N_{60}$ CORRELATIONS)

## Angle of internal friction (ϕ)

Peck et al. (1974) give a correlation between $(N_1)_{60}$ and $\phi'$ in a graphical form, which can be approximated as (Wolff, 1989)

$$\phi'(\text{deg}) = 27.1 + 0.3(N_1)_{60} - 0.00054[(N_1)_{60}]^2$$

Schmertmann (1975) provided the correlation among $N_{60}$, $\sigma'_o$, and $\phi'$. Mathematically, the correlation can be approximated as (Kulhawy and Mayne, 1990)

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

where

$N_{60}$ = field standard penetration number

$\sigma'_o$ = effective overburden pressure

$p_a$ = atmospheric pressure in the same unit as $\sigma'_o$

$\phi'$ = soil friction angle

Hatanaka and Uchida (1996) provided a simple correlation between $\phi'$ and $(N_1)_{78}$ that can be expressed as

$$\phi' = \sqrt{20(N_1)_{78}} + 20$$

In terms of $(N_1)_{60}$, Eq. (3.31a) becomes

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20$$

# SPT ($N_{60}$ CORRELATIONS)

Approximate borderline values for $D_r$, $N_{60}$, $(N_1)_{60}$, and $\dfrac{(N_1)_{60}}{D_r^2}$

| | | *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 | | |
| **$\phi'$ (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)

# SPT ($N_{60}$ CORRELATIONS)

## Modulus of Elasticity ($E_s$)

Kulhawy and Mayne (1990)

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

where

$p_a =$ atmospheric pressure (same unit as $E_s$)

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

# EXAMPLE 3.2

## EXAMPLE 3.2

In a sand with unit weight of 17.76 kN/m$^3$, a standard penetration test is carried out. The $N_{60}$ values are as follows:

| Depth (m) | 3 | 4.5 | 6 | 7.5 | 9 |
|---|---|---|---|---|---|
| $N_{60}$ | 16 | 20 | 22 | 24 | 26 |

Determine the friction angles at these depths using Peck et al. (1974), Schmertmann (1975), and Hatanaka and Uchida (1996) correlations.

### SOLUTION

| | | | | | Friction angle (degrees) | | |
|---|---|---|---|---|---|---|---|
| Depth (m) | $\sigma'_o$ (kN/m$^2$) | $N_{60}$ | $C_N$ (Liao & Whitman) | $(N_1)_{60}$ | Peck et al. | Schmertmann | Hatanaka & Uchida |
| 3.0 | 58.28 | 16 | 1.37 | 21.9 | 33.4 | 41.2 | 38.4 |
| 4.5 | 79.92 | 20 | 1.19 | 23.8 | 33.9 | 41.3 | 39.1 |
| 6.0 | 106.56 | 22 | 0.969 | 21.3 | 33.2 | 40.5 | 38.1 |
| 7.5 | 133.2 | 24 | 0.866 | 20.8 | 33.1 | 39.8 | 37.9 |
| 9.0 | 159.84 | 26 | 0.791 | 20.6 | 33.1 | 39.4 | 37.8 |

# VANE SHEAR TEST (VST)

# VANE SHEAR TEST (VST)

**Vane shear test is used to evaluate the in-situ undrained shear strength ($c_u$) of soft to stiff clays and silts. Both peak and remolded strengths can be measured and their ratio is termed soil sensitivity.**

***Advantages of VST*:**

- **Simple test and equipment**
- **Long history of use in practice**

***Disadvantages of VST:***

- **Limited application to soft to stiff clays and silts**
- **Slow and time-consuming**
- **Raw $c_u$ values need (empirical) correction**

# VANE SHEAR TEST (VST)

- VST consists of inserting a simple four-bladed vane into either clay or silt and rotating the device about a vertical axis and measuring the torque.
- Limit equilibrium is used to relate the measured torque to the undrained shear strength mobilized. Both peak and remolded strengths can be measured.
- A selection of vanes is available in terms of size, shape and configuration, depending on the consistency and strength of the soils.
- The standard vane (ASTM D 2573) has a rectangular geometry with a blade height to diameter ratio of 2.

This figure shows typical field vane
A standard 10 $cm^2$ cone penetrometer
is shown for scale.

# VANE SHEAR TEST (VST)

## Test Procedure

- Test procedures are outlined in ASTM D 2573.

- The test is often carried out by pushing the vane into the soil from the bottom of a borehole and the vane should be pushed at least four borehole diameters below the base of the borehole to avoid disturbance from drilling.

- The test can also be carried out using direct-push equipment pushing from the ground surface when there are no hard layers.

- Within 5 minutes after insertion, rotation should be carried out at a constant rate of 6 degrees per minute ($0.1^\circ$/s) with frequent measurements of the mobilized torque.

- Depending on the type of equipment used, there is the potential for friction to develop along the push rods. This friction needs to be either minimized or accounted for in the measurements.

# VANE SHEAR TEST (VST)

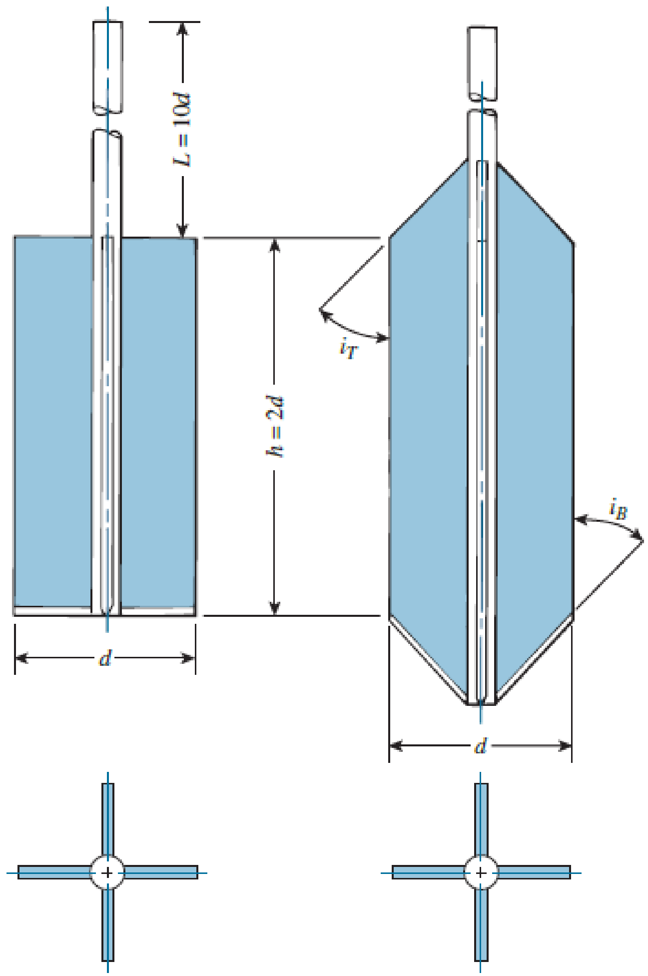


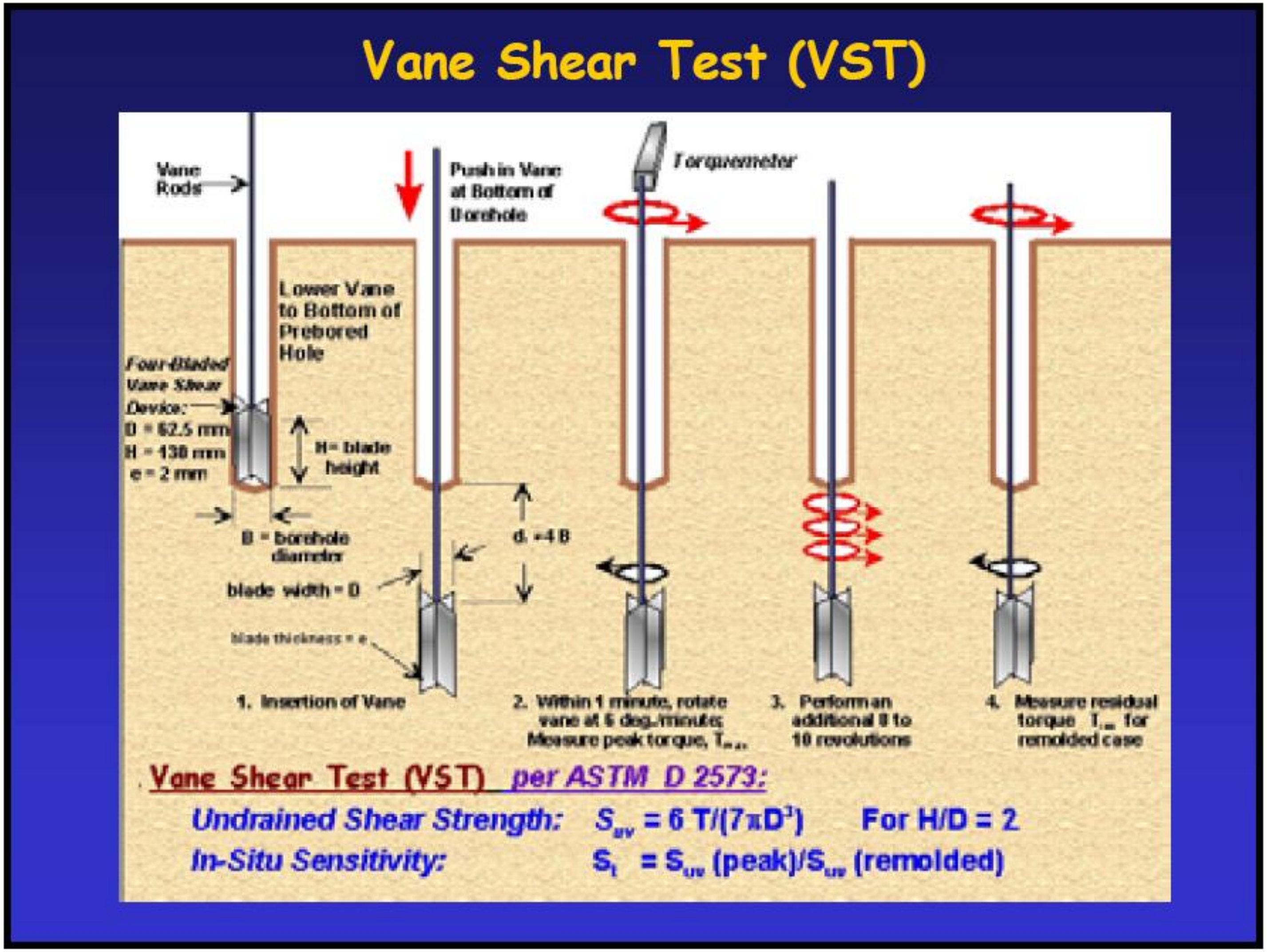

**TABLE 3.8** ASTM Recommended Dimensions of Field Vanes[a] (Based on *Annual Book of ASTM Standards, Vol. 04.08*)

| Casing size | Diameter, *d* mm | Height, *h* mm | Thickness of blade mm | Diameter of rod mm |
|---|---|---|---|---|
| AX | 38.1 | 76.2 | 1.6 | 12.7 |
| BX | 50.8 | 101.6 | 1.6 | 12.7 |
| NX | 63.5 | 127.0 | 3.2 | 12.7 |
| 101.6 mm[b] | 92.1 | 184.1 | 3.2 | 12.7 |

[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.

# VANE SHEAR TEST (VST)



# VANE SHEAR TEST (VST)

## Undrained Shear Strength

**The conventional interpretation to obtain the VST undrained shear strength from the maximum torque ($T_{max}$) assumes a uniform distribution of shear stresses both top and bottom and along the blades and a vane with a height-to-width ratio H/D = 2:**

According to ASTM (2014), for rectangular vanes,

$$K = \frac{\pi d^2}{2}\left(h + \frac{d}{3}\right)$$

If $h/d = 2$,

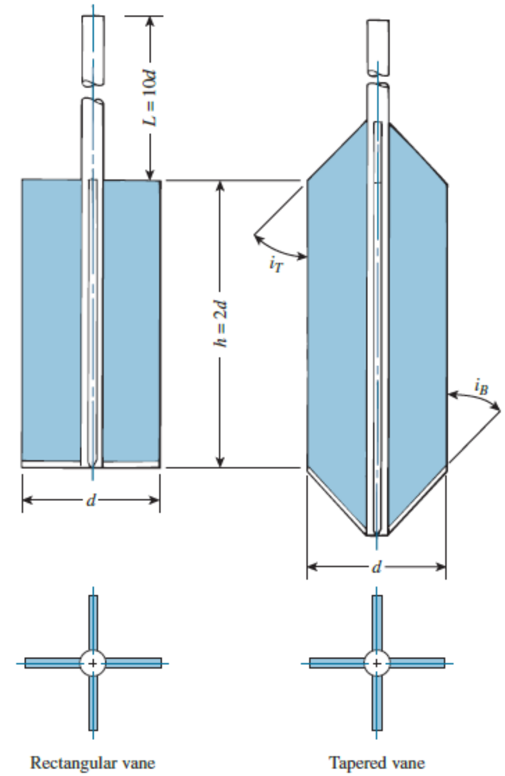
$$K = \frac{7\pi d^3}{6}$$

Thus,

$$c_u = \frac{6T}{7\pi d^3}$$

For tapered vanes,

$$K = \frac{\pi d^2}{12}\left(\frac{d}{\cos i_T} + \frac{d}{\cos i_B} + 6h\right)$$



# VANE SHEAR TEST (VST)

## Sensitivity

**After the peak $c_u$(peak) is obtained, the vane is rotated quickly through 10 complete revolutions and the test repeated to measure the remolded values($c_u$ (remolded)).**

**The sensitivity, $S_t$ is then:**

$$S_t = c_u(\text{peak}) / c_u(\text{remolded})$$

Vane Sensitivity Classification

| *Category* | *Sensitivity, $S_t$* |
|---|---|
| Insensitive | ~ 1 |
| Slightly sensitive | 1 - 2 |
| Medium sensitive | 2 - 4 |
| Very sensitive | 4 - 8 |
| Slightly quick | 8 - 16 |
| Medium quick | 16 - 32 |
| Very quick clay | 32 - 64 |
| Extra quick | > 64 |

# VANE SHEAR TEST (VST)

## Vane Correction Factor

**Since there is no unique value for the undrained shear strength of fine grained soils, it is common that the VST strength is corrected prior to application in stability analyses involving embankments on soft ground, bearing capacity and excavations in soft ground.**

$$C_u(\text{corrected}) = \lambda\, C_u(\text{VST})$$

**Where $\lambda$ is an empirical correction factor that has been related to plasticity index (PI) and void ratio.**

Bjerrum (1972) $\lambda = 1.7 - 0.54 \log[\text{PI}(\%)]$

Morris and Williams (1994)

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

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

# VANE SHEAR TEST (VST)

## Vane Correction Correlation

### Correlation between $c_u$ and Preconsolidation pressure

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

$\sigma'_c$ = preconsolidation pressure (kN/m$^2$)

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

### Correlation between $c_u$ and and overconsolidation ratio

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

where $\sigma'_o$ = effective overburden pressure.

- Mayne and Mitchell (1988):

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

- Hansbo (1957):

$$\beta = \frac{222}{w(\%)}$$

- Larsson (1980):

$$\beta = \frac{1}{0.08 + 0.0055(\text{PI})}$$

# EXAMPLE 3.3

## EXAMPLE 3.3

Refer to Figure 3.23. Vane shear tests (tapered vane) were conducted in the clay layer. The vane dimensions were 63.5 mm ($d$) × 127 m ($h$), and $i_T = i_B = 45°$. For a test at a certain depth in the clay, the torque required to cause failure was 20 N · m. For the clay, liquid limit was 50 and plastic limit was 18. Estimate the undrained cohesion of the clay for use in the design by using each equation:

a. Bjerrum's λ relationship [Eq. (3.40a)]
b. Morris and Williams' λ and PI relationship [Eq. (3.40b)]
c. Morris and Williams' λ and LL relationship [Eq. (3.40c)]
d. Estimate the preconsolidation pressure of clay, $\sigma'_c$.

### SOLUTION

**Part a**

Given: $h/d = 127/63.5 = 2$

From Eq. (3.38),

$$\begin{aligned}
K &= \frac{\pi d^2}{12}\left(\frac{d}{\cos i_T} + \frac{d}{\cos i_B} + 6h\right) \\
&= \frac{\pi(0.0635)^2}{12}\left[\frac{0.0635}{\cos 45} + \frac{0.0635}{\cos 45} + 6(0.127)\right] \\
&= (0.001056)(0.0898 + 0.0898 + 0.762) \\
&= 0.000994
\end{aligned}$$

From Eq. (3.34),

$$\begin{aligned}
c_{u(\mathrm{VST})} &= \frac{T}{K} = \frac{20}{0.000994} \\
&= 20{,}121\ \mathrm{N/m^2} \approx 20.12\ \mathrm{kN/m^2}
\end{aligned}$$

From Eqs. (3.40a) and (3.39),

$$\begin{aligned}
c_{u(\mathrm{corrected})} &= [1.7 - 0.54 \log(\mathrm{PI}\%)]c_{u(\mathrm{VST})} \\
&= [1.7 - 0.54 \log(50 - 18)](20.12) \\
&= \mathbf{17.85\ kN/m^2}
\end{aligned}$$

# EXAMPLE 3.3

**Part b**

From Eqs. (3.40b) and (3.39),

$$\begin{aligned} c_{u(\text{corrected})} &= [1.18e^{-0.08(\text{PI})} + 0.57]c_{u(\text{VST})} \\ &= [1.18e^{-0.08(50-18)} + 0.57](20.12) \\ &= \mathbf{13.3\ kN/m^2} \end{aligned}$$

**Part c**

From Eqs. (3.40c) and (3.39),

$$\begin{aligned} c_{u(\text{corrected})} &= [7.01e^{-0.08(\text{LL})} + 0.57]c_{u(\text{VST})} \\ &= [7.01e^{-0.08(50)} + 0.57](20.12) \\ &= \mathbf{14.05\ kN/m^2} \end{aligned}$$

**Part d**

From Eq. (3.41),

$$\sigma'_c = 7.04[c_{u(\text{VST})}]^{0.83} = 7.04(20.12)^{0.83} = \mathbf{85\ kN/m^2}$$

# CONE PENETRATION TEST (CPT)

# CONE PENETRATION TEST (CPT)

**The Cone Penetration Test (CPT)) has extensive applications in a wide range of soils. Although the CPT is limited primarily to softer soils, with modern larger pushing equipment and more robust cones, the CPT can be performed in stiff to very stiff soils, and in some cases soft rock.**

***Two types:***

1. **Mechanical friction-cone penetrometer**
2. **Electric friction-cone penetrometer**

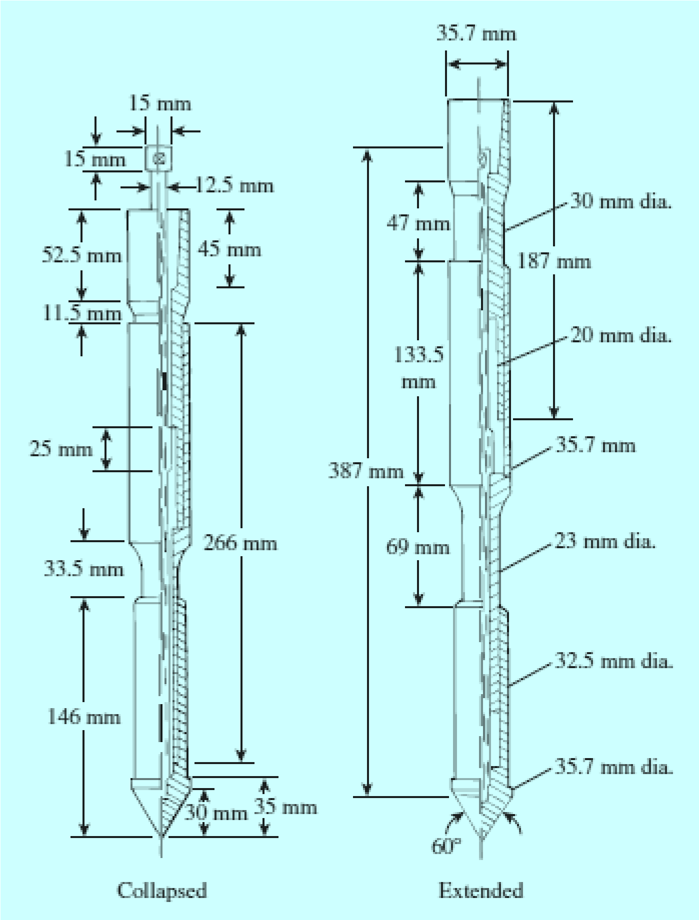
***Advantages of CPT:***

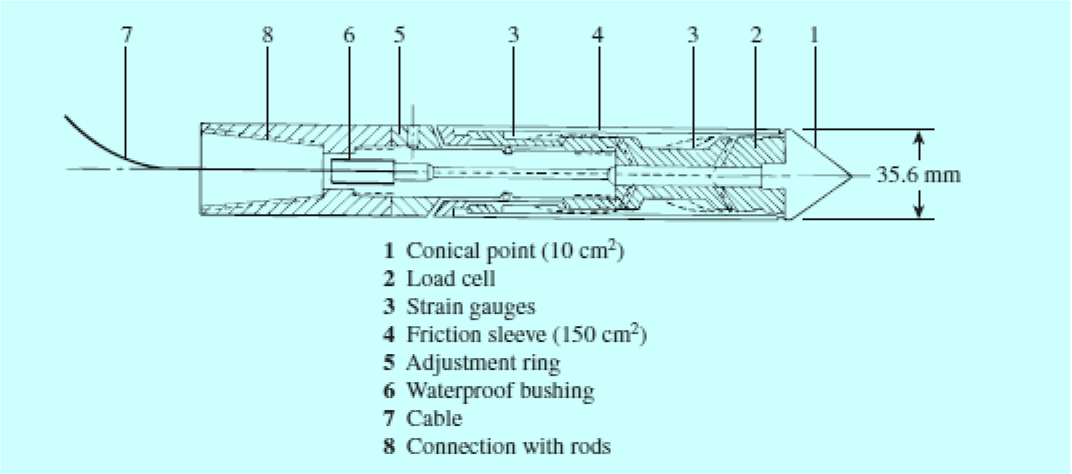
- **Fast and continuous profiling**
- **Repeatable and reliable data (not operator-dependent)**
- **Economical and productive**
- **Strong theoretical basis for interpretation**

***Disadvantage of CPT:***

- **High capital investment**
- **Requires skilled operators**
- **No soil sample**
- **Penetration can be restricted in gravel/cemented layers**

# CONE PENETRATION TEST (CPT)




Electric friction-cone penetrometer

Mechanical friction-cone penetrometer

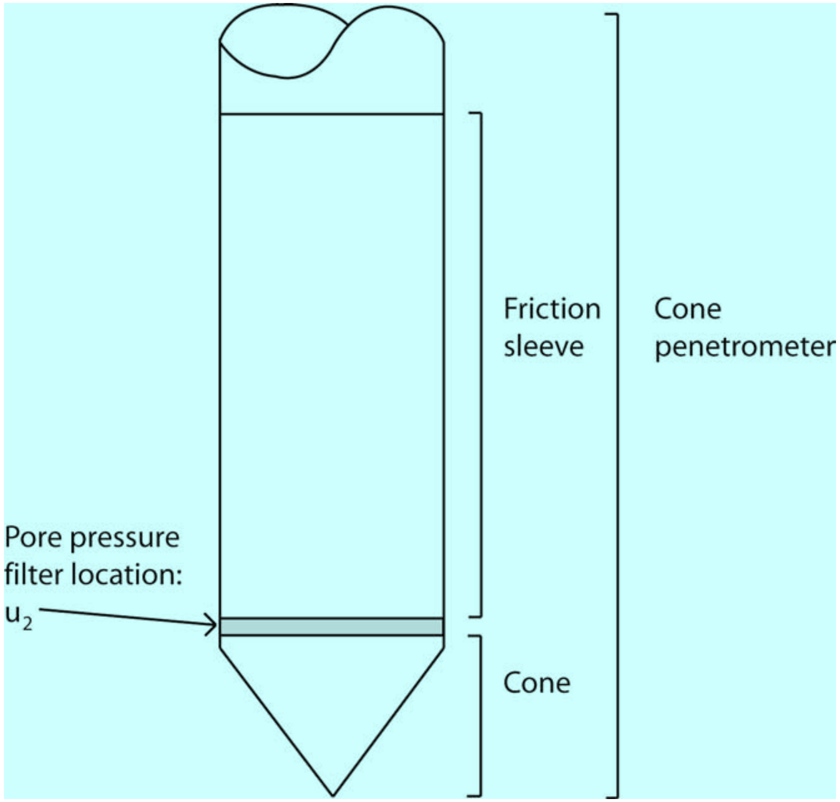
# CONE PENETRATION TEST (CPT)

In the Cone Penetration Test (CPT), a cone on the end of a series of rods is pushed into the ground at a constant rate and continuous measurements are made of the resistance to penetration of the cone and of a surface sleeve.

The total force acting on the cone, $Q_c$, divided by the projected area of the cone, $A_c$, produces the cone resistance, $q_c$.

The total force acting on the friction sleeve, $F_s$, divided by the surface area of the friction sleeve, $A_s$, produces the sleeve friction, $f_s$.

In a piezocone, pore pressure is also measured.



# CONE PENETRATION TEST (CPT)

Cone penetrometers come in a range of sizes with the 10 $cm^2$ and 15 $cm^2$ probes the most common and specified in most standards.

Figure shows a range of cones from a mini-cone at 2 $cm^2$ to a large cone at 40 $cm^2$. The mini cones are used for shallow investigations, whereas the large cones can be used in gravely soils.

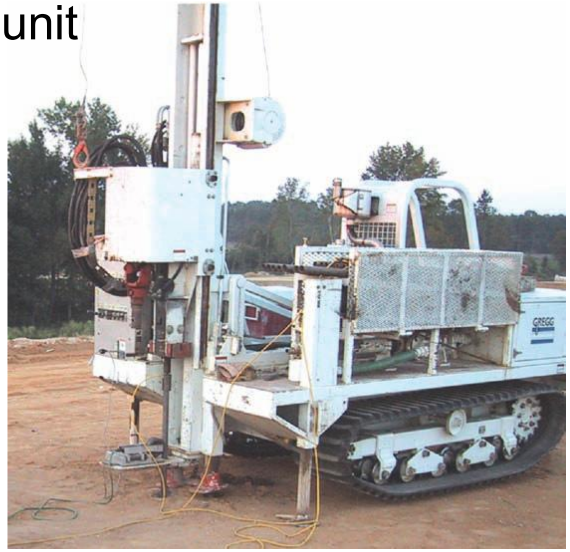
# CONE PENETRATION TEST (CPT)

# CONE PENETRATION TEST (CPT)

**Pushing equipment for on land applications generally consist of specially built units that are either truck or track mounted. CPT's can also be carried out using an anchored drill-rig.**

Truck mounted 25 ton CPT unit

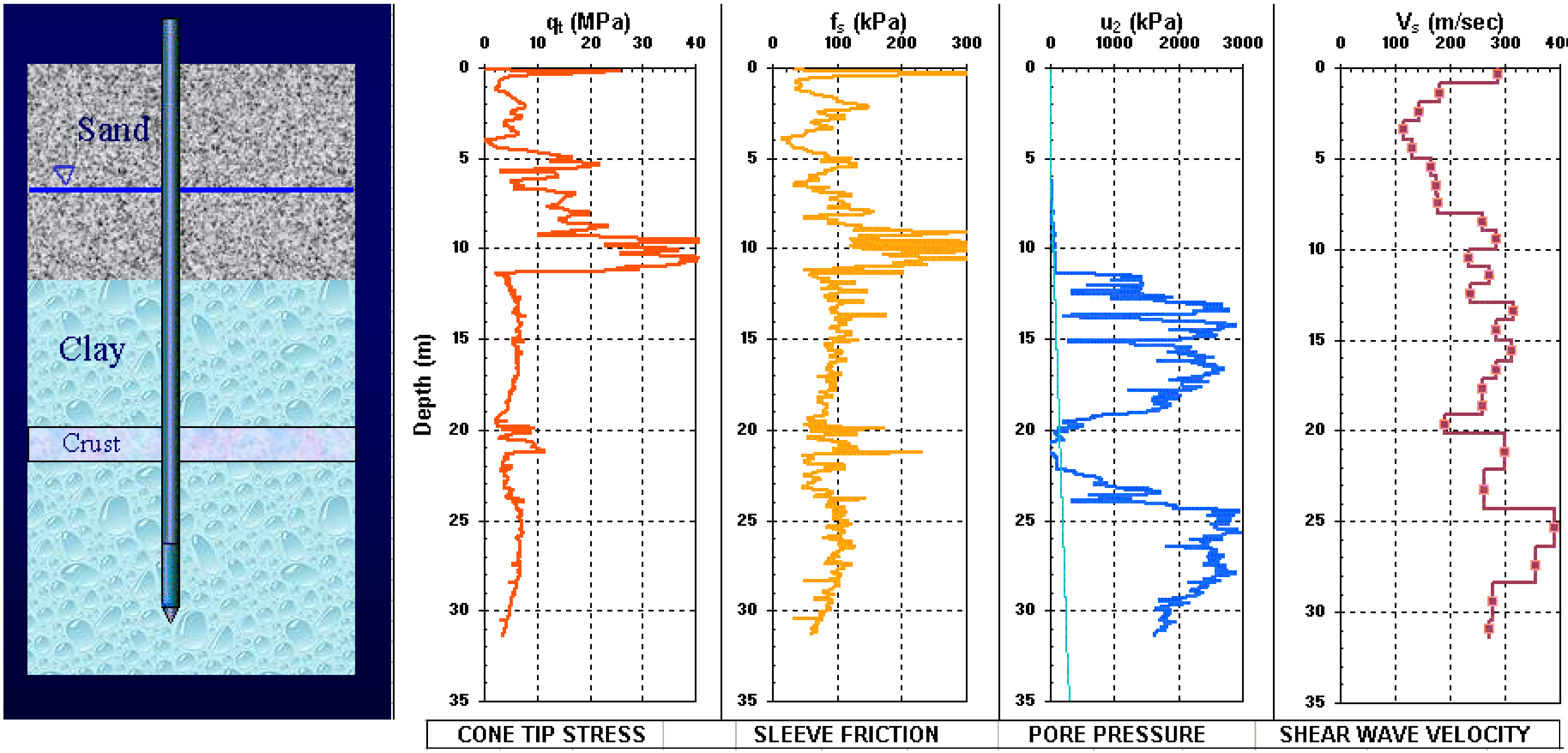
Small anchored drill-rig unit



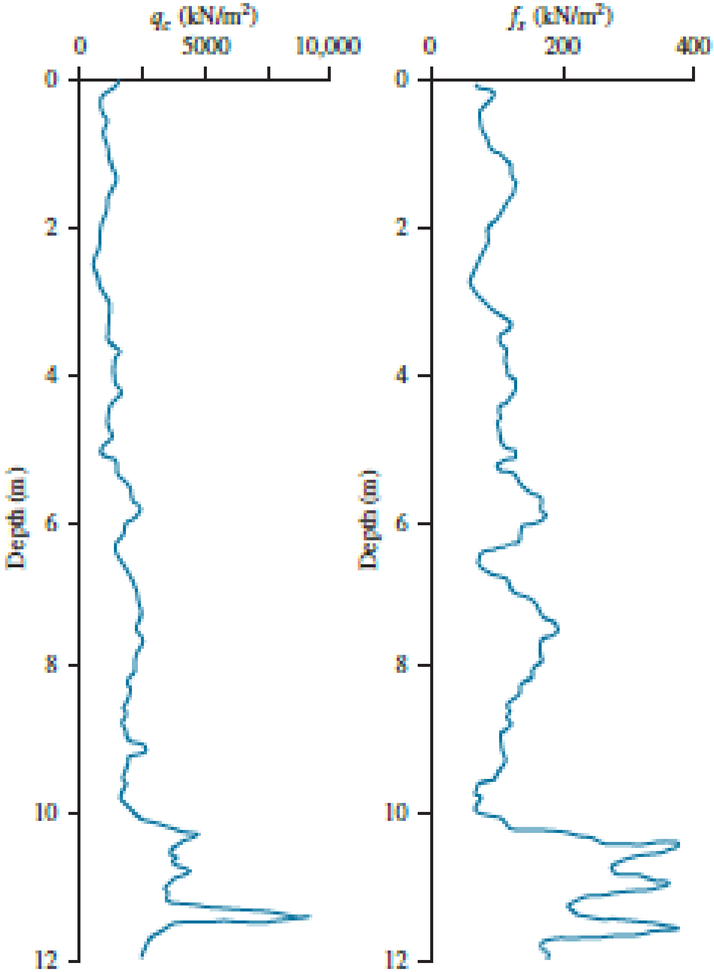
CPT inside buildings or limited access

# CONE PENETRATION TEST (CPT)

## Real-Time readings in computer screen



# CONE PENETRATION TEST (CPT)



# FRICTION RATIO (Fr)

$$F_r(\%) = \frac{\text{sleeve friction, } f_s}{\text{cone resistance, } q_c} \times 100$$

It varies in the range of 0–10%, with the lower end of the range for granular soil and the upper end for cohesive soil.

Anagnostopoulos et al. (2003) expressed $F_r$ as

$$F_r(\%) = 1.45 - 1.36 \log D_{50} \text{ (electric cone)}$$

$$F_r(\%) = 0.7811 - 1.611 \log D_{50} \text{ (mechanical cone)}$$

where $D_{50}$ = size through which 50% of soil will pass through (mm).
The $D_{50}$ for soil ranged from 0.001 mm to about 10 mm.

# $q_c$ CORRELATIONS

$q_c$ can be used for calculating some important parameters such as:

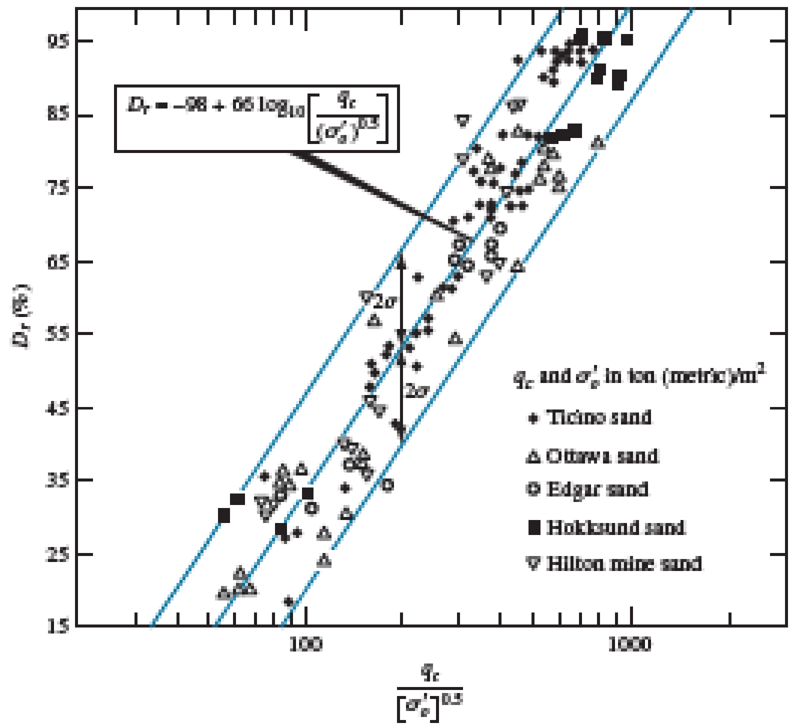
- Relative Density ($D_r$)
- Angle of internal friction ($\phi$)
- $N_{60}$
- Soil Type
- Undrained shear strength ($C_u$)
- Preconsolidation pressure
- Overconsolidation ratio (OCR)

# $q_c$ CORRELATIONS

## Relative Density ($D_r$)

### Lancellotta (1983) and Jamiolkowski et al. (1985)

$$D_r(\%) = A + B \log_{10}\left(\frac{q_c}{\sqrt{\sigma'_v}}\right)$$



# $q_c$ CORRELATIONS

## Relative Density ($D_r$)

### Kulhawy and Mayne, 1990

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

$$D_r = \sqrt{\left[\frac{1}{305 Q_c \mathrm{OCR}^{1.8}}\right]\left[\frac{\frac{q_c}{p_a}}{\left(\frac{\sigma'_o}{p_a}\right)^{0.5}}\right]}$$

where

$p_a$ = atmospheric pressure ($\approx 100$ kN/m²)

$\sigma'_o$ = vertical effective stress

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

# $q_c$ CORRELATIONS

## Angle of internal friction (ϕ)

Robertson and Campanella (1983)

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

Ricceri et al. (2002) for soil with classifications of ML and SP-SM

$$\phi' = \tan^{-1}\left[0.38 + 0.27\log\left(\frac{q_c}{\sigma'_o}\right)\right]$$

Lee et al. (2004)

$$\phi' = 15.575\left(\frac{q_c}{\sigma'_h}\right)^{0.1714}$$

$(\sigma'_h)$ = horizontal effective stress

# $q_c$ CORRELATIONS

## $N_{60}$

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = cD_{50}^{a}$$

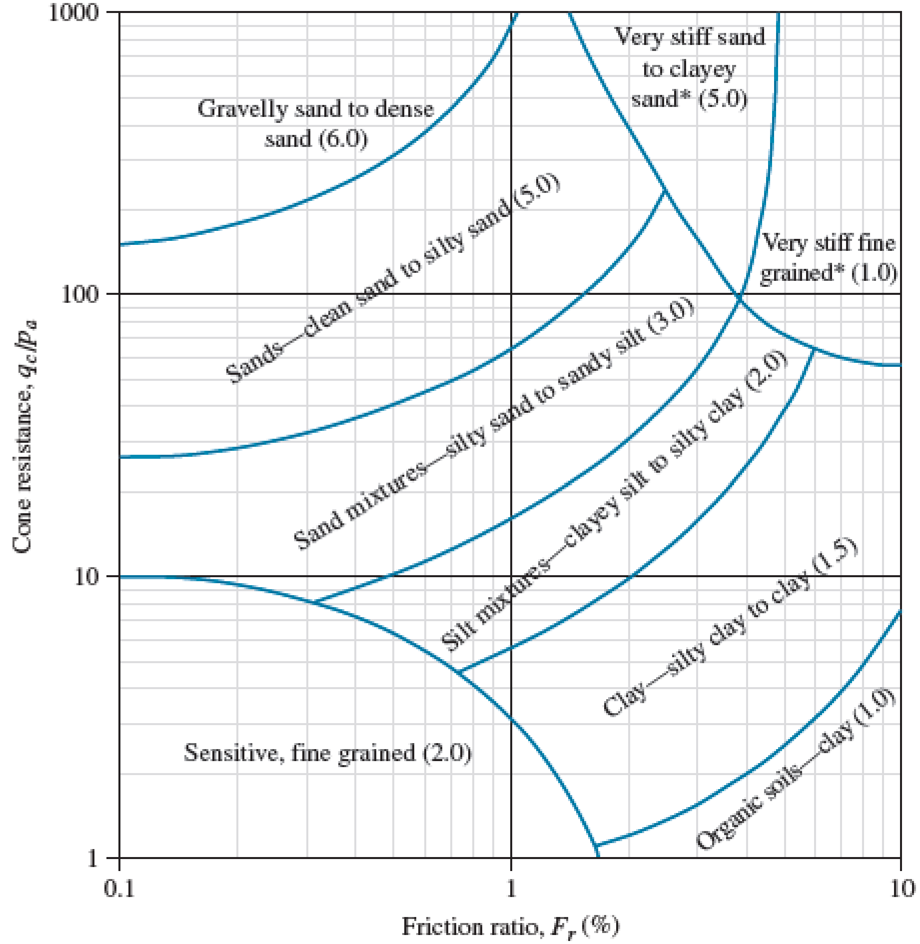
TABLE 3.9 Values of $c$ and $a$ [Eq. (3.55)]

| Investigator | | $c$ | $a$ |
|---|---|---|---|
| Burland and Burbidge (1985) | Upper limit | 15.49 | 0.33 |
| | Lower limit | 4.9 | 0.32 |
| Robertson and Campanella (1983) | Upper limit | 10 | 0.26 |
| | Lower limit | 5.75 | 0.31 |
| Kulhawy and Mayne (1990) | | 5.44 | 0.26 |
| Anagnostopoulos et al. (2003) | | 7.64 | 0.26 |

# $q_c$ CORRELATIONS

## Soil Type

The values of $(q_c/p_a)/N_{60}$ are shown in parentheses.




Note: *Heavily overconsolidated or cemented

# $q_c$ CORRELATIONS

## Undrained shear strength ($C_u$)

$$c_u = \frac{q_c - \sigma_o}{N_k}$$

$\sigma_o$ = total vertical stress

$N_k$ = bearing capacity factor

$$c_u = \frac{f_s}{1.26} \text{ (for mechanical cones)}$$

$$c_u = f_s \text{ (for electrical cones)}$$

Mayne and Kemper (1988)

$N_k = 15$ (for electric cone)

$N_k = 20$ (for mechanical cone)

Anagnostopoulos et al. (2003)

$N_k = 17.2$ (for electric cone)

$N_k = 18.9$ (for mechanical cone)

# $q_c$ CORRELATIONS

## Preconsolidation pressure

Mayne and Kemper (1988)

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

# $q_c$ CORRELATIONS

## Overconsolidation ratio (OCR)

Mayne and Kemper (1988)

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

where $\sigma_o$ and $\sigma'_o$ = total and effective stress, respectively.

# EXAMPLE 3.4

EXAMPLE 3.4

At a depth of 12.5 m in a *moderately compressible sand deposit*, a cone penetration test showed $q_c = 20$ MN/m$^2$. For the sand given, $\gamma = 16$ kN/m$^3$ and OCR = 2. Estimate the relative density of the sand.

## SOLUTION

Vertical effective stress $\sigma'_o = (12.5)(16) = 200$ kN/m$^2$.

$Q_c$ (moderately compressible sand) $\approx 1$.

$$D_r = \sqrt{\left[\frac{1}{305 Q_c \mathrm{OCR}^{1.8}}\right]\left[\frac{\dfrac{q_c}{p_a}}{\left(\dfrac{\sigma'_o}{p_a}\right)^{0.5}}\right]}$$

$$= \sqrt{\frac{1}{(305)(2)^{1.8}}\left[\frac{\left(\dfrac{20{,}000 \text{ kN/m}^2}{100 \text{ kN/m}^2}\right)}{\left(\dfrac{200 \text{ kN/m}^2}{100 \text{ kN/m}^2}\right)^{0.5}}\right]}$$

$$= \sqrt{(0.00094)(141.41)} = 0.365$$

$$D_r = 36.5\%$$

# PRESSUREMETER TEST (PMT)

# PRESSUREMETER TEST (PMT)

**The pressuremeter test can be used to evaluate the stress-strain response of a wide range of soils and rock. It consists of a probe with three cells. The top and bottom ones are guard cells and the middle is the measuring cell.**
**There are three basic types of pressuremeter devices, Pre-bored, Self-bored and Full-displacement, each with different abilities and challenges.**

***Advantages of PMT*:**

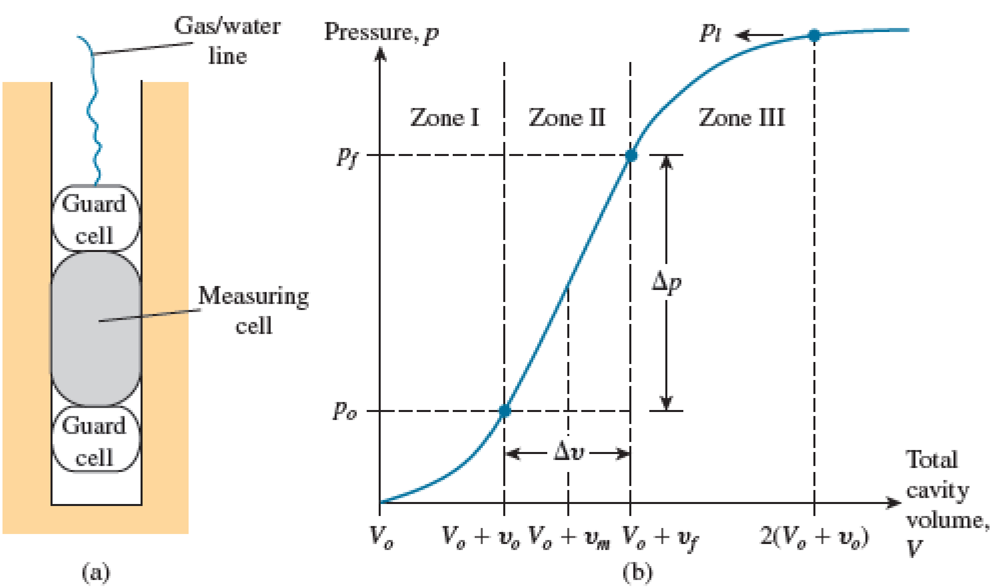
- **Strong theoretical basis for interpretation**
- **Tests large volume of ground**

***Disadvantages of PMT:***

- **Complicated equipment and procedures**
- **Requires skilled operator**
- **Time consuming and expensive**
- **Equipment can be easily damaged**

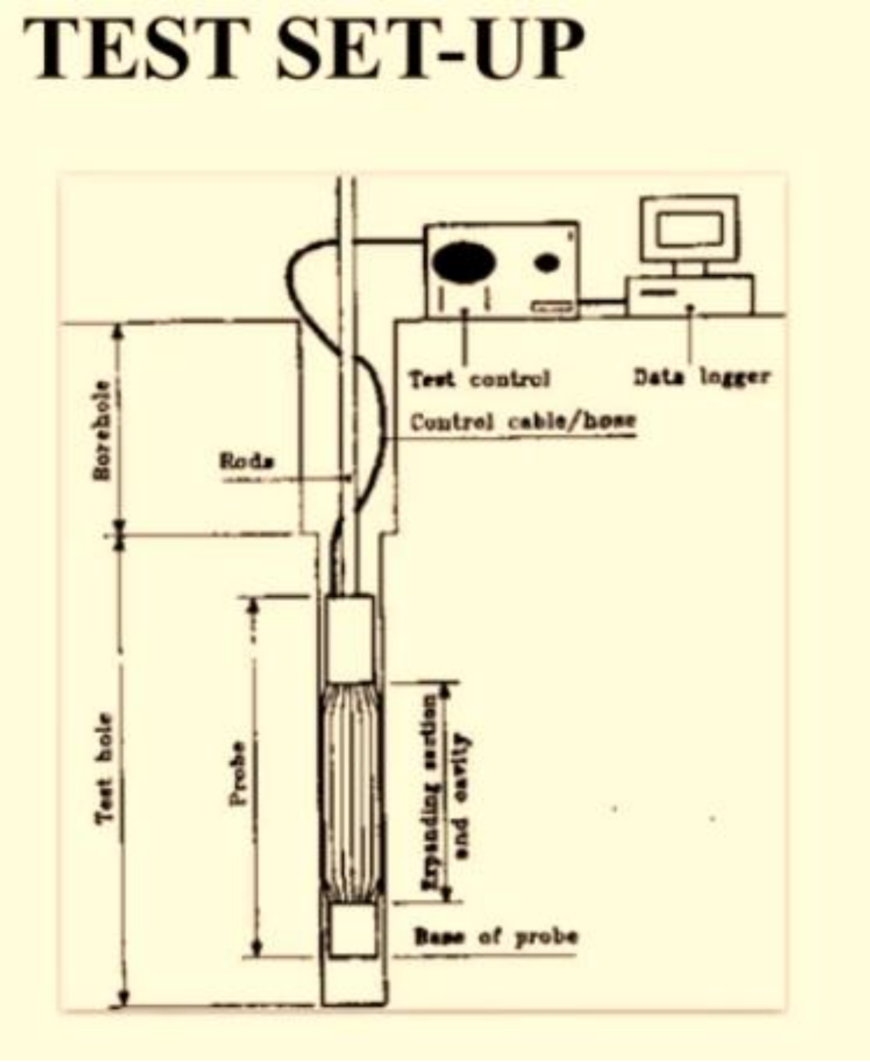
# PRESSUREMETER TEST (PMT)

| Probe diameter (mm) | Borehole diameter | |
|---|---|---|
| | Nominal (mm) | Maximum (mm) |
| 44 | 45 | 53 |
| 58 | 60 | 70 |
| 74 | 76 | 89 |




(a) Pressuremeter; (b) plot of pressure versus total cavity volume

# PRESSUREMETER TEST (PMT)



# PRESSUREMETER TEST (PMT)

(a) the pressuremeter probe

(b) drilling the bore hole by wet rotary method

(c) pressuremeter control unit with probe in the background

(d) getting ready to insert the pressuremeter probe into the borehole

(a)

(c)

(b)

(d)

# PRESSUREMETER TEST (PMT)

## PROCEDURE

Test procedure consists of three steps as follows:

- Drilling borehole.

Separate drilling equipment is used and preferably which causes least disturbance to the soil while drilling. The diameter of the borehole should be in between 1.03 times to 1.20 times the diameter of the probe.

- Positioning of probe in the bore bole.

The probe should be lowered slowly without disturbing the surrounding soil and the apparatus itself. After reached desired elevation, the probe is fixed using clamping device.

- Conducting test.

Cells of probe with water and gas. This action is done by using control unit of the pressuremeter. Equal pressure is maintained in both the measuring and guard cells. Now, using measuring cell pressure is applied on the soil wall of borehole.

# PRESSUREMETER TEST (PMT)

Baguelin et al. (1978)

$$c_u = \frac{(p_l - p_o)}{N_p}$$

$c_u$ = undrained shear strength of a clay

$$N_p = 1 + \ln\left(\frac{E_p}{3c_u}\right)$$

$$\text{Clay: } E_p(\text{kN/m}^2) = 1930\, N_{60}^{0.63}$$

$$\text{Sand: } E_p(\text{kN/m}^2) = 908\, N_{60}^{0.66}$$

# FLAT PLATE DILATOMETER TEST (DMT)

# FLAT PLATE DILATOMETER TEST (DMT)

**The flat plate dilatometer test (DMT) can be used to estimate a wide range of geotechnical parameters in primarily softer soils.**

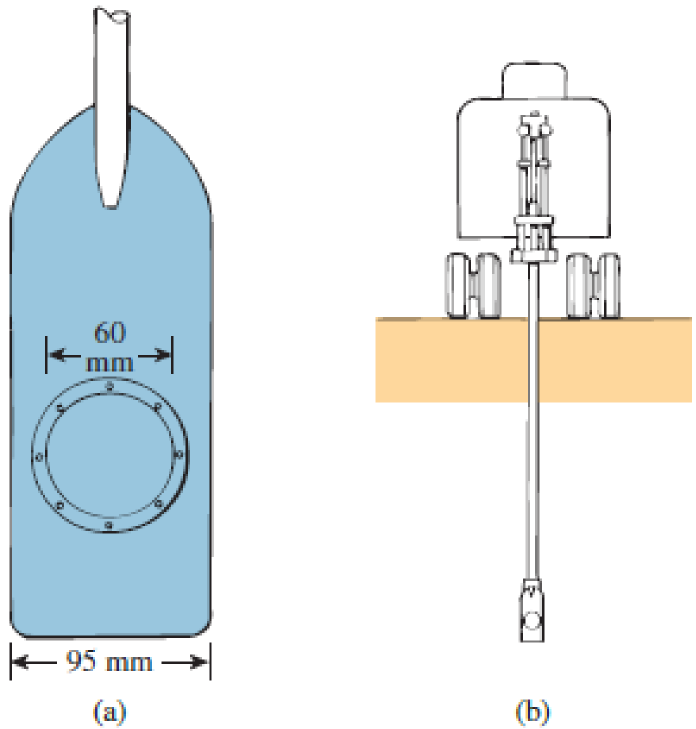
***Advantages of DMT:***

- **Simple and robust**
- **Repeatable and reliable data (not operator-dependent)**
- **Economical**

***Disadvantage of DMT:***

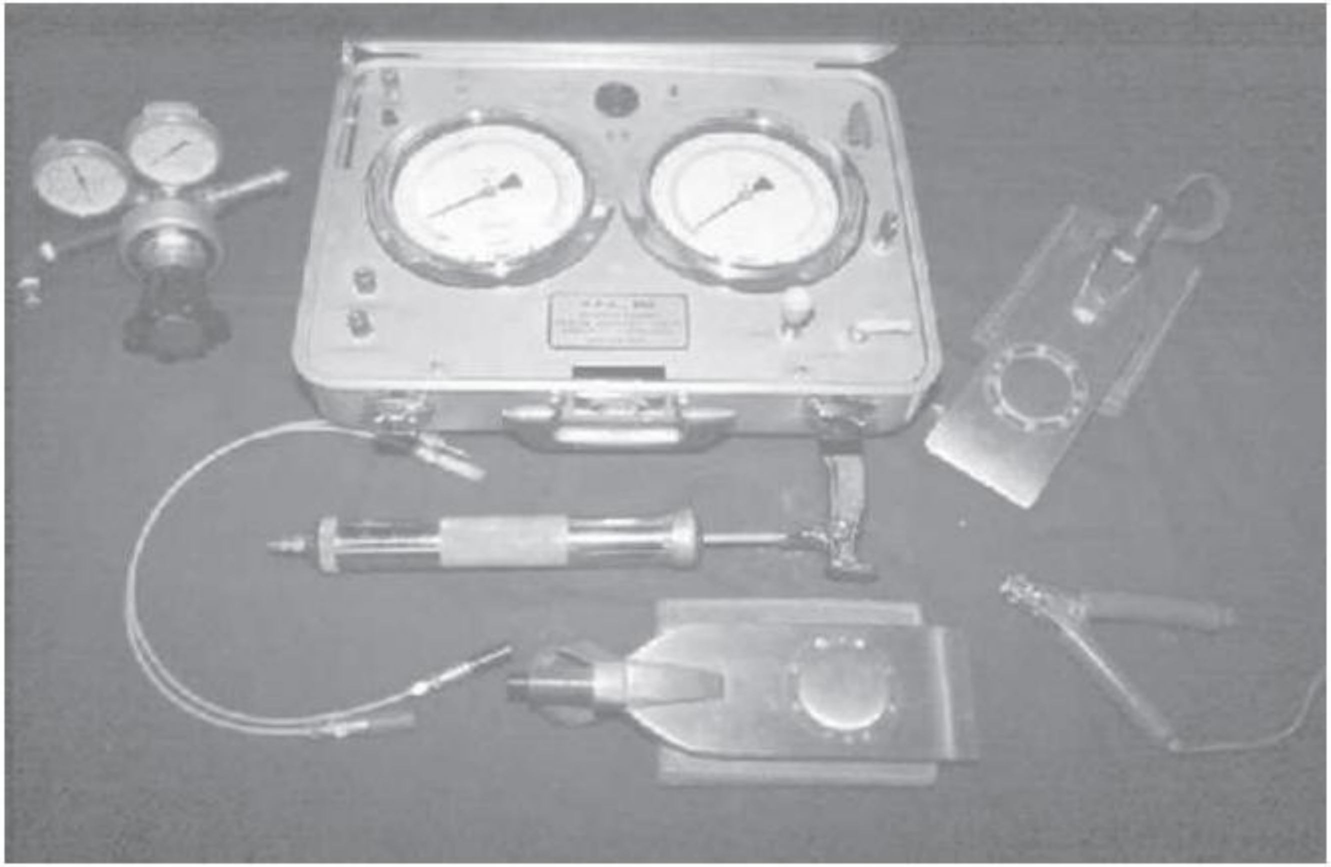
- **Difficult to push into dense and hard materials**
- **Weak theoretical basis for interpretation**
- **No soil sample**
- **Penetration can be restricted in gravel/cemented layers**

# FLAT PLATE DILATOMETER TEST (DMT)




(a) Schematic diagram of a flat-plate dilatometer (b) dilatometer probe inserted into ground

# FLAT PLATE DILATOMETER TEST (DMT)



Dilatometer and other accessories

# FLAT PLATE DILATOMETER TEST (DMT)

The $A$ and $B$ readings are corrected as follows (Schmertmann, 1986):

Contact stress, $p_o = 1.05(A + \Delta A - Z_m) - 0.05(B - \Delta B - Z_m)$

Expansion stress, $p_1 = B - Z_m - \Delta B$

where

$\Delta A$ = vacuum pressure required to keep the membrane in contact with its seating

$\Delta B$ = air pressure required inside the membrane to deflect it outward to a center expansion of 1.1 mm

$Z_m$ = gauge pressure deviation from zero when vented to atmospheric pressure

The test is normally conducted at depths 200 to 300 mm apart. The result of a given test is used to determine three parameters:

1. Material index, $I_D = \dfrac{p_1 - p_o}{p_o - u_o}$
2. Horizontal stress index, $K_D = \dfrac{p_o - u_o}{\sigma'_o}$
3. Dilatometer modulus, $E_D(\text{kN/m}^2) = 34.7(p_1 \text{ kN/m}^2 - p_o \text{ kN/m}^2)$

where

$u_o$ = pore water pressure

$\sigma'_o$ = *in situ* vertical effective stress

# FLAT PLATE DILATOMETER TEST (DMT)

$$K_o = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.6$$

$$\text{OCR} = (0.5K_D)^{1.56}$$

$$\frac{c_u}{\sigma'_o} = 0.22 \qquad \text{(for normally consolidated clay)}$$

$$\left(\frac{c_u}{\sigma'_o}\right)_{\text{OC}} = \left(\frac{c_u}{\sigma'_o}\right)_{\text{NC}} (0.5K_D)^{1.25}$$

$$E_s = (1 - \mu_s^2)E_D$$

where

$K_o$ = coefficient of at-rest earth pressure
OCR = overconsolidation ratio
OC = overconsolidated soil
NC = normally consolidated soil
$E_s$ = modulus of elasticity

- For undrained cohesion in clay (Kamei and Iwasaki, 1995):

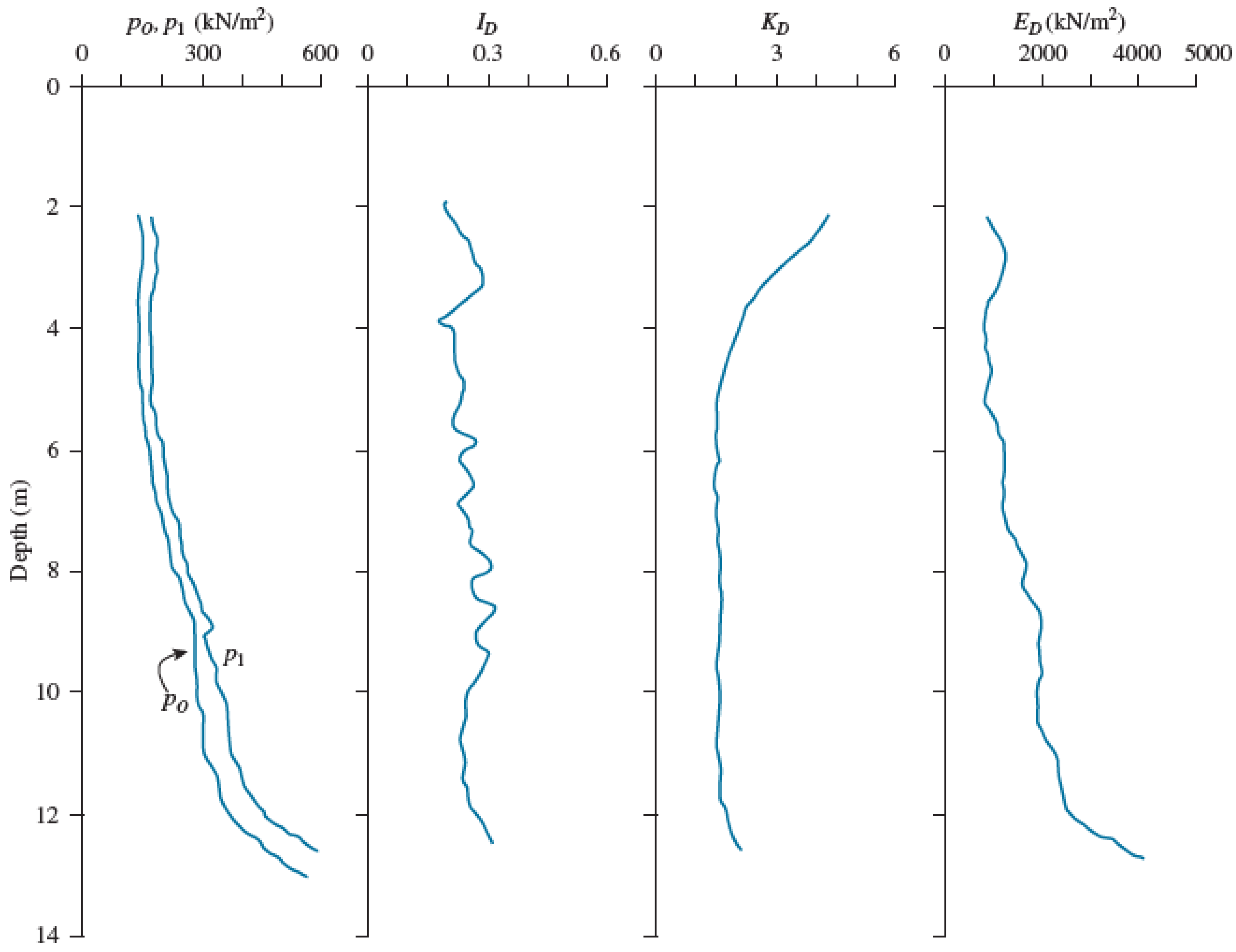
$$c_u = 0.35\,\sigma'_0\,(0.47K_D)^{1.14}$$

- For soil friction angle (ML and SP-SM soils) (Ricceri et al., 2002):

$$\phi' = 31 + \frac{K_D}{0.236 + 0.066K_D}$$

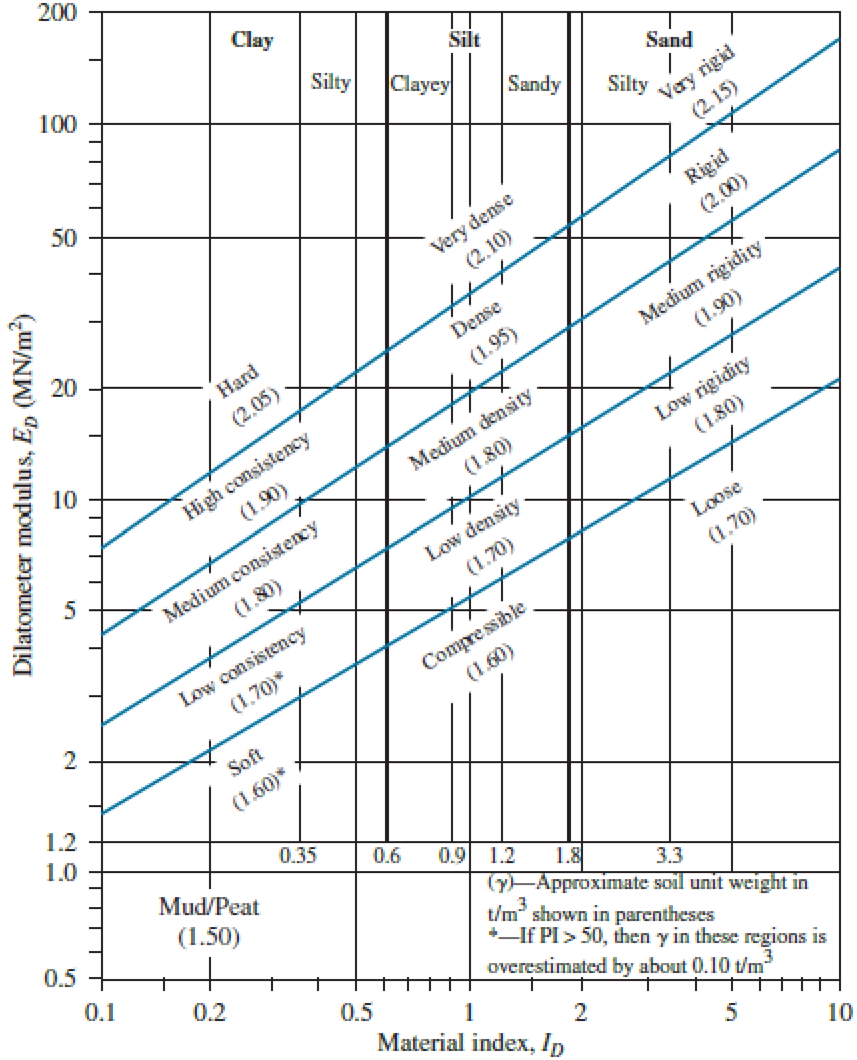
$$\phi'_{ult} = 28 + 14.6\log K_D - 2.1(\log K_D)^2$$

# FLAT PLATE DILATOMETER TEST (DMT)




A dilatometer test result conducted on soft Bangkok clay

# FLAT PLATE DILATOMETER TEST (DMT)

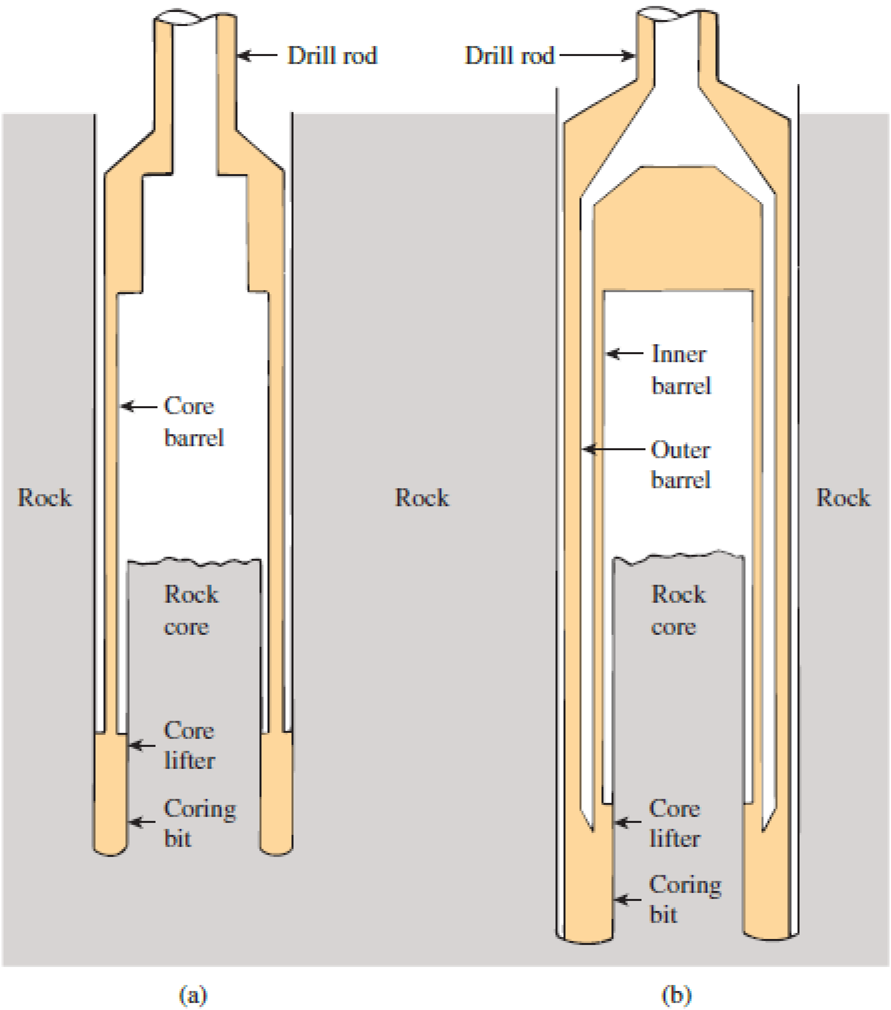



(*Note:* 1 t/m$^3$ = 9.81 kN/m$^3$)

Chart for determination of soil description and unit weight

# CORING OF ROCKS

# CORING OF ROCKS




Rock coring: (a) single-tube core barrel; (b) double-tube core barrel

# CORING OF ROCKS

Standard Size and Designation of Casing, Core Barrel, and Compatible Drill Rod

| Casing and core barrel designation | Outside diameter of core barrel bit (mm) | Drill rod designation | Outside diameter of drill rod (mm) | Diameter of borehole (mm) | Diameter of core sample (mm) |
|---|---|---|---|---|---|
| EX | 36.51 | E | 33.34 | 38.1 | 22.23 |
| AX | 47.63 | A | 41.28 | 50.8 | 28.58 |
| BX | 58.74 | B | 47.63 | 63.5 | 41.28 |
| NX | 74.61 | N | 60.33 | 76.2 | 53.98 |

# CORING OF ROCKS

Diamond coring bit

# CORING OF ROCKS

Diamond coring bit attached to a double-tube core barrel: (a) end view
(b) side view

# CORING OF ROCKS

**Core barrel samplers are originally designed to sample rock.**

**Single tube sampler**

**The core barrel of the sampler rotates and this poses the possibility of disturbing the sample by shearing the sample along certain weak planes. Moreover, the cored samples are subjected to erosion and disturbance by the drilling fluid.**
**The rock cores obtained can be highly disturbed and fractured because of torsion.**

**Double tube samplers**

**The tube samplers do not rotate with the core barrels and the samplers are not protected against the drilling fluid. The logging of samples presents difficulty for highly fractured rock.**

# CORING OF ROCKS

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

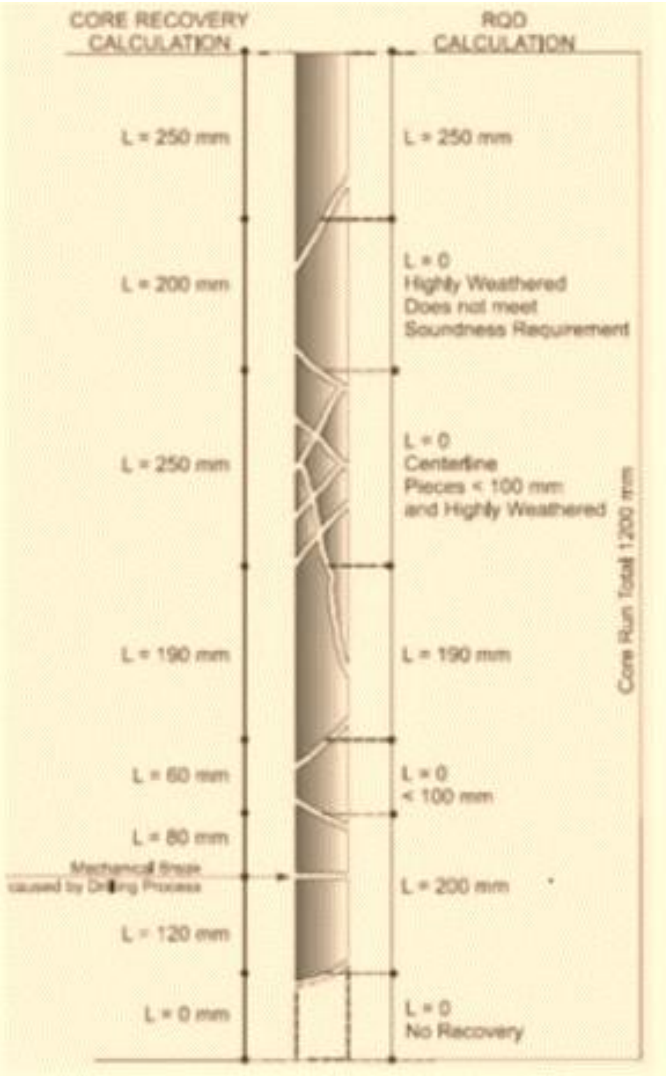
Rock quality designation (RQD)

$$= \frac{\Sigma \text{ length of recovered pieces equal to or larger than 101.6 mm}}{\text{theoretical length of rock cored}}$$

**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 |

# CORING OF ROCKS




- **Core recovery (CR)** = (total length of rock recovered / Total core run length)x100.
- ➢ Total length of rock recovered = 250+200+250+190+60+80+120 = 1150mm
- ➢ Total core run length = 1200mm. Therefore, Core recovery (CR) = (1150/1200)x100 = 96%.

- **Rock quality designation RQD** = (SUM(length of sound pieces >100mm)/Total core run length)x100
- ➢ SUM (length of sound pieces >100mm) = 250+190+200 = 640mm
- ➢ Therefore, RQD = (640/1200) x100 = 53% which is fair quality rocks, i.e. rocks are moderately weathered.

# 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. Figure 3.44 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).

# BORING LOGS

Boring Log

Name of the Project: Two-story apartment building

Location: Johnson & Olive St. Date of Boring: March 2, 2005

Boring No. 3 Type of Boring: Hollow-stem auger Ground Elevation: 60.8 m

| Soil description | Depth (m) | Soil sample type and number | $N_{60}$ | $w_n$ (%) | Comments |
|---|---|---|---|---|---|
| Light brown clay (fill) | | | | | |
| Silty sand (SM) | 1 | | | | |
| | 2 | SS-1 | 9 | 8.2 | |
| °G.W.T. 3.5 m | 3 | SS-2 | 12 | 17.6 | LL = 38; PI = 11 |
| Light gray clayey silt (ML) | 4 | | | | |
| | 5 | ST-1 | | 20.4 | LL = 36; $q_u$ = 112 kN/m$^2$ |
| | 6 | SS-3 | 11 | 20.6 | |
| Sand with some gravel (SP) | 7 | | | | |
| End of boring @ 8 m | 8 | SS-4 | 27 | 9 | |

$N_{60}$ = standard penetration number
$w_n$ = natural moisture content
LL = liquid limit; PI = plasticity index
$q_u$ = unconfined compression strength
SS = split-spoon sample; ST = Shelby tube sample

°Groundwater table observed after one week of drilling

*Figure 3.44* A typical boring log

# GEOPHYSICAL EXPLORATION

# GEOPHYSICAL EXPLORATION

- Although boring and test pits provide definite results but they are time consuming and expensive.
- Subsurface conditions are known only at the bore or test pit location.
- The subsurface conditions between the boring need to be interpolated or estimated.
- Geophysical methods are more quick and cheaper.
- They provide thorough coverage of the entire area.
- The results of geophysical testing however are less definitive and require subjective interpretation.
- Therefore both methods are important. In case geophysical testing in major in scope, few borings and sampling will be required for accurate determination of soil properties.
- If boring is major in scope then few geophysical lines will be required to know the conditions in-between the borings.

# GEOPHYSICAL TEST METHODS

## Advantages

- ✓ Many geophysical tests are non-invasive and thus offer significant benefits in cases where conventional drilling, testing, and sampling are difficult (e.g., deposits of gravel, talus deposits) or where potentially contaminated soils may occur in the subsurface.
- ✓ In general, geophysical testing covers a relatively large area, thus providing the opportunity to characterize large areas with few tests. It is particularly well-suited to projects that have large longitudinal extent compared to lateral extent (such as for new highway construction).
- ✓ Geophysical measurement assesses the characteristics of soil and rock at very small strains, typically on the order of 0.001 percent thus providing information on truly elastic properties.
- ✓ For the purpose of obtaining information on the subsurface, geophysical methods are relatively inexpensive when considering cost relative to the relatively large areas over which information can be obtained.

# GEOPHYSICAL TEST METHODS

## Disadvantages

- Most methods work best for situations in which there is a large difference in stiffness between adjacent subsurface units.
- It is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material over soft material
- Results are generally interpreted qualitatively and therefore useful results can only be obtained by an experienced engineer or geologist familiar with the particular testing method.
- Specialized equipment is required (compared to more conventional subsurface exploration tools).

# GEOPHYSICAL TEST METHODS

There are a number of different geophysical in-situ tests that can be used for stratigraphic information and in the determination of engineering properties. The most common methods are:

**Three methods**

1. Seismic Refraction Survey
2. Cross-Hole Seismic Survey
3. Electrical Resistivity Survey

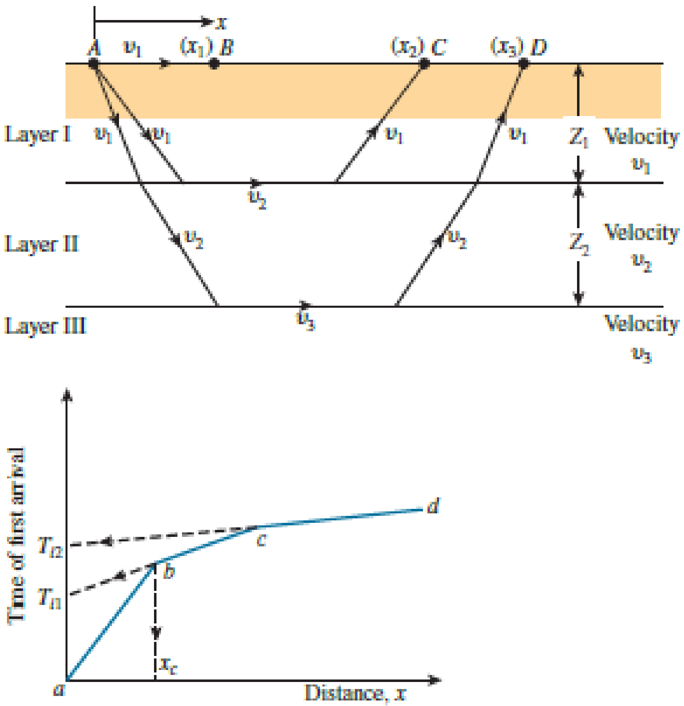
# SEISMIC REFRACTION SURVEY

# SEISMIC REFRACTION SURVEY

- **Useful in obtaining preliminary information about the <u>thickness of the layering</u> of various soils and the <u>depth to rock or hard soil</u>.**
- **It is conducted by impacting the surface and observing the first arrival of the disturbance (stress wave) at several other points.**
- **The impact can be created by a hammer blow or by a small explosive charge.**
- **The first arrival of disturbance waves at various points can be recorded by geophones.**
- **A graph of travel time versus distance is established**
- **Two types of stress waves:**
  - **P waves (plane waves)**
  - **S waves (shear waves)**

**P faster than S.**

# SEISMIC REFRACTION SURVEY




Seismic refraction survey

$$v = \sqrt{\frac{E_s}{\left(\frac{\gamma}{g}\right)} \frac{(1-\mu_s)}{(1-2\mu_s)(1+\mu_s)}}$$

where

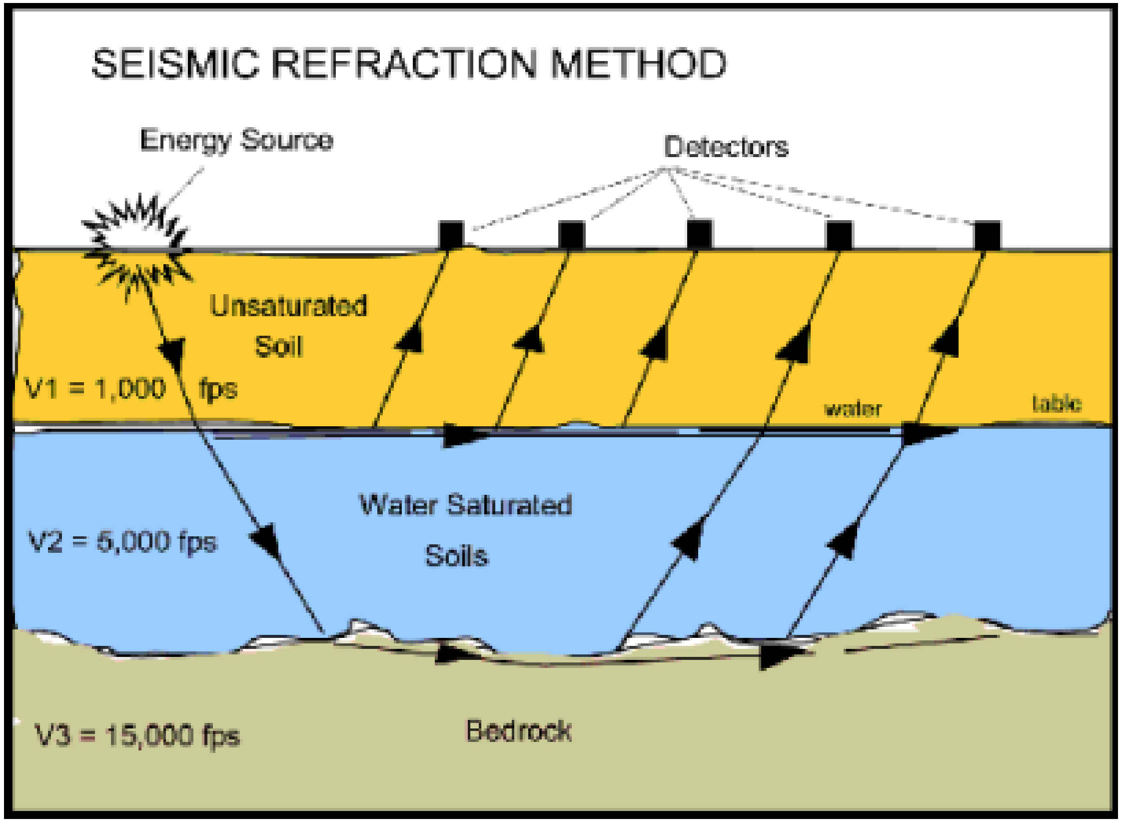
$E_s$ = modulus of elasticity of the medium

$\gamma$ = unit weight of the medium

$g$ = acceleration due to gravity

$\mu_s$ = Poisson's ratio

# SEISMIC REFRACTION SURVEY



# SEISMIC REFRACTION SURVEY

To determine the velocity $v$ of $P$ waves in various layers and the thicknesses of those layers, we use the following procedure:

***Step 1.*** Obtain the times of first arrival, $t_1, t_2, t_3, \ldots$, at various distances $x_1, x_2, x_3, \ldots$ from the point of impact.

***Step 2.*** Plot a graph of time $t$ against distance $x$.

***Step 3.*** Determine the slopes of the lines $ab$, $bc$, $cd$, . . . :

$$\text{Slope of } ab = \frac{1}{v_1}$$

$$\text{Slope of } bc = \frac{1}{v_2}$$

$$\text{Slope of } cd = \frac{1}{v_3}$$

Here, $v_1, v_2, v_3, \ldots$ are the $P$-wave velocities in layers I, II, III, . . . , respectively

***Step 4.*** Determine the thickness of the top layer:

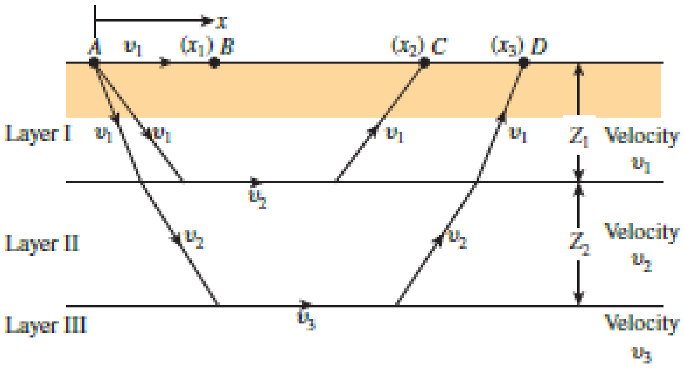
$$Z_1 = \frac{1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}\,x_c$$

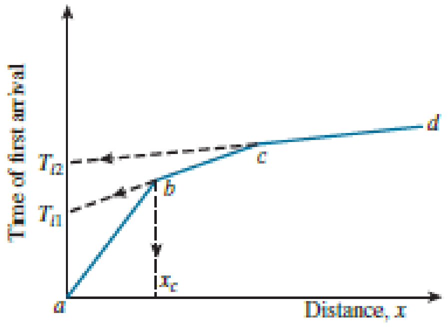
The value of $x_c$ can be obtained from the plot,

***Step 5.*** Determine the thickness of the second layer:

$$Z_2 = \frac{1}{2}\left[T_{i2} - 2Z_1\frac{\sqrt{v_3^2 - v_1^2}}{v_3 v_1}\right]\frac{v_3 v_2}{\sqrt{v_3^2 - v_2^2}}$$

Here, $T_{i2}$ is the time intercept of the line $cd$ in Figure extended backward.



# SEISMIC REFRACTION SURVEY

The velocities of *P* waves in various layers indicate the types of soil or rock that are present below the ground surface. The range of the *P*-wave velocity that is generally encountered in different types of soil and rock at shallow depths is given in the table:

**Range of *P*-Wave Velocity in Various Soil and Rocks**

| Type of soil or rock | *P*-wave velocity m/s |
|---|---|
| *Soil* | |
| Sand, dry silt, and fine-grained topsoil | 200–1000 |
| Alluvium | 500–2000 |
| Compacted clays, clayey gravel, and dense clayey sand | 1000–2500 |
| Loess | 250–750 |
| *Rock* | |
| Slate and shale | 2500–5000 |
| Sandstone | 1500–5000 |
| Granite | 4000–6000 |
| Sound limestone | 5000–10,000 |

# EXAMPLE 3.5

**EXAMPLE 3.5**

The results of a refraction survey at a site are given in the following table:

| Distance of geophone from the source of disturbance (m) | Time of first arrival ($s \times 10^3$) |
|---|---|
| 2.5 | 11.2 |
| 5 | 23.3 |
| 7.5 | 33.5 |
| 10 | 42.4 |
| 15 | 50.9 |
| 20 | 57.2 |
| 25 | 64.4 |
| 30 | 68.6 |
| 35 | 71.1 |
| 40 | 72.1 |
| 50 | 75.5 |

Determine the *P*-wave velocities and the thickness of the material encountered.

# EXAMPLE 3.5

**SOLUTION**

**Velocity**

In Figure the times of first arrival of the $P$ waves are plotted against the distance of the geophone from the source of disturbance. The plot has three straight-line segments. The velocity of the top three layers can now be calculated as:

$$\text{Slope of segment } 0a = \frac{1}{v_1} = \frac{\text{time}}{\text{distance}} = \frac{23 \times 10^{-3}}{5.25}$$

or

$$v_1 = \frac{5.25 \times 10^3}{23} = \mathbf{228\ m/s\ (top\ layer)}$$

$$\text{Slope of segment } ab = \frac{1}{v_2} = \frac{13.5 \times 10^{-3}}{11}$$

or

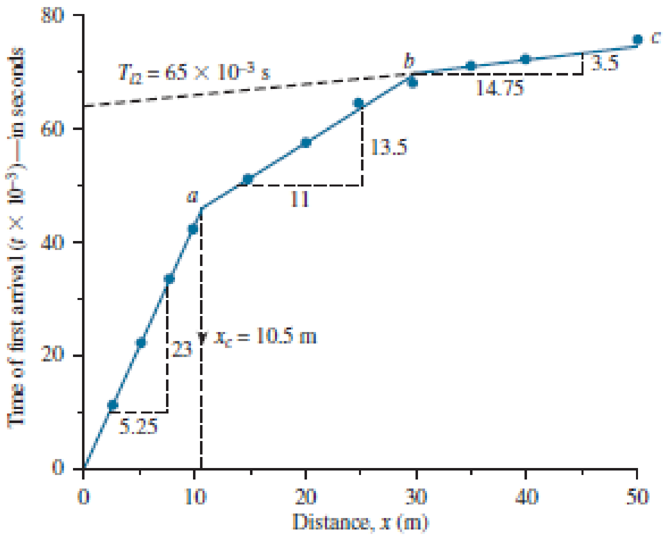
$$v_2 = \frac{11 \times 10^3}{13.5} = \mathbf{814.8\ m/s\ (middle\ layer)}$$

$$\text{Slope of segment } bc = \frac{1}{v_3} = \frac{3.5 \times 10^{-3}}{14.75}$$

or

$$v_3 = \mathbf{4214\ m/s\ (third\ layer)}$$

Comparing the velocities obtained here with those given in Table indicates that the third layer is a *rock layer.*




Plot of first arrival time of $P$ wave versus distance of geophone from source of disturbance

# EXAMPLE 3.5

**Thickness of Layers**

From Figure , $x_c = 10.5$ m, so

$$Z_1 = \frac{1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}x_c$$

Thus,

$$Z_1 = \frac{1}{2}\sqrt{\frac{814.8 - 228}{814.8 + 228}} \times 10.5 = \mathbf{3.94\ m}$$

Again, from Eq. (3.81),

$$Z_2 = \frac{1}{2}\left[T_{i2} - \frac{2Z_1\sqrt{v_3^2 - v_1^2}}{(v_3 v_1)}\right]\frac{(v_3)(v_2)}{\sqrt{v_3^2 - v_2^2}}$$

The value of $T_{i2}$ (from Figure 3.46) is $65 \times 10^{-3}$ s. Hence,

$$Z_2 = \frac{1}{2}\left[65 \times 10^{-3} - \frac{2(3.94)\sqrt{(4214)^2 - (228)^2}}{(4214)(228)}\right]\frac{(4214)(814.8)}{\sqrt{(4214)^2 - (814.8)^2}}$$

$$= \frac{1}{2}(0.065 - 0.0345)830.47 = \mathbf{12.66\ m}$$

Thus, the rock layer lies at a depth of $Z_1 + Z_2 = 3.94 + 12.66 =$ **16.60 m** from the surface of the ground.

# SEISMIC REFRACTION SURVEY

*Advantages :*

- It is fast and not hindered by the presence of boulders
- Equipment is lightweight and can be carried in the field.
- Two persons are enough

*Disadvantages :*

- It can not detect a subsurface layer whose sonic velocity is slower than that of the layer above (peat, soft clay,…)
- Wrong interpretation of the subsurface materials when the soil is saturated and the ground water table is not detected.

# CROSS-HOLE SEISMIC SURVEY

# CROSS-HOLE SEISMIC SURVEY

- To find the shear modulus of the soil
- Two holes are drilled into the ground, spacing L distance
- A vertical impulse is created at the bottom of one hole by means of an impulse rod. The shear waves (generated) are recorded by a vertically sensitive transducer.

The shear modulus $G_s$ of the soil at the depth at which the test is taken can be determined from the relation

$$v_s = \sqrt{\frac{G_s}{(\gamma/g)}}$$

or

$$G_s = \frac{v_s^2 \gamma}{g}$$

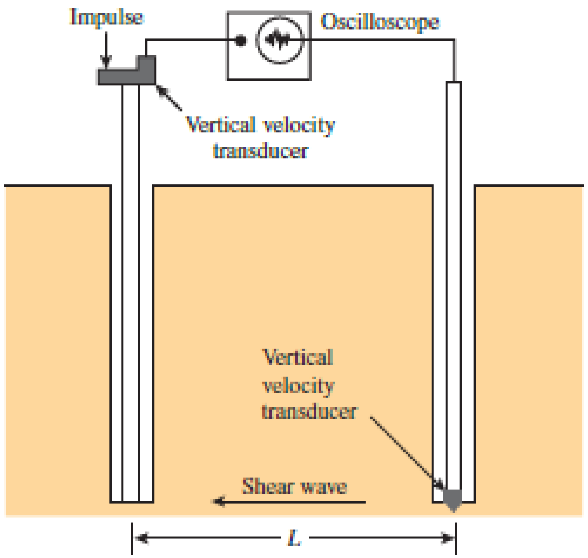
where

$v_s$ = velocity of shear waves

$\gamma$ = unit weight of soil

$g$ = acceleration due to gravity

The shear modulus is useful in the design of foundations to support vibrating machinery and the like.




Cross-hole method of seismic survey

# ELECTRICAL RESISTIVITY SURVEY

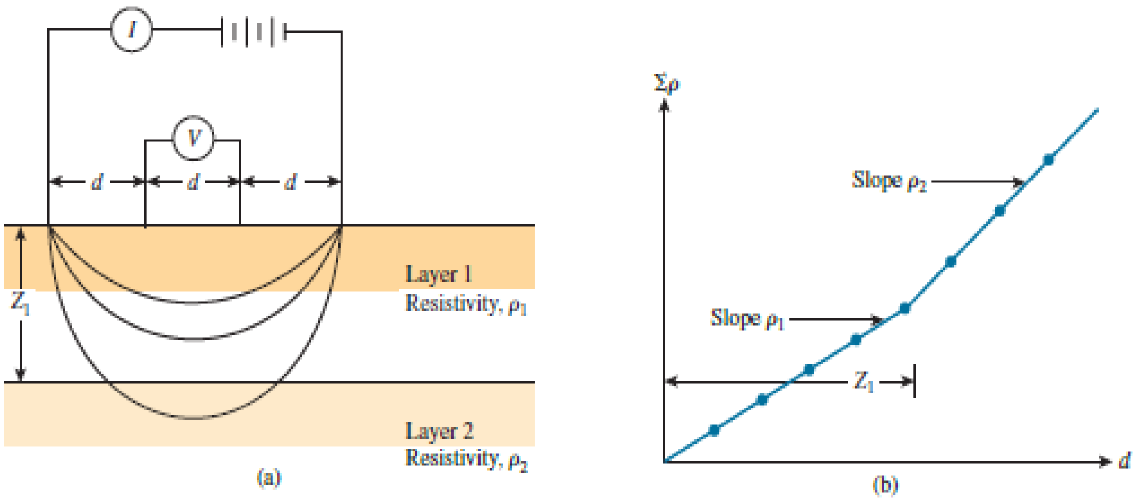
# ELECTRICAL RESISTIVITY SURVEY

- To obtain information about the stratification of the subsurface
- Different soils have different electrical resistivity
- Saturated soils $\longrightarrow$ very low resistivity
- Dry soils and rock $\longrightarrow$ high resistivity
- It consists of :
- Four electrodes are driven into the ground, spaced equally along a straight line (Wenner method).
- Two electrodes supply current to the ground, the other two detect the current between the exciting electrodes $\rho = \frac{2\pi dV}{I}$
- After each measurement, the spacing "d" can be expanded to penetrate greater depths.
- Plot $\Sigma\rho$ vs. d can be obtained, from which the thickness of various layers can be estimated.

# ELECTRICAL RESISTIVITY SURVEY

**Representative Values of Resistivity**

| Material | Resistivity (ohm · m) |
|---|---|
| Sand | 500–1500 |
| Clays, saturated silt | 0–100 |
| Clayey sand | 200–500 |
| Gravel | 1500–4000 |
| Weathered rock | 1500–2500 |
| Sound rock | >5000 |




Electrical resistivity survey:
(a) Wenner method;
(b) empirical method for determining resistivity and thickness of each layer

# ELECTRICAL RESISTIVITY SURVEY

***Advantages :***

- **It is fast and low cost**
- **It can detect underlying layer whose resistivity are either higher of lower than overlying layers**

***Disadvantages :***

- **Sensitive to variations in both soil conditions and electrode placement**
- **Can not distinguish between soft and stiff clays.**

# GEOTECHNICAL REPORT

# GEOTECHNICAL REPORT

- Upon completion of the geotechnical investigation and analysis, the information and findings must be compiled in a standard report format.
- The report serves as the permanent record of all geotechnical data known to be pertinent to the project and is referred to throughout the design, construction, and service life of the project.
- The data and recommendations are typically compiled in a Geotechnical Report. The intent of the Geotechnical Report is to present the data collected in a clear manner, to draw conclusions from the data, and to make recommendations for the geotechnical aspects of the project.
- The primary clients that use the report are roadway designers, Bridge Engineers, construction personnel, and contractors.

# SUBSOIL EXPLORATION REPORT

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.

# THE END