EWCE 343: Geotechnical & Foundation Engineering

L3/T2: 3.00 Credit Hours 3 Hours per Week

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Subsurface Investigation

Topics to be Covered

- Planning of Subsurface Investigation
- Methods of Exploration
- Various Types of Sample, Sampler and Sampling
- Penetration Tests
- Geophysical Methods of Exploration

1.0 What is Subsurface Investigation?

The proper design of civil engineering structures requires **adequate knowledge of the subsurface conditions at the site of the structure; that is, the physical properties and arrangements of underlying materials.**

The field and laboratory investigations required to obtain these essential information are called site investigation or subsurface exploration or soil exploration or soil investigation or exploratory programme.

1.0 What is Subsurface Investigation?

Subsoil investigation mainly depends on the type of structure. There are mainly three categories of civil engineering structures:

1) Structure for which the basic problem is the interaction of soil and structure e.g. foundations.
2) Structures constructed of soil e.g. embankment.
3) Structures of natural earth e.g. canal.

2.0 Information Required from a Subsurface Investigation

1) Nature of the deposit (Geology, recent history of filling etc.)
2) Depth, thickness and aerial extent of each soil stratum
3) Location of ground water table; artesian pressure
4) Engineering properties of soil e.g. shear strength, compressibility, permeability etc. that affects the structure

3.0 Use of Information from a Subsoil Investigation

Interpretation of data obtained through soil exploration generally provides information to decide on:

1) Type and the depth of foundation suitable for a given structure.
2) Evaluation of Safe load capacity of foundation
3) Estimation of the probable settlement of a structure.
4) Causes of settlements, tilts and cracks in the existing structures

3.0 Use of Information from a Subsoil Investigation

5) Determination of potential foundation problems (e.g, expansive soil, collapsible soil, sanitary landfill, and so on).
6) Establishment of ground water table.
7) Prediction of lateral earth pressure for structures like retaining walls, sheet pile bulkheads, and braced cuts.
8) Establishment of construction methods for changing subsoil conditions.
9) Identification and solution of excavation problems

4.0 Planning of Site Investigation

The planning of site investigation aims at obtaining the maximum information with minimum investment.

There are mainly two methods, namely **indirect and direct,** of obtaining the required information from a subsurface investigation.

Indirect method includes Phase-1 only, whereas, the **direct method** requires to cover **all the 4 phases** mentioned as under.

Phase 1: Obtaining Available Information (Desk Study)

Collection of geologic map, topographic map, aerial photograph, soil survey report, hydrological data etc.

4.0 Planning of Site Investigation

Phase II: Field Reconnaissance

Examination of the area by the engineer along with other specialist such as geologist, geotechnical engineer, land surveyor etc. The main jobs are:

- To study the existing structures for its foundation construction, defects, cracks.
- To identify exposed soil profile and rock outcrop.
- To locate high water marks on bridge abutment.
- To gather history of flood and scour level from local people.

4.0 Planning of Site Investigation

Phase III: Preliminary Investigation

This stage should undertake the following steps:

1) Thorough understanding of geology of site
2) More details of the subsoil strata like thickness of the individual strata by performing 1 or 2 bore holes.

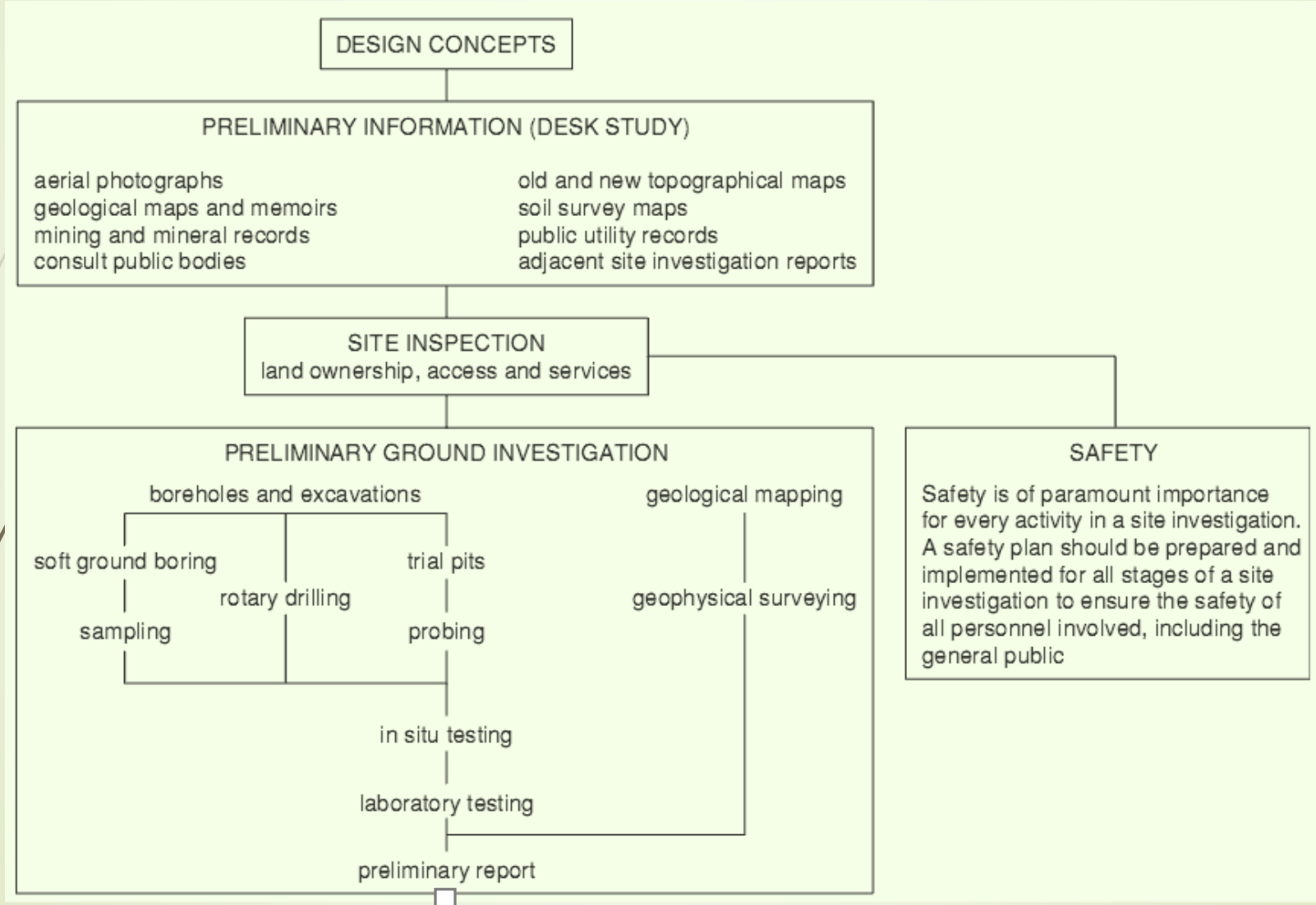
4.0 Planning of Site Investigation

Phase IV: Detailed Investigation

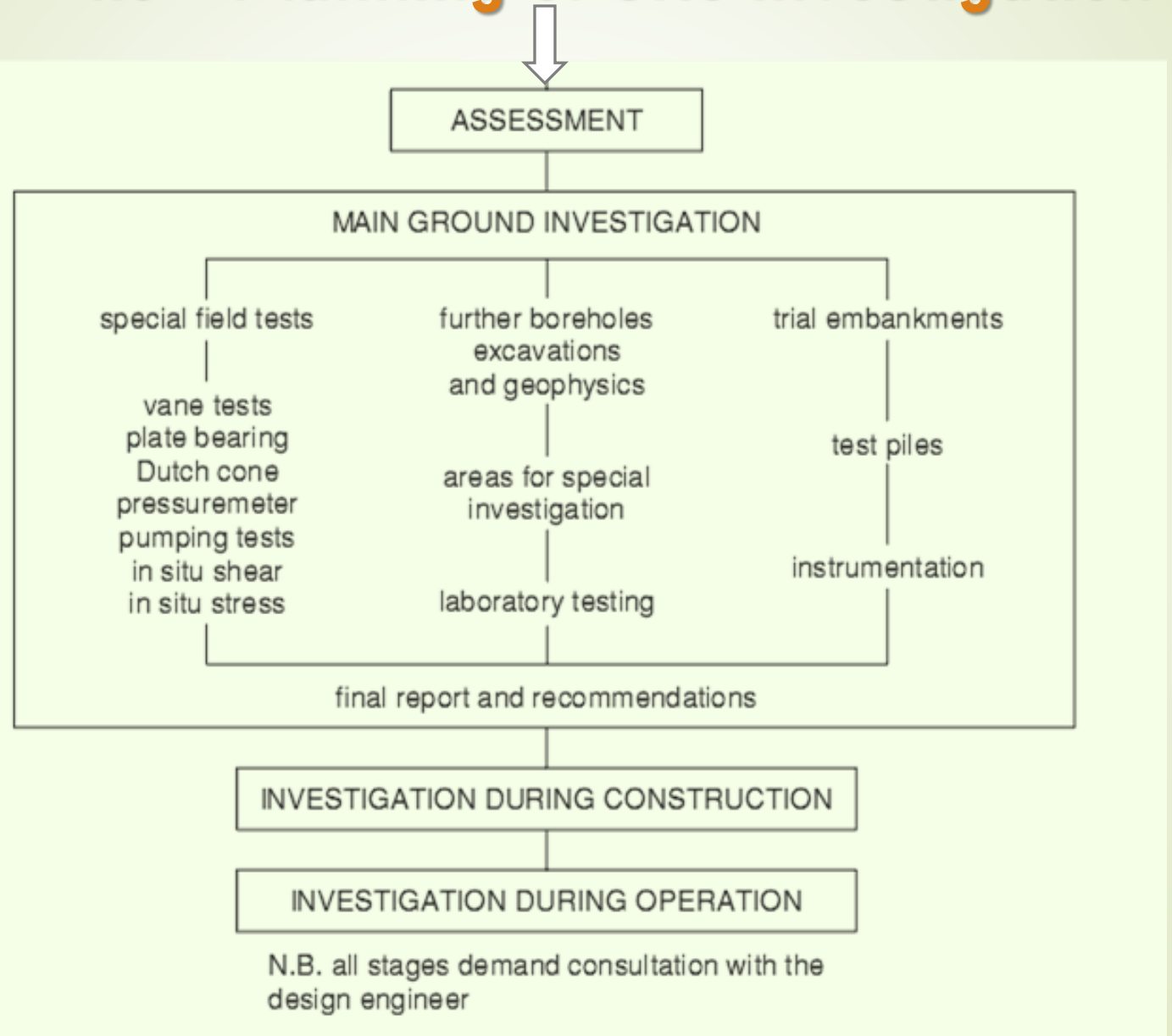
Mainly consists of:

1) Planning and execution of open pits and/ or additional boring, as dictated by the preliminary investigation.
2) Collection of soil samples for laboratory tests.
3) Field tests including sounding, probing and geophysical tests.

4.0 Planning of Site Investigation



4.0 Planning of Site Investigation



5.0 Considerations in a Subsoil Investigation

The following factors are to be considered in a site investigation:

a) Cost of Exploration
b) Number of Boring
c) Location of Boring
d) Spacing of Boring
e) Depth of Boring

5.1 Cost of Exploration

The cost of subsoil investigation depends on the extent and scope of the work to be done. However, as a general guide line the cost of the investigation should be estimated as

- 0.05 to 0.2 % of the total cost of the project or
- 0.5 to 1% of the cost of the structure.

5.2 Number and Location of Exploration Boring

Both the number and location of borings is governed by the degree of sensitivity of the structure to be built and the heterogeneity of the project site indicated by the preliminary investigation.

In exploration of areas of limited extent, the number of borings **should preferably be not less than 4 and should preferably be located near the corners of the area.**

5.2 Number and Location of Exploration Boring

For **small buildings** number of borings is limited to **3 to 5**.

For building sites covering an area of 1000 m^2, one borehole at each corner and one in center, that is **5 boreholes** in all, should be adequate.

At least three borings should be made on a project site where the surface is level, especially if the site is somewhat even.

5.2 Number and Location of Exploration Boring

For railway and embankment construction the boring should be done along the chainage line of alignment.

One hole for each **900 m^2** is often used for projects covering large areas. For such projects usually a grid system is used for the borehole locations. One borehole is drilled at each grid point.

Table 1 provides some of the information about the number of boring.

5.3 Spacing of Boring

The spacing of boring depends on the **subsoil condition of the site**. For uniform soil condition, spacing between the borings is more. Where as, for erratic or non-uniform soil conditions borings are usually closely spaced.

A guideline is provided for the spacing of the boring for different structures as presented in Table 1.

5.3 Spacing of Boring

Table 1: Spacing (metre) and Minimum Number of Boreholes for Various Structures				
Type of Structure ↓	Type of Soil in Horizontal Strata			Minimum Number of Boring
	Uniform	Average	Erratic	
One or two storied structures	50	25	10 - 15	3
Multistoried structures	30	15	8 - 10	5
Bridge piers and abutments	50	20	10	1 or 2 for each foundation
Highways and airports	300	150	50	---
Earth dam	60	30	15	---
Borrow pit	150 - 300	60 - 150	30 - 60	---

5.4 Depth of Boring

The basic criteria used in establishing the depths to which soil borings are extended is to ensure that the pressures imposed on the soil by the proposed construction at these depths are not of significant magnitude. Just where the pressures cease to be significant is a matter of opinion and experience.

Rules of thumb, such as "boring depths should be at least twice the width of the largest footing," are sometimes used.

Generally, one should extend borings through any unsuitable or questionable soil materials and sufficiently deep into firm soils that significant settlement will not result from compression of underlying soils.

5.4 Depth of Boring

A commonly used rule is that, unless bed rock is encountered, boring should be carried to such a depth that the net increase in soil stress caused by the proposed construction is less than 10% of the maximum value.

A second technique employs the concept of drilling to depths where the net increase in soil stress caused by the proposed construction is limited to less than 5% of the effective stress caused by the weight of the soil. These concepts are illustrated graphically in Fig. 1.

5.4 Depth of Boring

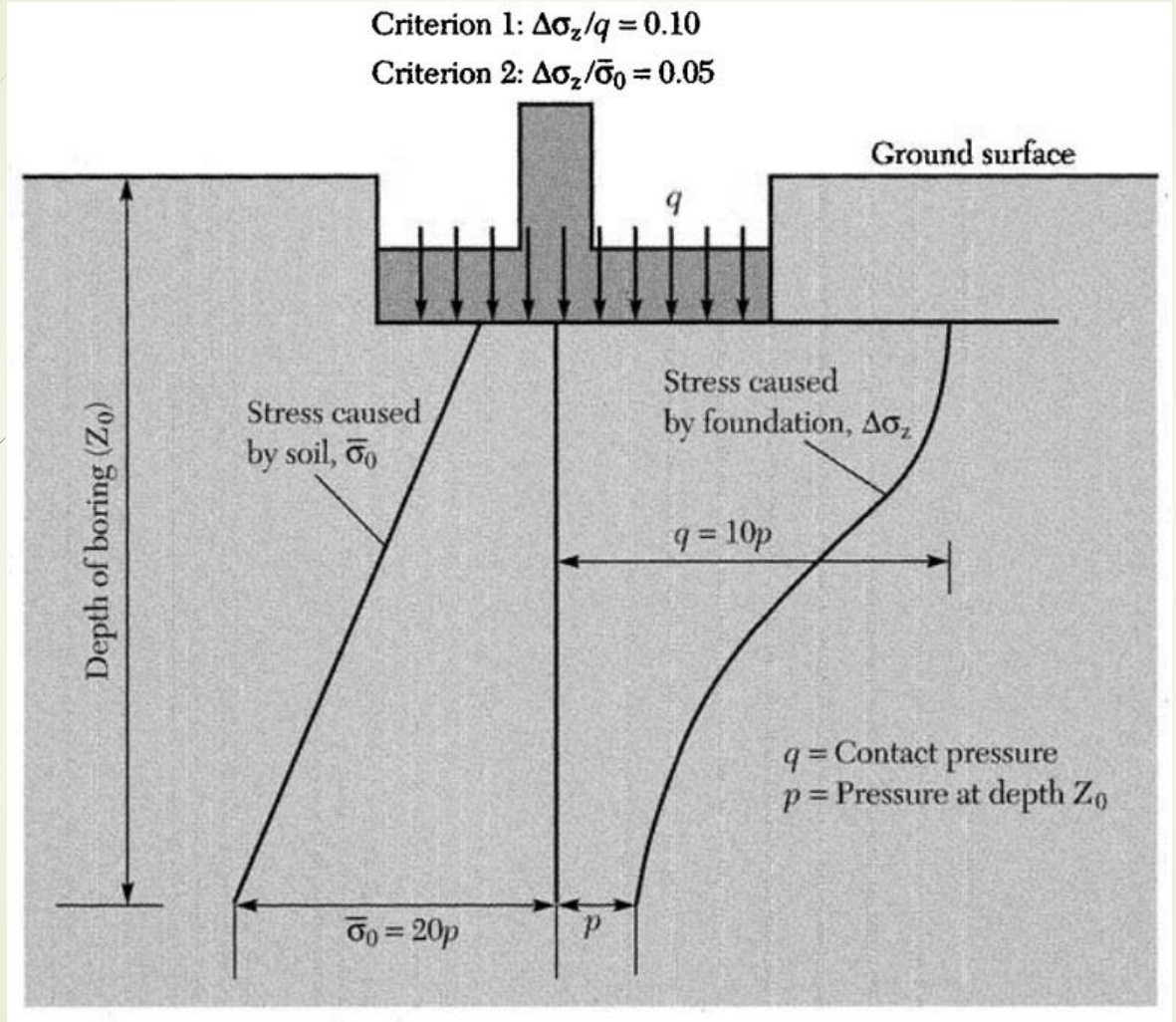
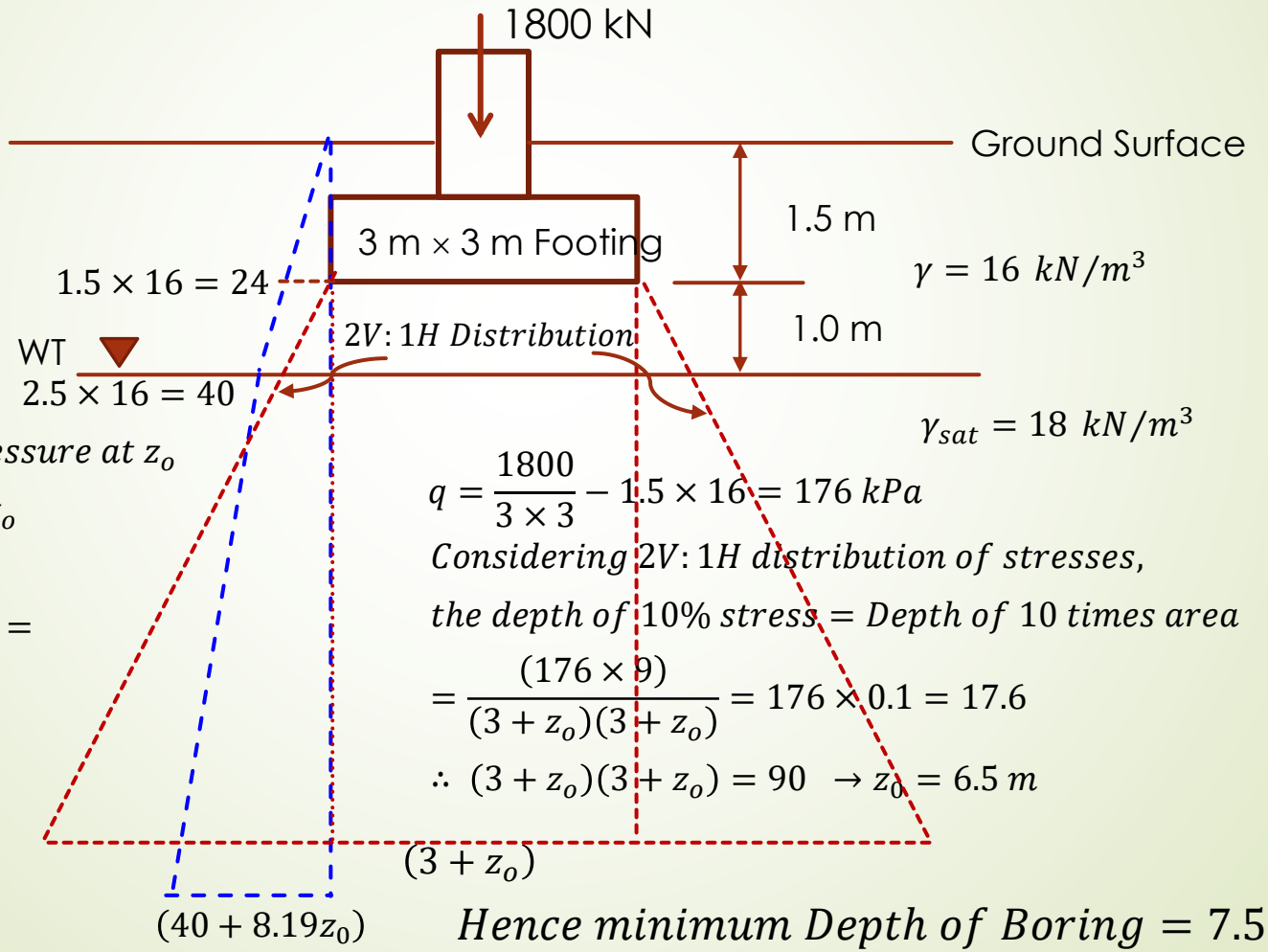


Fig. 1: Illustration of procedures used to approximate boring depths.

5.4 Depth of Boring

Example: As illustrated.



Solution:

$Effective\ vertical\ pressure\ at\ z_o$

$= 40 + (18 - 9.81) \times z_o$

$= 40 + 8.19z_0$

$\therefore 0.05 \times (40 + 8.19z_o) = \dfrac{(176 \times 9)}{(3 + z_o)(3 + z_o)} \rightarrow$

$z_o = \dfrac{1368}{(3 + z_o)^2} - 4.88$

$\rightarrow\ z_o = 7.5\ m$

$q = \dfrac{1800}{3 \times 3} - 1.5 \times 16 = 176\ kPa$

$Considering\ 2V:1H\ distribution\ of\ stresses,$

$the\ depth\ of\ 10\%\ stress = Depth\ of\ 10\ times\ area$

$= \dfrac{(176 \times 9)}{(3 + z_o)(3 + z_o)} = 176 \times 0.1 = 17.6$

$\therefore\ (3 + z_o)(3 + z_o) = 90\ \rightarrow z_o = 6.5\ m$

$Hence\ minimum\ Depth\ of\ Boring = 7.5\ m$

5.4 Depth of Boring

Depending on this, recommended depths of borings for building are about 3.5 m and 6.5 m for single and two storied building respectively.

Tables 2a and 2b, and Fig. 2 present some guidelines for depth of boring. Tomlinson has illustrated the probable depth of investigation as shown in Fig. 3.

5.4 Depth of Boring

Table 2a. Guidelines for Depth of Boring	
Type of Foundation	**Depth of Exploration**
Isolated spread footing or raft	1.5 times the width of the footing
Adjacent footings with clear spacing less than twice the width	1.5 times the length of the footing
Adjacent rows of footing	Refer to Fig. 2
Pile and well foundation	To a depth of $1\frac{1}{2}$ times the width of the structure from the bearing level (toe of pile or bottom of well).
Road cuts	Equal to the bottom width of the cut
Fill	2 m below ground level or equal to height of the fill whichever is greater.

5.4 Depth of Boring

Table 2b. Guidelines for Depth of Exploration for Buildings							
Width of Building (m) ↓	Number of Storeys						
	1	2	3	4	6	8	16
	Boring Depth (m)						
30	3.0	6.0	8.0	10.0	13.0	16.0	24.0
60	3.5	6.5	9.0	12.0	16.5	21.0	32.5
120	3.5	7.0	10.0	13.5	19.0	24.0	41.0

5.4 Depth of Boring

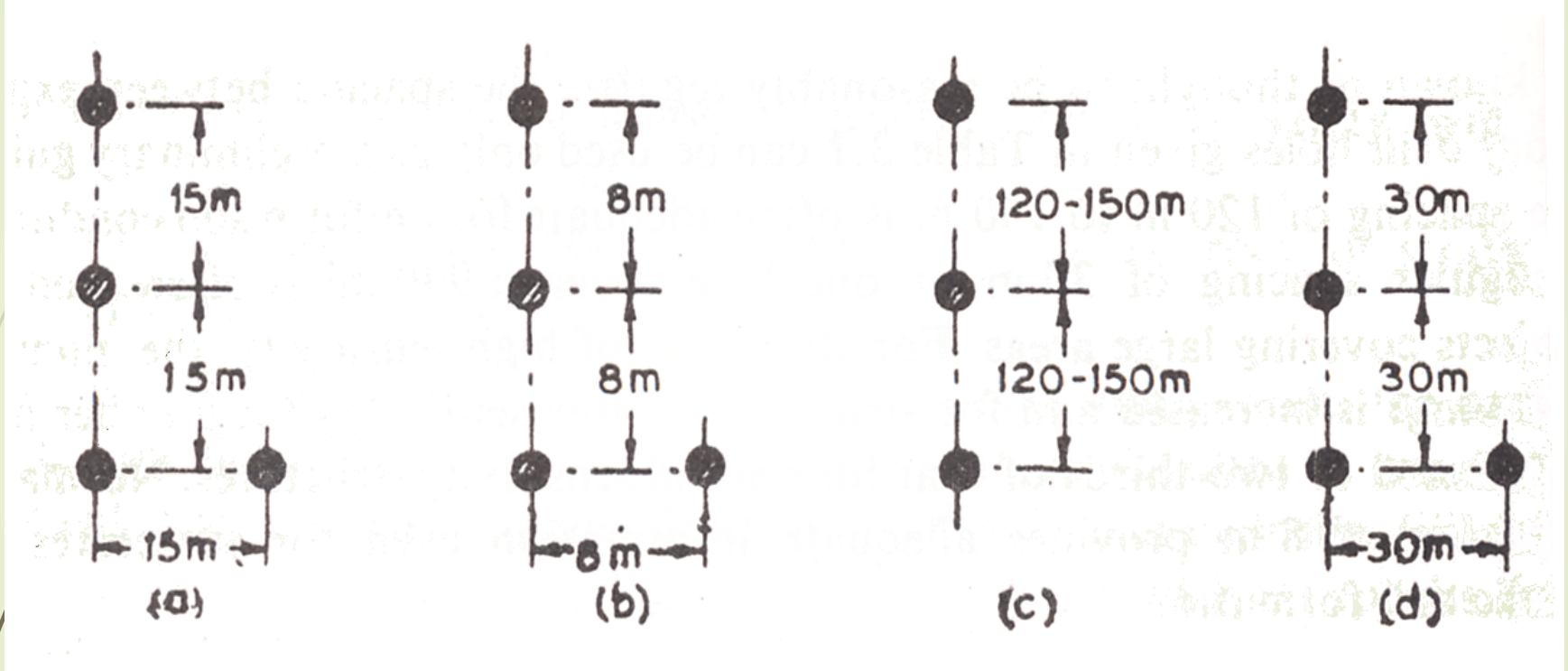


Fig. 2: Depth of Exploration; (a) Normal building on average soil; (b) Structures on erratic soils; (c) Structures on uniform soil and project of long length; (d) Project covering large area on uniform soil.

5.4 Depth of Boring

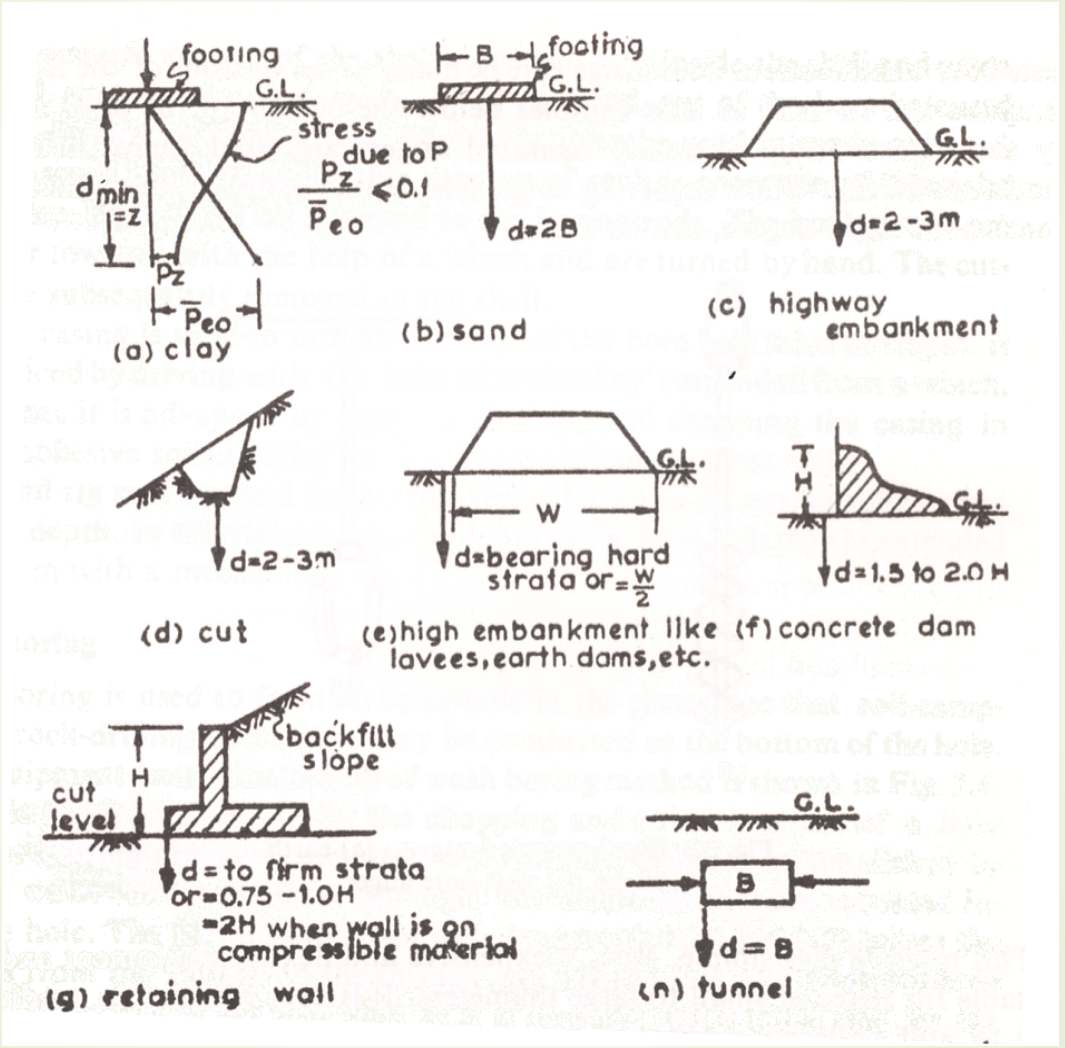


Fig. 3: Depth of boreholes for various foundations.

5.4 Depth of Boring

Sowers and Sowers (1981) proposed empirical formula to determine the depth of boring (unless rock is encountered):

For narrow concrete and light steel building:

$$Z_b = 3S^{0.7} \qquad (1a)$$

For wide concrete and heavy steel building:

$$Z_b = 6S^{0.7} \qquad (1b)$$

Where, Z_b is the depth of boring in metre and S is the number of equivalent storey. Roughly, a load of 10 kN/m^2 may be considered as a one equivalent storey.

5.4 Depth of Boring

If bed rock is expected to provide support to the load at least **one borehole must be drilled 3 m into bed rock** to establish whether bedrock does exist or merely a boulder has been encountered.

Prieto-Portat (2009) provides charts as under to determine the number and depth of boring for the construction of building (on shallow foundation) that may be considered very handy for the geotechnical engineer.

5.4 Depth of Boring

Subsurface Condition	Structural Footprint Area for Each Boring	
	(m)	(ft)
Poor and/or Erratic	200	2000
Average	300	3000
High Quality and Uniform	600	6000
Subsurface Condition	**Minimum Depth of Boring**	
	(m)	**(ft)**
Poor and/or Erratic	$6S^{0.7} + D$	$20S^{0.7} + D$
Average	$5S^{0.7} + D$	$15S^{0.7} + D$
High Quality and Uniform	$3S^{0.7} + D$	$10S^{0.7} + D$
S = No. of stories; D = Anticipated depth of foundation.		

6.0 Method of Boring

Sinking and advancing borehole into the ground with a view to obtaining soil or rock samples from specified depths is termed as boring. Borings are generally used for investigating the soil strata:

- For a depth greater than 3.0 metre.
- When difficult ground water conditions are met.

6.0 Method of Boring

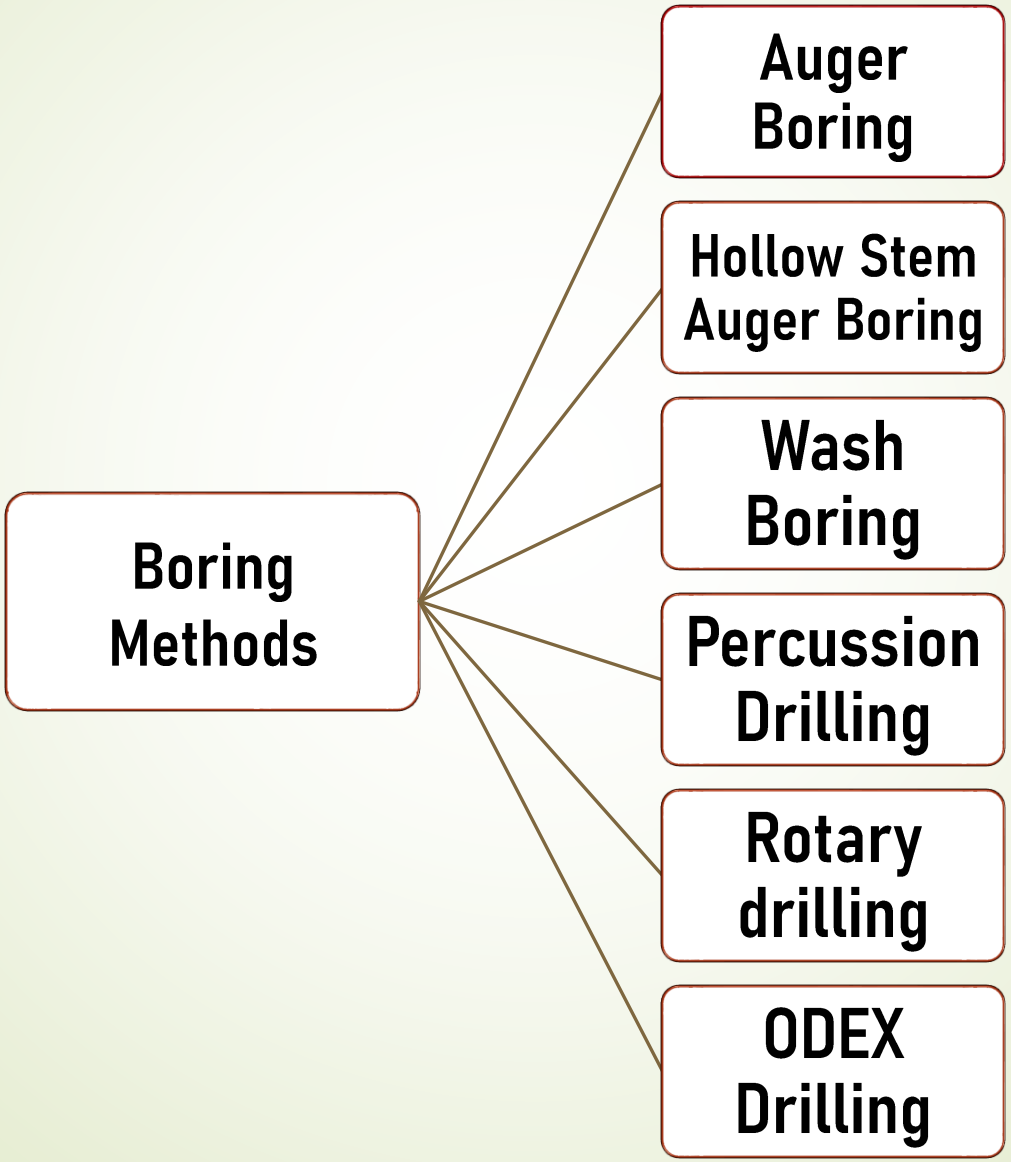
The diameter of the borehole usually varies from 50 mm to 250 mm. The borehole diameter depends on:

- Type of investigation
- Size of required soil samples
- Type of available equipment

Due to soil and ground water conditions the sides of the borehole may not be left unsupported. The sides are prevented from collapsing by:

- Casing pipe, and
- Filling the hole with a drilling mud, a suspension of bentonite in water.

6.0 Methods of Boring



6.1 Suitability of Boring Methods

The suitability of a particular boring method depends mainly on the nature of the soils and the position of the water table. Table 3 outlines the suitability of the various methods of boring.

6.1 Suitability of Boring Methods

Table 3a. Suitability of Boring Methods	
(1) Auger Boring	
Item	**Usage**
(a) Suitable soil type:	Partially saturated sands, silts & medium to stiff cohesive soils.
(b) Observation for change in materials:	Soil shavings removed with the auger.
(c) Sample obtained:	Representative sample, good for identification and classification tests.
(d) Utility in soil exploration:	Reconnaissance or detailed exploration; depths up to 10 m.

6.1 Suitability of Boring Methods

Table 3b. Suitability of Boring Methods	
(2) Hollow Stem Auger Boring	
Item	**Usage**
(a) Suitable soil type:	Partially saturated sands, silts & medium to stiff cohesive soils.
(b) Observation for change in materials:	Soil shavings removed with the auger.
(c) Sample obtained:	Representative sample, good for identification & classification tests..
(d) Utility in soil exploration:	Reconnaissance or detailed exploration; depths up to 60 m.

6.1 Suitability of Boring Methods

Table 3c. Suitability of Boring Methods	
(3) Wash Boring	
Item	**Usage**
(a) Suitable soil type:	All types of soil except hard, cemented soils and rocks.
(b) Observation for change in materials:	Cutting settled from wash water or drilling fluid.
(c) Sample obtained:	Non-representative sample; used only for strata identification.
(d) Utility in soil exploration:	Reconnaissance and special exploration & water table.

6.1 Suitability of Boring Methods

Table 3d. Suitability of Boring Methods	
(4) Percussion Drilling	
Item	**Usage**
(a) Suitable soil type:	All types of soils and rocks. Difficult in soft clay and loose sand.
(b) Observation for change in materials:	---
(c) Sample obtained:	Non representative sample; inadequate for any useful tests.
(d) Utility in soil exploration:	Penetrating rocks and boulders; useful with auger and wash boring.

6.1 Suitability of Boring Methods

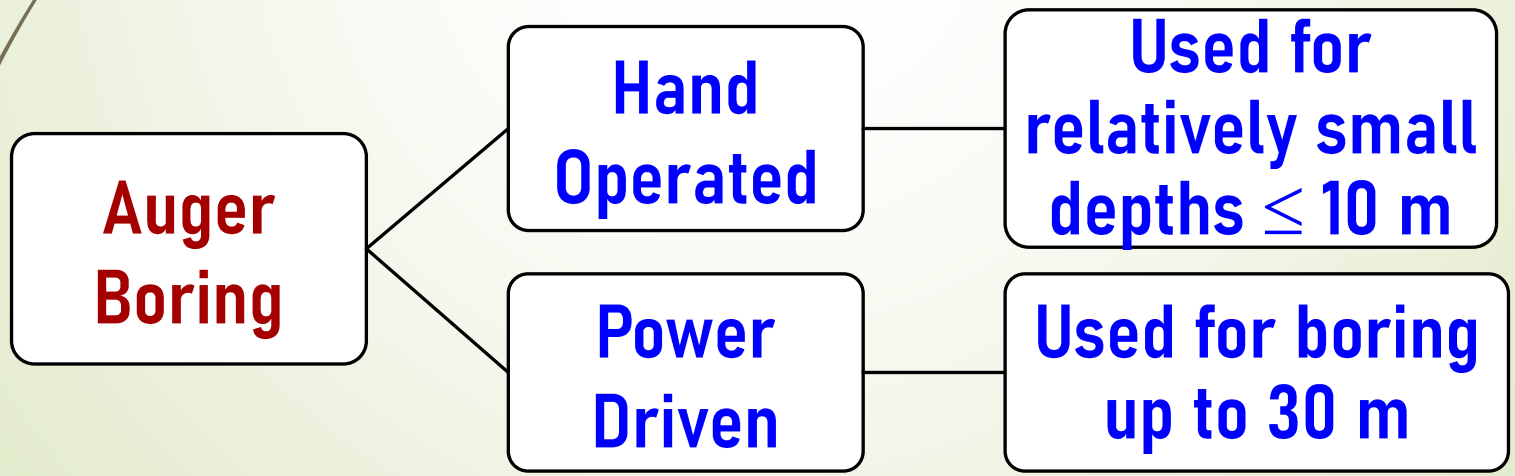
Table 3e. Suitability of Boring Methods	
(5) Rotary Drilling	
Item	**Usage**
(a) Suitable soil type:	Soils & rock except gravelly, porous soils & fissured rock.
(b) Observation for change in materials:	---
(c) Sample obtained:	Cores are obtained & used for classification & other tests.
(d) Utility in soil exploration:	Detailed and special exploration in rock.

6.1 Suitability of Boring Methods

Table 3f. Suitability of Boring Methods	
(6) ODEX Drilling	
Item	**Usage**
(a) Suitable soil type:	Soils with cobble and boulder.
(b) Observation for change in materials:	---
(c) Sample obtained:	Cores are obtained; suitable for all tests.
(d) Utility in soil exploration:	Suitable for soils with boulders.

6.2 Auger Boring

An auger is a tool used for drilling a borehole into the ground. Auger boring is generally used in soils which can stay open without casing. Clays, silts and partially saturated sands can stand unsupported. Auger are of two types used for various depths: Hand operated and Power driven.



6.2 Auger Boring

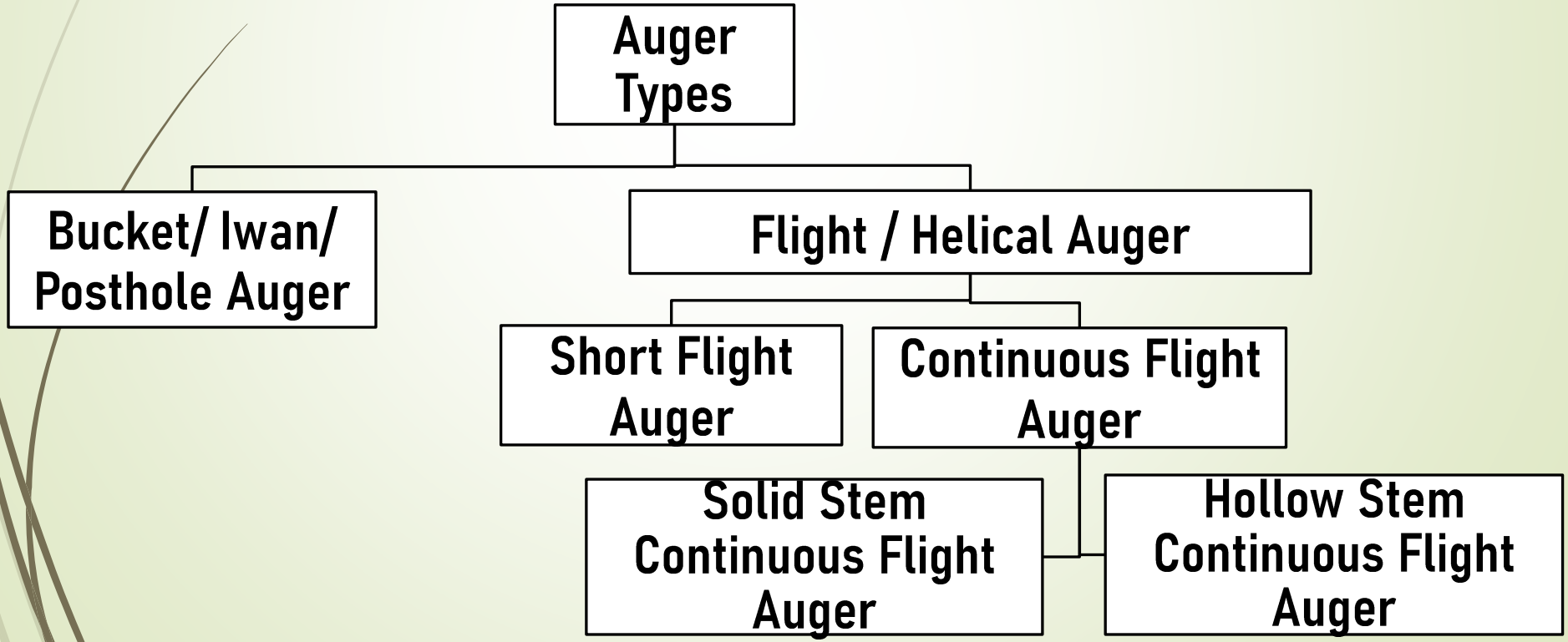
Hand operated auger is rotated and pressed into the soil by means of a T-handle fixed at the top of the upper rod of the auger.

Hand augers can be used to obtain soil samples to a depth of **approximately 10** m depending upon the soil conditions. Downward pressure and a torque are applied to the hand auger.

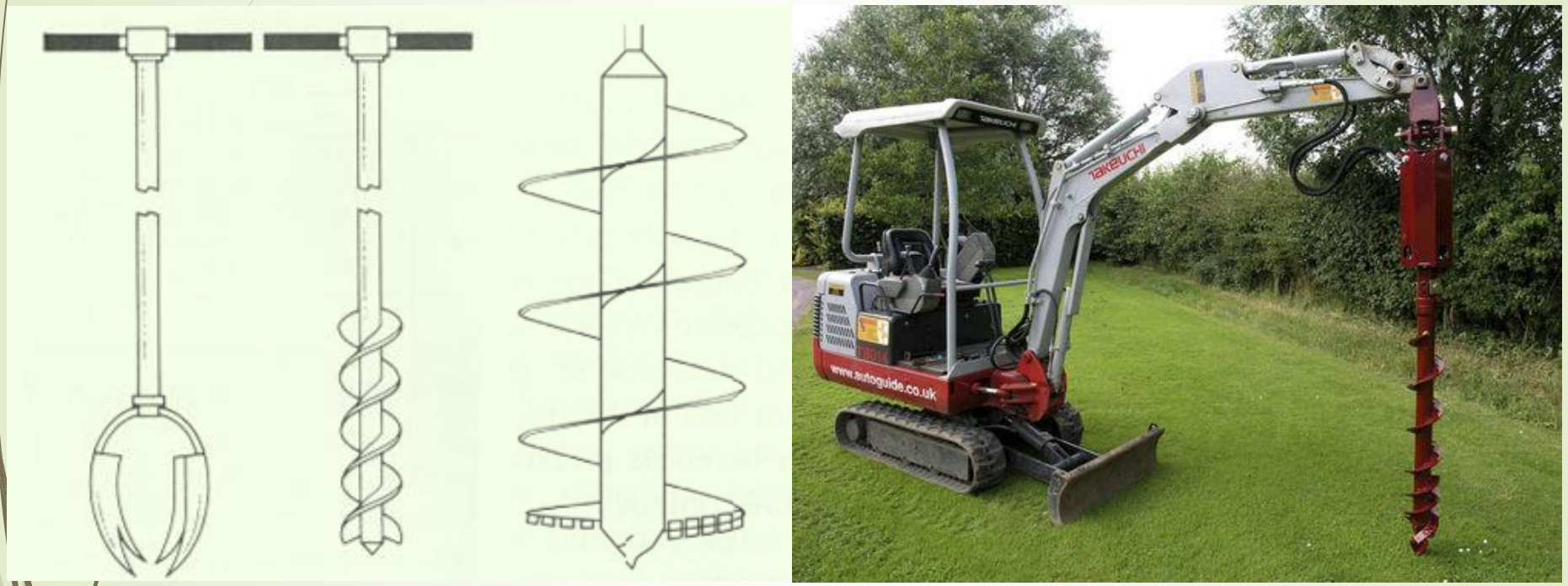
Due to the torque and the downward pressure, the hand auger penetrates into the ground. The process stops when human strength is not sufficient to generate enough torque or pressure.

6.2 Auger Boring

As soon as the auger gets filled with soil , it is taken out , cleaned and soil sample collected. The clean auger is again inserted and the process repeated. **Power driven auger operated by electrical power and is used for depths more than 10 m and up to 30 m.**



6.2 Auger Boring



(a) (b) (c) (d)

Fig. 4 (a): Posthole auger; (b) Helical auger; (c) Short flight auger; (d) Solid stem continuous fight auger

6.3 Hollow Stem Auger Boring

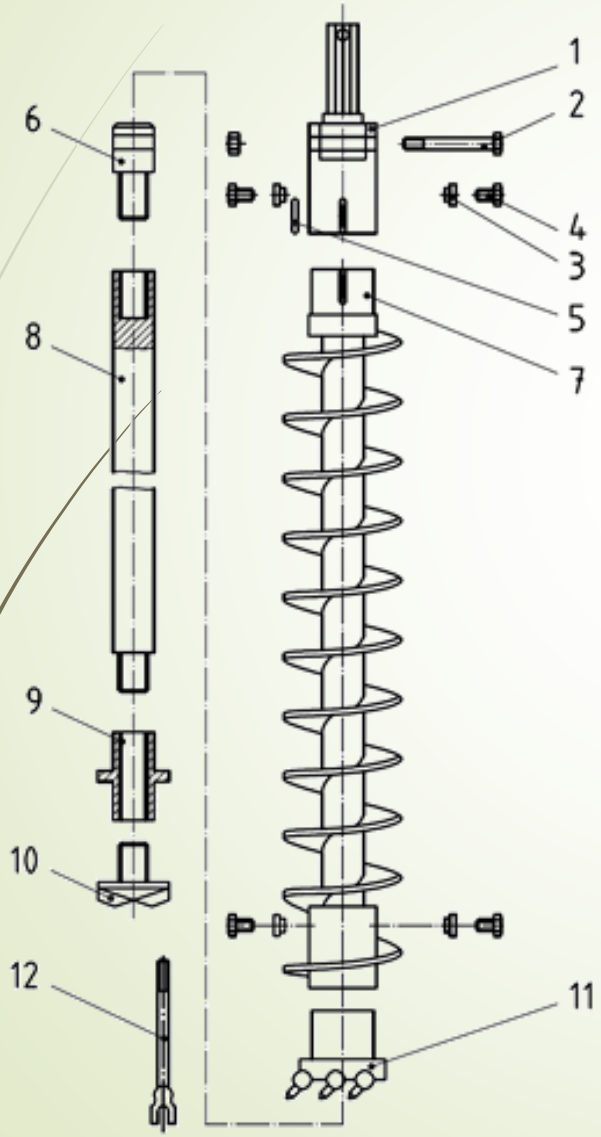
Holes may be drilled rapidly to depths of over 50 m using continuous flights of auger with hollow stem and a drilling rig mounted on a tractor or a truck. The hollow stem facilitates preventing the caving of the hole, and sampling tools may be operated through it. The auger is used to advance and case the hole simultaneously.

6.3 Hollow Stem Auger Boring

A removable plug attached to a center rod is used to block to entry of soil into the stem until the desired sample depth is reached. The plug and the centre rod are then retracted and the sampler with the sampler rod is inserted.

Schematic diagram and picture of hollow stem auger are shown in Figs. 5 and 6 respectively. Whenever the casing is to be extended, the auger has to be withdrawn. This sometimes hampers the quick progress of work.

6.3 Hollow Stem Auger Boring



1 Drive cap
2 Nut and bolt for rod to cap adaptor
3 Bushing nut
4 Lock nut
5 Drive key
6 Rod-to-cap adaptor
7 Hollow stem auger
8 Drill rod
9 Pilot bit connector
10 Pilot bit
11 Cutter head
12 Knock-out wrench

Fig. 5: Schematic diagram of hollow stem auger boring

6.3 Hollow Stem Auger Boring



Fig. 6: Picture showing hollow stem auger boring.

6.4 Wash Boring

Wash boring is the most commonly used method of making hole into the ground for soil investigation purposes. Except for the very stiff soil and rock, it can be conveniently used in any type of soil even below water table. Initially, a hole is advanced a short depth by an auger. A casing pipe is pushed or driven to prevent the caving of the hole. The system is then ready for wash boring.

In wash boring a hollow drill bit is screwed to a hollow drill rod connected to a rope passing over a pulley and supported by a tripod. A schematic diagram of the arrangement is shown in Fig. 7.

6.4 Wash Boring

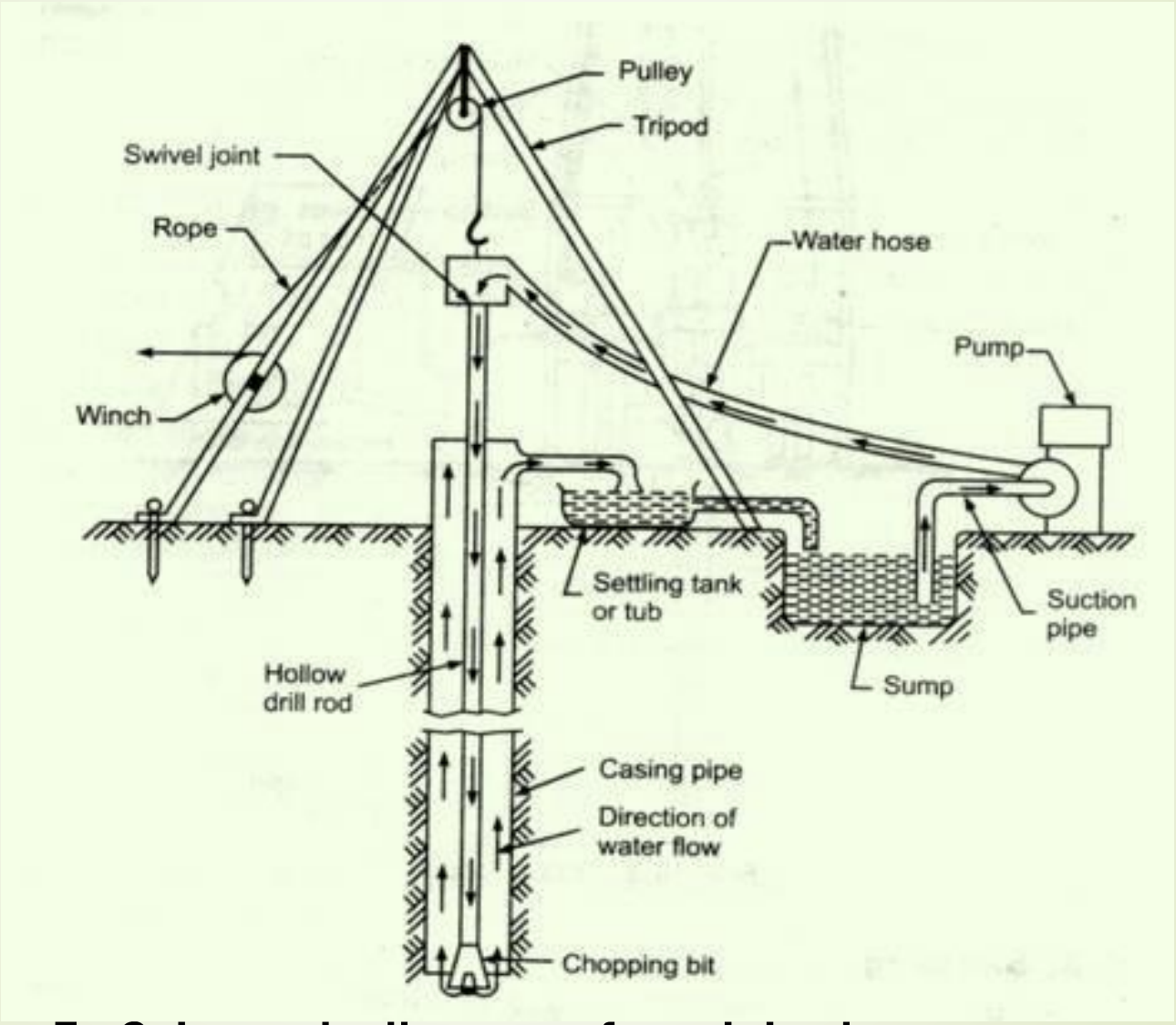


Fig. 7: Schematic diagram of wash boring setup.

6.4 Wash Boring

The basic principle of this method is that the hole is advanced by a sequence of chopping and jetting the soil using the drill bit. This breaks the soil into fragments and washes them out of the hole through the annular space between the drill rod and wall of the hole. The soil suspension is led to a tank where the soil particles settle while the water overflows into a sump. This water is used for circulation to the drill rod. The soil particles collected represents a very disturbed sample and is only useful to identify the change of the layer.

Wash boring is primarily used for advancing the borehole; whenever a soil sample is required, the chopping bit is replaced by a sampler.

6.5 Percussion or Cable Tool Drilling

In case of exceptionally hard strata of soil or rock percussion or cable tool drilling is used. A heavy drilling tool (bit) is alternately raised and dropped in such a manner that it grinds the underlying material. A small amount of water is added to facilitate the breaking of stiff materials that eventually forms slurry.

When the accumulation of slurry interferes with drilling, the drilling tools are removed and the hole is cleaned out. A percussion drilling setup is shown in Fig. 8.

6.5 Percussion or Cable Tool Drilling

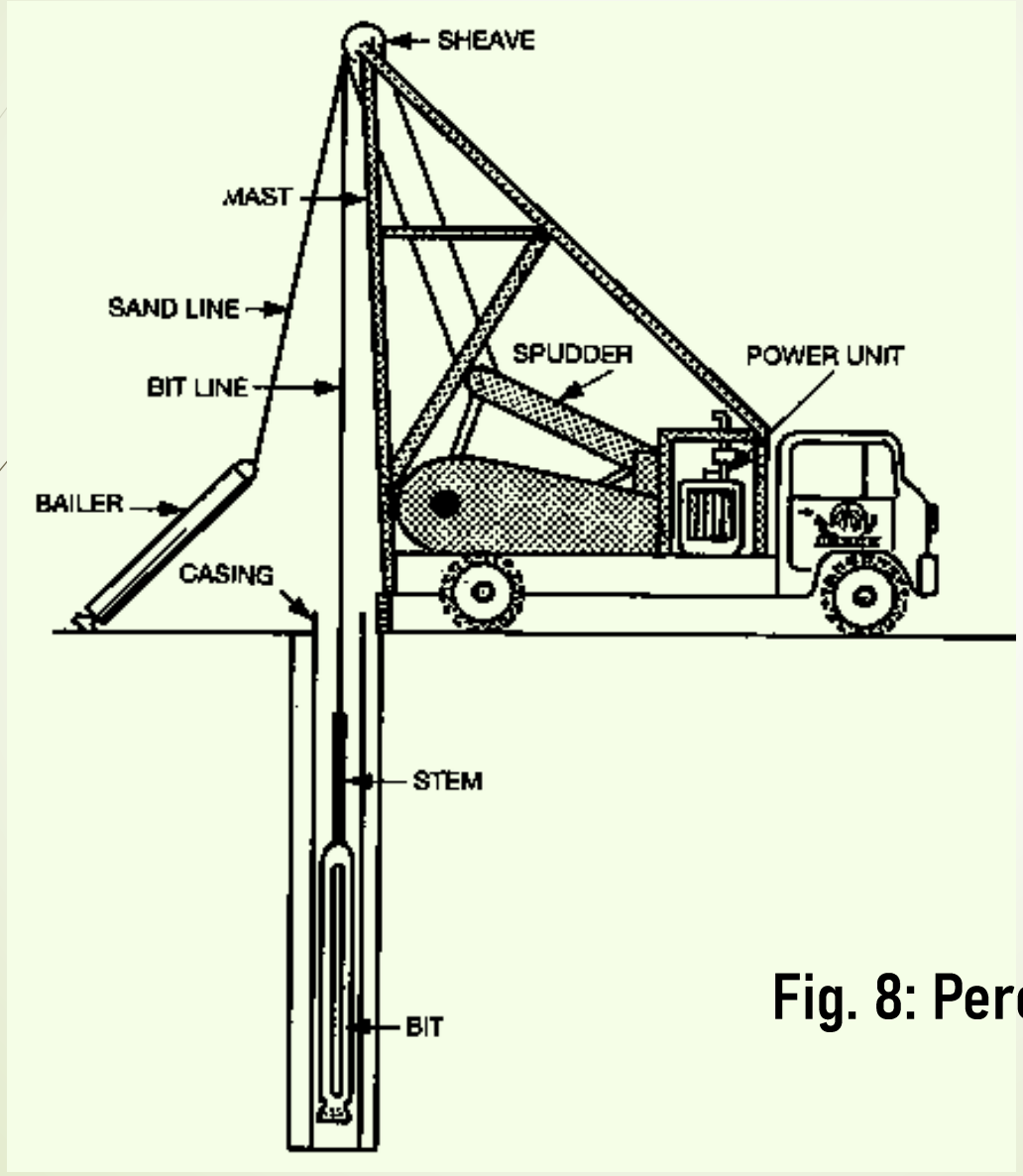


Fig. 8: Percussion drilling.

6.6 Rotary Drilling

Rotary drilling may be used in rock, in clay or even in sand. It is the most rapid method for penetrating highly resistant materials unless the deposit is very loose or badly fissured. In this method a rapidly rotating drilling bit cuts or grinds the materials at the bottom of the hole into small fragments.

The fragments are then removed circulating water or drilling fluid in a manner similar to that of wash boring. To obtain a sample, the drilling bit is removed and replaced by a sampler. A rotary drilling set up is shown in Fig. 9.

6.6 Rotary Drilling

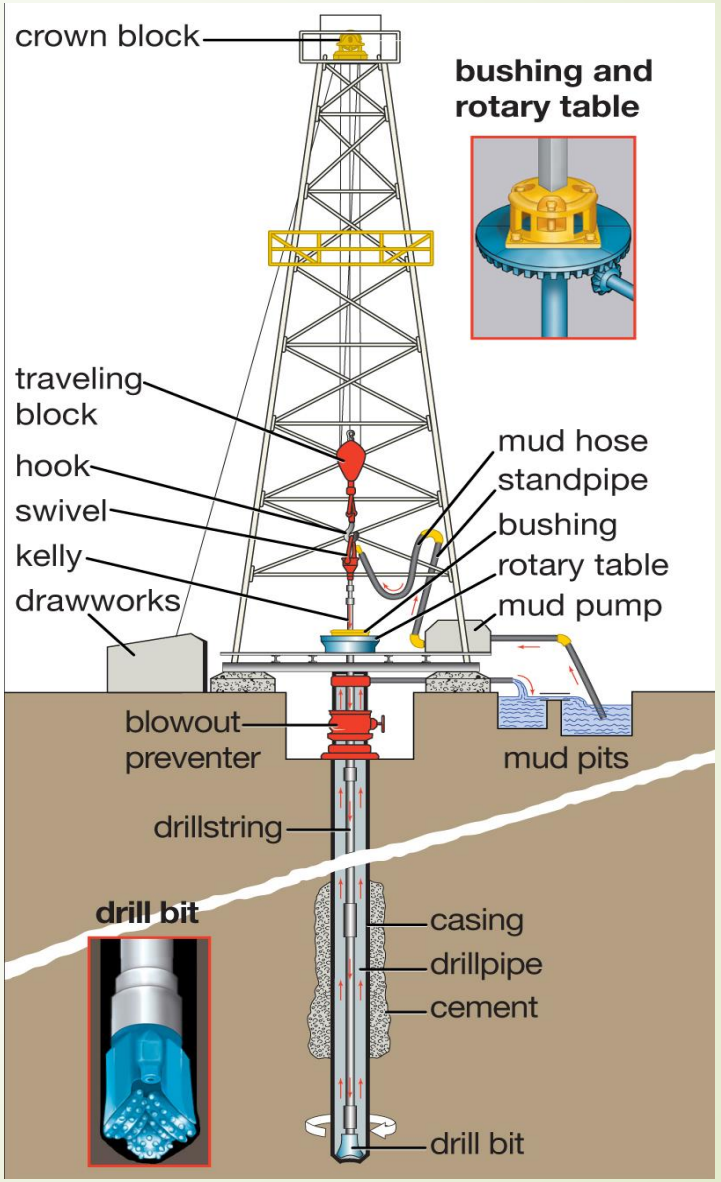


Fig. 9: Rotary drilling rig.

6.7 ODEX Drilling

This drilling method was developed for drilling through soils with cobbles and boulders. The name ODEX is an acronym of Overburden Drilling with an Eccentric bit.

The bit consists of a central lower portion that chops and rotates and an upper portion, which during drilling, moves outward to enlarge the borehole so that the casing can readily follow the bit. The casing shoe has a shoulder that is engaged by the bit unit. The casing is thus pulled down by a bit as it advances.

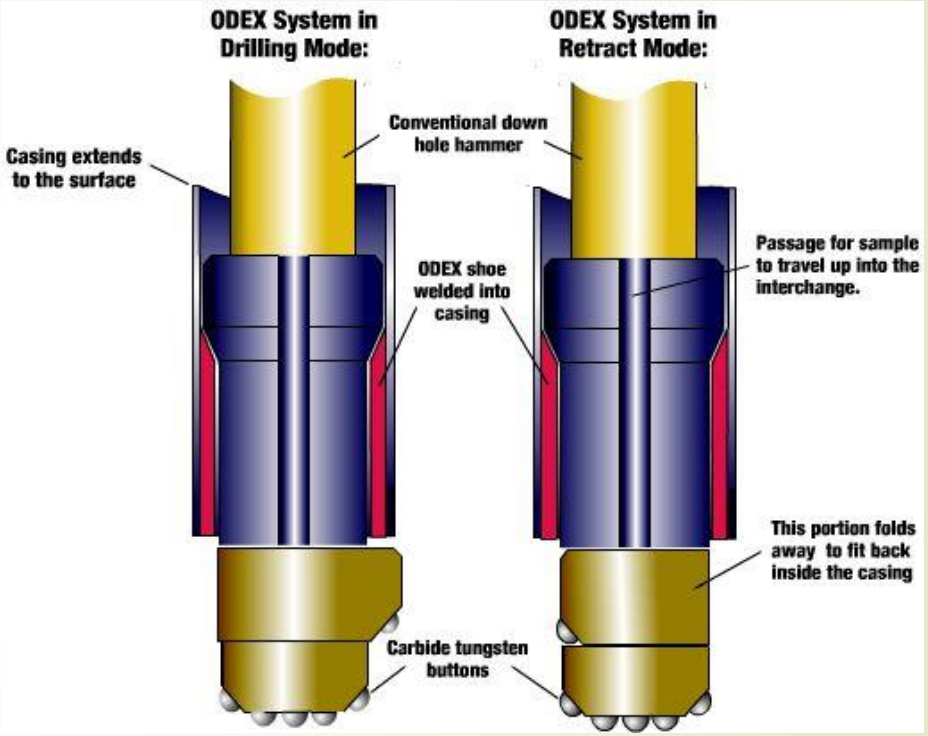


Fig. 10: ODEX drilling bit.

7.1 Soil Sample

The soil that is collected from borehole, for testing, is known as soil sample. The method of obtaining soil sample is termed as soil sampling technique or simply sampling. The tool by which soil samples are collected is the soil sampler. Depending on the presence of soil constituents in the sampled soil as compared to the insitu, soil sample may be classified as:

- Representative Sample
- Non-representative Sample

7.1 Soil Sample

A representative sample is one in which there is no change in soil constituents but whose structure, water content and void ratio may be altered.

A non-representative sample is one in which soils from different strata have been mixed or some of the soil constituents of the sampled soil are missing.

Depending on the structure and arrangement of the soil sampled as compared to the insitu, soil samples may be classified as:

- Undisturbed Sample
- Disturbed Sample

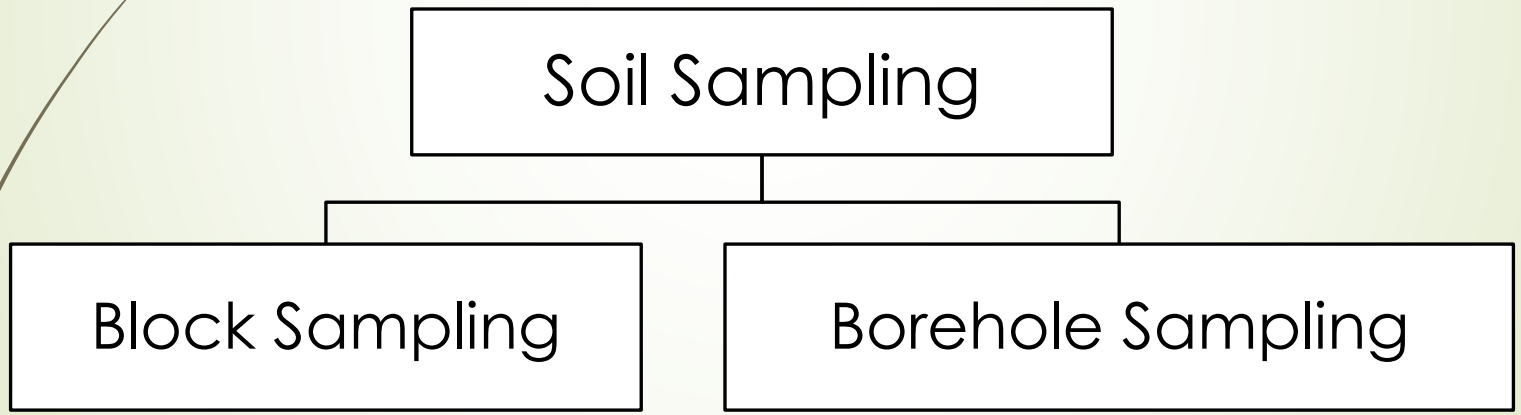
7.1 Soil Sample

A sample is said to be disturbed if all the constituents of the sampled soil are present in the sample in the similar arrangement and structure. That is the soil sample is the replica of the insitu soil.

However, true undisturbed sample is difficult to be obtained, as there would be a stress relief due to removal of overburden, during drilling of borehole. For all practical purposes this point of stress relief is disregarded in defining the undisturbed sample. A soil sample is considered as disturbed while any sort of disturbance occurs in it.

7.2 Soil Sampling

The process of collecting soil samples is known as soil sampling. There are mainly two methods of soil sampling.



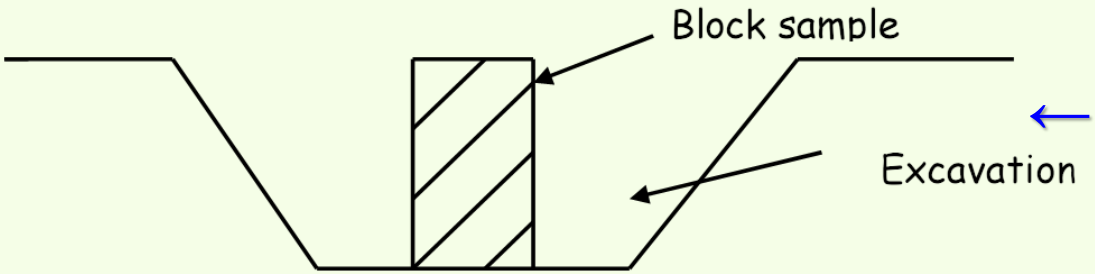
7.2 Soil Sampling

Hand cut block sample, collected by using this technique, is considered to be the best quality of soil samples. In this method soil surrounding the sample to be collected is removed by excavation and the sample is collected in a box. However, block sampling technique is restricted because of economy and other practical reasons. A schematic diagram of block sampling technique is sown in Fig. 11.

7.2 Soil Sampling

Bore hole sampling: This is the most common method of obtaining soil samples from a depth. A hole is formed in the ground removing the soil to the desired depth using any techniques. A sampler is screwed to the drill rod. It is taken down to the bottom of the borehole. The sampler is then either pushed or driven into the soil. When sufficient sample is entered into the sampler, it is rotated to shear at the base. A schematic diagram of borehole sampling is shown in Fig. 12.

7.2 Soil Sampling



← Fig. 11: Block sampling

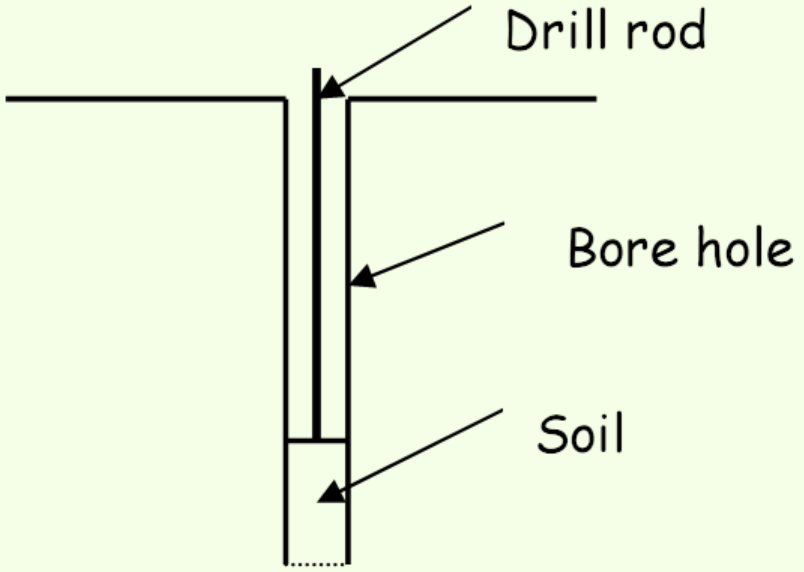


Fig. 12: Borehole sampling →

7.2 Soil Sampling

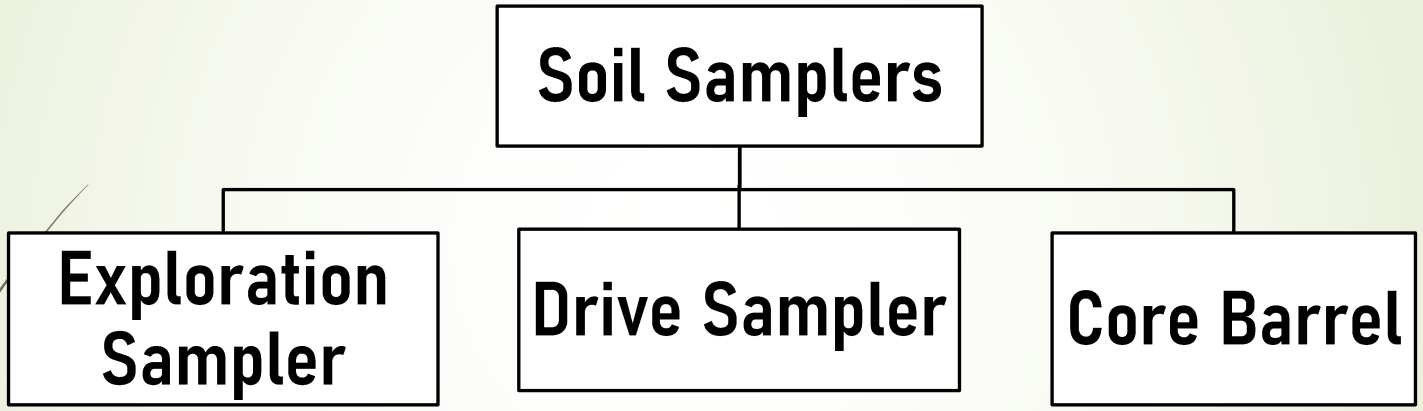
Frequency of sampling - In intermittent sampling, undisturbed soil samples are obtained at every change in stratum and at intervals not exceeding l.5 m within a continuous stratum.

On important investigations such as the foundations for an earth dam, **continuous core sampling in any soft clay layers** may be necessary.

7.3 Soil Samplers

A tool by which soil sample is collected is known as soil sampler.

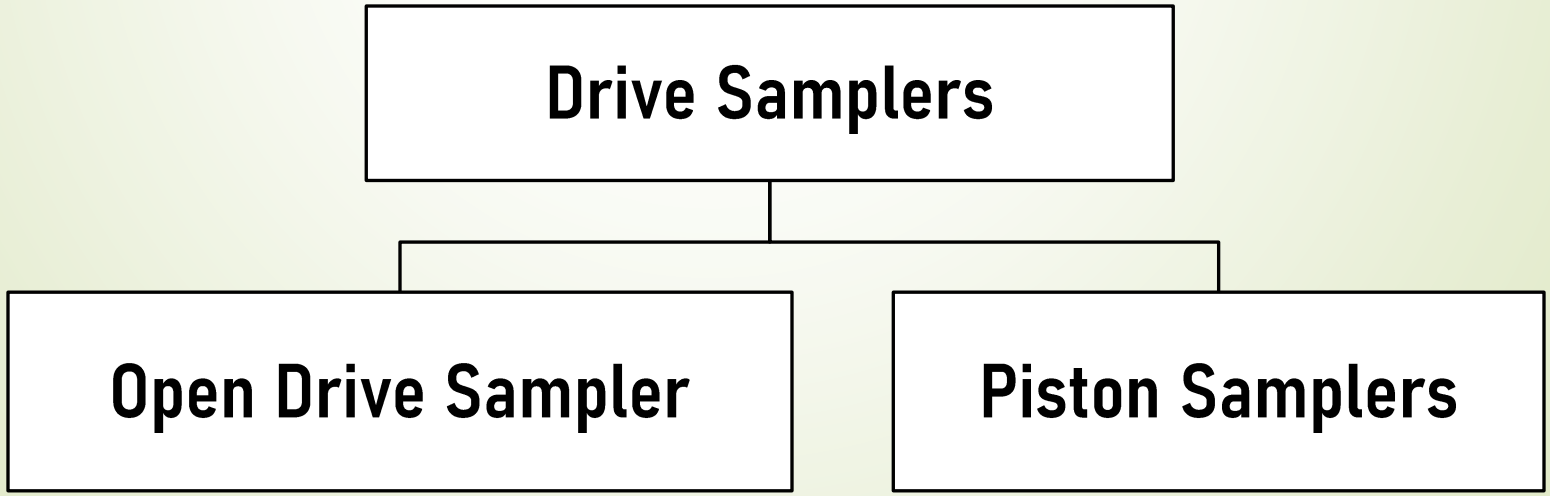
Hvorslev (1949) classified soil samplers in 3 categories.



Exploration samplers are generally used when the subsurface conditions are unknown at the proposed site, and reconnaissance survey is required to determine the type of soil. Usually, the boring tool acts as an exploration sampler; e.g. auger, wash boring. Core barrel is used to collect rock sampler and very stiff soil.

7.3 Soil Samplers

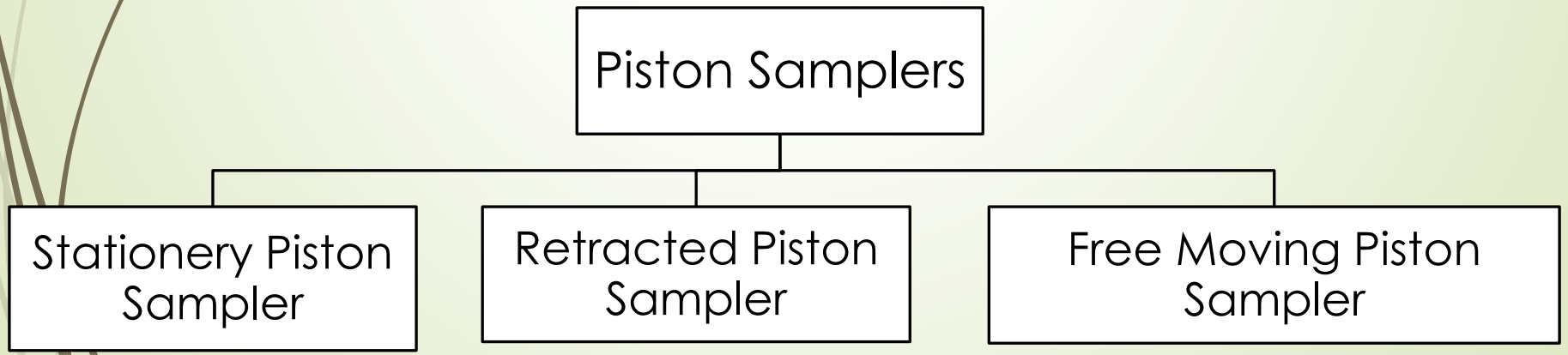
A drive sampler consists of a cylindrical tube that is pushed or driven without rotation into the soil. When the drive is complete, the sampler is rotated which separates the soil sample from the parent material. Drive samplers can be further classified into two categories depending on the inside arrangement of the sampler.



7.3 Soil Samplers

An open drive sampler consists of a cylindrical tube beveled at the lower end and secured to the bottom of a drill pipe at the other end. A schematic diagram is shown in Fig. 13(a).

Piston sampler is a drive sampler with a piston. The piston assists the driving operation by (i) preventing the entrance of excess soil and (ii) helping to hold the specimen into the tube. Piston sampler is illustrated in Fig. 13(b). Piston samplers can be further classified into 3 categories as shown in the following line diagram.



7.3 Soil Samplers

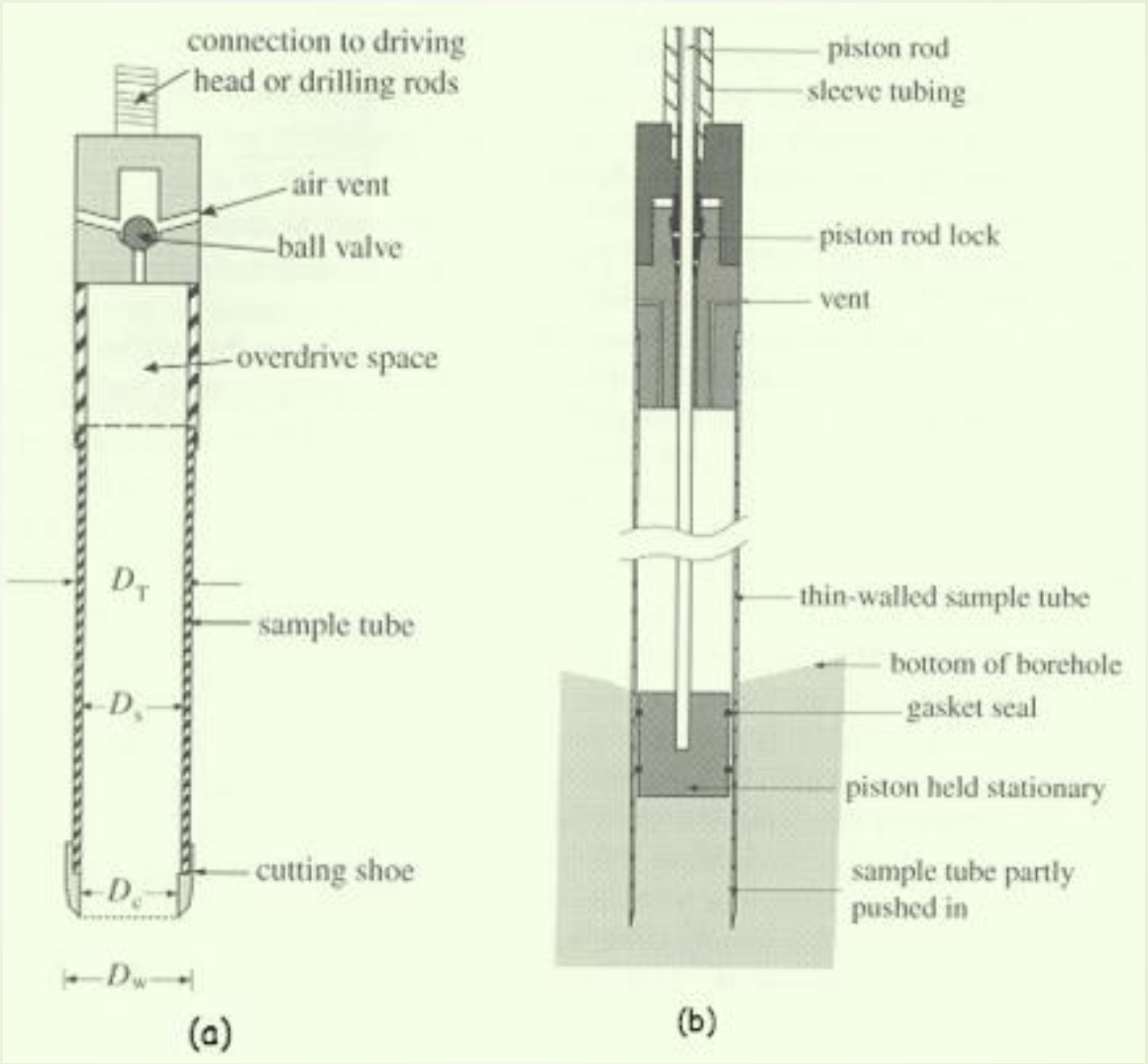


Fig. 13: Drive samplers; (a) Open drive sampler; (b) Piston sampler

7.3 Soil Samplers

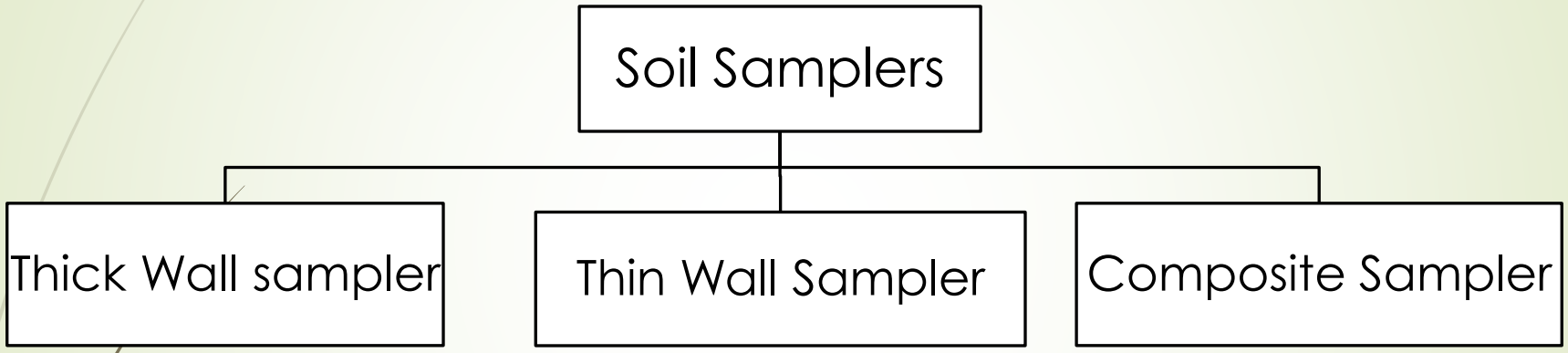
Stationary piston sampler, which functions with the piston secured at a fixed elevation during sampling

Retracted piston sampler, where casing is driven with the piston extended to the bottom of the sampler. The piston is then retracted to the top of the sampler to recover the sample.

Free moving piston sampler or Rotary piston sampler, in which the piston has the free movement inside the sampler during sampling.

7.3 Soil Samplers

Depending on the thickness of the wall of the sampler, it can be further classified as



Thick wall samplers are those having a smaller diameter to wall thickness ratio. The common type of thick wall sampler is Spilt Spoon Sampler or Spilt Barrel Sampler. The composite samplers are samplers having a liner inside to reduce the friction during sampling. Thin metal sheets or plastic tubes can be used as a liner.

7.3 Soil Samplers: Thick Wall Sampler

The Spilt Spoon sampler consists of a longitudinally split tube 50.8 mm in outside diameter and having 34.9 mm inside diameter. The splitting of the sampler permits it to be opened so that the specimen can be easily examined. To obtain the sample it is driven into the ground with a hammer. Usually disturbed samples are taken using split spoon samplers for identification purposes. Schematic diagram and picture of split spoon sampler is shown in Figs. 14(a) and 14(b) respectively.

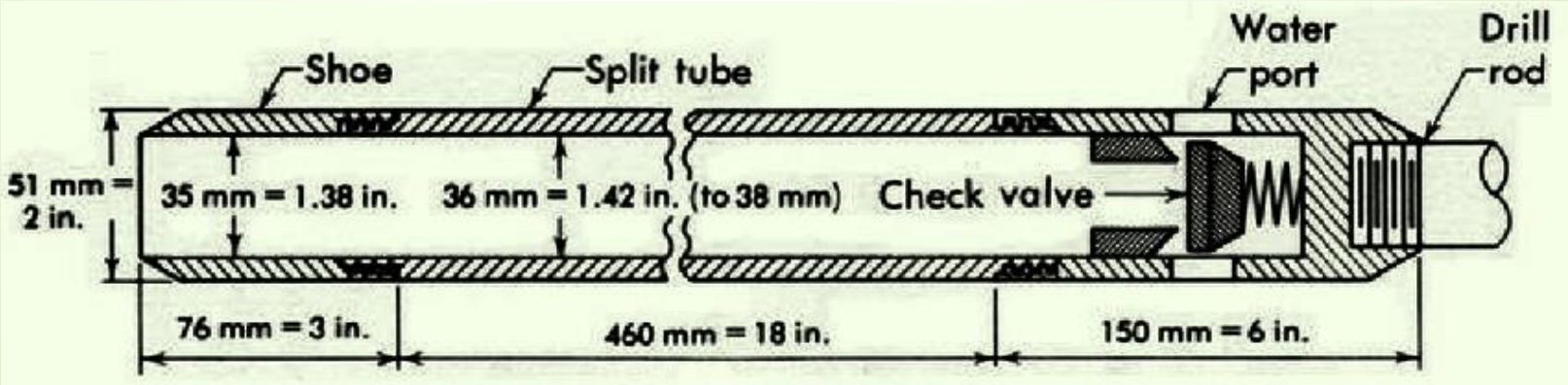


Fig. 14(a): Split spoon sampler

7.3 Soil Samplers: Thick Wall Sampler



Fig. 14(b): Split spoon sampler

7.3 Soil Samplers: Thin Wall Sampler

A thin wall sampler is a sampler, having a higher diameter to wall thickness ratio. The common thin walled sampler is Shelby tube and it is usually used to take undisturbed sample. The sampler is a seamless metal tube having a limited wall thickness. To obtain the sample, the tube is hydraulically pushed into the ground. Schematic diagrams and picture of Shelby tube are shown in Figs. 15(a), (b) and (c).

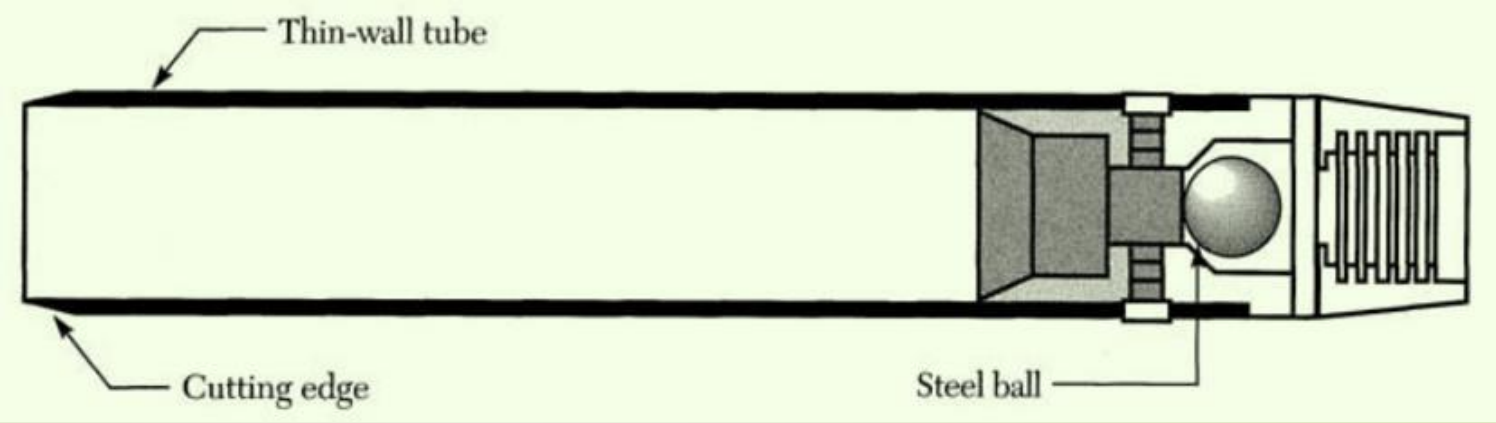


Fig. 15(a): Shelby tube

7.3 Soil Samplers: Thin Wall Sampler

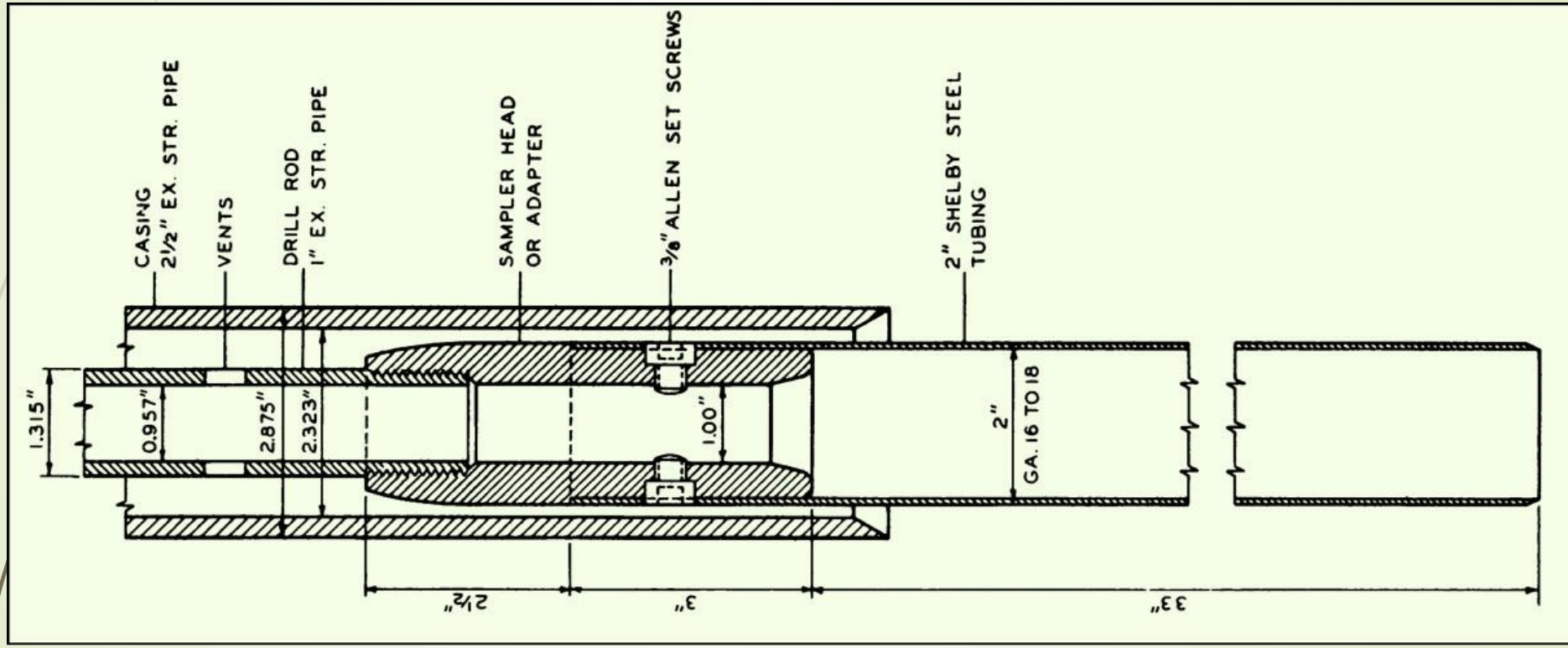


Fig. 15(b): Shelby tube

7.3 Soil Samplers: Thin Wall Sampler



Fig. 15(c): Shelby tube

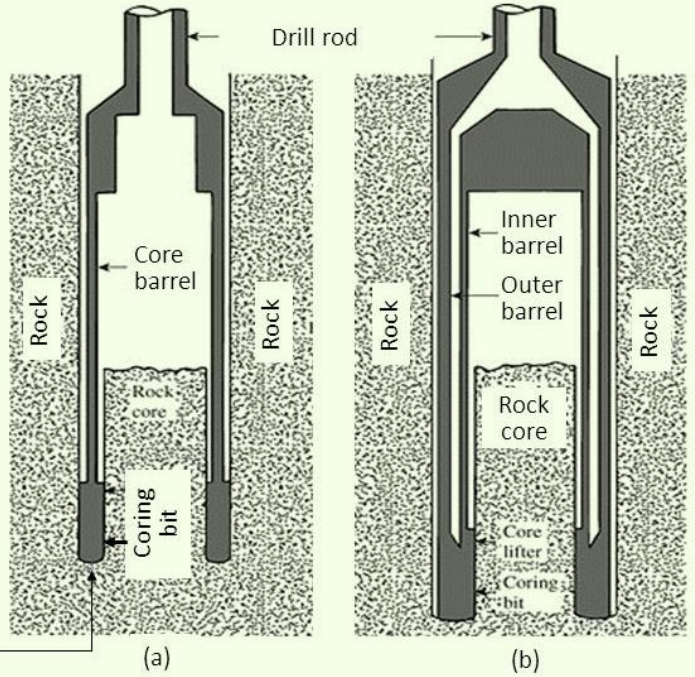
7.3 Soil Samplers: Core Barrel

Core drilling equipment

- Coring is done with either tungsten carbide or diamond core bits.
- Rock sampler is called "**core barrel**" which usually has a single tube.
- Double or triple tube core barrel is used when sampling of weathered or fractured rock.



Diamond Drill Bit



Core barrel: (a) Single-tube; (b) double-tube

44

Fig. 16: Core barrel; (a) Single tube; (b) Double tube

Core barrel sampler may be a single or double walled sampler that rotates or cores its way into undisturbed soil. The outer barrel rotates to cut into the soil. The sample is obtained with the inner barrel. It has the advantage of being able to obtain samples in hard or cemented soils. Core barrel samplers are often called Denison sampler or Denison core barrel. Diagrams of core barrels are shown in Fig. 16.

7.3 Soil Samplers: Core Barrel

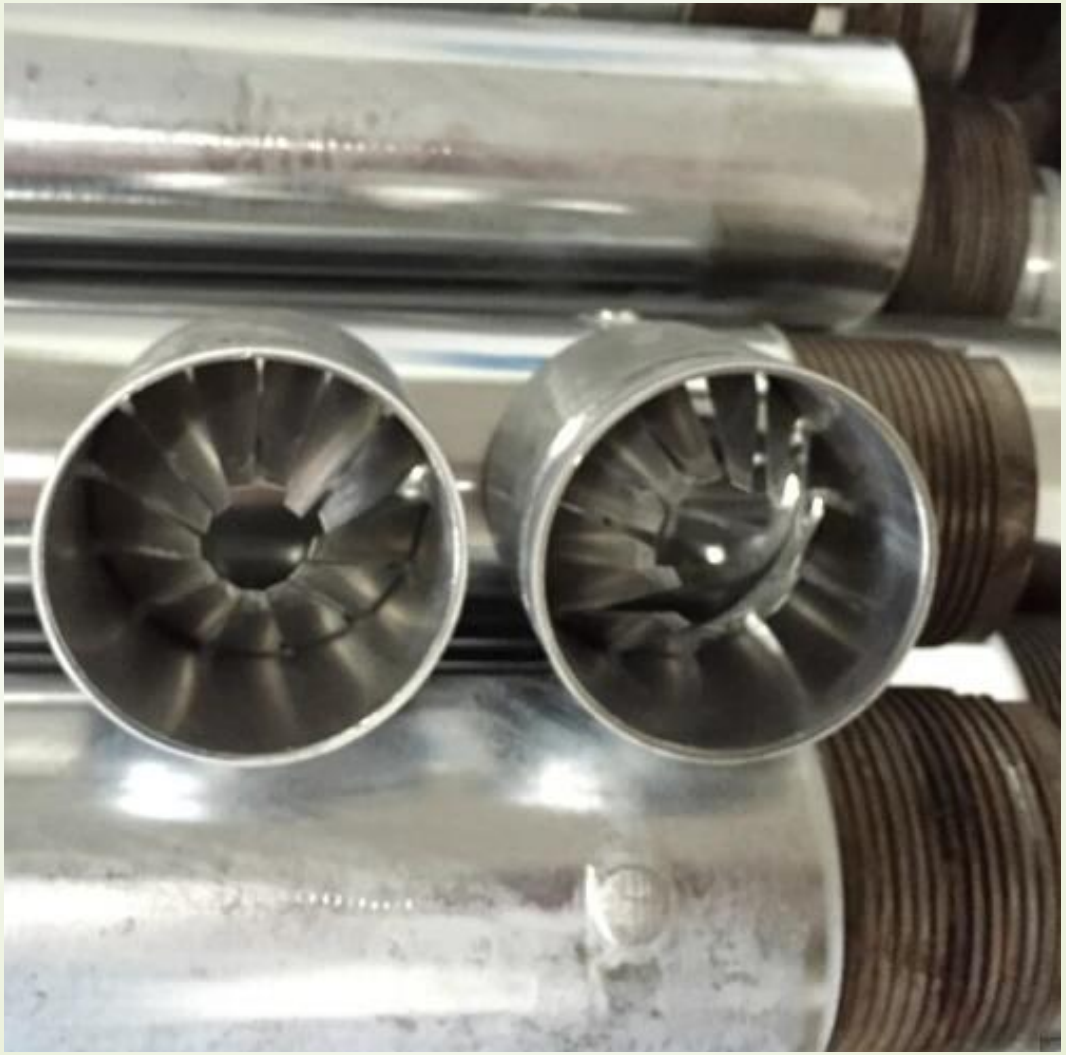
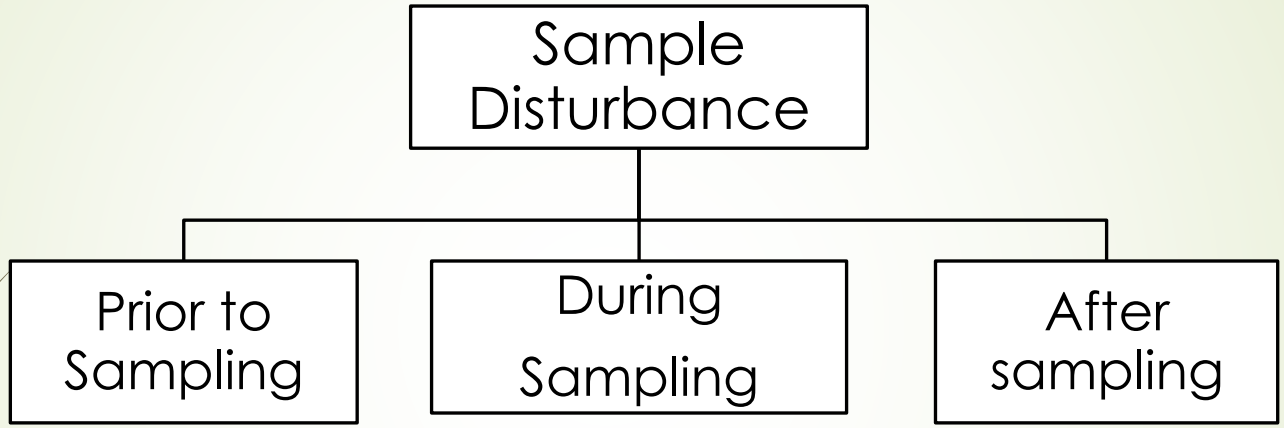


Fig. 16(c): Picture of core barrel

8.0 Sample Disturbance

A soil sample gets disturbed on three occasions:



Prior to soil sampling, the vertical stress at the bottom of the borehole is reduced due to removal of overburden. The soil layer below the borehole may also be deflected upward because of this stress relief. In continuous sampling, additional disturbance to the soil at the bottom occurs when the previous sample is sheared from the parent soil by torsion.

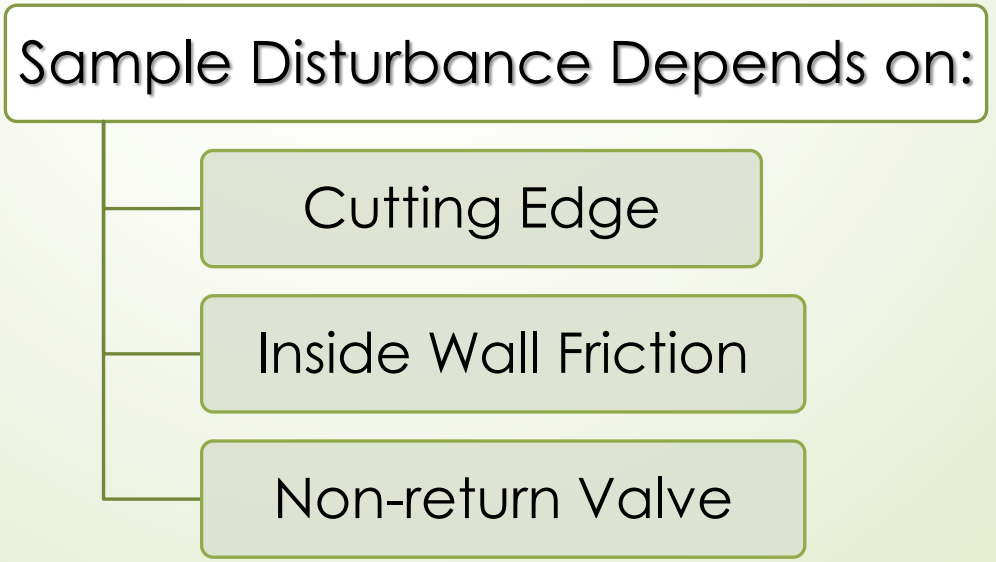
8.0 Sample Disturbance

During sampling, the sample disturbance is primarily due to displacement of soil by the wall of the sampler and friction and adhesion between soil sample and sampler. The other reason is attributed to the inside clearance between the sampler and the cutting shoe.

After sampling, a soil sample usually gets disturbed during transportation. It may also lose moisture because of improper sealing of the tube.

8.0 Sample Disturbance

While compared, the disturbance during sampling is much more significant and can be reduced designing proper soil sampler. The fundamental requirements of a sampling tool are that while forced into the ground it should cause minimum displacement, remoulding and disturbance. The extent of disturbance to the samples due to the sampler depends on three features of its design.



8.0 Sample Disturbance

A typical cutting edge is shown in Fig. 17. The parameters that determine the sample disturbance are given by:

$$Inside\ clearance\ ratio, C_i = \frac{100(D_s - D_c)}{D_c}(\%)$$

$$Outside\ clearance\ ratio, C_i = \frac{100(D_w - D_T)}{D_T}(\%)$$

$$Area\ ratio, A_r = \frac{100(D_w^2 - D_c^2)}{D_c^2}(\%)$$

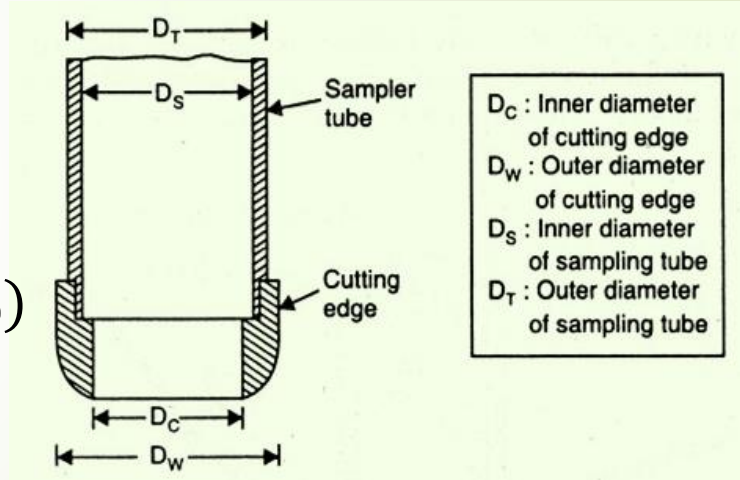


Fig. 17: Cutting edge of a sampling tube

8.0 Sample Disturbance

For a satisfactory undisturbed sample:

- Inside clearance ratio, Ci should be in between 1% and 3%.
- Outside clearance ratio, C_o should not be much greater than C_i.
- Area ratio, A_r should be kept as low as possible

The area ratio should be consistent with the strength requirement of the sampling tube. For stiff soils, A_r should not be greater than 20%; For soft sensitive clays, A_r should not greater 10%. If A_r is less than 10%, the sample disturbance is supposed to be small. A_r may be as high as 30% for thick wall sampler like split spoon and may be as low as 6 to 9% for thin wall samplers like Shelby tube.

8.0 Sample Disturbance

For a satisfactory undisturbed sample:

- According to ASTM, thin-walled sampling tubes have C_i ranging from 0% to 1.5%. For sampling in soft clays, C_i of 0 or less than 0.5% is used. In stiffer formations, larger C_i of 1%–1.5% may be required.
- Outside clearance ratio, C_o should not be much greater than C_i.
- Area ratio, A_r should be kept as low as possible

8.0 Sample Disturbance

Recovery Ratio - For a satisfactory undisturbed sample, taking into consideration the influence of the inside clearance when excess soil is prevented from entering the tube, the recovery ratio calculated as follows should be between 98 to 96 percent.

$$R_r = \frac{L}{H} \qquad (2)$$

Where,

$R_r =$ recovery ratio,

$L =$ length of the sample within the tube, and

$H =$ depth of penetration of the sampling tube.

8.0 Sample Disturbance

Inside Friction - This can be reduced by:

(a) suitable inside clearance,

(b) a smooth finish to the sample tube, and

(c) oiling the tube properly.

Non-return Valve - The valve should have a large orifice to allow the air and water to escape quickly and easily when driving the sampler.

8.0 Sample Disturbance

The degree of disturbance is also measured by recovery ratio given by the ratio of the length of sample recovered to the length of insertion of the sampling tube expressed as a percentage. The recovery should be in between 96% to 98% for undisturbed sample.

The quality of sampled rock is often defined as 'Rock Quality Designation' or RQD. It is the ratio of the total length recovered of pieces of core having length greater than or equal to 100 mm to the length of the sampler penetrated into the rock. This is expressed as a percentage. Table 4 gives classification of rock quality based on RQD.

Table 4. Classification of Rock Quality

RQD (%)	Rock Quality		RQD (%)	Rock Quality
90 - 100	Excellent		25 - 50	Poor
75 - 90	Good		0 - 24	Very Poor
50 - 75	Fair			

9.0 Groundwater Level

The position of GWT below ground level is needed for estimation of consolidation settlements and bearing capacity of foundation. As the GWT changes seasonally, it is necessary to establish the highest and the lowest water level. The position of GWT in a test pit can be directly measured with a tape.

A correct indication of the general ground water level is found by allowing the water in the borehole to reach an equilibrium level. For determining the water level in a borehole, a chalk-coated tape is lowered. The depth can also be measured by lowering the leads of an electrical circuit. When the open ends of leads touch the water, the circuit is closed glowing an indicator lamp. These methods are suitable in soils of high permeability, where the water level in the borehole stabilizes within 24 h at the most.

9.0 Groundwater Level

In soils of very low permeability, the groundwater level in the borehole does not stabilize even after several weeks. Casagrande's piezometer may be used in such cases for location of the water level.

A piezometer basically consists of a porous tip or a perforated pipe with a plastic standpipe attached to it. The tube is packed around with gravel or coarse sand and sealed in position with bentonite clay. Fig. 18 shows the general placement of a piezometer in a borehole.

The sand surrounding the porous tube should be kept saturated, during and after installation of the piezometer. The groundwater level is determined from the level of the water in the plastic tube.

9.0 Groundwater Level

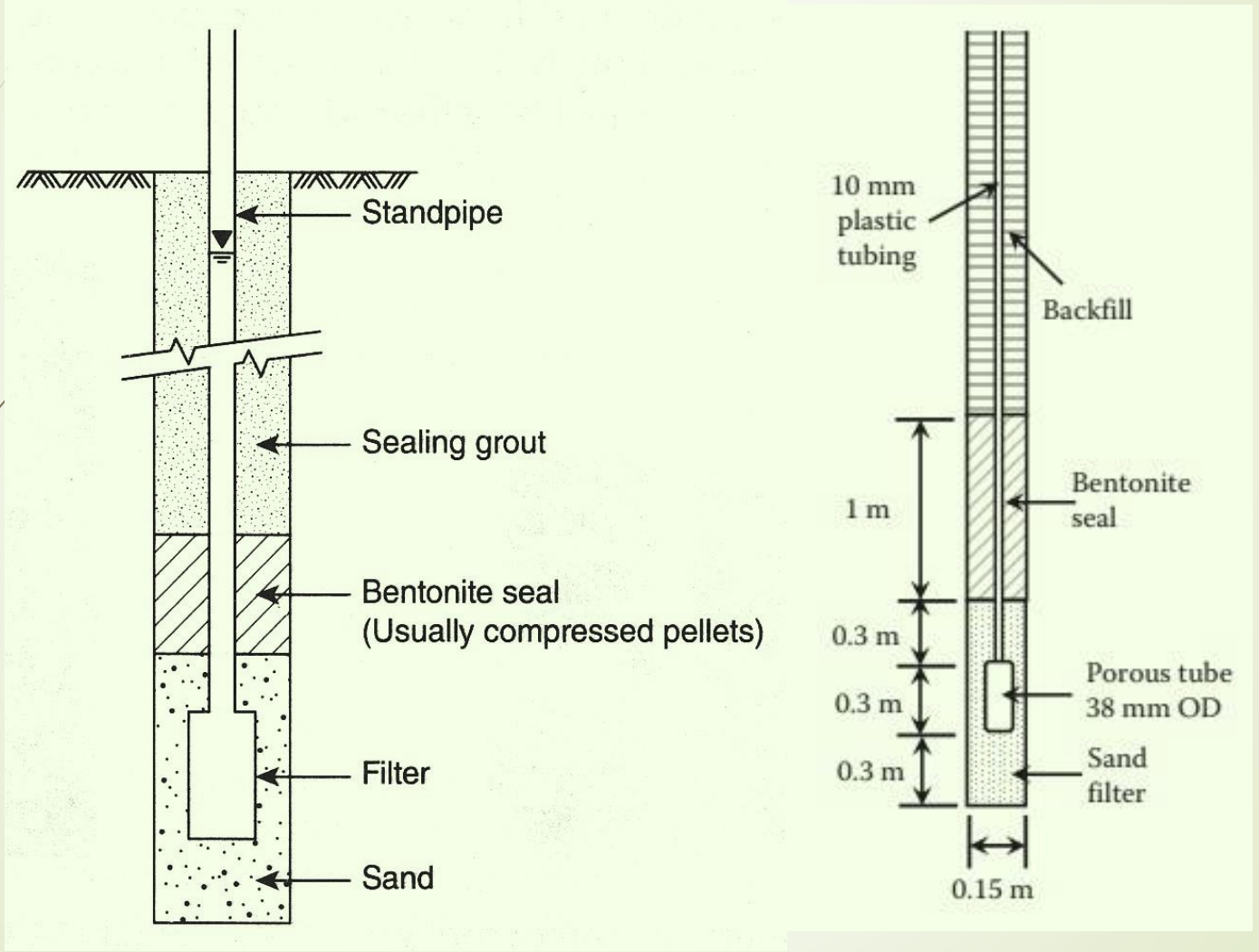


Fig. 18: Casagrande's piezometer for determining water table in fine grained soil

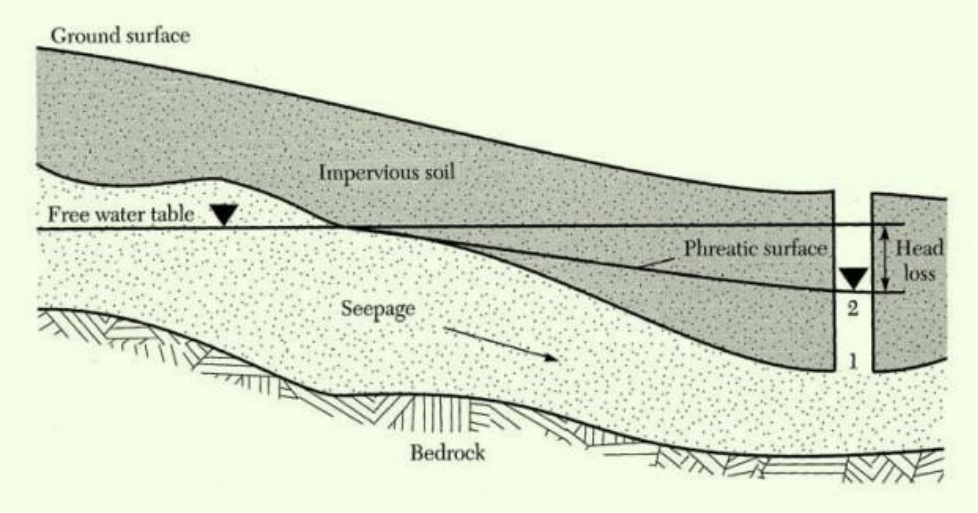
9.0 Groundwater Level

The recording of water levels at drilling sites is an important aspect of subsurface exploration. Failure to obtain an accurate water level may invalidate otherwise useful boring data. This is because groundwater may occur due to a free water surface, artesian conditions, or a perched water table. Consequently, groundwater may significantly affect various aspects of foundation design and construction.

9.0 Groundwater Level

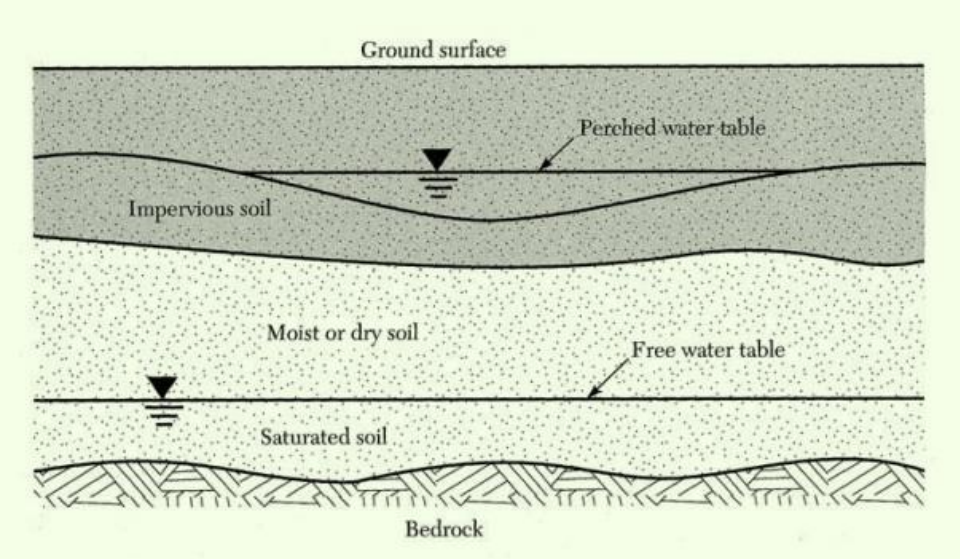
Artesian conditions are difficult to handle especially when dealing with a project that requires an excavation to be made below the water level. In such cases, proper precautions must be taken so that excess water is removed by pumping or trenching. This type of water occurs due to the presence of sloping confined aquifers as shown in Fig. 19. In this case, water is first encountered at the bottom of the borehole (level 1 in Fig. 19a), then rises to a stable elevation (level 2 in Fig. 19).The difference between the free water table and level 2 is due to energy loss caused mainly by friction between water and the soil particles.

9.0 Groundwater Level



← Fig. 19(a): Artesian ground water

Fig. 19(b): Perched ground water →



9.0 Groundwater Level

Perched water is generally of no consequence during construction but is of extreme importance when one is calculating the effective overburden pressures within a given soil deposit. The implication is that if perched water is erroneously assumed to be the free water table, then estimates of settlements and the bearing capacity of the foundation will be in error. A perched water table is developed when the water is an accumulation above a sagging impervious formation (Fig. 19b).

9.0 Groundwater Level

It is evident that, in this case, the free water table may be located several feet below the perched water level. Perched water generally disappears a few hours after the borehole is extended through a permeable soil layer and will not rise as free water would. Note that in coarse sand and gravel formations, the groundwater level may rise and fall due to changes in weather, discharge, and recharge. In such deposits, the free groundwater surface would normally slope in the same manner as the ground surface. These variations in the water table are not as pronounced in impervious formations such as clay and silt.

9.0 Groundwater Level

Water levels in a borehole can be estimated by the drilling crew by lowering a float attached to a tape measure. For good results to be obtained, the borehole should be cased and left open and the water level recorded at least two different times (preferably at 24-h increments). In most cases, the final water level recorded is assumed to be the depth of the free water table.

One can use the procedure proposed by Hvorslev (1949) for estimating the correct water table depth. This procedure involves bailing water out of the borehole, then recording the depth of the water level D_o at time t_o. The water level at times $t_o+\Delta_t$ and $t_o+2\Delta_t$ is recorded as D_1 and D_2 respectively, where Δ_t is the time increment (Δ_t=24 hours is recommended).

9.0 Groundwater Level

This approach is illustrated graphically in Fig. 20. Although the water is shown to rise in the borehole, it can also fall. The recorded depth values D_1 and D_2 can then be used to approximate the water table depth as follows:

$$D_w = \frac{D_1 + D_2}{2} - \frac{1}{2}\left[\frac{Z_1^2 + Z_2^2}{Z_1 - Z_2}\right] \qquad (3)$$

Where, D_w is the depth of the free water table, $Z_1=D_o-D_1$, and $Z_2= D_1-D_2$. Clearly Z_1 and Z_2 must not be equal for Eq. (3) to be valid. Additionally a piezometer, an instrument for measuring pressure head in soil deposits should be used for water level determination if more precise field data are required.

9.0 Groundwater Level

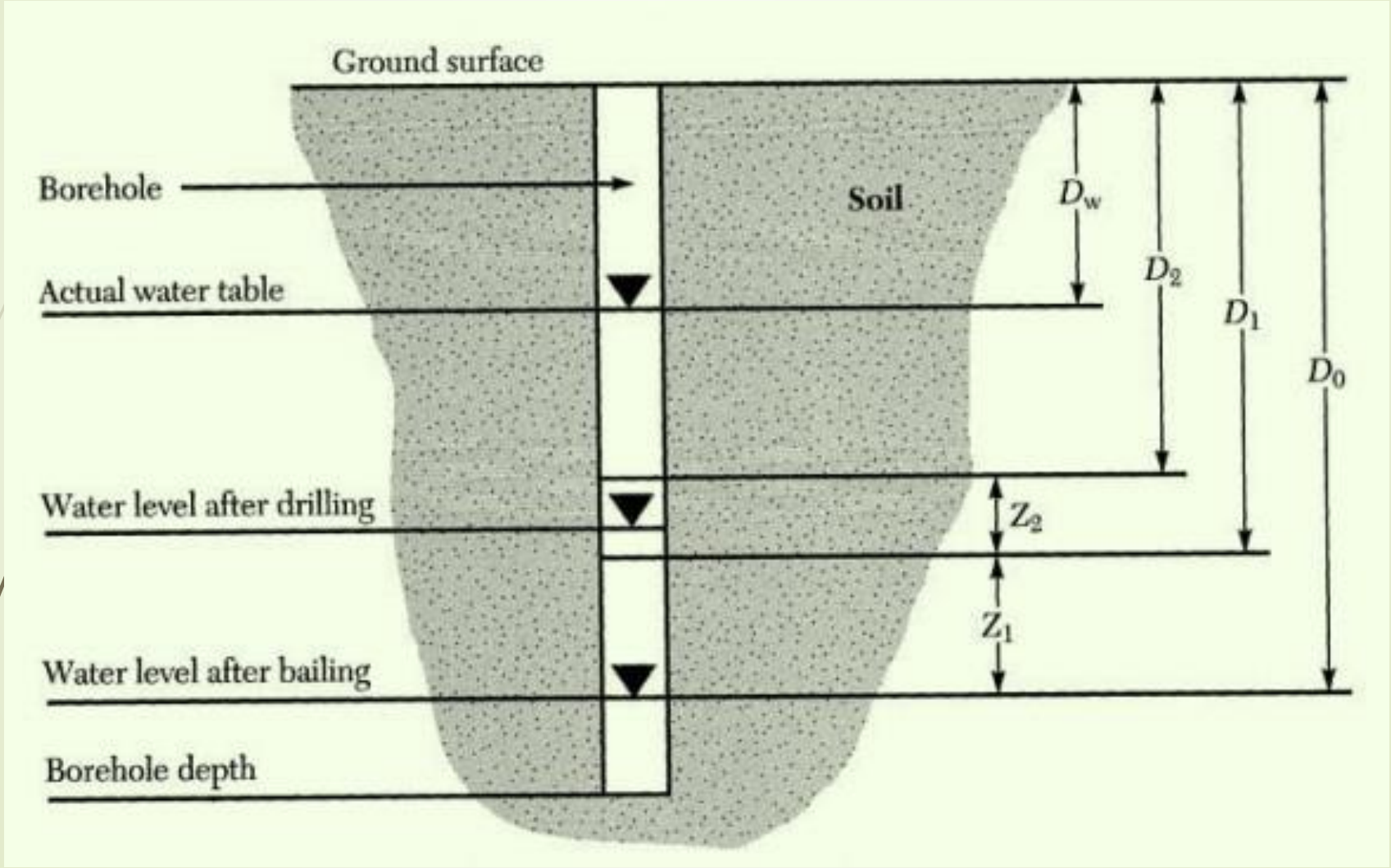


Fig. 20: Methods of estimating actual ground water

10.0 Sounding and Penetration Tests

Subsurface sounding are used for exploring soil strata of erratic nature. They are useful to determine the presence of any soft pockets between drill holes and also to determine the density index of cohesionless soils and the consistency of cohesive soils at desired depth.

10.0 Sounding and Penetration Tests

Methods of sounding consist of driving or pushing a standard sampling tube or a cone. The devices are also termed as penetrometers, since they penetrate the subsoil with a view to measuring the resistance to penetrate the soil strata. The tests are also known as penetration tests.

If a sampling tube is used to penetrate the soil, the test is referred to as Standard Penetration Test (or simply SPT). If a cone is used, the test is called a Cone Penetration Test. If the penetrometer is pushed steadily into the soil, the procedure is known as Static Penetration Test. If driven into the soil, it is known as Dynamic Penetration Test.

A field test called vane shear test may be used to determine the shearing strength of the soil located at a depth below the ground.

10.1 Standard Penetration Test

The Standard Penetration Test (SPT) is widely used to determine the insitu properties of soil. The test is especially suited for cohesionless soils as the correlation between the SPT value and φ is now well established.

The test consists of driving a split spoon sampler (Figs. 14, 21a, 21b) into the soil through a borehole 55 to 100 mm in diameter at the desired depth. It is done by a hammer weighing 65 kg (140 1b) dropping onto a drill rod from a height of 750 mm (30 inch).

10.1 Standard Penetration Test



Fig. 21(a:) Standard penetration test penetrometer (Split Spoon Sampler)

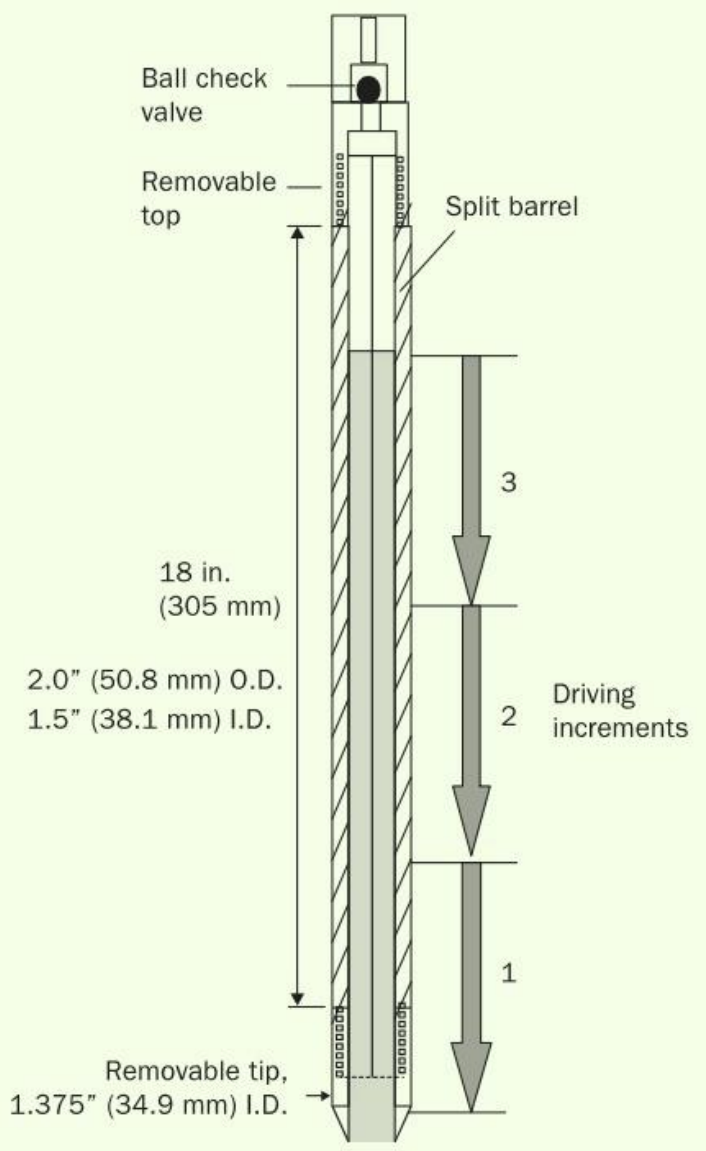
10.1 Standard Penetration Test



Fig. 21(b): Split-spoon sampler empty and with a disturbed sample

10.1 Standard Penetration Test

The number of blows, N required to produce a penetration of 300 mm (12 inches) is regarded as the penetration resistance. To avoid seating errors, the blows for the first 150 mm (6 inches) of penetration are not taken into account. The blow count for the next two 150-mm increments is reported as the penetration resistance (or penetration count) N unless the last increment cannot be completed (refusal).



10.1 Standard Penetration Test

According to ASTM, the boring should continue until one of the following (refusal) occurs:

1) A total of 50 blows during any one of the three 6.0 in [150 mm] increments described earlier.
2) A total of 100 blows have been applied.
3) There is no observed advance of the sampler during the application of 10 successive blows of the hammer. For automated systems an advance of less than 0.1 in. [2 mm] per blow can be considered refusal.

The sampler is advanced the entire 1.5 ft. [0.45 m] without the limiting blow counts occurring as described in (1) to (3).

10.1 Standard Penetration Test

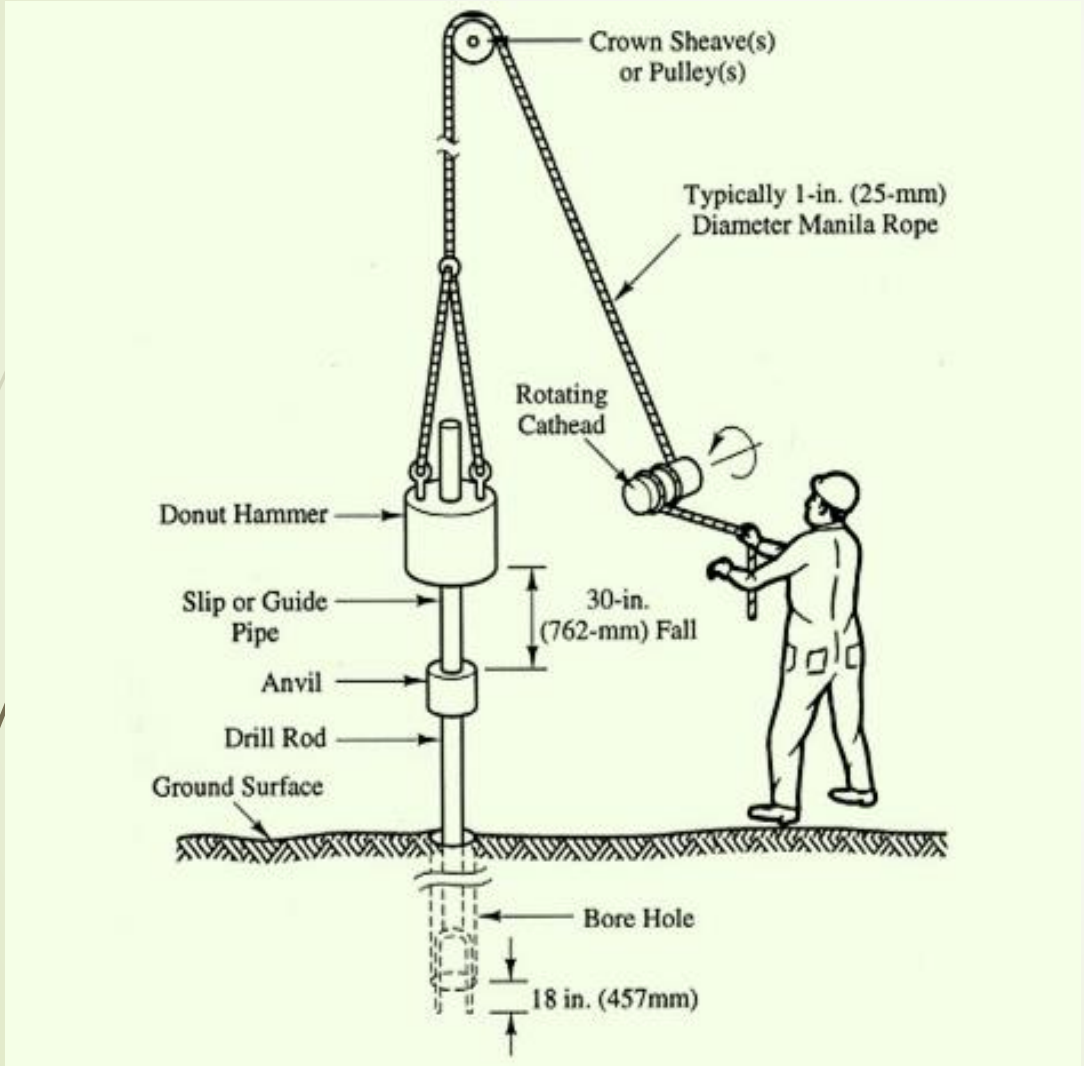


Fig. 22(a): Standard penetration test setup

10.1 Standard Penetration Test

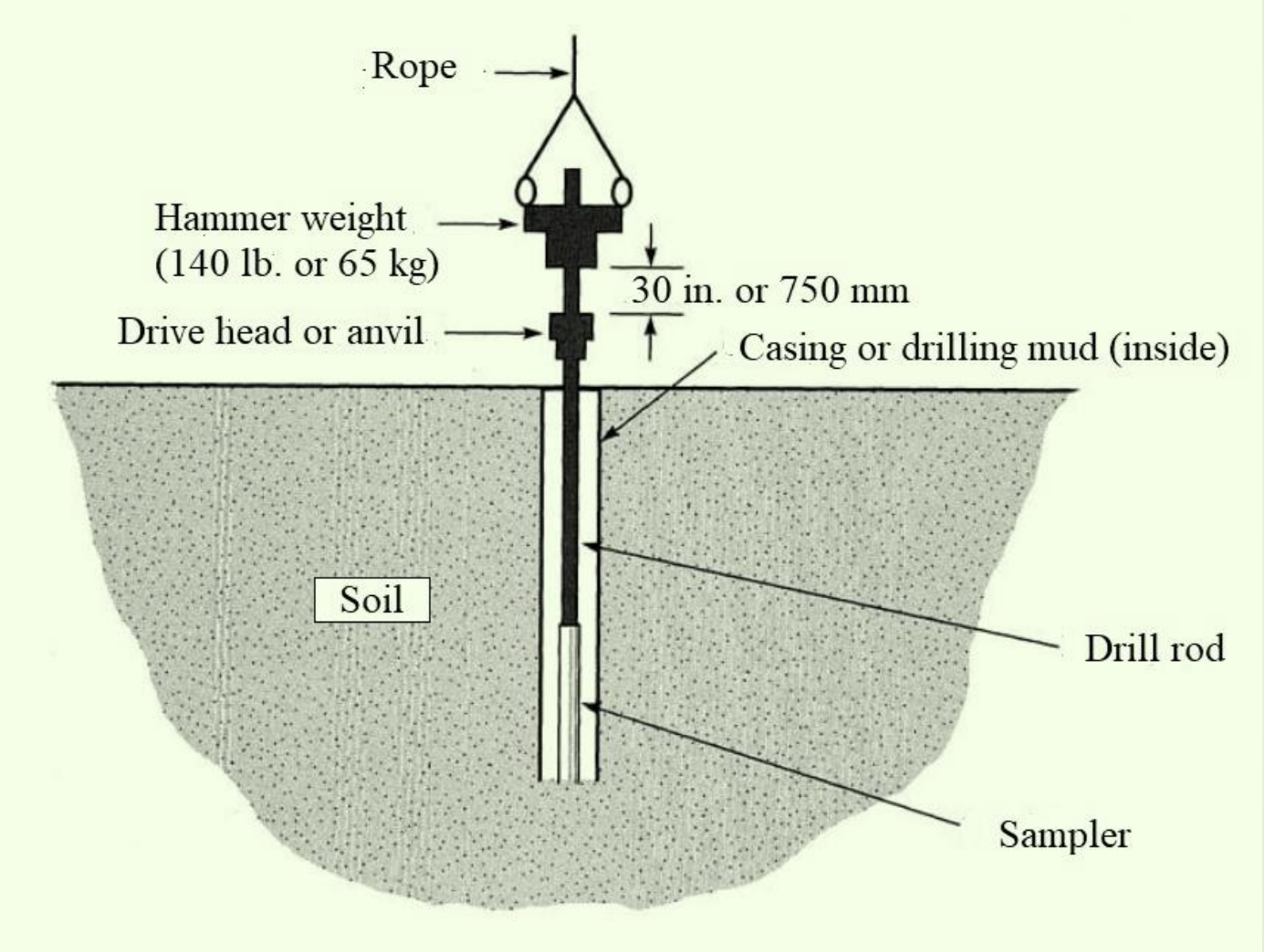


Fig. 22(b): Cross section of a typical borehole for SPT

10.1 Standard Penetration Test

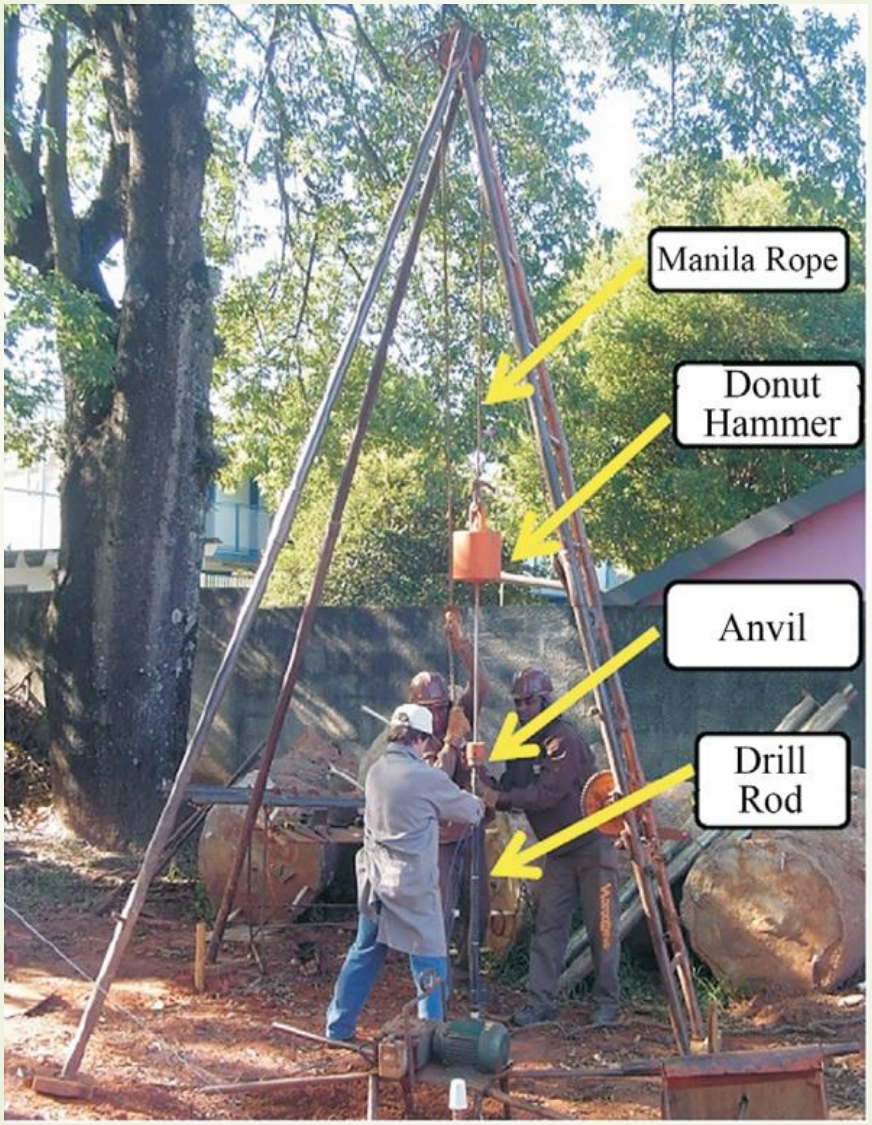


Fig. 23: Standard penetration test

10.1 Standard Penetration Test

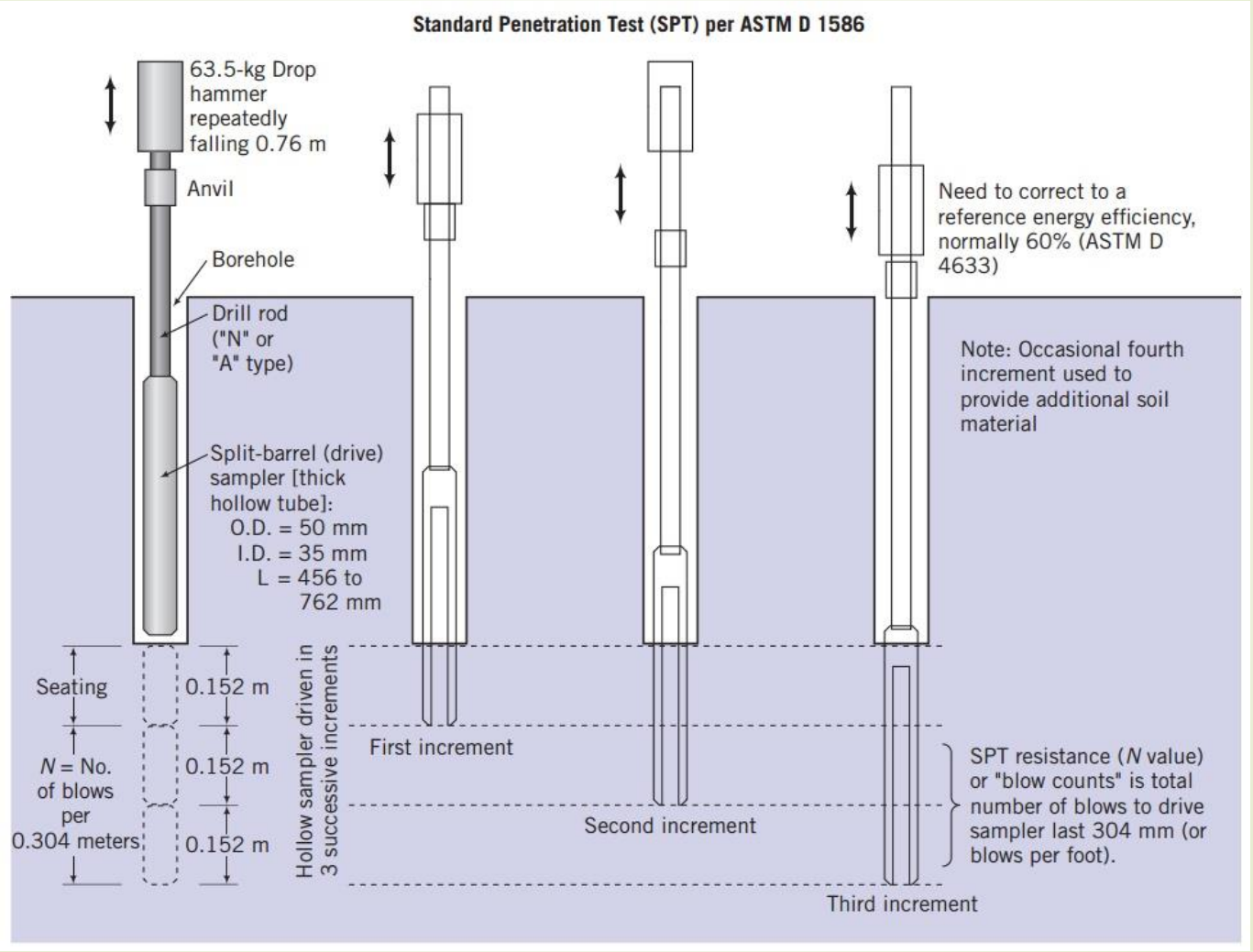


Fig. 24(a): Driving sequence in an SPT (as per ASTM)

10.1 Standard Penetration Test

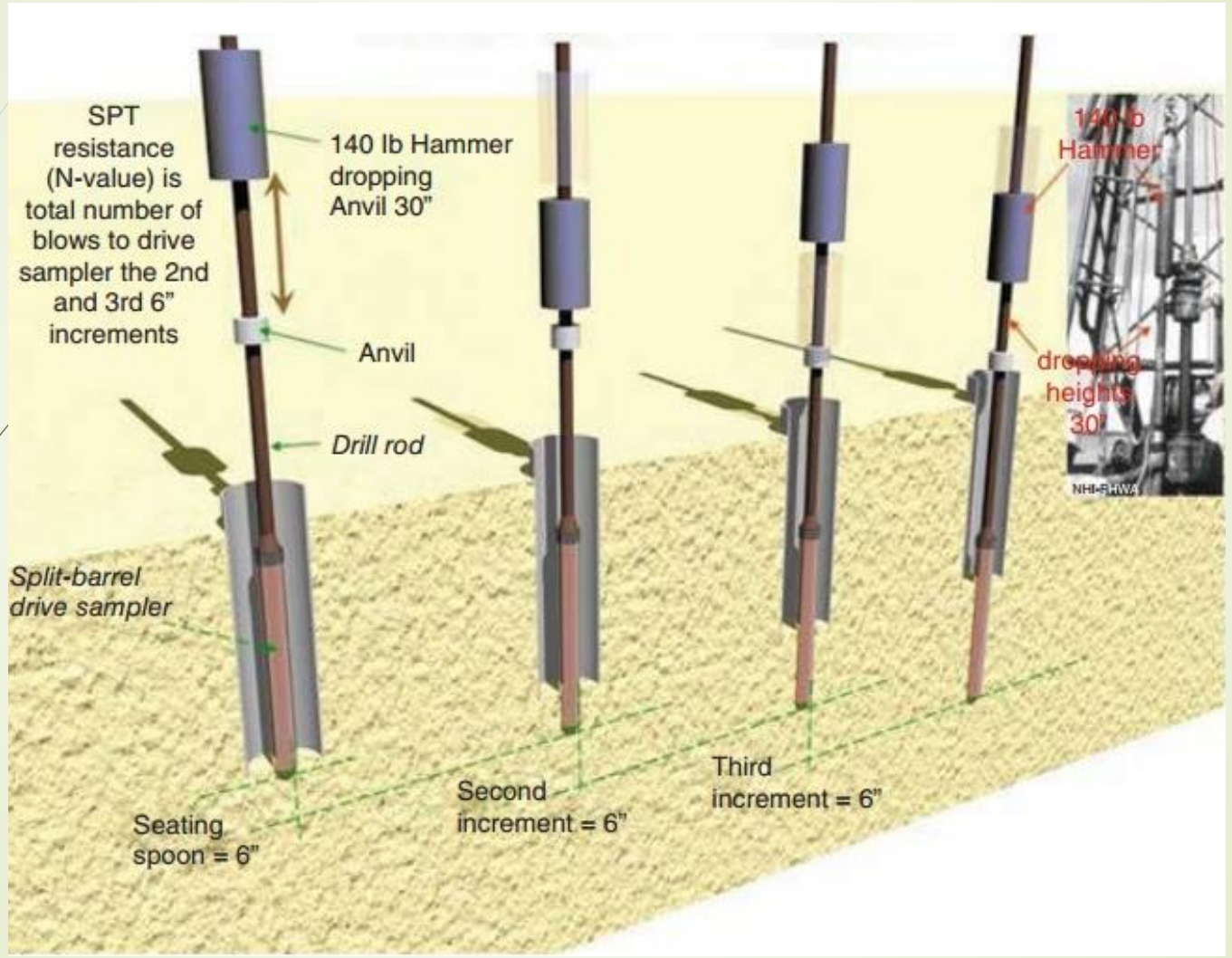


Fig. 24(b): Driving sequence in an SPT (as per ASTM)

10.1 Standard Penetration Test

If the sampler cannot be driven a 6-in. distance because of cemented hardpan, very dense hard soil, or rocks, the N-value is reported as number blows/ number of inches driven. For example, if 80 blows were required to drive the sampler a distance of 3in., then the blow count is reported as 80/3. Drilling operations are normally stopped at refusal. ASTM, however, defines one of the criteria of refusal as 100 blows/ft. This is also referred to as hard driving and is very expensive to the client.

10.1 Standard Penetration Test

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 hammer efficiency, borehole diameter, sampling method, rod length, water table and overburden pressure important.

The most two common types of SPT hammers used in the field are the safety hammer and donut hammer. They are usually dropped using a rope with two wraps around a pulley. The hammers are shown in Fig. 25. Usually SPT is conducted at every 1.5 m depth or at the change of stratum.

10.1 Standard Penetration Test

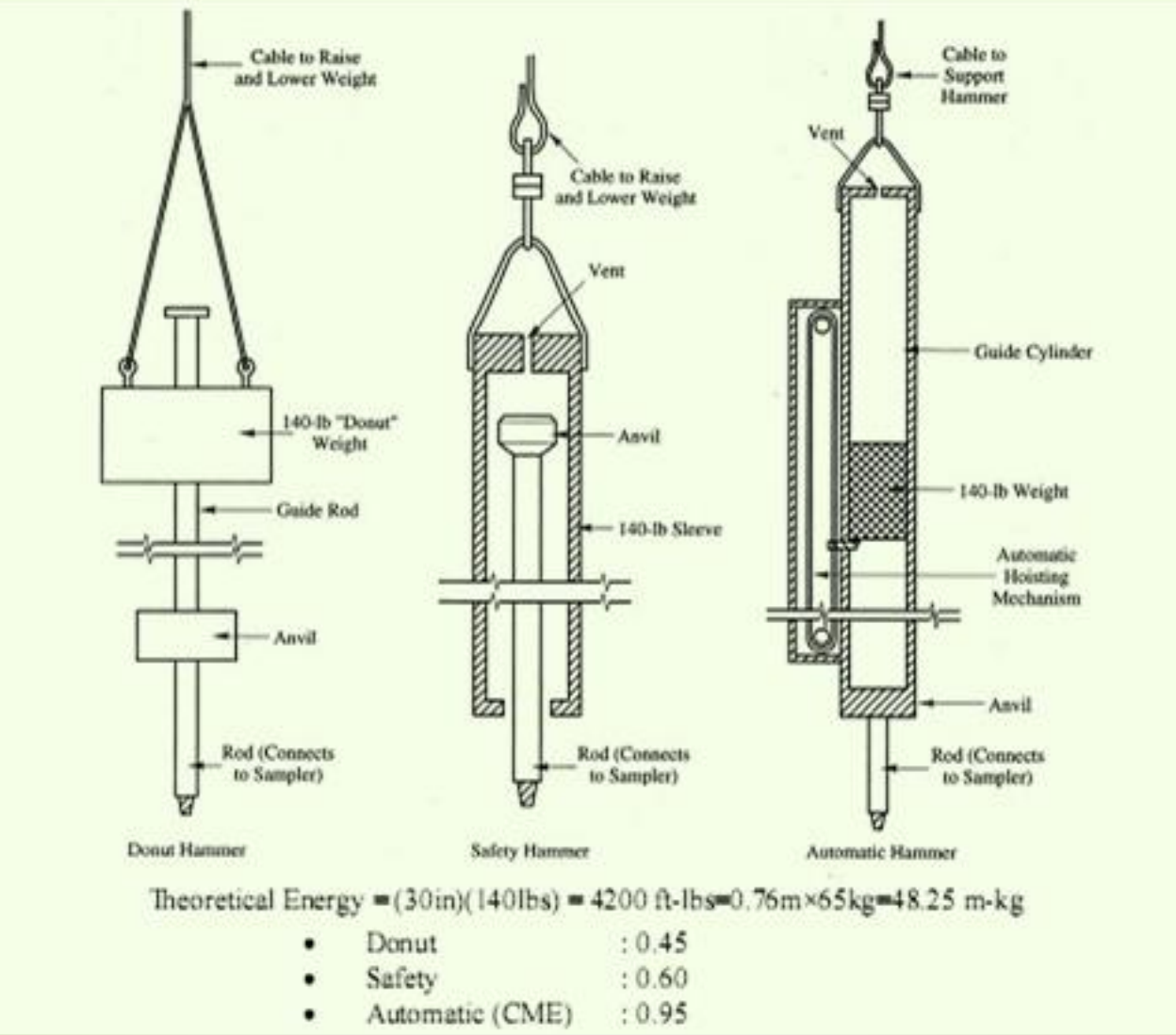


Fig. 25(a): Various SPT hammers

10.1 Standard Penetration Test

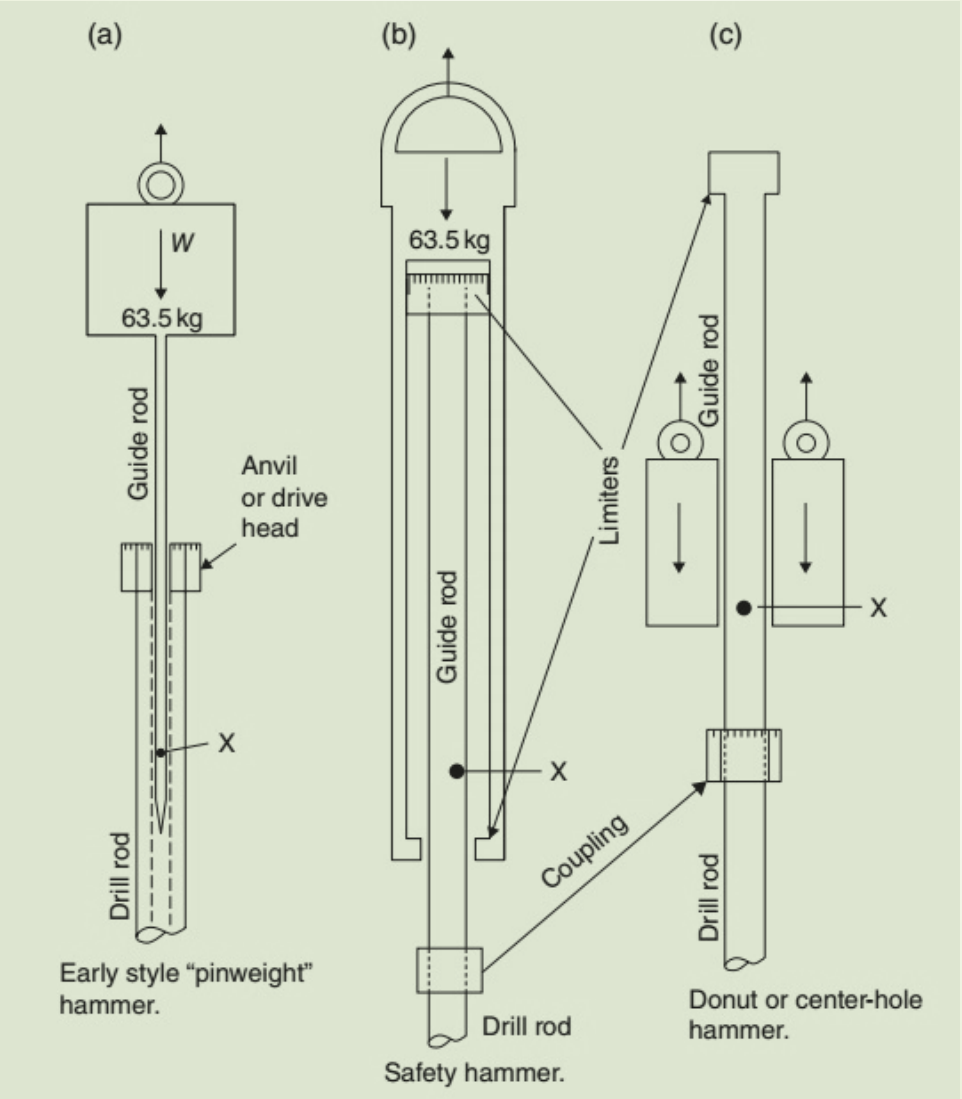


Fig. 25(b): Various SPT hammers

10.1 Standard Penetration Test

(A) Corrections/ Standardization of SPT Value for Field Procedures for All Types of Soil

On the basis of field observations, it appears reasonable to standardize the field SPT number as a function of the input driving energy and its dissipation around the sampler around the surrounding soil. The variations in testing procedures may be at least partially compensated by converting the measured N to N_{60} as follows. This correction has to be done irrespective to the type of soil.

10.1 Standard Penetration Test

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} \qquad (4)$$

Where,

$N_{60} = Corrected\ SPT\ N - value\ for\ field\ procedures$

$E_H = Hammer\ efficiency\ (Table\ 5)$

$C_B = Borehole\ diameter\ correction\ (Table\ 6)$

$C_S = Sampler\ correction\ (Table\ 6)$

$C_R = Drill\ rod\ correction (Table\ 6)$

$N = Measured\ SPT\ N - value\ at\ field$

10.1 Standard Penetration Test

Table 5: SPT Hammer Efficiency

Country	Hammer Type	Release Mechanism	Efficiency (E_H)
Argentina	Donut	Cathead	0.45
Bangladesh	Donut	Hand dropped	0.50
Brazil	Pin weight	Hand dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
	Donut	Cathead	0.50
Columbia	Donut	Cathead	0.50
India	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
Japan	Donut	Tambi trigger	0.78 – 0.85
	Donut	Cathead 2 turns + special release	0.65 – 0.67
UK	Automatic	Trip	0.73
USA	Safety	Cathead 2 turns	0.55 – 0.60
	Donut	Cathead 2 turns	0.45
Venezuala	Donut	Cathead	0.43

10.1 Standard Penetration Test

Table 6: Borehole, Sampler and Rod Length Correction Factors		
Factor Item	Equipment dimensions	Correction Factors
Borehole diameter correction factor, C_B	65 mm – 115 mm	1.00
	150 mm	1.05
	200 mm	1.15
Sampling Correction factor, C_S	Sampler with liner	1.00
	Sampler without liner	1.20
Rod length correction factor, C_R	3 m – 4 m	0.75
	4 m – 6 m	0.85
	6 m – 10 m	0.95
	> 10 m	1.00

10.1 Standard Penetration Test

(B) Corrections of SPT Value for Water Table in Case of Fine Sand and Silty Sands

In addition to corrections for field procedures, investigators suggested corrections of SPT-value for water table in the case of fine sand or silt below water table. Apparently, high N-values may be observed especially when observed value is higher than 15 due to dilatancy effect. In such cases, following correction is recommended (Terzaghi and Peck, 1948).

10.1 Standard Penetration Test

$$N_{WTC} = 15 + \frac{1}{2}(N_{60} - 15) \qquad (5)$$

Where, N_{WTC} and N_{60} are corrected N-values after water table and field procedures respectively. For SPT values less than 15 and for coarse sands N_{WTC} equals N_{60} .

(C) Corrections of SPT Values for Overburden Pressure for All Types of Cohesionless Soils

For SPT made at shallow levels, the values are usually too low. At a greater depth, the same soil of similar density index would give higher penetration resistance. It was only as late as in 1957 that Gibbs & Holtz suggested that corrections should be made for field SPT values for depth.

10.1 Standard Penetration Test

As the correction factor come to be considered only after 1957, all empirical data published before 1957 like those by Terzaghi is for uncorrected values of SPT. Since 1957 a number of investigators have suggested overburden correction. Gibbs & Holtz took standard pressure as 280 kN/m^2 (corresponding to a depth of 14 m) and duly made overburden correction for other values of overburdens. Thornburn suggested a standard pressure of 138 kN/m^2 (corresponding to a depth of 7 m).

Finally, in 1974, Peck, Hanson and Thornburn suggested a standard pressure of 100 kN/m^2 (Equivalent to 1 tsf or 1 kg/cm^2 overburden corresponding to a depth of 5 m).

10.1 Standard Penetration Test

According to them all SPT field values are to be corrected using an overburden correction factor C_N.

$$C_N = 0.77 \log\left(\frac{2000}{\sigma_o'}\right) \quad for\ \sigma_o' \geq 0.25\ kN/m^2$$

Where, σ_0' is the effective overburden pressure at the depth of SPT. Another correction factor is given by Liao and Whitman (1986) as follows.

$$C_N = \sqrt{\frac{1}{\sigma_o'}} \quad Where, \sigma_o'\ is\ in\ tsf\ or\ kg/cm^2$$

The value of C_N should not be larger than 2.0 in both the cases.

10.1 Standard Penetration Test

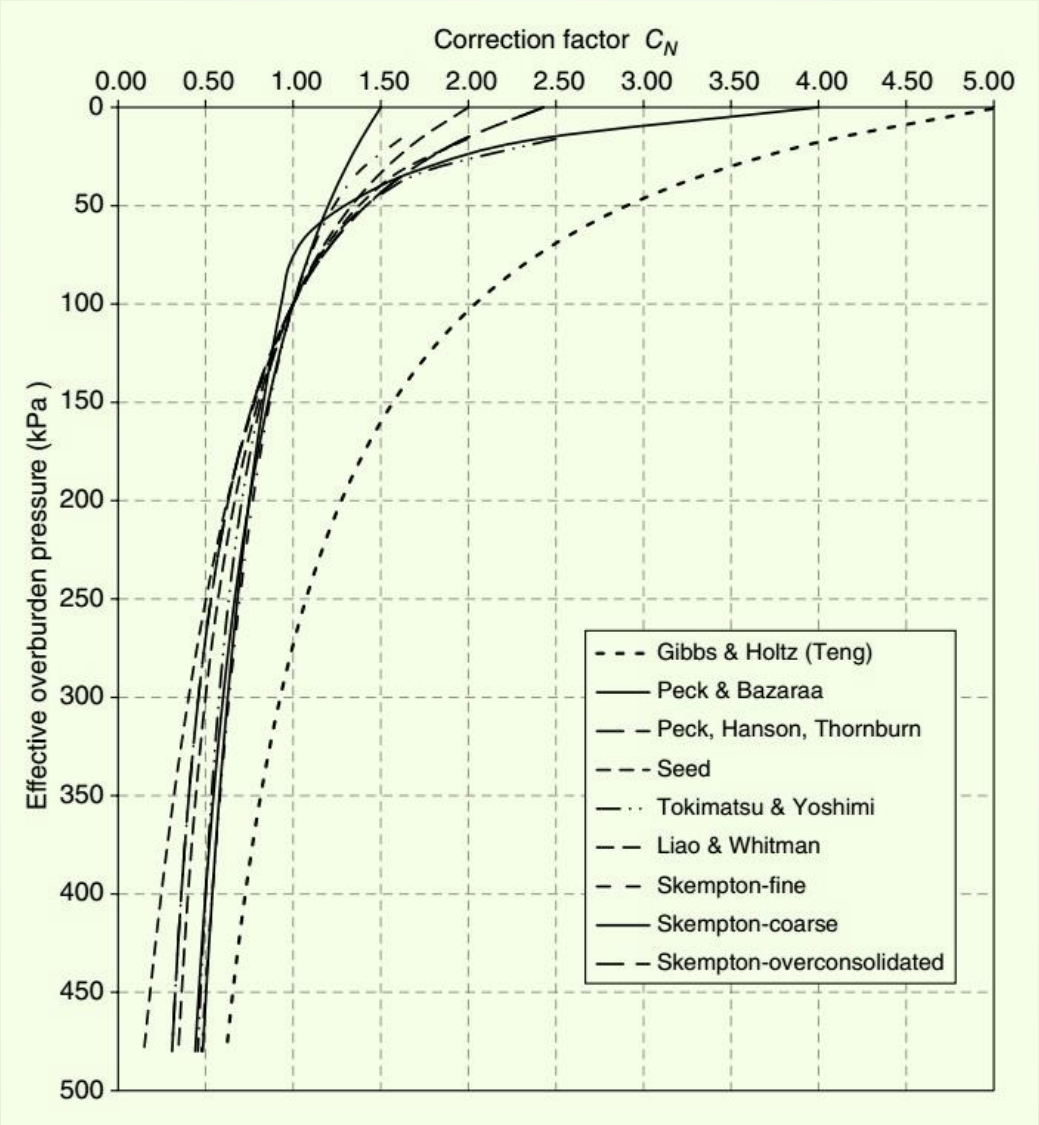


Fig. 26: SPT correction factors for overburden pressure

10.1 Standard Penetration Test

In case of fine sands, the water correction should be done prior to overburden correction. While using the correction factor given by the above equations, the corrected N value is termed as $(N_1)_{60}$ as this correction corresponds to an overburden pressure of 1 tsf or 1 kg/cm^2. Thus,

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

There is no substantial evidence to suggest the use of overburden correction factors when using the SPT in fne-grained soils. However, Oskorouchi & Mehdibeigi (1988) suggested that an overburden correction factor be applied to SPT N-values in medium to stiff clays (N>10) for estimating undrained shear strength.

10.1 Standard Penetration Test

(D) SPT Value, Density Index and ϕ

Though, SPT is not considered to be a refined and a completely reliable method of investigation, it gives useful information with regard to relative density of cohesionless soil and consistency of cohesive soils.

Terzaghi and Peck presented correlations (Tables 7) between SPT value and angle of internal friction of sandy soil. Several investigators also presented relations between SPT value and peak angle of internal friction.

10.1 Standard Penetration Test

Table 7: SPT and Soil Properties (Cohesionless soil: Fairly Reliable			
N- Value	Soil Condition	Relative Density, D_r (%)	Angle of internal friction, φ^o
0 - 4	Very loose	0 - 15	< 28
4 - 10	Loose	15 - 35	28 - 30
10 - 30	Medium	35 - 65	30 - 36
30 - 50	Dense	65 - 85	36 - 42
> 50	Very dense	> 85	> 42

10.1 Standard Penetration Test

(D) SPT Value, Density Index and ϕ

A number of early correlations of SPT N-values and relative density (relative density), D_r, of coarse-grained soils were presented at a time when either donut or safety hammer were used to perform the SPT. Differences in these correlations are related to variations in the composition, geologic origin, stress history, and moisture conditions of coarse-grained soils and the procedures and equipment used. For example, the correlation given by Gibbs & Holtz (1957) used a spoon with a constant internal diameter and no relief. Therefore, the blow counts may be higher than those obtained if a spoon with an internal relief had been used. A number of correlations developed after 1975 are summarized in Table 7a, as reported by Lutenegger (2021).

10.1 Standard Penetration Test

Table 7a: Some reported correlations between N and relative density, D_r (1)

Correlation	*Notes*	*References*
$D_r = 8.6 + 0.83\left((N + 10.4 - 3.2\ OCR\ - 0.24\ \sigma'_{vo})/0.0045\right)^{0.5}$ OCR = over-consolidation ratio	Fine sand σ'_{vo} in psi	Marcuson & Beiganousky (1977a)
$D_r = 12.2 + 0.75\left(222N + 2311 - 711\ OCR - 53\ \sigma'_{vo} - 50\ C_u^2\right)^{0.5}$ OCR = over-consolidation ratio C_u = uniformity coefficient	Coarse sand σ'_{vo} in psi	Marcuson & Beiganousky (1977b)
$D_r = 11.7 + 0.76\left(222N + 1600 - 53\ \sigma'_{vo} - 50\ C_u^2\right)^{0.5}$	NC σ'_{vo} in psi	Marcuson (1978)
$D_r = 0.118 + 0.441 \log N$		Borowczyk & Frankowski (1981)
$D_r/100 = (N)^{0.5}/\left(4.188 + 0.639\ \sigma'^{0.606}_{vo}\right)$	σ'_{vo} in metric tons/m^2	
$D_r = 16(N_1)^{0.5}$ $C_n = 1.7/(\sigma'_{vo} + 0.7)$	Clean sands σ'_{vo} in kg/cm^2 or tsf Sands with fines	Tokimatsu & Yoshimi (1983)

10.1 Standard Penetration Test

Table 7a: Some reported correlations between N and relative density, D_r (2)

$D_r = 16(N_1 + \Delta N_f)^{0.5}$

Fines content (FC)	ΔN_f
0 – 5%	0
5 – 10%	Interpolate
>10%	0.1FC + 4

$(N_1)_{60}/D_r^2 = 60$	
$N_1/D_r^2 = 17 + 22\,\sigma'_{vo}$	
$D_r = 22(N)^{0.57}\,\sigma'^{-0.14}_{vo}$	$D_r > 0.35$ — Skempton (1986)
$D_r = 18(N)^{0.57}\,\sigma'^{-0.14}_{vo}$	$\sigma'_{vo} = 0.5$ to 1.5 kg/cm^2, $D_r = 0.4$ to 0.9 — Skempton (1986)
$D_r = 25(N)^{0.44}\,\sigma'^{-0.13}_{vo}$	Fine sand — Yoshida et al. (1988)
$D_r = 25(N)^{0.46}\,\sigma'^{-0.12}_{vo}$	Gravel content 25% Gravel content 50% All soils σ'_{vo} in kPa
$D_r = 100\left[(N_1)_{60}/60\right]^{0.5}$	Normally consolidated, unaged sands — Kulhawy & Mayne (1990)
$D_r = 1.55N_1 + 40.0$	$0 \le N_1 \le 25$ — Hatanaka & Feng (2006)
$D_r = 0.84N_1 + 57.8$	$25 \le N_1 < 50$
$N_1 = N(98/\sigma'_v)^{0.5}$	
(σ'_v in kPa)	

10.1 Standard Penetration Test

(D) SPT Value, Density Index and ϕ

The results of the SPT may be useful in estimating the shear strength of granular soils by providing an estimate of the drained friction angle, φ'. It should be recognized that the value of φ' is not unique but depends on stress level, stress path, loading conditions, etc., and therefore, any estimate does not take these factors into account. A number of suggestions have been proposed by various researchers for evaluating φ' from SPT N-values. Direct correlations between N and φ' have been presented and are summarized in Table 7b. In most cases, the reference values for φ' have not been given.

10.1 Standard Penetration Test

Table 7b: Reported correlations between N and φ' for coarse-grained soils

Correlation	*References*
$\varphi' = (10N)/35 + 27^0$	Meyerhof (1956)
$\varphi' = (20N)^{0.5} + 15^0$	Kishida (1967)
$\varphi' = 3.5(N)^{0.5} + 20^0$	Muromachi et al. (1974)
$\varphi' = \left(N/\sigma'_{vo}\right)^{0.5} + 26.9^0$ $\left(\sigma'_{vo} \text{ in } MN/m^2\right)$	Parry (1977)
$\varphi' = (15N)^{0.5} + 15^0$	Shioi & Fukui (1982)
$\varphi' = \left[15.4(N_1)_{60}\right]^{0.5} + 20^0$	Kulhawy & Mayne (1990)
$\varphi' = (12N)^{0.5} + 23.7^0$	Bergado et al. (1993)
$\varphi' = (20N_1)^{0.5} + 20^0$ $N_1 = N/\left(\sigma'_{vo}/98\right)^{0.5}$ $\left(\sigma'_{vo} \text{ in kPa}\right)$	Hatanaka & Uchida (1996)

10.1 Standard Penetration Test

The following correlations may be considered reasonable. Relations are also presented graphically in Fig. 27.

$$\varphi^o = 27 + 0.3N_{60} + 0.00054{N_{60}}^2$$

$$\varphi^o = \sqrt{N_{60}} + 15$$

Relations are also presented graphically in Fig. 27.

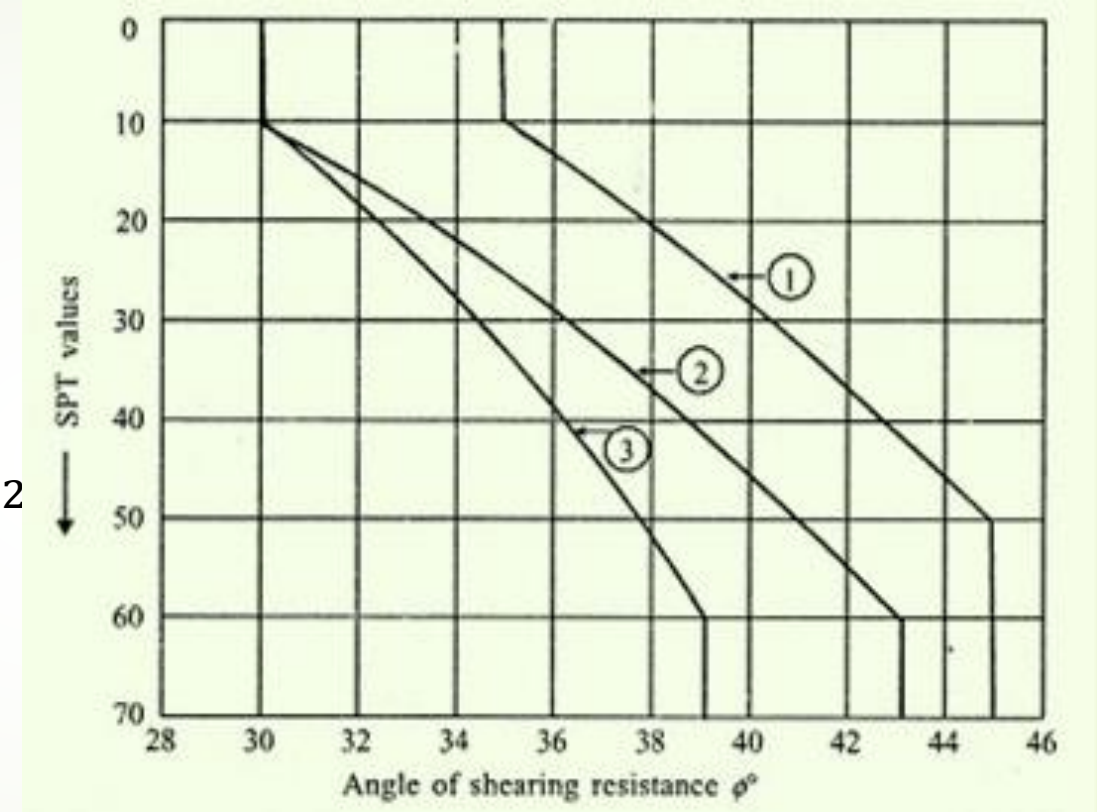


Fig. 27: SPT – φ relations for granular soils; (1) Well graded sand and gravel; (2) Uniform fine sand; and (3) Silty sand

10.1 Standard Penetration Test

(E) SPT Value and Consistency of Cohesive Soil

Terzaghi and Peck presented correlations between SPT value and consistency of cohesive soil, Table 8 and Fig. 28.

Table 8: SPT and Soil Properties (Cohesive soil: Rather Unreliable

N- Value	Soil Consistency	UC Strength (kN/m^2)
0 - 2	Very soft	< 25
2 -4	Soft	25 - 50
4 - 8	Medium	50 - 100
8 - 15	Stiff	100 - 200
15 - 30	Very stiff	200 - 400
> 30	Hard	> 400

10.1 Standard Penetration Test

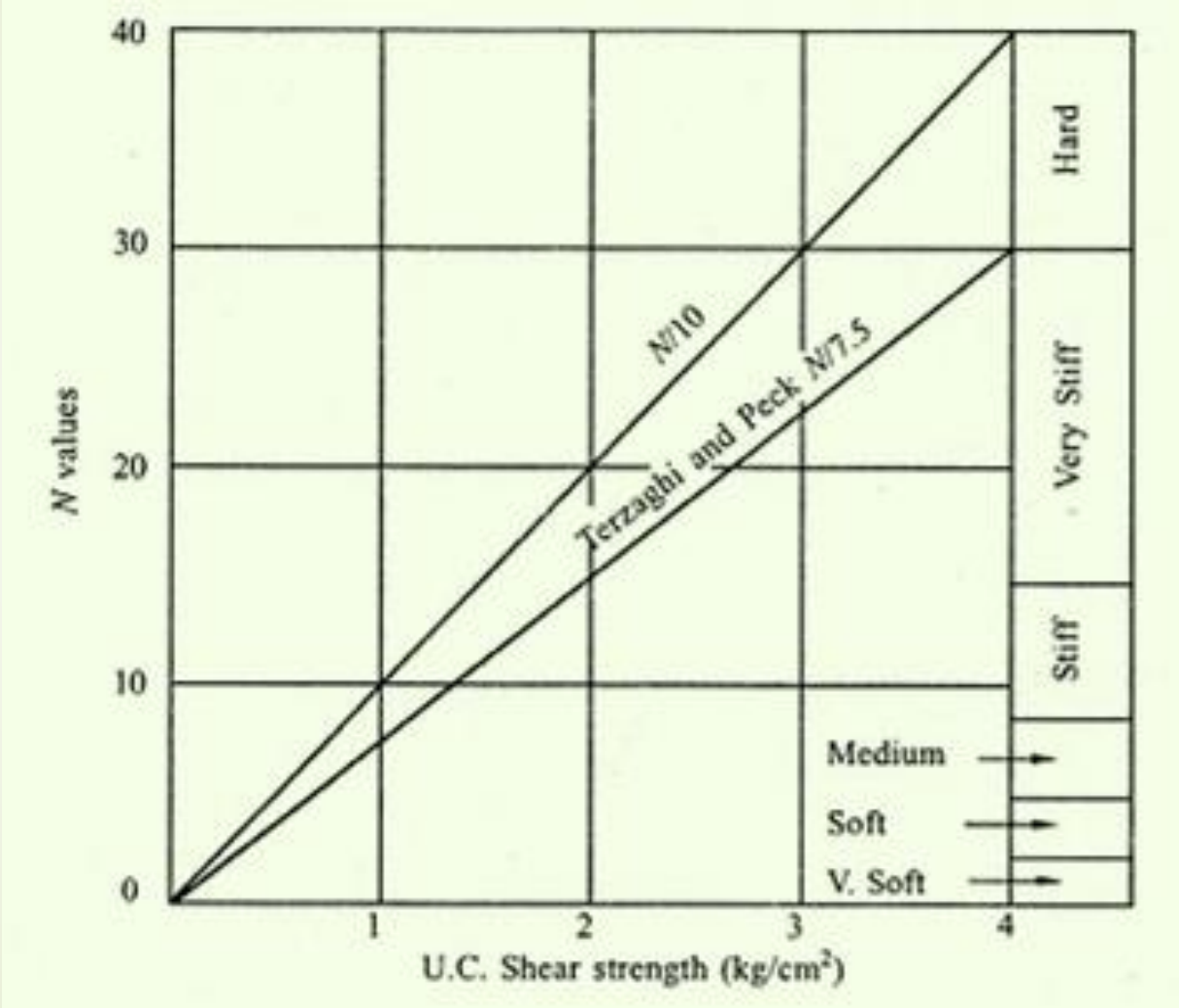


Fig. 28: SPT, consistency and unconfined compressive strength of clay soil

10.1 Standard Penetration Test

Even though, SPT values are not considered to be good measure of the strength of clays, yet it is being used extensively to assess the consistency of clays. One reasonable estimate can be as follows.

$$q_u(kN/m^2) = 12.5N_{60}$$

Where, q_u is the unconfined compressive strength. For highly plastic clays ($PI > 30$) the relation may be expressed as:

$$q_u(kN/m^2) = 10N_{60}$$

The correlation for clays is rather unreliable. Hence, vane shear test is recommended for more reliable information. However, as reported by Luteneggar (2021), various researchers suggested many relations and they are presented in Tables 8a, 8b and Fig. 28a.

10.1 Standard Penetration Test

Table 8a: Reported correlations between SPT N and undrained shear strength (1)

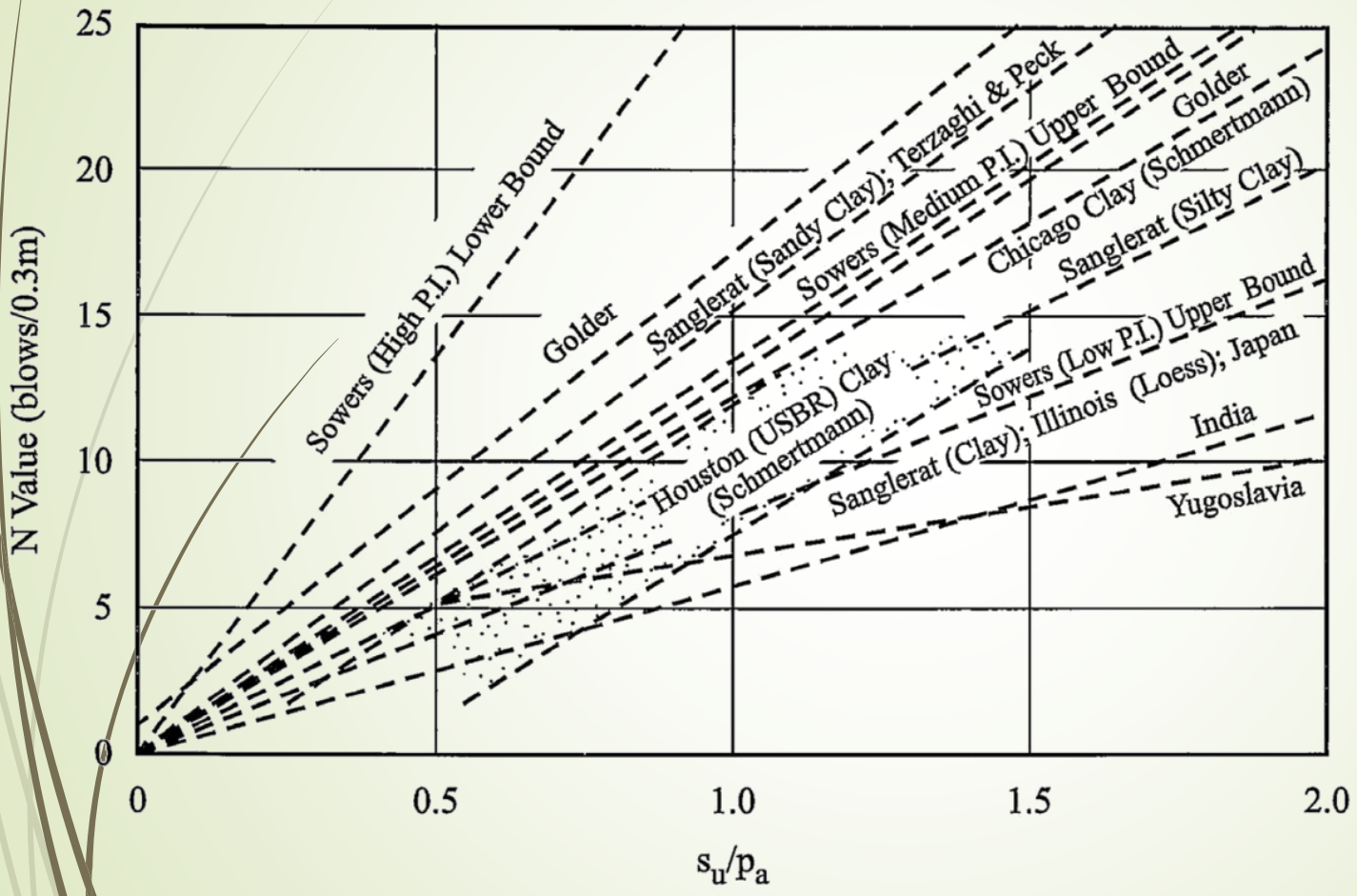
Correlation	*Units of s_u*	*Soil type*	*References*
$s_u = 29N^{0.72}$	kPa	Japanese cohesive soils	Hara et al. (1974)
$s_u/N_{60} = f(P.I.)$	kPa	Stiff UK soils	Stroud (1974)
$s_u = N/15$	Tsf	Stiff clays in Houston	Reese et al. (1976)
$s_u = 8N$ $N<10$ $s_u = 7N$ $10<N<20$ $s_u = 6N$ $20<N<30$ $s_u = 5N$ $30<N<40$	kPa	Guabirotuba Clay	Tavares (1988)
$s_u = 1.39N + 74.2$	kN/m^2	Tropical soil	Ajayi & Balogun (1988)
$s_u = 12.5N$ $s_u = 15N_{60}$	kPa	Sao Paulo Over-consolidated clay	Decourt (1989)
$s_u = 0.059N + 0.2$	tsf	Clay and soft shale	Nevels & Laguros (1993)
$s_u = 7.8N_{60}$ (CH) $s_u = 5.35N_{60}$ (CL)	kPa	Claysi – Turkey	Sivrikaya & Togrol (2006)
$s_u/p_a = 0.17N^{0.58}$	–	Ankara (Turkey) clay	Akbas & Kulhawy (2010)
$s_u = 2.1N_{60} + 17.6$	kPa	Iran clay	Nassaji & Kalantari (2011)
$s_u/N = 14.7 - 0.35w - 2.4 \log (PI)$ (w = water content in %) (PI = Plasticity Index in %)	kPa	2 clays in Greece	Plytas et al. (2011)
$s_u = 6.932N_{70}$	kN/m^2	Silty clays – Turkey	Cangir & Dipova (2017)
$s_u = 8.32N_{60}$	kPa	Stiff glacial till in Canada	Balachandran et al. (2017)
$S_u = 5.7N$	kN/m^2	London Clay	White et al. (2019)

10.1 Standard Penetration Test

Table 8b: Reported correlations between SPT N and unconfined compressive strength strength

Correlation	*Units of q_u*	*Soil type*	*References*
$q_u = 12.5N$	kPa	Fine-grained	Terzaghi & Peck (1967)
$q_u = N/8$	tsf	Clay	Golder (1961)
$q_u = 25N$	kPa	Clay	Sanglerat (1972)
$q_u = 20N$	kPa	Silty clay	
$q_u = 25N$	kPa	Highly plastic clay	Sowers (1979)
$q_u = 15N$		Medium plastic clay	
$q_u = 7.5N$		Low plasticity clay	
$q_u = 24N$	kPa	Clay	Nixon (1982)
$q_u = 62.5\ (N-3.4)$	kPa		Sarac & Popovic (1982)
$q_u = 1.37N$	t/m^2	CH Bangkok clay	Sambhandharaksa & Pitupakorn (1985)
$q_u = 1.04N$		CL Bangkok clay	
$q_u = 15N$	kPa	CL and CL-ML	Behpoor & Ghahramani (1989)
$q_u = 58N^{0.72}$	kPa	Fine-grained	Kulhawy & Mayne (1990)
$q_u = 14.3N$ (LL≤35)	kPa	Bangladesh clays	Serajuddin & Chowdhury (1996)
$q_u = 16.9N$ (LL 36–50)			
$q_u = 17.8N$ (LL>50)			
$q_u = 13.6\ N_{60}$	kPa	CH	Sivrikaya & Togrol (2002)
$q_u = 9.8N_{60}$		CL	
$q_u = 8.6N_{60}$		Fine-grained	
$q_u = (0.19P.I. + 6.2)N_{60}$		Fine-grained	

10.1 Standard Penetration Test



Pa is atmospheric pressure = 100 kPa

Fig. 28s: SPT, consistency and unconfined compressive strength of clay soil suggested by various authorities.

10.1 Standard Penetration Test

For Bangladesh soil, Serajuddin and Chowdhury suggested the following relationships between SPT N-value and unconfined compressive strength of clay soil.

$$q_u(kN/m^2) = 14.3N \qquad [LL \leq 35]$$

$$q_u(kN/m^2) = 16.9N \qquad [36 \leq LL \leq 50\,]$$

$$q_u(kN/m^2) = 17.8N \qquad [LL > 50]$$

10.2 Dynamic Cone Penetration Test

The dynamic cone penetration test can be considered only as a variation of SPT. Instead of the spoon sampler used for SPT, a special solid cone of 60 degree and 50 mm diameter is used as a penetrometer. This probe can be used either in the borehole as an SPT test or without a borehole as a continuous penetration test. The latter may be of the recoverable type or the expendable type.

Experience shows that in most soils, the solid cone penetrometer test tends to give a slightly higher value than SPT. These tests are to be used along with SPT tests so that a correlation between the two can be worked out for each site under investigation.

10.2 Dynamic Cone Penetration Test

It is then used to determine the nature of deposits in other locations at the same site without putting an expensive borehole. As this test is very much cheaper than SPT tests in boreholes, a large number of dynamic cone penetration tests can be made at various locations at nominal cost along with SPT tests. The blow count for every 100 mm penetration due to a 65 kg weight falling through 750 mm is taken. The total blows for one foot (300 mm) penetration is the dynamic cone value N_{dc}.

To save the equipment from damage, driving may be stopped when the number of blows exceeds 35 for 100 mm penetration. Typical cone used dynamic cone penetration test is shown in Fig. 29.

10.2 Dynamic Cone Penetration Test

Dynamic cone test can be used with or without bentonite (mud) slurry. But when depth of investigation is more than 6 m, use of bentonite or mud slurry is recommended as otherwise friction on the rods would be tremendous. Data from dynamic cone test is plotted as a curve of penetration resistance, N_{dc} number of blows per 30 cm of penetration, versus depth.

The N_{dc} values from dynamic cone penetration tests needed to be corrected for overburden pressure in cohesionless soil like N-values of SPT.

10.2 Dynamic Cone Penetration Test

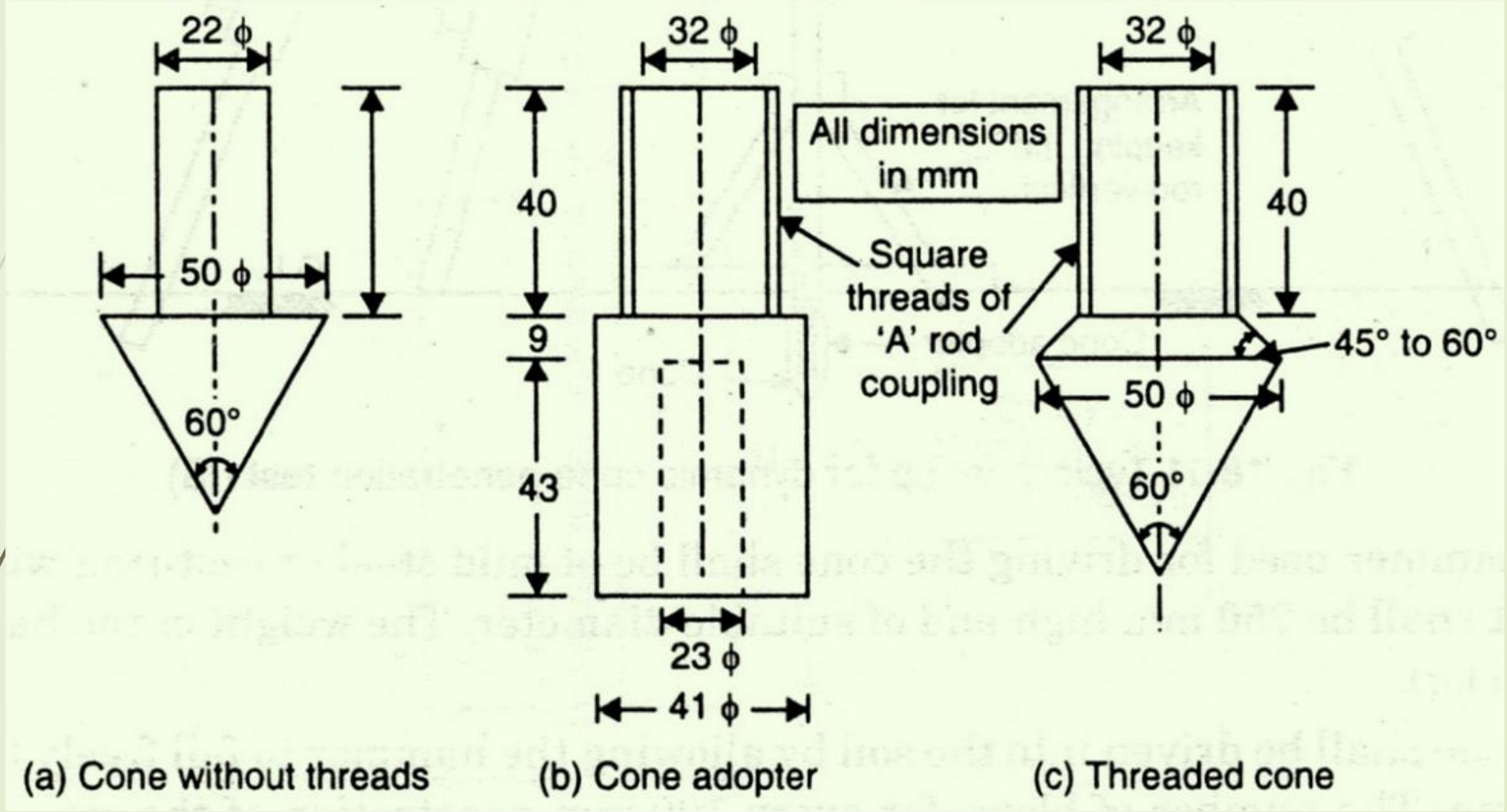


Fig. 29: Typical cone details for dynamic cone penetration test

10.2 Dynamic Cone Penetration Test

Dynamic probes come in a variety of cone sizes, hammer weights and hammer drops, but modern probes are generally the heavy probe (DPH) and the super-heavy (DPSH) probe. Results are typically expressed as blows per 100 mm.

The super-heavy probe has the same dimensions as an SPT, and interpretation is the same as for SPT data, simply by combining results for 100 mm penetration to obtain the number of blows for each 300 mm penetration. Alternatively, the blows for each 100 mm penetration can be multiplied by 3.

10.2 Dynamic Cone Penetration Test

For other size probes, an equivalent SPT N-value may be obtained for granular soils by assuming that penetration is proportional to the energy per blow (hammer mass × fall) and inversely proportional to the cone area. That is:

$$SPT\ N\ Value = \frac{M_p F_p A_s P_s}{M_s F_s A_p P_p} \times probe\ blow$$

Where, M_p and F_p are the mass and fall, respectively, of the probe monkey (the falling weight that drives the probe), A_p is its end area; and P_p is the penetration for each blow count for the probe; and M_s and F_s are the mass and fall, respectively, of the SPT monkey, A_s is its end area; and P_s is the penetration for each blow count for the SPT.

10.2 Dynamic Cone Penetration Test

For SPT, $M_s = 65kg$, $F_s = 0.76\,m$, $A_s = \pi \times \frac{502}{4} = 1960\,mm^2$ and $P_s = 300\,mm$.

For DHP, $M_s = 50\,kg$, $F_s = 0.50$, $A_s = \pi \times \frac{43.72}{4} = 1500\,mm^2$ and $P_p = 100\,mm$.

Substituting these values into the Equation of SPT gives the equivalence:

$$SPT\ N\ Value\ (blows/300\,mm) \approx 2 \times DPH\ probe\ blow\ (blows/100\,mm)$$

In clays, there is no real equivalence between probe blows and SPT blows. Probe results may however be used in conjunction with correlations with specific soil properties.

10.2 Dynamic Cone Penetration Test

The static cone penetration test normally called the Dutch cone penetration test (CPT) has gained acceptance rapidly in many countries. The method was introduced nearly 50 years ago. This test is widely used in Holland, Belgium, Britain, Indonesia, Malaysia, Singapore, West Indies etc.

It is finding increasing use in India. The equipment consists essentially of a steel cone with an apex angle of 60° and overall base diameter of 35.7 mm giving a cross sectional area of 10 sq.cm. The cone is attached to rod which is in turn connected to other rods as necessary. These rods are protected by sleeves known as mantle tubes.

10.2 Dynamic Cone Penetration Test

Immediately above the cone a friction jacket, of outside diameter greater than mantle tube, is fitted. The cone and the friction jacket in combination or separately are pushed into the ground by hydraulic cylinder of a machine of capacities presently varying from 20 kN to 100 kN.

The necessary reaction is obtained by anchors and sometimes by surcharge loading. One of the greatest values of the CPT consists of its function as a scale model pile test. Empirical correlations established over many years permit the calculation of pile bearing capacity directly from the CPT results without the use of conventional soil parameters.

10.3 Static Cone Penetration Test (CPT)

The CPT is valuable for soil profiling as the soil type can be identified from the combined measurement of end resistance of cone and side friction on a jacket. The test lends itself to the derivation of normal soil properties such as density, friction angle and cohesion. Various theories have been developed for foundation design. The popularity of the CPT can be attributed to the following three important factors:

- General introduction of the electric penetrometer providing more precise measurements, and improvements in the equipment allowing deeper penetration.
- The need for the penetrometer testing in-situ technique in offshore foundation investigations in view of the difficulties in achieving adequate sample quality in marine environment.
- The addition of other simultaneous measurements to the standard friction penetrometer such as pore pressure and soil temperature.

10.3 Static Cone Penetration Test (CPT)

(A) The Penetrometer

There are a variety of shapes and sizes of penetrometers being used, Figs. 30. The one that is standard in most countries is the cone with an apex angle of 60° and a base area of 10 cm^2. The sleeve (jacket) has become a standard item on the penetrometer for most applications. On the 10 cm^2 cone penetrometer, the friction sleeve should have an area of 150 cm^2 as per standard practice. The ratio of side friction and bearing resistance, the friction ratio, enables identification of the soil type (Schmertmann 1975).

It provides useful information when no borehole data are available. Even when borings are made, the friction ratio supplies a check on the accuracy of the boring logs.

10.3 Static Cone Penetration Test (CPT)



Fig. 30(a): Assortment of cones used for cone penetration test

10.3 Static Cone Penetration Test (CPT)

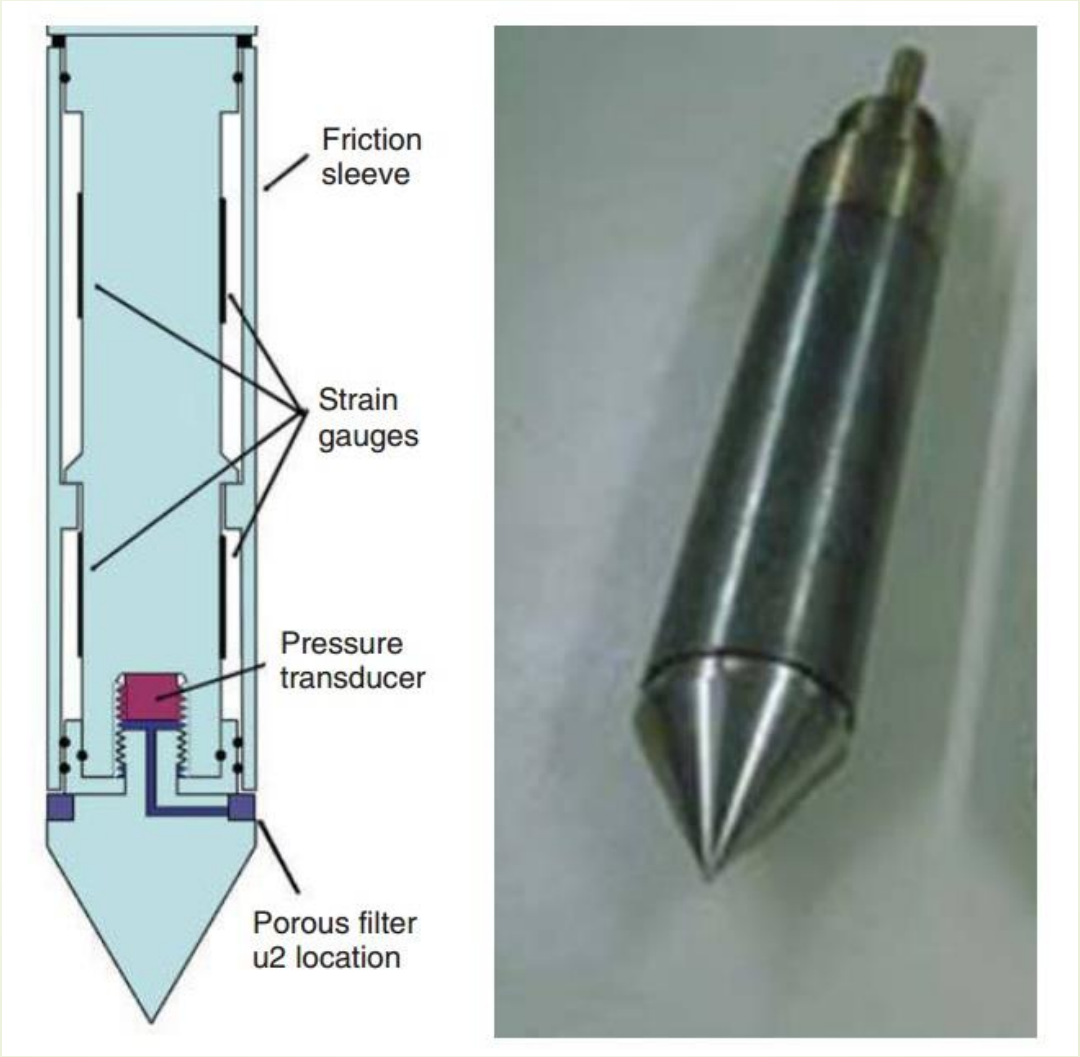


Fig. 30(b): Cone penetrometer.

10.3 Static Cone Penetration Test (CPT)

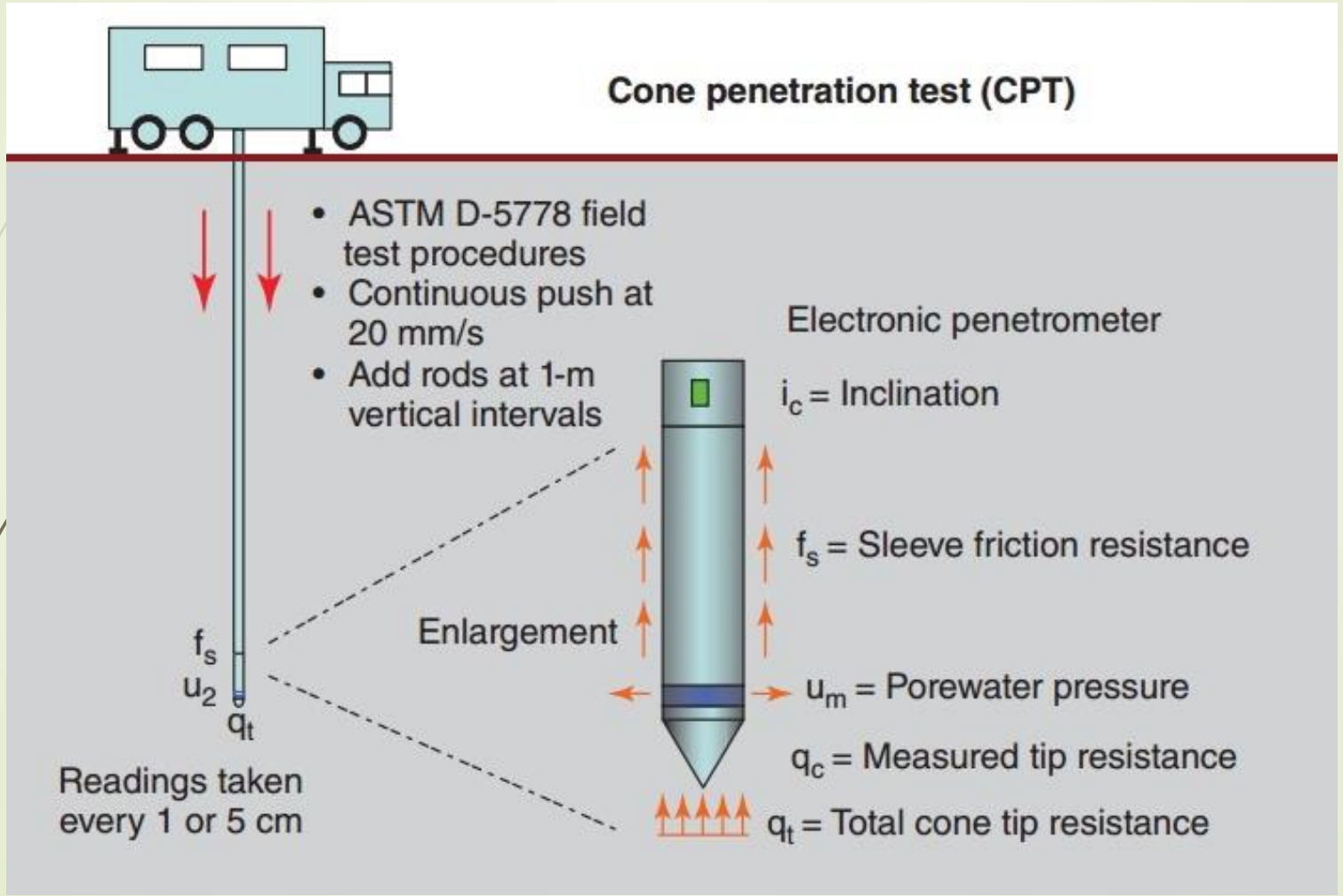


Fig. 30(c): Cone penetration test

10.3 Static Cone Penetration Test (CPT)

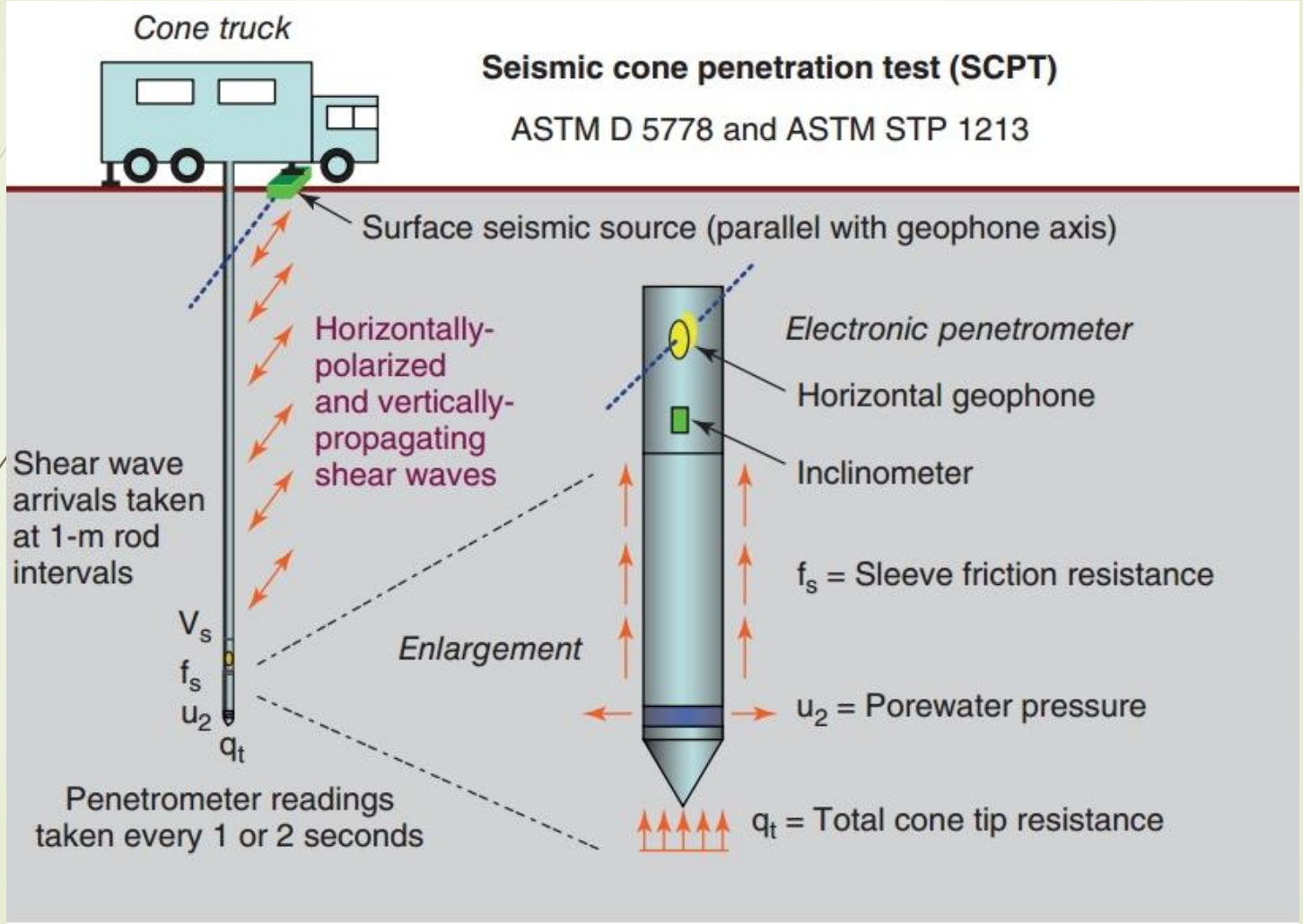


Fig. 30(d): Cone penetration test

10.3 Static Cone Penetration Test (CPT)

Two types of penetrometers are used for measuring cone resistance and friction. They are:

- The Mechanical Type
- The Electrical Type(B) Mechanical Penetrometer

The Begemann Friction Cone Mechanical type penetrometer is shown in Fig. 31. It consists of a 60° cone with a base diameter of 35.6 mm (sectional area 10 cm^2). A sounding rod is screwed to the base. Additional rods of one metre length each are used. The rods are screwed or attached together to bear against each other. The sounding rods move inside mantle tubes. The inside diameter of the mantle tube is just sufficient for the sounding rods to move freely, the outside diameter being is equal to or less than the base diameter of cone.

10.3 Static Cone Penetration Test (CPT)

(B) Mechanical Penetrometer

The Begemann Friction Cone Mechanical type penetrometer is shown in Fig. 31. It consists of a 60° cone with a base diameter of 35.6 mm (sectional area 10 cm^2). A sounding rod is screwed to the base. Additional rods of one metre length each are used. These rods are screwed or attached together to bear against each other. The sounding rods move inside mantle tubes. The inside diameter of the mantle tube is just sufficient for the sounding rods to move freely whereas the outside diameter is equal to or less than the base diameter of the cone.

10.3 Static Cone Penetration Test (CPT)

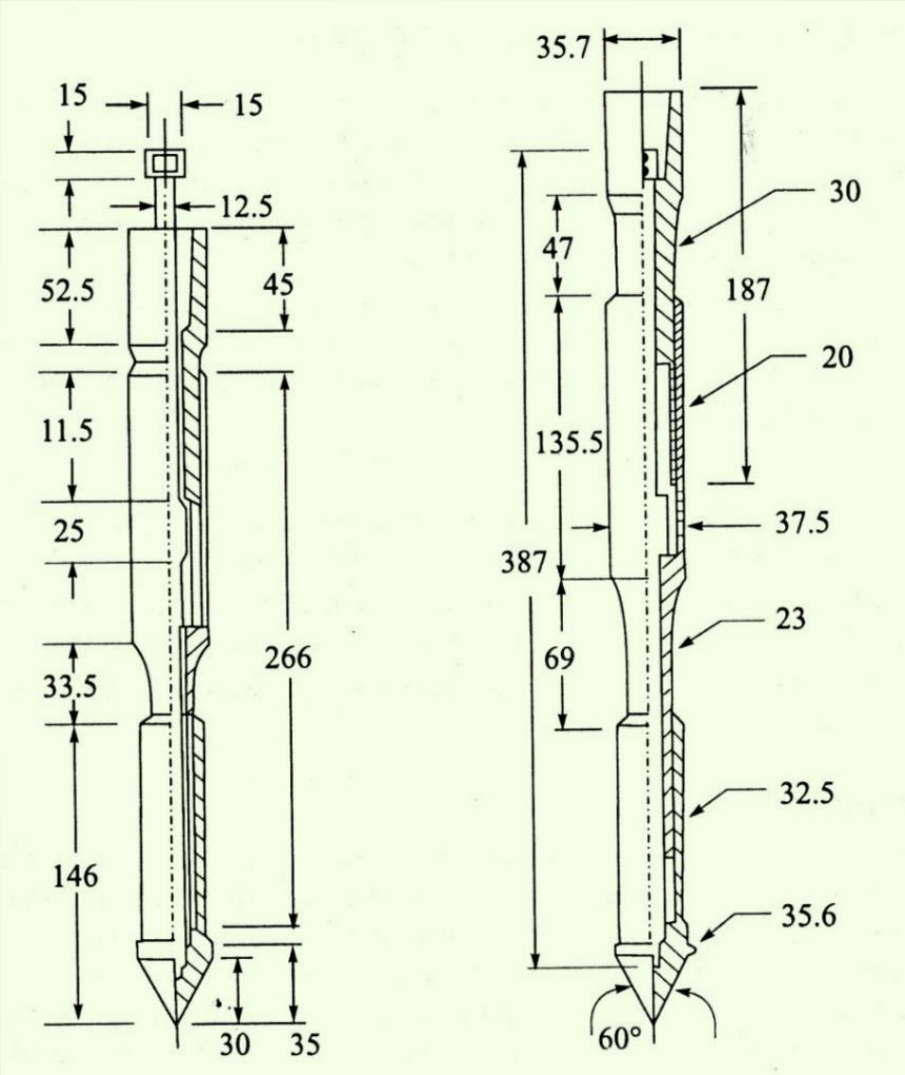


Fig. 31: Begeman friction cone mechanical type penetrometer (all dimensions are in mm)

10.3 Static Cone Penetration Test (CPT)

(C) Jacking System

The rigs used for pushing down the penetrometer consist basically of a hydraulic system. The thrust capacity for cone testing on land varies from 20 to 30 kN for hand operated rigs and 100 to 200 kN for mechanically operated rigs as shown in Fig. 32.

Bourden gauges are provided in the driving mechanism for measuring the pressures exerted by the cone and friction jacket either individually or collectively during the operation. The rigs may be operated either on the ground or mounted on heavy duty trucks. In either case, the rig should take the necessary up thrust. For ground based rigs screw anchors are provided to take up the reaction thrust.

10.3 Static Cone Penetration Test (CPT)

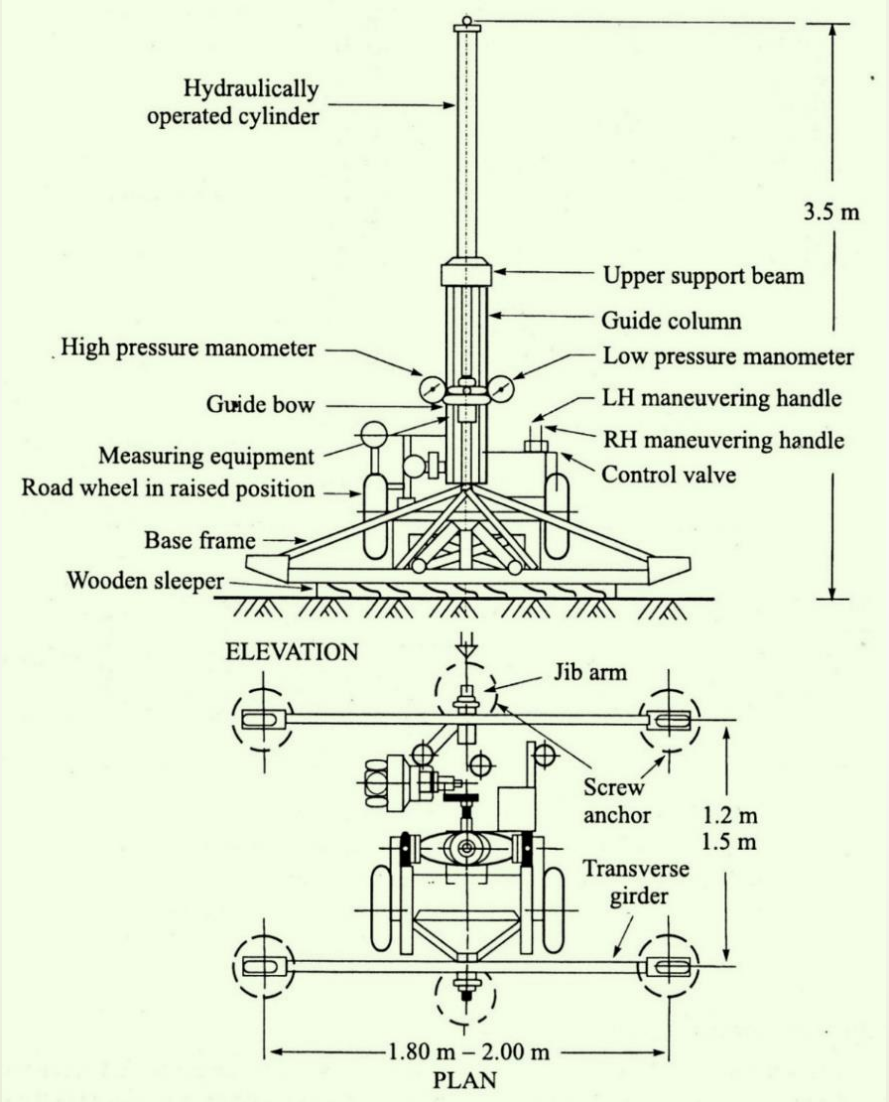


Fig. 32: Assembly of cone penetration rig

10.3 Static Cone Penetration Test (CPT)

(C) Operation of Penetrometer

The sequence of operation of the penetrometer shown in Fig. 33 is explained as follows:

Position 1: The cone and friction jacket assembly in a collapsed position.

Position 2: The cone is pushed down by the inner sounding rods to a depth until a collar engages the cone. The pressure gauge records the total force Q_c to the cone. Normally, a = 40 mm.

Position 3: The sounding rod is pushed further to a depth b. This pushes the friction jacket and the cone assembly together; the force is Q_t. Normally, b=40 mm.

10.3 Static Cone Penetration Test (CPT)

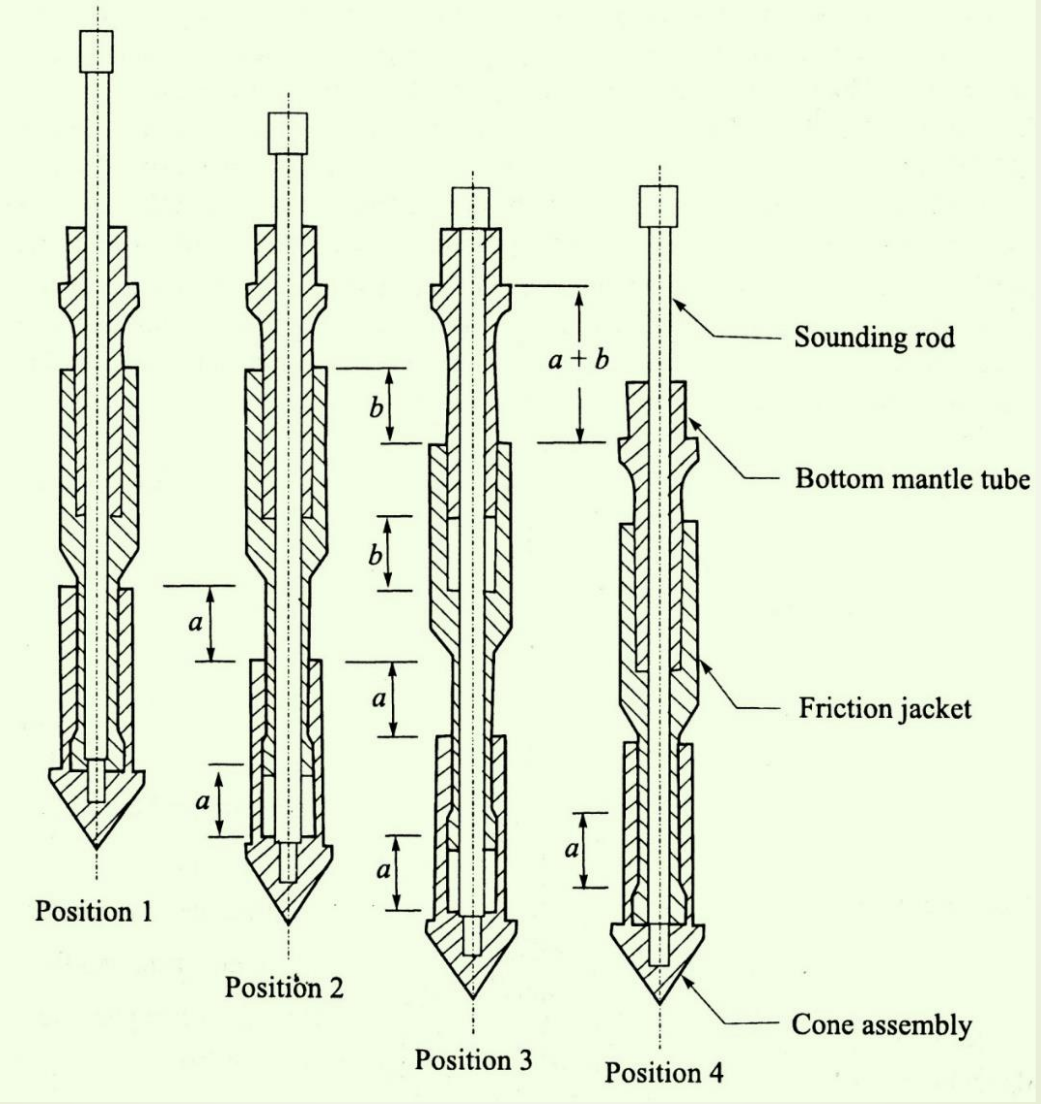


Fig. 33(a): Steps in cone penetration testing

10.3 Static Cone Penetration Test (CPT)

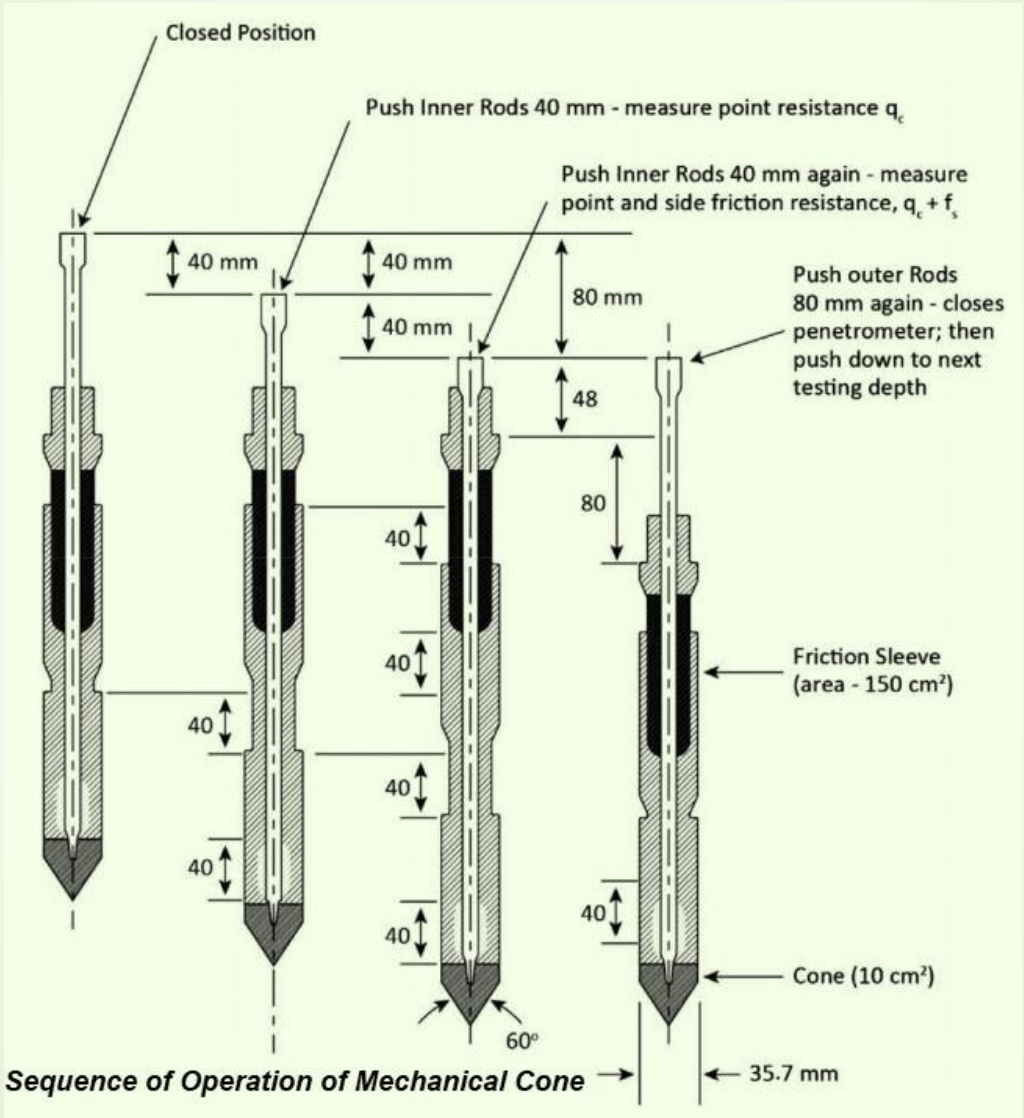


Fig. 33(b): Steps in cone penetration testing

10.3 Static Cone Penetration Test (CPT)

Position 4: The outside mantle tube is pushed down a distance (a+b) which brings the cone assembly and the friction jacket to position 1. The total movement = a + b = 80 mm.

The process of operation illustrated is continued until the proposed depth is reached. The cone is pushed at a standard rate of 20 mm per second. The mechanical penetrometer has its advantage as it is simple to operate and the cost of maintenance is low. The quality of the work depends on the skill of the operator. The depth of CPT is measured by recording the length of the sounding rods that have been pushed into the ground.

10.3 Static Cone Penetration Test (CPT)

(D) The Electric Penetrometer

The electric penetrometer is an improvement over the mechanical one. Mechanical penetrometers operate incrementally whereas the electric penetrometer is advanced continuously.

Fig. 34 shows an electric-static penetrometer with the friction sleeve just above the base of the cone. The sectional area of the cone and the surface area of the friction jacket remain the same as those of a mechanical type. The penetrometer has built in load cells that record separately the cone bearing and side friction. Strain gauges are mostly used for the load cells. The load cells have a normal capacity of 50 to 100 kN for end bearing and 7.5 to 15 kN for side friction, depending on the soils to be penetrated.

10.3 Static Cone Penetration Test (CPT)

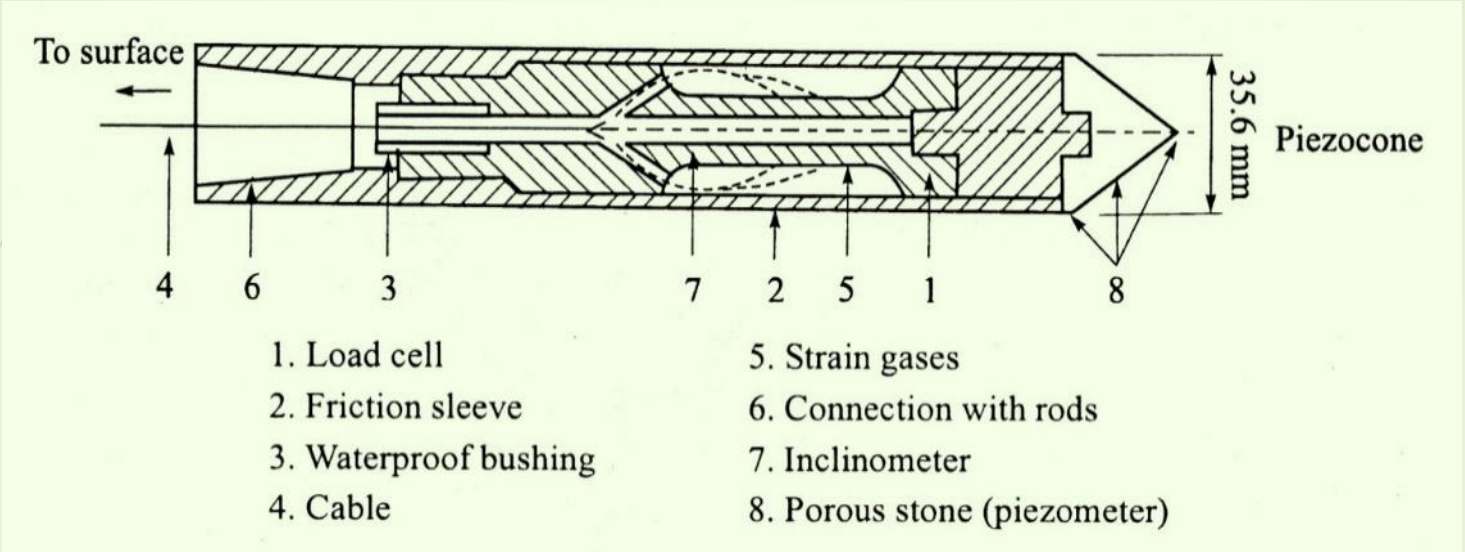


Fig. 34: An electrical static cone penetrometer

An electric cable inserted through the push rods (mantle tube) connects the penetrometer with the recording equipment at the surface which produces graphs of resistance versus depth. The electric penetrometer has many advantages. The repeatability of the cone test is very good.

10.3 Static Cone Penetration Test (CPT)

A continuous record of the penetration results reflects better the nature of the soil layers penetrated. However, electronic cone testing requires skilled operators and better maintenance. The electric penetrometer is indispensable for offshore soil investigation.

(E) Operation of Electric Penetrometer

The electric penetrometer is pushed continuously at a standard rate of 20 mm per second. A continuous record of the bearing resistance q_c and frictional resistance f_c against depth is produced in the form of a graph at the surface in the recording unit.

10.3 Static Cone Penetration Test (CPT)

(F) Piezocone

A piezometer element included in the cone penetrometer is called a piezocone, Fig. 35. There is now a growing use of the piezocone for measuring pore pressures at the tips of the cone. The porous element is mounted normally midway along the cone tip allowing pore water to enter the tip. An electric pressure transducer measures the pore pressure during the operation of the CPT. The pore pressure record provides a much more sensitive means to detect thin soil layers. This could be very important in determining consolidation rates in a clay soil within the sand seams.

10.3 Static Cone Penetration Test (CPT)

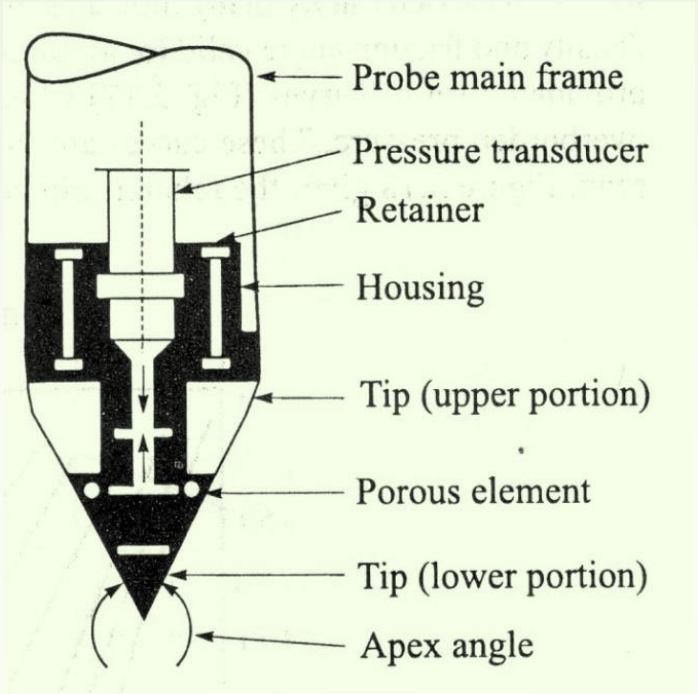


Fig. 35: Details of piezocone

(G) Temperature Cone

The Temperature of Soil Is required at certain localities to provide information about environmental changes.

10.3 Static Cone Penetration Test (CPT)

The temperature gradient with depth may offer possibilities to calculate the heat conductivity of the soil. Measurement of the temperature during CPT is possible by incorporating a temperature sensor in the electric penetrometer. Temperature measurements have been made in permafrost, under blast furnaces, beneath undercooled tanks, along marine pipe lines, etc.

(H) Effect of Rate of Penetration

Several studies have been made to determine the effect of the rate of penetration on cone bearing and side friction.

10.3 Static Cone Penetration Test (CPT)

Although the values tend to decrease for slower rates, the general conclusion is that the influence is insignificant for speeds between 10 and 30 mm per second. The standard rate of penetration has been generally accepted as 20 mm per second.

(I) Cone Resistance Q_c and Local Side Friction F_c

Cone penetration resistance q_c is obtained by dividing the total force Q_c acting on the cone by the base area A_c of the cone.

$$q_c = \frac{Q_c}{A_c} \qquad (6a)$$

10.3 Static Cone Penetration Test (CPT)

In the same way, the local side friction f_c is:

$$f_c = \frac{Q_f}{A_f} \qquad (6b)$$

Where,

$Q_f = Q_t - Q_c =$ force required to push the friction jacket,

$Q_t =$ the total force required to push the cone and friction jacket together in the case of a mechanical penetrometer,

$A_f =$surface area of the friction jacket.

10.3 Static Cone Penetration Test (CPT)

(J) Friction Ratio, R_f

Friction ratio, R_f is expressed as:

$$R_f = \frac{f_c}{q_c} \qquad (7)$$

Where, f_c and q_c are measured at the same depth. R_f is expressed as a percentage. Friction ratio is an important parameter for classifying soil.

(K) Relationships of Cone Resistance, q_c Relative Density, D_r and Friction Angle, φ of Sand

Research carried out by many indicates that a unique relationship between cone resistance, relative density and friction angle valid for all sands does not exist.

10.3 Static Cone Penetration Test (CPT)

Robertson and Campanella (1983a) have provided a set of curves (Fig. 36) which may be used to estimate D_r based on q_c and effective overburden pressure. These curves are supposed to be applicable for normally consolidated clean sand. Fig. 37 gives the relationship between $q_c$and φ (Robertson and Campanella, 1983b).

(L) Relationship between q_c and Undrained Shear Strength, c_u of Clay

Cone penetration resistance q_c and c_u may be related as:

$$q_c = N_k c_u + p_o \quad or, c_u = \frac{q_c - p_o}{N_k} \qquad (8)$$

Where, $N_k$cone factor, $p_o = \gamma h$ = overburden pressure.

10.3 Static Cone Penetration Test (CPT)

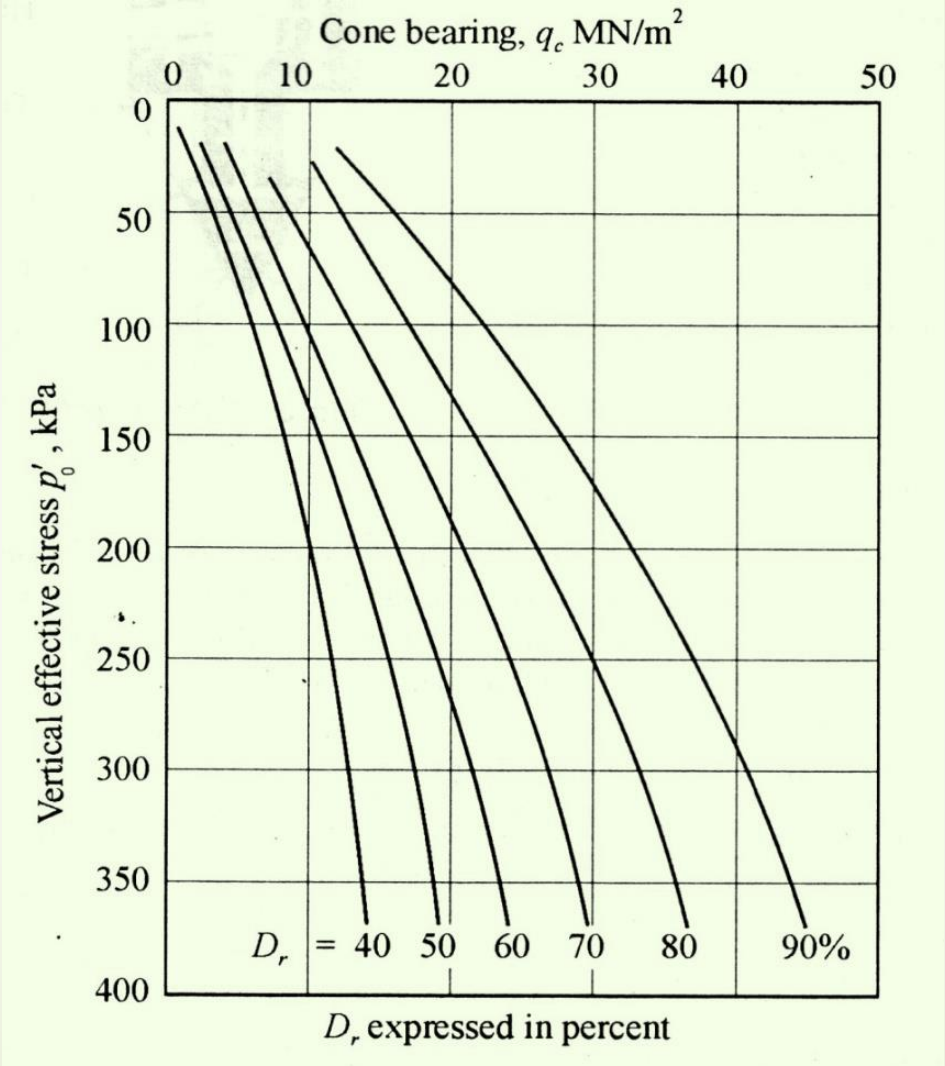


Fig. 36: Relationship between relative density, D_r and cone resistance , q_c for uncemented quartz sand

10.3 Static Cone Penetration Test (CPT)

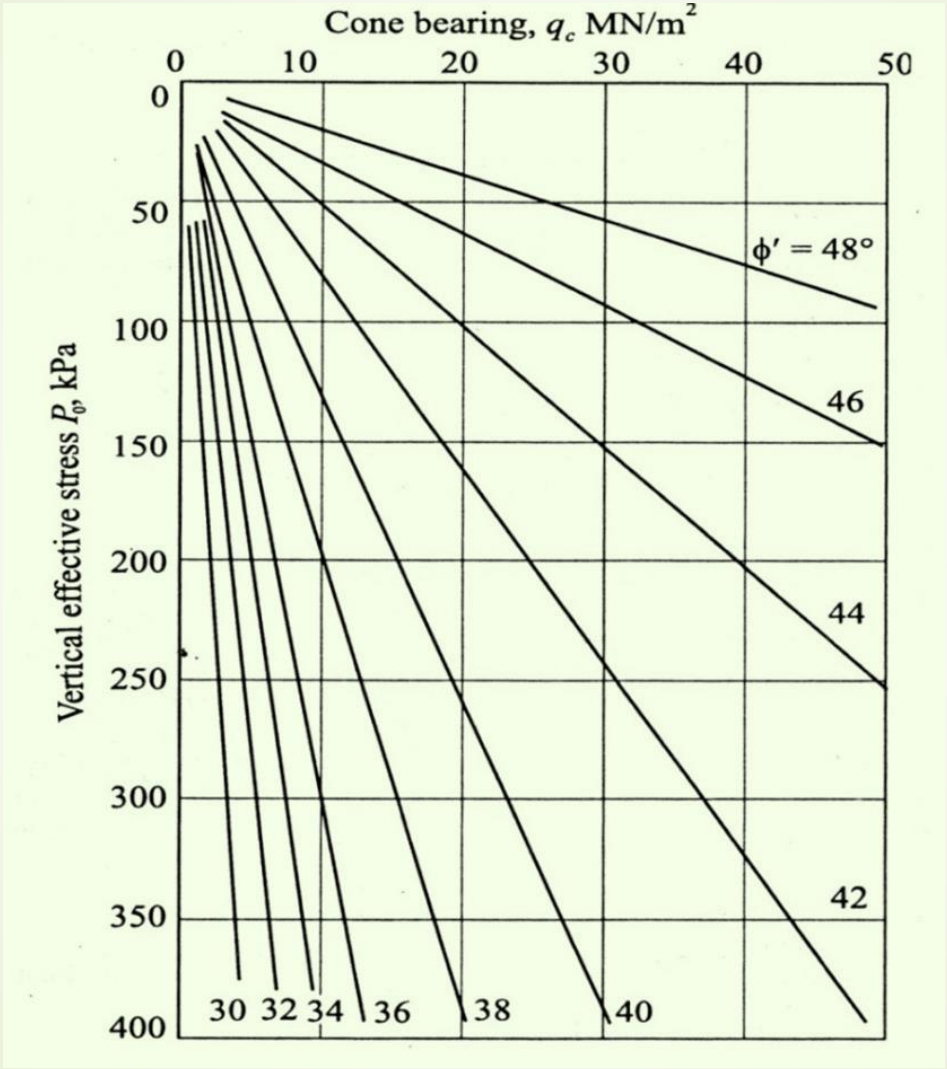


Fig. 37: Relationship between cone resistance, q_c and vertical effective stress, for uncemented quartz sand

10.3 Static Cone Penetration Test (CPT)

Rune and Kelven (1981), investigated the value of the cone factor N_k for both normally consolidated and over-consolidated clays. The values of N_k as obtained are given below in Table 9. Possibly a value of 20 for N_k for both types of clay may be satisfactory. Sanglerat (1972) recommends the same value for all cases where an overburden correction is of negligible value.

Table 9: Soil Type and q_c Value

Type of clay	Cone factor, N_k
Normally consolidated	10 - 19
Over-consolidated:	
At shallow depth	15 - 20
At depth	12 - 18

10.3 Static Cone Penetration Test (CPT)

(M) Soil Classification

One of the basic uses of CPT is to identify and classify soils. A CPT - Soil Behaviour Type Prediction System has been developed by Douglas and Olsen (1981) using an electric-friction cone penetrometer. The classification is based on the friction ratio f_c/q_c . The ratio f_c/q_c varies greatly depending on whether it applies to clays or sands. Their findings have been confirmed by hundreds of tests.

For clay soils, it has been found that the friction ratio decreases with increasing liquidity index, I_L. Therefore, the friction ratio is an indicator of the soil type penetrated. It permits approximate identification of soil type though no samples are recovered.

10.3 Static Cone Penetration Test (CPT)

Douglas (1984), presented a simplified classification chart shown in Fig. 38. His chart uses normalized cone resistance q_{cn} for overburden pressure using the equation:

$$q_{cn} = q_c(1 - 1.25p'_o) \qquad (9)$$

Where, p'_o = effective overburden pressure in tsf, and q_c = cone resistance in tsf.

In conclusion, CPT data provide a repeatable index of the aggregate behavior of in-situ soil. The CPT classification method provides a better picture of overall subsurface conditions than is available with most other methods of exploration. Typical sounding logs are given in Figs. 39(a), (b) and (c).

10.3 Static Cone Penetration Test (CPT)

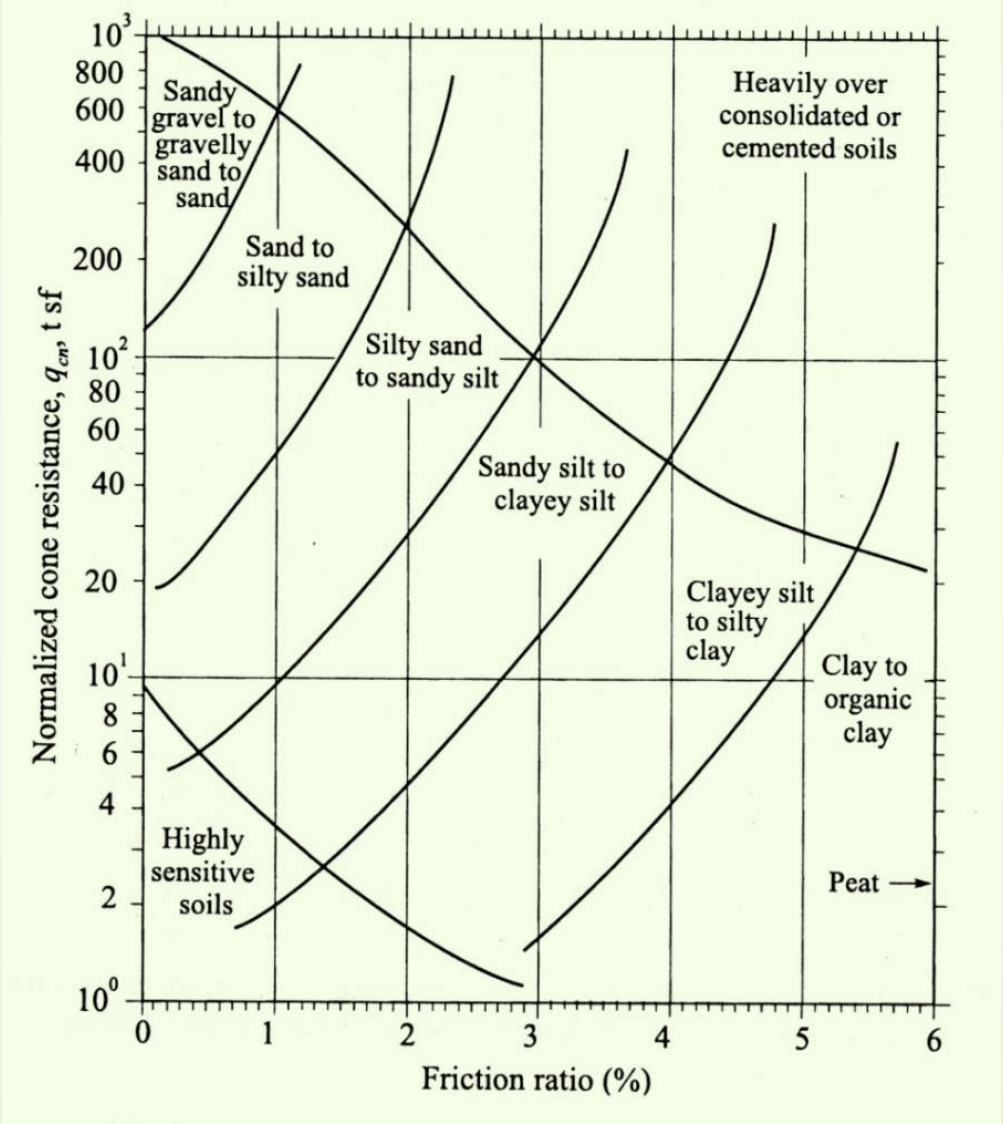


Fig. 38: Simplified classification chart of soil for CPT (Douglus, 1984)

10.3 Static Cone Penetration Test (CPT)

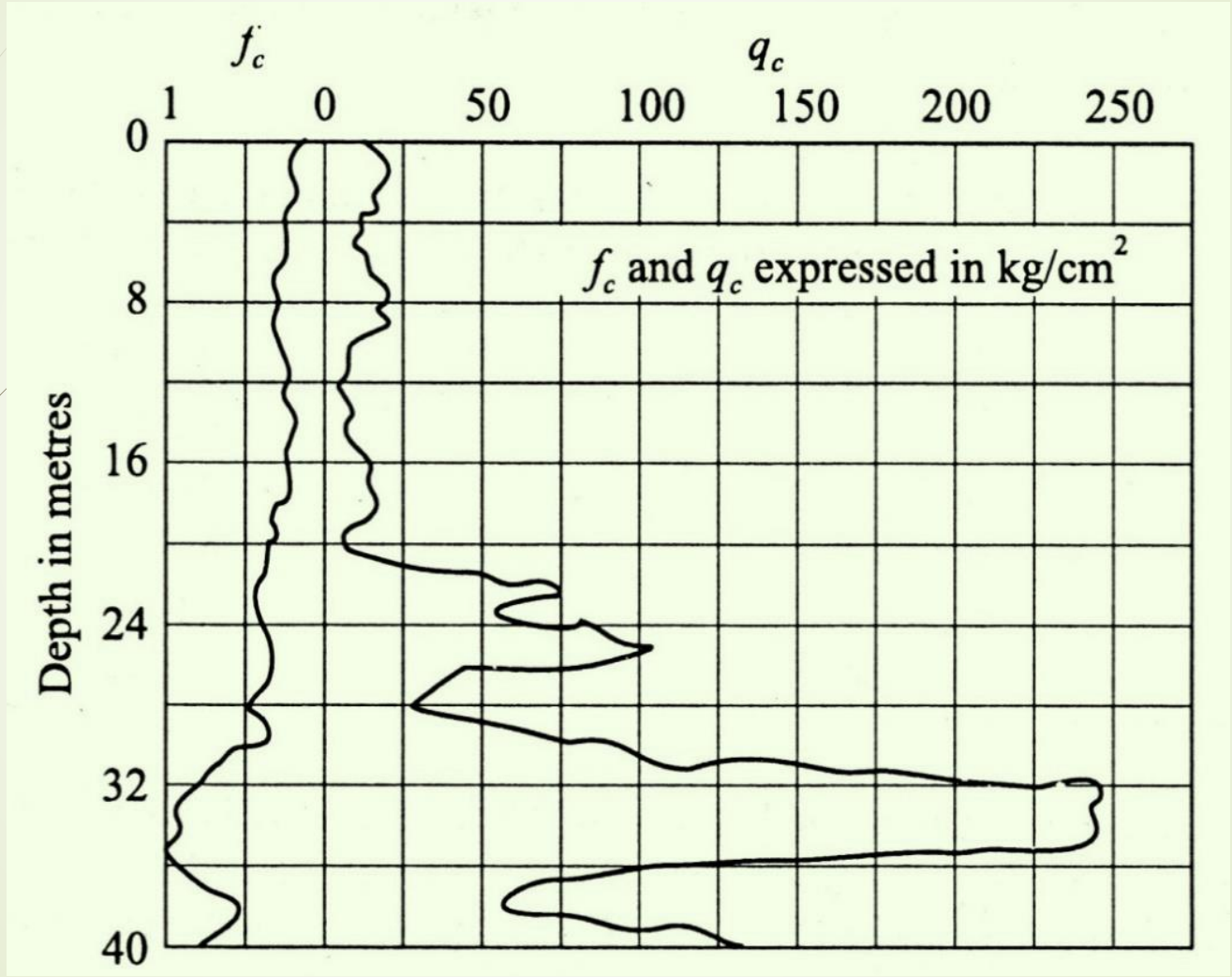


Fig. 39 (a): Typical sounding log

10.3 Static Cone Penetration Test (CPT)

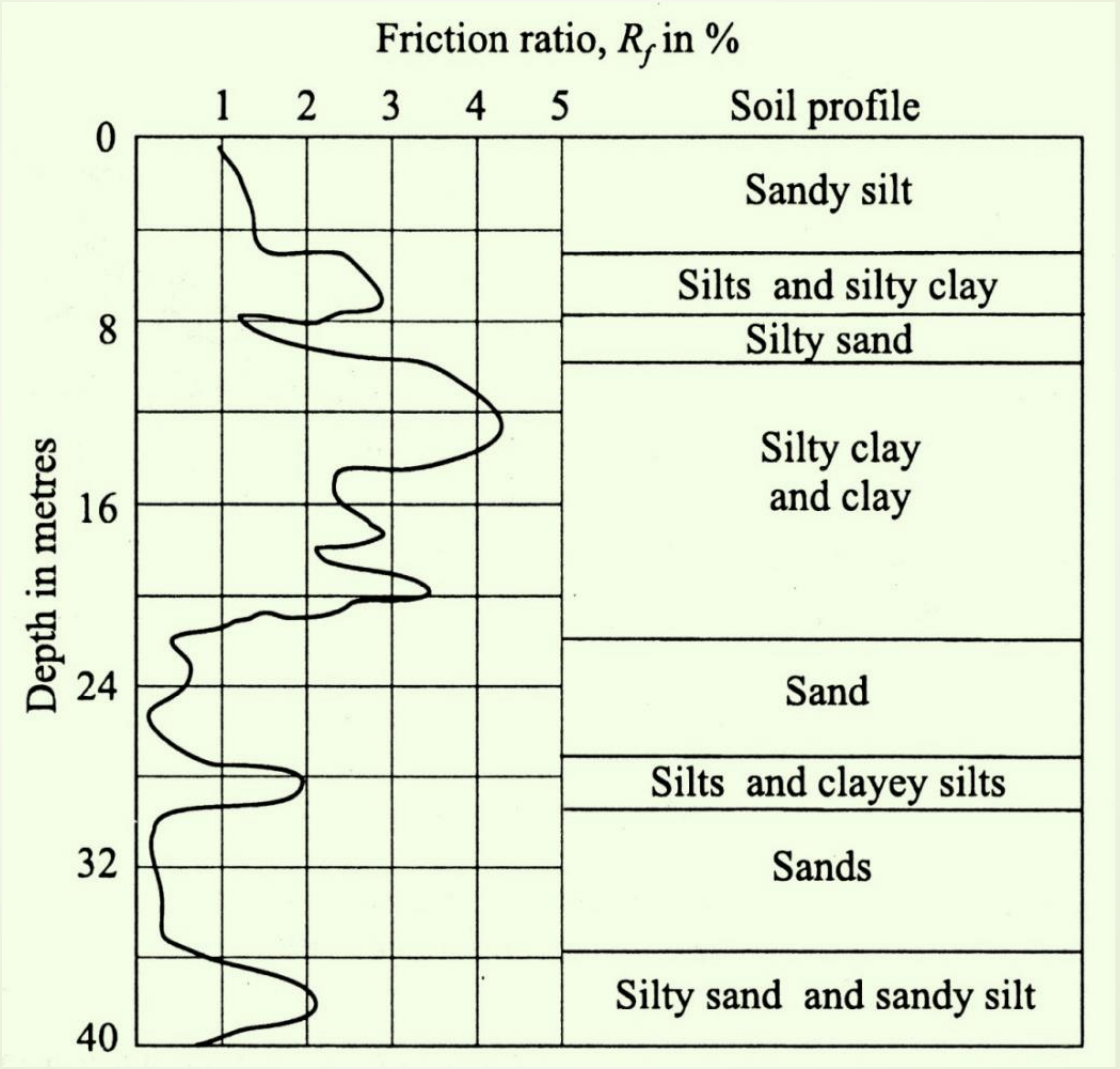


Fig. 39 (b): Typical sounding log showing soil layering

10.3 Static Cone Penetration Test (CPT)

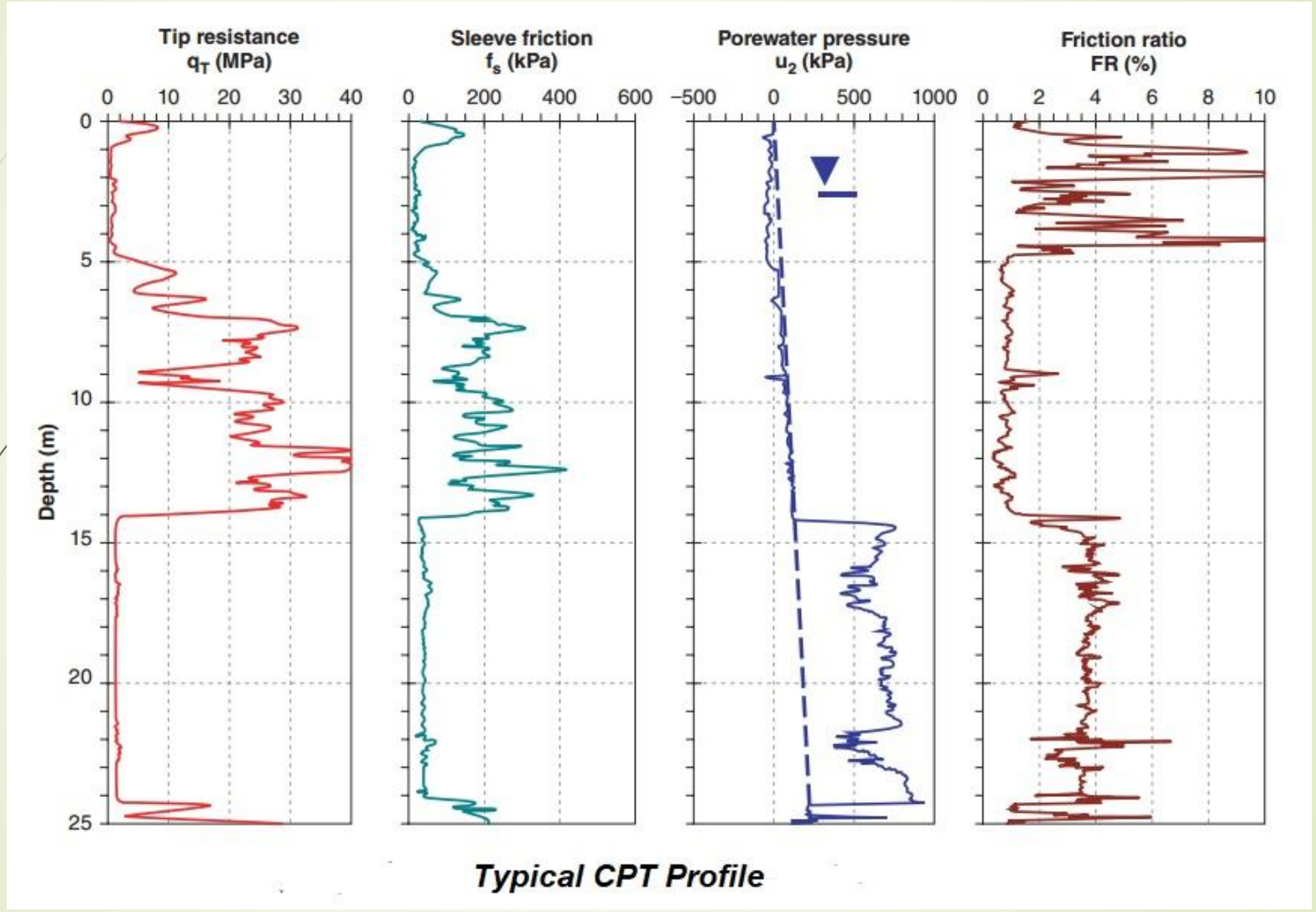


Fig. 39 (c): Typical sounding log showing friction, cone resistance and friction ratio

10.3 Static Cone Penetration Test (CPT)

The friction ratio R_f varies greatly with the type of soil. The variation of R_f for the various types of soils is generally of the order given in Table 10.

Table 10: Friction Ratio and Soil Classification

Friction Ratio, R_f (%)	Type of Soil
0.0 – 0.5	Loose gravel fill
0.5 – 2.0	Sands and gravels
2.0 – 5.0	Clay sand mixtures and silts
> 5.0	Clays, peats etc.

Meyerhof (1965), presented comparative data between SPT and CPT. For fine or silty medium loose to medium dense sands, he presents the correlation as:

$$q_c(kN/m^2) = 0.4N \qquad (10)$$

10.3 Static Cone Penetration Test (CPT)

Meyerhof's findings are as given in Table 11. The lowest values of the angle of φ given in Table 11 are conservative estimates for uniform, clean sand and they should be reduced by at least 5° for clayey sand. These values, as well as the upper values of the φ which apply to well graded sand, may be increased by 5° for gravelly sand.

Table 11: Soil Classification Based on Cone Resistance

State	D_r	$N_{60}\ or (N_1)_{60}$	q_c (Mpa)	φ^o
Very loose	< 0.2	< 4	< 2	< 30
Loose	0.2 – 0.4	4 – 10	2 – 4	30 – 35
Medium dense	0.4 – 0.6	10 – 30	4 – 12	35 – 40
Dense	0.6 – 0.8	30 – 50	12 – 20	40 – 45
Very dense	0.8 – 1.0	> 50	> 20	> 45

10.3 Static Cone Penetration Test (CPT)

(N) Correlation between SPT and CPT

Figure 40 shows a correlation presented by Robertson and Campanella (1983) between the ratio of q_c/N and mean grain size, D50. It can be seen from the figure that the ratio varies from 1 at $D_{50} = 0.001$ mm to a maximum value of 8 at $D_{50} = 1.0$ mm. The soil type also varies from clay to sand.

It is clear from the above discussions that the value of n $(= q_c/N)$ is not a constant for any particular soil. Designers must use their own judgement while selecting a value for n for a particular type of soil.

10.3 Static Cone Penetration Test (CPT)

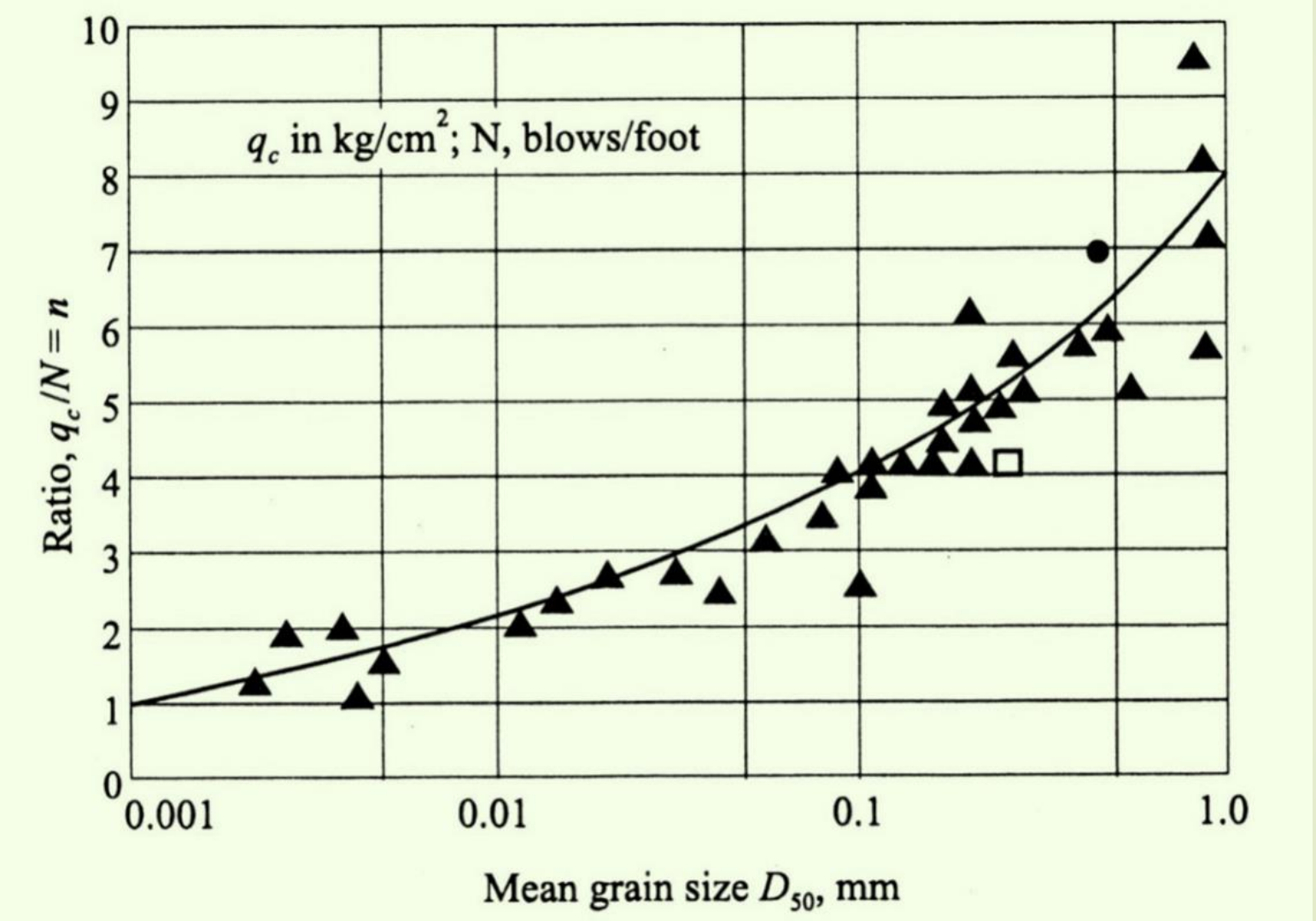


Fig. 40(a): Relation between q_c and N

10.3 Static Cone Penetration Test (CPT)

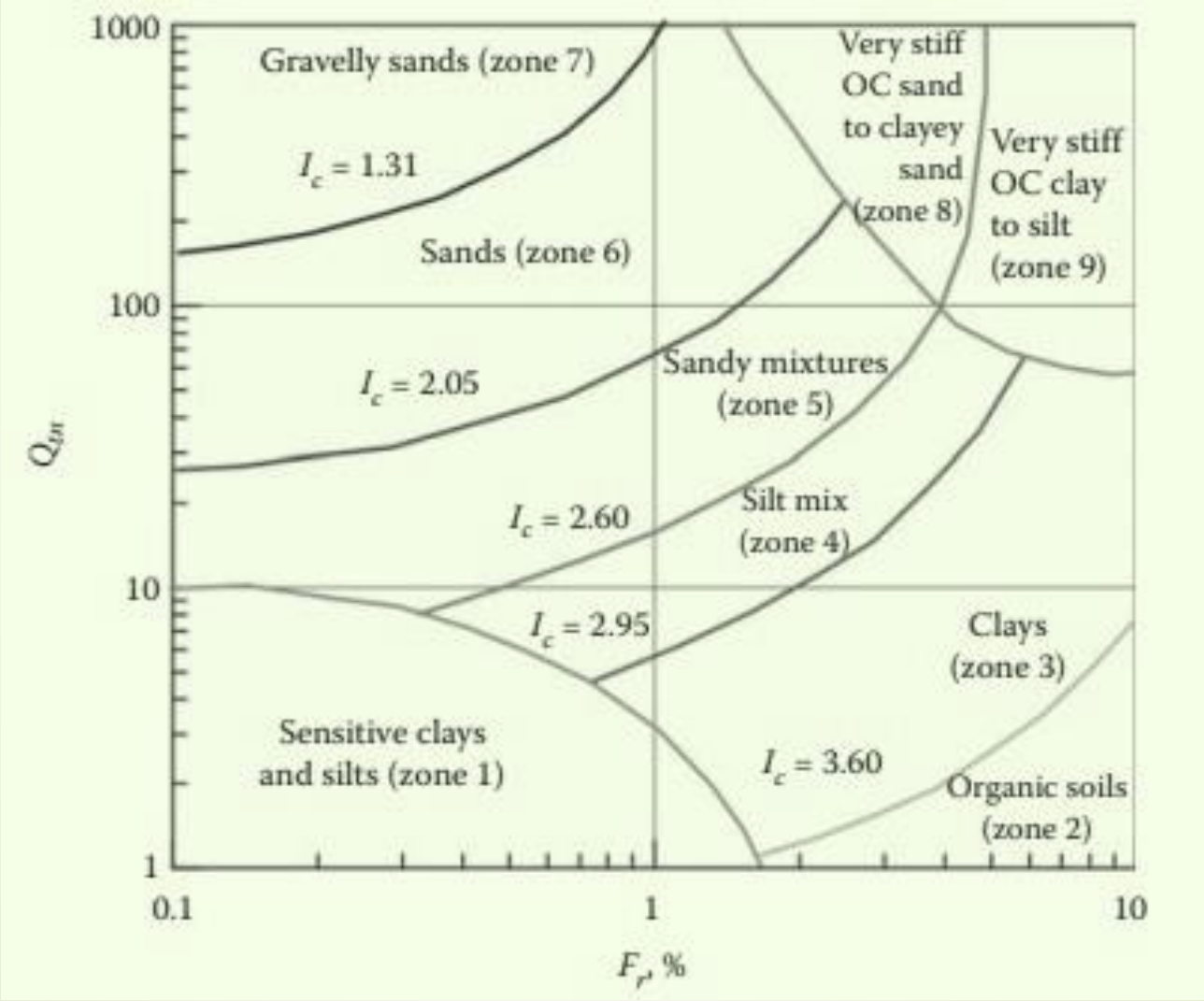


Fig. 40(b): Relation between q_c and N

11.0 Geophysical Methods of Exploration

Geophysics is an area of science dealing with the physics of the Earth. In its broadest sense it includes seismology, geodesy, atmospheric science, geomagnetometry, geothermometry, hydrology, oceanography, tectonophysics, geodynamics, glaciology, petrophysics, mineral physics, and exploration and engineering geophysics.

Exploration geophysics for civil engineering applications relies on a number of nondestructive geophysics tests aimed at obtaining soil and rock properties and soil and rock stratigraphy from the surface. Borehole geophysics and remote sensing are also parts of geophysical methods.

11.0 Geophysical Methods of Exploration

Geophysical methods include:

- Seismic techniques
- Gravity techniques
- Magnetic techniques
- Electrical techniques
- Electromagnetic techniques
- Borehole techniques, and
- Remote sensing techniques.

Gravity and magnetic techniques are not used very often in geotechnical engineering. They essentially consist of measuring the gravity field and the magnetic field to infer stratigraphy.

11.0 Geophysical Methods of Exploration

Geophysical techniques differ from geotechnical techniques in that they tend to give average soil and rock properties of large masses (many cubic meters) nondestructively, whereas geotechnical techniques give soil and rock properties at a much smaller scale (a few cubic decimeters) through mechanical testing.

Geophysical methods are extremely useful in geotechnical engineering because they allow the engineer to infer the large-scale properties between sites of geotechnical measurements and because some of them give parameters that are directly useful in design. Engineering geology contributes to the geotechnical engineering knowledge of a site at an even larger scale.

11.0 Geophysical Methods of Exploration

Several types of geophysical exploration techniques permit a rapid evaluation of subsoil characteristics. These methods also allow rapid coverage of large areas and are less expensive than conventional exploration by drilling. However, in many cases, definitive interpretation of the results is difficult. For that reason, such techniques should be used for preliminary work only.

Three types of common geophysical exploration like seismic refraction survey, cross-hole seismic survey and electrical resistivity survey are described here.

11.1 Principles of Seismic Geophysical Technique

Seismic geophysical techniques are based on seismic waves of energy (particle motion) that travel through soil, rock, or water. They may be created by a natural event (for example, an earthquake) or an artificial impact (as in seismic testing).

Seismic waves propagate because the disturbance created by a shock at a point A influences the particles at point B next to point A, which influence the particles at point C next to point B, and so on. The disturbance in this case is the motion of particles. The velocity of the particle is u and the velocity of the wave is v. The particle shakes at a frequency f when the wave passes by the particle location. After the wave has passed the particle location, the particle stops shaking.

11.1 Principles of Seismic Geophysical Technique

If the wave propagation in one direction is frozen at a given time, it shows a wave crest followed by a wave trough, followed by a wave crest and so on (Fig. 41). Waves are defined by a number of parameters.

The wave velocity v is the speed at which the particle motion is propagated from one particle to the next.

The particle velocity u is the speed at which the particle is moving around its own location.

The wave amplitude, a, is the maximum displacement of the particle from its equilibrium position.

The period T of a wave is the time between the arrival of two consecutive **c**rests (or troughs) at a given location.

11.1 Principles of Seismic Geophysical Technique

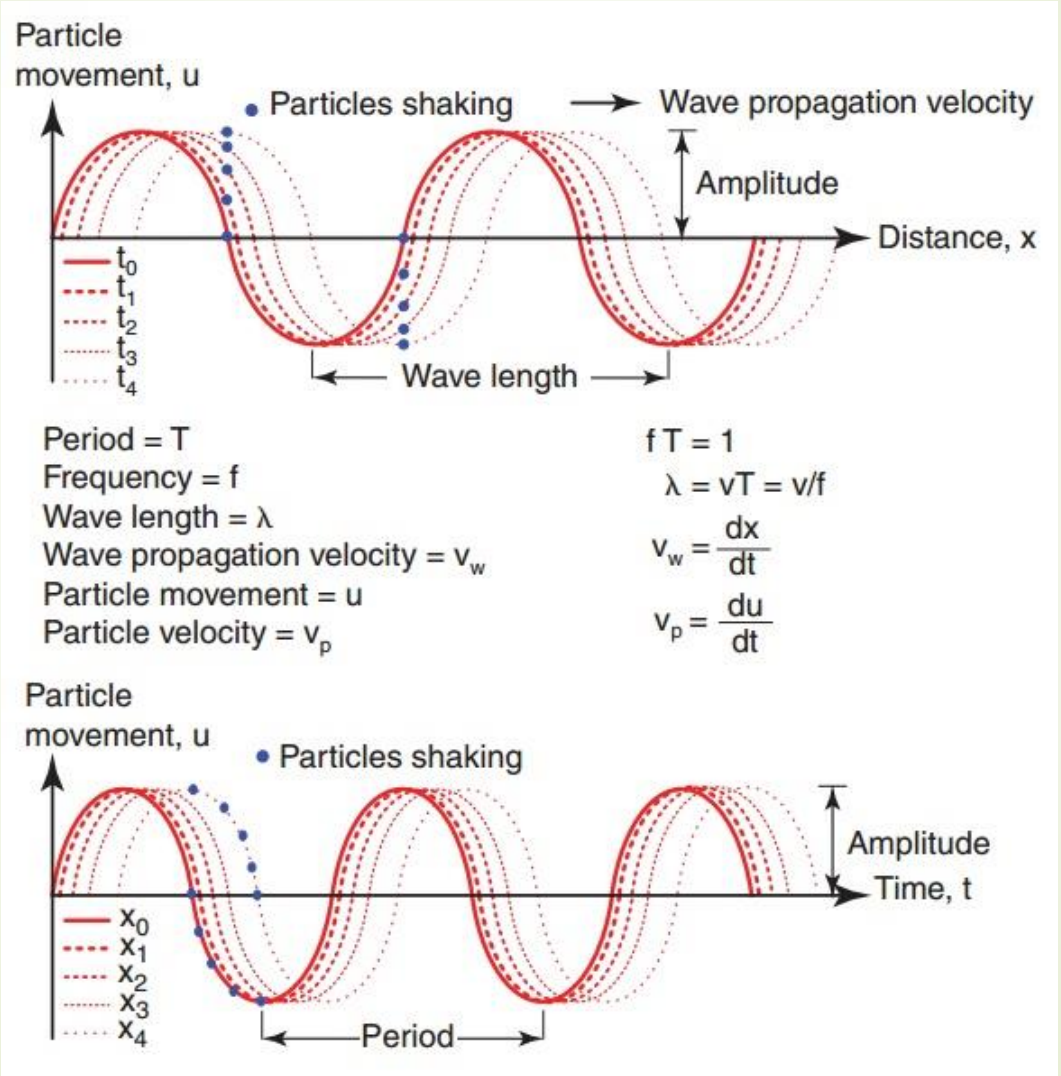


Fig. 41: Propagation of waves

11.1 Principles of Seismic Geophysical Technique

The wave frequency f is the number of periods per unit time (frequency with which the particle shakes).

The wavelength λ is the distance between two adjacent crests (or troughs) at a given time.

The frequency is set by whatever creates the initial shock; the wave speed is set by the medium through which it propagates.

Two categories of waves are identified: body waves and surface waves. Body waves propagate throughout the soil mass, whereas surface waves propagate along the ground surface. Body waves are of two types: compression waves or longitudinal waves called P waves (primary waves or pressure waves) and shear waves or transverse waves called S waves (secondary waves or shear waves) (Fig. 42).

11.1 Principles of Seismic Geophysical Technique

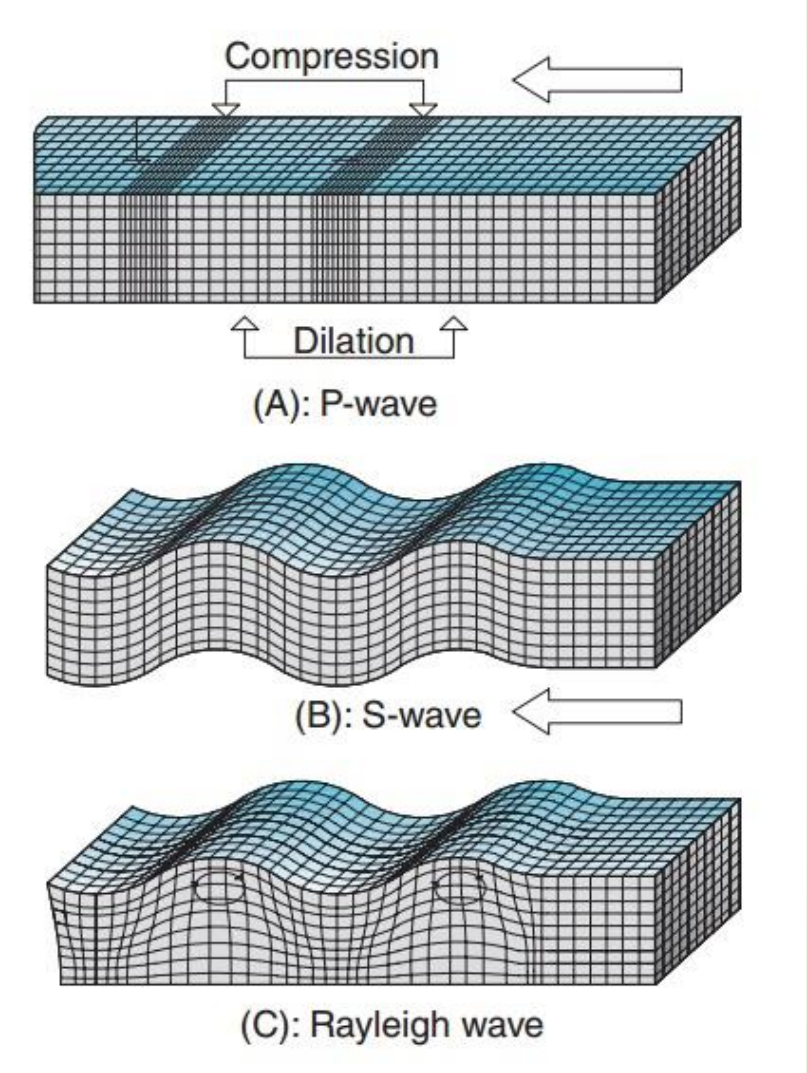


Fig. 42: Propagation of seismic body waves and surface waves.

11.1 Principles of Seismic Geophysical Technique

The P waves propagate by displacing a particle in the same direction as the direction of wave propagation; S waves propagate by displacing a particle perpendicular to the direction of wave propagation. In air, P waves are called sound waves and propagate at the speed of sound or v_p = 330 m/s. In water they propagate at v_p = 1450 m/s; in ordinary concrete at about 4000 m/s; and in granite at up to v_p = 6000 m/s.

Table 12 gives some estimates of wave velocities in earth materials.

11.1 Principles of Seismic Geophysical Technique

Table 12: Approximate soil and rock wave velocities as per ASTM D7128

Material	Wave velocity (m/s)		Density (kg/m^3)
	P-wave	S-wave	
Organic soil	300 - 700	100 - 300	1400 - 1700
Dry sand/ gravel	400 - 1500	100 - 600	1500 - 1800
Saturated sand	1000 - 2000	350 - 600	1900 - 2100
Saturated clay	1000 - 2000	200 - 600	2000 - 2200
Shale	2000 - 3500	700 - 1500	2100 - 2500
Marl	2000 - 3000	750 - 1500	2100 - 2600
Sandstone	2000 - 3500	800 - 1800	2100 - 2400
Chalk	2300 - 2600	1100 - 1300	2100 - 2600
Limestone	3500 - 6000	2000 - 3300	2400 - 2800
Granite	4500 - 6000	2500 - 3500	2500 - 2700
Water	1450 - 1500	---	1000
Ice	3400 - 3800	1700 - 1900	900

11.1 Principles of Seismic Geophysical Technique

Rayleigh waves have large amplitude, large wave length, and long duration, and propagate further than shear waves and P waves along the surface. Because their wave length is related to the depth being affected by the waves, different frequencies can be used to investigate the variation of soil properties with depth.

Love waves are slightly faster than Rayleigh waves, and are named after Augustus Love in the UK who discovered them in 1911.

11.2 Seismic Refraction Survey

Seismic refraction survey is a type of seismic geophysical technique which is useful in obtaining preliminary information about the thickness of the layering of various soils and the depth to rock or hard soil at a site.

Refraction surveys are conducted by impacting the surface, such as point A in Fig. 43(a), and observing the first arrival of the disturbance (stress waves) at several other points (e.g. B, C, D, ….). 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.

11.2 Seismic Refraction Survey

The impact on the ground surface creates two types of stress wave: P waves (or plane waves) and S waves (or shear waves). P waves travel faster than S waves; hence the first arrival of disturbance waves will be related to the velocities of the P waves in various layers. The velocity of P waves in a medium is:

$$v = \sqrt{\frac{E_s(1-\mu_s)}{\frac{g}{\gamma}(1-2\mu_s)(1+\mu_s)}} \qquad (11)$$

where, E_s = modulus of elasticity of the medium

γ = unit weight of the medium

g = acceleration due to gravity

μ_s = Poisson's ratio of soil

11.2 Seismic Refraction Survey

To determine the velocity v of P waves in various layers and thicknesses of those layers, the following procedure is used.

(a) The first arrival times t_1, t_2, t_3 … at various distances x_1, x_2, x_3 ….. from the point of impact is obtained.

(b) A graph of time t against distance x is plotted, as shown in Fig. 43(a).

(c) The slopes of the line ab, bc, cd, … are determined.

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

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

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

Here v_1, v_2, v_3, …… are the P-wave velocities in layers I, II, III, … respectively, Fig. 43(a)

11.2 Seismic Refraction Survey

(d) The thickness of the top layer is determined as:

$$z_1 = \frac{x_c}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}} \qquad (12)$$

The value of x_c can be obtained from Fig. 43(a).

(e) The thickness of the second layer can be determined from:

$$z_2 = \frac{1}{2}\left[T_{i2} - 2z_1\frac{\sqrt{v_3^2 - v_1^2}}{v_3 v_1}\right] \times \frac{v_3 v_2}{\sqrt{v_3^2 - v_2^2}} \qquad (13)$$

Where, T_{i2} is the time intercept of the line cd (Fig. 43a). The velocities of P-waves in various layers indicate the types of soil or rock that are present below the ground surface. They are listed in Tables 12 and 13.

11.2 Seismic Refraction Survey

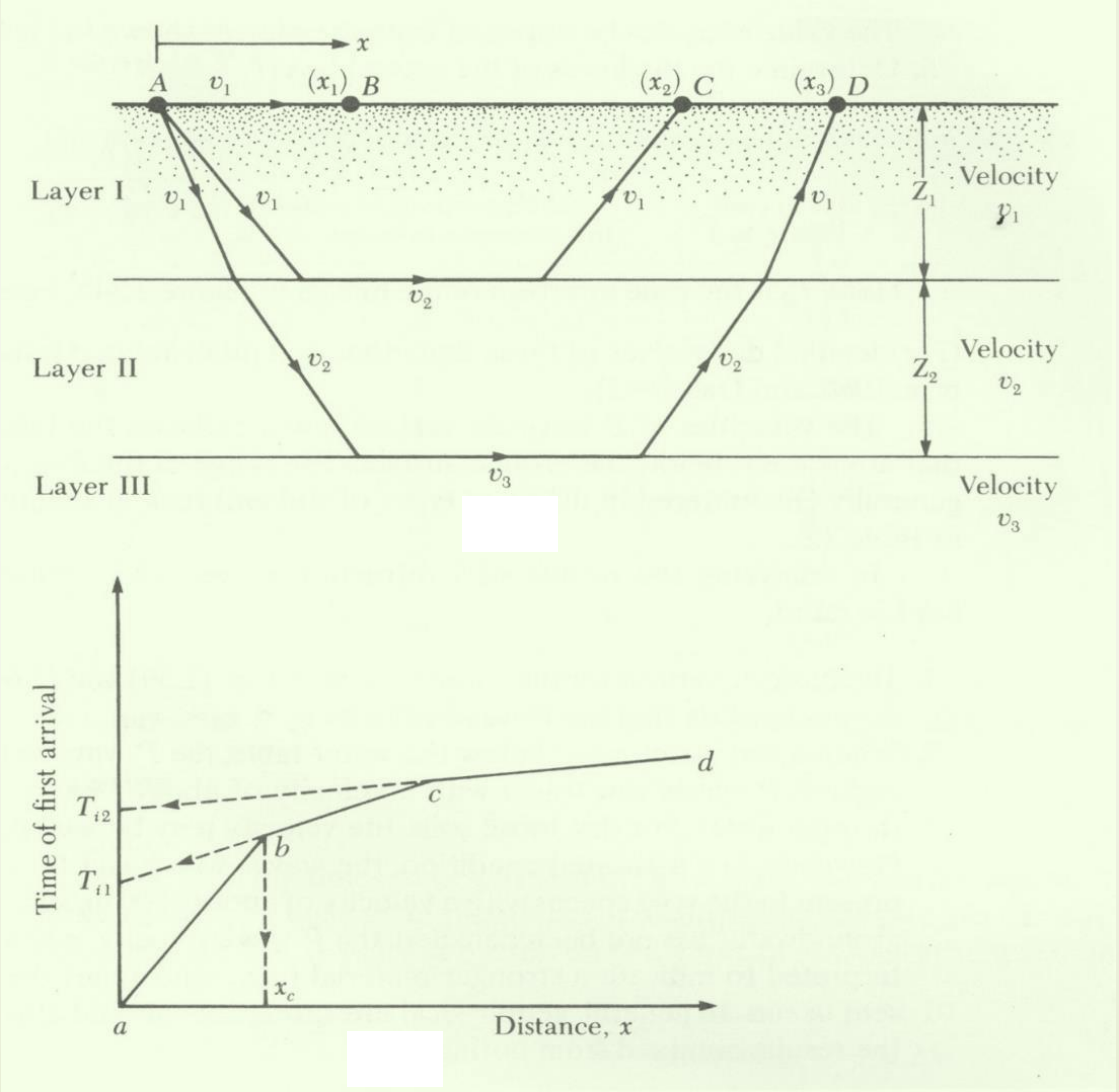


Fig. 43(a): Seismic refraction survey

11.2 Seismic Refraction Survey

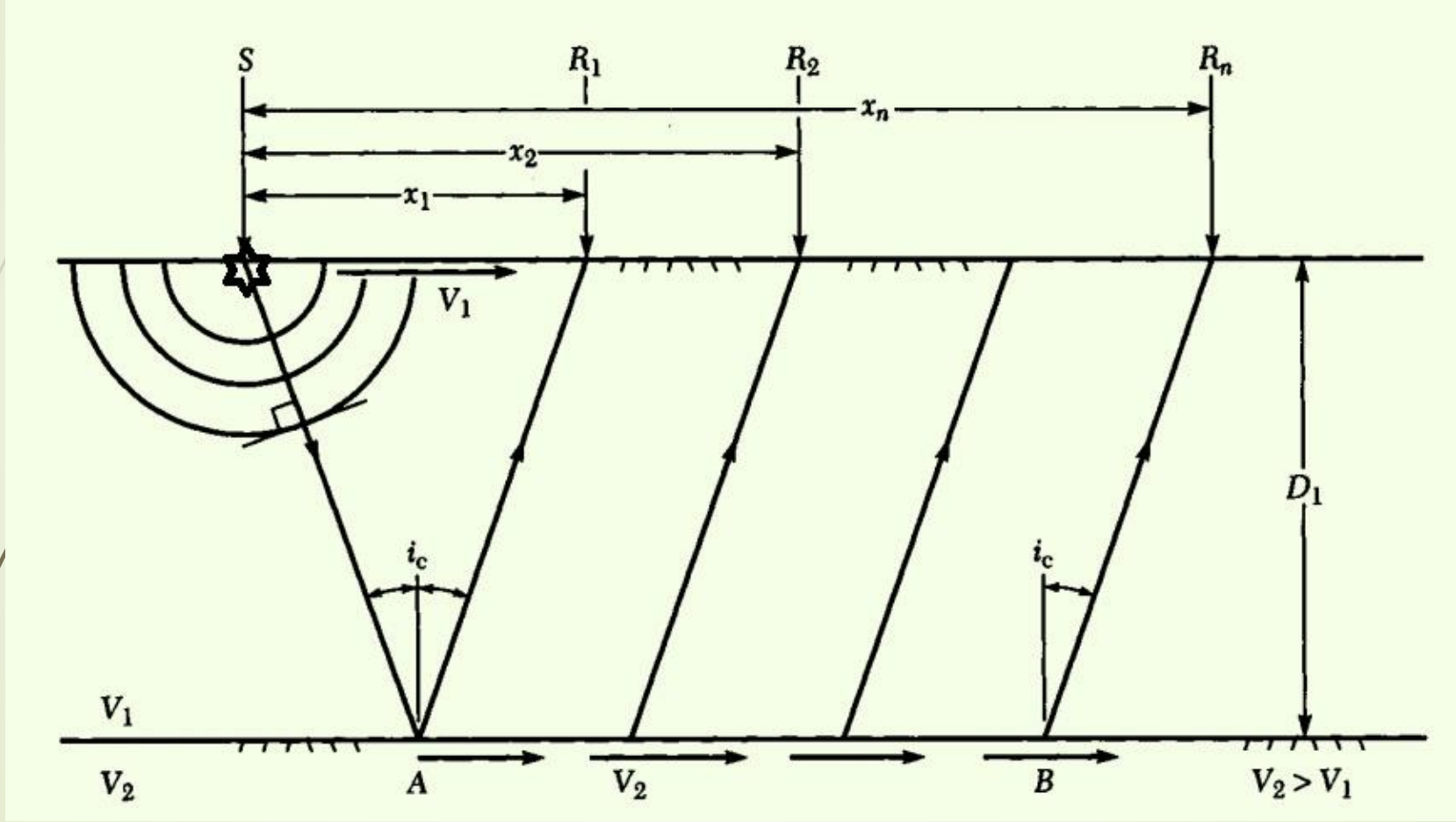


Fig. 43(b): Seismic refraction survey; ray paths for direct (primary) and refracted (shear) waves

11.2 Seismic Refraction Survey

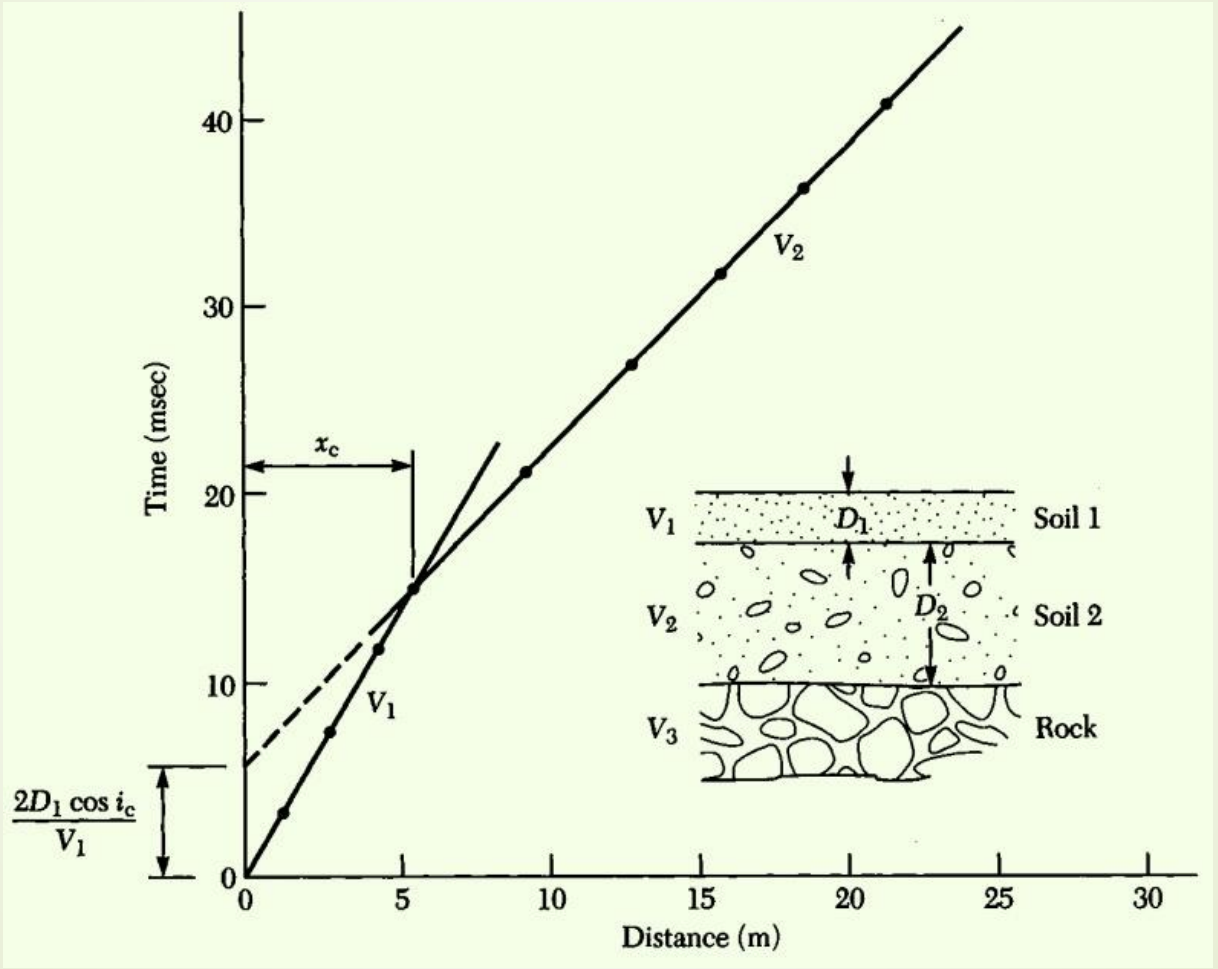


Fig. 43(c): Seismic refraction survey; travel time curves for direct (primary) and refracted (shear) waves

11.2 Seismic Refraction Survey

Table 13: Range of P-wave velocity in various soils and rocks

Type of soil or rock	P-wave velocity	
	m/sec	ft/sec
SOIL		
Sand, dry silt, and fine grained top soil	200 - 1000	650 - 3300
Alluvium	500 - 2000	1650 - 6600
Compacted clays, clayey gravel and dense clayey sand	1000 - 2500	3300 - 8200
Loess	250 - 750	800 - 2450
ROCK		
Slate and shale	2500 - 5000	8200 - 16400
Sandstone	1500 - 5000	4900 - 16400
Granite	4000 - 6000	13100 - 19700
Sound limestone	5000 - 10000	16400 - 32800

11.2 Seismic Refraction Survey

In analyzing the results of a refraction survey, two limitations need to be kept in mind.

(a) The basic equation for the survey that is Eqs. (12) and (13) are based on the assumption that the P-wave velocity $v_1 < v_2 < v_3$.

(b) When a soil is saturated below the water table, the P-wave velocity may be deceptive. P-wave can travel with a velocity of about 1500 m/sec through water. For dry, loose soils the velocity may be well below. If the presence of groundwater has not been detected, the P-wave velocity may be erroneously interpreted to indicate a stronger material (sandstone). In general, geophysical interpretations should always be verified by the results obtained from borings.

11.3 Cross-Hole Seismic Survey

The velocity of shear waves created as a result of an impact to a given layer of soil can be effectively determined by the cross-hole seismic survey. The principle of this technique is illustrated in Fig. 44. Two holes are drilled into the ground at a distance L apart. A vertical impulse is created at the bottom of one borehole by means of an impulse rod. The shear waves thus generated are recorded by means of a vertically sensitive transducer. The velocity of shear waves can be calculated as:

$$v_s = \frac{L}{t} \qquad (14)$$

Where, t is the travel time of the wave. The shear modulus G_s of the soil at the depth at which the test is done can be determined from the relation:

11.3 Cross-Hole Seismic Survey

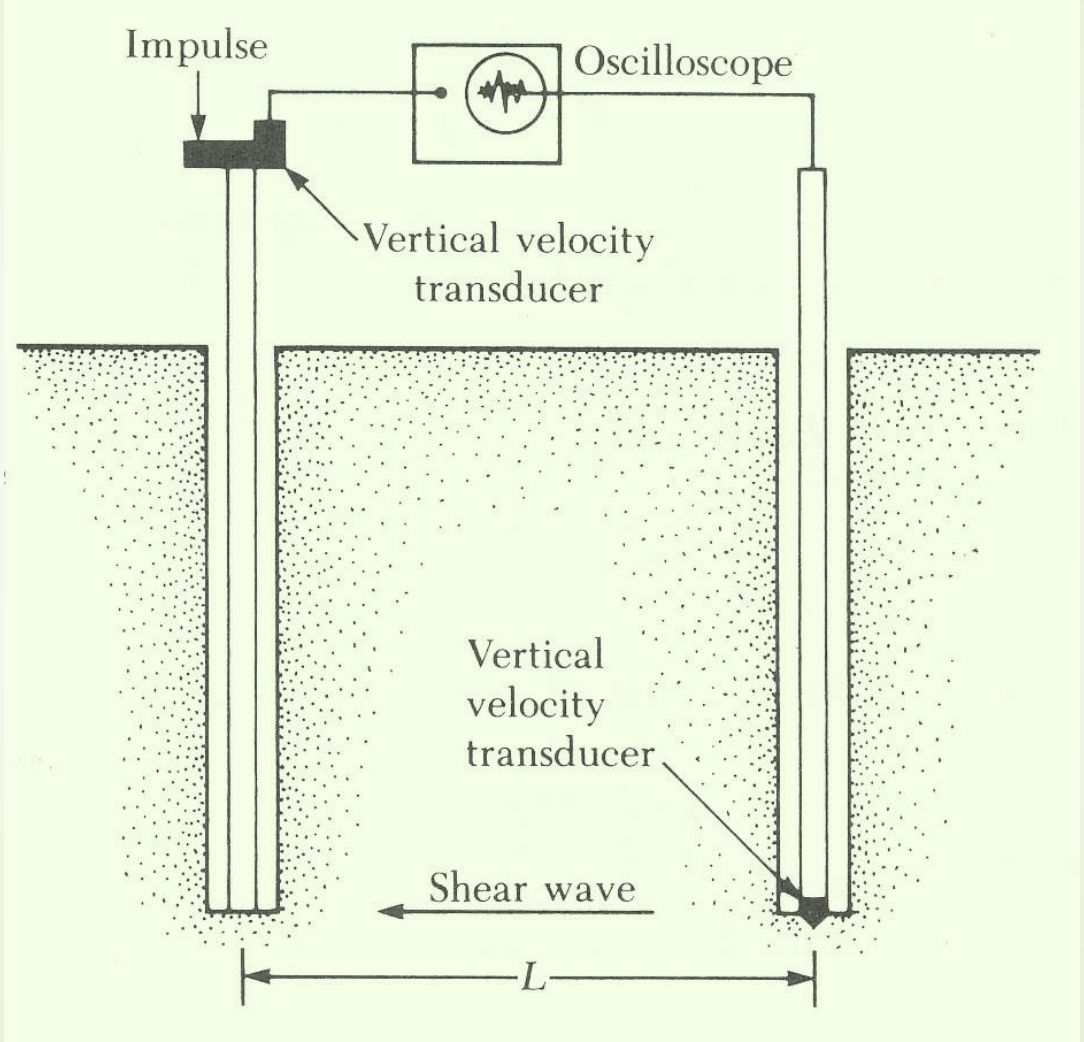


Fig. 44: Cross-hole method of seismic survey

11.2 Seismic Refraction Survey

$$v_s = \sqrt{\frac{G_s}{(\gamma/g)}} \qquad (15a)$$

Or, $$G_s = \frac{v_s^2 \gamma}{g} \qquad (15b)$$

where, v_s = velocity of shear waves

γ = unit weight of soil

g =acceleration due to gravity

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

11.4 Electrical Resistivity Survey

The electrical resistivity of any conducting material having length L, cross sectional area A and electrical resistance R can be defined as:

$$\rho = \frac{RA}{L} \qquad (16)$$

The unit of resistivity is ohm-metre. The resistivity of various soils depends primarily on their moisture content and also on the concentration of dissolve ions in them. The range of resistivity of various soils and rocks are given in Table 13.

The most common procedure of measuring the electrical resistivity of a soil profile makes use of four electrodes driven into the ground spaced equally along a straight line. The procedure is generally known as Wenner method.

11.4 Electrical Resistivity Survey

Table 13. Electrical Resistivity of Various Types of Soils

Soil type	Resistivity (ohm-m)
Sand	500 - 1500
Clays, saturated silts	0 - 100
Clayey sand	200 - 500
Gravel	1500 - 4000
Weathered rock	1500 - 2500
Sound rock	> 5000

The two outside electrodes are used to send an electrical current I (using dc current) into the ground. The current is typically in the range of 50-100 milli-amperes. The voltage drop V is measured between the two inside electrodes. The test arrangements are shown in Fig. 45. If the soil profile is homogeneous, its electrical resistivity is given by:

11.4 Electrical Resistivity Survey

$$\rho = \frac{2\pi dV}{I} \qquad (17)$$

In most cases, the soil profile may consists of various layers with different resistivities and Eqn.(17) will yield the apparent resistivity.

To obtain actual resistivity of various layers and their thicknesses, an empirical method may be used. It involves conducting test at various electrode spacing. Thus, the sum of the apparent resistivity $\sum \rho$ obtained is plotted against the spacing d as shown in Fig. 45. The plot thus obtained has relatively straight segments, the slopes of which give the resistivity of individual layers. The determination of thickness of layers is illustrated in Fig. 43. The resistivity survey is particularly useful in location gravel deposits within a fine grained soil.

11.4 Electrical Resistivity Survey

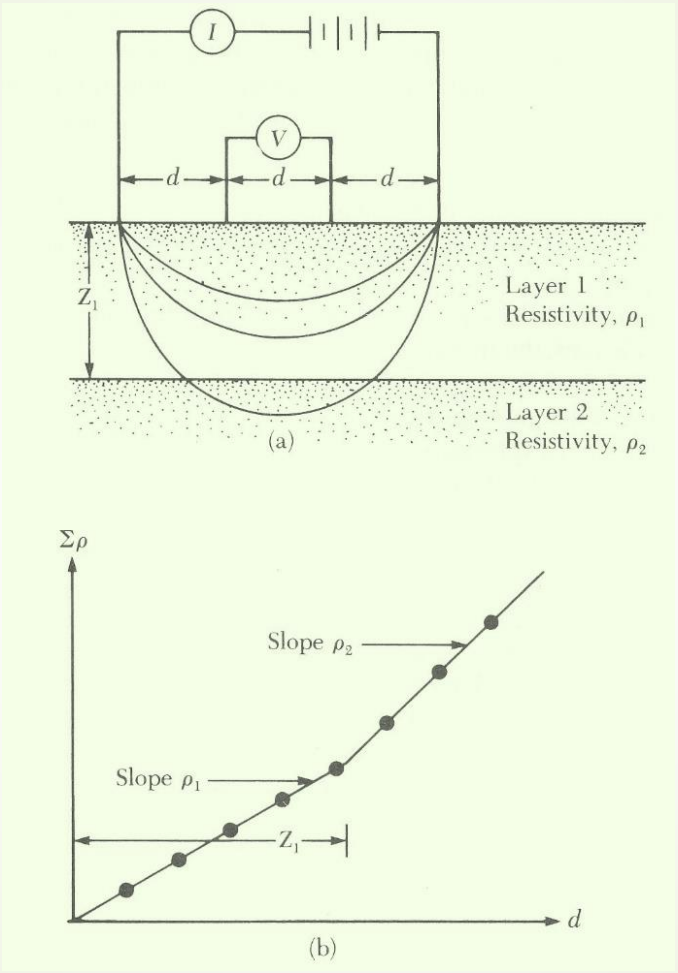


Fig. 45: Electrical resistivity survey; (a) Wenner method of placing electrode; (b) empirical method of determining resistivity and thickness of soil layer.

12.0 Soil Exploration Report

At the end of the soil exploration programme, the soil and rock samples collected from the field are subjected to visual observation and laboratory tests. Then a soil exploration report is prepared, containing the following information.

(i) Scope of investigation

(ii) General description of the proposed structure to be built

(iii) Geologic condition of the site

(iv) Drainage facilities at the site

(v) Details of boring

(vi) Description of subsoil condition

12.0 Soil Exploration Report

(vii) Groundwater table

(viii) Probale foundation alternatives

(ix) Any anticipated construction problems

(x) Limitations of the investigation.

The following information should also be furnished in a subsoil investigation report.

- Site location map
- Location of boring with respect to proposed structure
- Boring logs
- Laboratory test results
- Other special presentations.

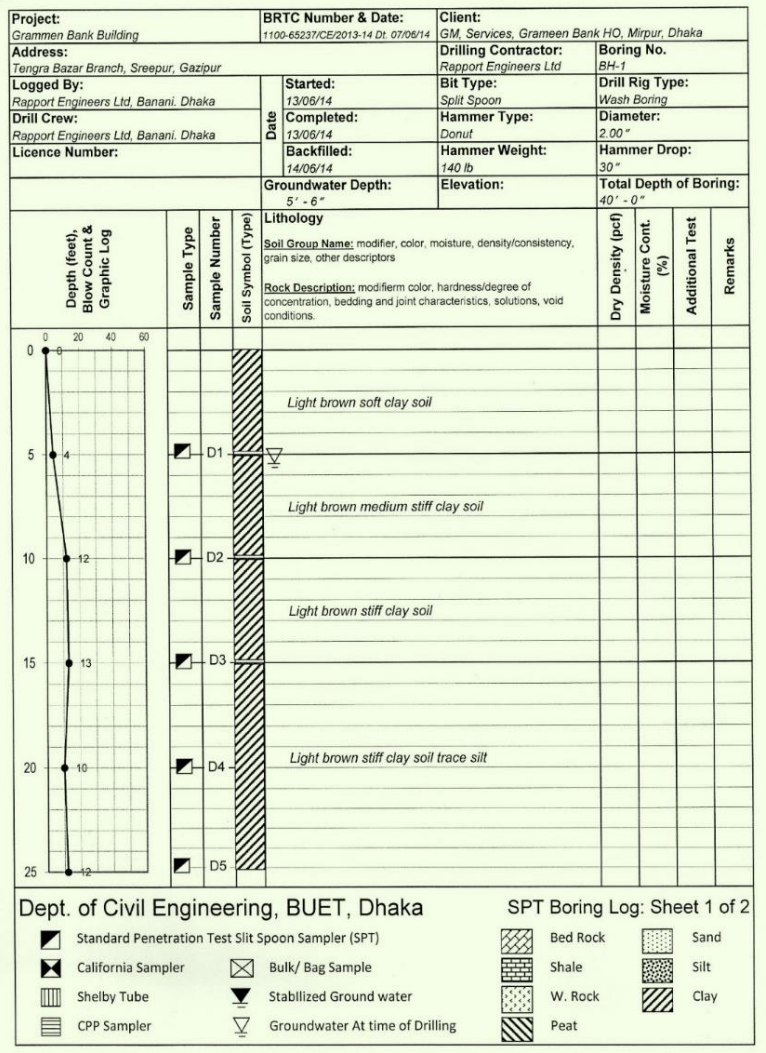
12.0 Soil Exploration Report

A typical boring log is presented in Fig. 44. It is to be noted that percentages of gravel, sand and fines in a soil layer may be stated in terms indicating a range of percentages as follows (MSL FT);

- Trace Less than 5%
- Few 5 to 10%
- Little 15 to 25%
- Some 30 to 45%
- Mostly 50 to 100%

12.0 Soil Exploration Report

Project: Grammen Bank Building	**BRTC Number & Date:** 1100-65237/CE/2013-14 Dt. 07/06/14	**Client:** GM, Services, Grameen Bank HO, Mirpur, Dhaka	
Address: Tengra Bazar Branch, Sreepur, Gazipur		**Drilling Contractor:** Rapport Engineers Ltd	**Boring No.** BH-1
Logged By: Rapport Engineers Ltd, Banani. Dhaka	**Date** **Started:** 13/06/14	**Bit Type:** Split Spoon	**Drill Rig Type:** Wash Boring
Drill Crew: Rapport Engineers Ltd, Banani. Dhaka	**Completed:** 13/06/14	**Hammer Type:** Donut	**Diameter:** 2.00"
Licence Number:	**Backfilled:** 14/06/14	**Hammer Weight:** 140 lb	**Hammer Drop:** 30"
	Groundwater Depth: 5' - 6"	**Elevation:**	**Total Depth of Boring:** 40' - 0"



PROJECT TITLE Shoping Center
LOCATION 7th mile DATE June 22, 2004
BORING NUMBER 4 TYPE OF BORING Hollow stem auger GROUND EL. 20 m

DESCRIPTION OF SOIL	DEPTH (m) & SAMPLE NUMBER	SPT VALUE (N)	MOISTURE CONTENT (%)	COMMENTS
Tan sandy silt	1	8		
	SS-1 2	8	11	LL= 25, PI= 8
	3			
	4	6	20	
Light brown silty clay (CL)	5			
	6	10	24	
Groundwater table June 22, 2004	SS-2 7			LL= 32, PI= 9
	8	12	26	
	9			
	ST-1 10	10	35	LL= 38, PI= 20
Soft clay (CL)	11			
	12	23	22	
	13			
	SS-3 14	32	28	LL= 40, PI= 20
Compact sand & gravel End of boring 15.2 m	15	35	26	

Fig. 44a: Typical borehole logs

12.0 Soil Exploration Report

Name of Site Investigation Company

BOREHOLE RECORD

BOREHOLE 3

Client: Department of Transport

Site: Proposed By-pass

Sheet 1 of 1

Job Ref. 94/56

Casing depth / Daily progress	Water level	Samples in situ tests and coring runs: Depth (m) From	To	Type	N value or core recov.	Depth (m)	Description of strata	Reduced level	Legend
							Ground level (metres OD)	9.50	287.043 N 654.529 E
12.7.94						0.30	Topsoil	9.20	
	17.7	1.00	1.45	U			Soft becoming firm with depth grey organic silty CLAY with thin bands or partings of silt		
		1.50		D					
		2.00	2.45	U		(4.70)	Vertical root holes in upper horizon		
		2.50		D			Layer of peat at 3.50-4.00 m		
		4.00	4.45	U			(Estuarine Clay)		
		4.50		D					
		5.00	5.45	SD	N=5	5.00		4.50	
(6.00)							Medium dense brown fine and medium SAND and fine to coarse GRAVEL		
13.7.94	17.7	6.00	6.45	SD	N=15	(3.00)	Gravel mostly subrounded to rounded		
		7.50	7.95	SD	N=25		(Flood Plain Gravel)		
		8.00	8.45	U		8.00	Very stiff fissured brown silty CLAY with much fine to coarse subangular to subrounded gravel and occasional cobbles	1.50	
		8.50		D					
		10.00	10.45	U					
		10.50		D		(6.00)	Lenses of sand at 10.2 and 12.5 m		
		12.00	12.45	U					
		12.50		D			(Glacial Clay)		
		14.00	14.45	SD	N=76	14.00	Greyish brown highly becoming moderately weathered weak becoming moderately strong thinly bedded fine to medium grained SANDSTONE	-4.50	
(10.00)							Bedding subhorizontal		
15.7.94		15.00	16.50	H	T=78 S=47 R=32	(>4.0)	Frequent joints at 45º (Coal Measures)		
		16.50	18.00	H	T=95 S=75 R=45		(Westphalian B)	-8.50	
						18.00	END		

Piezometer sealed at 6.00 m, water level at 2.00 m 17.7.94

Piezometer sealed at 15.00 m, water level at 7.00 m 17.7.94

T – Total core recovery
S – Solid core recovery
R – Rock quality designation

Boring equipment and methods

Cable percussion, 150 mm, GL – 15.00 m

Rotary coring, water flush 15.00 – 18.00 m

Logged by: GEB Scale 1:100

Remarks

Chiselling sandstone 1 hour

Very fast seepage at 5.00 m, level rose to 2.20 m after 20 mins.

Sealed off by casing at 10.00 m

Moderate seepage at 14.00 m, level rose to 12.00 m after 20 mins

Water level at 2.00 m on morning of 13.7.94 and at 8.00 m on 15.7.94

bentonite/cement

sand

bentonite/cement

sand

Fig. 44a: Typical borehole logs

12.0 Soil Exploration Report

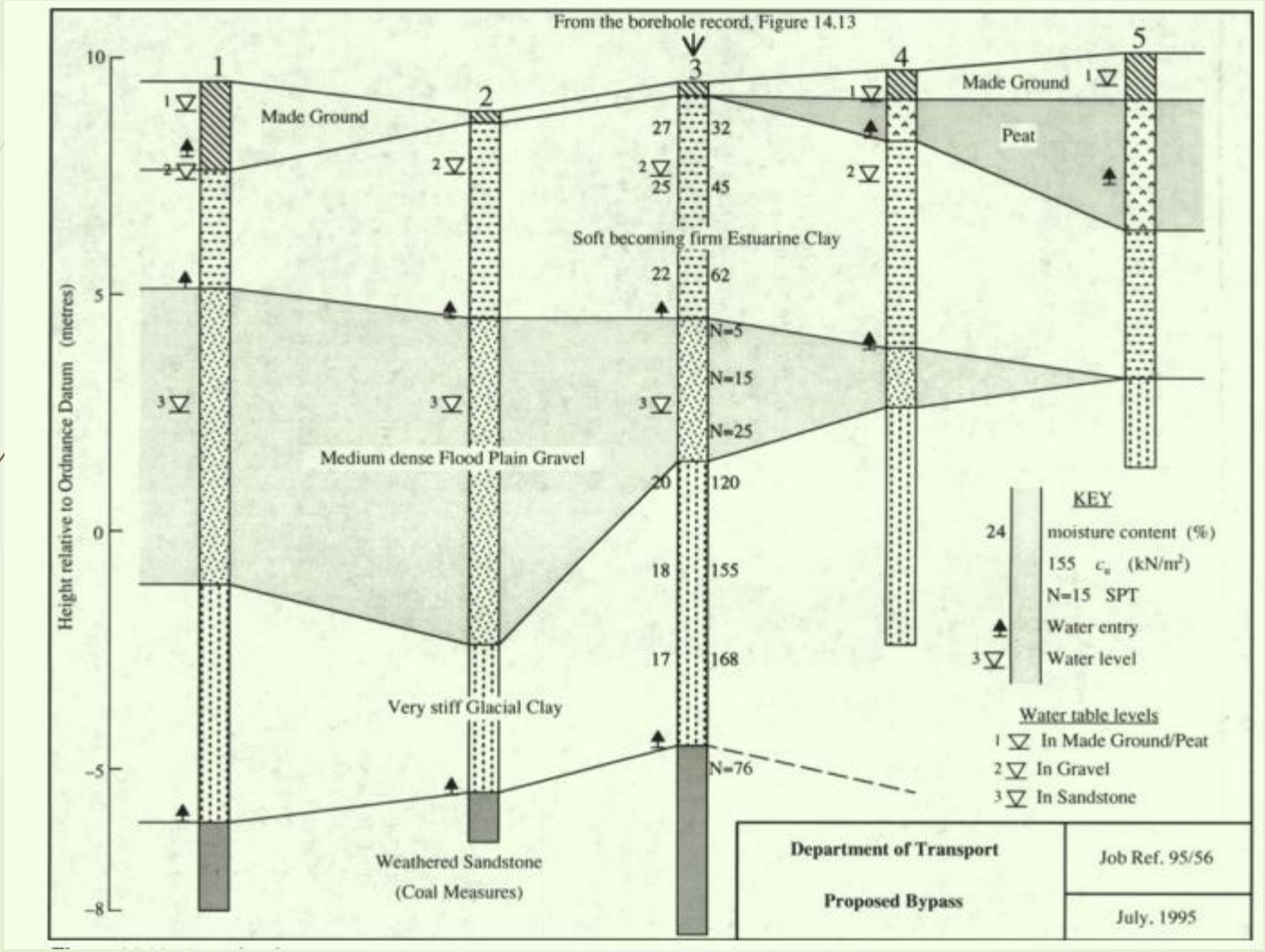


Fig. 44b: Typical soil profile

13.0 Typical Borehole Symbols and Abbreviations

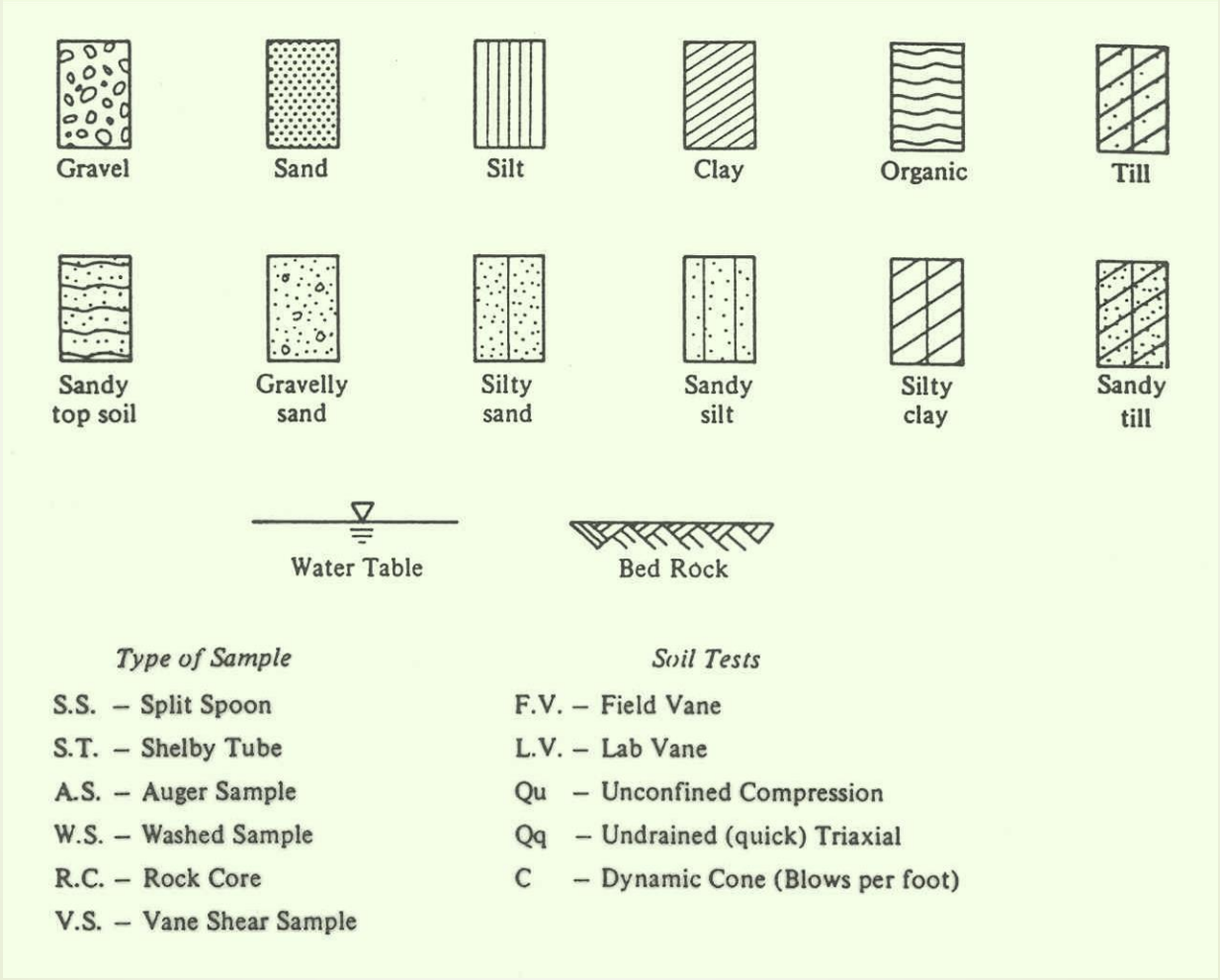


Fig. 45: Typical symbols of soil used in borehole

14.0 Numerical Example

A manufacturing plant and warehouse will be a one storey building with a floor load of 35 kN/m^2 constructed on a fill of 1.5 m thick. The building plant will be 100 m × 300 m. What will be a reasonable boring plan and depth? Consider uniform subsoil condition.

14.0 Numerical Example

Oskorouchi, A.M. and Mehdibeigi, A., 1988. Effect of Overburden Pressure on SPT N-Values in Cohesive Soils of North Iran. Proceedings of the 1st International Symposium on Penetration Testing, Vol. 1, pp. 363–367.

Serajuddin, M. and Chowdhury, M., 1996. Correlation between Standard Penetration Resistance and Unconfined Compression Strength of Bangladesh Cohesive Deposits. Journal of Civil Engineering, The Institution of Engineers, Bangladesh, Vol. CE24, No. 1, pp. 69–83.

Luteneggar, A.J. (2021), In Situ Testing Methods in Geotechnical Engineering, CRC Press, Taylor and Francies Group, USA.