Al-Mansour University College

قسم الهندسة المدنية

Civil Eng. Dept.

المرحلة الرابعة

4th. Stage

Foundation Engineering

2022 – 2023

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هندسة الاسس

CHAPTER 1

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INTRODUCTION

1.1 DEFINITIONS

All types of structures consist of two parts; the upper or *superstructure* and the lower *substructure or (foundation).*

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foundation. But, in general, it is the lowest part of a structure or building that transmits its weight safely to the underlying soil or rock.

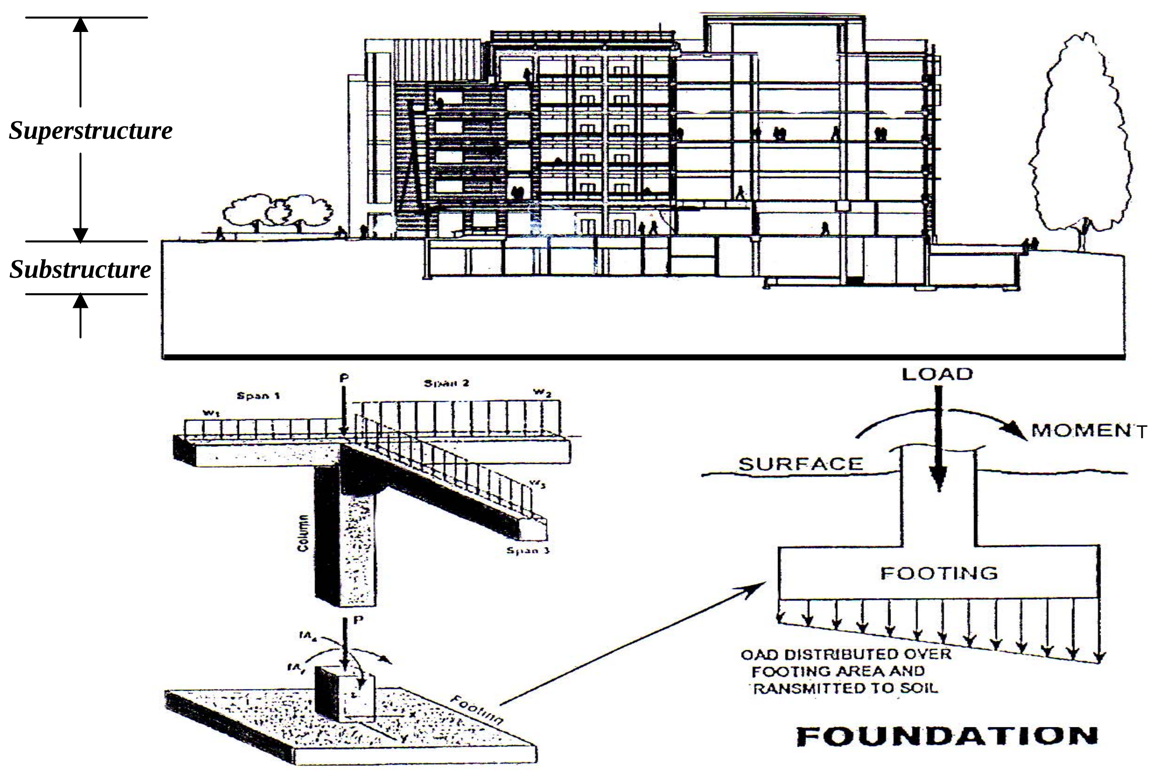


Fig. (1.1): Foundation definition.

- **FOUNDATION ENGINEERING:** is the science of applying engineering judgments and principles of soil mechanics to solve interfacing problems and retaining structures. Or it is the branch of engineering science which deals with two problems:

1. Evaluate the ability of soil to carry a load without shear failure or excessive settlement.
2. To design a proper structural member which can transmit the load from superstructure to soil taking economics into consideration.

1.2 CLASSIFICATION OF FOUNDATIONS

Foundations can be classified basically into two types: **shallow** and **deep.**

- **Shallow Foundations:**

These types of foundations are so called because they are placed at a shallow depth (relative to their dimensions) beneath the soil surface. Their depth may range from the top of soil surface to about 3 times the breadth (about 6 meters). They include ***spread footings*** as circular or square or rectangular in plan which support columns, and strip footings which support walls and other similar structures. In addition to, combined and mat foundations and soil retaining structures (retaining walls, sheet piles, excavations and reinforced earth).

- **Deep Foundations:**

The most common of these types of foundations are ***piles*** *and* ***drilled shafts***. They are called deep because they are embedded very deep (relative to their dimensions) into the soil. Their depths may run over several tens of meters. They are usually used when the top soil layers have low bearing capacities (the soil located immediately below the structure is weak, therefore the load of the structure must be transmitted to a greater depth).

The shallow foundation shown in **Fig. (1.2)** has a width B and a length L. The depth of embedment below the ground surface is equal to D_f. This depth must be adequate to avoid:

1. Lateral expulsion of soil beneath the foundation.
2. Seasonal volume changes such as freezing or the zone of active organic materials.
3. The depth be sufficient enough that the foundation should be safe against overturning, sliding, rotational failure, and overall soil shear failure and excessive settlement.

Theoretically, when B/L is equal to zero (that is, $L = \infty$), a plane strain case will exist in the soil mass supporting the foundation. For most practical cases when B/L (1/5 to 1/6), the plane strain theories will yield fairly good results.

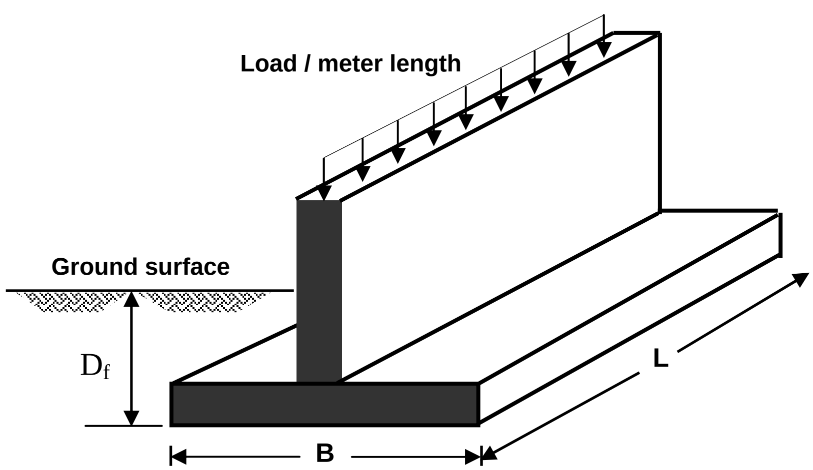


Fig. (1.2): Individual footing.

Terzaghi defined a shallow foundation as one in which the depth, D_f, is less than or equal to the width B ($D_f / B \leq 1$). Otherwise, it is considered as deep foundation.

In some cases, there is a different depth of embedment below the ground surface on both sides of a foundation as shown in **Fig. (1.3)**. For those cases, D_f should be *the depth at shallow side*, in addition to, the overburden pressure must be compared with soil cohesion to decide the type of footing required for design as follows:

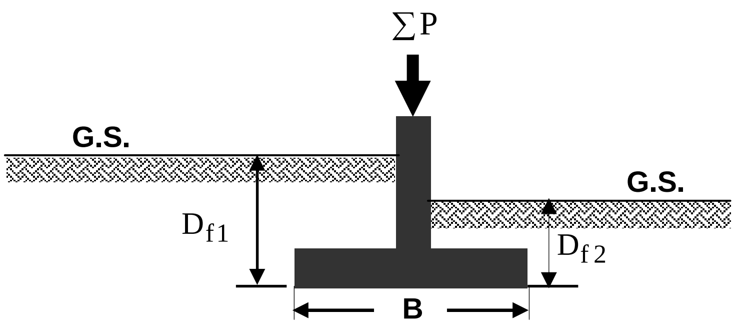


Fig. (1.3): Depth of embedment.

If $(D_{f1}.\gamma - D_{f2}.\gamma) > \dfrac{q_u}{2}$ **……….. Design the member as a retaining wall.**

If $(D_{f1}.\gamma - D_{f2}.\gamma) \leq \dfrac{q_u}{2}$ **…………Design the member as a footing.**

where q_u is unconfined compressive strength of soil.

From soil mechanics principles $\sigma_1 = \sigma_3.\tan^2(45+\phi/2) + 2.c.\tan.(45+\phi/2)$

For Unconfined Compressive Strength Test (U.C.T.): $\sigma_1 = q_u$ **and** $\sigma_3 = 0$ **; Therefore:**

- *For Pure Cohesive Soil* **(**$\phi_u = 0$**):** $q_u = 2.C_u$
- *For* $c-\phi$ *Soil:* $q_u = 2.C_u.\tan.(45+\phi/2)$

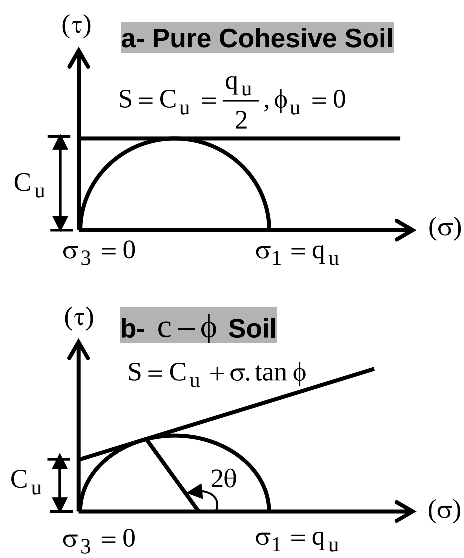


Fig. (1.4): Unconfined compressive strength test.

1.3 SETTLEMENT AT ULTIMATE LOAD

Settlement means a vertical displacement of a structure or footing or road,…etc.. The settlement of the foundation at ultimate load, S_u, is quite variable and depends on several factors. Based on laboratory and field test results, the approximate ranges for S_u values of soils are given below.

Soil	D_f / B	S_u / B (%)
Sand	0	5 to 12
Sand	Large	25 to 28
Clay	0	4 to 8
Clay	Large	15 to 20

For any foundation, one must ensure that the load per unit area of foundation does not exceed a limiting value, thereby causing shear failure in soil. This limiting value is the ***ultimate bearing capacity***, $q_{ult.}$ and generally using a factor of safety of 3 to 4 the allowable bearing capacity, $q_{all.}$ can be calculated as:

$$q_{all.} = \frac{q_{ult.}}{F.S} \qquad \textbf{(1.1)}$$

However, based on limiting settlement conditions, there are other factors which must be taken into account in deriving the allowable bearing capacity. The total settlement, S_T, of a foundation will be the sum of three components:

1. Elastic or immediate settlement, S_i **; (major in sand),**
2. Primary and Secondary consolidation settlements, S_c and S_{cs} **; (major in clay).**

$$S_T = S_i + S_c + S_{cs} \qquad \textbf{(1.2)}$$

Most building codes provide an allowable settlement limit for a foundation which may be well below the settlement derived corresponding to $q_{all.}$ given by **Eq.** (**1.1**). Thus, the bearing capacity corresponding to the allowable settlement must also be taken into consideration. A given structure with several shallow foundations may undergo two types of settlement:

1. Uniform or equal total settlement, and
2. Differential settlement.

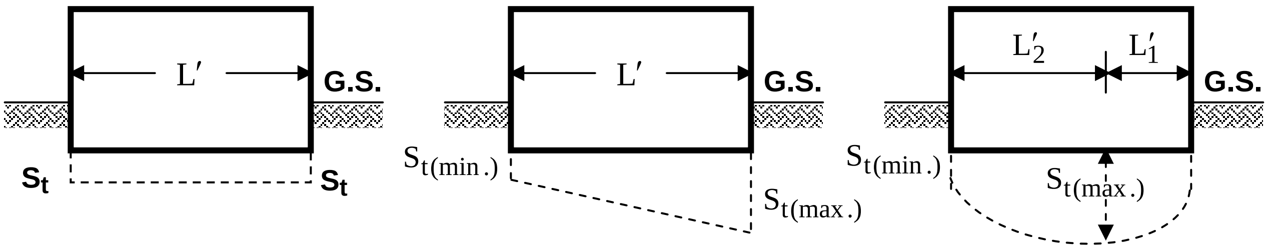
Fig. (1.5a) shows a uniform settlement which occurs when a structure is built over rigid structural mat. However, depending on the load of various foundation components, a structure may experience differential settlement. A foundation may also undergo uniform tilt (**Fig. 1.5b**) or non-uniform settlement (**Fig. 1.5c**). In these cases, the angular distortion, Δ, can be defined as:

$$\Delta = \frac{S_{t(max)} - S_{t(min)}}{L'} \qquad \textit{(for uniform tilt)} \qquad \textbf{(1.3)}$$

$$\Delta = \frac{S_{t(max)} - S_{t(min)}}{L'_1} \qquad \textit{(for non-uniform settlement)} \qquad \textbf{(1.4)}$$

Limits for allowable differential settlement of various structures are available in building codes. Thus, the final design of a foundation depends on:

(a) the ultimate bearing capacity, (b) the allowable settlement, and
(c) the allowable differential settlement for the structure.



(a) Uniform settlement **(b) Uniform tilt** **(c) Non-uniform settlement**

Fig. (1.5): Settlement of a structure.

Example (1.1):

A 30 cm x 30 cm column is loaded with 40 Ton. Check whether the column can be placed on soil directly or not if the allowable bearing capacity of soil is:

(a) $q_{all.}$ = 50 kg/cm^2, and

(b) $q_{all.}$ = 1.0 kg/cm^2.

Solution:

(a) $q_{all.} = \dfrac{Q}{A}$

or $A = \dfrac{40000}{50} = 800 \text{ cm}^2$ (minimum required area) < 900 cm^2 (area of column)……… **O.K.**

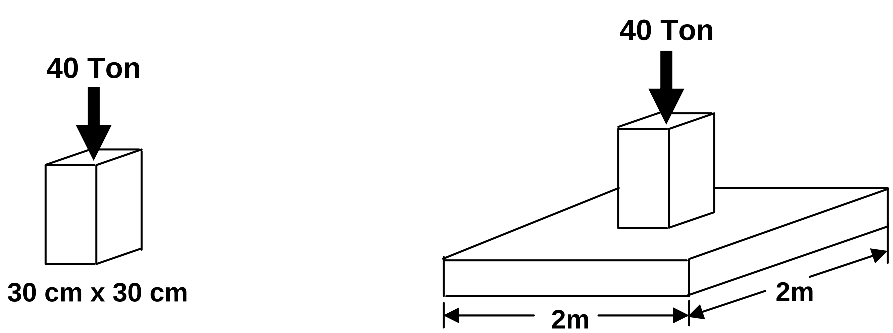
or $q_{all.} = \dfrac{40000}{30x30} = 44.4 \text{ kg/cm}^2 < 50 \text{ kg/cm}^2$ …… **O.K.**

$\therefore$ No failure may happen; and the column can be placed directly on the soil.

(b) $A = \dfrac{40000}{1.0} = 40000 \text{ cm}^2$ (minimum required area) > 900 cm^2 (area of column) ..... **N.O.K.**

$\therefore$ (Not safe) and the column in this case cannot be placed directly on soil, therefore, an enlarged base is required.

A = 40000 cm^2 = 4 m^2, assuming square area: $B = \sqrt{A} = \sqrt{4} = 2$m. (see **Fig. (1.6)**).



(a) $q_{all.} = 50.kg/cm^2$ **(b)** $q_{all.} = 1.0.kg/cm^2$

Fig. (1.6): Solution of example (1.1).

1.4 TYPES OF FAILURE IN FOOTINGS

It is possible due to load that a footing fails by one or two of the following:

(1) Shear failure: this failure must be checked against:-

(i) punching shear and **(ii)** wide beam shear. No shear failure is satisfied by providing an adequate thickness of concrete (see **Fig. 1.7**).

(2) Tension failure: this failure decides the locations and positions of steel distribution. No tension failure is satisfied by providing an adequate steel reinforcement (see **Fig. 1.7**).

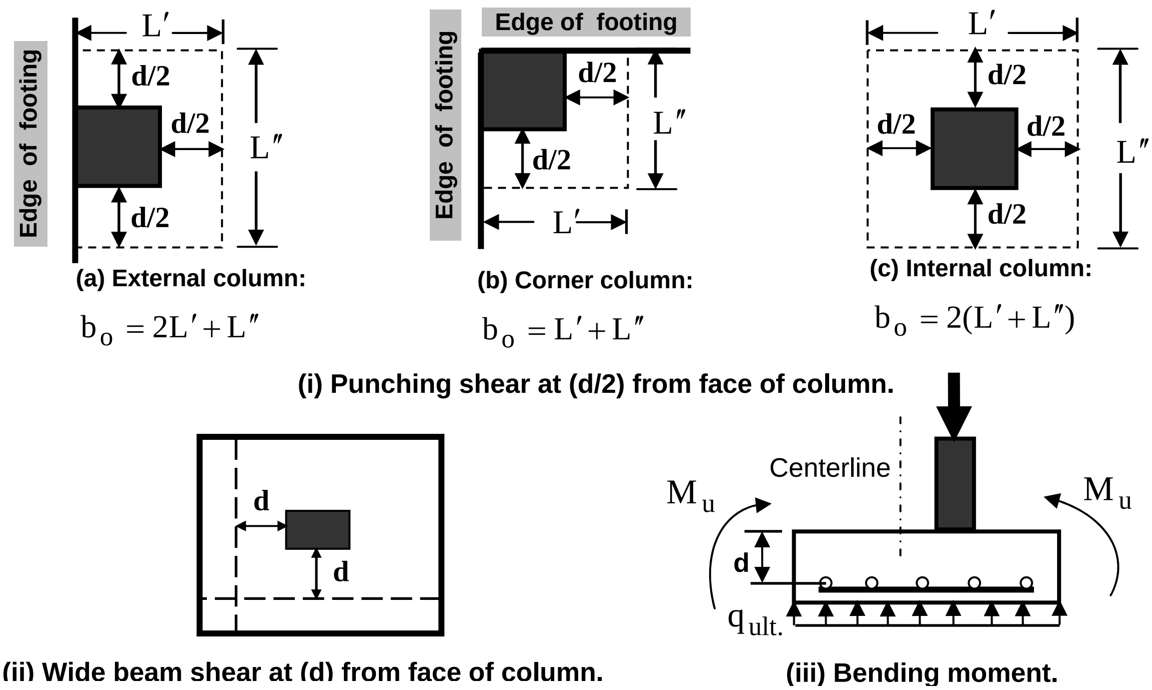


Fig. (1.7): Types of failure in footing.

1.5 TYPES OF FOOTINGS

(1) Spread or Isolated or Individual Column Footing:

It is a footing of plain or reinforced concrete that supports a single column. It may be either a square or circular or rectangular in shape or cross sectional area (se to make the resultant of loads within the middle third of footing.

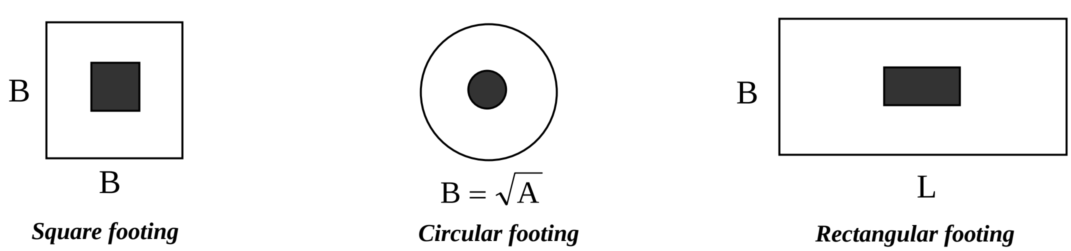


Fig. (1.8): Spread footing.

(2) Combined Footing (reinforced concrete only)**:**

It is a footing that connects several columns and can take one of the following shapes:

- **Rectangular Combined Footing** (see **Fig. 1.9**):

(a) Used along the walls of building at property lines where the footing for a several columns are transmitted to the same footing, the footing should be proportioned so that its centroid coincides with the center of gravity of the column loads.

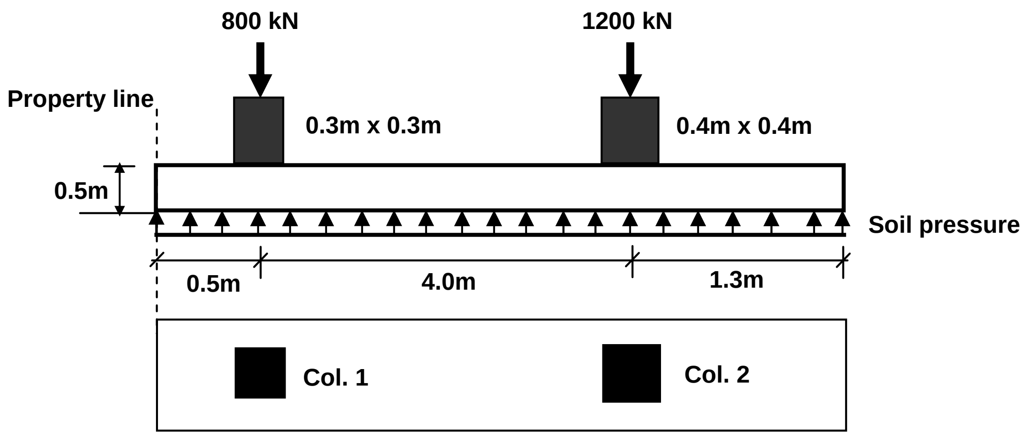


Fig. (1.9: Rectangular combined footing.

- **Trapezoidal Combined Footing** (see **Fig. 1.10**)**:**

(a) If the maximum load exists at the exterior column,

(b) It is not possible to make the resultant of loads passes through the centroid of the footing.

(i.e., If $\frac{L}{2} > \bar{x} > \frac{L}{3}$).

- **Strap or Cantilever Combined Footing** (see **Fig. 1.11**)**:**

(a) If there is an eccentricity, and/ or

(b) If $(\bar{x} < \frac{L}{3})$.

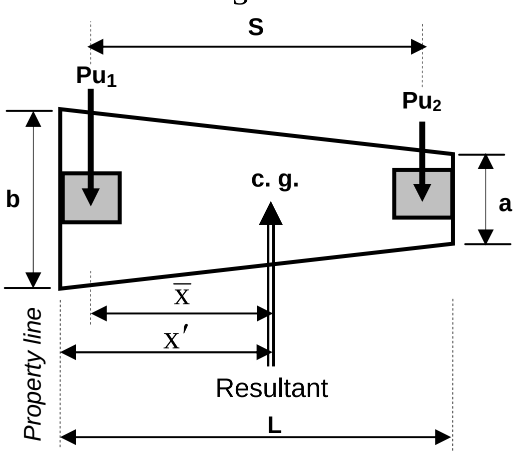


Fig. (1.10): Trapezoidal combined footing.

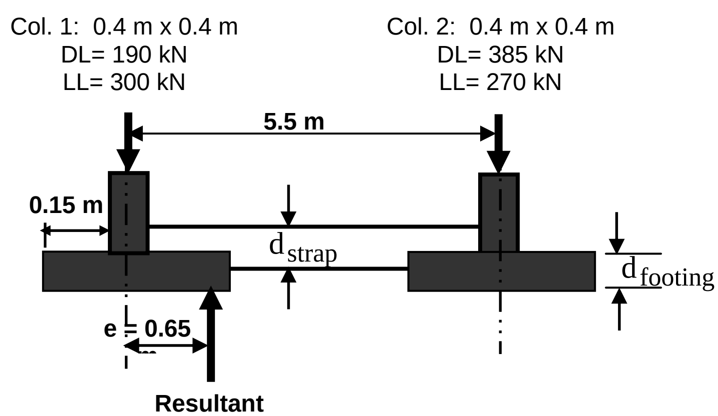


Fig. (1.11): Strap combined footing.

(3) Wall or Strip Footing *(plain or reinforced concrete only)* (see **Fig. 1.12**):

th a wall. In this case,

the footing area is calculated as: $\text{Area} = B.x.1 = \dfrac{\text{Total..load/unit..length}}{q_{all.}}$

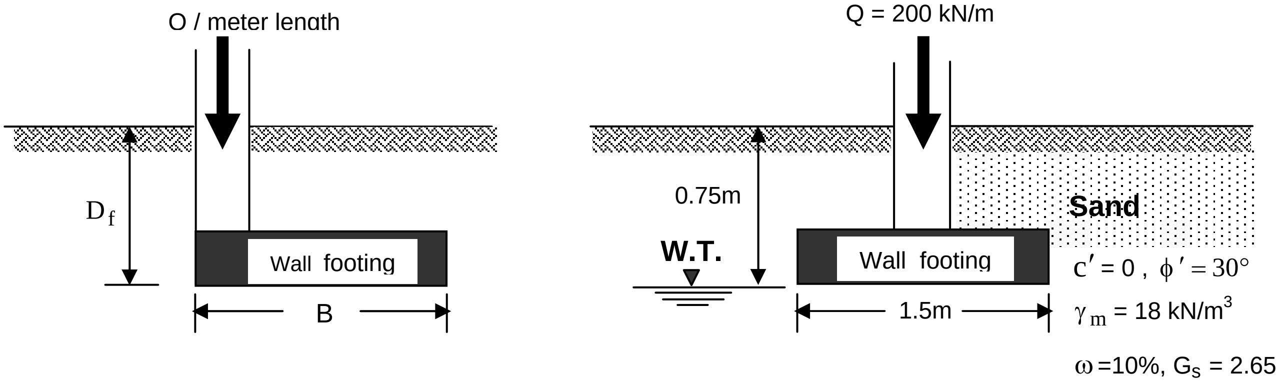


Fig. (1.12): Wall footing.

(4) Raft Foundation (see **Fig. 1.13**):

Is a combined footing that covers the entire area beneath a structure and supports all the walls and columns, such that: $\dfrac{\sum Q}{A} = q_{applied} < q_{all.}$

It is used when:

- All spread footings areas represent greater than 50 % of the entire site area,
- If there is a basement and ground water table problems, A large differential settlement is expected to occur.

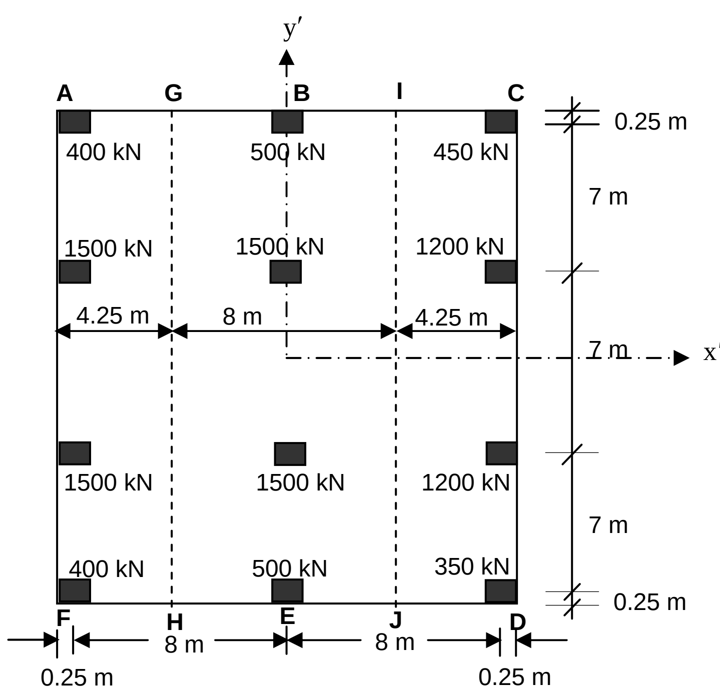


Fig. (1.13): Raft foundation.

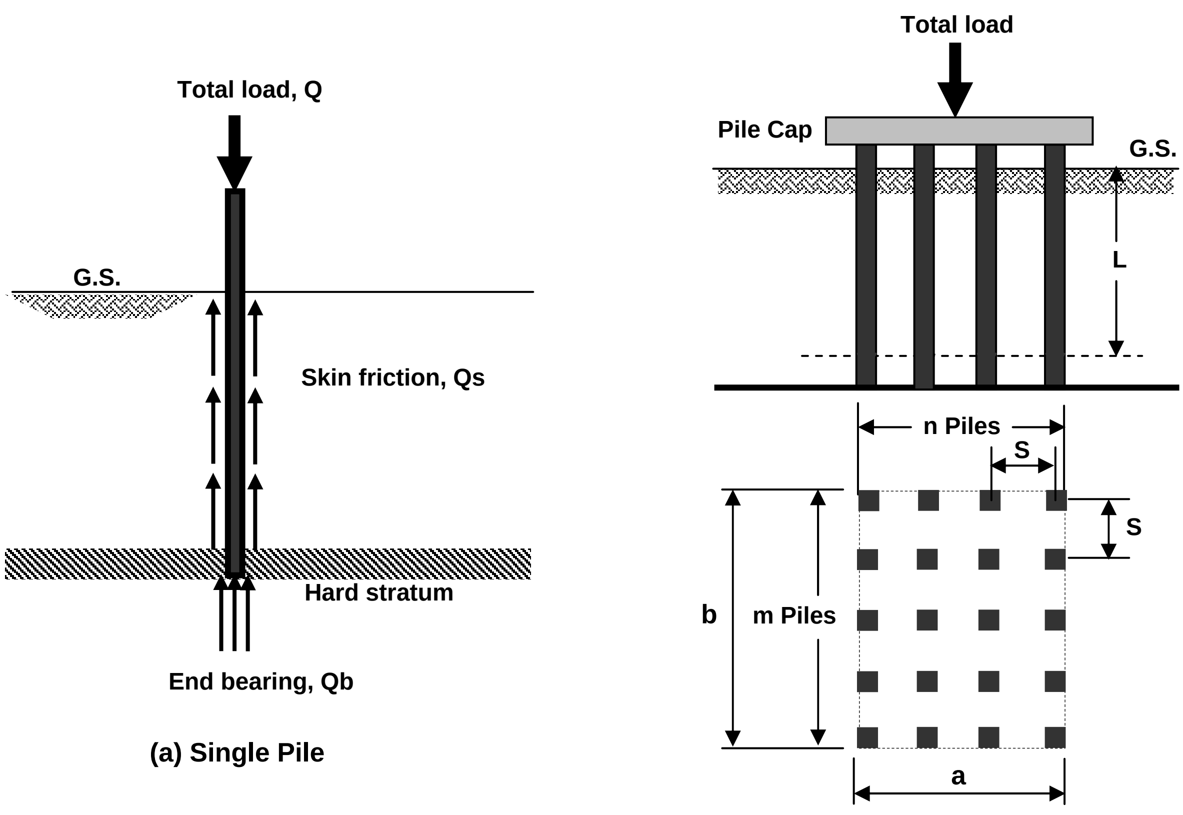
(5) Pile Foundation (see Fig. 1.14):

Pile is a structural member made of wood, steel or concrete used to transmit the load from superstructure to underlying soil stratum in the following cases:

- When the soil profile consists of weak compressible soils,
- If $q_{applied} > q_{all.}$,
- To resist tension or uplift forces induced by horizontal forces acting on superstructure due to wind or earthquakes loads.

Piles usually are of two types:

(a) Driven piles, suitable for granular soils,
(b) Bored piles, suitable for clayey soils,
Each type of these piles can be made of precast concrete or cast in place.



(a) Single Pile

(b) Layout of Piles in groups.

Fig. (1.14): Single and group piles.

(6) Pier Foundation (see Fig. 1.15):

It is an underground structural member that serves the same purpose as a footing. However, the ratio of the depth of foundation to the base width of piers is usually greater than 4 $(D_f/B > 4)$, whereas, for footings this ratio is commonly less than unity $(D_f/B \leq 1)$. A drilled pier is a cylindrical column that has essentially the same function as piles. The drilled pier foundation is used to transfer the structural load from the upper unstable soils to the lower firm stratum.

A part of the pier above the foundation is known as **a pier shaft**. The base of a pier shaft may rest directly on a firm stratum or it may be supported on piles. A pier shaft located at the end of a bridge and subjected to lateral earth pressure is known as an **abutment.**

Essentially, piers and piles serve the same purpose. The distinction is based on the method of installation. A pile is installed by driving and a pier by auger drilling. In general, a single pier is used to support the same heavy column load resisted by group of piles.

Fig. (1.15): Pier foundations.

(7) Floating Foundation:

If the weight of the constructed structure or building equal to the weight of the replaced excavated soil a foundation is known as fully compensated foundation. But if this condition is not satisfied, it is considered as semi-compensated foundation.

(8) Retaining Walls:

Retaining walls are structures used to provide stability for earth or other materials at their natural slopes. In general, they are used to support soil banks and water or also to maintain difference in the elevation of the ground surface on each of wall sides. Retaining walls are commonly supported by soil (or rock) underlying the base slab, or supported on piles; as in case of bridge abutments and where water may undercut the base soil as in water front structures. There are many types of retaining walls, each type serves different purposes and fit different requirements. They're mainly classified according to its behavior against the soil as:

(a) Gravity Retaining Walls are constructed of plain concrete or stone masonry. They depend mostly on their own weight and any soil resting on the wall for stability. This type of construction is not economical for walls higher than 3m (see **Fig. 1.16a**).

(b) Semi-Gravity Retaining Walls are modification of gravity wall in which small amounts of reinforcing steel are introduced. This helps minimizing the wall section (see **Fig. 1.16b**).

(c) Cantilever Retaining Walls are the most common type of retaining walls that used for wall height up to 8m. It derives its name from the fact that its individual parts behave as, and are designed as, cantilever beams. The stability of this type is a function of the strength of its individual parts (see **Fig. 1.16c**).

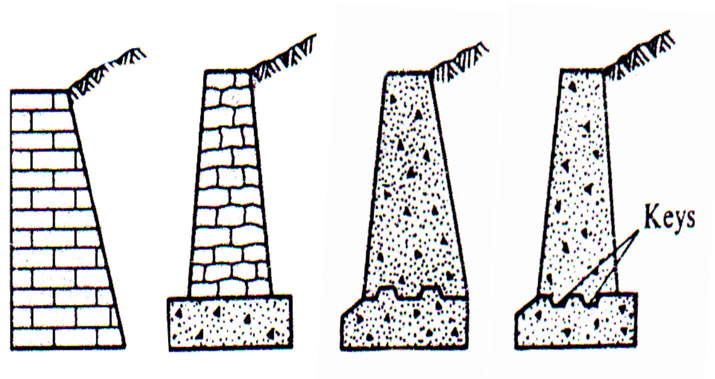
(d) Counterfort Retaining Walls are similar to cantilever retaining walls, at regular intervals, however, they have thin vertical concrete slabs behind the wall known as counterforts that tie the wall and base slab together and reduce the shear and bending moment. They're economical when the wall height exceeds 8m (see **Fig. 1.16d**).

(e) Buttress Retaining Walls this type is similar to counterfort retaining wall, except the bracing is in front of the wall and is in compression instead of tension.

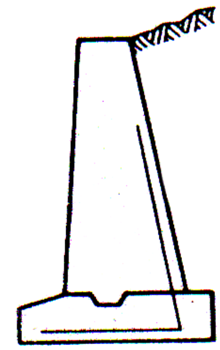
(f) Bridge Abutments are special type of retaining walls, not only containing the approach fill, but serving as a support for the bridge superstructure (see **Fig. 1.16f**).

(g) Crib Walls are built-up members of pieces of precast concrete, metal, or timber and are supported by anchor pieces embedded in the soil for stability (see **Fig. 1.16g**).

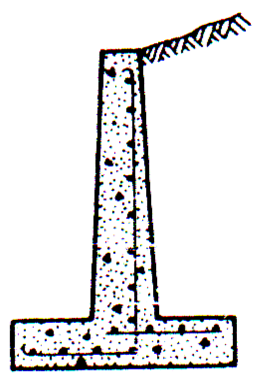
Among these walls, only the ***cantilever retaining walls*** *and* ***bridge abutments*** *are much used.*



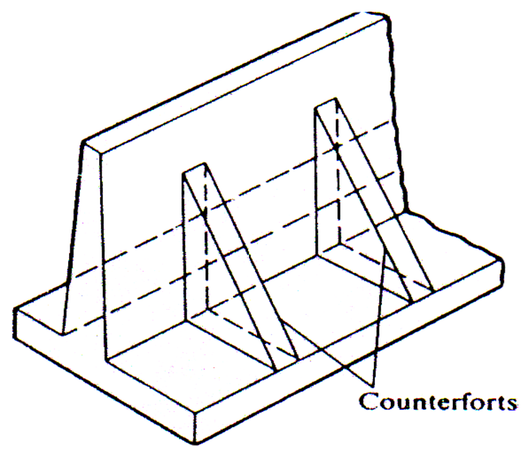
(a) Gravity walls



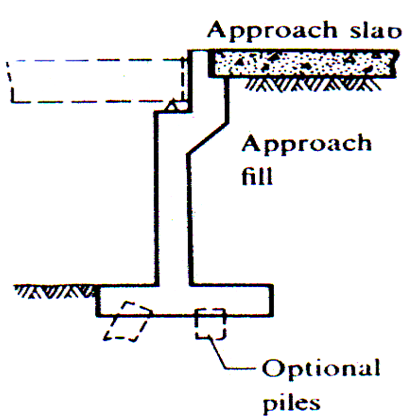
(b) Semi-gravity wall



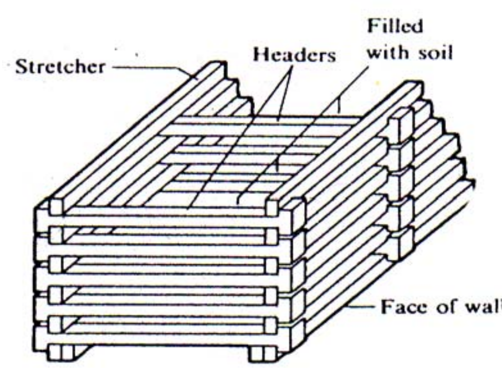
(c) Cantilever wall



(d) Counterfort wall



(f) Bridge abutment



(g) Crib walls

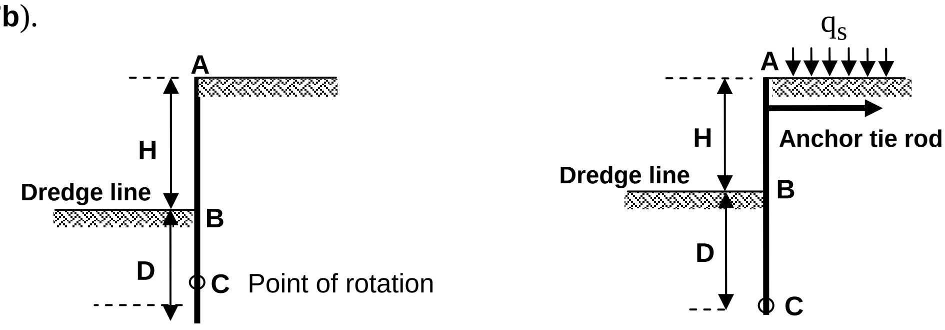
Fig. (1.16): Common types of retaining walls.

(9) Sheet Piles Walls:

These are classified as; anchored and cantilevered sheet pile walls; each kind of them may be used in single or double sheets walls.

(a) Cantilever or Free Sheet-Pile Walls are constructed by driving a sheet pile to a depth sufficient to develop a cantilever beam type reaction to resist the active pressures on the wall. That is, the embedment length which must be adequate to resist both lateral forces as well as a bending moment (see **Fig. 1.17a**).

(b) Anchored or Fixed Sheet-Pile Walls are types of retaining walls found in waterfront construction, which are used to form wharves or piers for loading and unloading ships (see **Fig. 1.17b**).

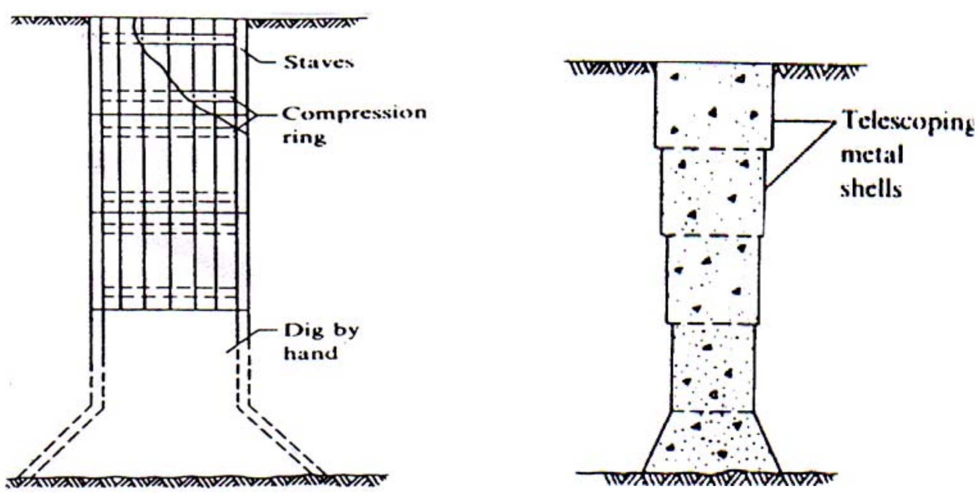


(a) Cantilever sheet pile wall. (b) Anchored sheet pile wall.

Fig. (1.17): Types of sheet piling walls.

(10) Caissons:

A hollow shaft or box with sharp ends or cutting edges for ease penetrating into soil used to isolate the site of project from the surrounding area. The material inside the caisson is removed by dredged through openings in the top or by hand excavation. Whereas, the lower part of it may be sealed from atmosphere and filled with air under pressure to exclude water from work space (see **Fig. 1.18**).



(a) The Chicago method. (b) The Gow method.

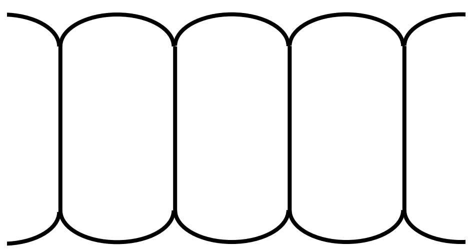
Fig. (1.18): Methods of caisson construction.

(11) Cofferdams:

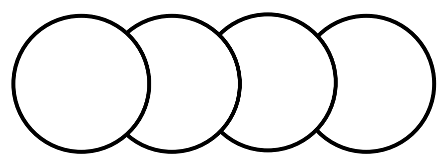
(a) Single and Double Sheet Pile Cofferdams: used for depth of water not exceeds 3.0 m.
(b) Cellular Cofferdams: used for higher depths of water, i.e., greater than 3.0 m.

These are relatively watertight enclosures of wood or steel sheet piles. Before the cofferdam is pumped out, one set of bracing is installed just above the water line. The water level is then lowered to the elevation at which another set of bracing must be installed. Successive lowering of water level and installation of bracing continue until the cofferdam is pumped out (see **Fig. 1.19**).

(a) Circular, economical for deep cells.



(b) Diaphragm, economical in quiet water.



(c) Modified circular.

Fig. (1.19: Cellular cofferdams.

CHAPTER 2

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SUBSOIL EXPLORATION

2.1 SOIL EXPLORATION

*All office, laboratory and field works*are done in order to *explore the subsurface of soil or rock conditions* at any given site to *obtain the necessary information required in design and construction*. Subsoil exploration is the first step in the design of a foundation system. Soil exploration consists essentially of boring, sampling and testing.

Mainly, planning of subsoil exploration involves three phases; reconnaissance phase, preliminary site investigation phase, and detailed site investigation phase.

2.1.1 RECONNAISSANCE PHASE

This phase consists of:
(a) Collection of all available information, and
(b) Reconnaissance of the site.
So that, it will indicate any settlement limitations and help to estimate foundation loads.

2.1.2 A PRELIMINARY SITE INVESTIGATION PHASE

This phase consists of:
(a) Preliminary design data that satisfy building code requirements, and
(b) Number and depth of boreholes.

So,it involves knowing of the distribution of structural loads which is required in the design of foundations. Also, a few borings or tests pits are to be opened to establish the stratification types of soil and location of water table. In addition to, one or more borings should be taken to rock when the initial boreholes indicate that the upper soil is loose or highly compressible.

2.1.3 A DETAILED SITE INVESTIGATION PHASE

In this phase, additional boreholes, samples will be required for zones of poor soil at smaller spacing and locations which can influence the design and construction of the foundation.

2.2 DRILLING OR BORING

- **Definition:** It is a procedure of advancing a hole into ground.
- **Drilling Methods:**

(1) Test Pits

(2) Auger Drilling
   (a) Hand-auger drilling.
   (b) Power-auger drilling.

(3) Wash Boring
   (a) Jetting.
   (b) Sludging (reverse drilling).

(4) Rotary Drilling
   (a) Rotary drilling with flush.
   (b) Rotary-percussion drilling.

(5) Percussion Drilling

Each of these methods has its merits and its drawbacks. However, Table (2.1) gives a guide for selecting the most appropriate drilling method.

Table (2.1): Drilling method selection.

Type of soil		Hand auger drilling	Wash boring		Rotary drilling		Percussion drilling
			Jetting	Sludging	Rotary drilling with flush	Rotary percussion drilling	
Gravel	Unconsolidated formations	X	X	X	X	✓ ?	✓ ?
Sand		✓	✓	✓	✓	✓ ?	✓ ?
Silt		✓	✓	✓	✓	✓ ?	✓ ?
Clay		✓	?	✓	✓	✓ slow	✓ slow
Sand with pebbles or boulders		X	X	X	X	✓ ?	✓ ?
shale	Low to medium strength formations	X	X	X	✓	✓ slow	✓
Sandstone		X	X	X	✓	✓	✓
Limestone	Medium to high strength formations	X	X	X	✓ slow	✓	✓ slow
Igneous (granite, basalt)		X	X	X	X	✓	✓ slow
Metamorphic (slate, gneiss)		X	X	X	X	✓	✓ V slow
Rock with fractures or voids		X	X	X	✓	✓	✓ !
Above water-table		✓	?	X	✓	✓	✓
Below water-table		?	✓	✓	✓	✓	✓

✓ = Suitable drilling method	✓ ? = Danger of hole collapsing	✓ ! = Flush must be maintained to continue drilling
? = Possible problems	x = Inappropriate method of drilling	

2.2.1 TEST PITS

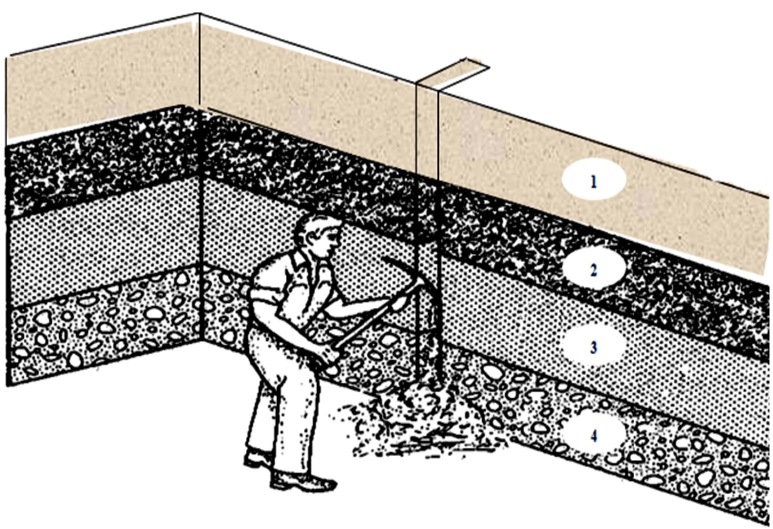
A pit is dug either by hand or by a backhoe. Probably in a test pit, the engineer can examine in detail the subsoil strata and take disturbed or undisturbed samples at the desired location (see Fig. 2.1):

Advantages:

- Inexpensive.
- Provide detailed information of stratigraphy.
- Large quantities of disturbed soils can be obtained for testing.
- Large blocks of undisturbed samples can be carved out from the pits.
- Field tests can be conducted at the bottom of the pit.

Disadvantages:

- Depth limited to about 6m.
- Deep pits uneconomical such as in case of investigationthat involves basement construction.
- Excavation below groundwater (high water table) and into rock difficult and costly.
- Too many pits may scar site and require backfill soils.
- When the soil is unstable and has a tendency to collapse, this prevents the engineer from entering the pit and accompanied by certain risks.
- Unsuitable in granular soils belowwater levelor when the standard penetration resistancetest (N-value) is required.



Walls of test pit indicate four layers
(1) Clayey silt (2) Sandy silt (3) Clean sand (4) Sandy gravel

Fig. (2.1): Test pits.

2.2.2 AUGER DRILLING

(a) Hand-Augers

The auger of (10-20) cm in diameter is rotated by turning and pushi o n the handlebar. Then withdrawing g the soil-laden auger to remove the excavated soil. Several new auger sections are added up to the required depth is reached. These augers can be available in different types such as (see Fig. 2.2):

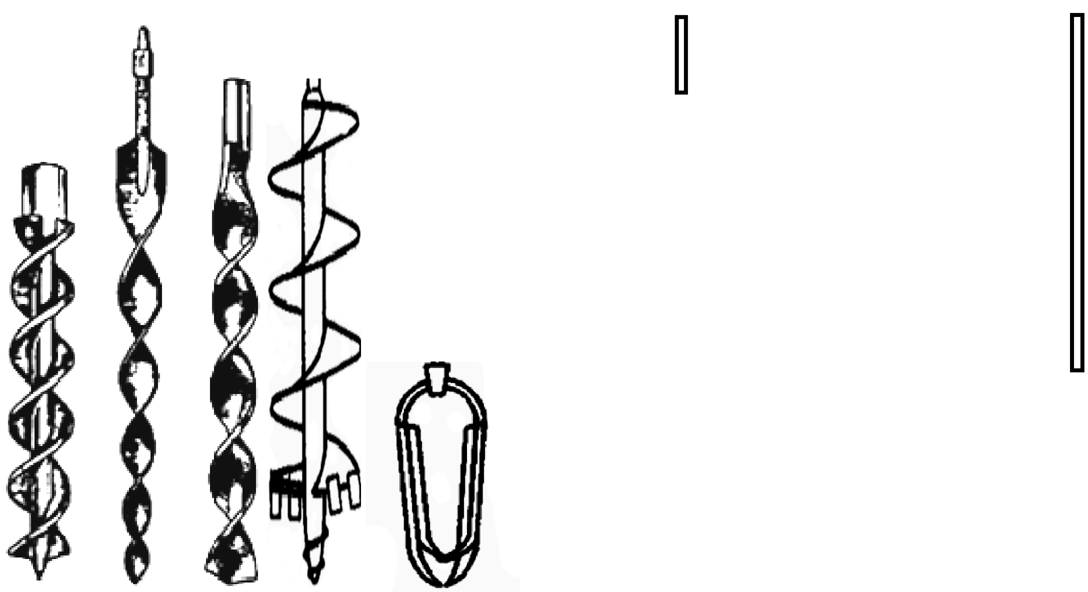
a. Helical Auger.
b. Short flight Auger, and
c. Iwan Auger.

Advantages:

- Inexpensive.
- Simple to operate and maintain.
- Not dependent on terrain.
- Portable.
- Used in uncased holes, and
- Groundwater location can easily be identified and measured.

Disadvantages:

- Slow compared with other methods.
- Depth limited to about 6m.
- Labor intensive.
- Undisturbed samples can be taken only for soft clay deposit, and
- Cannot be used in rock, stiff clays, dry sand, or caliches soils.



a. Helical (worm types) Augers **b. Shortr** **c.Iwan (posthole) Auger**

Fig. (2.2): Hand-augers.

(b) Power-Augers

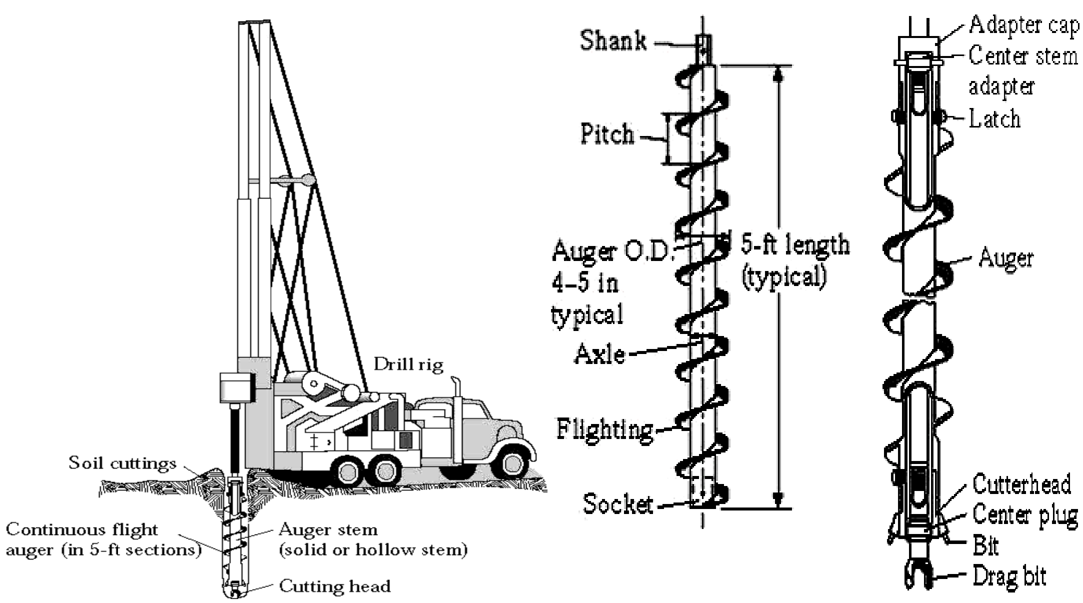
Truck or tractor mounted type rig and equipped with continuous flight at bore a hole 100 to 250 mmin di . These augers can have a solid or hollow stem of (20-75) cm in diameter (see Fig.2.3).

Advantages:

- Used in clay or sand or silt soils.
- Quick.
- Used in uncased holes, therefore no need for using drilling mud.
- Undisturbed samples can be obtained quite easily, and
- Groundwater location can easily be identified and measured.

Disadvantages:

- Depth limited to about 15m.At greater depth, drilling becomes expansive, and
- Site must be accessible to ehicle.



a. Continuous flight augers. **b. Solid-stem auger** **c. Hollow-steam auger**

Fig. (2.3): Power or mechanical-augers.

2.2.3 WASH BORING

Wate d to bottom of borehole hings are returned to surface. A drill bit is rotated and dropped to produce a chopping action (see Fig. 2.4).

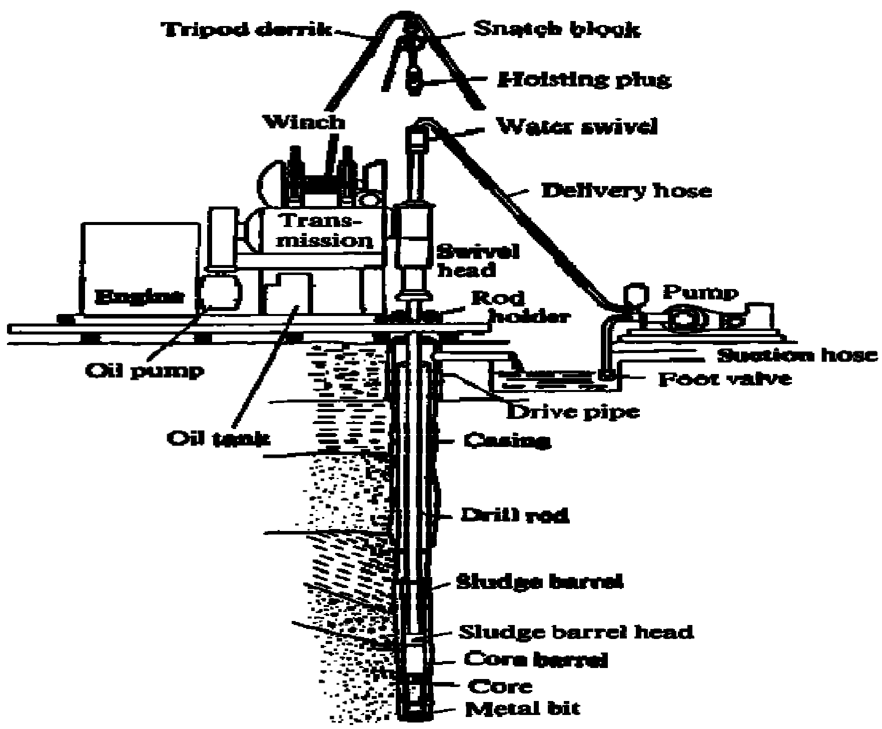


Fig. (2.4): Wash boring rig.

(a)Jetting Method

Method: Water is pumped down the centre of the drill-rods, emerging as a jet. It then returns up the borehole or drill-pipe bringing with it cuttings and debris. The washing and cutting of the formation is helped by rotation, and by the up-and-down motion of the drill-string. A foot-powered treadle pump or a small internal-combustion pump is equally suitable.

(b) Sludging(Reverse Jetting)

Method: A hollow pipe of steel is moved up and down in the borehole while a one-way valve can be used to impro lly – provid ng action. Water flows down the borehole annulus (ring) and back up the dri ing debris with it. A small reservoir is needed at the top of the borehole for recirculation. Simple teeth at the bottom of the drill-pipe, preferably made of metal, help cutting efficiency.

Advantages:

- The equipment can be made from local, low-cost materials, and it is simple to use.
- Possible above and below the water-table.
- Suitable for clay to silt clay, silt soils and unconsolidated rocks, and
- Used in uncased holes.

Disadvantages:

- Slow drilling through stiff clays and gravels.
- Undisturbed soil samples cannot be obtained. out 30m.

2.2.4ROTARY DRILLING

(a) Rotary Drilling with Flush

Method:A drill-pipe and bit are rotated to cut the rock. Air, water, or drilling mud is pumped down the drill-pipe to flush out the debris. The velocity of the flush in the borehole annulus must be sufficient to lift the cuttings (**see Fig. 2.5**).

Advantages:

- Quick.
- Can drill any type of soil or rock.
- Possible to drill to depths of over 40 meters.
- Operation is possible above and below the water-table.
- Undisturbed soil samples or rock cores can easily be recovered.
- Water and mud supports unstable formations, and
- Possible to use compressed air flush.

Disadvantages:

- Expensive equipment.
- Terrain must be accessible to motorized vehicle.
- Water is required for pumping.
- Difficulty in obtaining accurate location of groundwater level.
- There can be problems with boulders, and
- Rig requires careful operation and maintenance (additional time required for setup and cleanup).

(b) Rotary-Percussion Drilling

Method: In very hard rocks, such as granite, the only way to drill a hole is to pulverize the rock, using a rapid-action pneumatic hammer, often known as a 'down-the-hole hammer' (DTH). Compressed air is needed to drive this tool. The air also flushes the cuttings and dust from the borehole. Rotation of 10-30 rpm ensures that the borehole is straight, and circular in cross-section (**see Fig. 2.5**).

Advantages:

- Drills hard rocks.
- Possible to avel.
- Fast, and
- Operation is possible above and below the water-table.

Disadvantages:

- Higher tool cost than other tools illustrated here.
- Air compressor required, and
- Requires experience to operate and maintain.

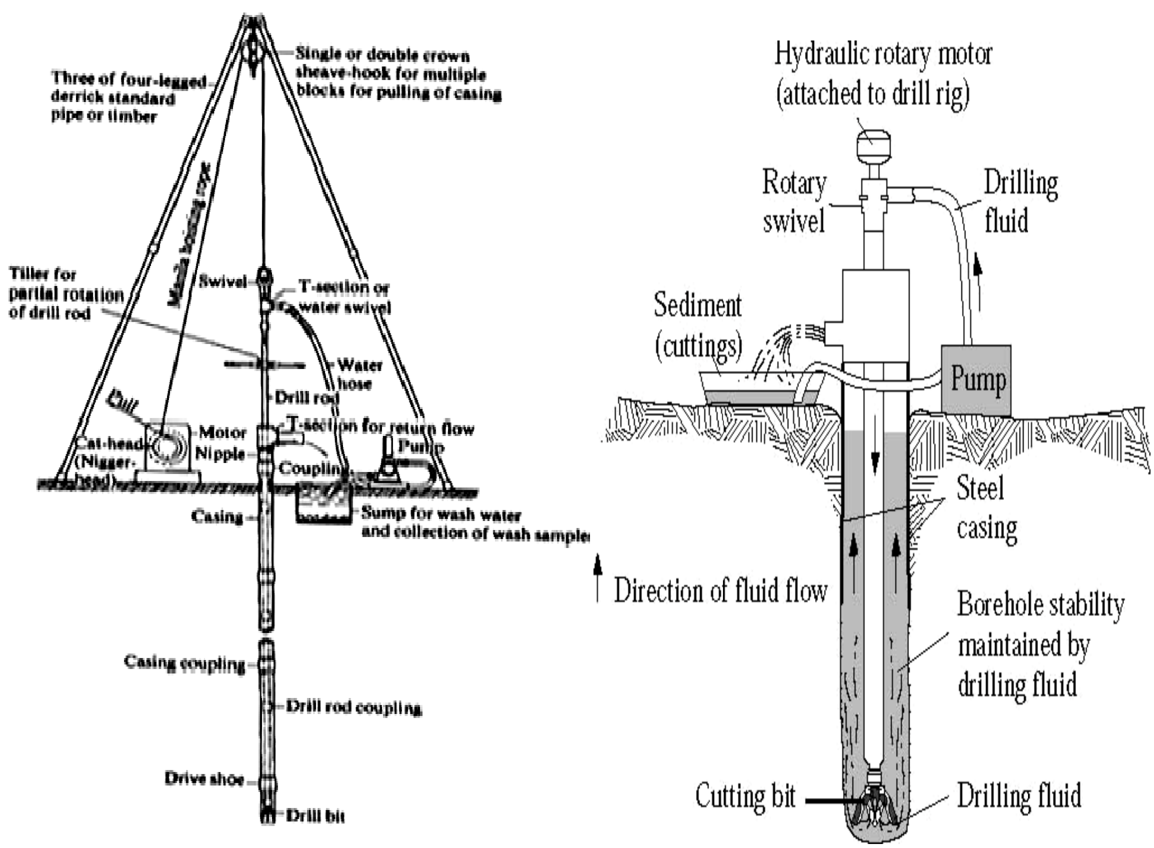


Fig. (2.5): Rotary drilling.

2.2.5 PRECUSSION DRILLING

Advantages:

- Simple to operate and maintain.
- Suitable for a wide variety of rocks.
- Operation is possible above and below the water-table.
- It is possible to drill to considerable depths, and
- Can be used for boring observation wells.

Disadvantages:

- Slow, compared with other methods.
- Equipment can be heavy.
- Problems can occur with unstable rock formations.
- Water is needed for dry holes to help remove cuttings, and
- Due to high disturbance of soil, the obtained samples can not be used for testing.

2.3UNDERGROUND WATER IN THE TEST HOLE

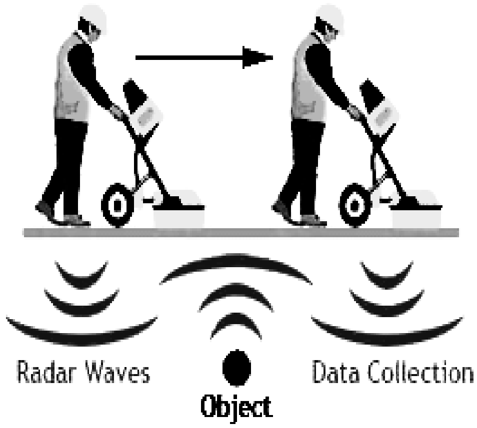
The depth of the water table (W.T.) as measured during drilling and sampling should be carefully evaluated. It is always necessary to wait for at least 24 hours to check on the stabilized water table for the final measurement.The technician should plug the top of the drill holes and flag them for identification. Care is required to ensure that the water level in the drill hole is always maintained. Any sudden drop or rise of the water table or a sudden change in the penetration resistance should be carefully recorded in the field logs of borings.

2.4 GEOPHYSICAL METHODS

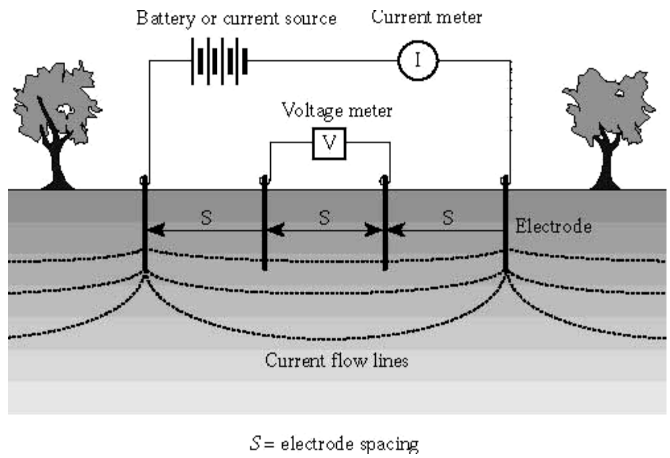
These methodsrepresent indirect methods of subsoilexploration and mainly consist of:

(1) Ground Penetration Radar (GPR). **(2) Electrical Resistivity Method (ERM)**
(2) Electromagnetic Method (EM), and **(4) Seismic Methods.**

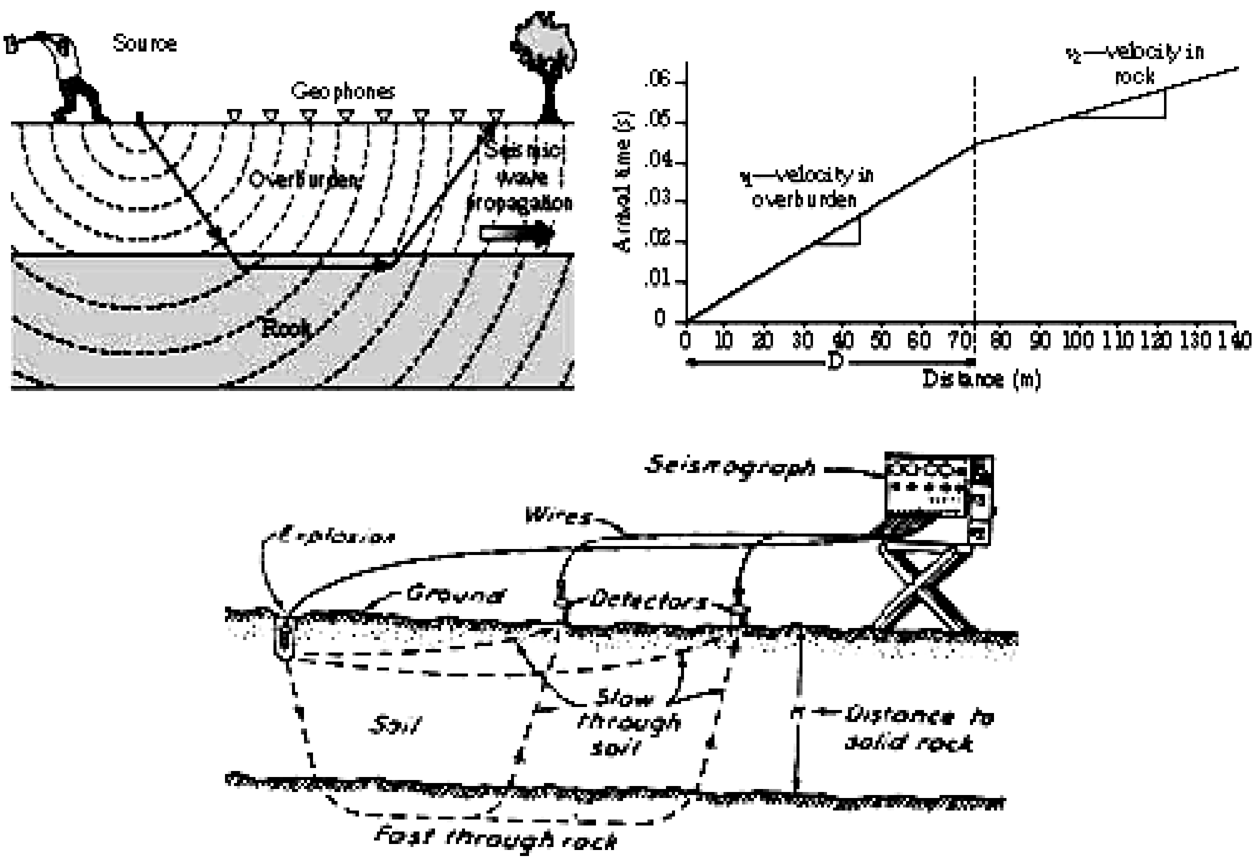
In subsoil investigation, the seismic methodsare most frequently used. Thesemethods are based on the variation of the wave velocity in different earth materials.They involve in generating a sound wave in the rock or soil, using a sledgehammer,a falling weight, or a small explosive charge, and then recording its receptionat a series of geophones located at various distances from the shot point, as shownin **Fig.(2.6)**. The time of the refracted sound arrival at each geophone is noted froma continuous reader. Typical seismic velocities of earth materials in (m/sec) are shownin **Table (2.1)**.



(b) Ground penetration radar.



(a) Electrical resistivity method.



(c) Seismic survey method.

Fig. (2.6): Geophysical methods.

Table (2.1): Typical seismic velocities ofdifferent earth materials
(after Peck, Hanson, and Thornburn,1974).

Type of soil	Seismic Velocity (m/sec)
Dry silt, silt, loose gravel. loose rocks, talus, and moist fine-grained soil	150 –180
Compacted till, indurated clays, gravel below water table, compacted clayey gravel, cemented sand, and sandy clay	750 – 2250
Rock, weathered, fractured, or partly decomposed	600 – 3000
Sandstone, sound	1500 – 4200
Limestone, chalk, sound	1800 – 6000
Igneous rock, sound	360 – 6000
Metamorphic rock, sound	300 – 4800

Requirements of seismic exploration:

- Equipment to produce an elastic wave, such as a sledgehammer used tostrike a plate on the surface.
- A series of detectors, or geophones, spaced at intervals along a li **ic exploration:**

1. Permits a rapid coverage of largeareas at a relatively small cost.
2. Not hampered by boulders andcobbles which obstruct borings, and
3. Used in regions notaccessible to boring equipment, such as the middle of a rapid river.

Disadvantages of seismic exploration:

1. Lack of unique interpretation.
2. It is particularly serious when the strata are not uniform in thickness nor horizontal,
3. Irregular contacts often are not identified, and
4. The strata of similar geophysical properties sometimes have greatly different properties.

Note:*Whenever possible, seismic data should be verified by one or two borings beforedefinite conclusions can be reached.*

2.5 SAMPLING

During the boring, three types of representative soil samples should be collected which are valuable to geotechnical engineers; these are as follows:

(a) The disturbed samples (D): which were collected from auger cuttings at specified depths?

(b)

(c) The (SS) samples: which were taken from standard split spoon sampler used in a standard penetration test (S.P.T.) that performed at different intervals depending on soil stratification.

All these samples then sealed tightly in plastic bagsto retain its in situ moisture content, labeled and transported to the soil mechanics laboratory, to perform the required tests.

Fig.(2.7)shows some details of standard split-spoon and thin-wall tube samplers that commonly used in in-situ testing and sample recovery equipment.A modification in the design of the split spoon sampler allows the insertion of brass thin-wall liners into the barrel. Four sections of brass liners (each 4 inch long) can be used. Such a de g and penetration test at the same time. This method was initiated in Californiaand known as the "California" sampler.

Samples of rock are generally obtained by rotary core drilling. Diamond core drilling is primarily used in medium-hard ks. Special diamond core barrels nch in diameter are occasionally used and larger ones can be used. Such large samples enable the geologist to study the formation and texture of the foundation rock in detail.

A summary of different sampler types which can be used to obtain disturbed or undisturbed samples of each type of soil are listed in ***Table (2.2).***

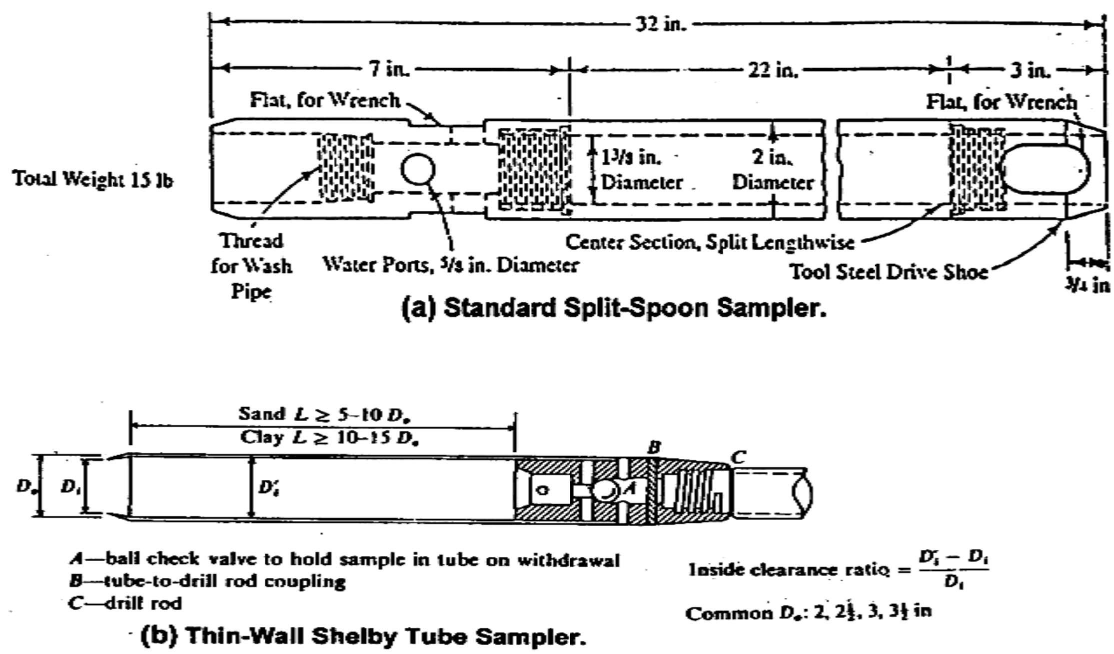


Fig.(2.7): Details of commonly used samplers for in-situ testing(*after Moore, 1980*).

Table (2.2):Types of samplers used for taking soil and rock samples from test holes.

Type of sampler	Procedure	Type of soil and Remarks
1. Highly disturbed sampler	Auger boring, wash boring, and precussion drilling.	• All types of soils, • Due to high disturbanceit is unsuitable forfoundation exploration.
2. Split spoon sampler	Standard Penetration Test.	• Cohesive, cohesionless soils and soft rocks, • For taking disturbed samples which are requiredfor physical and geotechnical analysis of
3. Thin wall Shelby tube	16gauge seamless steel tube (7.5-15)cmdia.; preferably pushed by static force instead of drivenby hammer.	• For taking undisturbed samples from cohesive soil, • Unsuitable for granular soils and hard materials.
4. Core barrel sampler: **(a)** Single tube, and **(b)**Double tube core barrel.	Rotary drilling	• For taking undisturbedcontinuous rock samples.
5. Piston samplers	Rotary drilling	• For taking undisturbed samples in soft and slightly stiff cohesive soils.
6. Hard carved samples: **(a)** Spring core catcher, and **(b)**Scraper bucket.	Cut by hand from side of test pit.	• For taking disturbed samples in cohesive or cohesionless soils.
7. Hand-cut samples	Cut by hand from side of test pit.	• For taking disturbe undisturbed block samples in cohesive soil.

2.6 SAMPLE DISTURBANCE

Certain amounts of disturbance during sampling must be regarded as inevitable:-

1. **Effect of stress relief:**
   Due to boring, the stress state in soil will be changed as a result of a stress relief.

2. **Effect of area ratio (Ar %):**
   It is the ratio of the volume of soil displacement to the volume of the collected sample.

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} x100 \qquad \textbf{(2.1)}$$

   For stiff clay < 20%, for soft clay $\leq 10\%$ and samples with $A_r > 20\%$ considered as disturbed samples.

3. **Effect of friction and adhesion:**
   If the length of sampler is large with respect to diameter, a bearing capacity failure may occur due to disturbance of sample.

$$C_i = \frac{D_o - D_i}{D_i} x100 \qquad \textbf{(2.2)}$$

where, $C_i =$ inside clearance = (0.3-0.4)% and not more than 1%.

4. **Effect of the way in which the force is applied to the spoon:**that meansby pushing or driving or by constant rate of penetration.

2.7 TESTING

The tests performed on each type of the three different soil samples are as follows:
As a rule, undisturbed samples (U) can betested fo test program of the samples includes the followings:

1. Classification Tests:
Sieve and hydrometer analysis,natural water content, Atterberge limits,specific gravity, andwet and dry unit weights.

2. Compaction Test:
Modified Procter compaction test must be carried out on some soil samples to obtain the maximum dry density $(\gamma_d^{max.})$ and the relevant optimum moisture content (OMC).

3. Shear Strength and Compressibility Tests:
Unconfined or Triaxial compressive strength test and one-dimensional consolidation test.

4. Chemical Tests:
Sulphate Content (SO_3^{-2})%,Total Soluble Salts(T.S.S.), Organic Matter Content (ORG.)%,PH- value, Carbonate Content (CO_3^{-2}), and Chlorides Content (Cl^{-1})%.

2.7.2 FIELD TESTS

During the subsoil exploration, several field tests as given in **Table (2.3)**,can be performed depending on the available testing equipments, required parameters for design of foundations, and the economic point of view.

Table (2.3): Types of field tests.

Purpose of test	Type of test
1.SPT N-value (for granular soil)	• Standard or Dynamic Penetration Test (SPT).
2.Undrained shear strength (for cohesive soil)	• Static Penetration Test (CPT) • Vane shear test (for soft to medium fine grained soil,clay and silt clay; up to Cu =1.0 kg/cm^2), obtain; $E_s ...and...G'$).
3.Bearing capacity	• Pavements: plate bearing ;CBR test, • Footings: plate bearing test, • Piles subjected to vertical loads: load test, • Batter piles: lateral load test.
4.Elastic and shear modulus	• Seismic Tests: **a-** Cross-hole, **b-** Down-hole, and **c-** Surface refraction (to measure R_D, E_s, G', liquefactionresistance and thickness of soil layers).
5. Permeability	• Pumping Test: **a-** Constant head test, **b-** Variable head test, **c-** Piezometers test (or ground water observation).
6. Compaction control	• Field or In-place Density: <u>For Sand:</u> a- Sand cone method, b- Rubber balloon method, <u>For Clay:</u> a- Penetration needle, b- Core cutter method.

2.8 LOGS OF BORINGS AND RECORDS OF TESTS RESULTS

At the beginning, a map giving specific locations of all borings should be available. Each boring should be identified (by number)and its location documented by measurement to permanent features. Such a map is shown in **Fig.(2.8)**. For each boring, all pertinent data should be recorded in the field on a boring log sheet. These sheets are preprinted fo

Depending on the results of the laboratory tests and the field observations, the actual subsoil profiles or logs of borings can more accurately be sketched (see **Fig.(2.11)**). In addition to, the actual description of soil strata in each borehole is summarized within records of tests results**.**

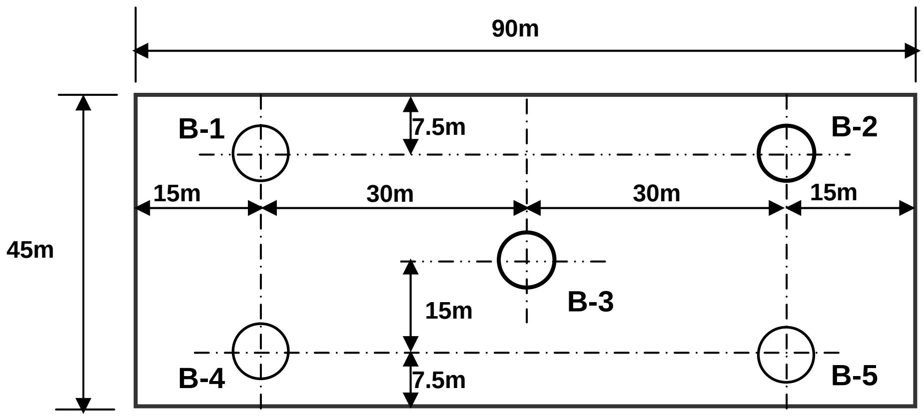


Figure (2.8): Example map showing boring locations on site plan.

DRILLING COMPANY, INC. BORE HOLE NO.: -------------
PROJECT: LOCATION: --------------------

Name --Date Time Depth Casing at
Address --- --
CASING (SIZE AND TYPE) --------------------------------
SAMPLE SPOON (SIZE AND TYPE) ---------------------
HAMMER (CSG): WT. ------------, DROP ------------------
(SPOON): WT. ------------, DROP ------------------
DATE: STARTED ---, COMPLETED ---------------------, DRILLER ---------------------

Field Samples		Depth of Sampling (m)		'N'- Value			Visual Description of Soil
No.	Type	From	To	6″	6″	6″	
1	D	0.0	2.0				Black and grey moist fill,
2	U	2.0	4.0				Black peat.
3	S.S	4.5	5.0	11	14	6	Sandy clay and silt mixture.
4	D	5.0	7.0				Sandy silt and clay mixture.
5	U	7.0	9.0				Silt fine gravel and traces of fine sand.
6	S.S	9.5	10.0	4	8	3	Sandy clay and silt mixture.

Fig.(2.9): boring log sheet.

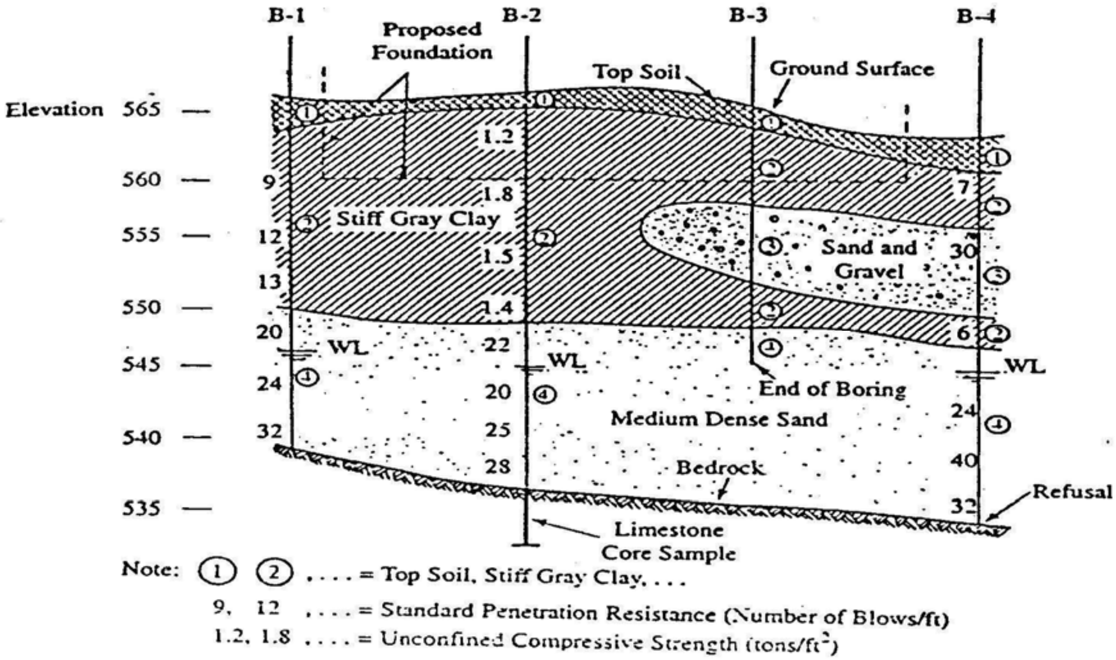


Fig.(2.10):Example of geologic profile.

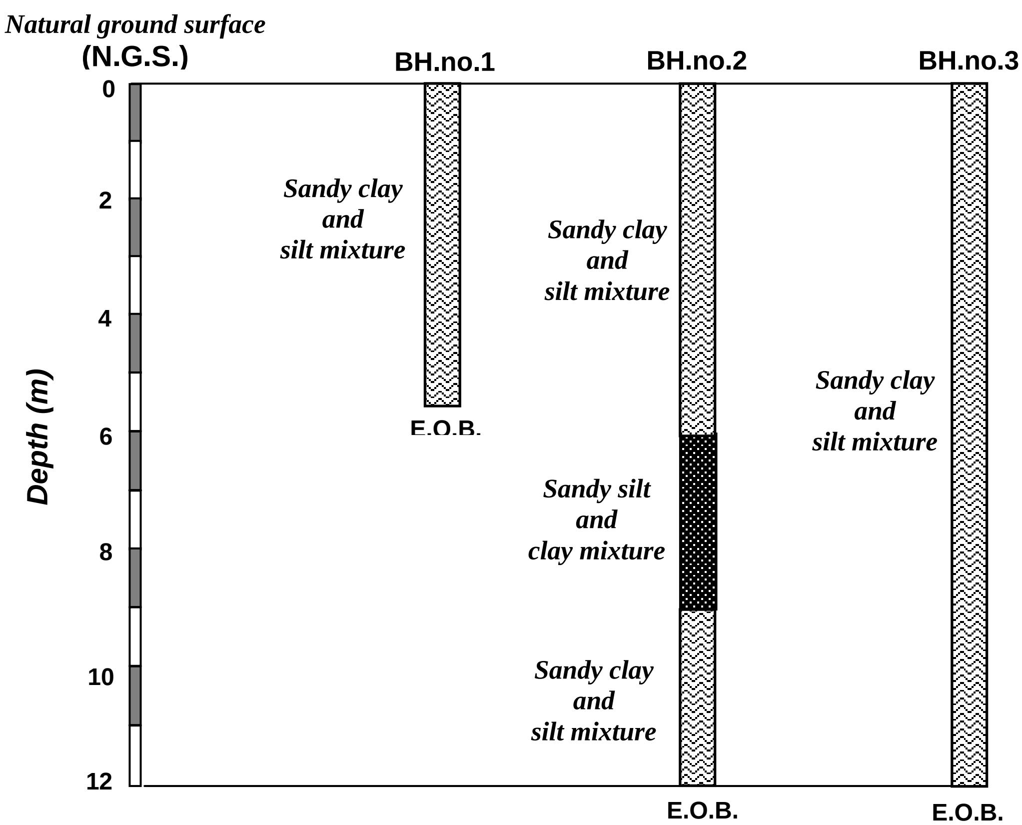


Fig.(2.11): Log of borings for 1st. stage of garden city housing project Tanahi District / Duhok city.

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RECORD OF TESTS RESULTS

PROJECT: Garden City Housing (1st. Stage)
LOCATION: Tanahi/ Duhok City
BORE HOLE NO.: 2
SHEET NO.: 1-2

1			2		3			4				5	6	7	8	9
Samples			Depth of Sampling (m)		Index Properties			Particle Size Distribution and Hydrometer Analysis				Specific Gravity 'Gs'	S.P.T 'N' Value	Symbol Unified Classification	Description of Soil	R.Q.D %
Number		Type	From	To	ω %	LL %	PI %	Gravel %	Sand %	Silt %	Clay %					
Field	Lab.															
1	100	D	0	2.0	22.8	53	27	0	10	55	35	2.70		CH	Sandy clay and silt mixture.	
2	101	U	2.0	2.5	21.7	51	27	0	8	54	38	2.70		CH	Sandy clay and silt mixture.	
3	102	S.S	2.5	3.0				0	12	54	34		12	CH	Sandy clay and silt mixture.	
4	103	D	3.0	5.0	22.2	51	27	0	10	50	40	2.72		CH	Sandy clay and silt mixture.	
5	104	U	5.0	5.5	20.1	56	28	0	8	55	37	2.72		CH	Sandy clay and silt mixture.	
6	105	S.S	5.5	6.0				0	10	44	46		32	CH	Sandy clay and silt mixture.	
7	106	D	6.0	8.0	19.1	59	32	0	13	43	44	2.73		CH	Sandy silt and clay mixture.	

	10		11	12				13							14						
	Unit Weight (kN/m^3)		Unconfined Compressive Strength qu (kN/m^2)	Shear Strength Parameters				Consolidation Test							Chemical Tests						
				Drained		Undrained															
Sample (Field)	Wet	Dry		C' (kPa)	φ'	C_U (kPa)	ϕ_U	eo	mv (m^2/kN)	Cv (m^2/year)	Cc (m^2/kN)	Cs (m^2/kN)	Pc (kN/m^2)	k (cm/sec)	SO_3^{-2} %	T.S.S %	ORG. %	PH- value	Gypsum %	$CaCO_3$ %	Cl^- %
1															0.65	0.14	2.70	8.0	----	30	0.06
2	20.94	17.21	76			38	0	0.539	0.66×10^{4}	0.371	0.073	0.045		0.77×10^{4}							
3																					
4															0.02	0.03	0.07	7.36	----	17	----
5	21.22	17.67	92			46	0	0.510	0.44×10^{4}	0.581	0.100	0.080		0.10×10^{4}							
6																					
7																					

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RECORD OF TESTS RESULTS

PROJECT: Garden City Housing (1st. Stage)
LOCATION: Tanahi/ Duhok City
BORE HOLE NO.: 2
SHEET NO.: 2-2

1			2		3			4				5	6	7	8	9	10		11	12				13							14						
Samples			Depth of Sampling (m)		Index Properties			Particle Size Distribution and Hydrometer Analysis				Specific Gravity 'Gs'	S.P.T 'N' Value	Symbol Unified Classification	Description of Soil	R.Q.D %	Unit Weight (kN/m^3)		Unconfined Compressive Strength qu (kN/m^2)	Shear Strength Parameters				Consolidation Test							Chemical Tests						
Number																				Drained		Undrained															
Field	Lab.	Type	From	To	ω %	LL %	PI %	Gravel %	Sand %	Silt %	Clay %						Wet	Dry		c' (kPa)	φ'	c_u (kPa)	ϕ_u	eo	mv (m^2/kN)	Cv (m^2/year)	Cc (m^2/kN)	Cs (m^2/kN)	Pc (kN/m^2)	k (cm/sec)	SO_3^{-2} %	T.S.S %	ORG. %	PH- value	Gypsum %	$CaCO_3$ %	Cl^- %
8	107	U	8.0	8.5	21.0	60	35	0	8	45	47	2.73		CH	Sandy silt and clay mixture.		21.60	17.85	136			68	0	0.500	0.39×10^{-4}	2.326	0.070	0.041		0.28×10^{-4}							
9	108	S.S	8.5	9.0				0	12	43	45		36	CH	Sandy silt and clay mixture.																						
10	109	D	9.0	11.0	16.4	62	34	0	6	50	44	2.73		CH	Sandy clay and silt mixture.																						
11	110	U	11.0	11.5	18	57	31	0	10	48	42	2.73		CH	Sandy clay and silt mixture.		21.20	17.96	168			84	0	0.491	1.6×10^{-4}	2.311	0.140	0.050		1.17×0^{-4}							
12	111	S.S	11.5	12.0				0	14	46	40		>50	CH	Sandy clay and silt mixture.																						

No water table is encountered at the time of boring and sampling (12/2/2007).

2.9 NUMBER OF BOREHOLES

It is a good practice in the beginning to take a few numbers of borings so that a soil profile can be drawn with reasonable accuracy and then the preliminary program can be adjusted to suit subsoil conditions.

For rough guidelines, if soil conditions are relatively uniform or the geological data are limited, Tables (2.4) and (2.5) can be used as a guide in planning of the preliminary program:

Table (2.4): Number and spacing of boreholes according to the type of project *(afterHvorslev 1949, and Road Research Laboratory 1954).*

Project	Distance between borings (m)			Minimum number of boreholes
	Horizontal stratification of soil			
	uniform	average	erratic	
Multi-story building	45	30	15	4
1 or 2 story building	60	30	15	3
Bridge, pier, abutment, Tv.Tower	----	30	7.5	1-2
Highways	300	150	30	----
Borrow pits	150-300	60-150	15-30	----
Isolated small structures:such as small houses.				1
Compact projects: such as buildings, dams, bridges or small landslips				4 deeper and closelyspaced
Extended projects:such as motorways, railways, reservoirs and land reclamation schemes.				shallower and widely spaced

Table (2.5): Number of boringsfor medium to heavy weight buildings, tanks, and other similar structures on shallow foundations*(after Sowers, 1979)***.**

Subsurface Conditions	Structure Footprint Area for Each Exploratory Boring (m^2)
Poor quality and / or erratic	100 – 300
Average	200 – 400
High quality and uniform	300 – 1000

2.10 DEPTH OF BORINGS

Hvorslev (1949) suggested a number of general rules which remain applicable:

- The soft strata should be penetrated even when they are covered with a surface layer of high bearing capacity;
- In case of very heavy loads or when seepage or other considerations are governing, the borings may be stopped when rock is encountered or after a short penetration into strata of

exceptional bearing capacity and stiffness, provided it is known from explorations in the vicinity of the area that these strata have adequate thickness or are underlain by still stronger formations. But, if these a re not satisfied, some of the borings must be extended until it has been established that the strong strata have adequate thickness irrespective of the character of the underlying material;

- When the structure is to be founded on rock, it must be verified that bedrock and not boulders have been encountered, and it is advisable to extend one or more borings from 3 to 6m into solid rock in order to determine the extent and character of the weathered zone of the rock;

For rough guidelines,the following criteriacan be used for minimum depths,from considerations of stress distribution or seepage,:

1. Foundations:

- All borings should extend below all deposits such as top soils, organic silts, peat, artificial fills, very soft and compressible clay layers;
- Boring should be sufficiently deep for checking the possibility of a weaker soil at greater depth which may settle under the applied load;
- Deeper than any strong layer at the surface checking for a weaker layer of soil under it which may cause a failure (see Fig.(2.12a));
-
- The depth at which the net vertical total stress increase due to the foundation or building load is less than 10% of the stress applied at foundation level (contact pressure);
- For isolated spread footings or raft foundations, explore to a depth equal 1.5B (B = least width of the footing or the raft)(see Fig.(2.12b));
- For group of interfering footings, explore to a depth equal 1.5B′(where, B′= width of interferingfootings)(see Fig.(2.13));
- For heavy structures (pressure > 200 kPa), the depth of borings should be extended to 2B (width of footing);
- For strip footings, explore to not less than 3B (width of footing) for $B > 6\text{m}$ and $\frac{L}{B} \geq 10$.
- For multistory buildings, explore to:

(i) $D = D_f + 3.S^{0.7}$ (in meter)……….*for light steel or narrow concrete buildings*,

(ii) $D = D_f + 6.S^{0.7}$ (in meter) ……..*for heavy steel or wide concrete buildings*.

where: D = Depth of boring, D_f =Depth of footing, and S = Number of stories.

- If piled foundation is expected, the borehole depth D = ($D_f + \frac{2}{3}$ L+ 1.5B)or D= (L + 3m) into the bearing stratum (see Fig.(2.14a));

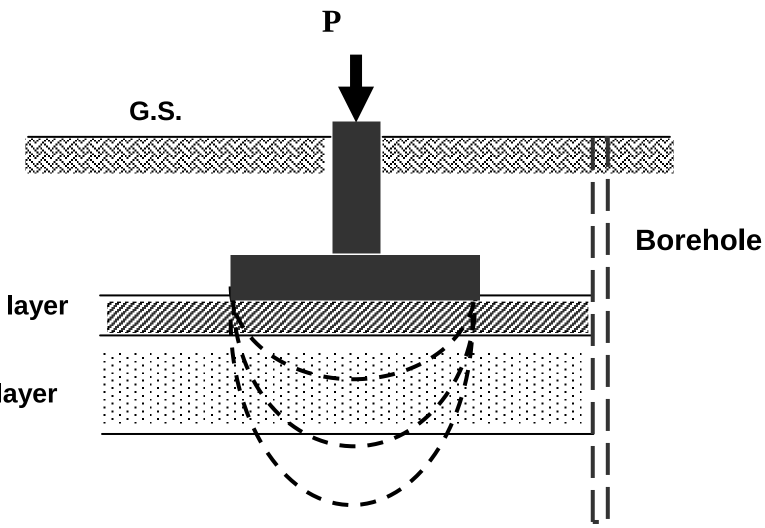
2. Reservoirs: Explore soil to:

(i) The depth of the base of the impermeable stratum, or

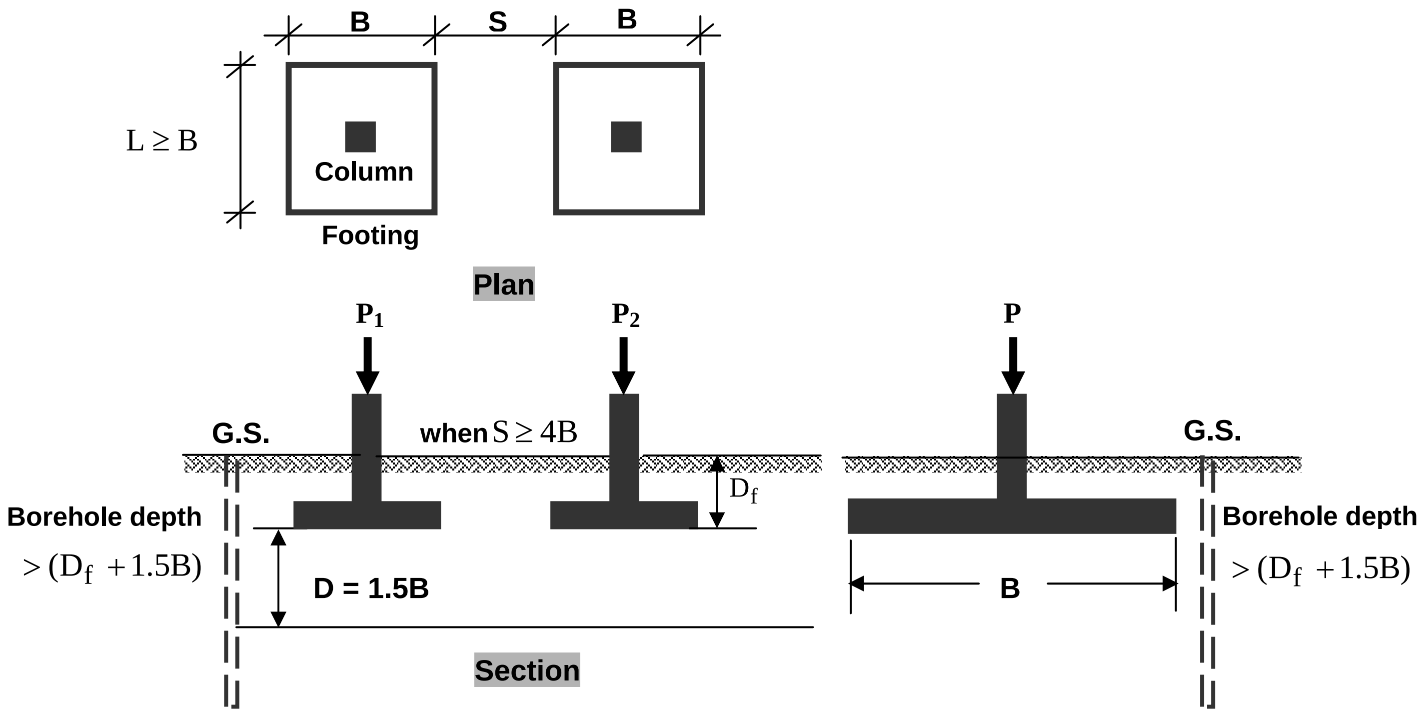
(ii) Not less than 2x maximum hydraulic head expected.

3. **Dams:** Because of thecritical factor is the safety against seepage and foundation failure, boreholes should penetrate not only soft or unstable materials, but also permeable materials to such a depth that seepage patterns can be predicted elow the finished level in cut, or
   - **(i)** B when $B \le H$, or
   - **(ii)** H when $B > H$ (see Figs.(2.15a and 2.15b)).

4. **Embankments:** The depth of exploration should be at least equal to the height of the embankment and should ideally penetrate all soft soils if stability is to be investigated. If settlements are critical then soil may be significantly stressed to depths below the bottom of the embankment equal to the embankment width (see Fig.(2.15c)).



(a) Existence of rock layer



(b) Isolated spread footing **(c) Raft or mat foundation.**

Fig.(2.12):Depth of borings for spread and raft foundations.

*

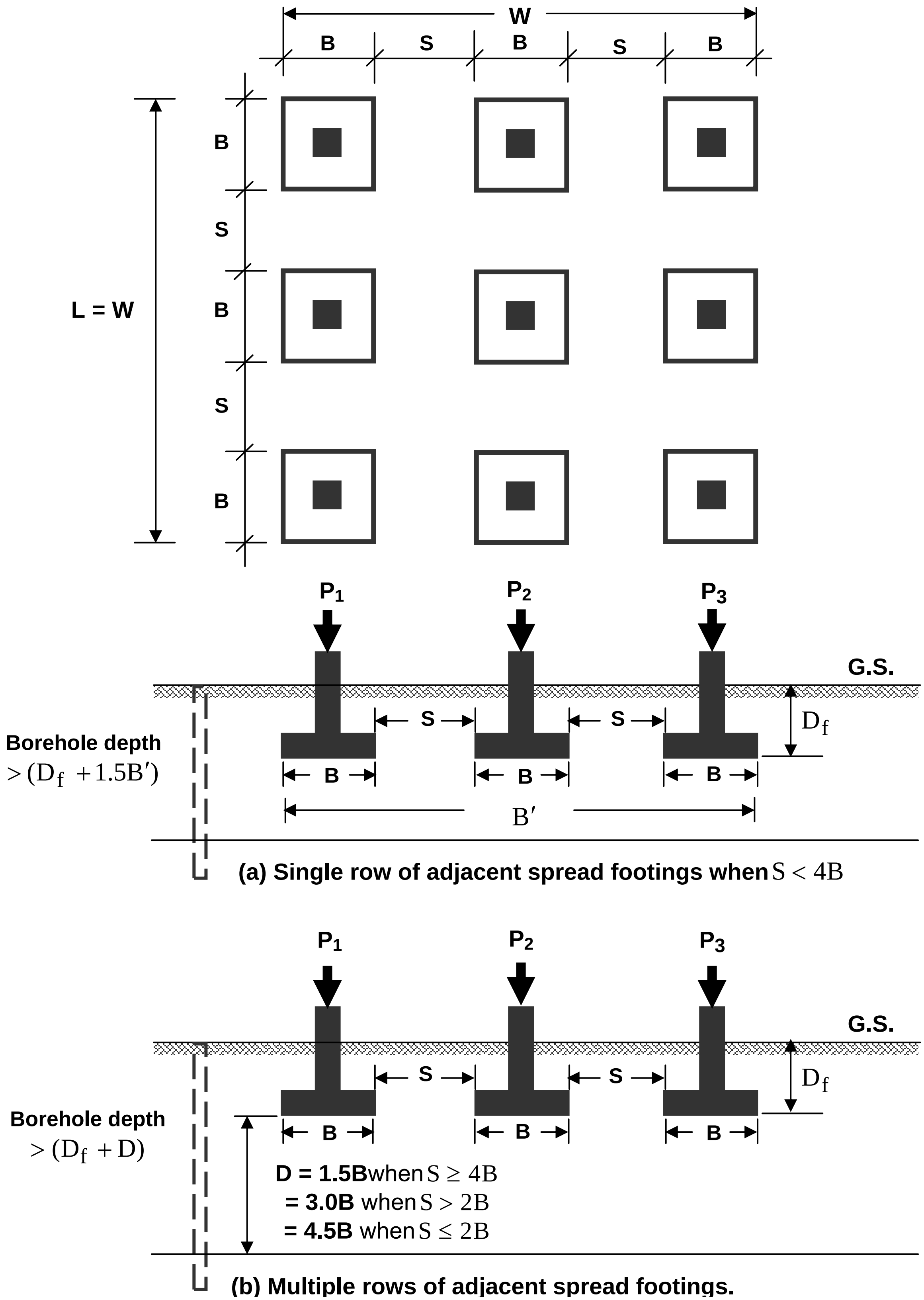


Fig.(2.13):Depth of borings for adjacent spread footings.

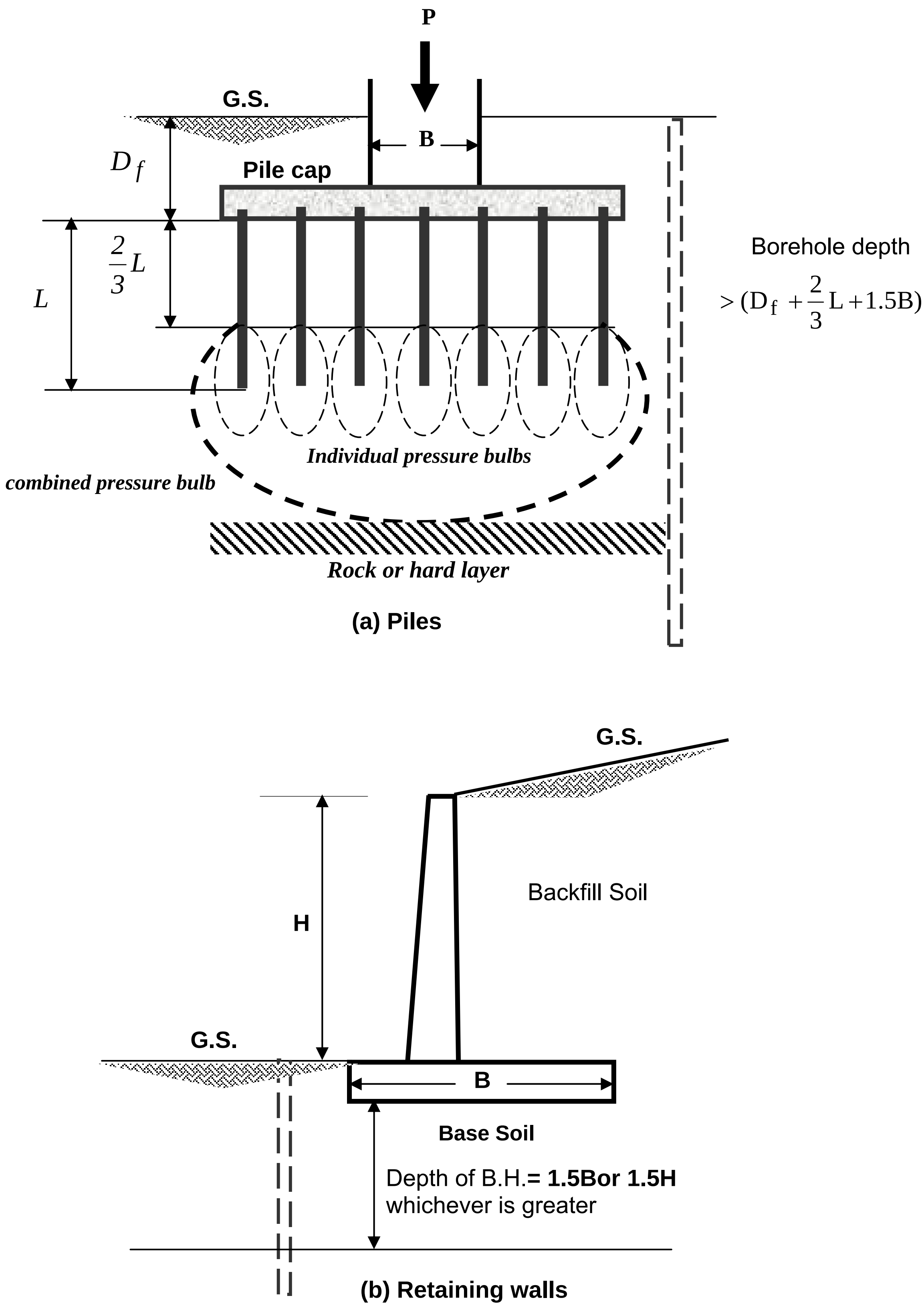
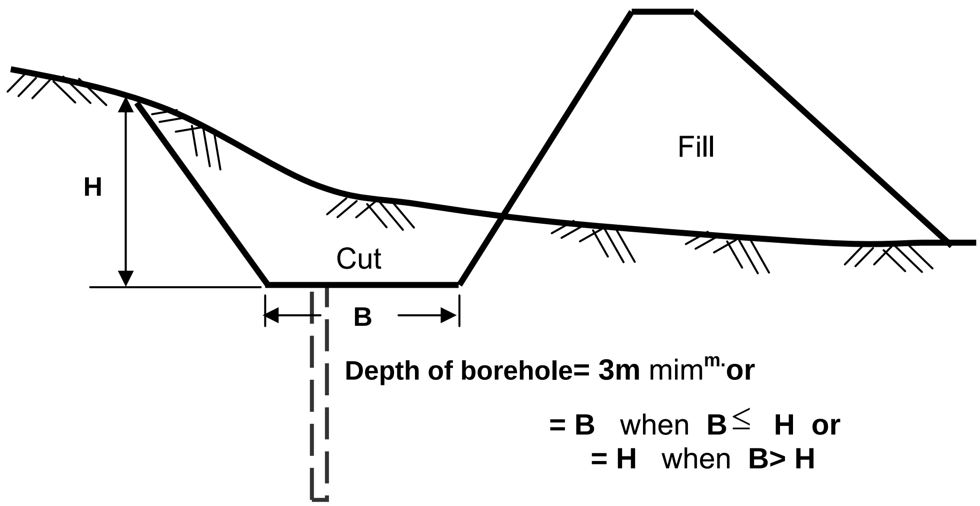
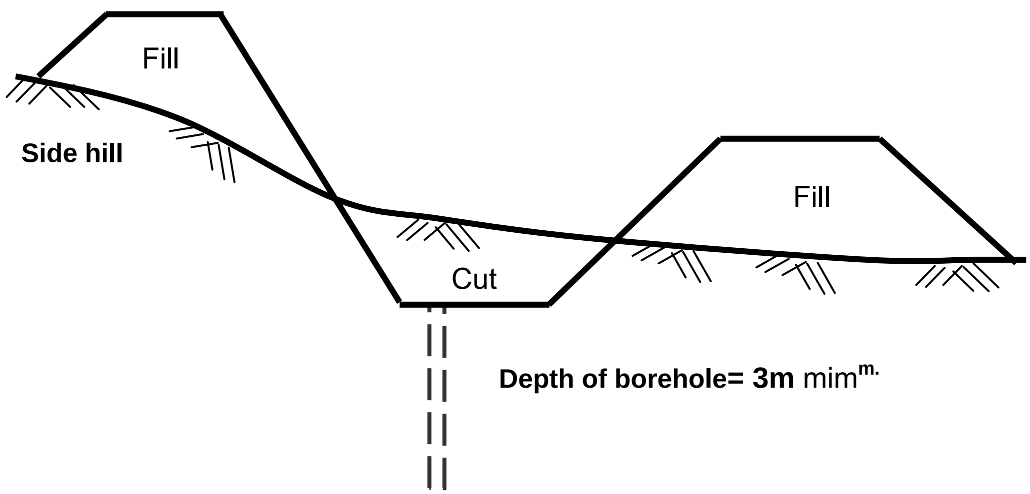


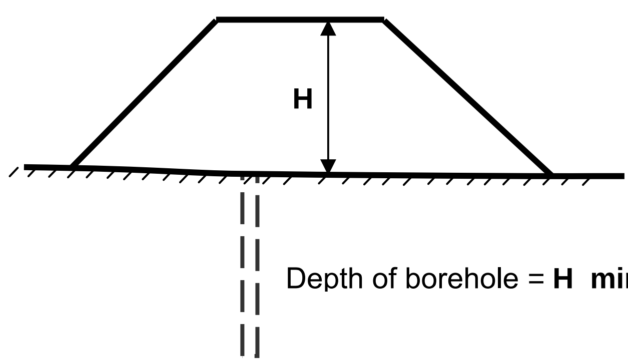
Fig.(2.14): Depth of borings for piles, and retaining walls.



(a) Deep cut and fills sections on side



(b) Normal section of a canal.



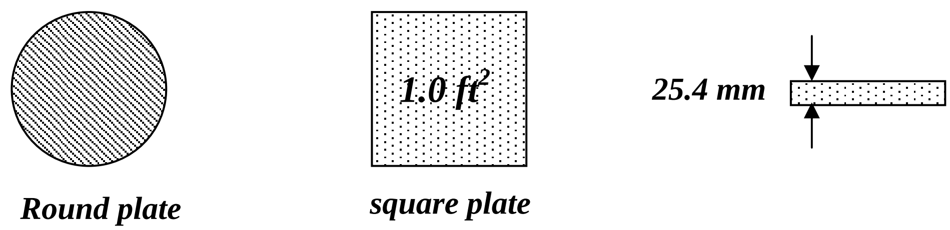
(c) High embankment

Fig.(2.15): Depth of borings for cuts and fills, canals, and embankments.

2.11 FIELD LOAD TEST

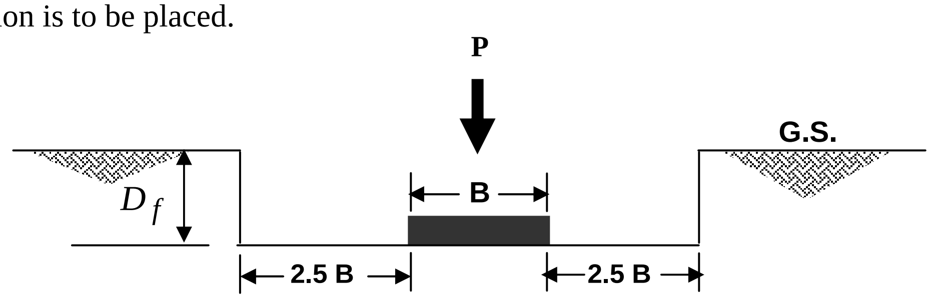
It is a method to investigate the stress-strain (or load-settlement) relationship of soils. Then, the results are used in estimating the bearing capacity. In this test, the load is applied on a model footing and the amount of load necessary to induce a given amount of settlement is measured.

Round plates from (150-750)mm in diameter by 150mm increment (i.e., 150, 300, 450, 600, 750)mm are available as well as square plates of (1.0 ft^2) area. The minimum thickness of plate (1 inch or 25.4mm).

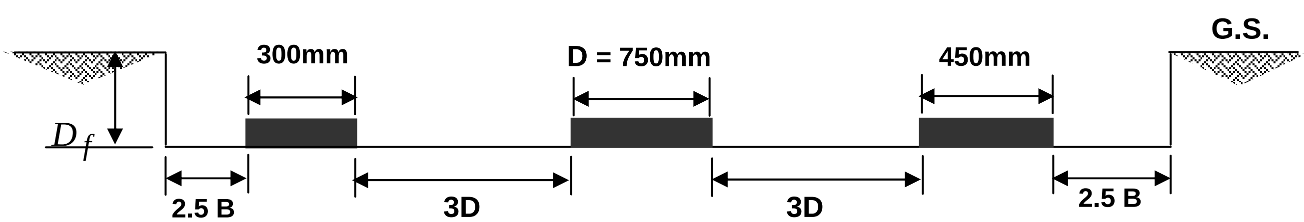


Procedure of load test as given by ASTM D110-72:

(1) Excavate a pit to width at least 6 times as wide as the used plate, and to the depth that the foundation is to be placed.



If it is specified that three sizes of plates are to be used for the test, the pit should be large enough so that, there is an available spacing between tests of 3 times the diameter (D) of the largest plate. This is useful for studying the size effect of footings.



(2) A square loading plate 2.5cm thick and (30 x 30)cm is placed on the surface of the soil at the bottom of the pit. There should not be any surcharge load placed on the soil within a distance of (60cm) from around the plate.

(3) A vertical load is placed on the plate in increments and settlements are recorded as an average from at least three dial gauges accurate to (0.025mm) that attracted to an independent suspension system. Load increment should be approximately 1/10 of the estimated allowable soil pressure. For each load increment, settlement readings should be taken at regular intervals of not less than (1 hr.) until there is no further settlement. The same time duration should be used for all the loading increments.

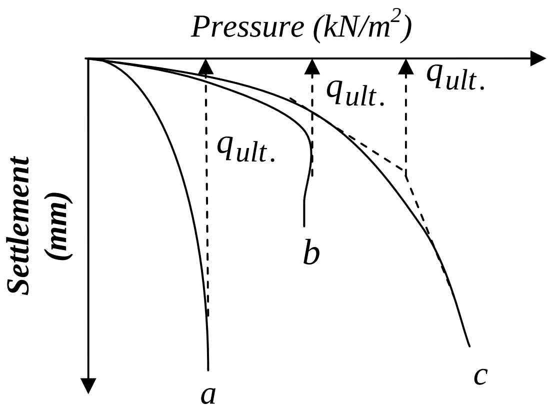
(4) The test is continued until a settlement of 25mm is observed or until the load increments reached 1.5 times the estimated allowable soil pressure.

(5) If the load is released, the elastic rebound of the soil should be recorded for a periods of time equal to the same time durations of each applied load increment.

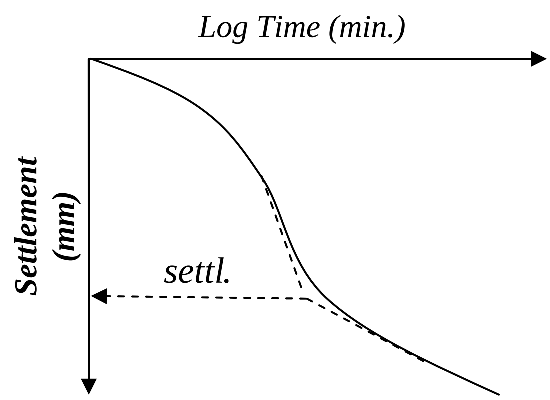
(6) The result of each test can be represented graphically as follows:-

(a) Settlement versus log time curve (for each load increment),

(b) Load-settlement curve (for all increments) from which $q_{ult.}$ is obtained.



(a) Load - settlement curve



(b) Log time-Settlement curve

Fig.(2.16): Typical load test results.

- **For cohesive soil***(bearing capacity is independent of footing size):*

$$\begin{cases} q_f = q_p \\ s_f = s_p \dfrac{B_f}{B_p} \end{cases} \quad \text{.......... (2.3)}$$

- **For cohesionless soil***(bearing capacity increases with size of footing):*

$$\begin{cases} q_f = q_p \dfrac{B_f}{B_p} \\ s_f = s_p \left(\dfrac{2B_f}{B_p + B_f} \right)^2 \end{cases} \quad \text{.......... (2.4)}$$

- **Settlement for both cohesive and cohesionless soils:**

$$\frac{s_f / B_f}{s_p / B_p} = (\frac{B_f}{B_p})^n \quad \text{.......... (2.5)}$$

where, s_f and s_p are settlements of footing and plate, B_f and B_p are their respective widths; provided that B_p = 1.0 ft for $\frac{B_f}{B_p} \geq 5$ as well as the footing and plate carries the same intensity of load, and (n) is an exponent depends on soil type; with some of its values are:

Type of soil	n
Clay	0.03-0.50
Sandy clay	0.08-0.10
Dense sand	0.40-0.50
Medium sand	0.25-0.35
Loose sand	0.20-0.25

- **For $c-\phi$ soils (*bearing capacity from two-plate load tests;* after Housel, 1929*):***

$$V = A.q + P.s \qquad \textbf{(2.6)}$$

where,

V = total load on a bearing area,
A = contact area of footing or plate,
q = bearing pressure beneath A,
P = perimeter of footing or plate, and
s = perimeter shear.

This method needs data from two-plate load tests so that Eq.(**2.6**) can be solved for q and s (for given settlement). After the values of q and s are known, then, the size of a footing required to carry a given load can be calculated.

2.12 FIELD PENETRATION TESTS

2.12.1 DynamicorStandard Penetration Test (SPT)

This test is preferred for very hard deposits, particularly of cohesionless soils for which undisturbed samples cannot easily be obtained. It utilizes a split-spoon sampler shown previously in **Fig.(2.7a)** driven into the soil.

The test of driving the standard split-barrel sampler of dimensions (680mm length, 30mm inside diameter and 50mm outside diameter) a distance of 460mm (18") into the soil at the bottom of the boring. This was done by using a 63.5kg (140Ib) driving mass (or hammer) falling "free" from a height of 760mm (30"). Then, counting the number of blows required for driving the sampler the last 305mm (12") to obtain the (N) number (neglecting the no. of blows for the upper first 150mm).

Note: The SPT- value is rejected or halted in any one of the following cases:

(a) if50 blows are required for any 150mm increment, or
(b) if100 blows are obtained, or
(c) if10 successive blow produce no advance.

The number of blows (N) can be correlated with the relative density (D_r) of Cohesionless soil (sand) and with the consistency of cohesiv

SPT- value N/30cm	Relative density $D_r = \frac{e_{max} - e_{insitu}}{e_{max} - e_{min}} x100$		$\phi°$
0-4	0-15	Very loose	28
4-10	15-35	Loose	28-30
10-30	35-65	Medium	30-36
30-50	65-85	Dense	36-41
> 50	85- 100	Very dense	> 41

Table (2.7): Relation of consistency of clay, SPT N-value, and unconfined compressive strength (q_u).

SPT- value N/30cm	consistency	q_u (ksf)	q_u (kg / cm^2)
Below	Very soft	0-0.5	0-0.25
2-4	Soft	0.5-1	0.25-0.5
4-8	Medium	1-2	0.5-1
8-15	Stiff	2-4	1-2
15-30	Very stiff	4-8	2-4
> 30	Hard	> 8	> 4

2.12.2Corrections for N-value

(1) W.T. Correction (in case of presence of W.T.):

For N > 15: $N_{corr.} = 15 + 0.5(N_{field} - 15)$ ……..…………………………..…**(2.7)**

and

For N $\leq$ 15: $N_{corr.} = N_{field}$ ……..……………..….....……..……………..…**(2.8)**

• ***If N-value is measured above water table, no need for this correction.***

(2) Overburden pressure, C_N ; Energy ratio, η_1 ; Rod length, η_2 ; Sampler; η_3 ; and Borehole dia., η_4 Corrections:

$N'_{70} = N_{field}.C_N..\eta_1..\eta_2..\eta_3..\eta_4$ ……………………..……………..……………..…….**(2.9)** where,

N'_{70} = corrected (N) using the subscript for the energy ratio E_{rb} and (') to indicate it has been adjusted or corrected,

C_N = adjustment for overburden pressure for $\overline{p} \geq 25.(\text{kPa})$ and can be calculated from the following formula:

$$C_N = 0.77 \log \frac{2000}{\overline{P}_o} \quad \text{...} \textbf{(2.10)}$$

- ***If*** $\overline{p} < 25.(\text{kPa})$***, no need for overburden pressure correction.***

where,

$\overline{p}_o$ = overburden pressure in (kPa),

η_i : *factors obtained from* **(Table 2.9)** *as*:

η_1 = hammer correction = (average energy ratio)/(drill rig energy) = $_r$ /

E E_{rb} ; η ehole diameter correction.

Table (2.9): Hammer, borehole, sampler, and rod η_i correction factors.

Hammer correction η_1					
Average energy ratio E_r					R-P = Rope -Pulley: $\eta_1 = E_r / E_{rb}$ For USA trip/auto w / E_r = 80 η_1 = 80/70 = 1.14
Country	Donut		Safety		
	R-P	Trip	R-P	Trip	
USA					
North America	45	----	70-80	80-100	
Japan	67	78	------	-------	
UK	----	----	50	60	
China	50	60	------	-------	
Rod length correction η_2					
Length	> 10m 6-10 4-6 0-4		η_2 = 1.00 = 0.95 = 0.85 = 0.75		N is too high for L <10 m
Sampling method correction η_3					
Without liner: With liner: Dense sand, Clay Loose sand.			η_3 = 1.00 = 0.80 = 0.90		N is too high with liner
Borehole diameter correction η_4					
Hole diameter	60-120 mm 150 mm 200 mm		η_4 = 1.00 = 1.05 = 1.15		N is too small for oversize hole

Notes:

1. It is evident that all $\eta_i = 1.0$ for the case of a small borehole, no sampler liner, length of drill rod >10 m and the given drill rig has $E_r = 70$. In this case the only adjustment is for overburden pressure (i.e., $N_{corr.} = N_{field}.C_N$).
2. Large values of E_r decrease the blow count (N) linearly (i.e., $N_2 = \frac{E_{r1}}{E_{r2}}..N_1$). This equation is used to convert any energy ratio to any other base.
3. If $N_{field} = 10...blows/10cm$, then $N_{corr.} = 10.\left(\frac{30}{10}\right) = 30...blows/30cm.$

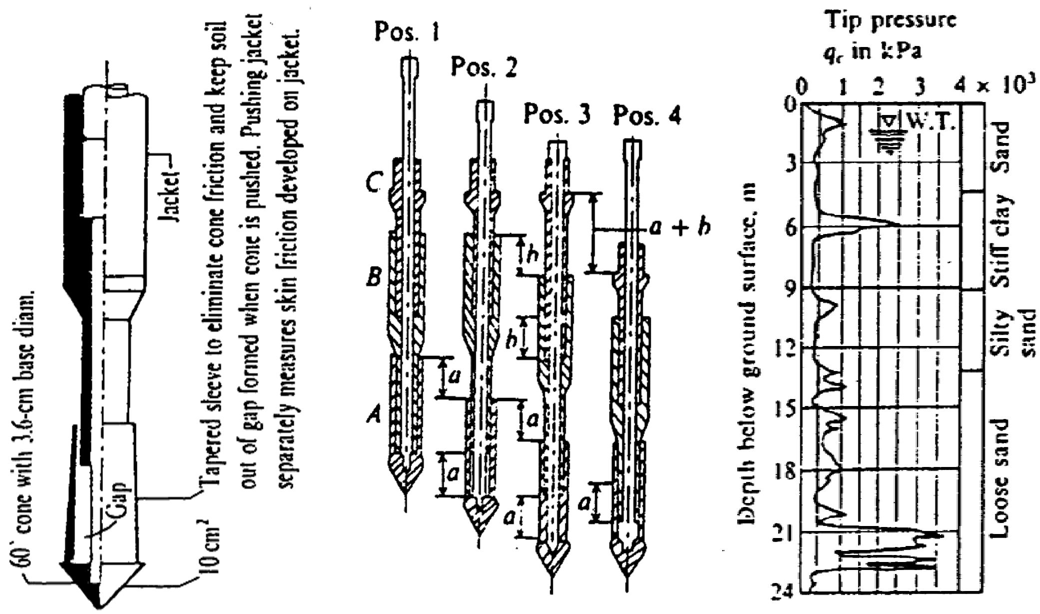
2.12.3Staticor ConePenetration Test (CPT)

1. Mechanical or the earliest "Dutch Cone Type",
2. Electric friction with strain gauges,
3. Electric piezo for pore water measurement,
4. Electric piezo/friction to measure q_c, q_s and u or (pwp), and
5. Seismic cone to compute dynamic shear modulus.

Fig.(2.17b) shows the operations sequence of a mechanical cone as: in position (1) the cone is seated; position (2) advances the cone tip to measure q_c; position (3) advances the friction sleeve to measure q_s ;and position (4) advances both tip and sleeve to measure $q_t = q_c + q_s$ .Therefore, at any required depth, the tip and sleeve friction resistances q_c and q_s are measured and then used to compute a friction ratio f_R as:

$$f_R(\%) = \frac{q_s}{q_c} x100 \; ; \; f_R < 1\% \text{ for sands; } f_R > 5 \text{ or } 6\% \text{ for clays and peat.}$$

The data collected from the CPT can be correlated to establishtheundrained shear strength S_u of cohesive soils,allowable bearing capacity of piles, to classify soils; and to estimate $\phi,..D_r$ for sands. A typical data set is shown in **Fig.(2.18b)**.

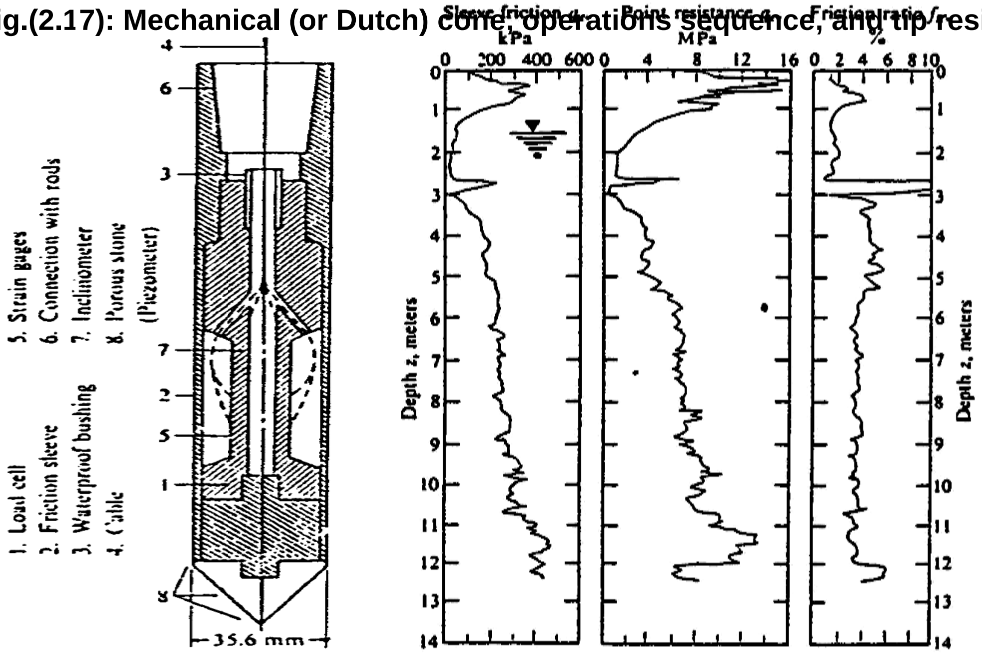


(a) Dutch cone modified to measure both point resistance q_c and skin friction q_f

(b) Positions of the Dutch cone during a pressure record.

(c) Typical output.

Fig.(2.17): Mechanical (or Dutch) cone, operations sequence, and tip resistance data.



(a) Piezocone.

(b) Cone Penetration record for clay soil.

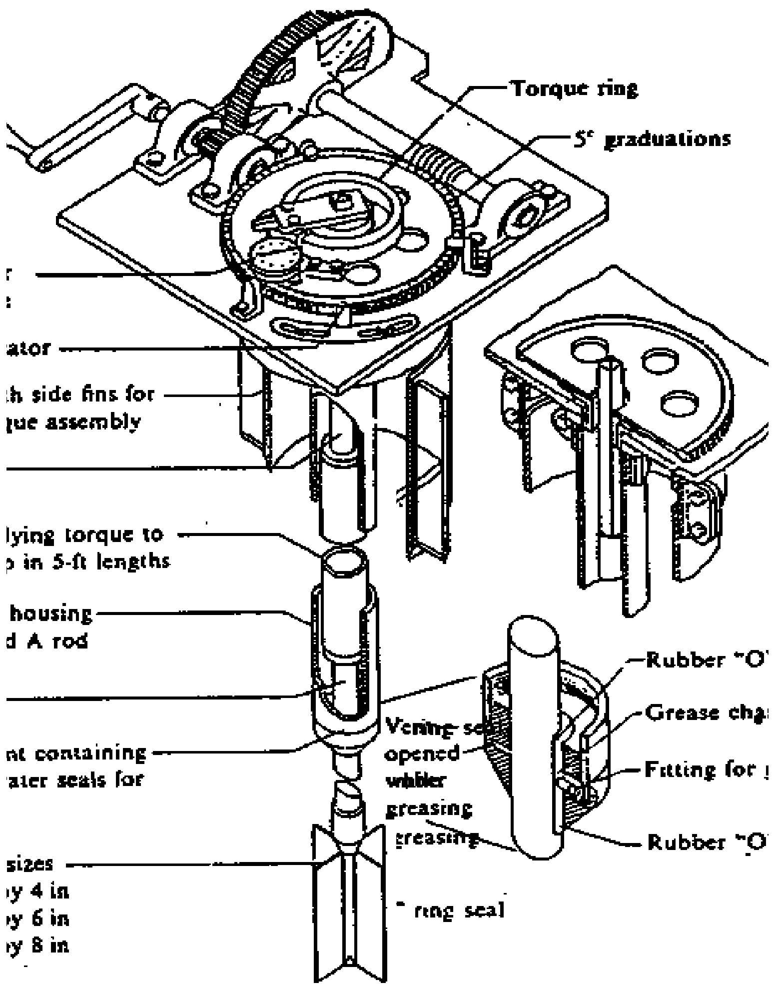
Fig.(2.18): Electric cone and CPT data.

2.13 VANE SHEAR TEST

It is a field test used to determine the in-situ shearing resistance(undrained shear strength) of soft to medium clay and silt clay having U.C.S.< 1.0 (kg/cm^2), then to be used for design of foundations and slopes.

- **Apparatus(see Fig.(2.19):**

1. Van shear test equipment;
2. Drilling rig;
3. Casing (as required); and
4. Other necessary tools and supplies such as stop watch, pipe,… etc..



u of Reclamation vane-shear test aparatus. [*Gibbs et al.* (1960).
bbs and Holtz of the USBR.)

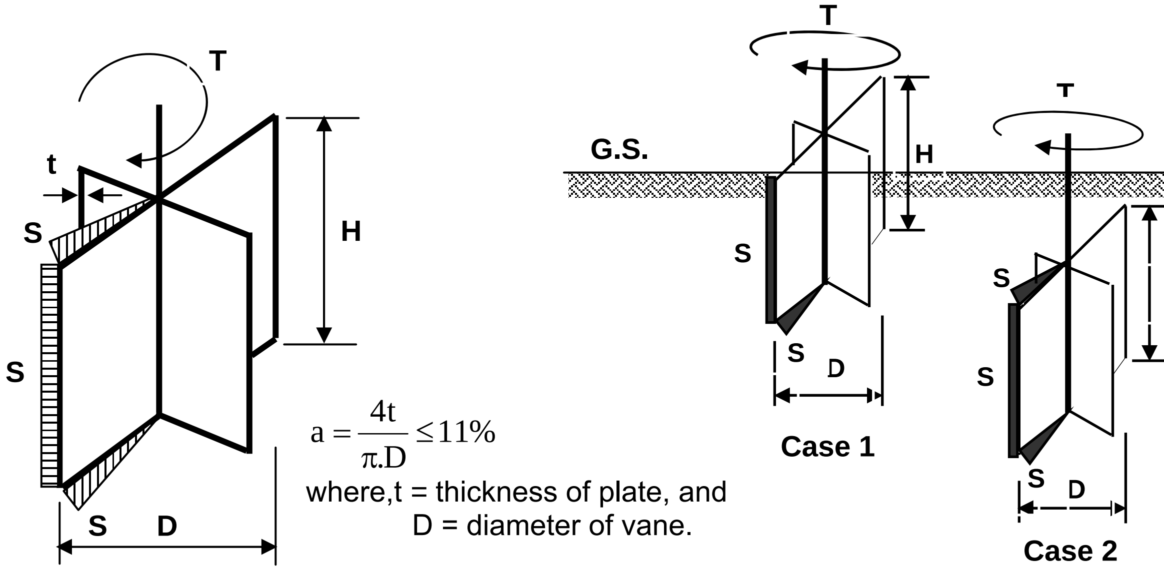
Fig.(2.19): Va pparatus.

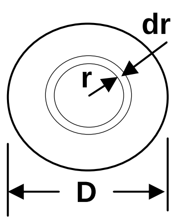
- **Procedure:**

1.

2. A torque is applied at a uniform rate of 0.1^o per sec. or (1^o-6^o per minute).

3. Readings are taken each minute interval until failure happens.





Standard dimensions of vane B.S. 1377 Rate of test (6 -12)deg./ min.		
Soil strength (kPa)	H (mm)	D (mm)
< 50	150	75
50-75	100	50
> 75	Not suitable	

Fig.(2.20): Vane shear standards.

- **Calculation:**

(i)Case (1): In this case, the vane is not embedded in soil, so that only the bottom end takes pant in shearing. If the soil is isotropic and homogenous, then:

(a) Total shear resistance at failure developed along cylindrical surface = $\pi.D.H.S$

(b) Totalresistance of bottom ends, considering a ring of radius r and thickness dr

$$= \int_0^{D/2} (2\pi.r.dr).S$$

(c) The torque T at failure will then equal: $T = (\pi.D.H.S)\frac{D}{2} + \int_{0}^{D/2}(2\pi.r.dr).S.r$

$$\text{or } T = \frac{\pi.D^2 S_u}{2}(H + \frac{D}{6}) \quad \text{..}\mathbf{(2.11)}$$

(ii) Case (2): If the top end of the vane is also embedded in soil, so shearing takesplace on top and bottom ends:

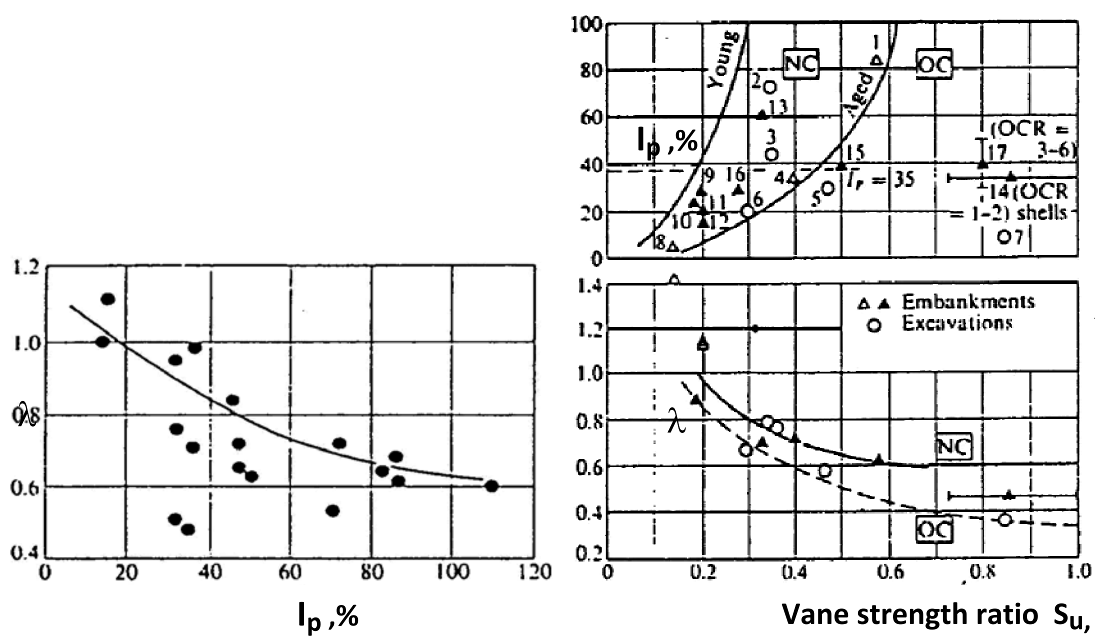
$$\text{or } T = \frac{\pi.D^2 S_u}{2}(H + \frac{D}{3}) \quad \text{..}\mathbf{(2.12)}$$

Notes:

- Use consistent units, such as: T in (kg-cm); S_u in (kg/cm^2);and H and D in (cm).
- It is found that the S_u values obtained by vane shear test are too large for design. Therefore, **Bjerrum's (1972)** proposed a reduction factor using the following formula:

$$S_{u,design} = \lambda..S_{u,field} \quad \text{..}\mathbf{(2.13)}$$

where, λ is a correction factor depends on plasticity index I_p and obtained from **Fig.(2.21a)**; Also, **Aas et al. (1986)** ther charts **(see Fig.(2.21b))** taking into account the effects of aging and OCR (Overconsolidation ratio).



(a) Bjerrum correction factor for vane-shear test.
[(Bjerrum, 1972) and Ladd etal., 1977)].

(b) Reinterpretation of the Bjerrum chart of part a by
(Aas et al. (1986) to include effects of aging and OCR).

Fig.(2.21): Vane shear correction factor λ .

SOLVED PROBLEMS

Problem (2.1): A thin-walled tube (OD = 76.2mm, ID = 73mm) was pushed into a soft clay at

Solution:

Recovery ratio: $L_r = \frac{575}{600} = 0.958$

Area ratio: $A_r = \frac{(76.2)^2 - (73)^2}{(73)^2} x100 = 8.96\%$

Problem (2.2): A three storysteel frame office building will be built on a site where the soils are expected to be of average quality and uniformity. The building will have a (30m x 40m) footprint and is expected to be supported on spread footing foundations located about (1m) below the ground surface. The site appears to be in its natural condition, with no evidence of previous grading. Bedrock is several hundred feet below the ground surface. Determine the required number and depth of the borings.

Solution:

- **Number of borings:**

  From **Table (2.5)**, one boring will be needed for every 200 to 400 m^2 of footprint area. Since the total footprint area is 30 x 40 =1200 m^2, use **(4)four borings**.

- **Depth of borings:**

  For subsurface condition of average quality, the minimum depth is:

  $5.S^{0.7} + D_f = 5(3)^{0.7} + 1 = 12\text{m}.$

  However, it would be good to drill at least one of the borings to a slightly greater depth to check lower strata.In summary, the exploration plan will be 4 borings with, **3 borings to 12 m, and 1 boring to 16 m.**

Problem (2.3): **Given:** Available information about:

Structure: Multistory building with 3 stories and basement

No. of columns = 16, Column load = 1000 kN

Raft dimensions: 16m x 16m x 1m, Foundation at 3m below G.S.

Soil profile: γ_d = 16 kN/m^3 , γ_{sat} = 20 kN/m^3 , W.T. at 6m below G.S.

Required: Number, layout, and depth of B.Hs.?

Solution:

- **Number and layout of borings:**

From Table (2.4b), for poor quality and/or erratic subsurface conditions, one boring is needed

- **Depth of borings:**

(a) $d = 1.5(16) = 24m$

(b) 10% of contact pressure:

$$q_{contact} = \frac{16.(1000) + 24(16)(16)(1)}{(16)(16)} - (3)(16) = 38.5..kPa$$

$$0.1.(38.5) = \frac{38.5(16)(16)}{(16+d)^2}, \ldots\ldots\ldots\ldots d = 34.6m$$

(c) 5% of overburden pressure:

$$0.05.[16(6) + (d-3)(20-10)] = \frac{38.5(16)(16)}{(16+d)^2}, \ldots\ldots\ldots d = 15.5m$$

From (**b** and **c**) take the smaller d = 15.5m

(d) $d = 6.S^{0.7} = 6.(4)^{0.7} = 15.83m$

From all (24m, 15.5m, and 1٥.٨٣m) take the larger d = 24m

$\therefore$ use...$D = 24 + 3 =$ **27m from G.S.**

Problem(2.4): A wide strip footing applying net pressure of 35 kPa is to be constructed 1m below the surface of uniform soil having unit weight of 19 kN/m^3. The footing is 5m wide and the water table is at ground surface. Is 12m depth of boring (measured from ground surface) sufficient for subsoil exploration program.

Solution:

(a) $d = 3(B) = 3(5) = 15m$

(b) 10% of contact pressure: $0.1.(35) = \frac{(35)(5)(1)}{(5+d)(1+d)}, \ldots\ldots\ldots d = 4.3m$

(c) 5% of overburden pressure: $0.05(9 + 9d) = \frac{(35)(5)(1)}{(5+d)(1+d)}, \ldots\ldots d = 5.2m$

From (**b** and **c**) take the smaller d = 4.3m

From all(15m, and 4.3m) take the larger d =15m, and so the depth from ground surface $D = 15 + 1 = \mathbf{16m}$, $\therefore$ **12m is not sufficient.**

liner. If the vertical effective stress at the test depth was 70 kN/m², determine N'_{60} ?

Solution:

The raw SPT value isN = 6 + 8 = 14

Since $p'_o = 70...kPa > 25$ kPa $\therefore C_N = 0.77.\log_{10}\dfrac{2000}{70} = 1.12$

From **(Table 2.9):**

$\eta_1 = E_r / E_{rb}$ =45/60 = 0.75

η_2 =0.85(forL $= 4.8$m (rod length) < 6m),

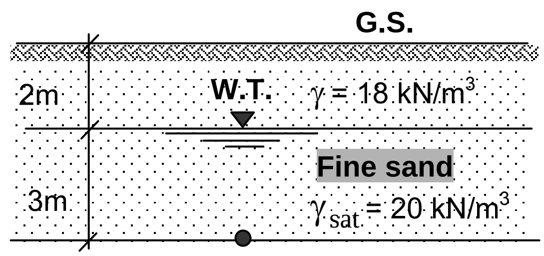
η_3 = 0.90 (for loose sand with liner),

η_4 =1.05 (for B.H. diameter = 150mm),

$N'_{60} = N_{field}..C_N..\eta_1..\eta_2..\eta_3..\eta_4$ = 14(1.12)(0.75)(0.85)(0.90)(1.05) =**10blows**

Problem (2.6): A standard penetration test was carried out in sand at 5m depth below the ground surface gave (N = 28) as shown in the figure below. Find the corrected N-value?

Solution:



$N_{field} = 28..blows/30cm$

- **Water table correction:**

For N > 15 …… $N' = 15 + 0.5.(N_{field} - 15)$

N' = 15 + 0.5(28 -15) = 21

- **Overburden correction:**

P'_o = 2(18) + 3(20 – 9.81) = 66.57 kPa $>$ 25 kPa

$\therefore C_N = 0.77\log\dfrac{2000}{P'_o} = 0.77\log\dfrac{2000}{66.57} = 1.14$

$\therefore N'_{corr.} = N'..C_N$ = 21(1.14) = **23 blows**

Problem (2.7): It is proposed to construct a spread wall footing of (3m width) in sand at (1.5m) below the ground surface to support a load of 12 Ton/m. The SPT results from a soil boring are as shown below. If the water table is located at 0.9m from G.S. and $\gamma_{soil(sat.)} = 17.6$ kN/m^3, determine the average corrected N-value required for design?

SPT sample depth (m)	1.5	2.25	3.0	3.75	4.5	5.25	6
N_{field}	31	25	22	20	28	33	31

Solution:

Find P'_o at each depth and correct N_{field} values up to at least a depth B below the foundation according to the magnitude of overburden pressure in comparison of 25 kPa.

Overburden pressure correction: $C_N = 0.77 \log \dfrac{2000}{P'_o}$

For 1.5m depth:

P'_o = 0.9(17.6) + (0.6)(17.6– 9.81)= 20.5 kPa $<$ 25 kPa, therefore, C_N =1.00

For 4.5m depth:

Find the average corrected N-value as a cumulative average down to the depth indicated, and then, choose the N-value for design as the lowest average N-value.

SPT sample depth (m)	N_{field}	P'_o (kN/m^2)	C_N	$N' = C_N.N_{field}$	$N'' = 15 + 0.5(N' - 15)$	$N''_{avg.}$
1.5	31	20.5	1.00	31	23	23
2.25	25	26.3	1.45	36	25	24
3.0	22	32.2	1.38	30	22	23
3.75	20	38.0	1.32	26	20	22
4.5	28	43.9	1.28	35	25	23

For 1.5m depth: $N'_{avg.} = 23$

For 2.25m depth: $N'_{avg.} = \dfrac{23 + 25}{2} = 24$

For 3.0m depth: $N'_{avg.} = \dfrac{23 + 25 + 22}{3} = 23$

For 3.75m depth: $N'_{avg.} = \dfrac{23 + 25 + 22 + 20}{4} = 22$

For 4.5m depth: $N'_{avg.} = \dfrac{23 + 25 + 22 + 20 + 25}{5} = 23$

N-value for design = $N'_{avg.}(\text{lowest}) =$ **22 blows**

Problem (2.8): The load-settlement data obtained from load test of a square plate of size (1ft) are as shown below. If a square footing of size (7ft) settles (0.75 inch), what is the allowable soil pressure of the footing? Consider sandy soil.

Load (Tsf)	2	5	8	10	14	16	19
Settlement (inch)	0.1	0.2	0.3	0.4	0.6	0.8	1.0

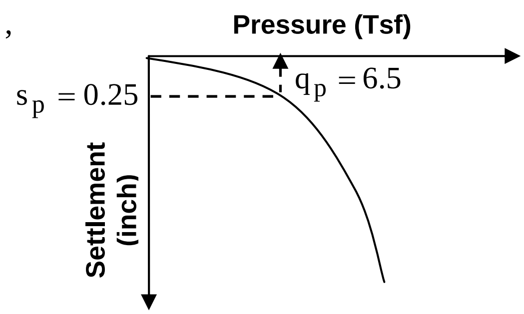
Solution:

For cohesionless soil: $$\left\{\begin{array}{l} q_f = q_p \dfrac{B_f}{B_p} \\ s_f = s_p\left(\dfrac{2B_f}{B_p + B_f}\right)^2 \end{array}\right. ,\therefore s_p = \left(\dfrac{0.75}{\left(\dfrac{2x7}{1+7}\right)^2}\right) = \dfrac{0.75}{3.05} = 0.25''$$

Now by drawing the given data and for $s_p = 0.25''$,

$q_p = 6.5\ \text{T/ft}^2$,and

$q_f = q_p \dfrac{B_f}{B_p} = 6.5\dfrac{7}{1} = \mathbf{45.5}\ \text{T/ft}^2$.



Problem (2.9): Use Housel method to determine the size of square footing required to carry a column load P = 45 tons if the two plate loading tests results are as given below:-

- plate size (1) = 35x35cms, corresponding load= 5.6 tons;relative to 1.0 cm settlement.
- plate size (2)=50x50 cms,corresponding load =10 tons; relative to 1.0 cm settlement.

Solution:

From **Housel's method**(Eq. 2.6):V = A. q + P. s

$$B = \frac{-0.24 \pm \sqrt{(0.24)^2 + 4(1)(1)(1.67)}}{(2)(1)} = \frac{-0.24 \pm 2.59}{2} = 1.18\ \text{m}$$

Take the footing **1.20 m x 1.20 m**.

Problem (2.10): A vane tester with a diameter d = 9.1cms and a height h = 18.2 cms requires a torque of 110 N-m to shear a clay soil sample, with a plasticity index of 48%. Find the soil un-drained cohesion S_u?

Solution:

For **CASE (2)** with top and bottom vane ends embedded in soil, the torque is given by:

$$T = \frac{\pi . D^2 . S_{u,field}}{2}(H + \frac{D}{3})$$

or $$S_{u,field} = \frac{T}{\frac{\pi . D^2}{2}(H + \frac{D}{3})} = \frac{0.110}{\frac{\pi .(0.091)^2}{2}\left[0.182 + \frac{0.091}{3}\right]} = 40\ \text{kN/m}^2$$

From **Fig.(2.27a)** for a plasticity index of 48%, Bjerrum'scorrection factor λ = 0.80, and

Therefore, $S_{u,design} = \lambda .. S_{u,field} = 0.8(40) = \mathbf{32\ kPa}$