First semester

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CHAPTER 1

INTRODUCTION

1.1 DEFINITIONS

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

- **FOUNDATION:** The soil beneath structures responsible for carrying the loads is the *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.

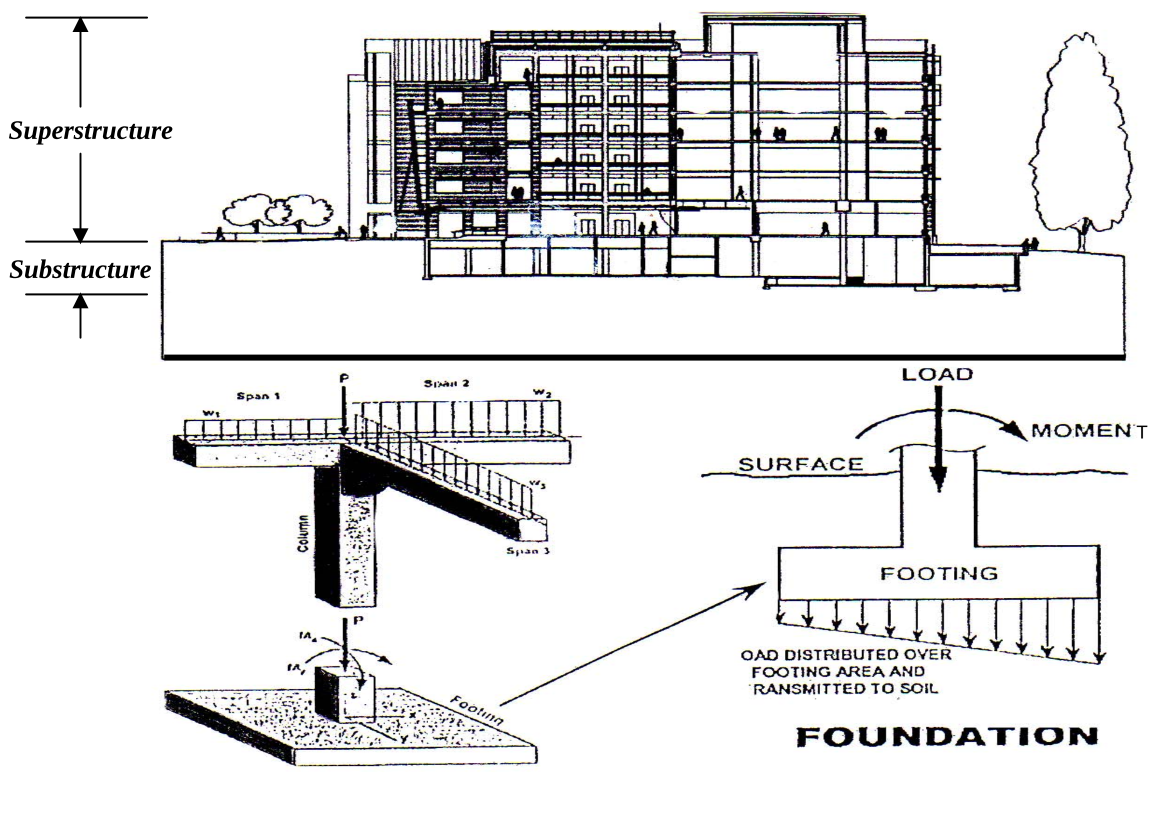


Fig. (1.1): Foundation definition.

- <u>**FOUNDATION ENGINEERING:**</u> 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.

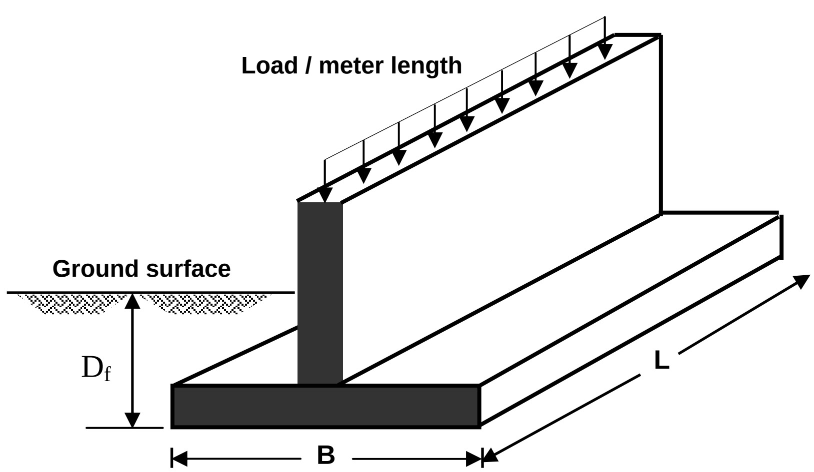


Fig. (1.2): Individual footing.

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:

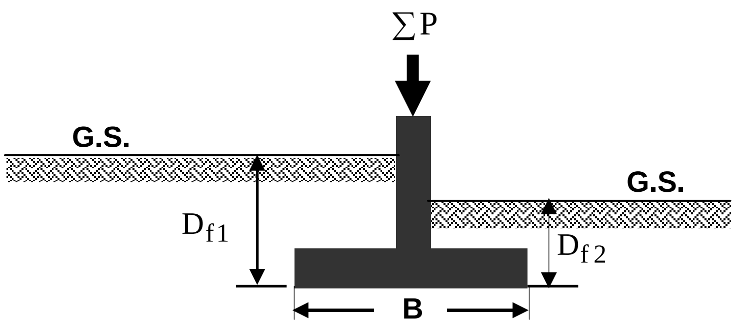


Fig. (1.3): Depth of embedment.

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

If $(D_{f1}.\gamma - D_{f2}.\gamma) \leq \frac{q_u}{2}$ …………**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:* $q_u = 2.C_u.\tan.(45+\phi/2)$

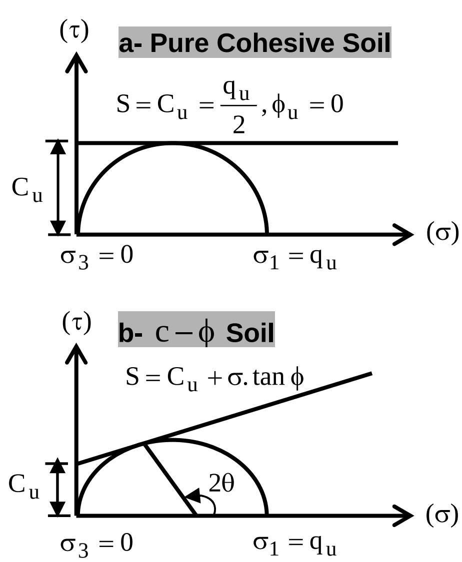


Fig. (1.4): Unconfined compressive strength test.

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:

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

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).**

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

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.

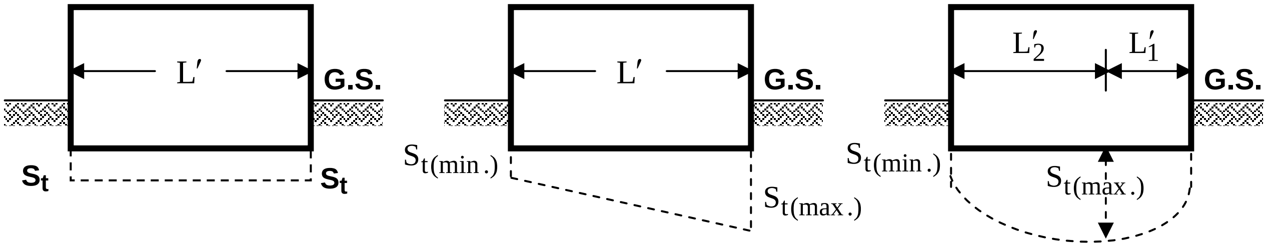
Fig. (1.5a) shows a uniform settlement which occurs when a structure is built over rigid structural mat. However, depending on the load of various foundation components, a structure may experience differential settlement. A foundation may also undergo uniform tilt (**Fig. 1.5b**) or non-uniform settlement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

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

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

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.**

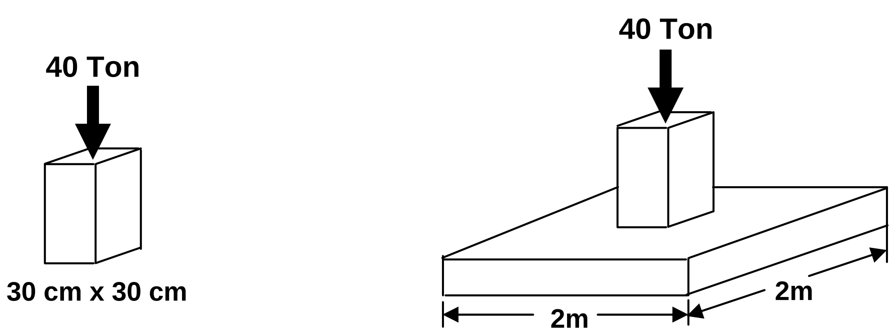
or $q_{all.} = \dfrac{40000}{30x30} = 44.4\ \text{kg/cm}^2 < 50\ \text{kg/cm}^2$ **…… O.K.**

$\therefore$ 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.

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



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

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**).

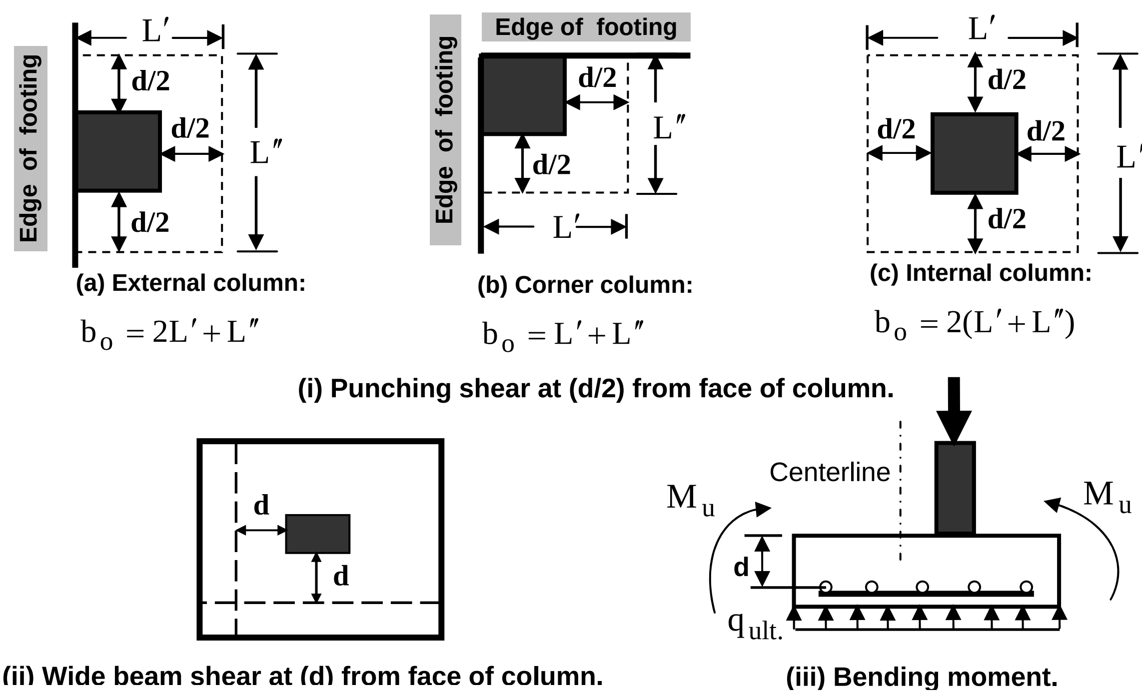


Fig. (1.7): Types of failure in footing.

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 (see **Fig. 1.8**). However, the design of square or circular spread footings is simpler than that of rectangular one. This is evident due to the twice calculation required for rectangular footing compared with other ones. The rectangular footing is preferred in case of a moment, since the length is increased in the direction of moment to make the resultant of loads within the middle third of footing.

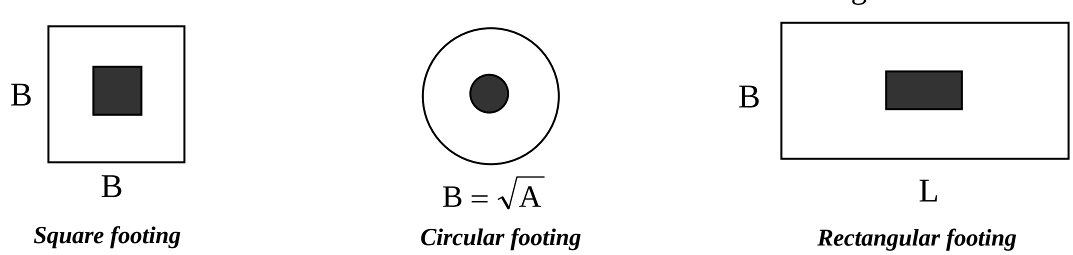


Fig. (1.8): Spread footing.

(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 wall column can not extend outside the limits of the structure.

(b) If the loads from 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.

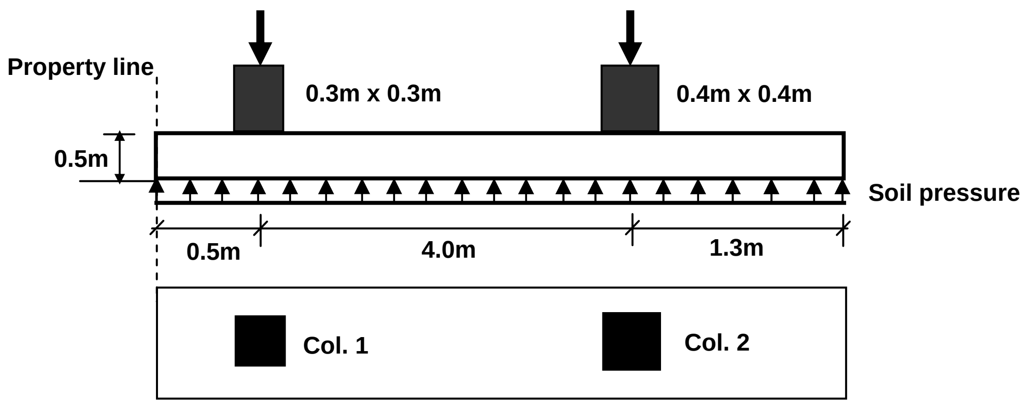


Fig. (1.9: Rectangular combined footing.

- **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 $(\bar{x} < \frac{L}{3})$.

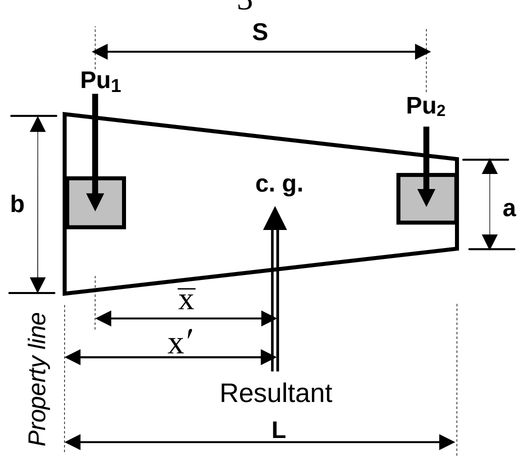


Fig. (1.10): Trapezoidal combined footing.

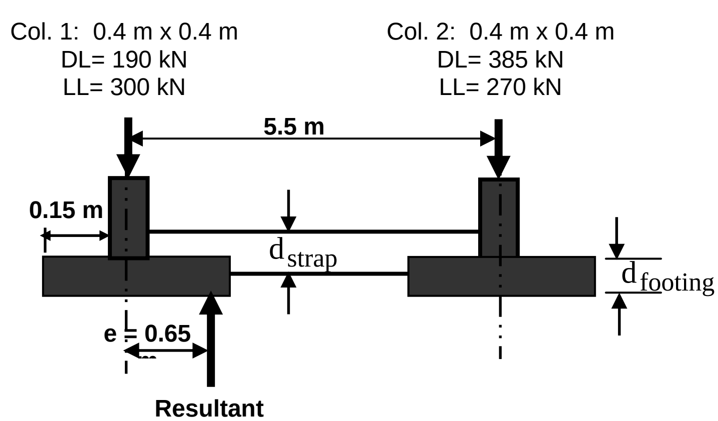


Fig. (1.11): Strap combined footing.

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

This footing represents a plain strain condition, such as a footing beneath a wall. In this case, the footing area is calculated as: $\text{Area} = B.x.1 = \dfrac{\text{Total..load/unit..length}}{q_{all.}}$

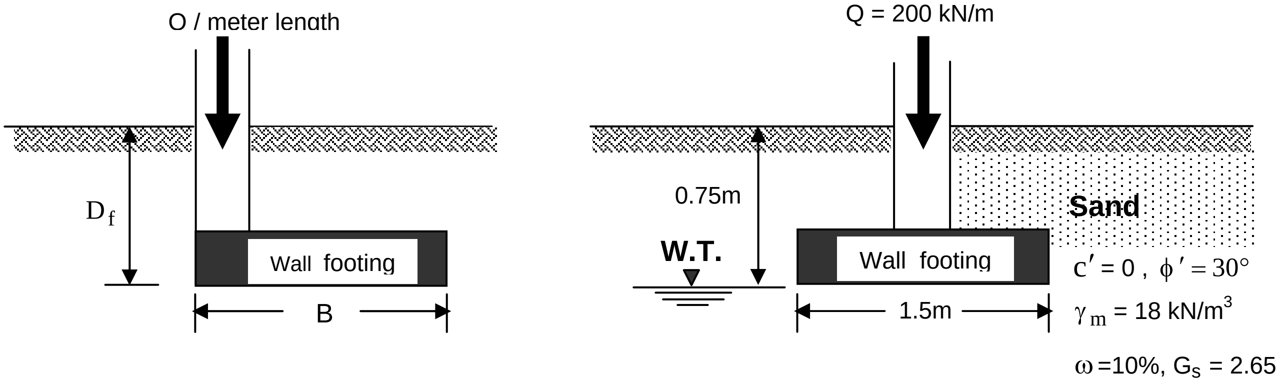


Fig. (1.12): Wall footing.

(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,
- The bearing capacity of soil is very low, and the building loads are so heavy, and
- A large differential settlement is expected to occur.

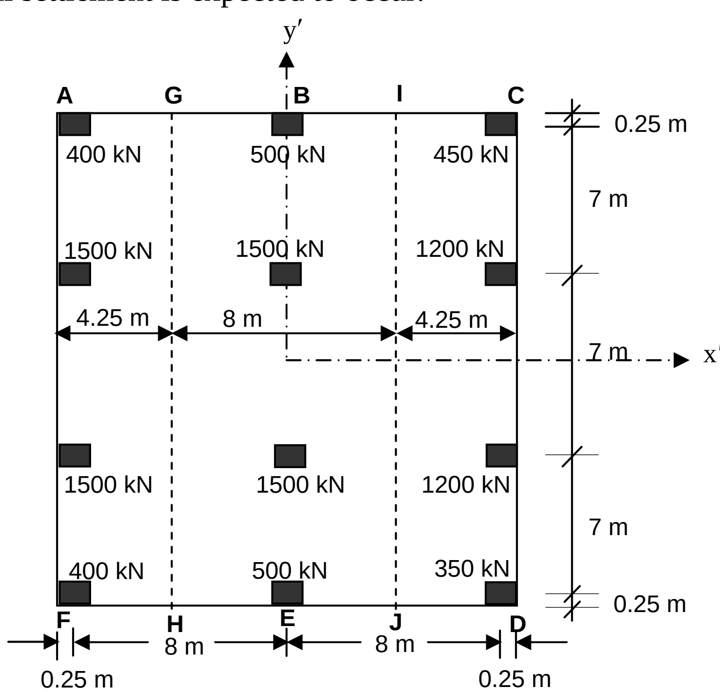


Fig. (1.13): Raft foundation.

(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.

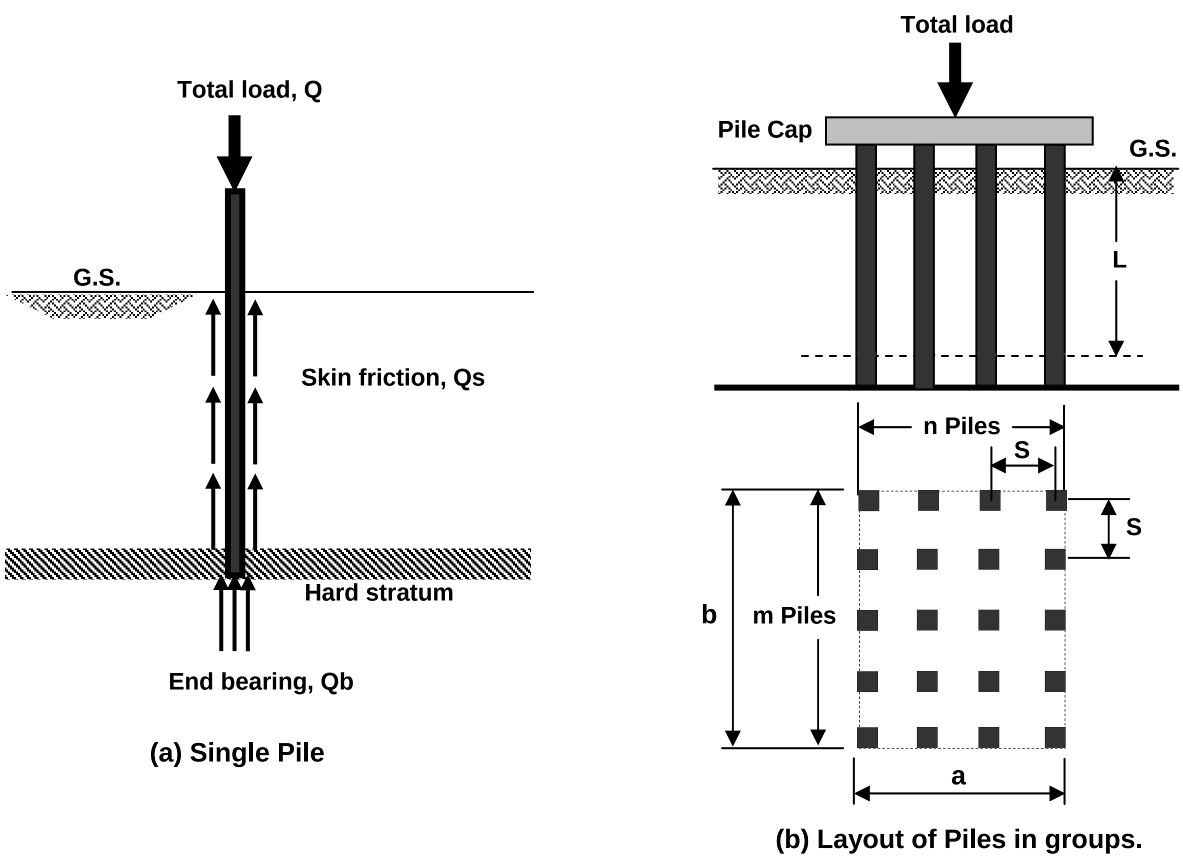


Fig. (1.14): Single and group piles.

(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**).

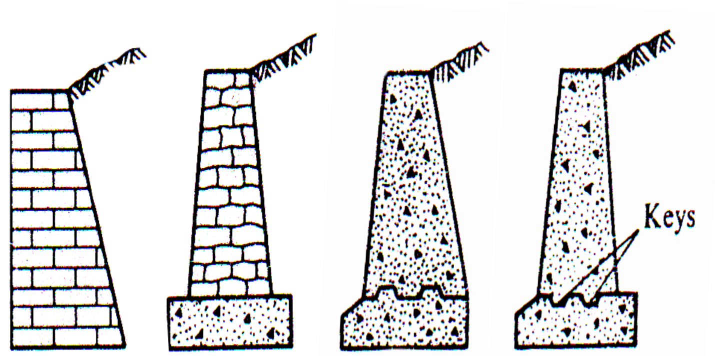
(d) Counterfort Retaining Walls are similar to cantilever retaining walls, at regular intervals, however, they have thin vertical concrete slabs behind the wall known as counterforts that tie the wall and base slab together and reduce the shear and bending moment. They're economical when the wall height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

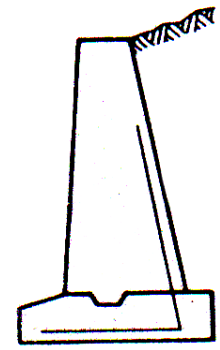
(f) Bridge Abutments are special type of retaining walls, not only containing the approach fill, but serving as a support for the bridge superstructure (see **Fig. 1.16f**).

(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**).

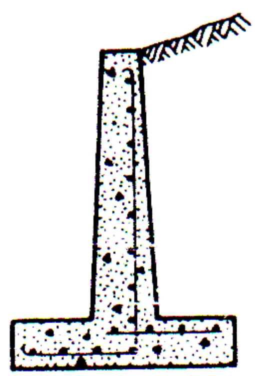
*Among these walls, only the **cantilever retaining walls** and **bridge abutments** are much used.*



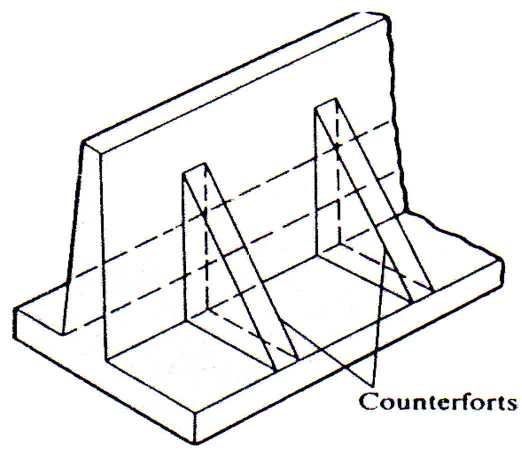
(a) Gravity walls



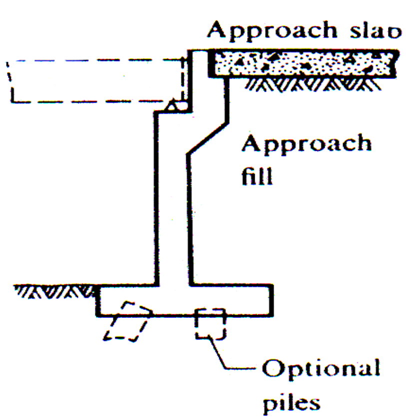
(b) Semi-gravity wall



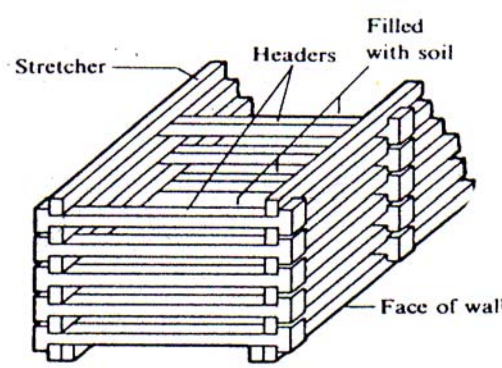
(c) Cantilever wall



(d) Counterfort wall



(f) Bridge abutment



(g) Crib walls

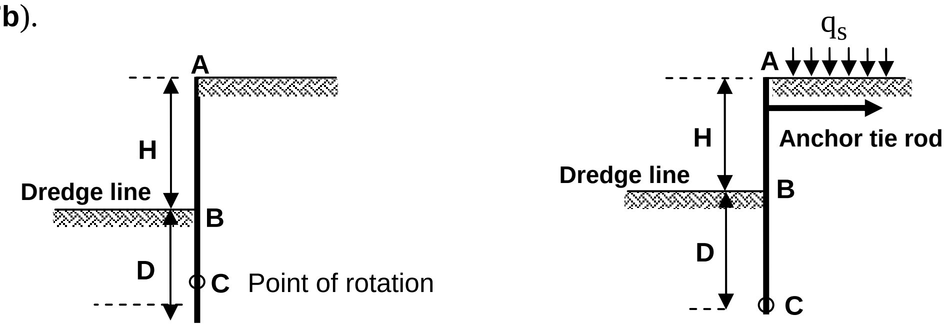
Fig. (1.16): Common types of retaining walls.

(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**).

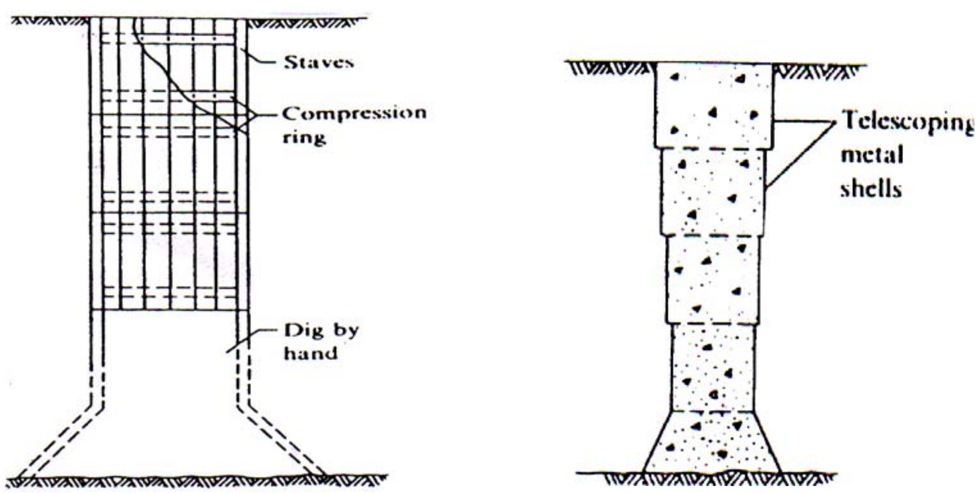


(a) Cantilever sheet pile wall. (b) Anchored sheet pile wall.

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.

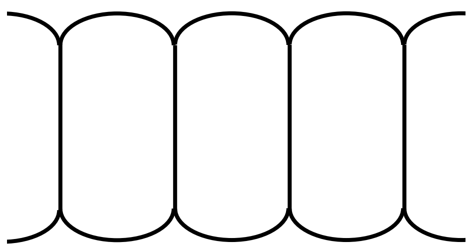
Fig. (1.18): Methods of caisson construction.

(11) Cofferdams:

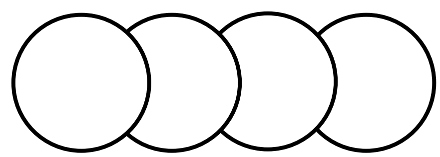
(a) Single and Double Sheet Pile Cofferdams: used for depth of water not exceeds 3.0 m.
(b) Cellular Cofferdams: used for higher depths of water, i.e., greater than 3.0 m.

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

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

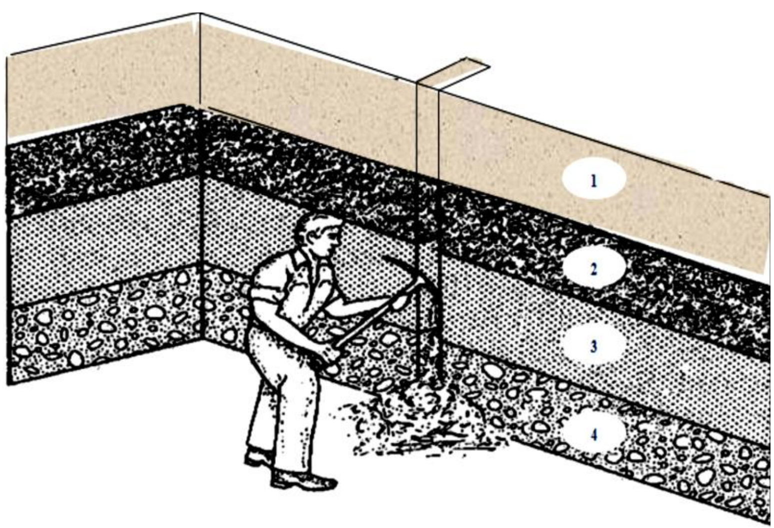
A pit is dug either by hand or by a backhoe. Probably in a test pit, the engineer can examine in detail the subsoil strata and take disturbed or undisturbed samples at the desired location (see Fig. 2.1):

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 investigation that 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 pushing down on the handlebar. Then withdrawing and emptying 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):

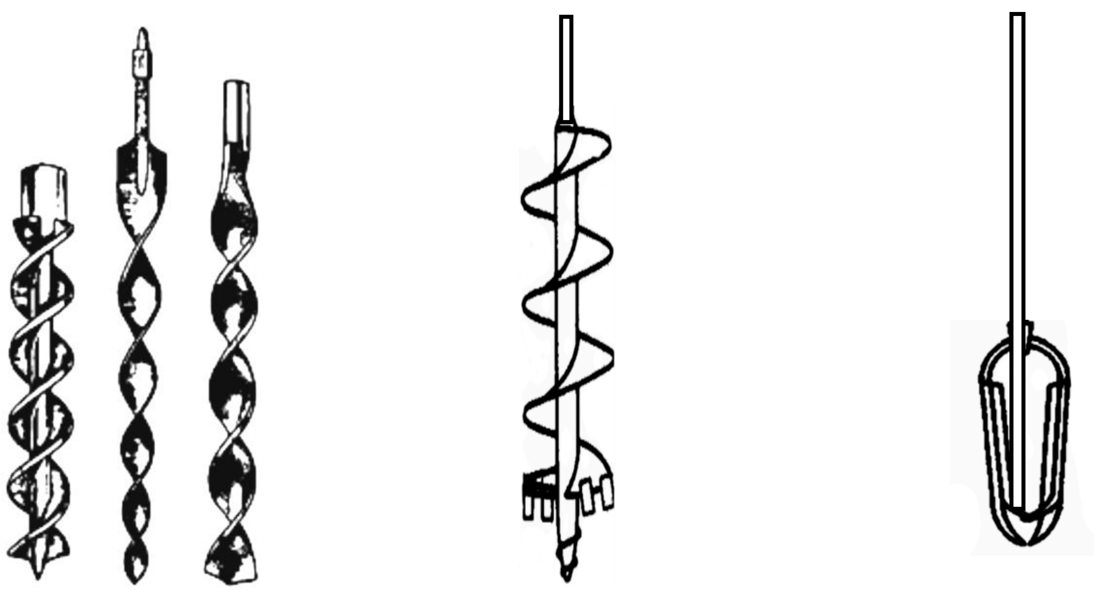
a. Helical Auger.
b. Short flight Auger, and
c. Iwan Auger.

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. Short flight Auger** **c. Iwan (posthole) Auger**

Fig. (2.2): Hand-augers.

(b) Power-Augers

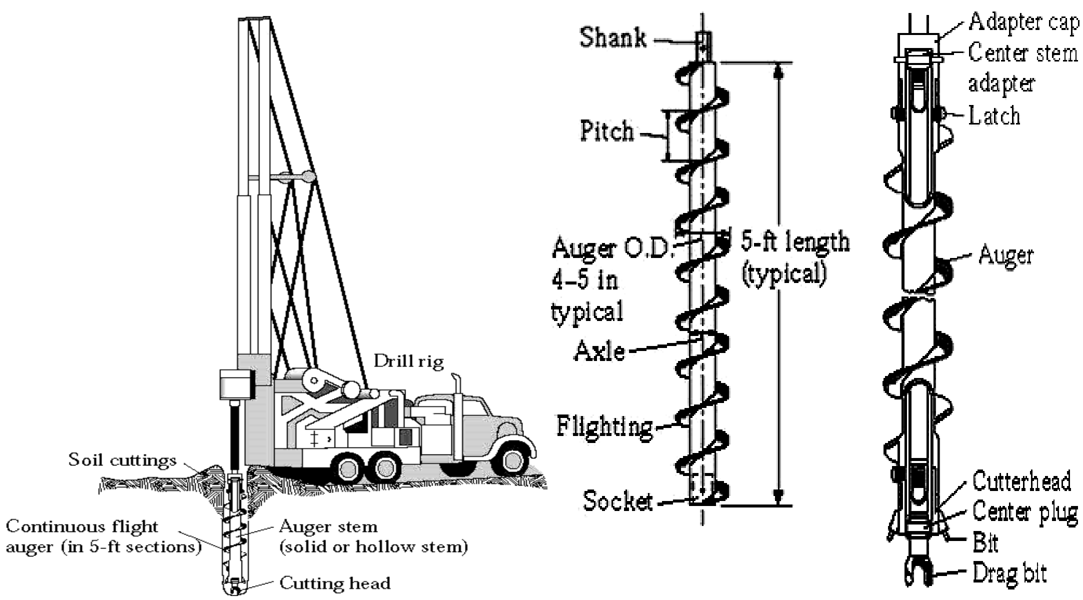
Truck or tractor mounted type rig and equipped with continuous flight augers that bore a hole of 100 to 250 mm in diameter. These augers can have a solid or hollow stem of (20-75) cm in diameter (see Fig.2.3).

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 motorized vehicle.



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

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

2.2.3 WASH BORING

Water is pumped to bottom of borehole and soil washings are returned to surface. A drill bit is rotated and dropped to produce a chopping action (see Fig. 2.4).

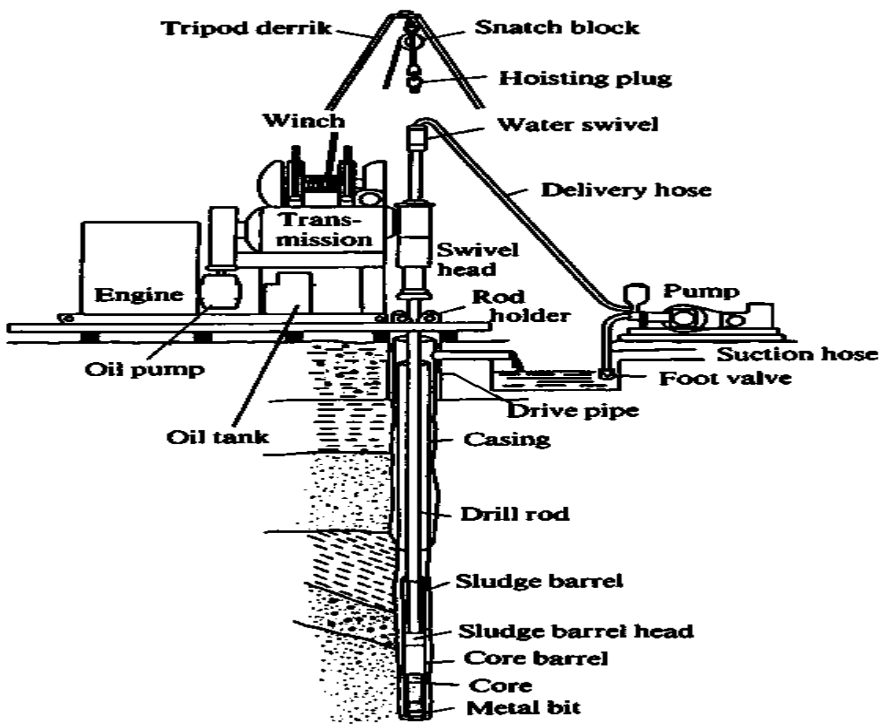


Fig. (2.4): Wash boring rig.

(a)Jetting Method

Method: Water is pumped down the center 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) Slugging (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 improvise successfully – provides a pumping action. Water flows down the borehole annulus (ring) and back up the drill pipe, bringing 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.
- Water is required for pumping.
- Difficulty in obtaining accurate location of groundwater level.
- Boulders can prevent further drilling, and
- Depth is limited to about 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 penetrate gravel.
- 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.

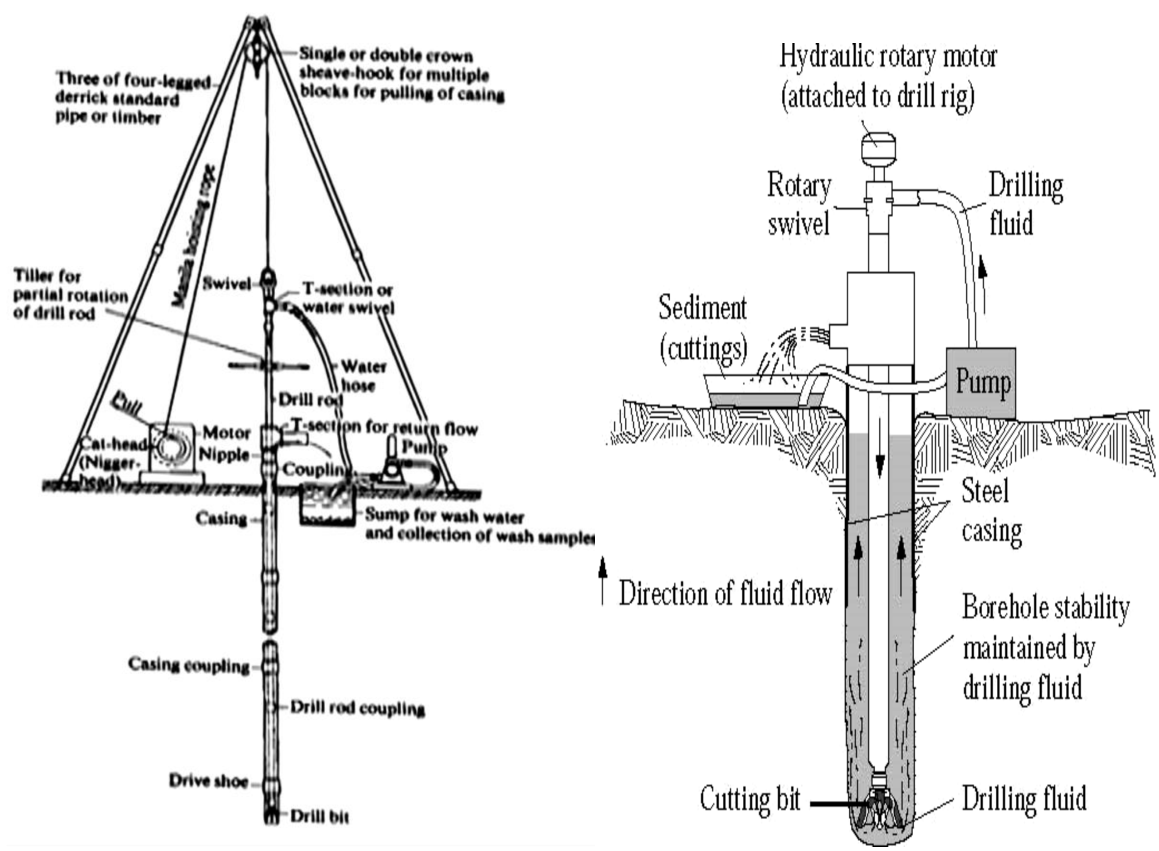


Fig. (2.5): Rotary drilling.

2.2.5 PERCUSSION DRILLING

Method: The lifting and dropping of a heavy (+50kg) cutting tool will chip and excavate material from a hole. The tool may be fixed to rigid drill-rods or to a rope or cable. With a mechanical winch, depths of hundreds of meters can be reached.

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.

2.3 UNDER GROUND WATER IN THE TEST HOLE

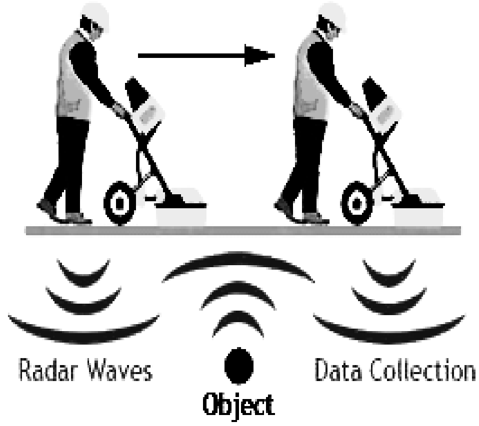
The depth of the water table (W.T.) as measured during drilling and sampling should be carefully evaluated. It is always necessary to wait for at least 24 hours to check on the stabilized water table for the final measurement. The technician should plug the top of the drill holes and flag them for identification. 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

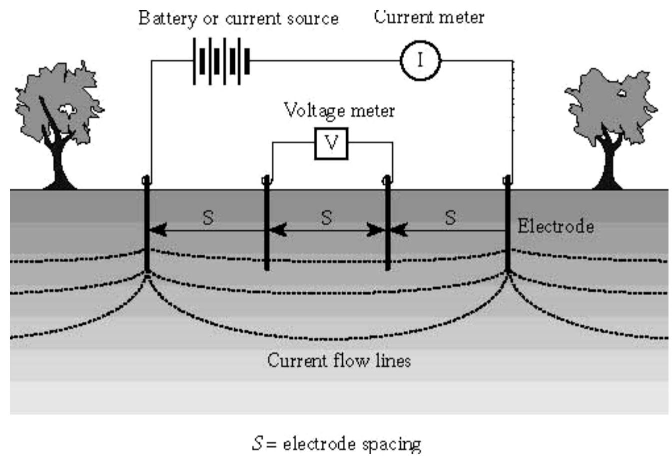
These methods represent indirect methods of subsoil exploration and mainly consist of:

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

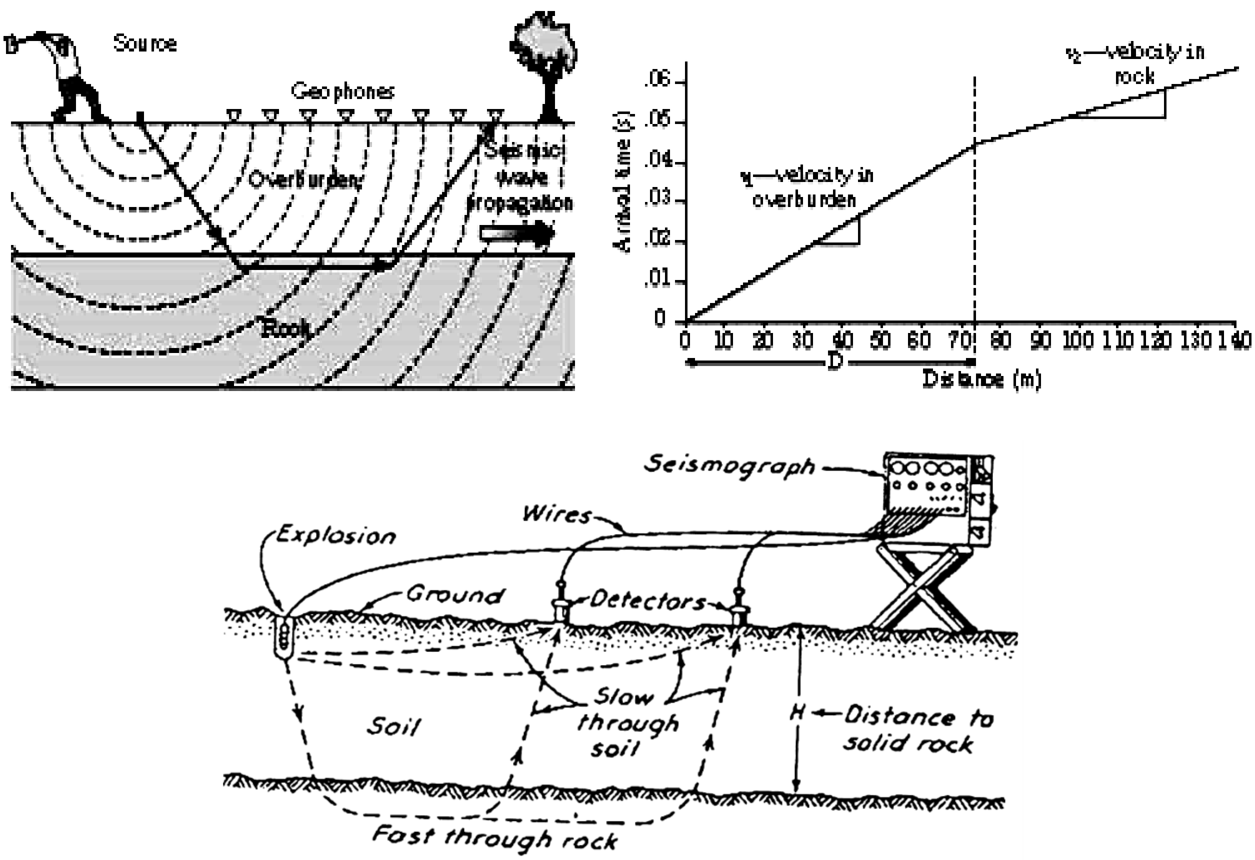
In subsoil investigation, the seismic methods are most frequently used. These methods are based on the variation of the wave velocity in different earth materials. They involve in generating a sound wave in the rock or soil, using a sledge-hammer, a falling weight, or a small explosive charge, and then recording its reception at a series of geophones located at various distances from the shot point, as shown in **Fig.(2.6)**. The time of the refracted sound arrival at each geophone is noted from a continuous reader. Typical seismic velocities of earth materials in (m/sec) are shown in **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 to strike a plate on the surface.
- A series of detectors, or geophones, spaced at intervals along a line from wave origin point, and
- A time-recording mechanism to record the time of origin of the wave and the time of its arrival at each detector.

Advantages of seismic exploration:

1. Permits a rapid coverage of large areas at a relatively small cost.
2. Not hampered by boulders and cobbles which obstruct borings, and
3. Used in regions not accessible 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 before definite 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) The undisturbed samples (U): which were obtained using a thin Shelby tubes of 100mm in diameter and (400-450)mm in length, and

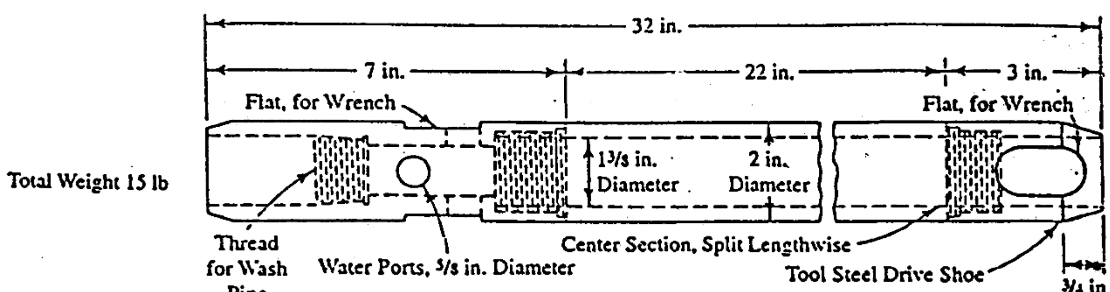
(c) The (SS) samples: which were taken from standard split spoon sampler used in a standard penetration test (S.P.T.) that performed at different intervals depending on soil stratification.

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

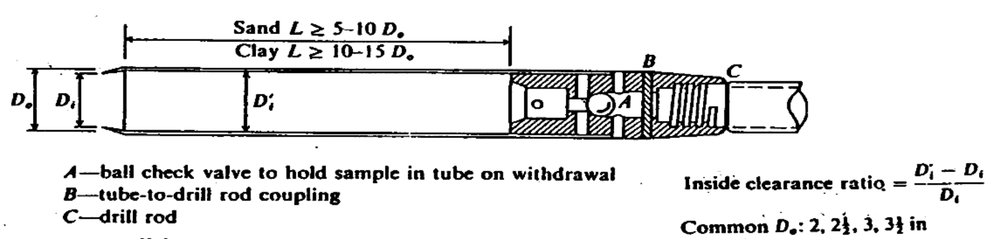
Fig.(2.7) shows some details of standard split-spoon and thin-wall tube samplers that commonly used in in-situ testing and sample recovery equipment. A modification in the design of the split spoon sampler allows the insertion of brass thin-wall liners into the barrel. Four sections of brass liners (each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

Samples of rock are generally obtained by rotary core drilling. Diamond core drilling is primarily used in medium-hard to hard rocks. Special diamond core barrels up to 8 inch 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).***



(a) Standard Split-Spoon Sampler.



(b) Thin-Wall Shelby Tube Sampler.

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 percussion drilling.	• All types of soils, • Due to high disturbance it is unsuitable for foundation exploration.
2. Split spoon sampler	Standard Penetration Test.	• Cohesive, cohesionless soils and soft rocks, • For taking disturbed samples which are required for physical and geotechnical analysis of soil as well as chemical tests. • In cohesionless soils, the penetration number (N) is used for making both strength and settlement estimates.
3. Thin wall Shelby tube	16gauge seamless steel tube (7.5-15) cm dia.; preferably pushed by static force instead of driven by 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 undisturbed continuous 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 disturbed samples in cohesionless soil or disturbed and 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.

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} x100 \quad \text{......(2.1)}$$

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.

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

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 means by 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 be tested for strength and compressibility to determine the stress-strain characteristics of the material, in addition to classification and chemical tests. Whereas, disturbed (D) or (SS) samples as available were mainly used for physical and geotechnical analysis of soil as well as chemical tests.

2.7.1 LABORATORY TESTS:

The obtained samples should be tested according to the procedure of the American Society for Testing and Materials (ASTM) or the British Standards (BS) whichever is appropriate. The test program of the samples includes the followings:

1. Classification Tests:
Sieve and hydrometer analysis, natural water content, Atterberg's limits, specific gravity, and wet 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 equipment, 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), • Tor vane shear test (for soft soil; up to Cu =5.0 kg/cm^2), • Pocket penetrometer, • Pressure meter test; it is of three types: **a-** Menard (to obtain; $R_D, \phi, S_u, E_s, G', m_v, C_c$), **b-** Self-boring (to obtain; $R_D, \phi, S_u, u, E_s, G', m_v, C_c, C_v$), **c-** Screw boring (to 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', liquefaction resistance 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 forms containing blanks for filling in appropriate data. **Fig.(2.9)** shows an example of a boring log sheet.

Soil data obtained from a series of test borings can best be presented by preparing a geologic profile, which shows the arrangement of various layers of soil, the groundwater table, existing and proposed structures, and soil properties data. An example of a geologic profile is shown in **Fig.(2.10)**.

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**.**

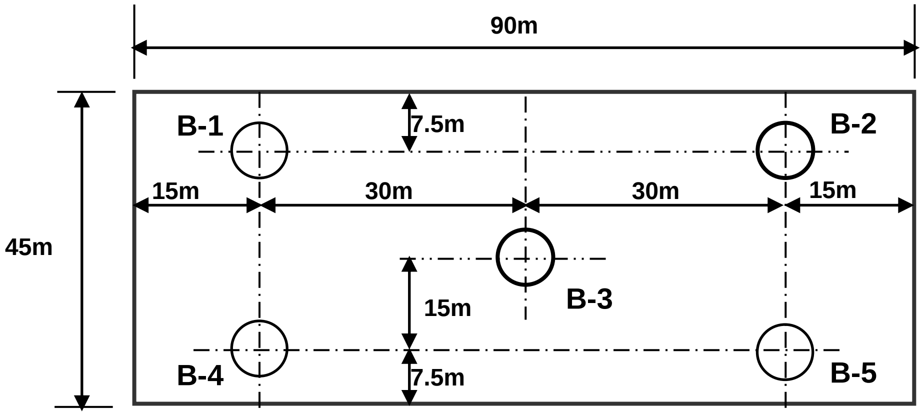


Figure (2.8): Example map showing boring locations on site plan.

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 with 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.

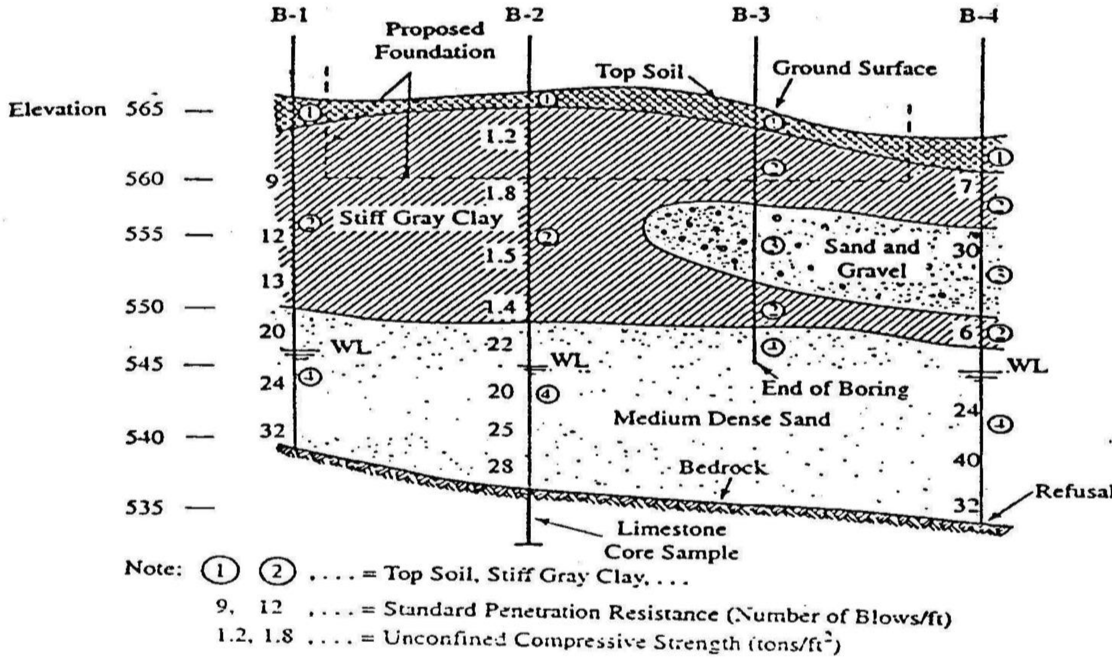


Fig.(2.10):Example of geologic profile.

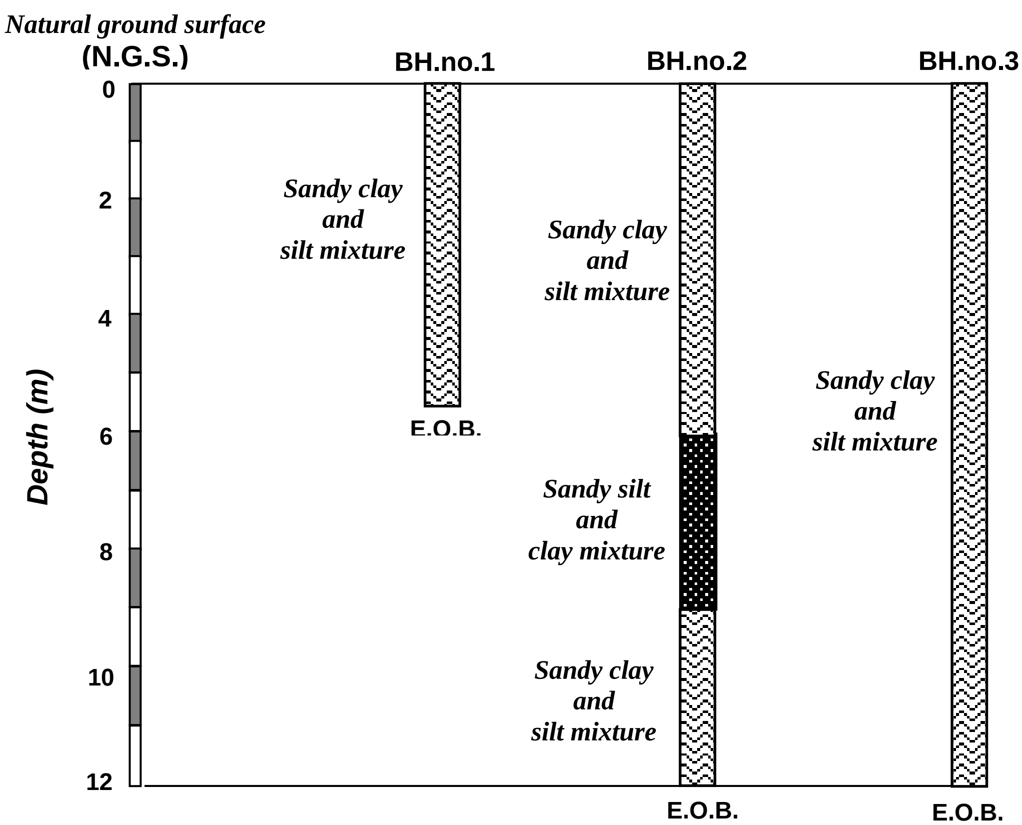


Fig.(2.11): Log of borings for 1st. stage of garden city housing project Tanahi District / Duhok city.

UNIVERSITY OF DUHOK
COLLEGE OF ENGINEERING
ENGINEERING CONSULTING BUREAU

RECORD OF TESTS RESULTS

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

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^- %
1	100	D	0	2.0	22.8	53	27	0	10	55	35	2.70		CH	Sandy clay and silt mixture.																0.65	0.14	2.70	8.0	—	50	0.06
2	101	U	2.0	2.5	21.7	51	27	0	8	54	38	2.70		CH	Sandy clay and silt mixture.		20.94	17.21	76			38	0	0.539	0.66×10^{-4}	0.371	0.073	0.045		0.77×10^{-4}							
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.																0.02	0.03	0.07	7.36	—	17	—
5	104	U	5.0	5.5	20.1	56	28	0	8	55	37	2.72		CH	Sandy clay and silt mixture.		21.22	17.67	92			46	0	0.510	0.44×10^{-4}	0.581	0.100	0.050		0.10×10^{-4}							
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.																						

UNIVERSITY OF DUHOK
COLLEGE OF ENGINEERING
ENGINEERING CONSULTING BUREAU

RECORD OF TESTS RESULTS

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

1 Samples			2 Depth of Sampling (m)		3 Index Properties			4 Particle Size Distribution and Hydrometer Analysis				5	6	7	8	9	10 Unit Weight (kN/m^3)		11	12 Shear Strength Parameters				13 Consolidation Test							14 Chemical Tests						
Number: Field	Number: Lab.	Type	From	To	ω %	LL %	PI %	Gravel %	Sand %	Silt %	Clay %	Specific Gravity 'Gs'	S.P.T 'N' Value	Symbol Unified Classification	Description of Soil	R.Q.D %	Wet	Dry	Unconfined Compressive Strength qu (kN/m^2)	Drained c' (kPa)	Drained ϕ'	Undrained c_u (kPa)	Undrained 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.

Obviously, the more boreholes and the closer they are spaced, the more accurate the resulting geologic profile. Boreholes number and layout may need to be changed as more information emerges, so that, an additional boreholes may be required during the survey.

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
(after Hvorslev 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
Special structures: - Retaining walls - Earth dams	120 25- 50			at centerline with some B.Hs. located on both sides
Slope stability problems	-	-	-	(3-4) B.Hs at critical zone and (1) B.H outside this zone

Table (2.5): Number of borings for 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 conditions are 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 criteria can 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 increase in stress due to the foundation or building load is less than 5% of the effective overburden pressure;
- 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 interfering footings)(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 the critical 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. Thus, Hvorslev (1949) recommends:-

- *For earth structures*, a depth equal to 1.5 times the base width of the dam, and
- *For concrete structures*, a depth between 1.5 and 2.0 times the height of the dam.

4. Roads, highways, and air fields: the minimum depth is 5m below the finished road level, provided that vertical alignment is fixed but should extend below artificial fill or compressible layers. In practice some realignment often occurs in cuttings, and side drains may be dug up to 6m deep or to bore to at least 1.5 times the embankment height in fill areas, and to at least 5m below finished road level in cut.

5. Retaining walls, slopes stability problems: Explore to:

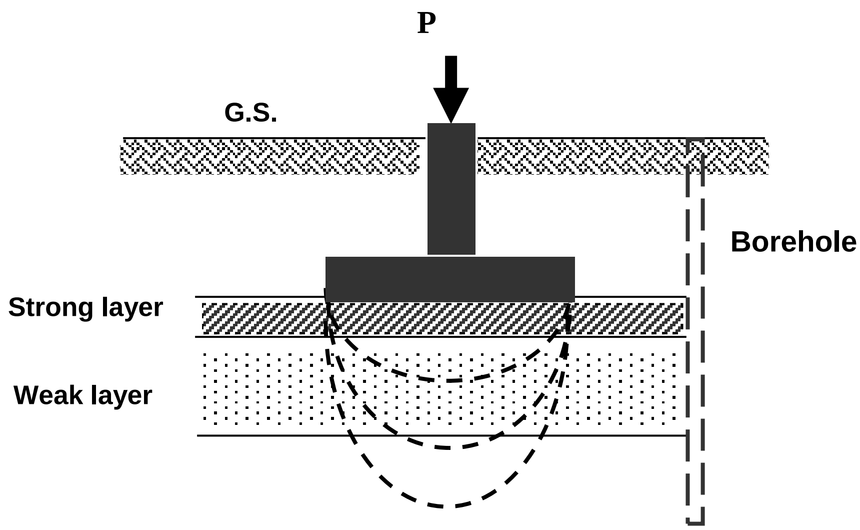
- 1.5B (wall base width) or 1.5H (wall height) whichever is greater below the bottom of the wall or its supporting piles (see Fig.(2.14b)), In addition to;
- It must be below an artificial fills or compressible layers, and deeper than possible surface of sliding;

6. Canals, deep cut and fill sections on side hills: Explore to at least to:

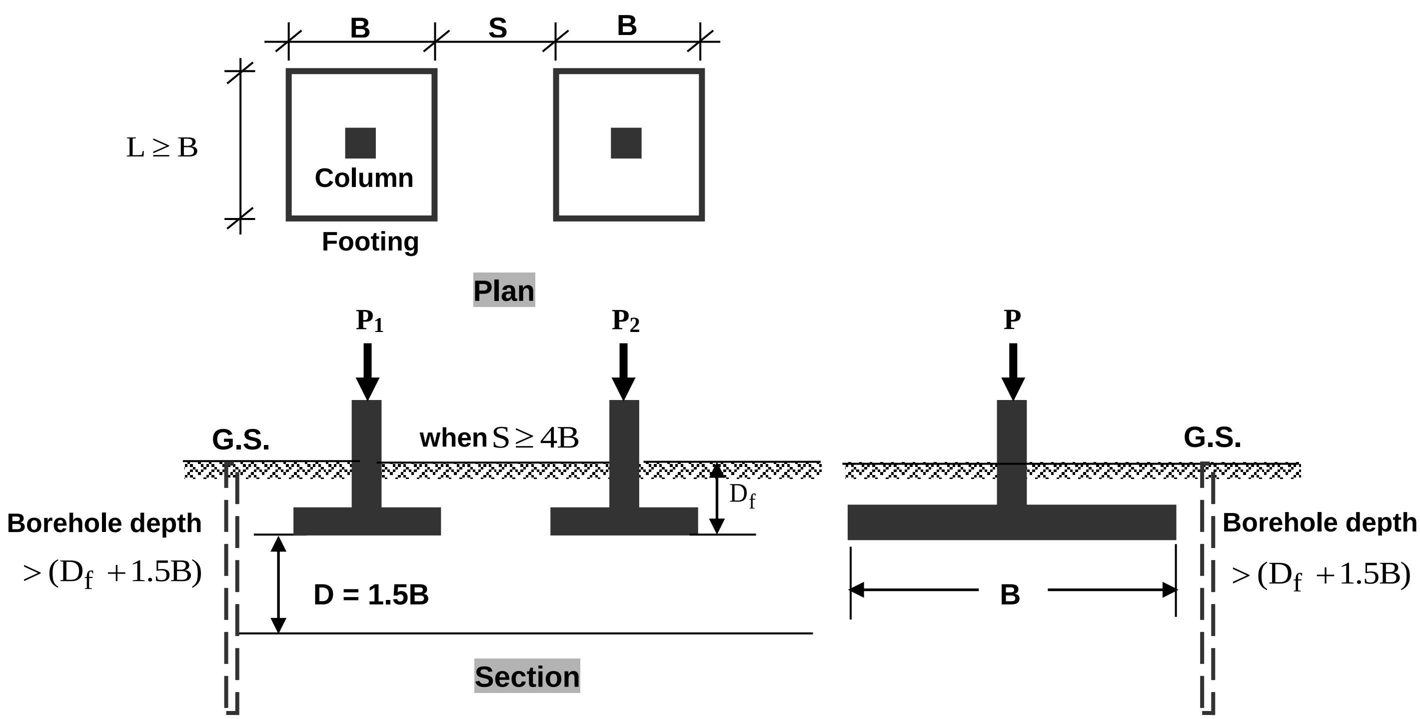
(i)3m below the finished level in cut, or
(ii)B when $B \leq H$, or
(iii) H when $B > H$(see Figs.(2.15a and 2.15b)).

7. 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.

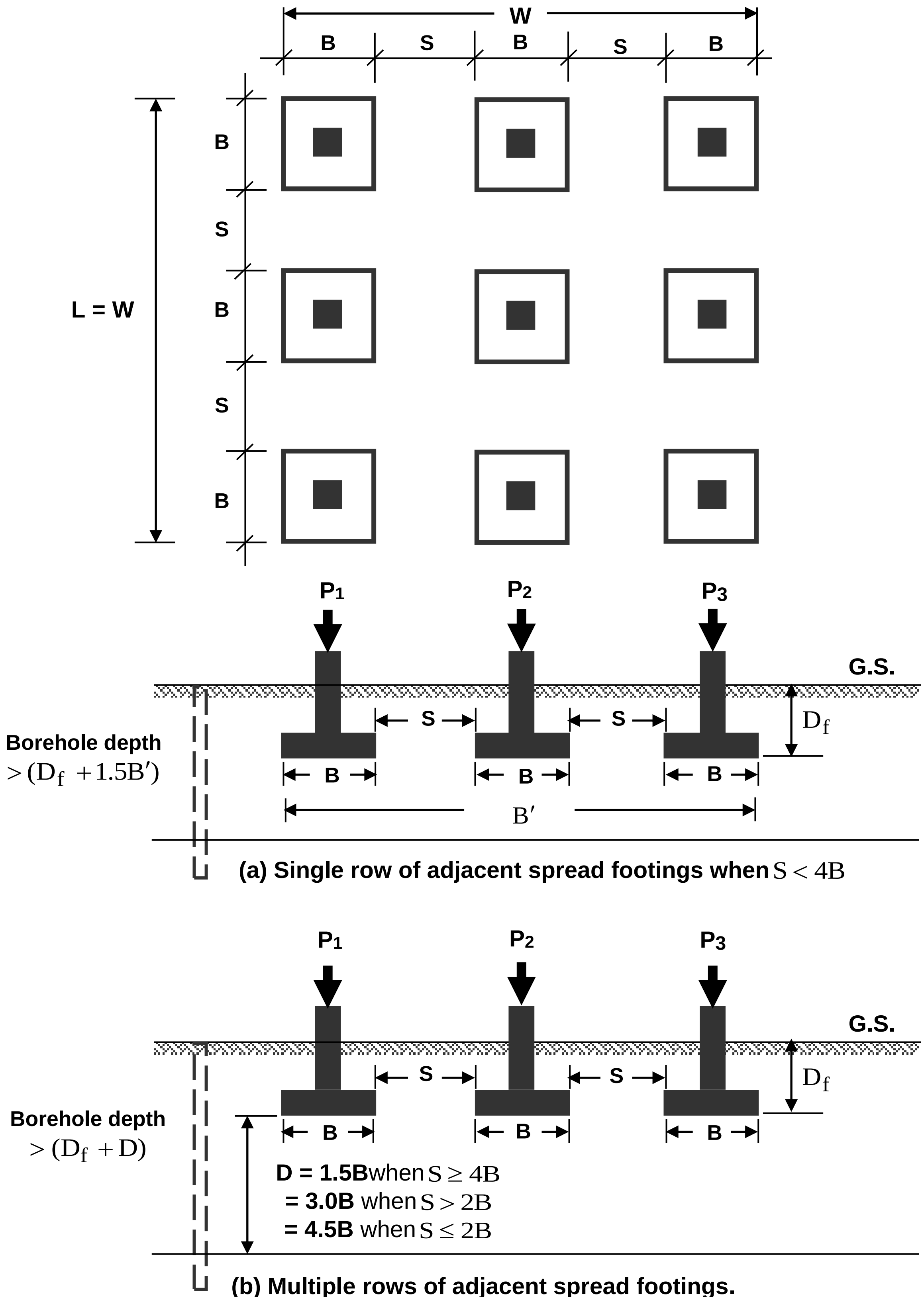
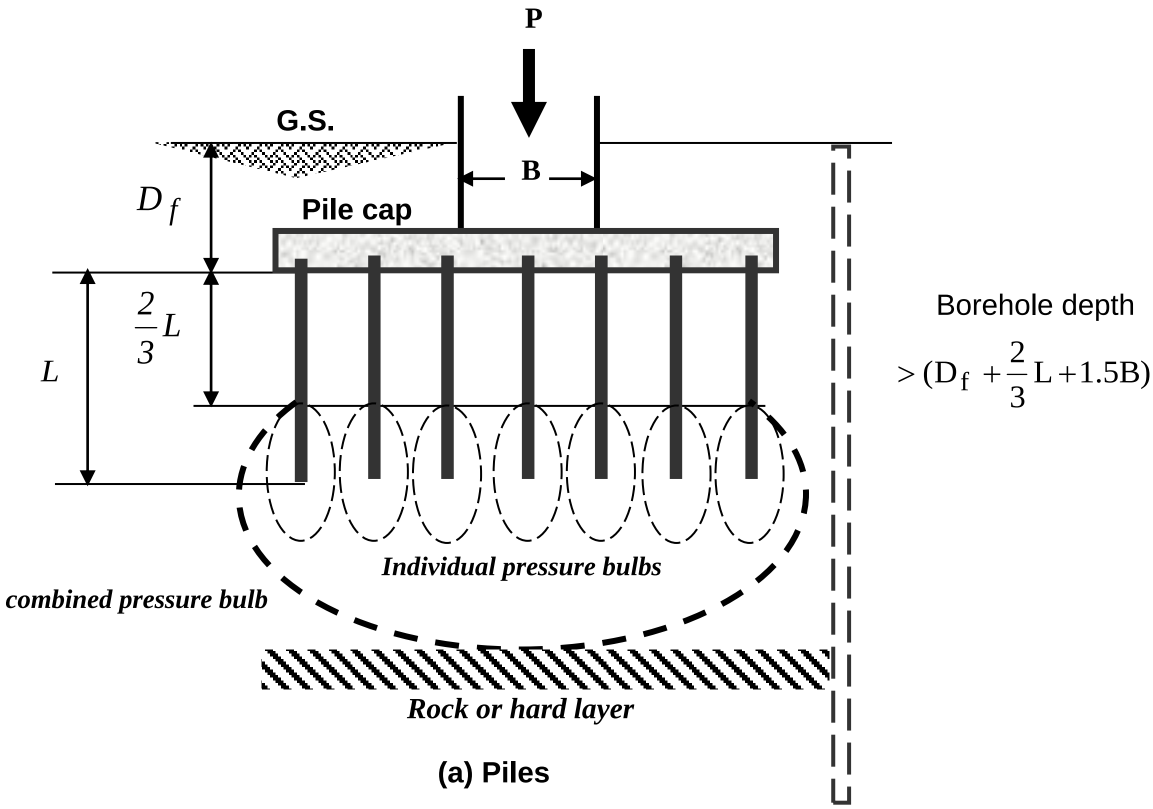
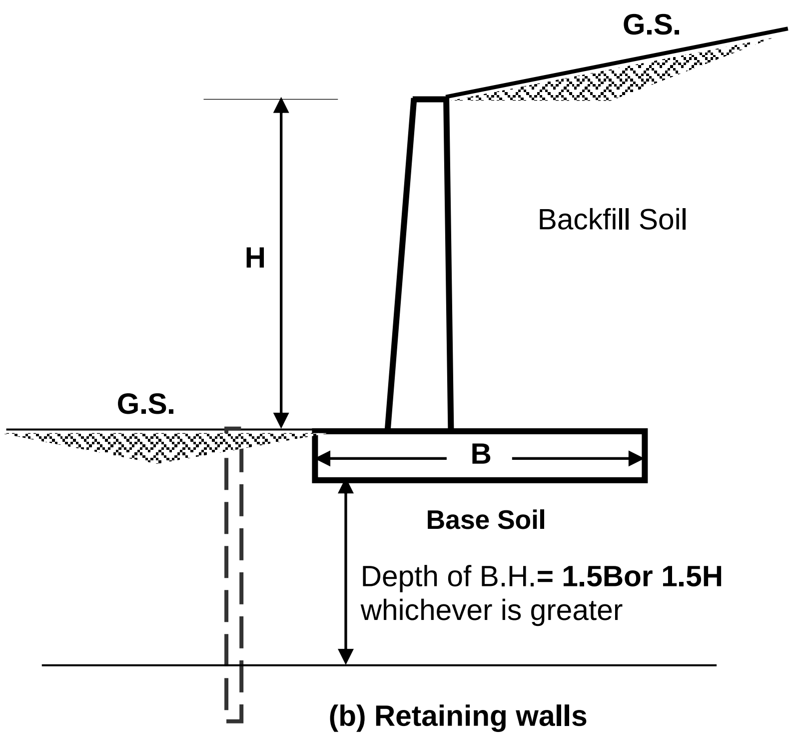


Fig.(2.13):Depth of borings for adjacent spread footings.



(a) Piles



(b) Retaining walls

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

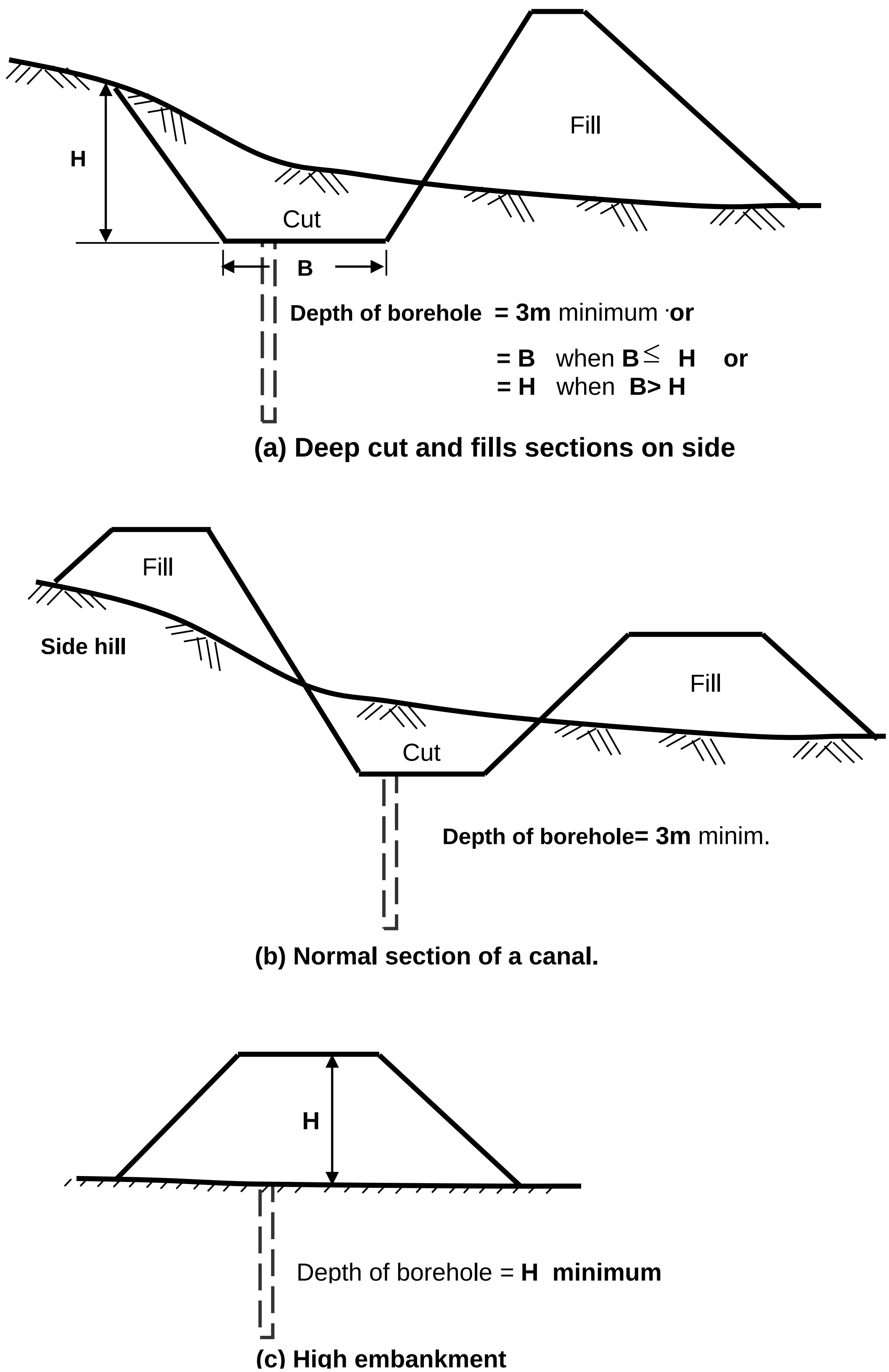
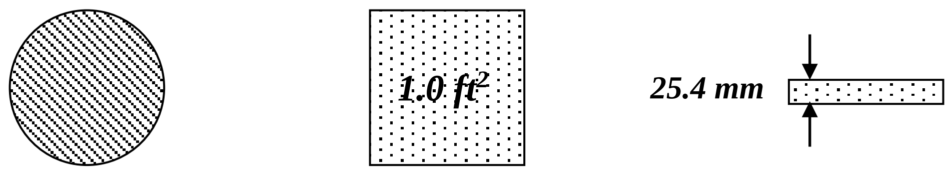


Fig.(2.15): Depth of borings for cuts and fills, canals, and embankments.

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.

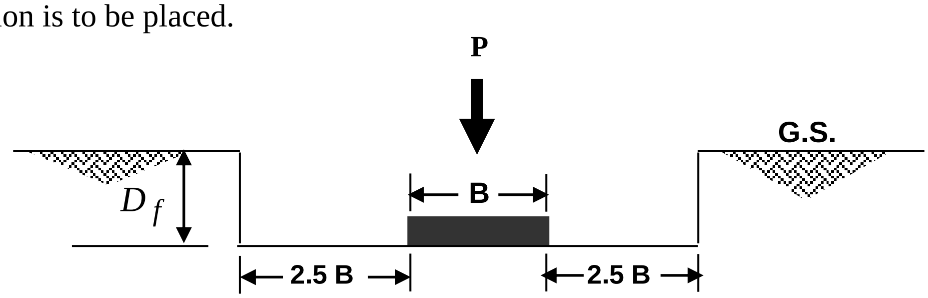
Round plates from (150-750)mm in diameter by 150mm increment (i.e., 150, 300, 450, 600, 750)mm are available as well as square plates of (1.0 ft^2) area. The minimum thickness of plate (1 inch or 25.4mm).



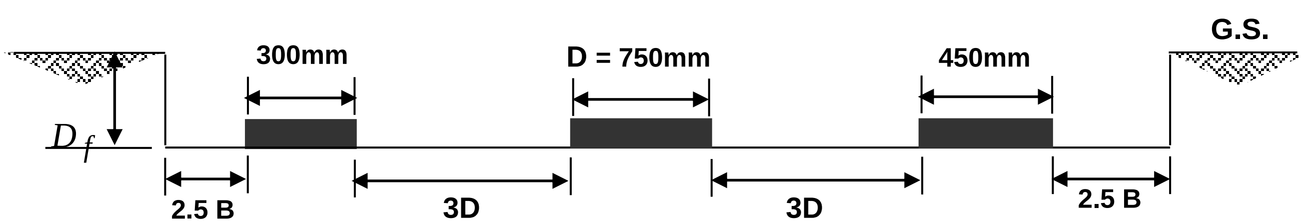
Round plate ***Square plate***

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.

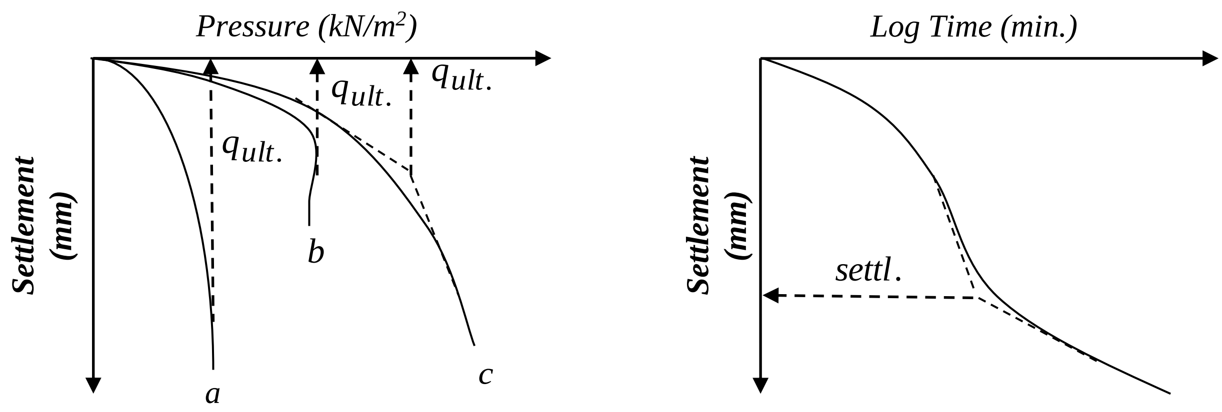
(4) The test is continued until a settlement of 25mm is observed or until the load increments reached 1.5 times the estimated allowable soil pressure.

(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**<u>*(bearing capacity is independent of footing size):*</u>

$$\begin{cases} q_f = q_p \\ s_f = s_p \dfrac{B_f}{B_p} \end{cases} \qquad \textbf{(2.3)}$$

- **For cohesionless soil**<u>*(bearing capacity increases with size of footing):*</u>

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

- **<u>Settlement for both cohesive and cohesionless soils:</u>**

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

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*):***

$$V = A.q + P.s \text{ .... } \mathbf{(2.6)}$$

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 Dynamic or Standard 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)** that driven into the soil.

The test consists 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 cohesive soil (clay) as shown in **Tables (2.6, and 2.7)**.

Table (2.6): Relative density of sands according to results of standard penetration test.

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

2.12.2 Corrections 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 ≤ 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:

$$N'_{70} = N_{field}.C_N..\eta_1..\eta_2..\eta_3..\eta_4 \qquad \textbf{(2.9)}$$

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:

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

- ***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) = E_r / E_{rb} ;

η_2 = rod length correction;

η_3 = sampling method correction; and

η_4 = borehole 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 η_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 $N_{corr.} = 10.\left(\frac{30}{10}\right) = 30...blows/30cm.$

2.12.3 Static or Cone Penetration Test (CPT)

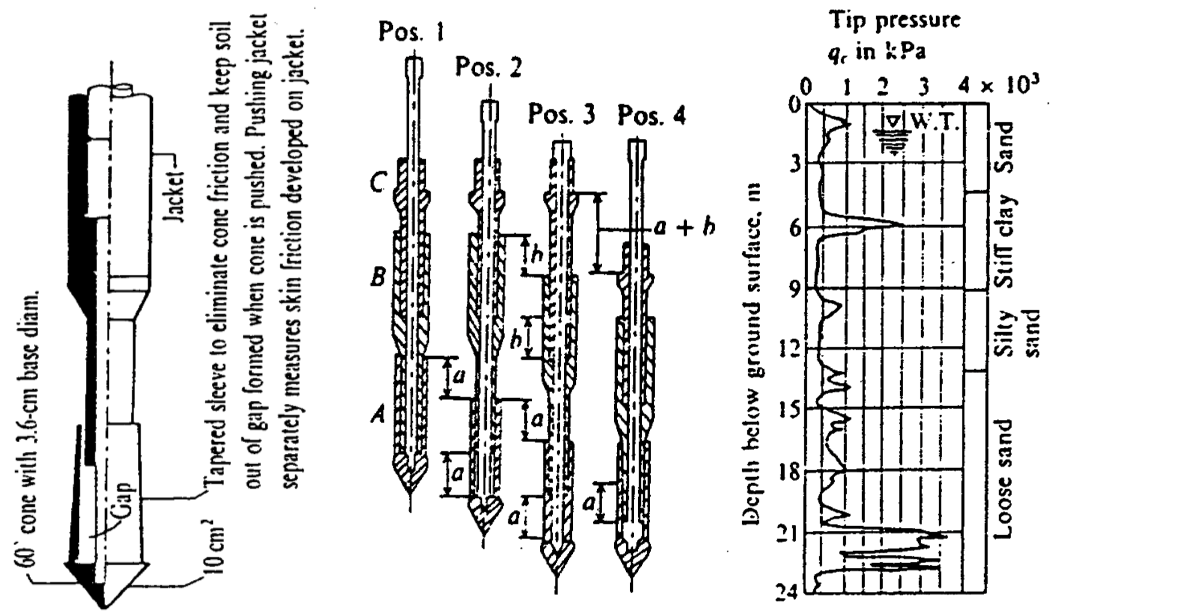
This is a simple static test used for soft clays and fine to medium coarse sands. The test is not applicable in gravels and stiff hard clays. It is performed by pushing the standard cone (according to **ASTM D3441** with a 60^o point and base diameter = 35.7mm with cross-section area of 10 cm^2) into the ground at a rate of (10 – 20) mm/sec. Several cone configurations can be used such as:

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 establish the undrained 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)**.

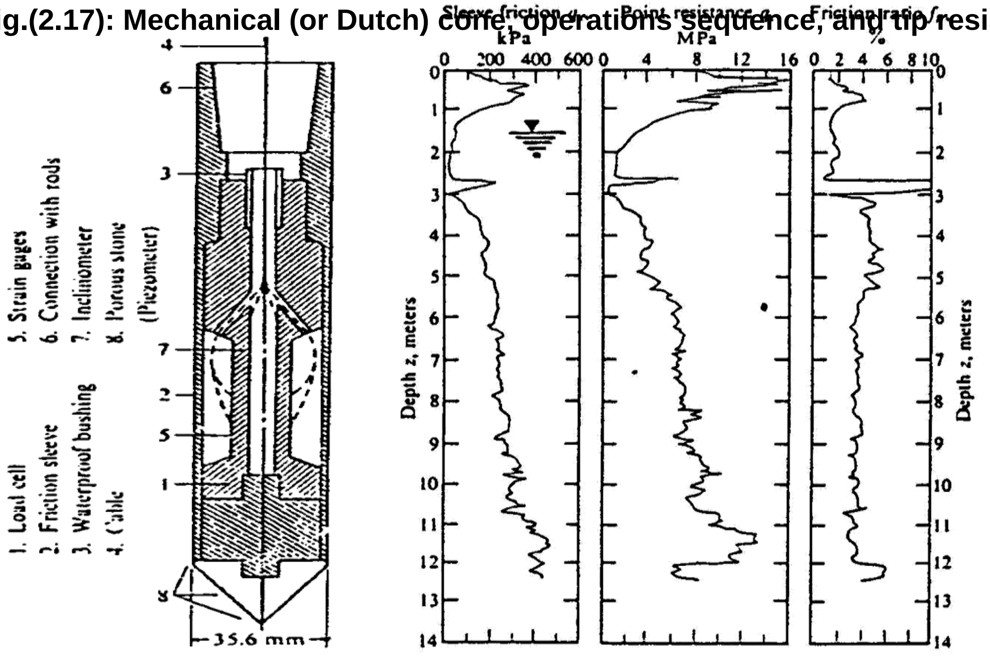


(a) Dutch cone modified to measure both point resistance q_c and skin friction q_f .

(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.

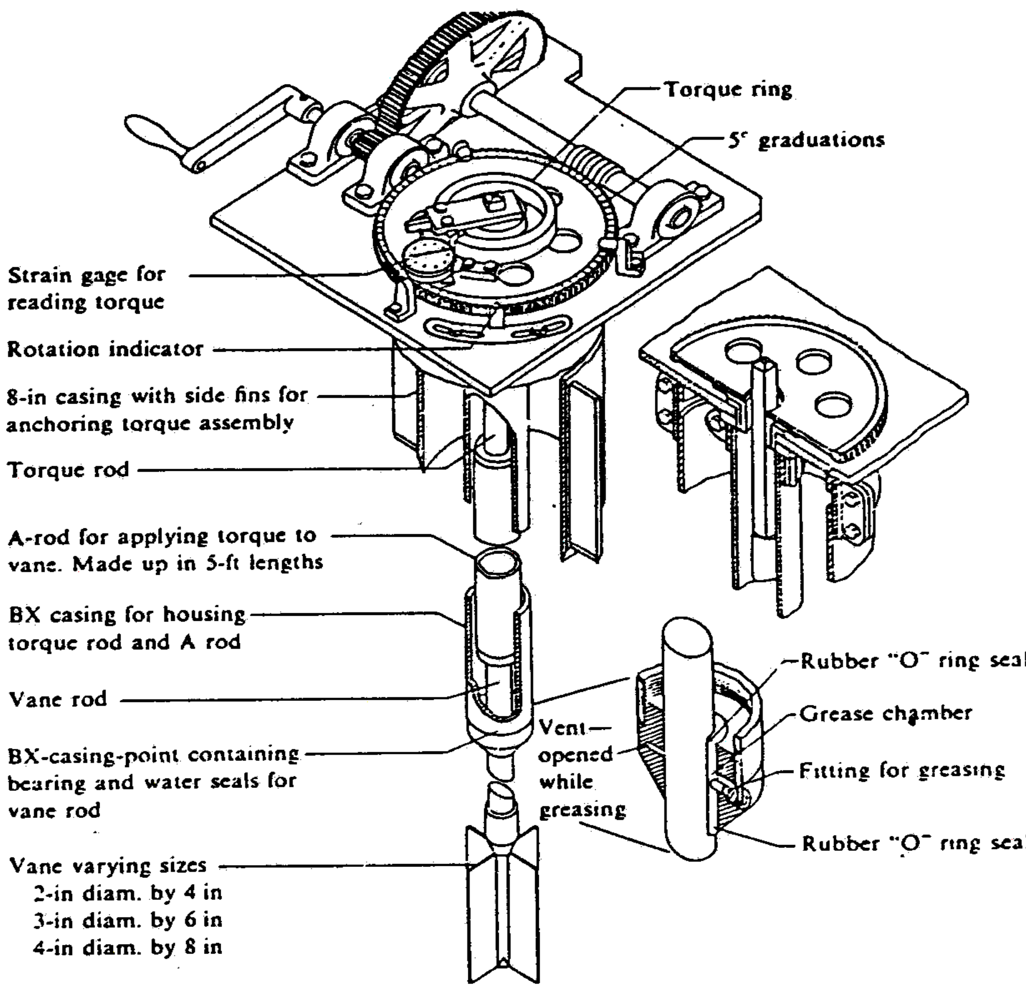
Fig.(2.18): Electric cone and CPT data.

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..

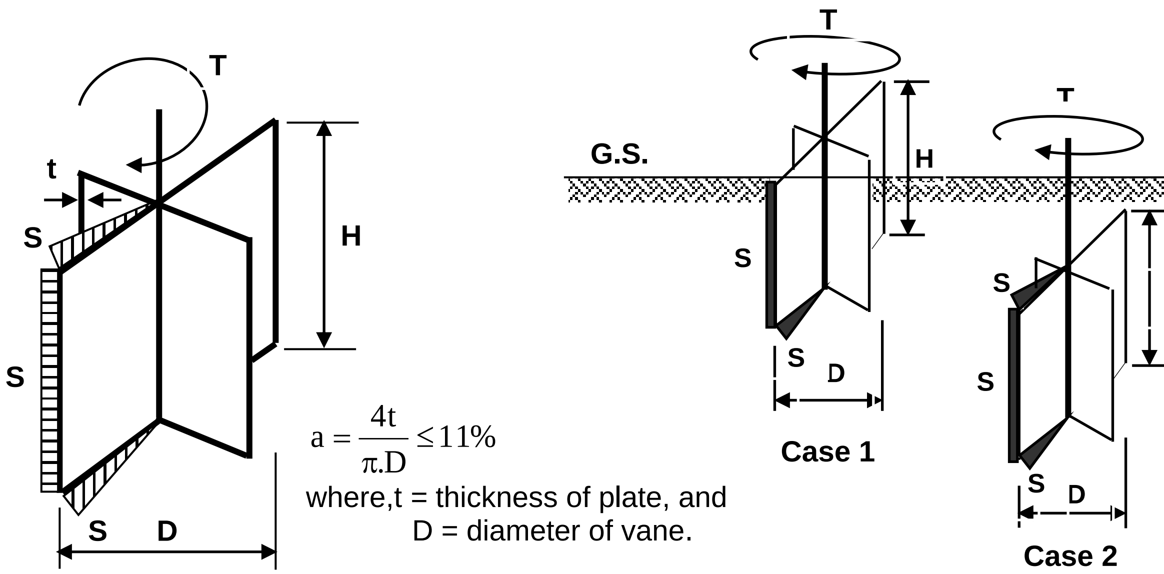


The Bureau of Reclamation vane-shear test aparatus. [*Gibbs et al.* (*1960*), *courtesy of Gibbs and Holtz of the USBR.*)

Fig.(2.19): Vane shear apparatus.

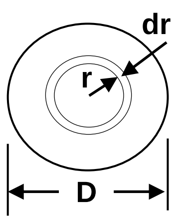
• Procedure:

1. The equipment is installed in place properly either at the ground surface without a hole **(case 1)** or at the bottom of a borehole **(case 2)** and then the vane is pushed into the soil layer to the required depth; **(see Fig.(2.20)).**
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.



$$a = \frac{4t}{\pi.D} \le 11\%$$

where, t = thickness of plate, and
D = diameter of vane.



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 part 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) Total resistance 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: $T = (\pi.D.H.S)\dfrac{D}{2} + \int_{0}^{D/2}(2\pi.r.dr).S.r$

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

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

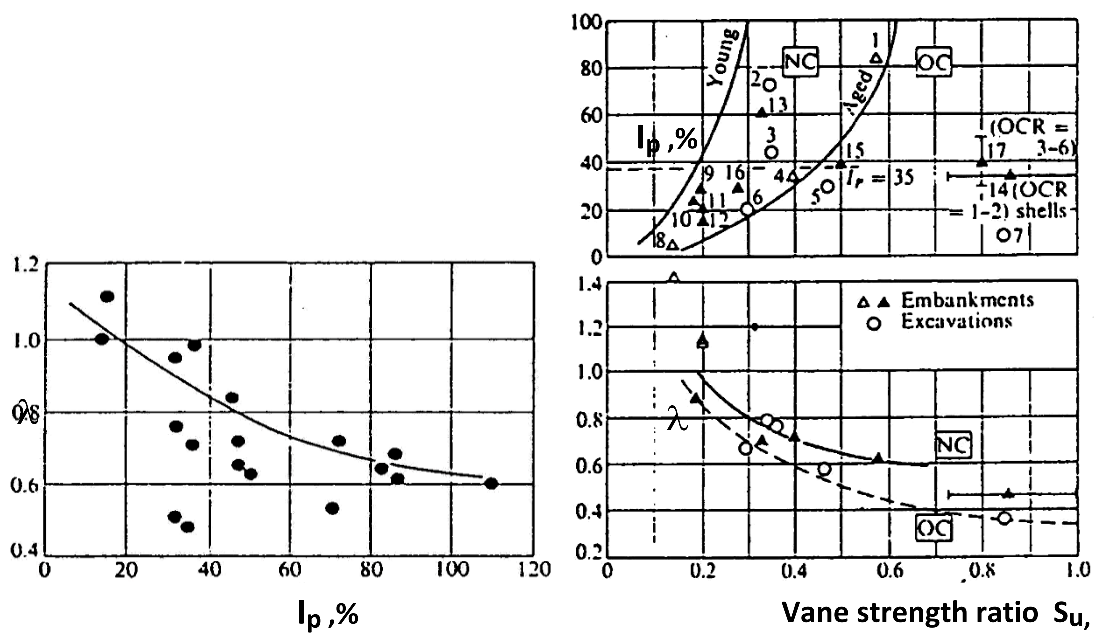
$$\text{or } T = \frac{\pi.D^2 S_u}{2}\left(H + \frac{D}{3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.12)$$

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:

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

where, λ is a correction factor depends on plasticity index I_p and obtained from **Fig.(2.21a)**; Also, **Aas et al. (1986)** proposed another 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 the bottom of a borehole a distance of 600mm. When the sampler was recovered a measurement done inside the tube indicated a recovered sample length of 575mm. Calculate the recovery and area ratios.

Solution:

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

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

Problem (2.2): A three story steel 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:
  $5.S^{0.7} + D_f = 5(3)^{0.7} + 1 = 12m.$
  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 for every (100 to 300) m^2 of footprint area. Since the total footprint area is 16x16=256m^2> 200m^2 (average value), **use one or two borings.**

- **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:

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

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 15.83m) 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: $0.1.(35) = \frac{(35)(5)(1)}{(5+d)(1+d)}, \ldots\ldots d = 4.3m$

(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.**

Problem (2.5): A standard penetration test SPT has been conducted in a coarse sand to a depth of 4.8m below the ground surface. The blow counts obtained in the field were as follows: 0 – 6 in: 4 blows; 6 -12 in: 6 blows; 12 -18 in: 8 blows. The test was conducted using a USA-style donut hammer in a 150mm diameter boring with a standard sampler and 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):**

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

$\eta_2 = 0.85$ (forL $= 4.8$m (rod length) < 6m),

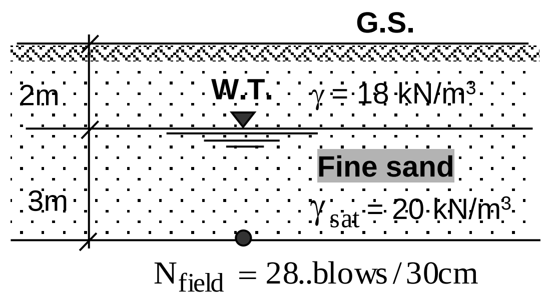
$\eta_3 = 0.90$ (for loose sand with liner),

$\eta_4 = 1.05$ (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:



- **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:

P'_o = 0.9(17.6) + (3.6)(17.6– 9.81) = 43.9 kPa $>$ 25 kPa, therefore, C_N =1.28

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

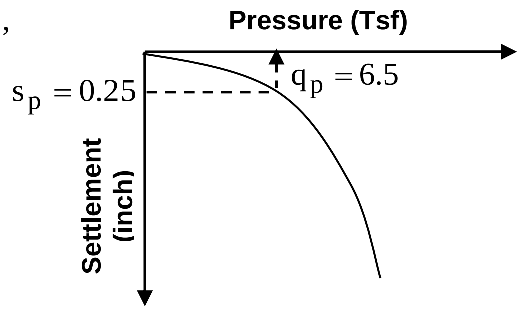
Solution:

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

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 cm, corresponding load =10 tons; relative to 1.0 cm settlement.

Solution:

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

5.6 = 0.123 q + 1.4 s

10 = 0.25 q + 2 s

Solving the two equations, gives: q = 26.9 and s = 1.63.

Again from Eq.(2.6) shown above, the footing area required to carry 45tons load is calculated as:

$45 = B^2 q + 4B s$

$45 = B^2(26.9) + 4B\ (1.63)$

$26.9 B^2 + 6.52 B – 45 = 0$

$B^2 + 0.24 B – 1.67 = 0$

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

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's correction factor λ = 0.80, and

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

CHAPTER 3

BEARING CAPACITY
OF SHALLOW FOUNDATIONS

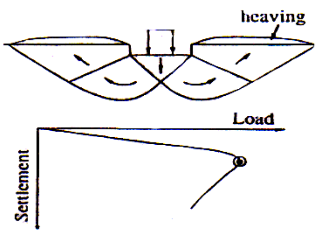
The ultimate soil bearing capacity for a foundation is the pressure that will cause failure in the supporting soil

3.1 MODES OF FAILURE

Failure is defined as mobilizing the full value of soil shear strength accompanied with excessive settlements. For shallow foundations it depends on soil type, particularly its compressibility, and type of loading. Modes of failure in soil at ultimate load are of three types; these are (see ***Fig. 1.5****):*

<table>
<tr><th>Mode of Failure</th><th>Characteristics</th><th>Typical Soils</th></tr>
<tr><td>

1. General Shear failure



</td><td>

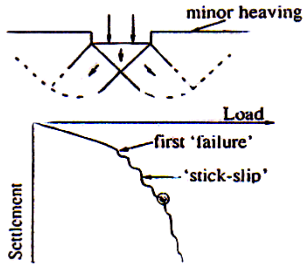
- Well defined continuous slip surface up to ground level,
- Heaving occurs on both sides with final collapse and tilting on one side,
- Failure is sudden and catastrophic,
- Ultimate value is peak value.

</td><td>

- Low compressibility soils
- Very dense sands,
- Saturated clays (NC and OC),
- Undrained shear (fast loading).

</td></tr>
<tr><td>

2. local Shear failure (Transition)



</td><td>

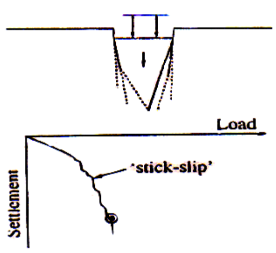
- Well defined slip surfaces only below the foundation, discontinuous either side,
- Large vertical displacements required before slip surfaces appear at ground level,
- Some heaving occurs on both sides with no tilting and no catastrophic failure,
- No peak value, ultimate value not defined.

</td><td>

- Moderate compressibility soils
- Medium dense sands,

</td></tr>
<tr><td>

3. Punching Shear failure



</td><td>

- Well defined slip surfaces only below the foundation, non either side,
- Large vertical displacements produced by soil compressibility,
- No heaving, no tilting or catastrophic failure, no ultimate value.

</td><td>

- High compressibility soils
- Very loose sands,
- Partially saturated clays,
- NC clay in drained shear (very slow loading),
- Peats.

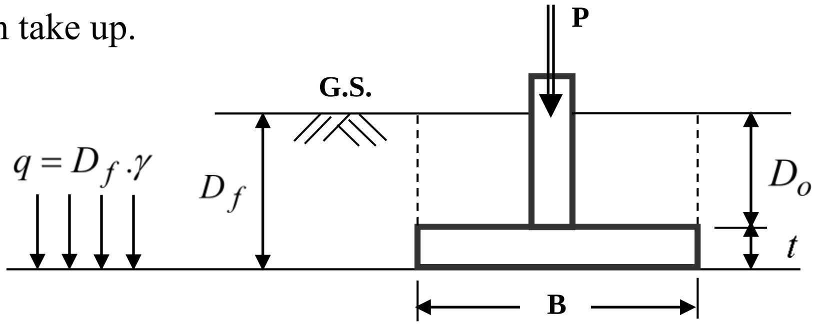
</td></tr>
</table>

<u>Note:</u> General shear failure no exists when: Dr < 30% for sandy soils.
S_t > 10 for clayey soils.

Fig. (3.1): Modes of failure.

3.2 BEARING CAPACITY CLASSIFICATION <u>*(According to column loads)*</u>

- **Gross Bearing Capacity (q_{gross}):** It is the total unit pressure at the base of footing which the soil can take up.



q_{gross} = total pressure at the base of footing = $\sum P_{footing} / area.of.footing$.

where $\sum P_{footing} = p.(column.load) +$ *own wt. of footing + own wt. of earth fill over the footing.* $q_{gross} = (P + \gamma_s.D_o.B.L + \gamma_c.t.B.L)/B.L$

$$q_{gross} = \frac{P}{B.L} + \gamma_s.D_o + \gamma_c.t \quad \textbf{(3.1)}$$

- **Ultimate Bearing Capacity ($q_{ult.}$):** It is the maximum unit pressure or the maximum gross pressure that a soil can stand without shear failure.

- **Allowable Bearing Capacity ($q_{all.}$):** It is the ultimate bearing capacity divided by a reasonable factor of safety.

$$q_{all.} = \frac{q_{ult.}}{F.S} \quad \textbf{(3.2)}$$

- **Net Ultimate Bearing Capacity:** It is the ultimate bearing capacity minus the vertical pressure that is produced on horizontal plain at level of the base of the foundation by an adjacent surcharge.

$$q_{ult.-net} = q_{ult.} - D_f.\gamma \quad \textbf{(3.3)}$$

- **Net Allowable Bearing Capacity ($q_{all.-net}$):** It is the <u>**net safe bearing capacity**</u> or the ultimate bearing capacity divided by a reasonable factor of safety.

Approximate: $$q_{all.-net} = \frac{q_{ult.-net}}{F.S} = \frac{q_{ult.} - D_f.\gamma}{F.S} \quad \text{....................(3.4)}$$

Exact: $$q_{all.-net} = \frac{q_{ult.}}{F.S} - D_f.\gamma \quad \text{...................................(3.5)}$$

3.3 FACTOR OF SAFETY IN DESIGN OF FOUNDATION

The general values of safety factor used in design of footings are 2.5 to 3.0, however, the choice of factor of safety (F.S.) depends on many factors such as:

1. The variation of shear strength of soil,

2. Magnitude of damages,

3. Reliability of soil data such as uncertainties in predicting the $q_{ult.}$,

4. Changes in soil properties due to construction operations,

5. Relative cost of increasing or decreasing F.S., and

6. The importance of the structure, differential settlements and soil strata underneath the structure.

3.4 BEARING CAPACITY REQUIREMENTS

Three requirements must be satisfied in determining bearing capacity of soil. These are:

(1) ***Adequate depth;*** the foundation must be deep enough with respect to environmental effects; such as: frost penetration, seasonal volume changes in the soil, to exclude the possibility of erosion and undermining of the supporting soil by water and wind currents, and to minimize the possibility of damage by construction operations,

(2) ***Tolerable settlements,*** the bearing capacity must be low enough to ensure that both total and differential settlements of all foundations under the planned structure are within the allowable values,

(3) ***Safety against failure,*** this failure is of two kinds:

- The structural failure of the foundation; which may be occur if the foundation itself is not properly designed to sustain the imposed stresses, and
- The bearing capacity failure of the supporting soils.

3.5 FACTORS AFFECTING BEARING CAPACITY

- Type of soil (cohesive or cohesionless).
- Physical features of the foundation; such as size, depth, shape, type, and rigidity.
- Amount of total and differential settlement that the structure can stand.
- Physical properties of soil; such as density and shear strength parameters.
- Water table condition.
- Original stresses.

3.6 METHODS OF DETERMINING BEARING CAPACITY

(a) Bearing Capacity Tables

The bearing capacity values can be found from certain tables presented in building codes, soil mechanics and foundation books; such as that shown in **Table (3.1)**. They are based on experience and can be only used *for preliminary design of light and small buildings as a helpful indication; however, they should be followed by the essential laboratory and field soil tests.*

Table (3.1) neglects the effect of: **(i)** underlying strata, **(ii)** size, shape and depth of footings, **(iii)** type of the structures supported by the footings, **(iv)** there is no specification of the physical properties of the soil in question, and **(v)** assumes that the ground water table level is at foundation level or with depth less than width of footing. Therefore, if water table rises above the foundation level, the hydrostatic water pressure force which affects the base of foundation should be taken into consideration.

Table (3.1): Bearing capacity values according to building codes.

Soil type	Description	Bearing pressure (kg/cm^2)	Notes
Rocks	1. bed rocks. 2. sedimentary layer rock (hard shale, sand stone, siltstone).	70 30	Unless they are affected by water.

	3. schist or erdwas. 4. soft rocks.	20 13		
Cohesionless soil		Dry	submerged	Footing width 1.0 m.
	1. well compacted sand or sand mixed with gravel. 2. sand, loose and well graded or loose mixed sand and gravel. 3. compacted sand, well graded. 4. well graded loose sand.	3.5-5.0 1.5-3.0 1.5-2.0 0.5-1.5	1.75-2.5 0.5-1.5 0.5-1.5 0.25-0.5	
Cohesive soil	1. very stiff clay 2. stiff clay 3. medium-stiff clay 4. low stiff clay 5. soft clay 6. very soft clay 7. silt soil	2-4 1-2 0.5-1 0.25-0.5 up to 0.2 0.1-0.2 1.0-1.5		It is subjected to settlement due to consolidation

(b) Field Load Test

This test is fully explained in **(chapter 2)**.

(c) Bearing Capacity Equations

Several bearing capacity equations were developed *for the case of **general shear failure*** *by many researchers as presented in Table (3.2); see **Tables** (3.3, 3.4 and 3.5) for related factors.*

Table (3.2): Bearing capacity equations by the several authors indicated.

- Terzaghi (see **Table 3.3** for typical values for $K_{P\gamma}$ values)

$$q_{ult.} = cN_c.S_c + \bar{q}N_q + 0.5.B.\gamma.N\gamma.S_\gamma$$

$$N_q = \frac{e^{2[0.75\pi.-\frac{\phi}{2}(\frac{\pi}{180})].\tan\phi}}{2\cos^2(45+\phi/2)}; \qquad N_c = (N_q - 1).\cot\phi; \qquad N_\gamma = \frac{\tan\phi}{2}\left(\frac{k_{P\gamma}}{\cos^2\phi} - 1\right)$$

where a close approximation of $k_{P\gamma} \approx 3.\tan^2\left(45 + \frac{(\phi+33)}{2}\right)$.

	Strip	circular	square	rectangular
S_c =	1.0	1.3	1.3	(1+ 0.3 B / L)
S_γ =	1.0	0.6	0.8	(1- 0.2 B / L)

- Meyerhof (see **Table 3.4** for shape, depth, and inclination factors)

Vertical load: $q_{ult.} = c.N_c.S_c.d_c + q.N_q.S_q.d_q + 0.5.B.\gamma.N_\gamma.S_\gamma.d_\gamma$

Inclined load: $q_{ult.} = c.N_c.d_c.i_c + q.N_q.d_q.i_q + 0.5.B.\gamma.N_\gamma.d_\gamma.i_\gamma$

$$N_q = e^{\pi.\tan\phi}\tan^2(45+\phi/2); \qquad N_c = (N_q - 1).\cot\phi; \qquad N_\gamma = (N_q - 1).\tan(1.4\phi)$$

- Hansen (see **Table 3.5** for shape, depth, and inclination factors)

$$\text{For..}\phi > 0: \quad q_{ult.} = cN_cS_cd_ci_cg_cb_c + qN_qS_qd_qi_qg_qb_q + 0.5.B.\gamma.N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$\text{For..}\phi = 0: \quad q_{ult.} = 5.14S_u(1 + S'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$$

$$N_q = e^{\pi.\tan\phi}\tan^2(45+\phi/2); \qquad N_c = (N_q - 1).\cot\phi; \qquad N_\gamma = 1.5(N_q - 1).\tan\phi$$

- Vesic (see **Table 3.5** for shape, depth, and inclination factors)

Use Hansen's equations above

$$N_q = e^{\pi.\tan\phi}\tan^2(45+\phi/2); \qquad N_c = (N_q - 1).\cot\phi; \qquad N_\gamma = 2(N_q + 1).\tan\phi$$

- *All the bearing capacity equations above are based on general shear failure in soil.*

- **Note:** *Due to scale effects,* N_γ *and then the ultimate bearing capacity decreases with increase in size of foundation.* Therefore, Bowle's (1996) suggested that for (**B > 2m**), with any bearing capacity equation of **Table (3.2)**, the term ($0.5B\gamma.N_\gamma S_\gamma d_\gamma$) must be multiplied by a reduction factor: $r_\gamma = 1 - 0.25\log\left(\frac{B}{2}\right)$; i.e., $0.5B\gamma.N_\gamma S_\gamma d_\gamma r_\gamma$

B (m)	2	2.5	3	3.5	4	5	10	20	100
r_γ	1	0.97	0.95	0.93	0.92	0.90	0.82	0.75	0.57

Table (3.3): Bearing capacity factors for Terzaghi's equation.

ϕ,..deg	N_c	N_q	N_γ	$K_{P\gamma}$
0	5.7 +	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

+ = 1.5 π + 1

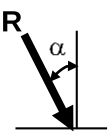
Table (3.4): Shape, depth and inclination factors for Meyerhof's equation.

For	**Shape Factors**	**Depth Factors**	**Inclination Factors**
Any ϕ	$S_c = 1 + 0.2.K_P\frac{B}{L}$	$d_c = 1 + 0.2\sqrt{K_P}\frac{D_f}{B}$	$i_c = i_q = \left(1 - \frac{\alpha^\circ}{90^\circ}\right)^2$
$\phi \geq 10^\circ$	$S_q = S_\gamma = 1 + 0.1.K_P\frac{B}{L}$	$d_q = d_\gamma = 1 + 0.1\sqrt{K_P}\frac{D_f}{B}$	$i_\gamma = \left(1 - \frac{\alpha^\circ}{\phi^\circ}\right)^2$
$\phi = 0$	$S_q = S_\gamma = 1.0$	$d_q = d_\gamma = 1.0$	$i_\gamma = 0$

Where: $K_P = \tan^2(45 + \phi/2)$

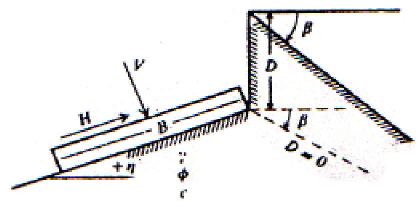
$\alpha =$ angle of resultant measured from vertical without a sign.

B, L , D_f = width, length, and depth of footing.



Note:- When $\phi_{triaxial}$ is used for plan strain, adjust ϕ as: $\phi_{Ps} = (1.1 - 0.1\frac{B}{L})\phi_{triaxial}$

Table (3.5): Shape, depth, inclination, ground and base factors for use in Hansen or Vesic bearing capacity equations of Table (3.2). **(1)** Factors apply to either method unless subscripted with (H) or (V). **(2)** Use primed factors when $\phi = 0$.

Shape factors	Depth factors	Inclination factors	Ground Factors (Base on slope)
$S'_c = 0.2\dfrac{B}{L}$ $S_c = 1 + \dfrac{N_q}{N_c}\cdot\dfrac{B}{L}$ $S_c = 1.0$ for strip $S_q = 1 + \dfrac{B}{L}\tan\phi$ $S_\gamma = 1 - 0.4\dfrac{B}{L}$	$d'_c = 0.4.k$ $d_c = 1 + 0.4.k$ $d_q = 1 + 2\tan\phi.(1-\sin\phi)^2 k$ $d_\gamma = 1.0$ for all ϕ $k = \dfrac{D_f}{B}$ for $\dfrac{D_f}{B} \le 1$ $k = \tan^{-1}\dfrac{D_f}{B}$ **(rad)** for $\dfrac{D_f}{B} > 1$	$i'_{c(H)} = 0.5 - 0.5\sqrt{1 - \dfrac{H}{A_f.C_a}}$ $i'_{c(V)} = 1 - \dfrac{m.H}{A_f.C_a.N_c}$ $i_c = i_q - \dfrac{1-i_q}{N_q - 1}$ (Hansen and Vesic) $i_{q(H)} = \left(1 - \dfrac{0.5H}{V + A_f C_a.\cot\phi}\right)^5$ $i_{q(V)} = \left(1 - \dfrac{H}{V + A_f C_a.\cot\phi}\right)^m$ $i_{\gamma(H)} = \left(1 - \dfrac{0.7H}{V + A_f C_a.\cot\phi}\right)^5$ for ($\eta = 0$) $i_{\gamma(H)} = \left(1 - \dfrac{(0.7 - \eta^\circ/450)H}{V + A_f C_a.\cot\phi}\right)^5$ for ($\eta > 0$) $i_{\gamma(V)} = \left(1 - \dfrac{H}{V + A_f C_a.\cot\phi}\right)^{m+1}$	$g'_c = \dfrac{\beta^\circ}{147^\circ}$ For Vesic use: $N_\gamma = -2\sin\beta$ for $\phi = 0$ $g_c = 1 - \dfrac{\beta^\circ}{147^\circ}$ $g_{q(H)} = g_{\gamma(H)} = (1 - 0.5\tan\beta)^5$ $g_{q(V)} = g_{\gamma(V)} = (1 - \tan\beta)^2$ **Base factors** (Tilted base) $b'_c = \dfrac{\eta^\circ}{147^\circ}$ $b_c = 1 - \dfrac{\eta^\circ}{147^\circ}$ $b_{q(H)} = \exp(-2\eta.\pi.\tan\phi/180)$ $b_{\gamma(H)} = \exp(-2.7\eta.\pi.\tan\phi/180)$ $b_{q(V)} = b_{\gamma(V)} = (1 - \eta.\pi.\tan\phi/180)^2$
Where $e_B, .e_L$ = Eccentricity of load from center of footing area A_f = Effective footing area $B'.x.L'$ C_a = Adhesion to base = cohesion or a reduced value D_f = Depth of footing (used with B and not B′) H = Horizontal component of load with $H \le C_a A_f + V\tan\delta$ V = Total vertical load on footing β = Slope of ground away from base with downward = (+) δ = Friction angle between base and soil; usually $\delta = \phi$ for concrete on soil η = Tilt angle of base from horizontal with (+) upward as usual case. **GENERAL NOTES** **1.** Do not use S_i in combination with i_i . **2.** Can use S_i in combination with d_i, g_i, and b_i **3.** For $L/B \le 2$ use $\phi_{tr.}$ For $L/B > 2$ use $\phi_{Ps} = 1.5\,\phi_{tr.} - 17$ For $\phi \le 34^\circ$ $\phi_{Ps} = \phi_{tr.}$		$m = m_B = \dfrac{2 + B/L}{1 + B/L}$ for **H** parallel to **B** $m = m_L = \dfrac{2 + L/B}{1 + L/B}$ for **H** parallel to **L** ***Note:*** $i_q, i_\gamma > 0$	*Note:* $\beta + \eta < 90^\circ$ and $\beta < \phi$ 

3.7 WHICH EQUATIONS TO USE?

Of the bearing capacity equations previously discussed, the most widely used equations are Meyerhof's and Hansen's. While Vesic's equation has not been much used (but is the suggested method in the American Petroleum Institute, RP2A Manual, 1984).

Table (3.6) : Which equations to use.

Use	*Best for*
Terzaghi	• *Very cohesive soils where D/B ≤ 1 or for a quick estimate of* q_{ult}. *to compare with other methods,* • *Somewhat simpler than Meyerhof's, Hansen's or Vesic's equations; which need to compute the shape, depth, inclination, base and ground factors,* • *Suitable for a concentrically loaded horizontal footing,* • *Not applicable for columns with moment or tilted forces,* • *More conservative than other methods.*
Meyerhof, Hansen, Vesic	• *Any situation which applies depending on user preference with a particular method.*
Hansen, Vesic	• *When base is tilted; when footing is on a slope or when D/B >1.*

3.8 EFFECT OF SOIL COMPRESSIBILITY <u>**(local shear failure)**</u>

1. For clays sheared in drained conditions, Terzaghi (1943) suggested that the shear strength parameters c and ϕ should be reduced as:

$$c^* = 0.67c' \quad \text{and} \quad \phi^* = \tan^{-1}(0.67 \tan\phi') \quad \textbf{(3.6)}$$

2. For loose and medium dense sands (when $D_r \le 0.67$), Vesic (1975) proposed:

$$\phi^* = \tan^{-1}(0.67 + D_r - 0.75D_r^2)\tan\phi' \quad \textbf{(3.7)}$$

where D_r is the relative density of the sand, recorded as a fraction.

<u>**Note:** For dense sands ($D_r > 0.67$) the strength parameters need not be reduced, since the general shear mode of failure is likely to apply.</u>

BEARING CAPACITY EXAMPLES (1)

Example (1): **Determine the allowable bearing capacity of a strip footing shown below using Terzaghi and Hansen Equations if c = 0, $\phi = 30°$, D_f = 1.0m , B = 1.0m , $\gamma_{soil} = 19$ kN/m^3, the water table is at ground surface, and SF=3.**

Solution:

(a) By Terzaghi's equation:

$$q_{ult.} = cN_c.S_c + qN_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma$$

Shape factors: from table (3.2), for strip footing $S_c = S\gamma = 1.0$

Bearing capacity factors: from table (3.3), for $\phi = 30°$, $N_q = 22.5,..N_\gamma = 19.7$

$q_{ult.} = 0 + 1.0\,(19\text{-}9.81)22.5 + 0.5x1(19\text{-}9.81)19.7x1.0 = 297\ kN/m^2$

$q_{all.}$=297/3 = $\boxed{99\ kN/m^2}$

(b) By Hansen's equation:

$for..\phi > 0:$

$$q_{ult.} = cN_cS_cd_ci_cg_cb_c + qN_qS_qd_qi_qg_qb_q + 0.5\gamma.B.N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

Since c = 0, any factors with subscript c do not need computing. Also, all $g_i..and..b_i$ *factors are 1.0; with these factors identified the Hansen's equation simplifies to:*

$q_{ult.} = \bar{q}N_qS_qd_q + 0.5\gamma'.B.N_\gamma S_\gamma d_\gamma$

From table (3.5): $\begin{cases} for..\phi \le 34°..use..\phi_{ps} = \phi_{tr} \\ for\ L/B > 2..use..\phi_{ps} = 1.5\phi_{tr} - 17 \end{cases}$, $\therefore .use.\phi_{ps} = 1.5\phi_{tr} - 17$

$$\therefore .use.\phi_{ps} = 1.5\phi_{tr} - 17, \quad 1.5\ x\ 30 - 17 = 28°\ ,$$

Bearing capacity factors: from table (3.4), for $\phi = 28°, N_q = 14.7,..N_\gamma = 10.9$

Shape factors: from table (3.5), $S_\gamma = S_q = 1.0$,

Depth factors: from table (3.5),

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D}{B},$$

$$d_q = 1 + 2.\tan 28(1-\sin 28)^2 \frac{1}{1} = 1.29, \quad and \quad d_\gamma = 1.0$$

$q_{ult.} = 1.0\ (19\text{-}9.81)14.7x1.29 + 0.5x1(19\text{-}9.81)10.9x1.0 = 224.355\ kN/m^2$

$q_{all.}$=224.355/3 = $\boxed{74.785\ kN/m^2}$

Example (2): A footing load test produced the following data:

D_f = **0.5m, B = 0.5m, L = 2.0m,** $\gamma'_{soil} = 9.31$ **kN/m³,** $\phi_{tr} = 42.5°$ **, c = 0,**

$P_{ult.}(measured) = 1863.kN$, $q_{ult.}(measured) = 1863/0.5x2 = 1863$ **kN/m².**

Required: compute $q_{ult.}$ by Hansen's and Meyerhof's equations and compare computed with measured values.

Solution:

(a) By Hansen's equation:

Since c = 0, and all $g_i..and..b_i$ factors are 1.0; the Hansen's equation simplifies to:

$$q_{ult.} = \bar{q} N_q S_q d_q + 0.5\gamma'.B.N_\gamma S_\gamma d_\gamma$$

From table (3.5): L/B = 2/0.5 = 4 > 2 $\therefore ..use..\phi_{ps} = 1.5\phi_{tr} - 17$,

1.5 x 42.5 - 17= 46.75° $\longrightarrow$ *take...*$\phi = 47°$

Bearing capacity factors: from table (3.2)

$N_q = e^{\pi.\tan\phi}..\tan^2(45+\phi/2)$, $\quad N_\gamma = 1.5(N_q - 1)\tan\phi$

for $\phi = 47°$: $\quad N_q = 187.2$, $\quad N_\gamma = 299.5$

Shape factors: from table (3.5),

$$S_q = 1 + \frac{B}{L}\tan\phi = 1 + \frac{0.5}{2.0}\tan 47 = 1.27, \qquad S_\gamma = 1 - 0.4\frac{B}{L} = 1 - 0.4\frac{0.5}{2.0} = 0.9$$

Depth factors: from table (3.5),

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 \frac{D}{B}, d_q = 1 + 2\tan 47(1 - \sin 47)^2 \frac{0.5}{0.5} = 1.155, d_\gamma = 1.0$$

$q_{ult.} = 0.5\ (9.31)187.2x1.27x1.155 + 0.5x0.5(9.31)299.5x0.9x1.0 = \boxed{1905.6\ kN/m^2}$

versus 1863 kN/m^2 measured.

(b) By Meyerhof's equation:

From table (3.2) for vertical load with c = 0:

$$q_{ult.} = \bar{q} N_q S_q d_q + 0.5\gamma'.B.N_\gamma S_\gamma d_\gamma$$

From table (3.4): $\phi_{ps} = (1.1 - 0.1\frac{B}{L})\phi_{tr},\ (1.1 - 0.1\frac{0.5}{2.0})42.5 = 45.7,\ \ take...\phi = 46°$

Bearing capacity factors: from table (3.2)

$$N_q = e^{\pi.\tan\phi}..\tan^2(45 + \phi/2),\ N_\gamma = (N_q - 1)\tan(1.4\phi)$$

for $\phi = 46°$: $N_q = 158.5,\ N_\gamma = 328.7$

Shape factors: from table (3.4)

$$K_p = \tan^2(45 + \phi/2) = 6.13, \quad S_q = S_\gamma = 1 + 0.1.K_p\frac{B}{L} = 1 + 0.1(6.13)\frac{0.5}{2.0} = 1.15$$

Depth factors: from table (3.4)

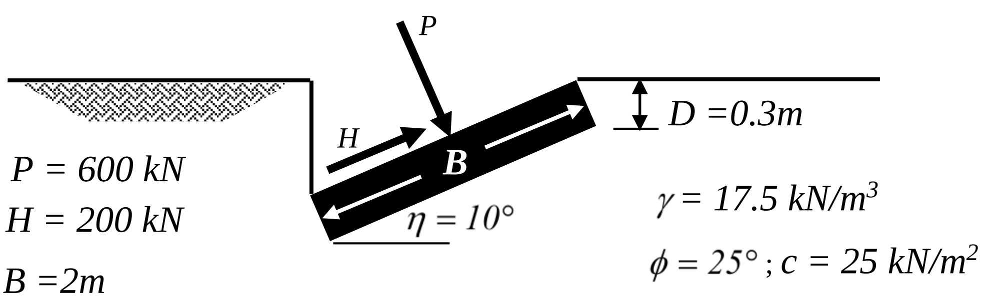
$$\sqrt{K_p} = 2.47, \quad d_q = d_\gamma = 1 + 0.1.\sqrt{K_p}\frac{D}{B} = 1 + 0.1(2.47)\frac{0.5}{0.5} = 1.25$$

$q_{ult.} = 0.5(9.31)158.5x1.15x1.25 + 0.5x0.5(9.31)328.7x1.15x1.25 = 2160.4\ kN/m^2$

versus 1863 kN/m^2 measured

$\therefore$ **Both Hansen's and Meyerhof's eqs. give over-estimated $q_{ult.}$ compared with measured.**

Example (3): **A 2.0x2.0m footing has the geometry and load as shown below. Is the footing adequate with a SF=3.0?.**



Solution:

We can use either Hansen's, or Meyerhof's or Vesic's equations. An arbitrary choice is Hansen's method.

Check sliding stability:

use $\delta = \phi;\ C_a = c$ *and* $A_f = 2x2 = 4m^2$

$H_{max.} = A_f C_a + V\tan\delta = 4x25 + 600\tan 25° = 280 > 200\ kN$ ***(O.K. for sliding)***

Bearing capacity By Hansen's equation:

$with..inclination..factors..all..S_i = 1.0$

$$q_{ult.} = cN_c.d_c.i_c.b_c + \bar{q}N_q.d_q.i_q.b_q + 0.5\gamma.B.N_\gamma.d_\gamma.i_\gamma.b_\gamma$$

Bearing capacity factors from table (3.2):

$N_c = (N_q - 1).\cot\phi$, $\quad N_q = e^{\pi.\tan\phi}..\tan^2(45 + \phi/2)$, $\quad N_\gamma = 1.5(N_q - 1)\tan\phi$

for $\phi = 25°$: $\quad N_c = 20.7$, $\quad N_q = 10.7$, $\quad N_\gamma = 6.8$

Depth factors from table (3.5):

for D =0.3m, and B = 2m, D/B = 0.3/2=0.15 < 1.0 (shallow footing)

$d_c = 1 + 0.4\dfrac{D}{B} = 1 + 0.4(0.15) = 1.06$,

, $d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\dfrac{D}{B} = 1 + 0.311(0.15) = 1.05$

$d_\gamma = 1.0$

Inclination factors from table (3.5):

$i_q = (1 - \dfrac{0.5H}{V + A_f.c.\cot\phi})^5 = (1 - \dfrac{0.5x200}{600 + 4x25x\cot 25})^5 = 0.52$,

$i_c = i_q - \dfrac{(1 - i_q)}{(N_q - 1)} = 0.52 - \dfrac{1 - 0.52}{10.7 - 1} = 0.47$,

$$for..\eta > 0 : i_\gamma = (1 - \frac{(0.7 - \eta^\circ / 450)H}{V + A_f .c.\cot\phi})^5 = (1 - \frac{(0.7 - 10/450)200}{600 + 4x25x\cot 25})^5 = 0.40$$

<u>The base factors</u> $for..\eta = 10^\circ(0.175..radians)$ *<u>from table (3.5):</u>*

$$b_c = 1 - \frac{\eta^\circ}{147^\circ} = 1 - \frac{10}{147} = 0.93,$$

$$b_q = e^{(-2\eta\tan\phi)} = e^{(-2(0.175)\tan 25)} = 0.85, b_\gamma = e^{(-2.7\eta\tan\phi)} = e^{(-2.7(0.175)\tan 25)} = 0.80$$

$$q_{ult.} = 25(20.7)(1.06)(0.47)(0.93) + 0.3(17.5)(10.7)(1.05)(0.52)(0.85)$$
$$+ 0.5(17.5)(2.0)(6.8)(1)(0.40)(0.80) = 304\ kN/m^2$$

$$q_{all.} = 304/3 = 101.3\ kN/m^2$$

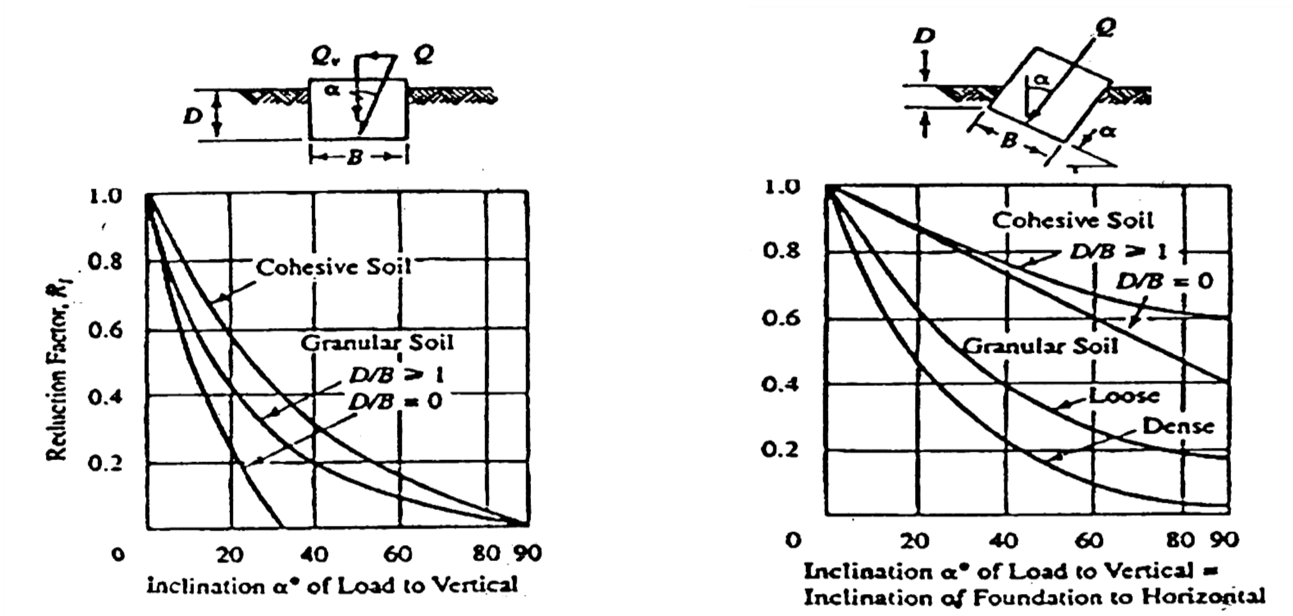
$P_{all.} = q_{all.}.A_f = 101.3(4) = 405.2\ kN < 600\ kN$ *(the given load),* $\therefore$ *B=2m is not adequate and, therefore it must be increased and* $P_{all.}$ *recomputed and checked.*

3.9 FOOTINGS WITH INCLINED OR ECCENTRIC LOADS

- **INCLINED LOAD:**

If a footing is subjected to an inclined load (see Fig.3.7), the inclined load Q can be resolved into vertical and horizontal components. The vertical component Q_v can then be used for bearing capacity analysis in the same manner as described previously (Table 3.2). After the bearing capacity has been computed by the normal procedure, it must be corrected by an R_i factor using Fig.(3.7) as:

$$\therefore \quad \boxed{q_{ult.(inclined..load)} = q_{ult.(vertical..load)} .x.R_i} \quad \textbf{......................................(3.8)}$$



(a) Horizontal foundation ***(b) Inclined foundation***

Figure (3.7): Inclined load reduction factors.

Important Notes:

- *Remember that in this case, Meyerhof's bearing capacity equation for inclined load (from Table 3.2) can be used directly:*

$$q_{ult.(inclined.load)} = \boxed{cN_c d_c i_c + \bar{q} N_q d_q i_q + 0.5\gamma'.B.N_\gamma d_\gamma i_\gamma} \quad \textbf{……………….(3.9)}$$

- The footings stability with regard to the inclined load's horizontal component also must be checked by calculating ***the factor of safety against sliding*** as follows:

$$Fs_{(slididing)} = \frac{H_{max.}}{H} \quad \textbf{…………………………………………..……………...(3.10)}$$

where:

H = *the inclined load's horizontal component,*

$\boxed{H_{max.} = the.maximum.resisting.force = A'_f .C_a + \sigma' \tan\delta \text{ .... for } (c-\phi) \text{ soils;}}$ or

$H_{max.} = A'_f .C_a$ *……. for the undrained case in clay* ($\phi_u = 0$); *or*

$H_{max.} = \sigma' \tan\delta$ *……. for a sand and the drained case in clay* ($c' = 0$).

$A'_f = effective..area = B'.L'$

$$C_a = adhesion = \alpha.C_u$$

$$where ... \alpha = 1.0 .... for.soft.to.medium.clays.; and$$

$$.\alpha = 0.5 .... for.stiff\ .clays\ .$$

σ' = the net vertical effective load = $Q_v - D_f .\gamma$; *or*

$\sigma' = (Q_v - D_f .\gamma) - u.A'_f$ *(if the water table lies above foundation level)*

δ = *the skin friction angle, which can be taken as equal to (ϕ'),and*

u = the pore water pressure at foundation level.

- **ECCENTRIC LOAD:**

Eccentric load result from loads applied somewhere other than the footing's centroid or from applied moments, such as those resulting at the base of a tall column from wind loads or earthquakes on the structure.

To provide adequate $SF_{(against.lifting)}$ of the footing edge, it is recommended that the eccentricity ($e \le B/6$). Footings with eccentric loads may be analyzed for bearing capacity by two methods: (1) the concept of useful width and (2) application of reduction factors.

(1) Concept of Useful Width:

In this method, only that part of the footing that is symmetrical with regard to the load is used to determine bearing capacity by the usual method, with the remainder of the footing being ignored.

- *First, computes eccentricity and adjusted dimensions:*

$$e_x = \frac{M_y}{V}; \quad L' = L - 2e_x; \quad e_y = \frac{M_x}{V}; \quad B' = B - 2e_y; \quad A'_f = A' = B'.L'$$

- *Second, calculates $q_{ult.}$ from Meyerhof's, or Hansen's, or Vesic's equations (Table 3.2) using B' in the ($\frac{1}{2}B.\gamma.N_\gamma$) term and B' or/and L' in computing the shape factors and not in computing depth factors.*

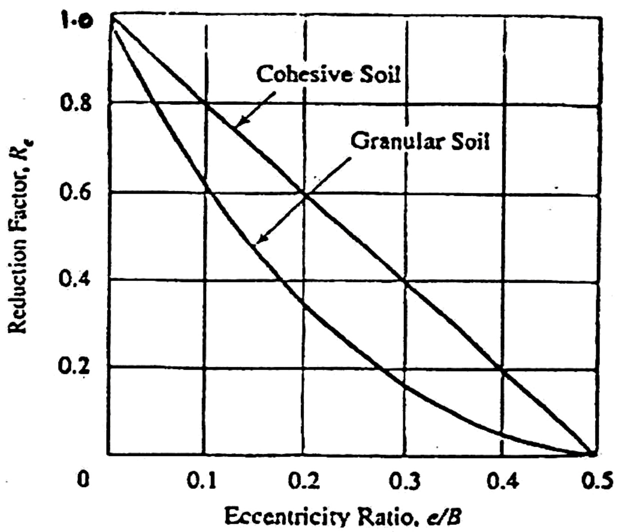
(2) Application of Reduction Factors:

First, computes bearing capacity by the normal procedure (using equations of Table 3.2), assuming that the load is applied at the centroid of the footing. Then, the computed value is corrected for eccentricity by a reduction factor (R_e) obtained from Figure (3.8) or from Meyerhof's reduction equations as:

$$\left.\begin{aligned} R_e &= 1 - 2(e/B) \text{..........for..cohesive..soil} \\ R_e &= 1 - (e/B)^{1/2} \text{........for.cohesionless.soil} \end{aligned}\right\} \textbf{.......................(3.11)}$$

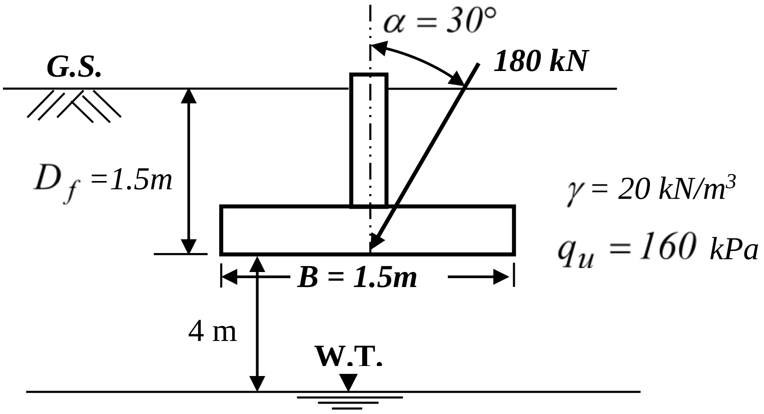
$$\therefore \quad q_{ult.(eccentric)} = q_{ult.(concentric)} .x. R_e$$

.................................(3.12)



Figure(3.8): Eccentric load reduction factors.

Example (4): **A square footing of 1.5x1.5m is subjected to an inclined load as shown in figure below. What is the factor of safety against bearing capacity (use Terzaghi's equation).**



Solution:

By Terzaghi's equation: $q_{ult.} = cN_c.S_c + qN_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma$

Shape factors: from table (3.2) for square footing $S_c = 1.3; S\gamma = 0.8, c = q_u / 2 = 80$ *kPa*

Bearing capacity factors: from table (3.3) for $\phi_u = 0$: $N_c = 5.7,..N_q = 1.0,..N_\gamma = 0$

$q_{ult.(vertical.load)} = 80(5.7)(1.3) + 20(1.5)(1.0) + 0.5(1.5)(20)(0)(0.8) = 622.8\ kN/m^2$

From Fig.(3.7) with $\alpha = 30°$ *and cohesive soil, the reduction factor for inclined load is 0.42.*

$q_{ult.(inclined.load)} = 622.8(0.42) = 261.576\ kN/m^2$

$Q_v = Q.\cos 30 = 180\ (0.866) = 155.88\ kN$

Factor of safety (against bearing capacity failure) $= \frac{Q_{ult.}}{Q_v} = \frac{261.576(1.5)(1.5)}{155.88} = 3.77$

Check for sliding:

$Q_h = Q.\sin 30 = 180\ (0.5) = 90\ kN$

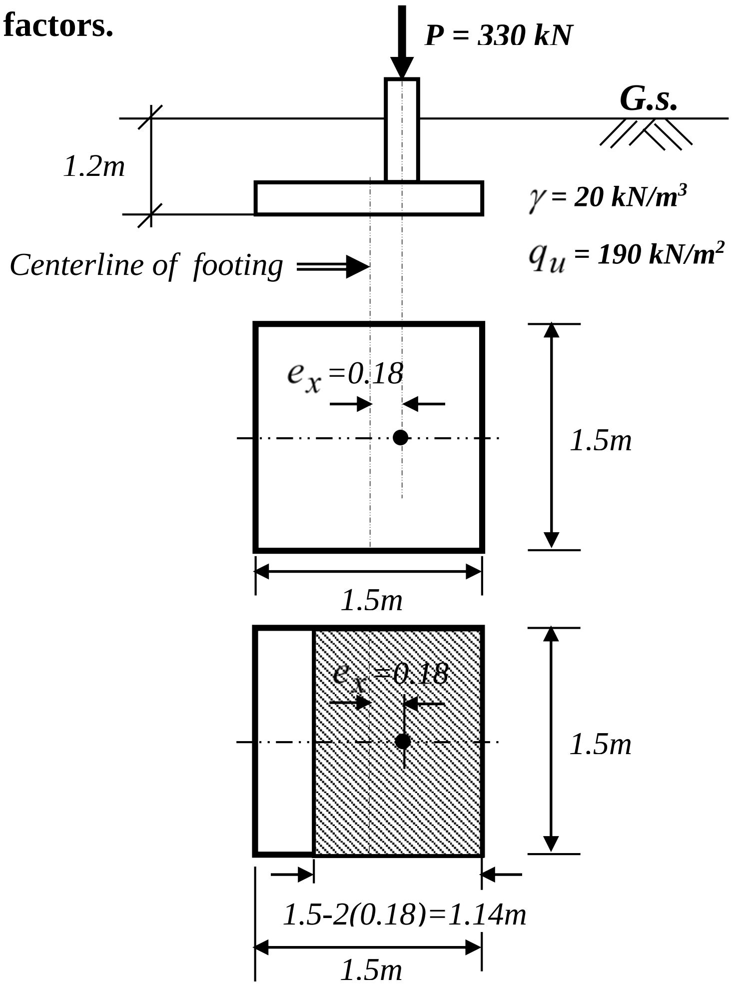
$H_{max.} = A'_f.C_a + \sigma' \tan\delta = (1.5)(1.5)(80) + (180)(\cos 30)(\tan 0) = 180\ kN$

Factor of safety (against sliding) $= \frac{H_{max.}}{Q_h} = \frac{180}{90} = 2.0$ **(O.K.)**

Example (5): **A 1.5x1.5m square footing is subjected to eccentric load as shown below. What is the safety factor against bearing capacity failure (use Terzaghi's equation):**

(a) By the concept of useful width, and

(b) Using Meyerhof's reduction factors.



Solution:

(1) Using concept of useful width:

from Terzaghi's equation:

$$q_{ult.} = cN_c.S_c + qN_q + \frac{1}{2}.B'.\gamma.N\gamma.S_\gamma$$

Shape factors: from table (3.2) for square footing $S_c = 1.3; S\gamma = 0.8, c = q_u / 2 = 95\ kPa$

Bearing capacity factors: from table (3.3) for $\phi_u = 0:\ N_c = 5.7, N_q = 1.0, N_\gamma = 0$

The useful width is: $B' = B - 2e_x = 1.5 - 2(0.18) = 1.14m$

$q_{ult.} = 95(5.7)(1.3) + 20(1.2)(1.0) + 0.5(1.14)(20)(0)(0.8) = 727.95\ kN/m^2$

Factor of safety (against bearing capacity failure) $= \frac{Q_{ult.}}{Q_v} = \frac{727.95(1.14)(1.5)}{330} = 3.77$

(2) Using Meyerhof's reduction factors:

In this case, $q_{ult.}$ *is computed based on the actual width: B = 1.5m*

from Terzaghi's equation:

$$q_{ult.} = 1.3cN_c + qN_q + 0.4B.\gamma.N\gamma$$

$q_{ult.(concentric.load)} = 1.3(95)(5.7) + 20(1.2)(1.0) + 0.4(1.5)(20)(0) = 727.95\ kN/m^2$

For eccentric load from figure (3.8):

with Eccentricity ratio $= \frac{e_x}{B} = \frac{0.18}{1.5} = 0.12$ *; and cohesive soil* $R_e = 0.76$

$\therefore q_{ult.(eccentric.load)} = 727.95\ (0.76) = 553.242\ kN/m^2$

Factor of safety (against bearing capacity failure) $= \frac{Q_{ult.}}{Q_v} = \frac{553.242(1.5)(1.5)}{330} = 3.77$

Example (6): **A square footing of 1.8x1.8m is loaded with axial load of 1780 kN and subjected to M_x = 267 kN-m and M_y = 160.2 kN-m moments. Undrained triaxial tests of unsaturated soil samples give $\phi = 36°$ and $c = 9.4$ kN/m^2. If D_f = 1.8m, the water table is at 6m below the G.S. and $\gamma = 18.1$ kN/m^3, what is the allowable soil pressure if SF=3.0 using (a) Hansen bearing capacity and (b) Meyerhof's reduction factors.**

Solution:

$$e_y = \frac{267}{1780} = 0.15m\ ; \qquad e_x = \frac{160.2}{1780} = 0.09m$$

$$B' = B - 2e_y = 1.8 - 2(0.15) = 1.5m\ ; \quad L' = L - 2e_x = 1.8 - 2(0.09) = 1.62m$$

(a) Using Hansen's equation:

$(with...all...i_i, g_i..and..b_i..factors...are...1.0)$

$$q_{ult.} = cN_c.S_c.d_c + \bar{q}N_q.S_q.d_q + 0.5\gamma.B'.N_\gamma.S_\gamma.d_\gamma$$

Bearing capacity factors from table (3.2):

$$N_c = (N_q - 1).\cot\phi, \qquad N_q = e^{\pi.\tan\phi}..\tan^2(45 + \phi/2), \qquad N_\gamma = 1.5(N_q - 1)\tan\phi$$

for $\phi = 36°$: $\quad N_c = 50.6, \qquad N_q = 37.8, \qquad N_\gamma = 40$

Shape factors from table (3.5):

$$S_c = 1 + \frac{N_q}{N_c}\frac{B'}{L'} = 1 + \frac{37.8}{50.6}\frac{1.5}{1.62} = 1.692, \qquad S_q = 1 + \frac{B'}{L'}\tan\phi = 1 + \frac{1.5}{1.62}\tan 36 = 1.673$$

$$S_\gamma = 1 - 0.4\frac{B'}{L'} = 1 - 0.4\frac{1.5}{1.62} = 0.629$$

<u>Depth factors from table (3.5):</u>

for D =1.8m, and B = 1.8m, D/B = 1.0 (shallow footing)

$$d_c = 1 + 0.4\frac{D}{B} = 1 + 0.4(1.0) = 1.4,$$

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\frac{D}{B} = 1 + 2\tan 36(1 - \sin 36)^2(1.0) = 1.246, \quad d_\gamma = 1.0$$

$q_{ult.}$ = 9.4(50.6)(1.692)(1.4) + 1.8(18.1)(37.7)(1.673)(1.246)

+ 0.5(18.1)(1.5)(40)(0.629)(1)= 4028.635 kN/m^2

$q_{all.} = 4028.635 / 3 = 1342.878\ kN/m^2$

Actual soil pressure ($q_{act.}$) = 1780/(1.5)(1.62)= 732.510 < 1342.878 **(O.K.)**

<u>(b) Using Meyerhof's reduction:</u>

$$R_{ex} = 1 - \left(\frac{e_x}{L}\right)^{1/2} = 1 - \left(\frac{0.09}{1.8}\right)^{0.5} = 0.78; \qquad R_{ey} = 1 - \left(\frac{e_y}{B}\right)^{1/2} = 1 - \left(\frac{0.15}{1.8}\right)^{0.5} = 0.72$$

Recompute $q_{ult.}$ *as for a centrally loaded footing, since the depth factors are unchanged.*

<u>The revised Shape factors from table (3.5) are:</u>

$$S_c = 1 + \frac{N_q}{N_c}\frac{B}{L} = 1 + \frac{37.8}{50.6}\frac{1.8}{1.8} = 1.75; \qquad S_q = 1 + \frac{B}{L}\tan\phi = 1 + \frac{1.8}{1.8}\tan 36 = 1.73$$

$$S_\gamma = 1 - 0.4\frac{B}{L} = 1 - 0.4\frac{1.8}{1.8} = 0.60$$

$$q_{ult.} = cN_c.S_c.d_c + \bar{q}N_q.S_q.d_q + 0.5\gamma.B.N_\gamma.S_\gamma.d_\gamma$$

$q_{ult.}$ = 9.4(50.6)(1.75)(1.4) + 1.8(18.1)(37.7)(1.73)(1.246)

+ 0.5(18.1)(1.8)(40)(0.60)(1)= 4212.403 kN/m^2

$q_{all.centrally.loaded.footing} = 4212.403 / 3 = 1404.134\ kN/m^2$

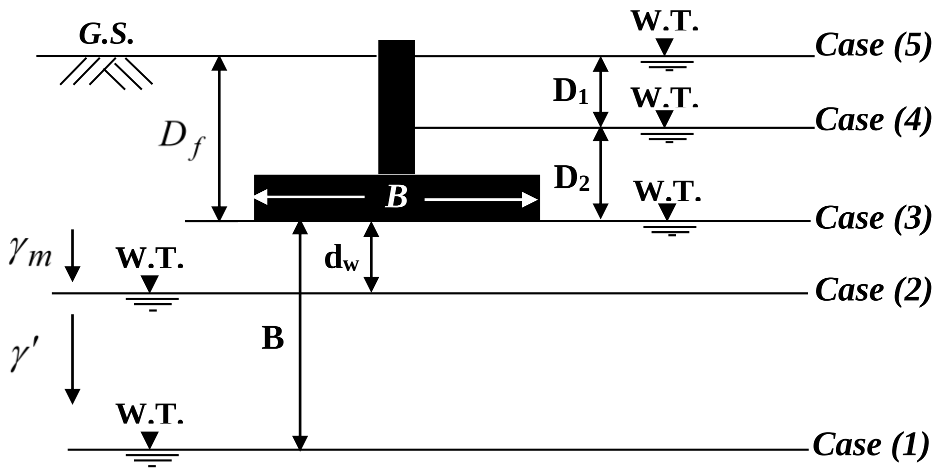
$$q_{all.eccentric.loaded..footing} = q_{all.centrally.loaded.footing}(R_{ex})(R_{ey})$$

$$=1404.134(0.78)(0.72) = 788.35 \text{ kN/m}^2 \quad \textit{(very high)}$$

Actual soil pressure ($q_{act.}$) $= 1780/(1.8)(1.8) = 549.383 < 788.35$ **(O.K.)**

3.10 EFFECT OF WATER TABLE ON BEARING CAPACITY

Generally the submergence of soils will cause loss of all apparent cohesion, coming from capillary stresses or from weak cementation bonds. At the same time, the effective unit weight of submerged soils will be reduced to about one-half the weight of the same soils above the water table. Thus, through submergence, all the three terms of the bearing capacity (B.C.) equations may be considerably reduced. Therefore, it is essential that the B.C. analysis be made assuming the highest possible groundwater level at the particular location for the expected life time of the structure.



Case (1):

If the water table (W.T.) lies at B or more below the foundation base; no W.T. effect.

Case (2):

- *<u>(from Ref.;Foundation Engg. Hanbook):</u> if the water table (W.T.) lies within the depth* ($d_w<B$) *; (i.e., between the base and the depth B), use* $\gamma_{av.}$ *in the term* $\frac{1}{2}\gamma.B.N_\gamma$ *as:*

$$\gamma_{av.} = \gamma' + (d_w / B)(\gamma_m - \gamma') \ldots\ldots\ldots\ldots\ldots\ldots \textit{(from Meyerhof)}$$

- *(from Ref.;Foundation Analysis and Design):* *if the water table (W.T.) lies within the wedge zone* $\{H = 0.5B.tan(45+\phi/2)\}$*; use* $\gamma_{av.}$ *in the term* $\frac{1}{2}\gamma.B.N_\gamma$ *as:*

$$\gamma_{av.} = (2H - d_w)\frac{d_w}{H^2}.\gamma_{wet} + \frac{\gamma'}{H^2}(H-d_w)^2 \text{ \textbf{.........}(from ,Bowles)}$$

where:

$H = 0.5B.tan(45+\phi/2)$.
γ' = *submerged unit weight* =$(\gamma_{sat.} - \gamma_w)$,
d_w = *depth to W.T. below the base of footing,*
$\gamma_m = \gamma_{wet}$ = *moist or wet unit weight of soil in depth* (d_w) *, and*

- *Snice in many cases of practical purposes, the term* $\frac{1}{2}\gamma.B.N_\gamma$ *can be ignored for conservative results, it is recommended for this case to use* $\gamma = \gamma'$ *in the term* $\frac{1}{2}\gamma.B.N_\gamma$ *instead of* $\gamma_{av.}$

$$(\gamma' < \gamma_{av.}(from..Meyerhof) < \gamma_{av.}(from..Bowles))$$

Case (3): *if* $d_w = 0$ *; the water table (W.T.) lies at the base of the foundation; use* $\gamma = \gamma'$

Case (4): *if the water table (W.T.) lies above the base of the foundation; use:*

$$q = \gamma_t.D_{1(above..W.T.)} + \gamma'.D_{2(below..W.T.)} \text{ and } \gamma = \gamma' \text{ in } \frac{1}{2}\gamma.B.N_\gamma \text{ term.}$$

Case (5): *if the water table (W.T.) lies at ground surface (G.S.); use:* $q = \gamma'.D_f$ *and*

$$\gamma = \gamma' \text{ in } \frac{1}{2}\gamma.B.N_\gamma \text{ term.}$$

Note: *All the preceding considerations are based on the assumption that the seepage forces acting on soil skeleton are negligible. The seepage force adds a component to the body forces caused by gravity. This component acting in the direction of stream lines is equal to* $(i.\gamma_w)$*, where* i *is the hydraulic gradient causing seepage.*

3.11 Bearing Capacity for Footings on Layered Soils

Stratified soil deposits are of common occurrence. It was found that when a footing is placed on stratified soils and the thickness of the top stratum form the base of the footing (d_1 or H) is less than the depth of penetration [$H_{crit.} = 0.5B\tan(45+\phi/2)$]; in this case the rupture zone will extend into the lower layer (s) depending on their thickness and therefore require some modification of ultimate bearing capacity ($q_{ult.}$).

Figure (3.18) shows a foundation of any shape resting on an upper layer having strength parameters c_1, ϕ_1 and underlain by a lower layer with c_2, ϕ_2.

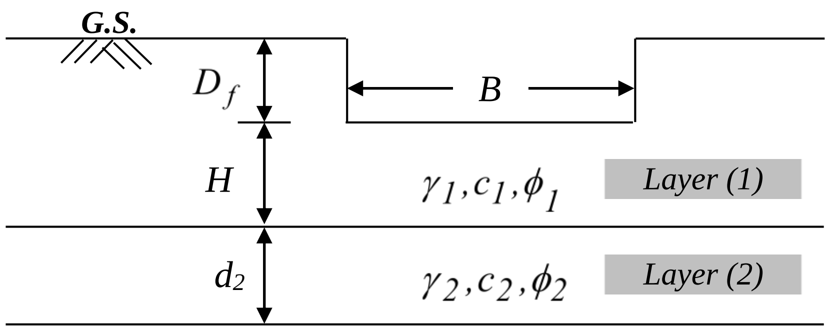


Figure (3.11): Footing on layered $c-\phi$ soils.

- **<u>Hansen Equation</u>** <u>(Ref., Bowles's Book, 1996)</u>

(1) *Compute $H_{crit.} = 0.5B\tan(45+\phi_1/2)$ using ϕ_1 for the top layer.*

(2) *If $H_{crit.} > H$ compute the modified values of c and ϕ as:*

$$c^* = \frac{Hc_1 + (H_{crit.} - H)c_2}{H_{crit.}} \;; \qquad \phi^* = \frac{H\phi_1 + (H_{crit.} - H)\phi_2}{H_{crit.}}$$

<u>Note:</u> *A possible alternative for $c-\phi$ soils with a number of thin layers is to use average values of c and ϕ in bearing capacity equations of Table (3.2) as:*

$$c_{av.} = \frac{c_1H_1 + c_2H_2 + \ldots\ldots + c_nH_n}{\sum H_i} \;; \qquad \phi_{av.} = \tan^{-1}\frac{H_1\tan\phi_1 + H_2\tan\phi_2 + \ldots\ldots + H_n\tan\phi_n}{\sum H_i}$$

(3) *Use Hansen's equation from Table (3.2) for $q_{ult.}$ with c^* and ϕ^* as:*

BEARING CAPACITY EXAMPLES (3)

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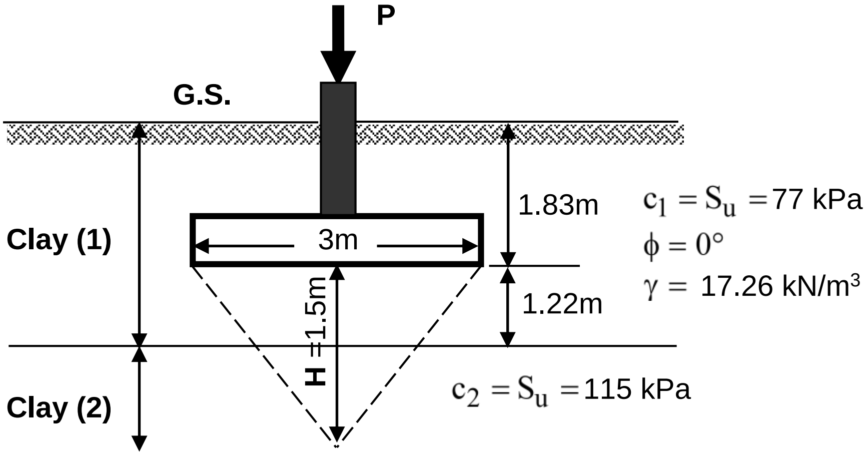
Civil Engineering Department – College of Engineering
Tikrit University

Example (8): *(footing on layered clay)*

A rectangular footing of 3.0x6.0m is to be placed on a two-layer clay deposit as shown in figure below. Estimate the ultimate bearing capacity.



Solution:

$$H_{crit.} = 0.5B\tan(45+\phi/2) = 0.5(3)\tan45 = 1.5m > 1.22m$$

$\therefore$ *the critical depth penetrated into the 2^nd.^ layer of soil.*

For case(1); clay on clay layers using Hansen's equation:

- ***F**rom Bowles's Book, 1996:*

$$q_{ult.} = 5.14 C_{avg.}(1 + S'_c + d'_c) + q'$$

where:

$$S_u = C_{avg.} = \frac{C_1 H + C_2\,[Hcrit - H]}{Hcrit} = \frac{77(1.22) + 115(1.5 - 1.22)}{1.5} = 84.093$$

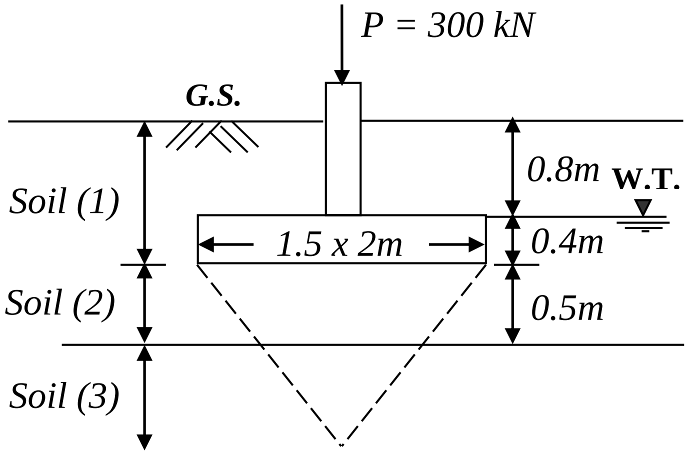
$$S'_c = 0.2B/L = 0.2(3/6) = 0.1; \quad for\ Df/B \le 1: \quad d'_c = 0.4D/B = 0.4(1.83/3) = 0.24$$

$$\therefore \quad q_{ult.} = 5.14(84.093)(1+0.1+0.24) + 1.83(17.26) = \boxed{610.784\ kPa}$$

Example (9): *(footing on $c-\phi$ soils)*

Check the adequacy of the rectangular footing 1.5x2.0m shown in figure below against shear failure (use F.S.= 3.0), γ_w =10 kN/m^3 .

parameter	Soil (1)	Soil (2)	Soil (3)
Gs	2.70	2.65	2.75
e	0.8	0.9	0.85
c (kPa)	10	60	80
$\phi°$	35	0	0



Solution:

$$\gamma_{d1} = \frac{G_s.\gamma_w}{1+e} = \frac{2.70(10)}{1+0.8} = 15 \ kN/m^3$$

$$\gamma_{sat1} = \frac{(G_s+e)\gamma_w}{1+e} = \frac{(2.70+0.8)10}{1+0.8} = 19.4 \ kN/m^3$$

$$\gamma_{d2} = \frac{G_s.\gamma_w}{1+e} = \frac{2.65(10)}{1+0.9} = 18.7 \ kN/m^3$$

$$\gamma_{sat2} = \frac{(2.75+0.85)10}{1+0.85} = 19.45 \ kN/m^3$$

$$H_{crit.} = 0.5B\tan(45+\phi/2) = 0.5(1.5)\tan 45 = 0.75m > 0.50m$$

$\therefore$ *the critical depth penetrated into the soil layer (3).*

Since soils (2) and (3) are of clay layers, therefore; by using Hansen's equation:

- *From Bowles's Book, 1996:*

$$q_{ult.} = 5.14C_{avg.}(1+S'_c+d'_c)+q'$$

where:

$$C_{avg.} = \frac{C_1 H + C_2\ [Hcrit - H]}{Hcrit} = \frac{60(0.5) + 80(0.75 - 0.50)}{0.75} = 66.67$$

$S'_c = 0.2B/L = 0.2(1.5/2) = 0.15$;

$for\ Df/B \le 1\ \ d'_c = 0.4D/B = 0.4(1.2/1.5) = 0.32$

$\therefore\quad q_{ult.} = 5.14(66.67)(1+0.15+0.32)+0.8(15)+0.4(19.45-10) = 519.5\ kPa$

$$q_{all}(net) = \frac{519.5}{3} - 15.78 = \boxed{157.4\ kPa}$$

$$q_{applied} = \frac{300}{1.5x2} = 100\ kPa < q_{all}(net) = 157.4\ kPa\quad \therefore\ \textbf{(O.K.)}$$

<u>Check for squeezing:</u>

For no squeezing of soil beneath the footing: $(q_{ult.} > 4c_1 + \bar{q})$

$4c_1 + \bar{q} = 4(60) + 0.8(15) + 0.4(19.45-10) = 255.78\ kPa < 519.5\ kPa\ \ \therefore$ **(O.K.)**

3.12 Skempton's Bearing Capacity Equation

- **<u>Footings on Clay and Plastic Silts:</u>**

From Terzaghi's equation, the ultimate bearing capacity is:

$$q_{ult.} = cN_c.S_c + qN_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma \quad \textbf{..................................(3.12)}$$

<u>For saturated clay and plastic silts:</u> ($\phi_u = 0$ and $N_c = 5.7, N_q = 1.0, and. N_\gamma = 0$),

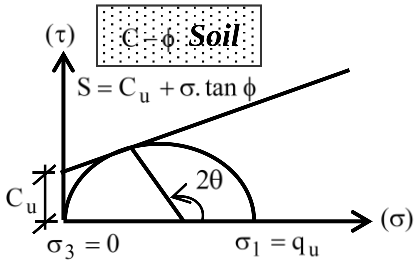
<u>For strip footing:</u> $S_c = S_\gamma = 1.0$

$$\boxed{q_{ult.} = cN_c + \bar{q}} \quad \textbf{..(3.30)}$$

$$q_{all.} = \frac{q_{ult.}}{3} \quad and \quad q_{all.(net)} = q_{all.} - \bar{q}$$

$$\therefore\quad q_{all.(net)} = \frac{q_{ult.}}{3} - \bar{q} = \frac{cN_c + \bar{q}}{3} - \bar{q} = \frac{cN_c}{3} + (\frac{\bar{q}}{3} - \bar{q}) \quad \textbf{.....................(3.30a)}$$

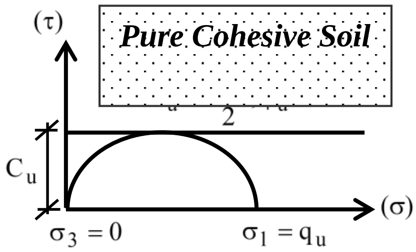
where: N_c *= bearing capacity factor obtained from* ***figure (3.12)*** *depending on shape of footing and* $\frac{D_f}{B} \cdot (\frac{\bar{q}}{3} - \bar{q})$ *is a small value can be neglected.*



for c − ϕ ***soil***: $\boxed{\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c\tan(45 + \phi/2)}$

for UCT: $\sigma_1 = q_u$ and $\sigma_3 = 0$; then $\boxed{q_u = 2c\tan(45 + \phi/2)}$

or $\phi_u = 0$; $c = \frac{q_u}{2}$ *and equation (3.30a) will be:*



$$q_{all.(net)} = q_u \frac{N_c}{6} \quad \text{.......................................(3.30b)}$$

From ***figure (3.12)*** *for* $\frac{D_f}{B}$ *=0:* $N_c = 6.2$ *for square or circular footings; 5.14 for strip or continuous footings* *If* $N_c = 6.0$*, then:*

$$\boxed{q_{all.(net)} \approx q_u} \quad \text{...(3.31)}$$

See figure (3.13) *for net allowable soil pressure for footings on clay and plastic silt.*

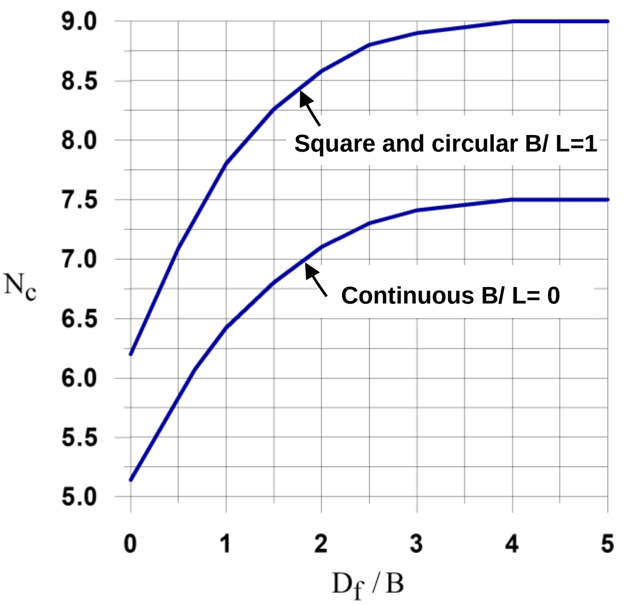


Figure (3.12): N_c **bearing capacity factor for Footings on clay under** $\phi = 0$ **conditions** (After Skempton, 1951).

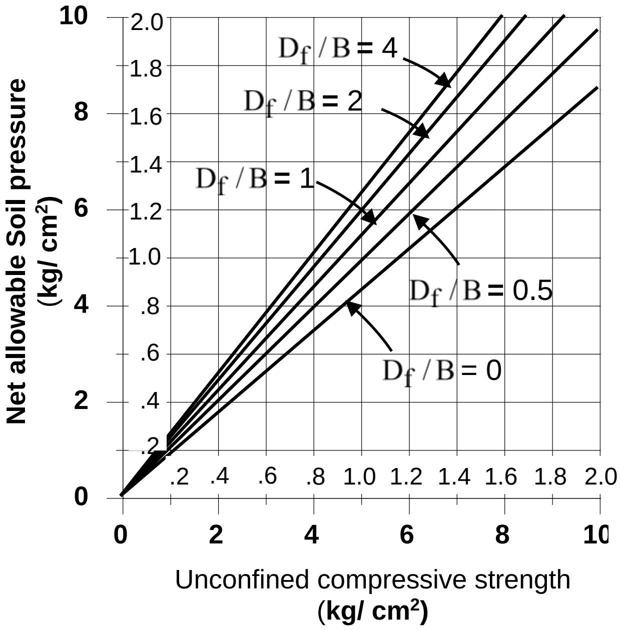
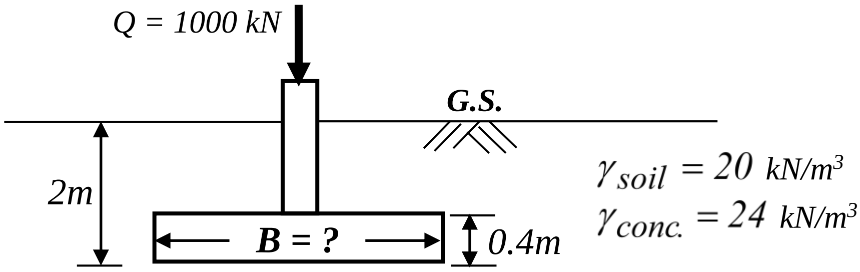


Figure (3.13): Net allowable soil pressure for footings on clay and plastic silt, determined for a factor of safety of 3 against bearing capacity failure ($\phi = 0$ conditions). Chart values are for strip footings (B/L=0); and for other types of footings multiply values by (1+ 0.2B/L).

$$\boxed{N_{c(net)} = N_{c(strip)}(1 + 0.2\frac{B}{L}) \quad \underline{\textbf{or}} \quad N_{c(net)} = N_{c(square)}(0.84 + 0.16\frac{B}{L})}$$

Example (10): *(footing on clay)*

Determine the size of the square footing shown in figure below. If q_u = 100 kPa and F.S.= 3.0?



Solution:

Assume B =3.5m, D/B *= 2/3.5 = 0.57 then from* **figure (3.12)**: $N_c = 7.3$

$q_{ult.} = cN_c + \bar{q}$ = 50(7.3) + 2(20) = 405 kPa

$$q_{all.(net)} = \frac{q_{ult.}}{3} - \bar{q} = \frac{405}{3} - 20(1.6) - 24(0.4) = 93.4 \text{ kPa}$$

Area=1000/93.4 = 10.71 m²; for square footing: $B = \sqrt{10.71} = 3.27 < 3.5m$

$\therefore$ *take B =3.25m , and* D/B *= 2/3.25 = 0.61 then from figure (3.15):* $N_c = 7.5$

$q_{ult.} = cN_c + \bar{q}$ = 50(7.5) + 2(20) = 415 kPa

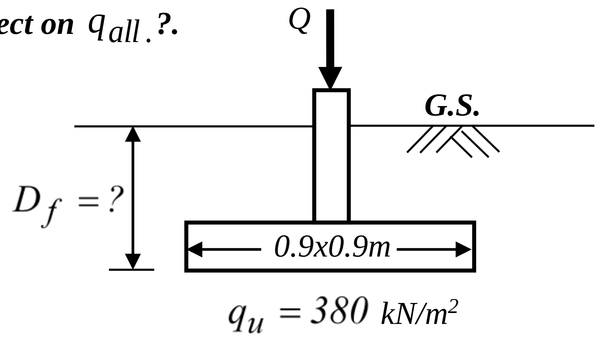
$$q_{all.(net)} = \frac{q_{ult.}}{3} - \bar{q} = \frac{415}{3} - 20(1.6) - 24(0.4) = 96.73 \text{ kPa}$$

Area=1000/96.73 = 10.34 m²; $B = \sqrt{10.34} = 3.21 \approx 3.25m$ **(O.K.)**

$\therefore$ <u>*use B x B = (3.25 x 3.25)m*</u>

Example (11): *(footing on clay)*

For the square footing shown in figure below. If q_u = 380 kPa and F.S.= 3.0, determine $q_{all.}$ and $D_f(min.)$ which gives the maximum effect on $q_{all.}$?.



Solution:

From Skempton's equation:

For strip footing: $q_{all.(net)} = \dfrac{cN_c}{3}$

For square footing: $q_{all.(net)} = \dfrac{cN_c}{3} x1.2$

*From Skempton's **figure (3.12)** at* $D_f / B = 4$ *and* $B/L=1$ *(square footing):* $N_c = 9$

$$\therefore\ q_{all.(net)} = \frac{\frac{380}{2}(9)}{3} = \boxed{570\ kPa} \quad and \quad D_f = 4(0.9) = \boxed{3.6m}$$

- ## Rafts on Clay:

If $q_b = \dfrac{\Sigma Q}{A} = \dfrac{Total.load(D.L.+L.L.)}{area} > q_{all}$. *use pile or floating foundations.*

From Skempton's equation, the ultimate bearing capacity (for strip footing) is:

$$q_{ult.} = cN_c + \bar{q} \quad \textbf{……………………………………………………………(3.30)}$$

$$q_{ult.(net)} = cN_c\ , \qquad q_{all.(net)} = \frac{cN_c}{F.S.} \qquad or \qquad F.S. = \frac{cN_c}{q_{all.(net)}}$$

Net soil pressure = $q_b - D_f.\gamma$

$$\therefore \qquad \boxed{F.S. = \frac{cN_c}{q_b - Df.\gamma}} \quad \textbf{……………………………………………………………(3.32)}$$

Notes:

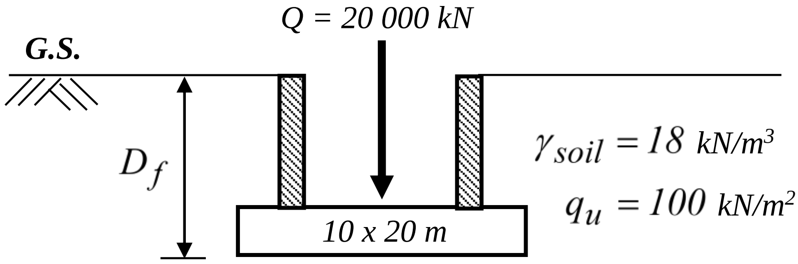
(1) *If* $q_b = D_f.\gamma$ *(i.e.,* $F.S. = \infty$*) the raft is said to be fully compensated foundation (in this case, the weight of foundation (D.L.+ L.L.) = the weight of excavated soil).*

(2) *If* $q_b > D_f.\gamma$ *(i.e.,* $F.S. = certain value$*) the raft is said to be partially compensated foundation such as the case of storage tanks.*

<u>Example (12):</u> *(raft on clay)*

Determine the F.S. for the raft shown in figure for the following depths: $D_f = 1m, 2m,$ ***and 3m?.***

<u>Solution:</u>



$$F.S. = \frac{cN_c}{q_b - Df.\gamma}$$

- <u>For</u> $D_f = 1m$:

From figure (3.12) $D_f / B = 1/10 = 0.1$ *and* $B/L = 0$:

$$N_{c\,strip} = 5.4 \text{ and } \boxed{N_{c\,rectangular} = N_{c\,strip}(1 + 0.2B/L)} = 5.4\,(1 + 0.2\frac{10}{20}) = 5.94$$

$$\therefore \quad F.S. = \frac{cN_c}{q_b - Df.\gamma} = \frac{(100/2)5.94}{\frac{20000}{10x20} - 1(18)} = \frac{50(5.94)}{100 - 18} = 3.62$$

- <u>For</u> $D_f = 2m$:

From figure (3.12) $D_f / B = 2/10 = 0.2$ *and* $B/L = 0$:

$$N_{c\,strip} = 5.5 \quad \text{and} \quad N_{c\,rectangular} = 5.5\,(1 + 0.2\frac{10}{20}) = 6.05$$

$$\therefore \quad F.S. = \frac{cN_c}{q_b - Df.\gamma} = \frac{(100/2)6.05}{\frac{20000}{10x20} - 2(18)} = \frac{50(6.05)}{100 - 36} = 4.72$$

- <u>For</u> $D_f = 3m$:

From figure (3.12) $D_f / B = 3/10 = 0.3$ *and* $B/L = 0$:

$$N_{c\,strip} = 5.7 \quad \text{and} \quad N_{c\,rectangular} = 5.7\,(1 + 0.2\frac{10}{20}) = 6.27$$

$$\therefore \quad F.S. = \frac{cN_c}{q_b - Df.\gamma} = \frac{(100/2)6.27}{\frac{20000}{10x20} - 3(18)} = \frac{50(6.27)}{100 - 54} = 6.81$$

3.13 Design Charts for Footings on Sand and Nonplastic Silt

From Terzaghi's equation, the ultimate bearing capacity is:

$$q_{ult.} = cN_c.S_c + \bar{q}N_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma$$

……..……………..……...…………..(3.12)

<u>*For sand*</u> $(c = 0)$ *and* <u>*for strip footing*</u> $(S_c = S_\gamma = 1.0)$, then, Eq.(3.12) will be:

$$q_{ult.} = \bar{q}N_q + \frac{1}{2}B.\gamma.N_\gamma \quad \text{....…………...……………………….……….….……..(3.33)}$$

$$q_{ult.(net)} = \bar{q}N_q + \frac{1}{2}B.\gamma.N_\gamma - \bar{q}$$

$$q_{ult.(net)} = D_f.\gamma.N_q + \frac{1}{2}B.\gamma.N_\gamma - D_f.\gamma$$

$$q_{ult.(net)} = D_f.\gamma(N_q - 1) + \frac{1}{2}B.\gamma.N_\gamma = B\left[\frac{D_f.\gamma}{B}(N_q - 1) + \frac{1}{2}\gamma.N_\gamma\right]$$

$$\boxed{q_{all.(net)} = \frac{B}{F.S.}\left[\frac{D_f.\gamma}{B}(N_q - 1) + \frac{1}{2}\gamma.N_\gamma\right]} \quad \text{.....………..………..………..(3.34)}$$

<u>***Notes:***</u>

*(1) the allowable bearing capacity shown by (**Eq.3.34**) is derived from the frictional resistance due to: (i) the weight of the sand below the footing level; and (ii) the weight of the surrounding surcharge or backfill.*

(2) <u>*the design charts for proportioning shallow footings on sand and nonplastic silts are shown in*</u>

<u>***Figures (3.15, 3.16 and 3.17).***</u>

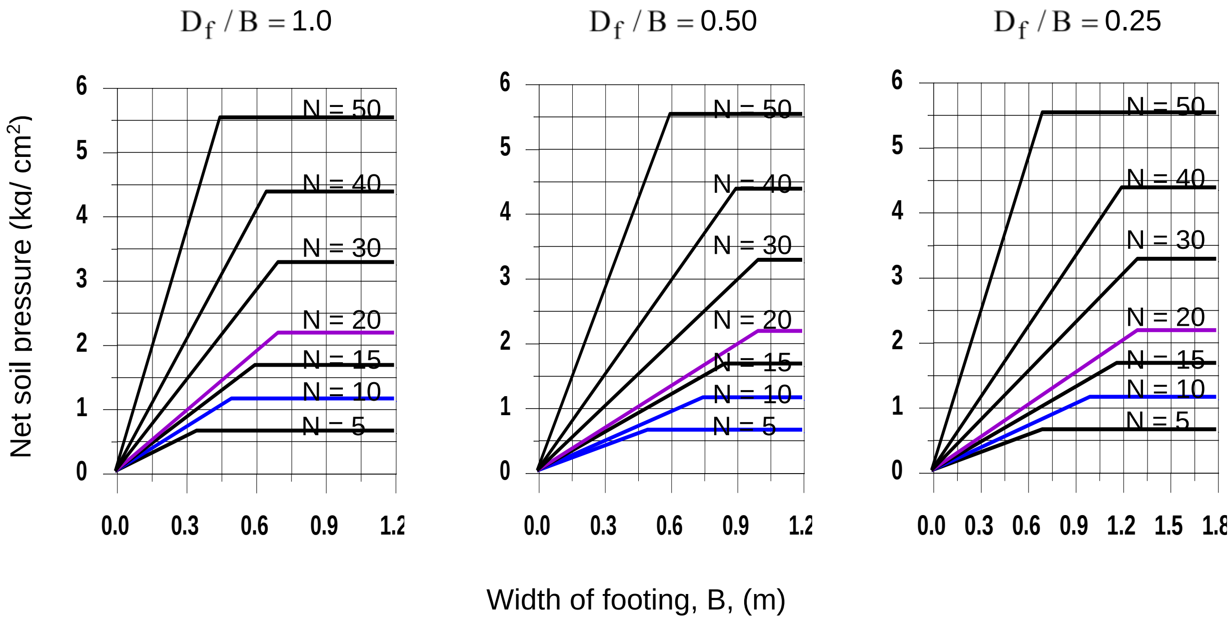


Fig.(3.15): Design charts for proportioning shallow footings on sand.

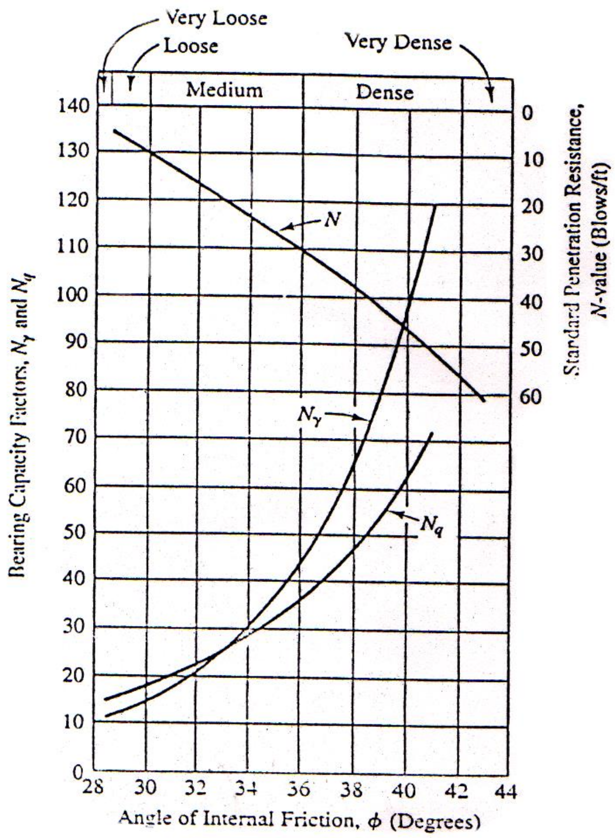


Fig.(3.16): Relationship between bearing capacity factors and ϕ.

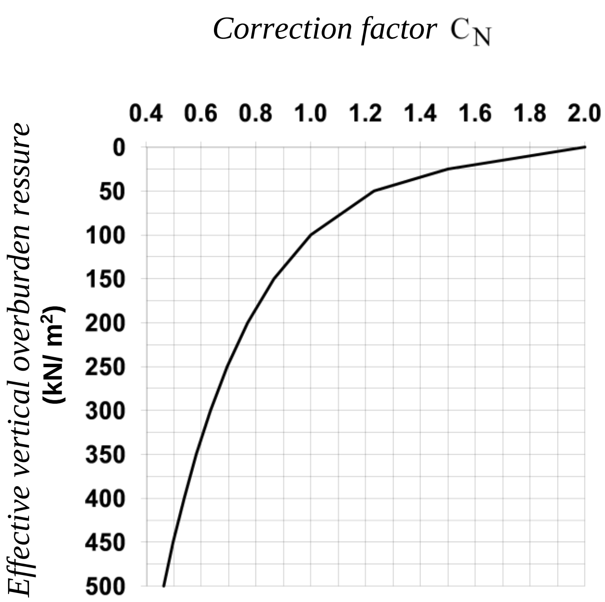


Fig.(3.17): Chart for correction of N-values in sand for overburden pressure.

Limitations of using charts (3.15, 3.16 and 3.17):

- *These charts are for strip footing, while for other types of footings multiply $q_{all.}$ by (1+ 0.2 B/L).*
- *The charts are derived for shallow footings ($D_f / B \le 1$); $\gamma = 100$ Ib/ft^3; settlement = 1.0 (inch); F.S. = 2.0; no water table (far below the footing); and corrected N-values.*
- *N-values must be corrected for:*

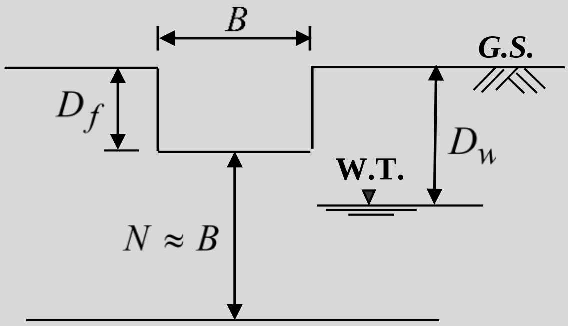
  (i) overburden pressure effect using figure (3.17) or the following formulas:

$$C_N = 0.77 \log \frac{20}{\overline{P}_o(Tsf)} \quad \text{or} \quad C_N = 0.77 \log \frac{2000}{\overline{P}_o(kPa)}$$

If $\bar{p}_o < 0.25(Tsf)$ or $< 25(kPa)$, (no need for overburden pressure correction).

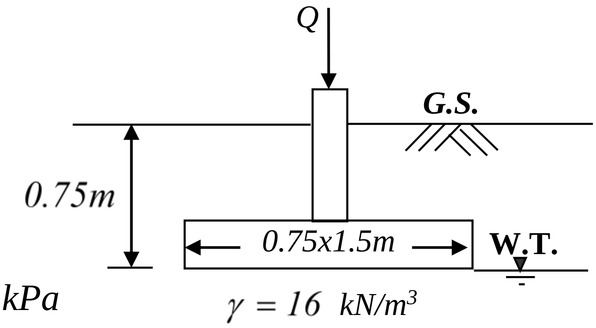
(ii) and water table effect:

$$C_w = 0.5 + 0.5 \frac{D_w}{B + D_f}$$



Example (13): *(footing on sand)*

Determine the gross bearing capacity and the expected settlement of the rectangular footing shown in figure below. If $N_{avg.}$(not corrected) =22 and the depth for correction = 6m?.



Solution:

$P'_o = 0.75(16) + 5.25(16-9.81) = 44.5$ kPa >25 kPa

$$C_N = 0.77 \log \frac{2000}{\overline{P}_o(kPa)} = 0.77 \log \frac{2000}{44.5} = 1.266$$

$$C_w = 0.5 + 0.5 \frac{D_w}{B + D_f} = 0.5 + 0.5 \frac{0.75}{0.75 + 0.75} = 0.75$$

$N_{corr.} = 22(1.266)(0.75) = 20.8$ (use N = 20)

*From **figure (3.15)** for footings on sand: at $D_f\ /B = 1$ and B = 0.75m (2.5ft) and N 20*

<u>for strip footing:</u> $q_{all.(net)} = 2.2(Tsf\ )x105.594 = 232.307\ kPa$

<u>for rectangular footing:</u> $q_{all.(net)} = 232.307\,x\ (1{+}0.2B/L) = 255.538\ kPa$

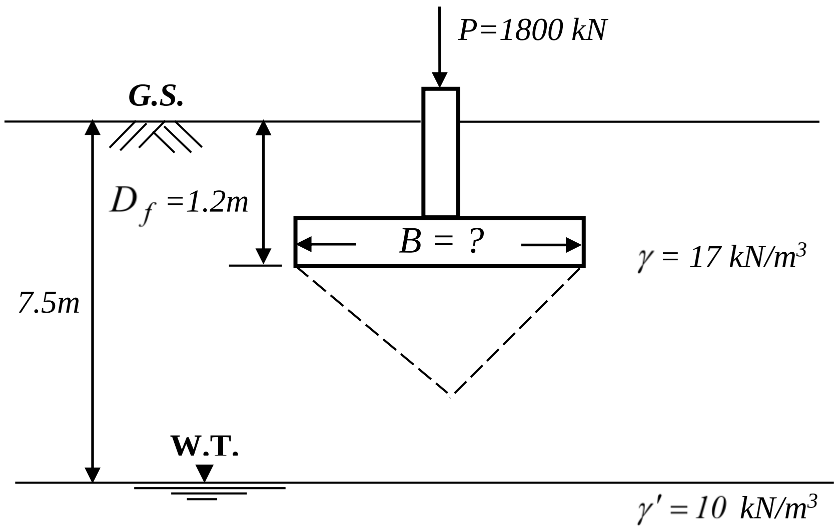
$q_{gross} = q_{all.(net)} + D_f\,.\gamma = 255.538 + 0.75(16) = \boxed{267.538\ kPa}$

And the maximum settlement is not more than (1 inch or 25mm).

<u>Example (14):</u> *(bearing capacity from field tests)*

SPT results from a soil boring located adjacent to a planned foundation for a proposed warehouse are shown below. If spread footings for the project are to be found (1.2m) below surface grade, what foundation size should be provided to support (1800 kN) column load? Assume that 25mm settlement is tolerable, W.T. encountered at (7.5m).

SPT sample depth (m)	N_{field}
0.3	*9*
1.2	*10*
2.4	*15*
3.6	*22*
4.8	*19*
6	*29*
7.5	*33*
10	*27*



<u>Solution:</u>

Find σ'_o *at each depth and correct* N_{field} *values.* *Assume B = 2.4 m*

At depth B below the base of footing (1.2+2.4) = 3.6m; $N'_{avg.} = (15+19+25)/3 = 20$

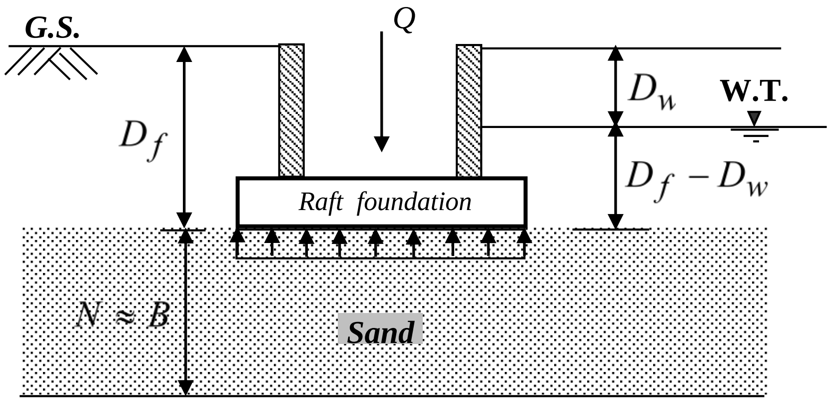
For $N'_{avg.} = 20$, *and* $D_f\ /B = 0.5$; $q_{all.} = 2.2\ T/ft^2 = 232.31\ kPa$ *from Fig.**(3.15).***

SPT sample depth (m)	N_{field}	σ'_o (kN/m²)	σ'_o (T/ft²)	C_N **(Fig.3.17)**	$N' = C_N . N_{field}$
0.3	9				
1.2	10	20.4	0.21	1.55	15
2.4	15	40.8	0.43	1.28	19
3.6	22	61.2	0.64	1.15	25
4.8	19	81.6	0.85	1.05	20
6	29	102	1.07	0.95	27
7.5	33	127.5	1.33	0.90	30
10	27	152.5	1.59	0.85	23

Say B = 2.5 m, $q_{all.} = \frac{P}{B.x.L}$, $L = \frac{1800}{232.31 X 2.5} = 3.10m$, $\therefore$ *use* ***(2.5 x 3.25)m*** *footing.*

- ## Rafts on Sand:

For allowable settlement = 2 (inch) and differential settlement >3/4 (inch) provided that $D_f \geq (8\,ft).or.(2.4m)\min$. *the allowable net soil pressure is given by:*



$$\boxed{q_{all.(net)} = C_w \frac{S_{all.}(N)}{9}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ for\ 5 \leq N \leq 50 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{(3.35)}$$

If C_w *=1 and* $S_{all.} = 2''$; *then* $\boxed{q_{all.(net)}} = 1.0\frac{2.0(N)}{9} = \boxed{0.22N(Tsf) = 23.23N(kPa)}$

and $q_{gross} = q_{all.(net)} + D_f.\gamma = \frac{\Sigma Q}{Area}$

where: $D_f.\gamma = D_w\gamma + (D_f - D_w)(\gamma - \gamma_w) + (D_f - D_w)\gamma_w$

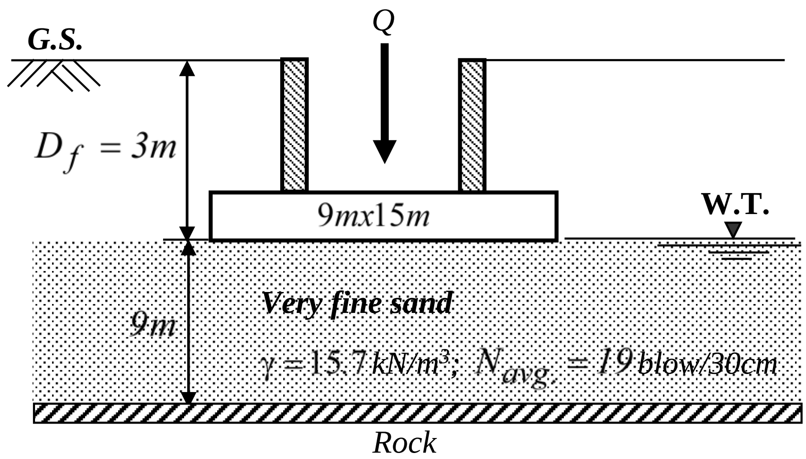
$$C_w = 0.5 + 0.5\frac{D_w}{B + D_f} = \text{(correction for water table)}$$

N = SPT number (corrected for both W.T. and overburden pressure).

Hint: *A raft-supported building with a basement extending below water table is acted on <u>by hydroustatic uplift pressure or buoyancy</u> equal to $(D_f - D_w)\gamma_w$ per unit area.*

Example (15): *(raft on sand)*

Determine the maximum soil pressure that should be allowed at the base of the raft shown in figure below If $N_{avg.}$(corrected) =19?.



Solution:

For raft on sand: $\boxed{q_{all.(net)} = 23.23N(kPa)}$ = 23.23(19) = 441.37 kPa

Correction for water table: $C_w = 0.5 + 0.5\dfrac{D_w}{B + D_f} = 0.5 + 0.5\dfrac{3}{9+3} = 0.625$

$\therefore\ q_{all.(net)} = 441.37(0.625) = 275.856\ \ kPa$

The surcharge = $D_f.\gamma$ = 3(15.7) = 47.1 kPa

and $q_{gross} = q_{all.(net)} + D_f.\gamma$ = 275.856 + 47.1 = 323 kPa

3.14 Bearing Capacity of Footings on Slopes

If footings are on slopes, their bearing capacities are less than if the footings were on level ground. In fact, bearing capacity of a footing is inversely proportional to ground slope.

- **Meyerhof's Method:**

In this method, the ultimate bearing capacity of footings on slopes is computed using the following equations:

$$(q_{ult.})_{continuous.footing.on.slope} = cN_{cq} + \frac{1}{2}\gamma.B.N_{\gamma q} \quad \text{.........(3.36)}$$

$$(q_{ult.})_{c.or.s.footing.on.slope} = (q_{ult.})_{continuous footing.on.slope}\left[\frac{(q_{ult.})_{c.or.s.footing.on.level.ground}}{(q_{ult.})_{continuous footing.on.level.ground}}\right] \quad \text{.....(3.37)}$$

where:

*N_{cq} and $N_{\gamma q}$ are bearing capacity factors for footings on or adjacent to a slope; determined from **figure (3.18)**,*

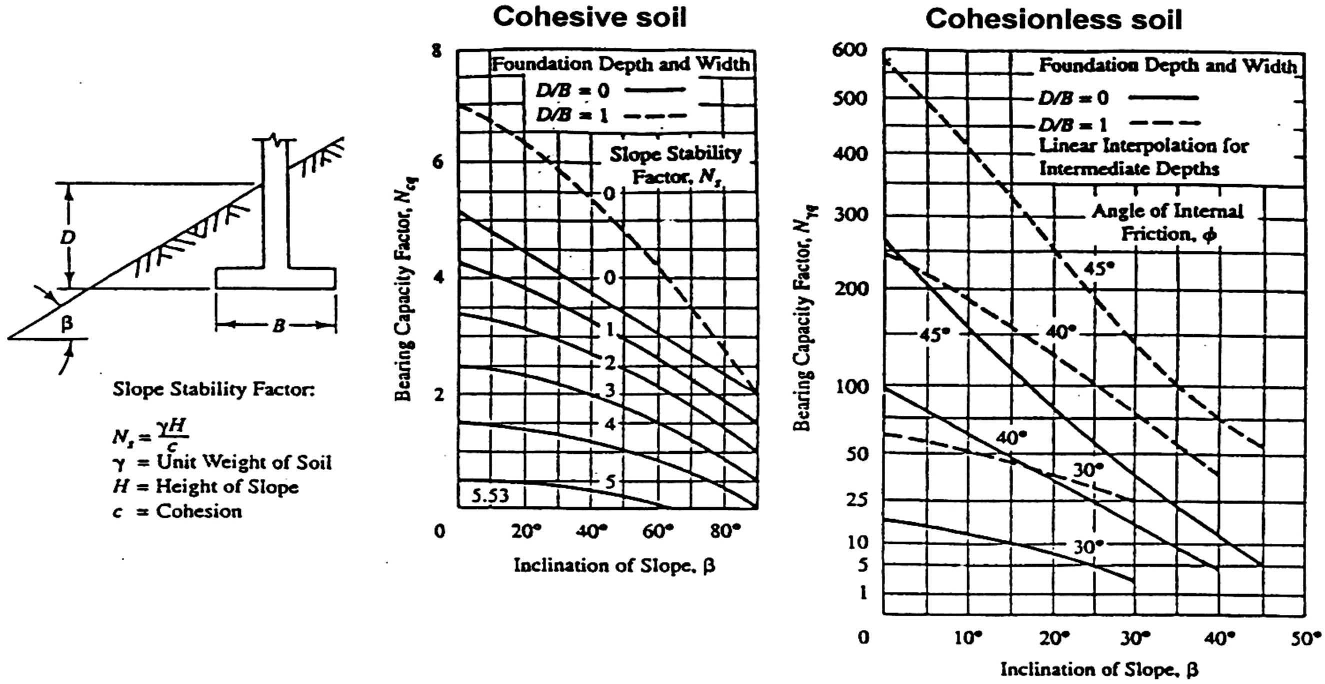
c or s footing denotes either circular or square footing, and

$(q_{ult.})$ of footing on level ground is calculated from Terzaghi's equation.

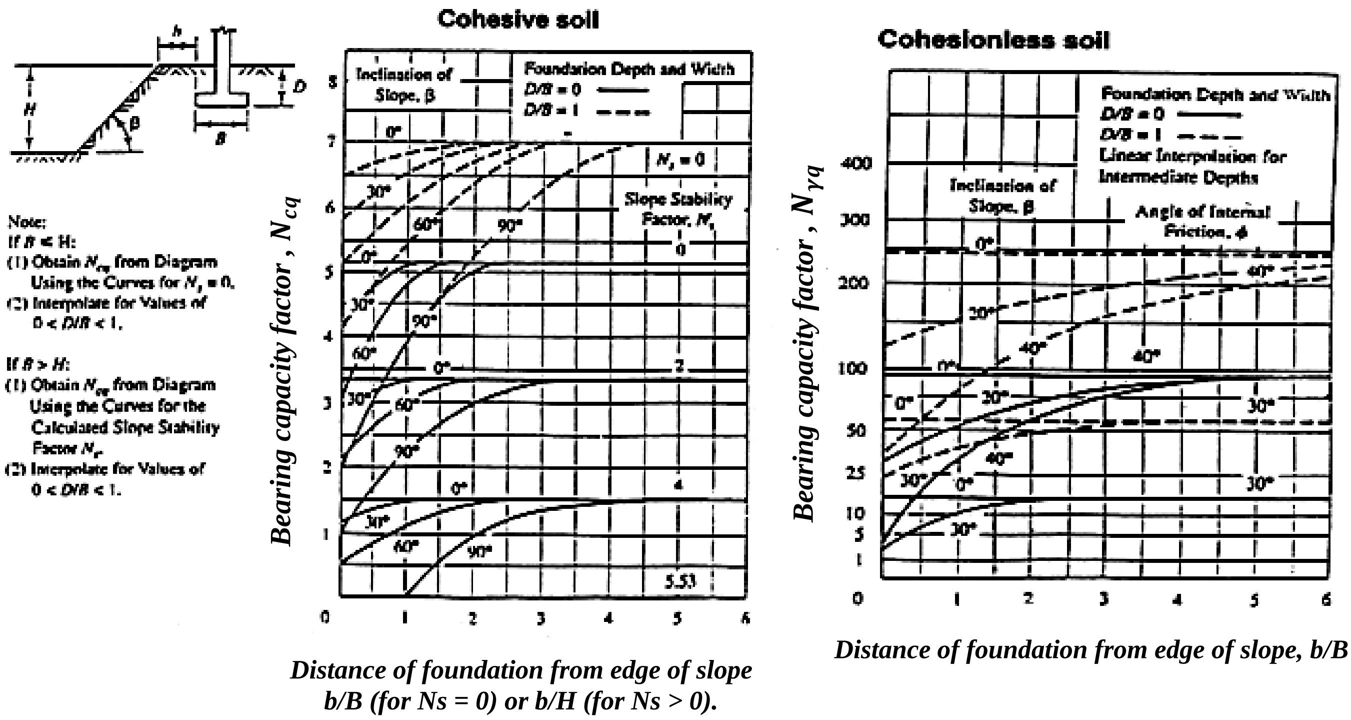
Notes:

(1) A $\phi_{triaxial}$ should not be adjusted to ϕ_{ps}, since the slope edge distorts the failure pattern such that plane-strain conditions may not develop except for large b/B ratios.

(2) For footings on or adjacent to a slope, the overall slope stability should be checked for the footing load using a slope-stability program or other methods *such as **method of slices by Bishop's***.



(a) On face of slope.



(b) On top of slope.

Figure (3.18): bearing capacity factors for continuous footing *(after Meyerhof).*

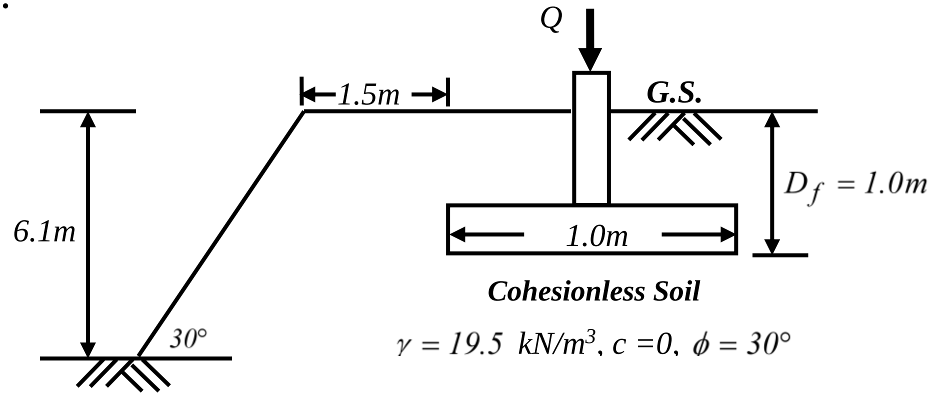
BEARING CAPACITY EXAMPLES (4)

Footings on slopes

Prepared by: Dr. Farouk Majeed Muhauwiss
Civil Engineering Department – College of Engineering
Tikrit University

Example (16): *(footing on top of a slope)*

A bearing wall for a building is to be located close to a slope as shown in figure. The ground water table is located at a great depth. Determine the allowable bearing capacity by Meyerhof's method using F.S. =3?.



Solution:

$$(q_{ult.})_{continuous.footing.on.slope} = cN_{cq} + \frac{1}{2}\gamma.B.N_{\gamma q} \quad \text{.....................................}\mathbf{(3.36)}$$

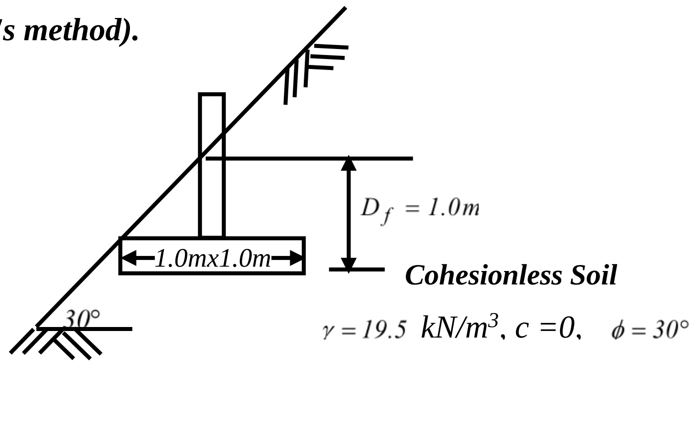
From figure (3.18-b): with $\phi = 30°$, $\beta = 30°$, $\frac{b}{B} = \frac{1.5}{1.0} = 1.5$, *and* $\frac{D_f}{B} = \frac{1.0}{1.0} = 1.0$ *(use the dashed line)* $\longrightarrow$ $N_{\gamma q} = 40$

$$(q_{ult.})_{continuous.footing.on.slope} = (0)N_{cq} + \frac{1}{2}(19.5)(1.0)(40) = 390 \text{ kN/m}^2$$

$$q_{all.} = 390/3 = \boxed{130 \text{ kN/m}^2.}$$

Example (17): *(footing on face of a slope)*

Same conditions as example (16), except that a 1.0m-by 1.0m square footing is to be constructed on the slope (use Meyerhof's method).



Solution:

$$(q_{ult.})_{c.or.s.footing.on.slope} = (q_{ult.})_{continuous footing.on.slope}\left[\frac{(q_{ult.})_{c.or.s.footing.on.level.ground}}{(q_{ult.})_{continuous footing.on.level.ground}}\right] \quad \textbf{.....(3.37)}$$

$$(q_{ult.})_{continuous.footing.on.slope} = (0)N_{cq} + \frac{1}{2}(19.5)(1.0)(25) = \boxed{243.75 kN/m^2}$$

$(q_{ult.})$ *<u>of square or strip footing on level ground is calculated from Terzaghi's equation:</u>*

$$q_{ult.} = cN_cS_c + qN_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma$$

<u>Bearing capacity factors from table (3.3):</u> *for* $\phi = 30°$; $N_c = 37.2,..N_q = 22.5,..N_\gamma = 19.7$

<u>Shape factors table (3.2):</u> *for square footing* $S_c = 1.3, S\gamma = 0.8$; *strip footing* $S_c = S_\gamma = 1.0$

$$(q_{ult.})_{square.footing.on.level.ground} = 0 + 1.0\,(19.5)(22.5) + 0.5(1.0)(19.5)(19.7)(0.8) = 592.4\ kN/m^2$$

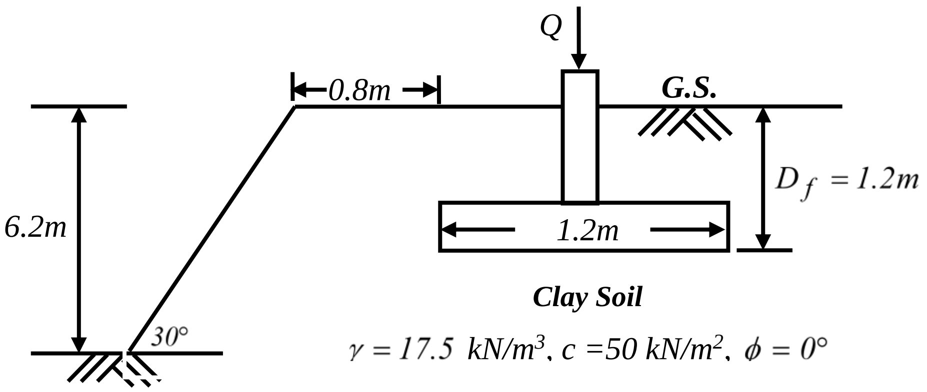
$$(q_{ult.})_{continuous.footing.on.level.ground} = 0 + 1.0\,(19.5)(22.5) + 0.5(1.0)(19.5)(19.7)(1.0) = 630.8\ kN/m^2$$

$$\therefore \quad (q_{ult.})_{square.footing.on.slope} = 243.75\frac{592.4}{630.8} = 228.912\ kN/m^2$$

and $(q_{all.})_{square.footing.on.slope} = \frac{228.912}{3} = \boxed{76\ kN/m^2}$

<u>Example (18):</u> *(footing on top of a slope)*

A shallow continuous footing in clay is to be located close to a slope as shown in figure. The ground water table is located at a great depth. Determine the gross allowable bearing capacity using F.S. = 4



<u>Solution:</u>

Since B<H assume the stability number $N_s = 0$ *and for* ***purely cohesive soil,*** $\phi = 0$

$$\boxed{(q_{ult.})_{continuous.footing.on.slope} = cN_{cq}}$$

From figure (3.18-b) for cohesive soil: with $\phi = 30°$ **,** $N_s = 0$ **,** $\frac{b}{B} = \frac{0.8}{1.2} = 0.67$ **,** *and*

$\frac{D_f}{B} = \frac{1.2}{1.2} = 1.0$ *(use the dashed line)* $\longrightarrow$ $N_{cq} = 6.3$

$(q_{ult.})_{continuous.footing.on.slope} = (50)(6.3) = 315\ kN/m^2$

$q_{all.} = 315/4 = \boxed{78.8\ kN/m^2.}$

3.15 Foundation with Tension Force

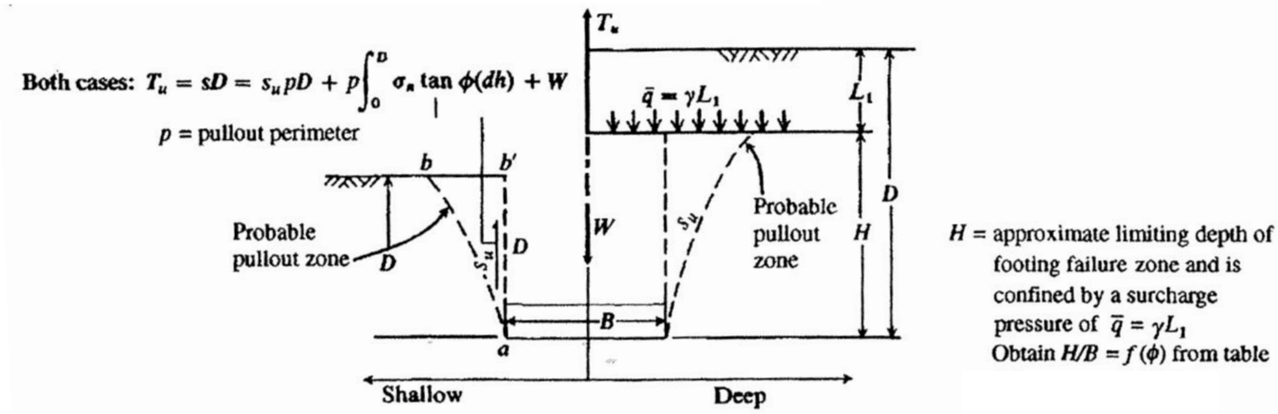


Figure (3.19) Footing for tension loads

For shallow footings

Round: $$T_u = \pi B s_u D + s_f \pi B \gamma \left(\frac{D^2}{2}\right) K_u \tan\phi + W$$

Rectangular: $$T_u = 2 s_u D(B + L) + \gamma D^2(2 s_f B + L - B) K_u \tan\phi + W$$

where the side friction adjustment factor $s_f = 1 + mD/B$.

For deep footings (base depth $D > H$)

Round: $$T_u = \pi s_u BH + s_f \pi B \gamma (2D - H)\left(\frac{H}{2}\right) K_u \tan\phi + W$$

Rectangular: $$T_u = 2 s_u H(B + L) + \gamma(2D - H)(2 s_f B + L - B) H K_u \tan\phi + W$$

where $s_f = 1 + mH/B$.

For footing shape

Round: B = diameter

Square: $L = B$

Rectangular: use B and L

W= W. footing +W. Soil + any additional load

$K_u = (1-\sin\varnothing)\sqrt{O.C.R}$

Obtain shape factor s_f, ratios m and H/B [all $f(\phi)$] from the following table—interpolate as necessary:

ϕ

$\phi =$		20°	25°	30°	35°	40°	45°	48°
imiting	H/B	2.5	3	4	5	7	9	11
	m	0.05	0.10	0.15	0.25	0.35	0.50	0.60
laximum	s_f	1.12	1.30	1.60	2.25	4.45	5.50	7.60

or example: $\phi = 20°$ so obtain $s_f = 1.12$, $m = 0.05$, and $H/B = 2.5$. Therefore, $H = 2.5B$, and total footing depth to be a "deep" footing $D > 2.5B$. If $B = 1$ m, D of Fig. 4-10 must be greater than 2.5 m, or else use "shallow footing"

3.16 Foundation on Rock

It is common to use the building code values for the allowable bearing capacity of rocks (***see Table 3.8***)**.** However, there are several significant parameters which should be taken into consideration together with the recommended code value; such as site geology, rock type and quality (as RQD).

Usually, the shear strength parameters c and ϕ of rocks are obtained from high Pressure Triaxial Tests. However, for most rocks $\phi = 45°$ except for limestone or shale $\phi = (38° - 45°)$ can be used. Similarly in most cases we could estimate $c = 5$ MPa with a conservative value.

Table (3.8): Allowable contact pressure $q_{all.}$ ***of jointed rock.***

RQD %	$q_{all.}$ **(T/ft²)**	$q_{all.}$ **(kN/m²)**	***Quality***
100	*300*	*31678*	*Excelent*
90	*200*	*21119*	*Very good*
75	*120*	*12671*	*Good*
50	*65*	*6864*	*Medium*
25	*30*	*3168*	*Poor*
0	*10*	*1056*	*Very poor*
1.0 (T/ft²) = 105.594 (kN/m²)			

Notes:

(1) *If* $q_{all.}(tabulated) > q_u(unconfined..compressive..strength)$ *of intact rock sample, then take* $q_{all.} = q_u$.

(2) *The settlement of the foundation should not exceed (0.5 inch) or (12.7mm) even for large loaded area.*

(3) *If the upper part of rock within a depth of about B/4 is of lower quality, then its RQD value should be used or that part of rock should be removed.*

Any of the bearing capacity equations from Table (3.2) with specified shape factors can be used to obtain $q_{ult.}$ of rocks, but with bearing capacity factors for sound rock proposed by *(Stagg and Zienkiewicz, 1968)* as:

$$N_c = 5\tan^4(45 + \phi/2), \qquad N_q = \tan^6(45 + \phi/2), \qquad N_\gamma = N_q + 1$$

Then, $q_{ult.}$ must be reduced on the basis of RQD as:

$$q'_{ult.} = q_{ult.}(RQD)^2$$

and
$$q_{all.} = \frac{q_{ult.}(RQD)^2}{F.S.}$$

where: *F.S.=safety factor dependent on RQD. It is common to use F.S. from (6-10) with the higher values for RQD less than about 0.75.*

- **<u>Rock Quality Designation (RQD):</u>**

It is an index used by engineers to measure the quality of a rock mass and computed from recovered core samples as:

$$RQD = \frac{\sum lengths..of..intact..pieces..of..core > 100mm}{length..of..core..advance}$$

<u>Example (19):</u> *(RQD)*

A core advance of 1500mm produced a sample length of 1310mm consisting of dust, gravel and intact pieces of rock. The sum of pieces 100mm or larger in length is 890mm.

<u>Solution:</u>

The recovery ratio $(L_r) = \frac{1310}{1500} = 0.87$**;** *and* $(RQD) = \frac{890}{1500} = 0.59$

<u>Example (20):</u> *(foundation on rock)*

A pier with a base diameter of 0.9m drilled to a depth of 3m in a rock mass. If RQD = 0.5, $\phi = 45°$ ***and c = 3.5 MPa ,*** γ_{rock} ***= 25.14 kN/m³, estimate*** $q_{all.}$ ***of the pier using Terzaghi's equation.***

<u>Solution:</u>

<u>By Terzaghi's equation:</u> $q_{ult.} = cN_c.S_c + qN_q + \frac{1}{2}.B.\gamma.N\gamma.S_\gamma$

<u>Shape factors</u>: from table (3.2) for circular footing: $S_c = 1.3$; $S_\gamma = 0.6$

Bearing capacity factors: $N_c = 5\tan^4(45+\phi/2)$, $N_q = \tan^6(45+\phi/2)$, $N_\gamma = N_q + 1$

for $\phi = 45°$, $N_c = 170$, $N_q = 198$, $N_\gamma = 199$

$$q_{ult.} = (3.5x10^3)(170)(1.3) + (3)(25.14)(198) + 0.5(25.14)(0.9)(199)(0.6) = 789.78 \ MPa$$

and

$$q_{all.} = \frac{q_{ult.}(RQD)^2}{F.S.} = \frac{789.78(0.5)^2}{3.0} = 65.815..MPa$$

CHAPTER 4

STRESSES IN SOIL MASS

4.1 DEFIFINTIONS

- **VERTTICAL STRESS**

  Occurs due to internal or external applied load such as, overburden pressure, weight of structure and earthquake loads.

- **HORIZONTAL STRESS**

  Occurs due to vertical stress or earth pressure, water pressure, wind loads or earthquake horizontal loads.

- **ISOBAR**

  It is a contour connecting all points below the ground surface of equal intensity of pressure.

- **PRESSURE BULB**

  The zone in a loaded soil mass bounded by an isobar of a given pressure intensity is called a pressure bulb for that intensity.

4.2 CONTACT PRESSURE

The analysis of Borowicka (1938) shows that the distribution of contact stress was dependent on a non-dimensional factor defined as:

$$k_r = \frac{1}{6}\left(\frac{1-\nu^2{}_s}{1-\nu^2{}_f}\right)\left(\frac{E_f}{E_s}\right)\left(\frac{T}{b}\right)^3 \quad \text{..(4.1)}$$

where: ν_s and ν_f = Poisson's ratio for soil and foundation materials, respectively,

E_s and E_f = Young's modulus for soil and foundation materials, respectively,

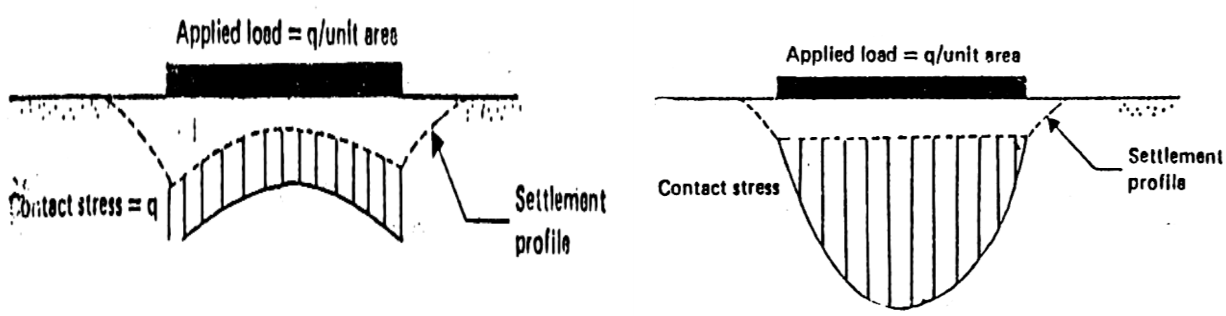
T = thickness of foundation,

B = half-width for strip footing; or radius for circular footing,

k_r = 0 indicates a perfectly flexible foundation; or

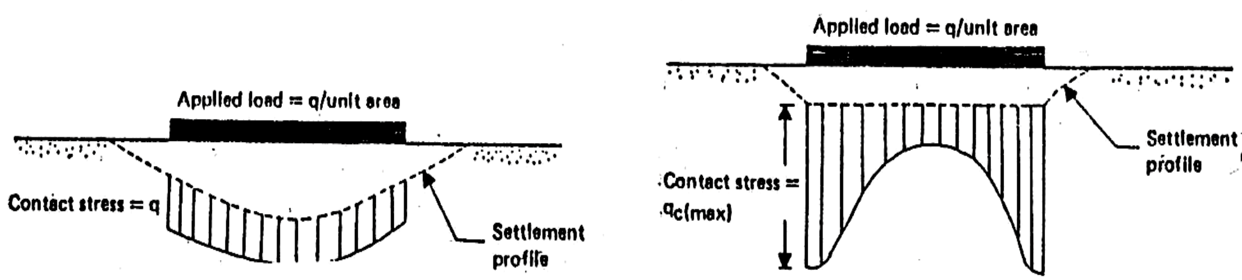
∞ means a perfectly rigid foundation.

The actual soil pressures distributions of rigid and flexible footings resting on sand and clay soils are shown in **Figures (4.1 and 4.2)**.



(a) Flexible **(b) Rigid**

Figure (4.1): Foundations on sand.



(a) Flexible **(b) Rigid**

Figure (4.2): Foundations on clay.

4.3 ASSUMPTIONS: The soil is assumed as:

(1) Semi-infinite in extent; x and y are infinite but the depth z has a limit value (Half-space),
(2) Isotropic; the soil has same properties in all directions,
(3) Homogeneous,
(4) Elastic and obeys Hook's law; the soil has linear relationship,
(5) Stresses at a point due to more than one surface load are obtained by superposition, and
(6) Negative values of loading can be used if the stresses due to excavations were required or the principle of superposition was used.

STRESS INCREASE DUE TO DIFFERENT LOADING

(1) POINT LOAD

- **BOUSSINESQ METHOD FOR HOMOGENEOUS SOIL:**

This method can be used for point loads acts directly at or outside the center. For a central load acting on the surface, its nature at depth z and radius r according to (simple radial stress distribution) is a cylinder in two-dimensional condition and a sphere in three-dimensional case.

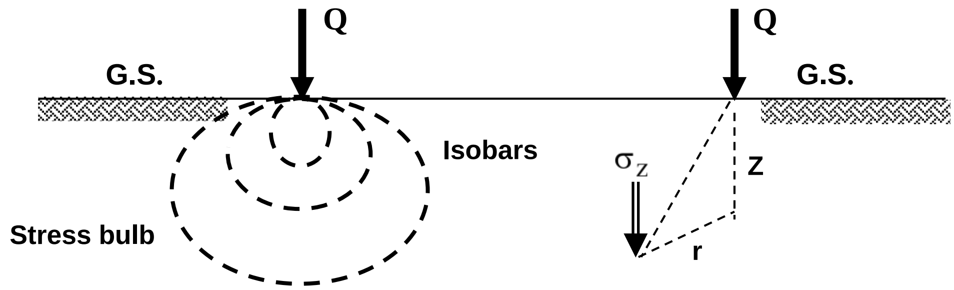


Figure (4.3): Vertical stress due to point load.

The vertical stress increase below or outside the point of load application is calculated as:

$$\sigma_z = \frac{Q}{.z^2} A_b \quad \text{……..........................………………...………...}\textbf{(4.2)}$$

where: $A_b = \dfrac{3/2\pi.}{\left[1+(\frac{r}{z})^2\right]^{5/2}}$

- **WESTERGAARD METHOD FOR STRATIFIED SOIL:**

$$\sigma_z = \frac{Q}{2\pi.z^2} \frac{\sqrt{(1-2\mu)/(2-2\mu)}}{\left\{[(1-2\mu)/(2-2\mu)]+(\frac{r}{z})^2\right\}^{3/2}}$$

……......................(4.3a)

where: $\mu =$ Poisson's ratio.

when $\mu = 0$: $\sigma_z = \dfrac{Q}{.z^2} A_W$ **……........................………………...………(4.3b)**

where: $A_W = \dfrac{1/\pi.}{\left[1+2(\frac{r}{z})^2\right]^{3/2}}$; Values of A_W can be tabulated for different values of μ as:

r/z	A_W		
	$\mu = \mathbf{0}$	$\mu = \mathbf{0.4}$	
0.0	0.3183	0.9549	**Note that :** At (r/z ≈ 1.8) both Boussinessq and Westergaard methods give equal values of σ_z.
0.2	0.2836	0.6916	
0.8	0.0925	0.0897	
1.0	0.0613	0.0516	
2.0	0.0118	0.0076	
3.0	0.0038	0.0023	
4.0	0.0017	0.0010	

Example (4.1): A concentrated point load Q acts vertically at the ground surface. Determine the vertical stress σ_z for each of the following cases:

a. Along the depth for r = 2m, and
b. At depth z = 2m.

Solution: From Boussinesq's equation: $\sigma_z = \frac{Q}{.z^2} A_b$ where: $A_b = \frac{3/2\pi.}{\left[1+(\frac{r}{z})^2\right]^{5/2}}$

(a) For r = 2m, the values of σ_z at various arbitrarily selected depths are given in the following table and the distribution of σ_z with depth is shown in **Figure (4.4 a)**.

z (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	∞	0	0	∞	Indeterminate
0.4	5.0	0.00014	0.16	6.250Q	0.0009Q
0.8	2.5	0.00337	0.64	1.563Q	0.0053Q
1.2	1.67	0.01712	1.44	0.694Q	0.0119Q
1.6	1.25	0.04543	2.56	0.391Q	0.0178Q
2.0	1.00	0.08440	4.00	0.250Q	0.0211Q
2.4	0.83	0.12775	5.76	0.174Q	0.0222Q
2.8	0.71	0.17035	7.84	0.128Q	0.217Q
3.6	0.56	0.24372	12.96	0.0772Q	0.0188Q
5.0	0.40	0.32946	25.00	0.0400Q	0.0132Q
10.0	0.20	0.43287	100.00	0.0100Q	0.0043Q

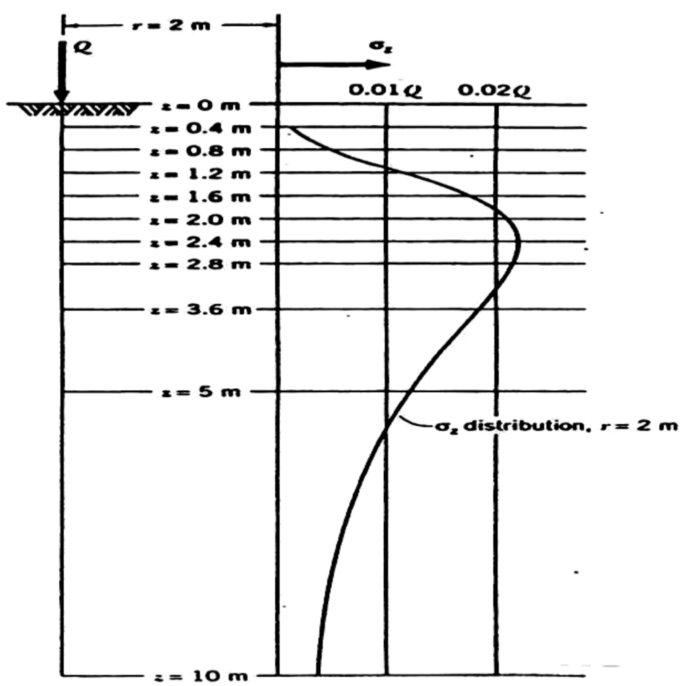


Figure (4.4 a): σ_z distribution with depth at a fixed radial distance from point of surface load.

(b) At depth z = 2m, the values of σ_z for various horizontal distances of r are given in the following table and the distribution of σ_z with r is shown in **Figure (4.4 b)**.

r (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	0	0.47746	4.0	0.25Q	0.1194Q
0.4	0.2	0.43287	4.0	0.25Q	0.1082Q
0.8	0.4	0.32946	4.0	0.25Q	0.0824Q
1.2	0.6	0.22136	4.0	0.25Q	0.0553Q
1.6	0.8	0.13862	4.0	0.25Q	0.0347Q
2.0	1.0	0.08440	4.0	0.25Q	0.0211Q
2.4	1.2	0.05134	4.0	0.25Q	0.0129Q
2.8	1.4	0.03168	4.0	0.25Q	0.0079Q
3.6	1.8	0.01290	4.0	0.25Q	0.0032Q
5.0	2.5	0.00337	4.0	0.25Q	0.0008Q
10.0	5.0	0.00014	4.0	0.25Q	0.0001Q

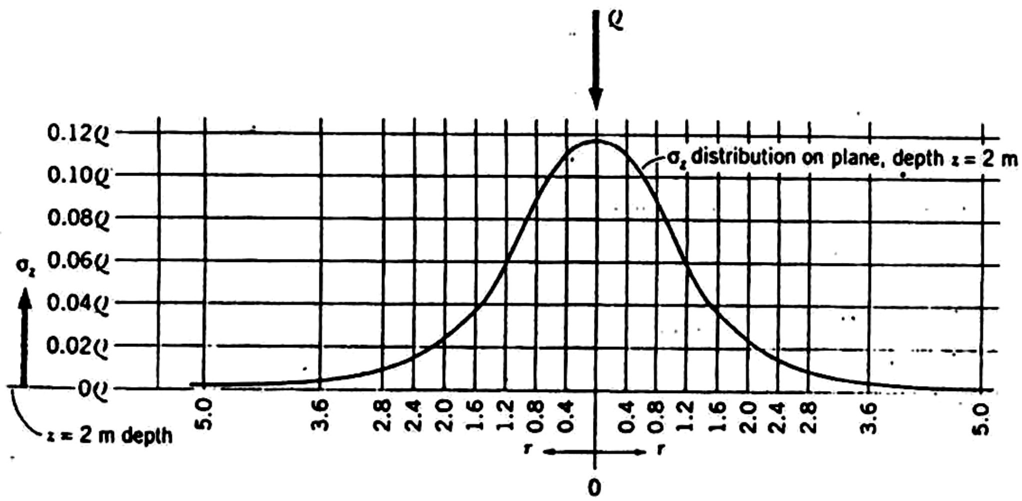


Figure (4.4 b): σ_z distribution with depth at a fixed radial distance from point of surface load.

Example (4.2): Q, is a concentrated point load acts vertically at the ground surface. Determine the vertical stress σ_z for various values of horizontal distances r and at z = 1, 2, 3, and 4m, then plot the σ_z distribution for all z depths.

Solution: From Boussinesq's equation: $\sigma_z = \frac{Q}{.z^2} A_b$ where: $A_b = \frac{3/2\pi.}{\left[1+(\frac{r}{z})^2\right]^{5/2}}$

σ_z for z = 1, 2, 3, and 4m depths is given in the following tables and their distributions with horizontal distances are shown in **Figure (4.5)**.

z = 1 m					
r (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	0	0.47746	1.0	Q	0.47746Q
0.4	0.4	0.32946	1.0	Q	0.32946Q
0.8	0.8	0.13862	1.0	Q	0.13862Q
1.2	1.2	0.05134	1.0	Q	0.05134Q
1.6	1.6	0.01997	1.0	Q	0.01997Q
2.0	2.0	0.00854	1.0	Q	0.00854Q
2.4	2.4	0.00402	1.0	Q	0.00402Q
2.8	2.8	0.00206	1.0	Q	0.00206Q
3.6	3.6	0.00066	1.0	Q	0.00066Q
5.0	5.0	0.00014	1.0	Q	0.00014Q

z = 2 m					
r (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	0	0.47746	4.0	0.25Q	0.1194Q
0.4	0.2	0.43287	4.0	0.25Q	0.1082Q
0.8	0.4	0.32946	4.0	0.25Q	0.0824Q
1.2	0.6	0.22136	4.0	0.25Q	0.0553Q
1.6	0.8	0.13862	4.0	0.25Q	0.0347Q
2.0	1.0	0.08440	4.0	0.25Q	0.0211Q
2.4	1.2	0.05134	4.0	0.25Q	0.0129Q
2.8	1.4	0.03168	4.0	0.25Q	0.0079Q
3.6	1.8	0.01290	4.0	0.25Q	0.0032Q
5.0	2.5	0.00337	4.0	0.25Q	0.0008Q

z = 3 m					
r (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	0	0.47746	9.0	Q/9	0.0531Q
0.4	0.1333	0.45630	9.0	Q/9	0.0507Q
0.8	0.2666	0.40200	9.0	Q/9	0.0447Q
1.2	0.4000	0.32950	9.0	Q/9	0.0366Q
1.6	0.5333	0.25555	9.0	Q/9	0.0284Q
2.0	0.6666	0.19060	9.0	Q/9	0.0212Q
2.4	0.8000	0.13862	9.0	Q/9	0.0154Q
2.8	0.9333	0.09983	9.0	Q/9	0.0111Q
3.6	1.2000	0.05134	9.0	Q/9	0.0057Q
5.0	3.3333	0.01710	9.0	Q/9	0.0019Q

Z = 4 m					
r (m)	r/z	A_b	z^2	Q/z^2	σ_z (Q/m^2)
0.0	0	0.47746	16	Q/16	0.02984Q
0.4	0.1	0.46573	16	Q/16	0.02911Q
0.8	0.2	0.43287	16	Q/16	0.02705Q
1.2	0.3	0.38492	16	Q/16	0.02406Q
1.6	0.4	0.32946	16	Q/16	0.02059Q
2.0	0.5	0.27332	16	Q/16	0.01708Q
2.4	0.6	0.22136	16	Q/16	0.01384Q
2.8	0.7	0.17619	16	Q/16	0.01101Q
3.6	0.9	0.10833	16	Q/16	0.00677Q
5.0	1.25	0.04543	16	Q/16	0.00284Q

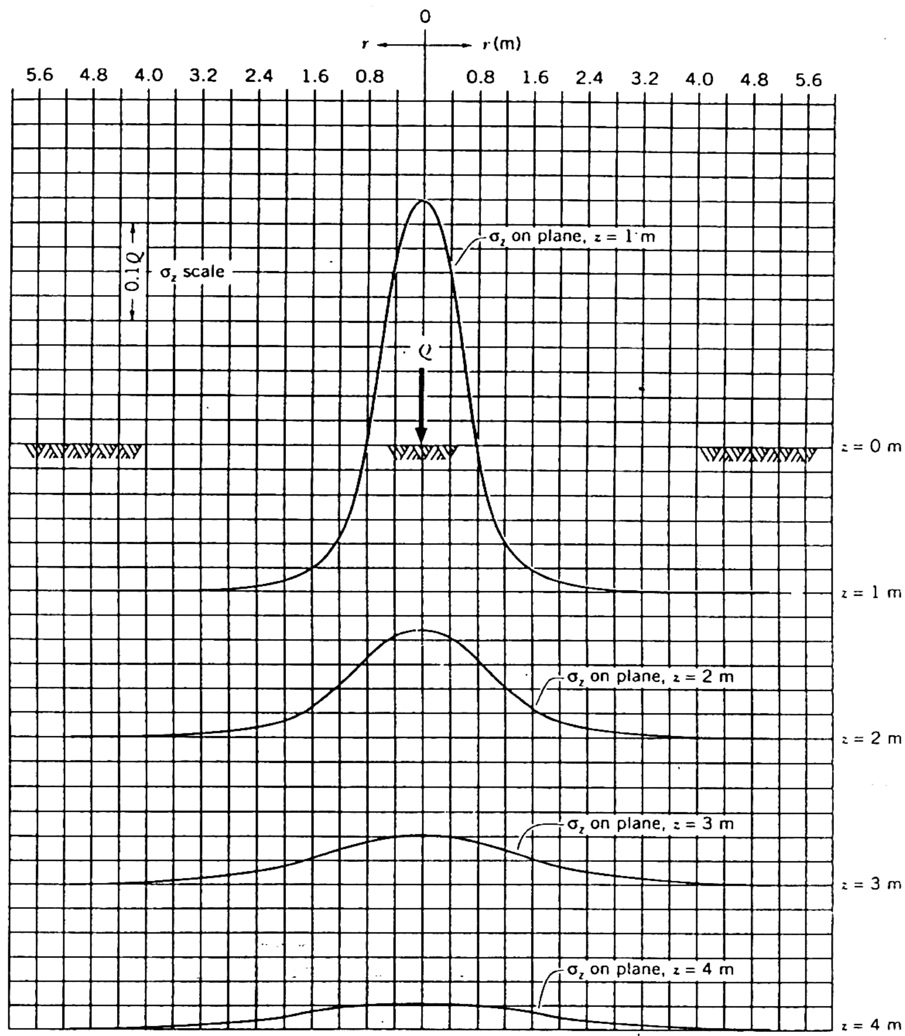
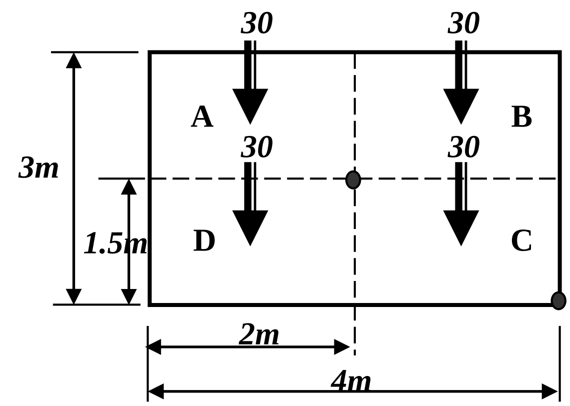


Figure (4.5): σ_z distribution with horizontal distance from point of surface load at several depths.

Example (4.3): An elastic soil medium of (4m x 3m) rectangular area is shown in figure. If the area is divided into 4 elementary areas of (2m x 1.5m) each that subjected at its surface a concentrated loads of (30 ton) at its centroid, use the Boussinesq's equation to find the vertical pressure at a depth of 6m below:

1. the center of the area,
2. one corner of the area.



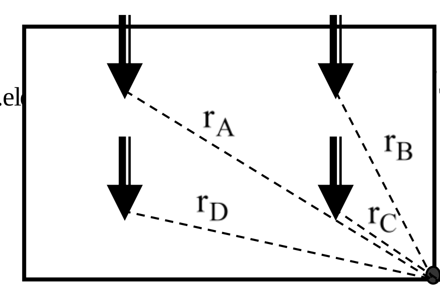
Solution:

From Boussinesq's equation: $\sigma_z = \frac{Q}{.z^2} A_b$ where: $A_b = \frac{3/2\pi.}{\left[1+(r/z)^2\right]^{5/2}}$

(a) At the center of the area:

r = 1.25m, r/z = 1.25/6 = 0.208

$A_b = 3/2\pi./.\left[1+(0.208)^2\right]^{5/2} = 0.4293$, $\sigma_{z\,(One.el}$ 5



T/m^2

$\sigma_{z\,(Total)} = (4).0.35775 = 1.43\,T/m^2$

At one corner of the area:

Elementary area	r (m)	r/z	A_b	$\sigma_z = \frac{Q}{.z^2} A_b$
A	3.750	0.625	0.209	0.174
B	2.462	0.410	0.324	0.270
C	1.250	0.208	0.429	0.358
D	3.092	0.515	0.265	0.221
$\sum$				1.023 T/m2

(2) 2:1 APPROXIMATION METHOD *for depths < 2.5 (width of loaded area):*

Total load on the surface = q.B.L; and Area at depth z = (L + z) (B + z)

$$\sigma_z = \frac{q.B.L}{(L+z)(B+z)} \quad \text{..(4.4)}$$

Type of footing	Area, A_z
Square	$(B+z)^2$
Rectangular	$(B+z)(L+z)$
Circular	$\pi(D+z)^2/4$
Strip or wall	$(B+z).1$

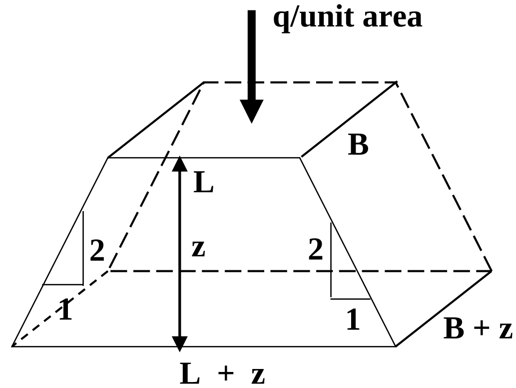


Figure (4.6): 2:1 Stress distribution method.

(3) UNIFORMLY LOADED LINE OF FINITE LENGTH:

Figure (4.7) shows a line load of equal intensity **q** applied at the surface. For an element selected at an arbitrary fixed point in the soil mass, an expression for σ_z could be derived by integrating Boussinesq's expression for point load as:

$$\sigma_z = \frac{Q}{.z^2} P_o \quad \text{..(4.5)}$$

where:

$$P_o = \frac{1}{2\pi(m^2+1)^2}\left[\frac{3n}{\sqrt{n^2+1+m^2}} - \left(\frac{n}{\sqrt{n^2+1+m^2}}\right)^3\right]$$

$m = x/z$, and $n = y/z$

Values of P_o for various combinations of **m** and **n** are given in **Table (4.1).**

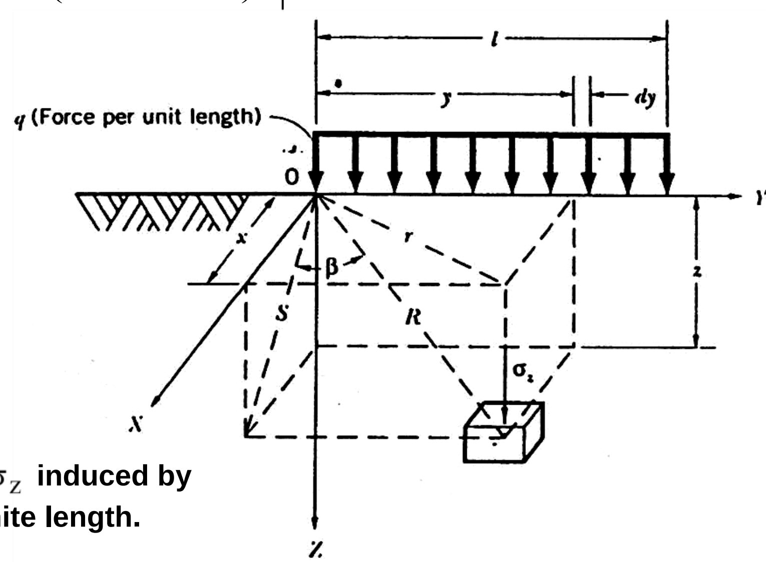


Figure (4.7): Vertical stress σ_z induced by Line loads of finite length.

Table (4.1): Influence values P_0 for case of uniform line load of finite length.

m	n											
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4
0.0	0.04735	0.09244	0.13342	0.16917	0.19929	0.22398	0.24379	0.25947	0.27176	0.28135	0.29464	0.30277
0.1	0.04619	0.09020	0.13023	0.16520	0.19470	0.21892	0.23839	0.25382	0.26593	0.27539	0.28853	0.29659
0.2	0.04294	0.08391	0.12127	0.15403	0.18178	0.20466	0.22315	0.23787	0.24947	0.25857	0.27127	0.27911
0.3	0.03820	0.07472	0.10816	0.13764	0.16279	0.18367	0.20066	0.21429	0.22511	0.23365	0.24566	0.25315
0.4	0.03271	0.06406	0.09293	0.11855	0.14058	0.15905	0.17423	0.18651	0.19634	0.20418	0.21532	0.22235
0.5	0.02715	0.05325	0.07742	0.09904	0.11782	0.13373	0.14694	0.15775	0.16650	0.17354	0.18368	0.19018
0.6	0.02200	0.04322	0.06298	0.08081	0.09646	0.10986	0.12112	0.13045	0.13809	0.14430	0.15339	0.15931
0.7	0.01752	0.03447	0.05035	0.06481	0.07762	0.08872	0.09816	0.10608	0.11265	0.11805	0.12607	0.13140
0.8	0.01379	0.02717	0.03979	0.05136	0.06172	0.07080	0.07862	0.08525	0.09082	0.09546	0.10247	0.10722
0.9	0.01078	0.02128	0.03122	0.04041	0.04872	0.05608	0.06249	0.06800	0.07268	0.07663	0.08268	0.08687
1.0	0.00841	0.01661	0.02441	0.03169	0.03832	0.04425	0.04948	0.05402	0.05793	0.06126	0.06645	0.07012
1.2	0.00512	0.01013	0.01495	0.01949	0.02369	0.02752	0.03097	0.03403	0.03671	0.03905	0.04281	0.04558
1.4	0.00316	0.00626	0.00927	0.01213	0.01481	0.01730	0.01957	0.02162	0.02345	0.02508	0.02777	0.02983
1.6	0.00199	0.00396	0.00587	0.00770	0.00944	0.01107	0.01258	0.01396	0.01522	0.01635	0.01828	0.01979
1.8	0.00129	0.00256	0.00380	0.00500	0.00615	0.00724	0.00825	0.00920	0.01007	0.01086	0.01224	0.01336
2.0	0.00085	0.00170	0.00252	0.00333	0.00410	0.00484	0.00554	0.00619	0.00680	0.00736	0.00836	0.00918
2.5	0.00034	0.00067	0.00100	0.00133	0.00164	0.00194	0.00224	0.00252	0.00278	0.00303	0.00349	0.00389
3.0	0.00015	0.00030	0.00045	0.00060	0.00074	0.00088	0.00102	0.00115	0.00127	0.00140	0.00162	0.00183
4.0	0.00004	0.00008	0.00012	0.00016	0.00020	0.00024	0.00027	0.00031	0.00035	0.00038	0.00045	0.00051

m	n										
	1.6	1.8	2.0	2.5	3.0	4.0	5.0	6.0	8.0	10.0	∞
0.0	0.30784	0.31107	0.31318	0.31593	0.31707	0.31789	0.31813	0.31822	0.31828	0.31830	0.31831
0.1	0.30161	0.30482	0.30692	0.30966	0.31080	0.31162	0.31186	0.31195	0.31201	0.31203	0.31204
0.2	0.28402	0.28716	0.28923	0.29193	0.29307	0.29388	0.29412	0.29421	0.29427	0.29428	0.29430
0.3	0.25788	0.26092	0.26293	0.26558	0.26670	0.26750	0.26774	0.26783	0.26789	0.26790	0.26792
0.4	0.22683	0.22975	0.23169	0.23426	0.23535	0.23614	0.23638	0.23647	0.23653	0.23654	0.23656
0.5	0.19438	0.19714	0.19899	0.20147	0.20253	0.20331	0.20354	0.20363	0.20369	0.20371	0.20372
0.6	0.16320	0.16578	0.16753	0.16990	0.17093	0.17169	0.17192	0.17201	0.17207	0.17208	0.17210
0.7	0.13496	0.13735	0.13899	0.14124	0.14224	0.14297	0.14320	0.14329	0.14335	0.14336	0.14338
0.8	0.11044	0.11264	0.11416	0.11628	0.11723	0.11795	0.11818	0.11826	0.11832	0.11834	0.11835
0.9	0.08977	0.09177	0.0318	0.09517	0.09608	0.09677	0.09699	0.09708	0.09713	0.9715	0.09716
1.0	0.07270	0.07452	0.07580	0.07766	0.07852	0.07919	0.07941	0.07949	0.07955	0.07957	0.07958
1.2	0.04759	0.04905	0.05012	0.05171	0.05248	0.05310	0.05330	0.05338	0.05344	0.05345	0.05347
1.4	0.03137	0.03253	0.03340	0.03474	0.03542	0.03598	0.03617	0.03625	0.03630	0.03632	0.03633
1.6	0.02097	0.02188	0.02257	0.02368	0.02427	0.02478	0.02496	0.02504	0.02509	0.02510	0.02512
1.8	0.01425	0.01496	0.01551	0.01643	0.01694	0.01739	0.01756	0.01765	0.01768	0.01769	0.01771
2.0	0.00986	0.01041	0.01085	0.01160	0.01203	0.01244	0.01259	0.01266	0.01271	0.01272	0.01273
2.5	0.00424	0.00453	0.00477	0.00523	0.00551	0.00581	0.00593	0.00599	0.00603	0.00605	0.00606
3.0	0.00201	0.00217	0.00231	0.00258	0.00277	0.00298	0.00307	0.00312	0.00316	0.00317	0.00318
4.0	0.00057	0.00063	0.00068	0.00078	0.00086	0.00096	0.00102	0.00105	0.00108	0.00109	0.00110

Problem (4.4): **Given:** q = 100 kN/m.
Find: The vertical stress σ_z at points 0, 1, and 2 shown in Fig. (4.8)?

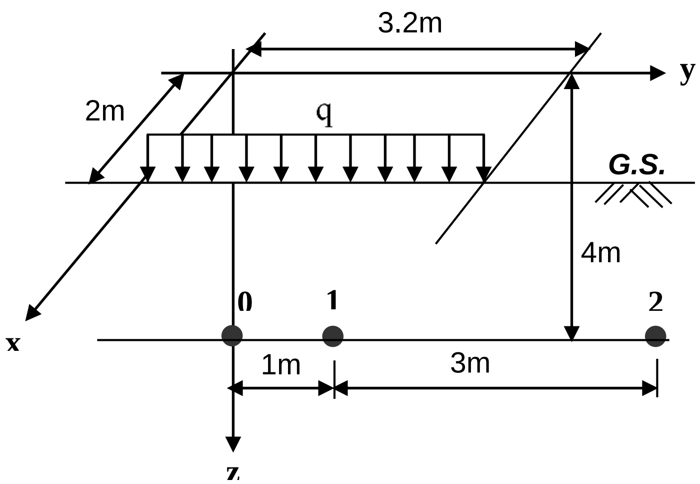


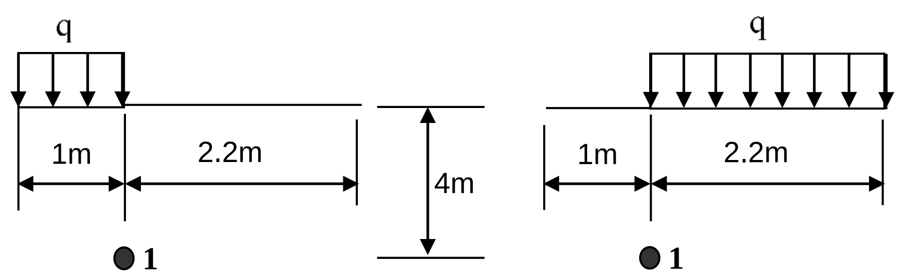
Figure (4.8): Uniformly line surface load of finite length.

Solution:

(a) σ_z at point (0):

For m = x/z = 2/ 4 = 0.5 and n = y/z = 3.2/ 4 = 0.8 from **Table (4.1)**: $P_0 = 0.15775$

$$\sigma_{z(0)} = \frac{Q}{z} P_0 = \frac{100}{4} 0.15775 = 3.944.kN/m^2$$



(a) Left load. **(b) Right load.**

Figure (4.9): σ_z **at point (1).**

(b) σ_z at point (1):

$$\sigma_{z(1)} = \sigma_{z(1L)} + \sigma_{z(1R)}$$

From Fig. (4.9-a)

For m = x/z = 2/ 4 = 0.5 and n = y/z = 1/ 4 = 0.25 from **Table (4.1)**: $P_0 = 0.06534$

$$\sigma_{z(1L)} = \frac{Q}{z} P_0 = \frac{100}{4}(0.06534) = 1.634.kN/m^2$$

From Fig. (4.9-b)

For m = x/z = 2/ 4 = 0.5 and n = y/z = 2.2/ 4 = 0.55 from **Table (4.1)**: $P_0 = 0.12578$

$$\sigma_{z(1R)} = \frac{Q}{z} P_0 = \frac{100}{4}(0.12578) = 3.144.kN/m^2$$

$$\sigma_{z(1)} = \sigma_{z(1L)} + \sigma_{z(1R)} = 1.634 + 3.144 = 4.778.kN/m^2$$

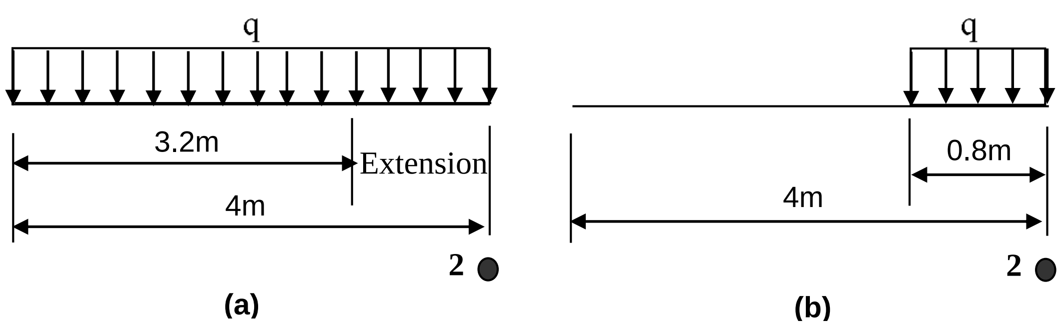


Figure (4.10): σ_z **at point (2).**

(c) σ_z at point (2):

$$\sigma_{z(2)} = \sigma_{z(2L)} - \sigma_{z(2R)}$$

From Fig. (4.10-a):

For m = x/z = 2/ 4 = 0.5 and n = y/z = 4/ 4 = 1.0 from **Table (4.1)**: $P_0 = 0.1735$

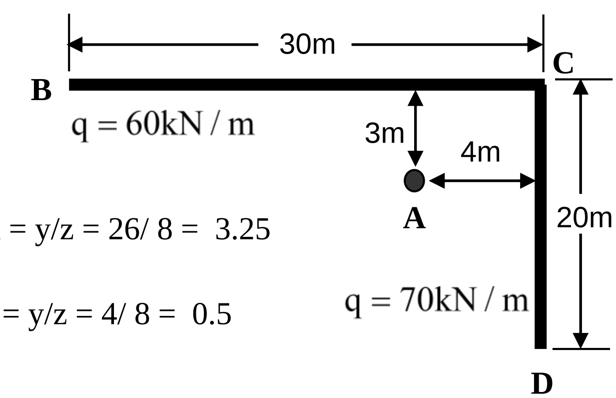
From Fig. (4.10-b):

For m = x/z = 2/ 4 = 0.5 and n = y/z = 0.8/ 4 = 0.2 from **Table (4.1)**: $P_0 = 0.0532$

$$\sigma_{z(2)} = \sigma_{z(2L)} - \sigma_{z(2R)} = \frac{100}{4}(0.1735 - 0.0532) = 3.01.kN/m^2$$

Problem (4.5): **Given:** Two walls loaded as shown in Fig..

Find: the vertical stress σ_z at z = 8m below point A?



Solution:

Wall BC:

26m: For m = x/z = 3/ 8 = 0.375 and n = y/z = 26/ 8 = 3.25

From **Table (4.1)**: $P_0 = 0.26$

4m: For m = x/z = 3/ 8 = 0.375 and n = y/z = 4/ 8 = 0.5

From **Table (4.1)**: $P_0 = 0.15$

$$\sigma_{z(BC)} = \frac{q}{z} P_0 = \frac{60}{8}(0.26 + 0.15) = 3.10.kN/m^2$$

Wall CD:

3**m:** For $m = x/z = 4/8 = 0.5$ and $n = y/z = 3/8 = 0.375$
From **Table (4.1)**: $P_0 = 0.09$

17m: For $m = x/z = 4/8 = 0.5$ and $n = y/z = 17/8 = 2.125$
From **Table (4.1)**: $P_0 = 0.20$

$$\sigma_{z(CD)} = \frac{q}{z} P_0 = \frac{70}{8}(009 + 0.20) = 2.50.kN/m^2$$

$$\sigma_{z(A)} = 3.10 + 2.50 = 5.6.kN/m^2$$

(4) UNIFORMLY LOADED STRIP AREA:

To calculate the vertical stress under uniformly loaded strip area **(see Figure 4.11)**.

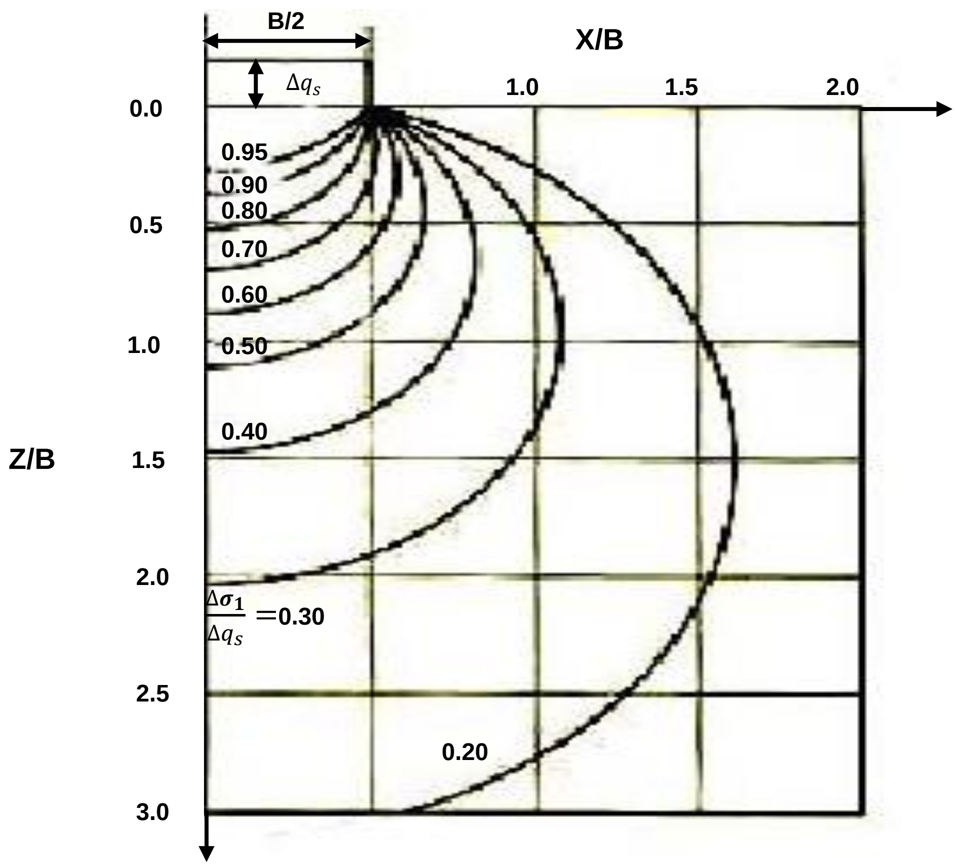


Figure (4.11): Pressure bulbs for vertical stresses under strip load.

(5) TRIANGULAR LOADED STRIP AREA:

To calculate the vertical stress under triangular loaded strip area **(see Figure 4.12)**.

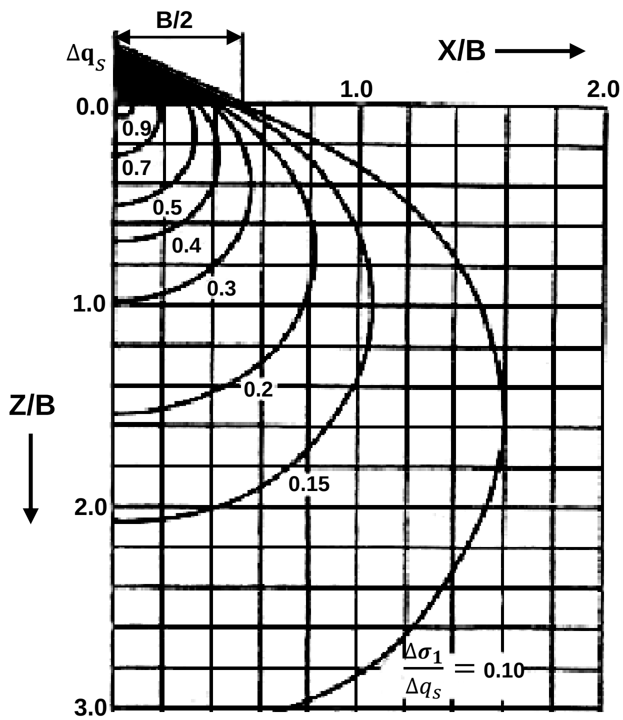
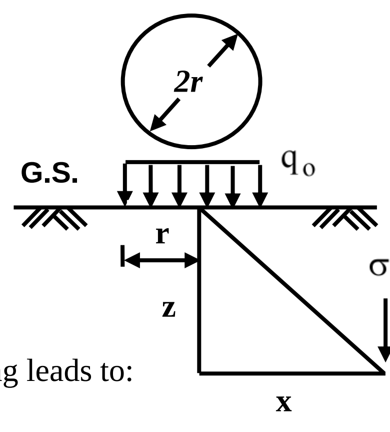


Figure (4.12): Pressure bulbs for vertical stresses under triangular strip load.

(6) UNIFORMLY LOADED CIRCULAR AREA:



$$\sigma_z = \int_{\theta=0}^{2\pi} \frac{3q}{.2\pi.z^2}\left[\frac{1}{1+(r/z)^2}\right]^{5/2} dA$$

where, $dA = \frac{1}{2} r^2 .d\theta$; which after integrating and simplifying leads to:

$$\sigma_z = \frac{I.xq_o}{100} \quad \text{...} \mathbf{(4.5)}$$

where, I = Influence factor depends on (z/r and x/r); expressed in percentage of surface contact pressure, q_o, for vertical stress under uniformly loaded circular area (**see Figure 4.13**).

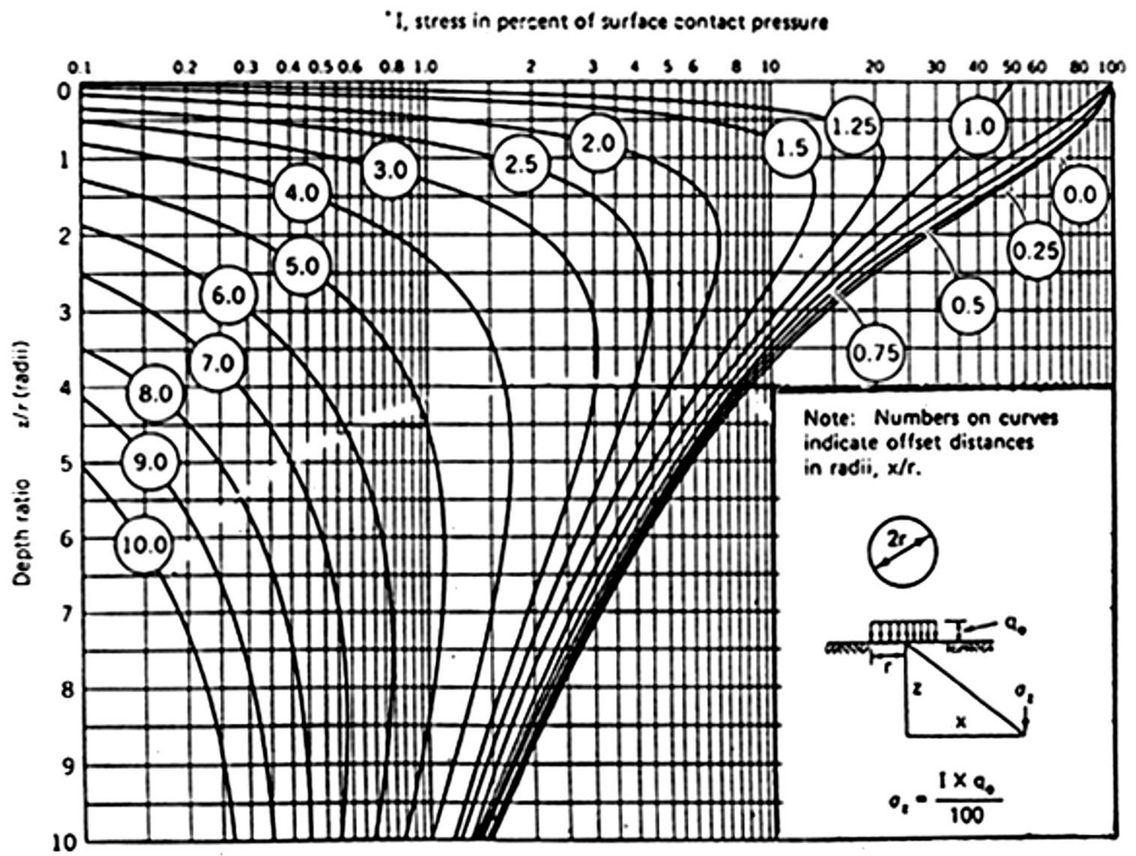


Figure (4.13): Influence values expressed in percentage of surface contact pressure for vertical stress under uniformly loaded circular area *(after Foster and Ahlvin, 1954, as cited by U.S. Navy, 1971).*

Problem (4.6):

Given: A circular area, r = 1.6m, induces a soil pressure at the surface of 100 kN/m^2.

Find: the vertical stress σ_z at:

(a) z = 2m directly under the center of the circular area.

(b) z = 2m below and 2m away from the center of the circle.

Solution:

a. For z/r = 2/1.6 = 1.25 and x/r = 0; *from* ***Fig. (4.13):*** $I = 52$

$$\sigma_z = \frac{I.q_o}{100} = \frac{52.(100)}{100} = 52 \text{ } \mathbf{kN/m^2}.$$

b. For z/r = 2/1.6 = 1.25 and x/r = 2/1.6 = 1.25; *from* ***Fig. (4.13):*** $I = 22$

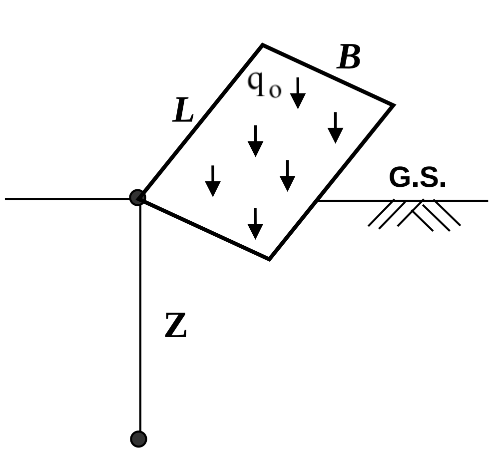
$$\sigma_z = \frac{I.q_o}{100} = \frac{22.(100)}{100} = 22 \text{ kN/m}^2.$$

(6) UNIFORMLY LOADED RECTANGULAR OR SQUARE AREA:

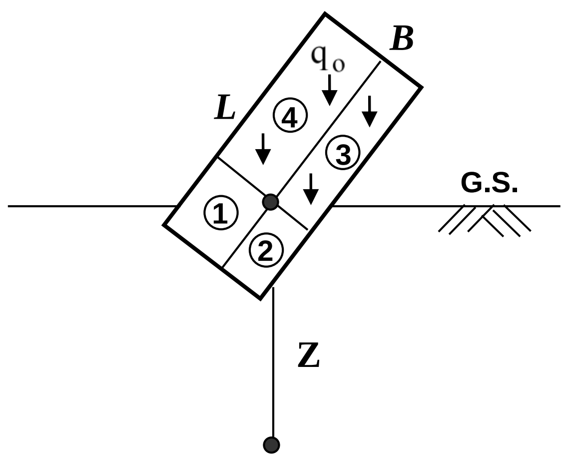
The vertical stress increase below the corner of a flexible rectangular or square loaded area is calculated as:

$$\sigma_Z = I.q_O \quad \textbf{(4.6)}$$

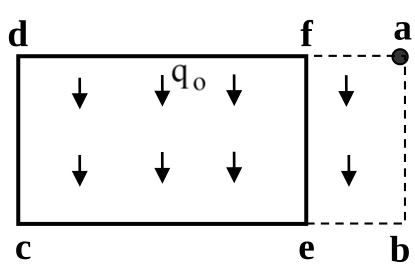
where, I = influence factor, depends on (m = B/z, and n = L/z) obtained from **(Figure 4.14).**



Example (1): $\sigma_z = I.q_o$



Example (2): $\sigma_z = q_o[I_1 + I_2 + I_3 + I_4]$



Example (3): $\sigma_{z(a)} = q_o[I_{abcd} - I_{abef}]$

Examples for Vertical stress under the corner of a uniformly loaded rectangular area.

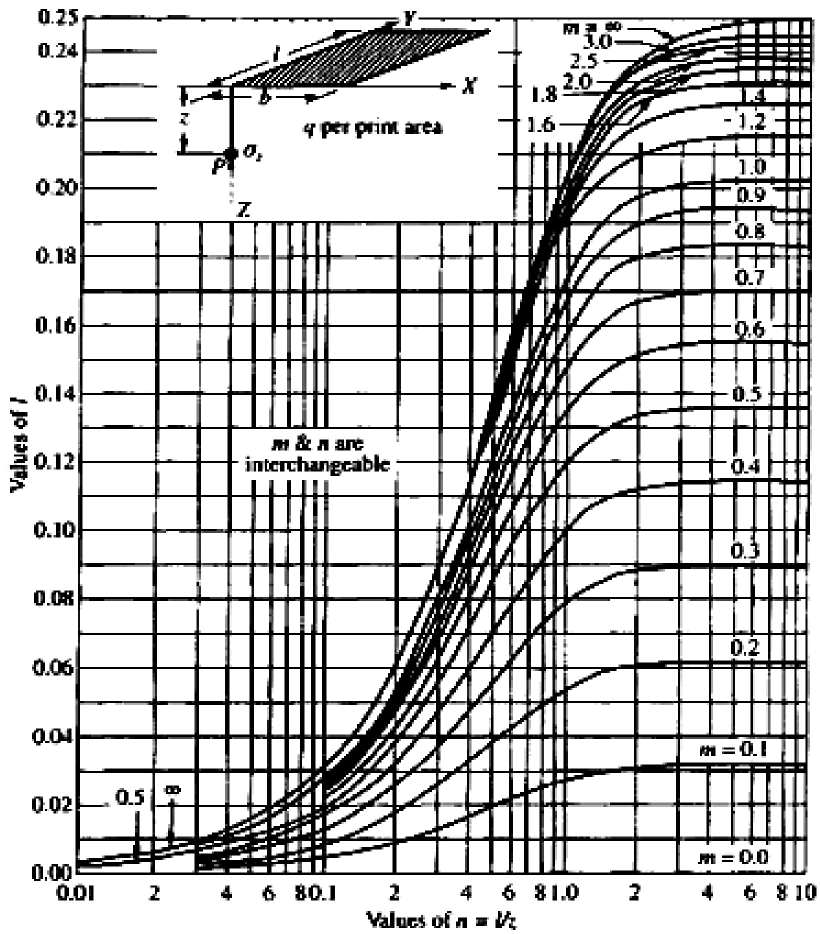
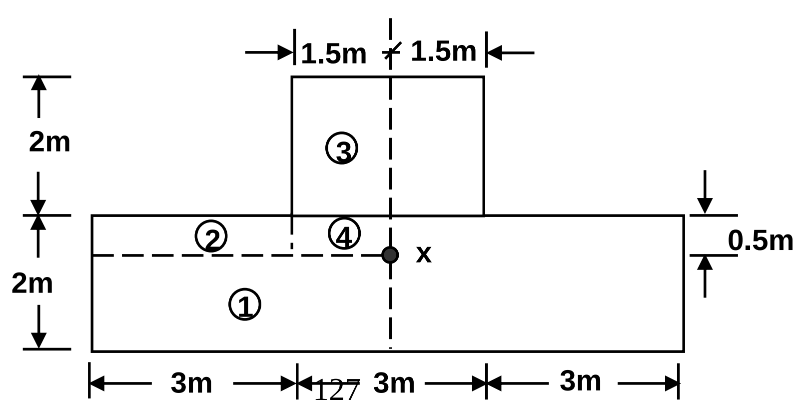


Figure (4.14): Values of I for vertical stress below the corner of a flexible rectangular area *(after Fadum, 1948).*

Problem (4.7): The plan of a foundation is given in the Fig. below. The uniform contact pressure is 40 kN/m^2. Determine the vertical stress increment due to the foundation at a depth of (5m) below the point (x).



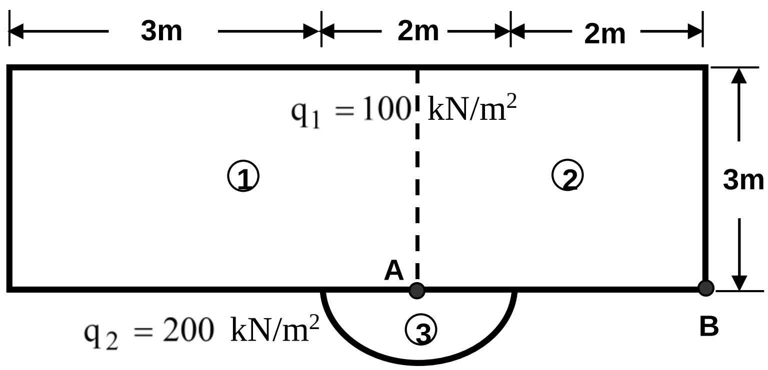
Solution:

- Using **Fig. (4.14)** the following table of results can be prepared for z = 5m

Segment	B	L	m = B/z	n = L/z	I
1	1.5	4.5	0.3	0.9	0.077
2	0.5	4.5	0.1	0.9	0.027
3	1.5	2.5	0.3	0.5	0.056
4	0.5	1.5	0.1	0.3	0.013

$\sigma_z = I.q_o = (2)(40)[\,0.077 + 0.027 + 0.056 - 0.013] = 11.76\ kN/m^2$

Problem (4.8): Determine the vertical stress increase at points (A) and (B) due to the loaded area shown in Fig. knowing that A and B points are located at depth of (5m) below the foundation level.



Solution:

(1) for point A:

- ***For half-circular area:***
  *From **Fig. (4.13):*** for z = 5m, z/r = 5/1 = 5 and x/r = 0: $\mathbf{I_1} = 5.7$
- ***For rectangular loaded area:***
  *From **Fig. (4.14):*** for z = 5m, **m =** B/z = 3/5 = 0.6 and **n** = L/z = 4/5 = 0.8: $\mathbf{I_2} = 0.125$
  for z = 5m, **m =** B/z = 3/5 = 0.6 and **n** = L/z =3/5 = 0.6: $\mathbf{I_3} = 0.107$

$\therefore\ \sigma_z = (0.5)(200)(5.7/100) + (100)(0.125 + 0.107) = 28.9\ kN/m^2$

(2) for point B:

- ***For half-circular area:***
  *From **Fig. (4.13):*** for z = 5m, z/r = 5/1 = 5 and x/r = 3/1 = 3: $\mathbf{I_1} = 2.7$
- ***For rectangular loaded area:***
  *From **Fig. (4.14):*** for z = 5m, **m =** B/z = 3/5 = 0.6 and **n** = L/z = 7/5 = 1.4: $\mathbf{I_2} = 0.147$

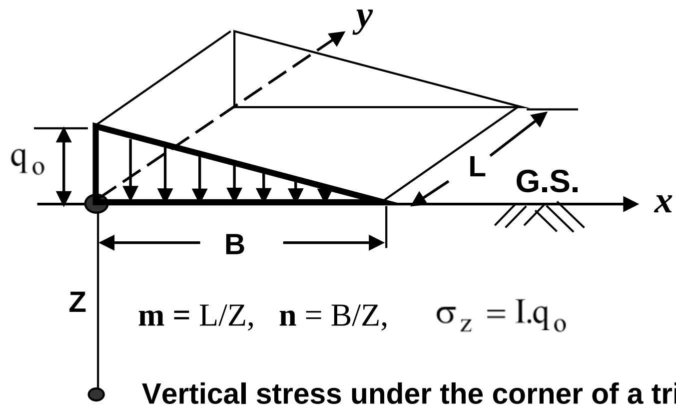
$\therefore\ \sigma_z = (0.5)(200)(2.7/100) + (100)(0.147) = 17.4\ kN/m^2$

(7) TRIANGULAR LOAD OF LIMITED LENGTH:

The vertical stress under the corners of a triangular load of limited length is calculated as:

$$\sigma_Z = I.q_O \quad \text{...(4.7)}$$

where, I = influence factor, depends on (m = L/z, and n = B/z) obtained from **(Figure 4.15).**



Vertical stress under the corner of a triangular load of limited length.

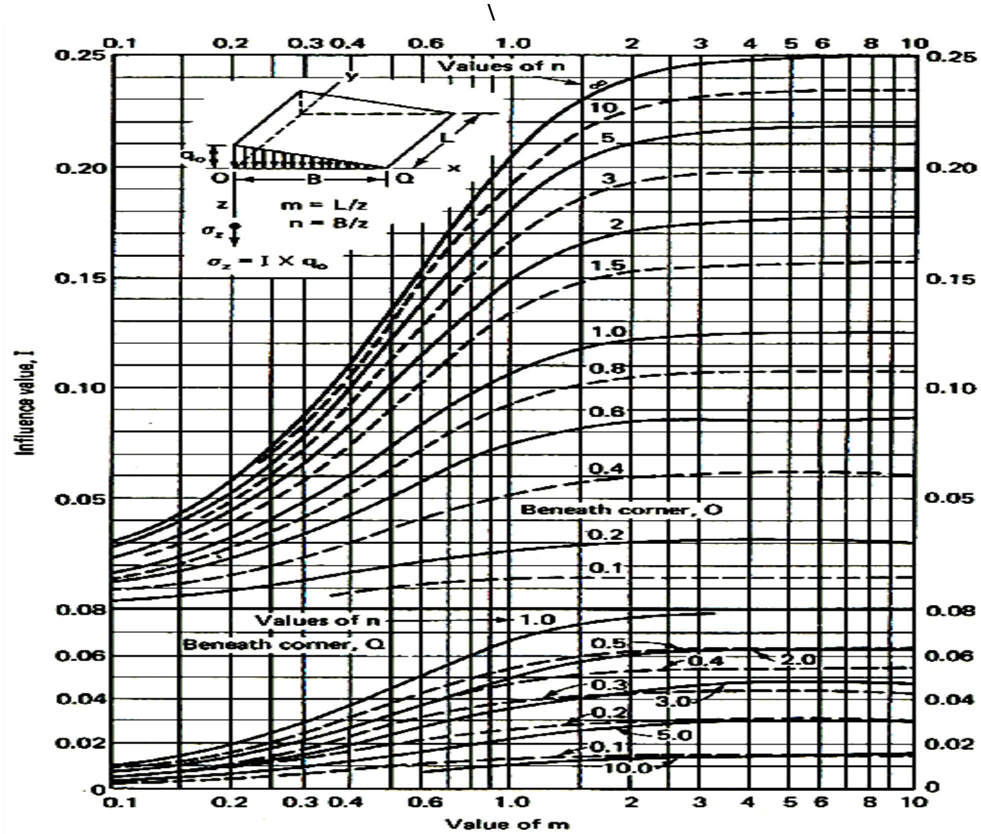


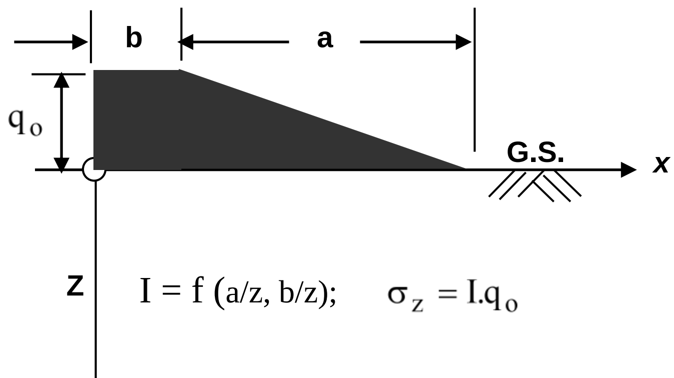
Figure (4.15): Influence values for vertical stress under the corners of a triangular load of limited length *(after U.S. Navy, 971).*

(8) EMBANKMENT LOADING:

The vertical stress under embankment loading is calculated as:

$$\sigma_z = I.q_o \quad \text{..} \quad \mathbf{(4.8)}$$

where, I = influence factor depends on (a/z, and b/z) determined from **(Figure 4.16).**



Vertical stress under embankment loading.

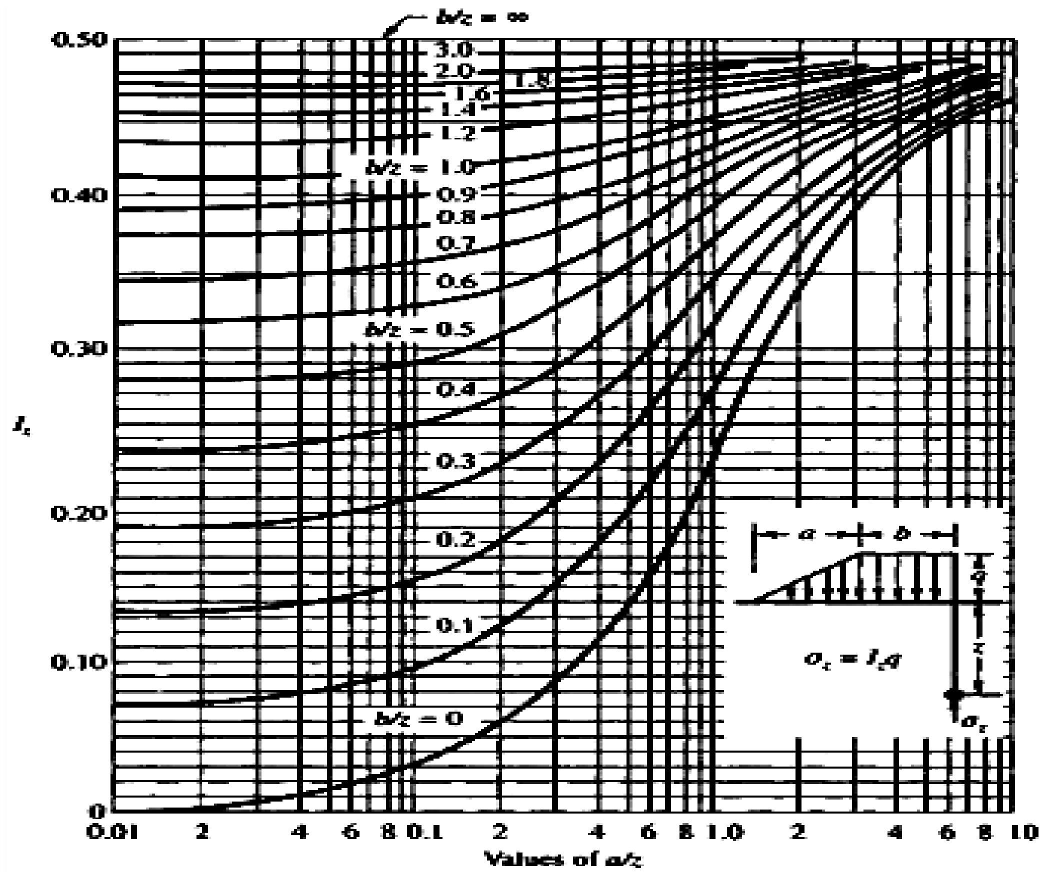
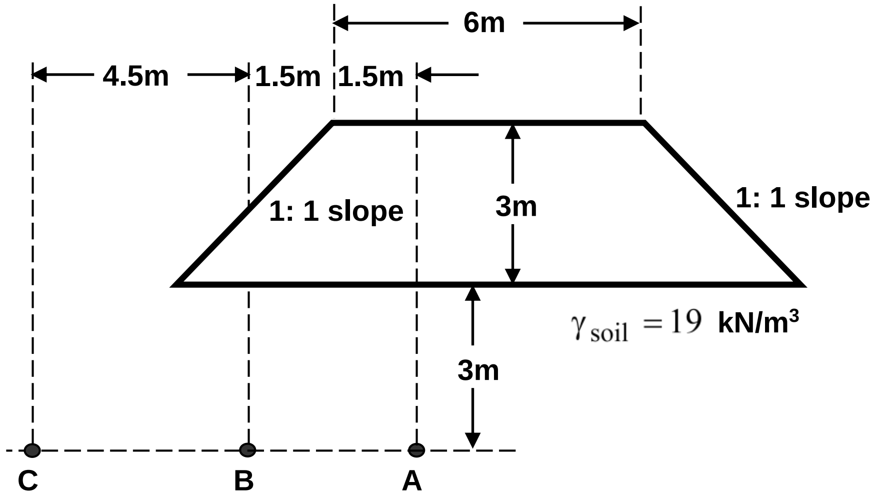


Figure (4.16): Influence factor for embankment loading *(after Osterberg, 1957).*

Problem (4.9): An embankment of (3m) high is to be constructed as shown in the figure below. If the unit weight of compacted soil is 19 kN/m^3, calculate the vertical stress due to the embankment loading at (A), (B), and (C) points.



Solution:

(1) Vertical stress at A:

From Fig. (4.17a): $\sigma_{zA} = \sigma_{z(1)} + \sigma_{z(2)}$

Left-hand section: b/z = 1.5/3 = 0.5 and a/z = 3/3 = 1.0, *from **Fig. (4.16);*** I_1 = 0.396

Right-hand section: b/z = 4.5/3 = 1.5 and a/z = 3/3 = 1.0, *from **Fig. (4.16);*** I_2 = 0.477

$\sigma_{zA} = (I_1 + I_2)q$ = [0.396 + 0.477](19)(3) = 49.761 kN/m^2

(2) Vertical stress at B:

From Fig. (4.17b): $\sigma_{zB} = \sigma_{z(1)} + \sigma_{z(2)} - \sigma_{z(3)}$

Left-hand section: b/z = 0/3 = 0 and a/z = 1.5/3 = 0.5, *from **Fig. (4.16);*** I_1 = 0.140

Middle section: b/z =7.5/3 = 2.5 and a/z = 3/3 = 1.0, *from **Fig. (4.16);*** I_2 = 0.493

Right-hand section: b/z = 0/3 = 0 and a/z = 1.5/3 = 0.5, *from **Fig. (4.16);*** I_3 = 0.140

$\sigma_{zB} = (I_1.q_1) + (I_2.q_2) - (I_3.q_3)$

= (0.14)(19)(1.5) + (0.493)(19)(3) - (0.14)(19)(1.5) = 28.101 kN/m^2

(3) Vertical stress at C:

Using Fig. (4.17c): $\sigma_{zC} = \sigma_{z(1)} - \sigma_{z(2)}$

Left-hand section, b/z = 12/3 = 4 and a/z = 3/3 = 1.0, *from **Fig. (4.16);*** I_1 = 0.498.

Right-hand section, b/z = 3/3 = 1.0 and a/z = 3/3 = 1.0, *from **Fig. (4.16);*** I_2 = 0.456.

$\sigma_{zC} = (I_1 - I_2)q$ = (0.498 - 0.456)(19)(3) = 2.394 kN/m^2

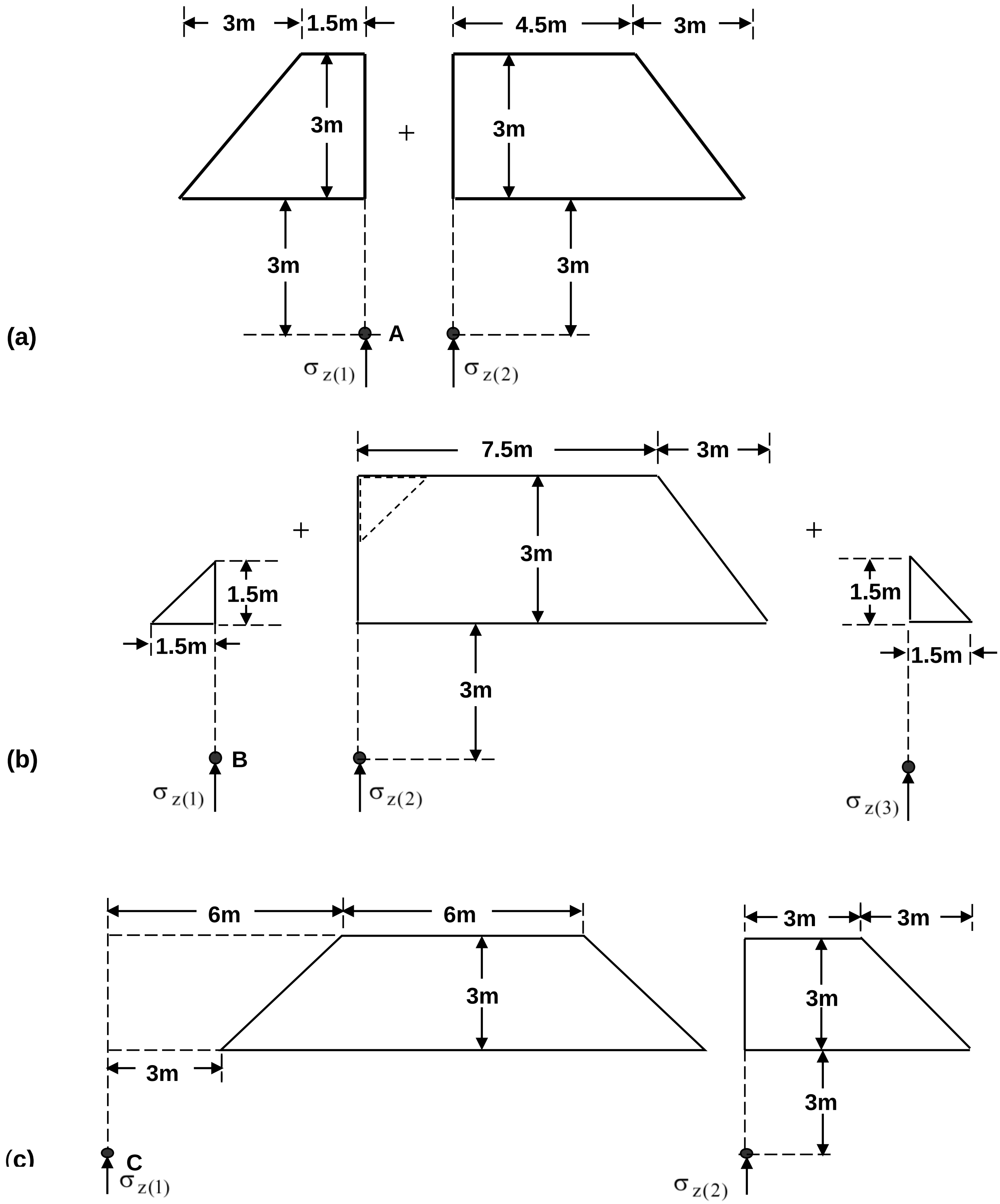


Figure (4.17): Solution of Example (4.9).

(9) ANY SHAPE LOADED AREA (NEWMARK CHART):

The stress on an elemental area dA of soil due to surface contact pressure q_o is calculated as:

$$dq = \frac{3q_o}{2\pi.z^2} \frac{1}{[1+(r/z)^2]^{5/2}} dA$$

but $dA = 2\pi.r.dr$ $\quad \therefore \quad q = \int_0^r \frac{3q_o}{2\pi.z^2} \frac{2\pi.r.dr}{[1+(r/z)^2]^{5/2}}$

or $\quad q = q_o \left\{ 1 - \frac{1}{[1+(r/z)^2]^{3/2}} \right\}$

$$(r/z) = \sqrt{(1-q/q_o)^{-2/3} - 1} \quad \textbf{.......................................(4.9)}$$

Prepare a chart on transparent paper with r_i circles as follows with $18°$ sectors:

q/q_o	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
(r/z)	0.270	0.400	0.518	0.637	0.766	0.918	1.110	1.387	1.908	∞

In this case, each circle of the chart is subdivided into 20 units, therefore the number of units for 10 circles = (20 units x 10 circles) = 200 and the influence value (I_V =1/200 = 0.005). If the scale distance (AB) is assumed = 5 cm, then:

r_i (cm)	1.35	2	2.59	3.18	3.83	4.59	5.55	6.94	9.54	∞

To estimate σ_z**:**

(1) Adopt a scale such that, the scale distance (AB) is equal to the required depth (z),

(2) Based on the scale adopted in (1), replot the plan of the loaded area,

(3) Place the plan plotted in (2) on the Newmark chart in such a way that the point (P) at which the vertical stress is required,

(4) Count the number of blocks, N of the chart which fall inside the plan, and

(5) calculate σ_z as**:**

$$\sigma_z = q.(I_V).(N) \quad \textbf{...(4.10)}$$

where, I_v = Influence value of the chart **(see Figure 4.18).**

- **Important Notes about Newmark Chart**

a. If the stress is required at different depth, then the plan is drawn again to a different scale such that the new depth z is equal to the distance (AB) on the chart.

b. The use of Newmark's chart is based on a factor termed the influence value, determined from the number of units into which the chart is subdivided. For example; **Fig.(4.18)** is subdivided into 200 units (20 units x 10 circles), therefore the influence value is (1/200 = 0.005). But if the series of rings are subdivided into 400 units, then, $I_V = 1/400 = 0.0025$.

c. In making a chart, it is necessary that the sum of units between two concentric circles multiplied by I_V be equal to the change in q/q_o of the two rings. (i.e., if the change in two rings is 0.1 q/q_o, then I_V x number of units should equal to 0.1).

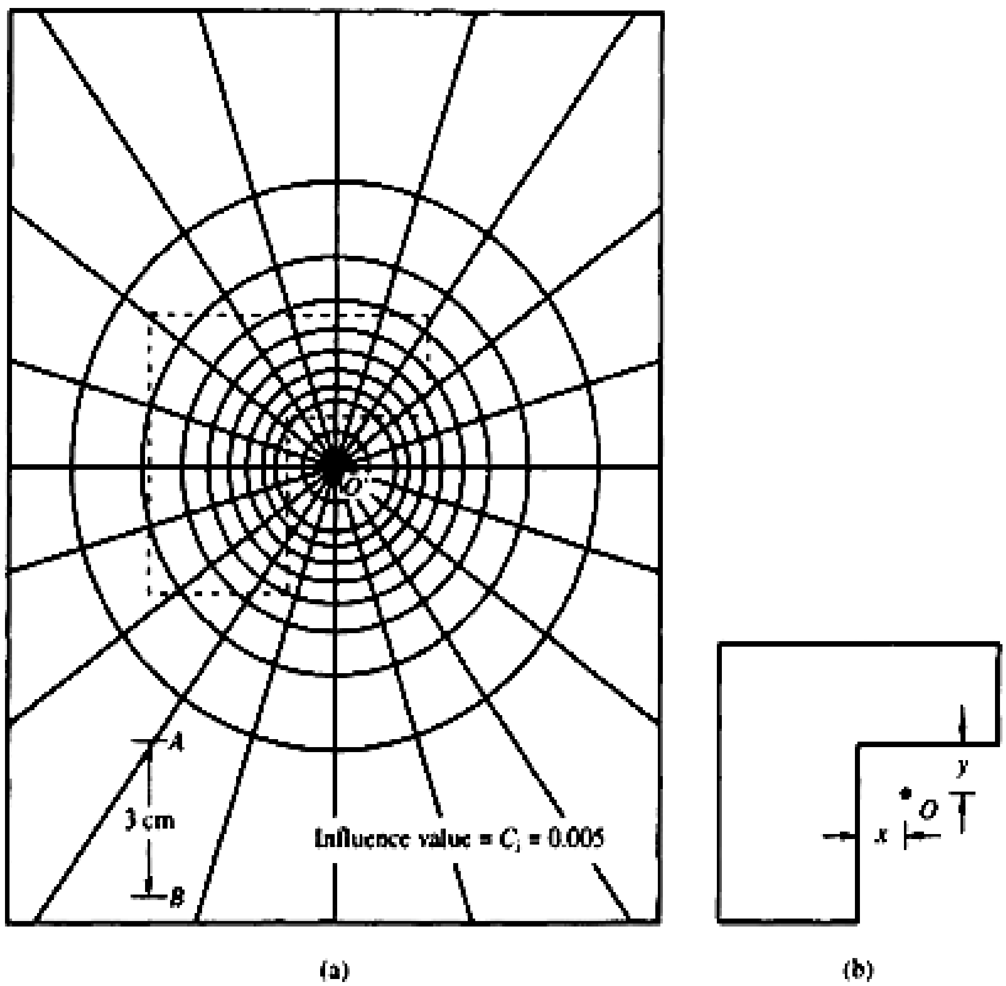
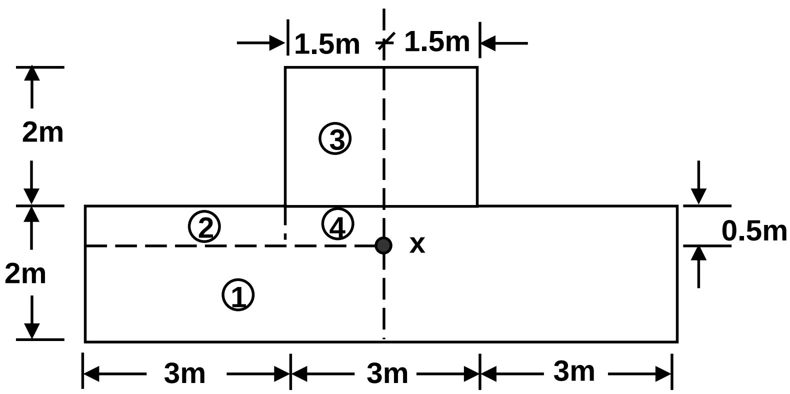


Figure (4.18): Influence chart for computation of vertical pressure *(after Newmark, 1942).*

Problem (4.10): The foundation plan shown in the figure below is subjected to a uniform contact pressure of 40 kN/m^2. Determine the vertical stress increment due to the foundation load at (5m) depth below the point (x).



Solution:

Using Fig. (4.18): N ≈ 58

$\sigma_z = q.(I_V).(N)$ = (40)(0.005)(58) = 11.6 kN/m^2