

## UNIT-IV

### SHALLOW FOUNDATION

#### Introduction

A foundation is a integral part of the structure which transfer the load of the superstructure to the soil. A foundation is that member which provides support for the structure and it's loads. It includes the soil and rock of earth's crust and any special part of structure that serves to transmit the load into the rock or soil.

The different types of the foundations are given in fig.

#### Different types of footings

#### Foundations

##### 1. Shallow foundations ( $D_f < B$ )

##### A) Footing

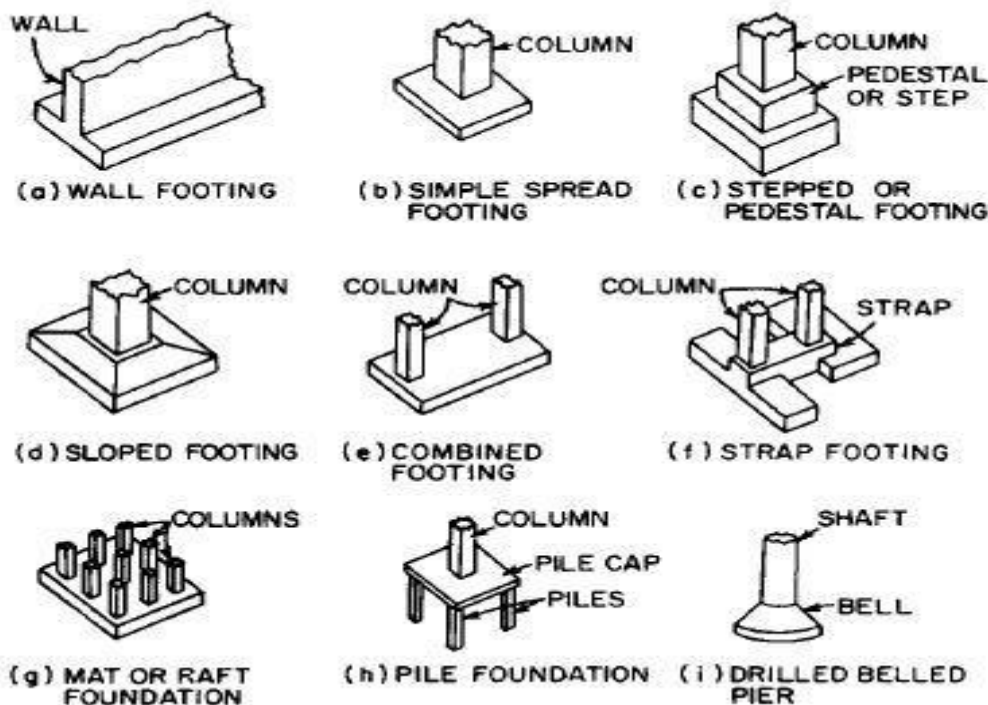
i) Spread      ii) Strap      iii) Combined

##### B) Raft Footing

i) Conventional ii) Buoyancy

##### 2. Deep foundations ( $D_f > B$ )

a) Deep footing      b) Pile    c) Pier    d) Caissons Well Foundation



## Shallow Foundation

Shallow foundation are those foundations in which the depth at which the foundation is placed is less than the width of the foundation ( $D < B$ ). Shallow foundations are generally termed as spread footing as they transmit the load of the super structure laterally into the ground.

### Classification of Shallow Foundation:

On the basis of design, the shallow foundations are classified as:

- Wall Footing
- Isolated column or Column Footing
- Combined Footing
- Cantilever (Strap) Footing
- Mat (Raft) Foundation

### Wall Footing

This type of foundation runs continuous along the direction of the wall and helps to transmit the load of the wall into the ground. Wall footings are suitable where loads to be transmitted are small and are economical in dense sands and gravels. In this type of foundation the width is 2-3 times the width of the wall at ground level. Wall footing may be constructed through stone, brick, plain or reinforced cement concrete.

### Spread Footing:

It is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped to spread the load over a larger area. When footing is provided to support an individual column, it is called “isolated footing”.

### Strap Footing

When an edge footing cannot be extended beyond the property line the edge footing is linked up with the other interior footing by means of a strap beam. Such footings are called as strap footing. It is also known as cantilever footing.

### Column Footing

Column footing are suitable and economical for the depth greater than 1.5m. In this type of foundation the base of the column is enlarged. Column footing is in the form of flat slab and may be constructed through plain or reinforced concrete.

### Combined Footing

Combined footings are those foundations that are made common for two or more columns in a row. It is used when the footing for a column may extend beyond the property line. It is also suitable when the two columns are closely spaced and the soil on which the structure rests is of low bearing capacity. It may be rectangular or trapezoidal in shape.

### Mat Foundation

A mat foundation is a combined footing which covers the entire area beneath of a structure and supports all the walls and columns. It is also known as raft foundation. Mat foundation is applicable when:

- Allowable bearing pressure is low.
- The structure is heavy.
- The site is with highly compressible layer.

The mat foundation can be further classified into following types:

- Flat slab type.
- Flat Slab thickened under column.
- Two way beam and slab type.
- Flat slab with pedestals.
- Rigid frame mat.
- Piled mat

### Deep Foundation

Deep Foundation are those foundations in which the depth of the foundation is greater than its width ( $D > B$ ). The  $D/B$  ratio is usually 4-5 for deep foundation. Unlike shallow foundation, the deep foundation transmits the load of the superstructure vertically to the rock strata lying deep. Deep foundations are used when the shallow foundation cannot support the load of the structure.

### Classification of Deep Foundation

The mat foundation can be further classified into following types:

- Pile Foundation
- Pier Foundation
- Well (Caissons) Foundation

### Pile Foundation

Pile is a slender member with small area of cross-section relative to its length. They can transfer load either by friction or by bearing. Pile foundations are used when:

- The load is to be transferred to stronger or less compressible stratum, preferably rock.
- The granular soils need to be compacted.
- The horizontal and the inclined forces need to be carried from the bridge abutments and the retaining walls.

### Classification of Pile Foundation

The pile foundation can be further classified into following types on various basis such as function, material, method of installation which are listed below:

### **Based on Function:**

- Bearing piles
- Friction piles
- Combined piles (Both bearing and friction)

### **Based on Material:**

- Timber piles
- Concrete piles
- Steel piles

### **Based on Method of Installation:**

- Large displacement piles
- Small displacement piles
- Non-displacement piles

### **Pier Foundation**

Pier foundations are underground cylindrical structural member that support heavier load of the structure which shallow foundations cannot resist. Unlike pile foundation, pier foundation can only transfer load by bearing. Pier foundations are shallower in depth than the pile foundation. Pier foundations are used when:

- The top stratum is a decomposed rock underlying as sound rock strata.
- The soil is stiff clay that occurs large resistance for driving the bearing pile.

### **Well (Caissons) Foundation**

The term caisson refers to box or a case. These are hollow inside and are usually constructed at the site and sunk in place into hard bearing strata. As they are expensive in construction, they are usually restricted to major foundation works. Well foundations are suitable when the soil contains large boulders obstructing the penetration during installation of pier or pile foundations. Caissons are used for bridge piers, abutments in rivers and lakes and other shore protection works. They are used to resist heavy vertical and horizontal loads and are used in the construction of large water front structures as pump houses.

#### **Classification of Well Foundation**

- Open Caissons
- Pneumatic Caissons
- Box Caissons

### **Factors affecting the selection of Foundation:**

#### **On the basis of ground/soil condition**

- Shallow foundations are preferred where soil close to the surface is capable of supporting structure loads.
- Where the ground close to the surface is not capable of supporting structural loads, hard strata is searched for and deep foundation is required.
- Uniform stable ground requires relatively shallow foundation whereas filled up ground has low bearing capacity thus requires deep foundation.

### On the basis of Loads from Building:

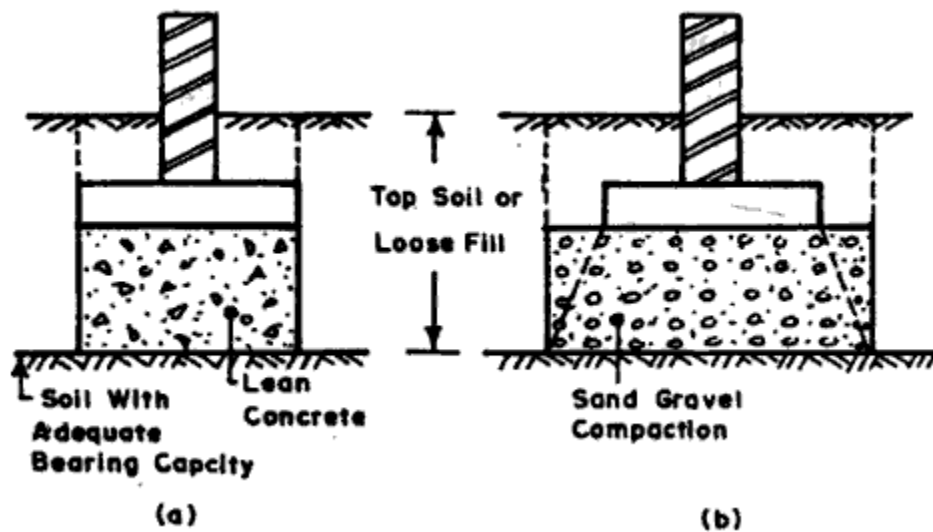
- In the case of low-rise building in a larger area, the extent of loading is relatively low, so shallow foundation can resist the load from the structures.
- In the case of the high-rise building built within less area have high loads. Therefore, the deep foundation is required as shallow foundation may not be able to resist such loads of greater intensity.

### Location of Depth:

The footings should be carried below the top (organic) soli, miscellaneous fill, abandoned foundation, debris or muck. If the top loose soil or fill is too deep, two alternatives may be used depending upon the relative economy and the time available:

- (i) Removing the top soil directly below the footing and replacing it with lean concrete [Fig. (a)]
- (ii) Removing the top soil in an area larger than the footing and replacing it with compacted sand and gravel fill. The area of the compacted sand and gravel fill should be sufficiently large to distribute the footing load, as shown in Fig. (b).

In either case, it is essential to reach the level of the strata which has the required bearing capacity adopted for the design of footing.



Sometimes, the top soil may be good and compact, and may have adequate bearing capacity. In that case it is desirable to keep the minimum depth of foundation given by Rankines formula: (Eq. )

$$D_{min} = \frac{q}{\gamma} \left[ \frac{1 - \sin \phi}{1 + \sin \phi} \right]^2$$

Where  $\phi$  is the angle of repose, the values of which may be taken from Table

**$q$  = Intensity of load at the base of footing  
( $\text{t/m}^2$  or  $\text{kN/m}^2$ )**

**$\gamma$  = Unit weight of soil ( $\text{t/m}^3$  or  $\text{kN/m}^3$ ).**

**Note:** It is to be noted that  $q$  is the actual load intensity and not the safe bearing capacity of soil. Sometimes, the actual load intensity may be less than the safe bearing capacity of soil, requiring lesser minimum depth. When footings are supported on very stiff soil, having very high safe bearing capacity, the minimum depth of foundation computed on the basis of safe bearing capacity would come out to be very large which is ridiculous. In such soils, the width of the footing (found from other considerations) would be larger than the one required from the bearing capacity considerations, thus giving rise to actual soil pressure lesser than the safe bearing capacity.

**TABLE . Values Of Unit Weight And Angle Of Repose**

Type of soil	Unit weight ( $\text{kg/m}^3$ )	Angle of Repose
1. Sand (dry)	1500 to 1650	25° to 35°
2. Sand (Damp)	1700 to 1850	30° to 40°
3. Sand (Wet)	1800 to 1900	15° to 30°
4. Sand (Dry and compact)	1700 to 1850	35° to 45°
5. Vegetable earth (Dry)	1600 to 1700	20° to 30°
6. Vegetable earth (Damp)	1650 to 1750	40° to 45°
7. Vegetable earth (Wet)	1700 to 1800	15° to 20°
8. Vegetable earth (Dry and consolidated)	1800	45°
9. Gravel	1700 to 1800	40° to 45°
10. Sand-Gravel mix	1800 to 1900	25° to 35°
11. Clay (Dry)	1700 to 1750	30°
12. Clay (Damp)	1750 to 1850	35° to 40°
13. Clay (Wet)	1850 to 1900	15°
14. Mud	1600 to 1850	zero
15. Ashes	600 to 800	40°

The depth of footing should also be such that the rate or angle of spread of the load from the wall base to the outer edge of the ground bearing does not exceed the permissible value, as envisaged in **Fig.**

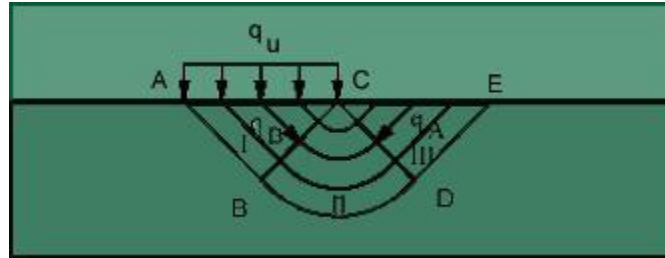
**Methods of determining bearing capacity** The various methods of computing the bearing capacity can be listed as follows:

- Presumptive Analysis
- Analytical Methods
- Plate Bearing Test

- Penetration Test
- Modern Testing Methods
- Centrifuge Test **Prandtl's Analysis**

**Prandtl (1920)** has shown that if the continuous smooth footing rests on the surface of a weightless soil possessing cohesion and friction, the loaded soil fails as shown in figure by plastic flow along the composite surface. The analysis is based on the assumption that a strip footing placed on the ground surface sinks vertically downwards into the soil at failure like a punch.

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**Prandtl's Analysis**

- Zone I is an active failure zone
- Zone II is a radial shear zone
- Zone III is a passive failure zone identical for  $\Phi=0$

Zone I consist of a triangular zone and its boundaries rise at an angle  $(45-\Phi/2)$  with the horizontal two zones on either side represent passive Rankine zones. The boundaries of the passive Rankine zone rise at angle of  $(45+\Phi/2)$  with the horizontal. Zones II located between I and III are the radial shear zones. The bearing capacity is given by (Prandtl 1921) as

$$q_d = CN_c$$

where  $c$  is the cohesion and  $N_c$  is the bearing capacity factor given by the expression

$$N_c = \cot\Phi [e^{2\tan\Phi} \tan^2[(45+\Phi/2)] - 1]$$

Reissner (1924) extended Prandtl's analysis is  $N_c$  for uniform load  $q$  per unit area acting on the ground surface.

He assumed that the shear pattern is unaltered and gave the bearing capacity expression as follows.

$$q_d = CN_c + qN_q$$

$$N_q = e^{2\tan\Phi} \tan^2[(45+\Phi/2)]$$

$$N_c = \cot\Phi [e^{2\tan\Phi} \tan^2[(45+\Phi/2)] - 1]$$

if  $\Phi=0$ , the logspiral becomes a circle and  $N_c$  is equal to  $(\pi+2)$ , also  $N_q$  becomes 1. Hence the bearing capacity of such footings becomes

$$\begin{aligned} q_d &= (\pi+2)c + q \\ &= 5.14c + q \end{aligned}$$

if  $q=0$ ,

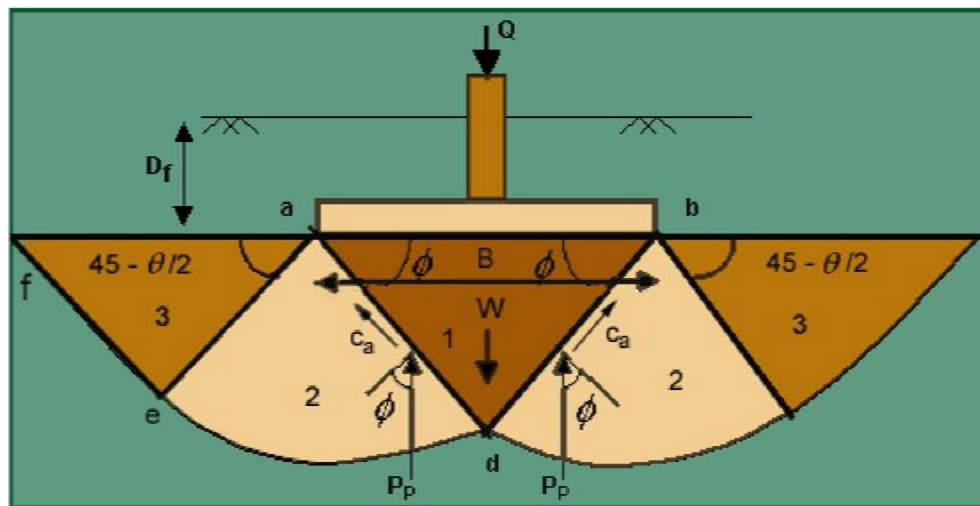
we get ,  $q_d = 5.14c = 2.57q_u$

Where  $q_u$  is the unconfined compressive strength

### Terzaghi's Bearing Capacity Theory

#### Assumptions in Terzaghi's Bearing Capacity Theory:

- Depth of foundation is less than or equal to its width.
- Base of the footing is rough.
- Soil above bottom of foundation has no shear strength; is only a surcharge load against the overturning load
- Surcharge up to the base of footing is considered.
- Load applied is vertical and non-eccentric.
- The soil is homogenous and isotropic.
- L/B ratio is infinite.



#### Terzaghi's Bearing Capacity Theory

Consider a footing of width  $B$  and depth  $D_f$  loaded with  $Q$  and resting on a soil of unit weight  $\gamma$ . The failure of the zones is divided into three zones as shown below. The zone 1 represents an active Rankine zone, and the zones 3 are passive zones. The boundaries of the active Rankine zone rise at an angle of  $(45 - \phi/2)$ , and those of the passive zones at  $(45 + \phi/2)$ , with the horizontal. The zones 2 are known as zones of radial shear, because the lines that constitute one set in the shear pattern in these zones radiate from the outer edge of the base of the footing. Since the base of the footings is rough, the soil located between it and the two surfaces of sliding remains in a state of equilibrium and acts as



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if it formed part of the footing. The surfaces ad and bd rise at  $\Phi$  to the horizontal. At the instant of failure, the pressure on each of the surfaces ad and bd is equal to the resultant of the passive earth pressure  $P_p$  and the cohesion

$$Q_d = 2P_p + 2C_a \sin \Phi = 2P_p + B \tan \Phi \text{----- (1)}$$

The passive pressure required to produce a slip on def can be divided into two parts,  $P'_p$  and  $P''_p$ . The force  $P'_p$  represents the resistance due to weight of the mass adef. The point of application of  $P'_p$  is located at the lower third point of ad. The force  $P''_p$  acts at the midpoint of contact surface ad.

The value of the bearing capacity may be calculated as :

$$Q_d = 2(P_p + P_c + P_q + \frac{1}{2} B c \tan \phi) \text{----- (2)}$$

by introducing into eqn(2) the following values:

$$N_c = \frac{2P_c}{Bc} + \tan \phi$$

$$N_q = \frac{2P_q}{B \cdot D_f}$$

Footing subjected to Concentric loading Problem 1 Shallow footing subjected to vertical load along with moment. Design a column footing to carry a vertical load of 40 t (DL+LL) and moment of 1000 Kg-m.

$$q_{allow} = 20 \text{ t/m}^2, f_y = 415 \text{ N/mm}^2, f_{ck} = 15 \text{ N/mm}^2$$

### Skempton's Theory for Cohesive Soil

Skempton 1951 suggested a bearing capacity theory for saturated clay for which  $\epsilon_s = 0$ . Skempton gives  $N_c$ , the bearing capacity factor on the basis of theory, laboratory tests and field observations. It was found that the value of  $N_c$  increased with the increase in  $D_f/B$  ratio. The expression for  $N_c$  proposed by Skempton is given below.

For Strip footings,

$$N_c = 5(1+0.2D_f/B), \text{ with a maximum limiting value of } 7.5 \text{ ---- (1)}$$

For square and circular footings,

$$N_c = 6(1+0.2D_f/B), \text{ with a maximum limiting value of } 9.0 \text{ ---- (2)}$$

For rectangular footings,

$$7.5(1+0.2D_f/B) (1+0.2B/L) \text{ for } D_f/B > 2.5 \text{ ---- (4)}$$

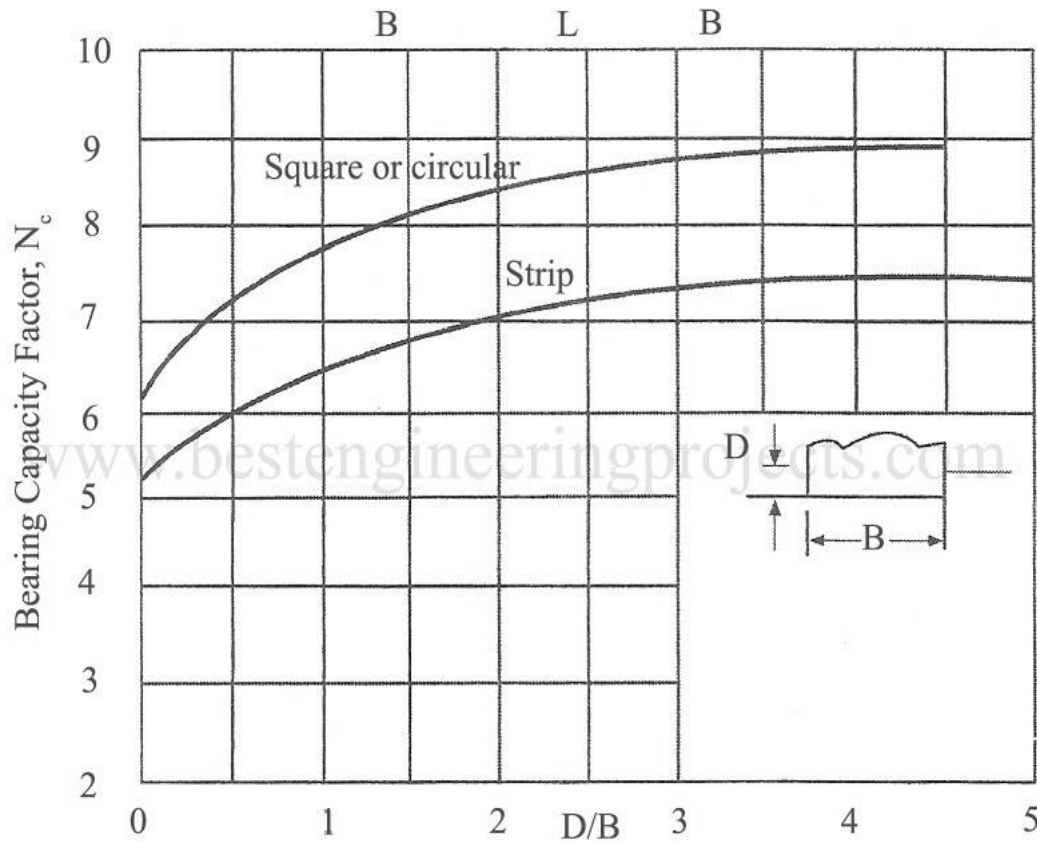


Fig 1 Skempton's Bearing Capacity Factor  $N_c$

For  $\bar{E}_s = 0$  condition, the net ultimate bearing capacity is given by:

$$Q_{net} = c_u N_c \quad \text{--- (5)}$$

Alternatively, the graph given in Fig.1 above can be used to find the bearing capacity factor  $N_c$ . Where,  $N_c(\text{rect}) = N_c(\text{sq})[0.84 + 0.16B/L]$ . As per Skempton, if the shear strength of the soil does not vary more than  $\pm 50\%$  of the average value for a depth  $2/3B$  below the footing, the average value of  $c_u$  can be used in the above equation.

## Terzaghi's Theory on Bearing Capacity Analysis

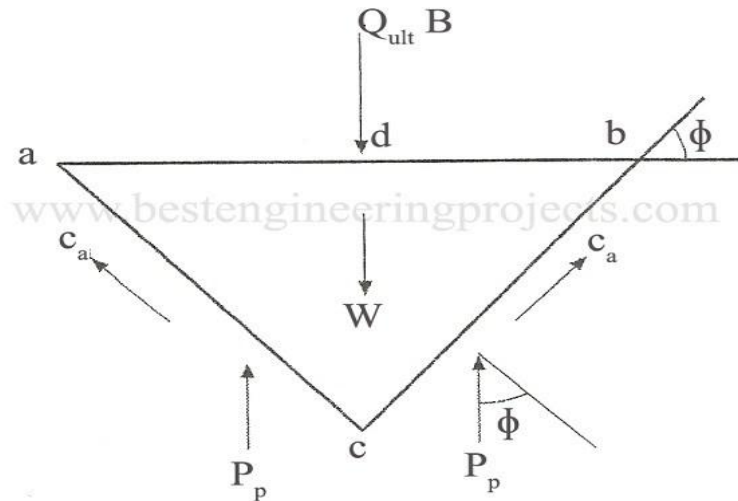
Terzaghi in 1943 gave a general bearing capacity theory for a strip foundation. For the first time, he developed his theory by incorporating weight of failure wedge in the analysis.



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The boundaries of passive Rankine's Zone III are inclined at angles  $(45 - \phi/2)$  with the horizontal.

The ultimate bearing capacity  $Q_{ult}$  is obtained by considering the equilibrium of the elastic wedge in Zone I as shown in Fig.2. The Various forces acting on the Wedge are.



**Fig 2 Equilibrium of Elastic Wedge**

$P_p$  = The resultant passive earth resistance at failure offered by the wedge bcfgb

$C_a$  = Soil adhesion acting along ac and bc

$\phi$  = Angle of internal friction

$Q_{ult}$  = Ultimate bearing capacity

For equilibrium,

$$Q_{ult} - B\gamma - 1 = 2P_p + 2c_a \tan \phi - ac \sin \phi - W$$

$$= 2P_p + 2c_a \tan \phi - (B/2 \cos \phi) \sin \phi - B/2 \gamma \tan \phi$$

$$= 2P_p + c_a B \tan \phi - 1/4 B \gamma \tan \phi$$

$$Q_{ult} = 2P_p/B + c_a \tan \phi - 1/4 \gamma \tan \phi \quad \text{--- (1)}$$

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The passive force  $P_p$  is determined by considering the equilibrium of the passive wedge bcdeb. This is obtained by carrying out the three independent operations. The operations comprise:

1. Soil is assumed to have weight and possess no cohesion and surcharge.
2. Soil is assumed to have surcharge only and possess no cohesion and weight.
3. Soil is assumed to have cohesion only and possess no weight and surcharge.

with the above assumptions the passive resistance  $P_{p\gamma}$ ,  $P_{pq}$  and  $P_{pc}$  respectively due to weight, surcharge and cohesion are determined and the values are superimposed. Hence,  $P_p$  can be written as:

$$P_p = (P_{p\gamma}) + (P_{pq}) + (P_{pc}) \quad \text{--- (2)}$$

Substituting the value of  $P_p$  in equation (1) we get,

$$Q_{ult} = \frac{2[(P_{p\gamma}) + (P_{pq}) + (P_{pc})]}{B} + c \tan \phi - \frac{1}{4} B \gamma \tan \phi \quad \text{--- (3)}$$

$$Q_{ult} = \left[ \frac{2(P_{p\gamma})}{B} - \frac{1}{4} B \gamma \tan \phi \right] + \frac{2(P_{pq})}{B} + \frac{2(P_{pc})}{B} + c \tan \phi \quad \text{--- (4)}$$

$$Q_{ult} = \frac{1}{4} B \gamma N_\gamma + q N_q + c N_c$$

$$N_\gamma = \frac{4(P_{p\gamma})}{\gamma B^2} - \frac{1}{2} \tan \phi$$

$$N_q = \frac{2(P_{pq})}{qB}$$

$$N_c = \frac{2(P_{pc})}{B} + \tan \phi$$

The factors  $N_\gamma$ ,  $N_q$  and  $N_c$  are called bearing capacity factors. These factors are functions of angle of internal friction  $\phi$ . The curve presented in Fig.3 below gives the bearing capacity factors for various values of the angle of internal friction  $\phi$ .

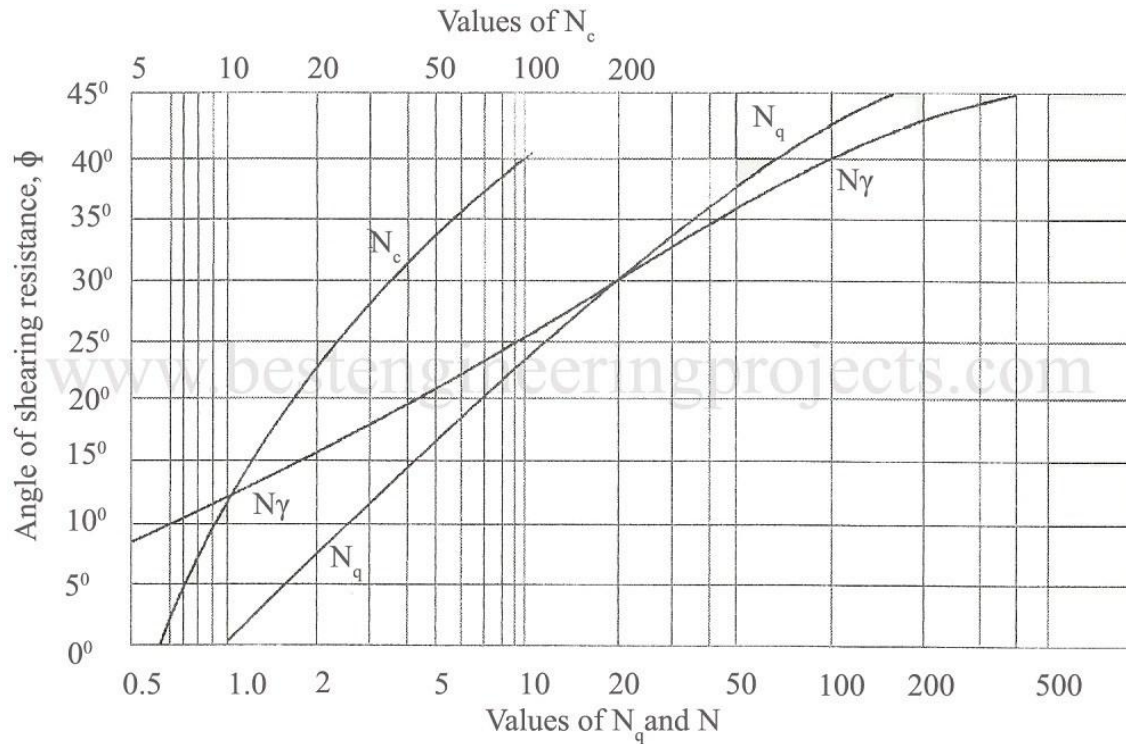


Fig 3 Terzaghi Bearing Capacity Factors

**Safe Bearing Capacity** – The safe bearing capacity is obtained as per the followings. Let  $Q_{net}$  be the net bearing capacity. The net bearing capacity, as per the definition is obtained as:

$$Q_{net} = Q_{ult} - q = Q_{ult} - \hat{\sigma}_3 D_f \quad \text{--- (9)}$$

$$Q_{net} = \frac{1}{2} \hat{\sigma}_3 B N_{\gamma} + q(N_q - 1) + c N_c$$

$$Q_{net} = \frac{1}{2} \hat{\sigma}_3 B N_{\gamma} + \hat{\sigma}_3 D_f (N_q - 1) + c N_c \quad \text{--- (10)}$$

$$Q_s = Q_{net}/F + \hat{\sigma}_3 D_f = \left[ \frac{1}{2} \hat{\sigma}_3 B N_{\gamma} + \hat{\sigma}_3 D_f (N_q - 1) + c N_c \right] + \hat{\sigma}_3 D_f \quad \text{--- (11)}$$

The limitation of the Terzaghi's theory is that it is applicable only for the shallow foundation. The theory has been derived for the case of general shear failure. For local shear failure the following modification has been proposed by Terzaghi.

$$C_m = \frac{2}{3} c \quad \text{--- (12)}$$

$$\hat{\sigma}_{3,m} = \frac{2}{3} \tan \hat{\sigma}_3 \quad \text{--- (13)}$$

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The reduction in shear parameters is due to the shear strength not being fully mobilized. The bearing capacity factors for use in general equation of Terzaghi should be based on the values of  $\hat{c}, \hat{m}$ .

Bearing capacity for footing having limited dimension – The equation developed by Terzaghi is for strip foundation, which is considered as two dimensional. The case of footings with finite dimensions is considered as three dimensional problem. Based on experimental results Terzaghi suggested following modification for other footings such as square, circular, rectangular etc.

### Square Footing

$$Q_{ult} = 1.2cN_c + \hat{\gamma}^3 D_f N_q + 0.4\hat{\gamma}^3 B N_{\gamma}^3 \quad \text{--- (14)}$$

### Circular Footing

$$Q_{ult} = 1.2cN_c + \hat{\gamma}^3 D_f N_q + 0.3\hat{\gamma}^3 B N_{\gamma}^3 \quad \text{--- (15)}$$

### Rectangular Footing

$$Q_{ult} = cN_c [1+0.2B/L] + \hat{\gamma}^3 D_f N_q + 0.5\hat{\gamma}^3 B N_{\gamma}^3 [1-0.2B/L] \quad \text{--- (16)}$$

### Meyerhof's general bearing capacity equations

Vertical load:

$$Q_u = c N_c S_c D_c + \gamma D N_q S_q D_q + 0.5 \gamma B N_{\gamma} S_{\gamma} D_{\gamma} \quad \text{--- [1]}$$

Inclined load:

$$Q_u = c N_c S_c D_c I_c + \gamma D N_q S_q D_q I_q + 0.5 \gamma B N_{\gamma} S_{\gamma} D_{\gamma} I_{\gamma} \quad \text{--- [2]}$$

Where:

$N_c, N_q, N_{\gamma}$ : Meyerhof's bearing capacity factors depend on soil friction angle,  $\phi$ .

$$N_c = \cot \phi (N_q - 1) \quad \text{--- [3]}$$

$$N_q = e^{\pi \tan \phi} \tan^2(45 + \phi/2) \quad \text{--- [4]}$$

$$N_{\gamma} = (N_q - 1) \tan(1.4\phi) \quad \text{--- [5]}$$

$S_c, S_q, S_{\gamma}$ : shape factors

$D_c, D_q, D_{\gamma}$ : depth factors

$I_c, I_q, I_{\gamma}$ : incline load factors

Friction angle	Shape factor	Depth factor	Incline load factors
Any $\phi$	$S_c = 1 + 0.2 K_p (B/L)$	$D_c = 1 + 0.2 \sqrt{K_p} (B/L)$	$I_c = I_q = (1 - \theta/90^\circ)^2$
$\phi = 0$	$S_q = S_{\gamma} = 1$	$D_q = D_{\gamma} = 1$	$I_{\gamma} = 1$
$\phi \geq 10^\circ$	$S_q = S_{\gamma} = 1 + 0.1 K_p (B/L)$	$D_q = D_{\gamma} = 1 + 0.1 \sqrt{K_p} (D/B)$	$I_{\gamma} = (1 - \theta/\phi)^2$

C: Cohesion of soil

$\gamma$  : unit weight of soil

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D: depth of footing

B, L: width and length of footing

$K_{pr} = \tan^2(45 + \phi/2)$ , passive pressure coefficient.

$\theta$  = angle of axial load to vertical axis

**Table 2: Meyerhof's bearing capacity factors**

$\phi$	$N_c$	$N_q$	$N_r$
0	5.1	1	0
5	6.5	1.6	0.1
10	8.3	2.5	0.4
15	11	3.9	1.2
20	14.9	6.4	2.9
25	20.7	10.7	6.8
30	30.1	18.4	15.1
35	46.4	33.5	34.4
40	75.3	64.1	79.4

**Meyerhof's bearing capacity factors**

