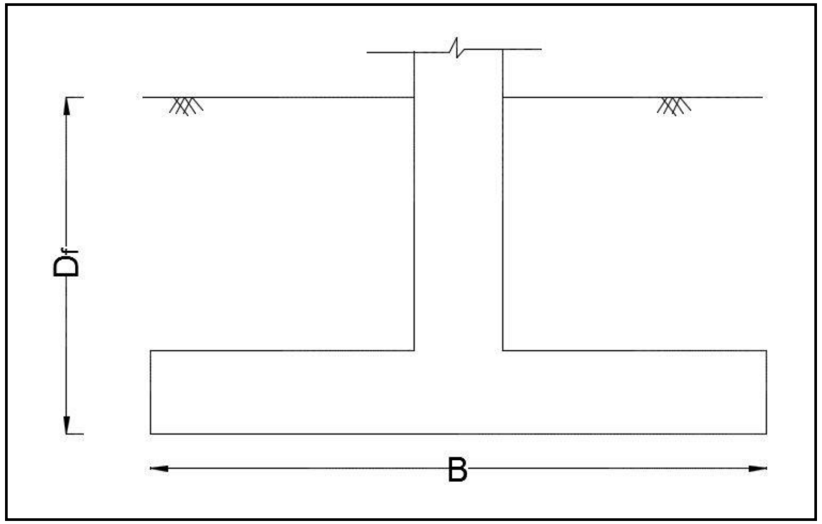
Foundation

A foundation is that part of structure which transfers the load of the structure to the sub soil.

Foundation

Shallow Foundation
$D_f / B \leq 1$



Deep Foundation

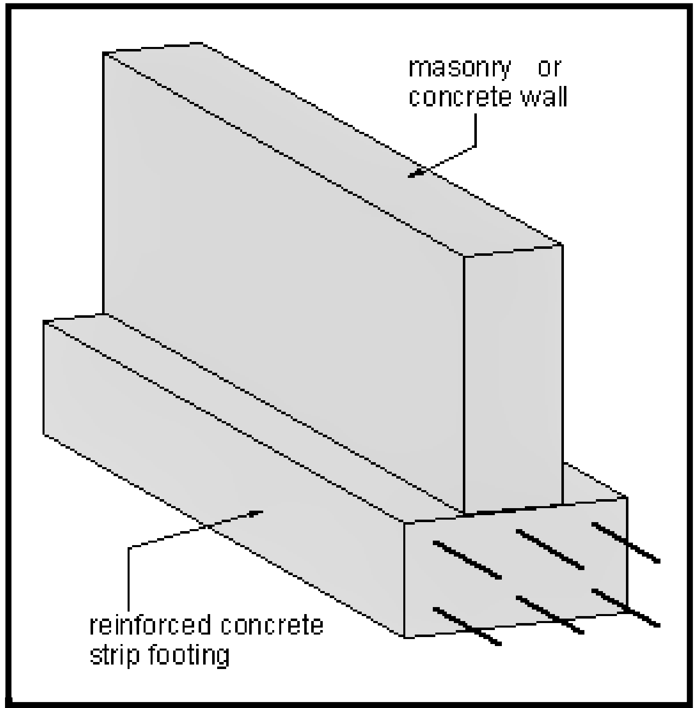
Moderately deep
$1 < D_f / B \leq 15$

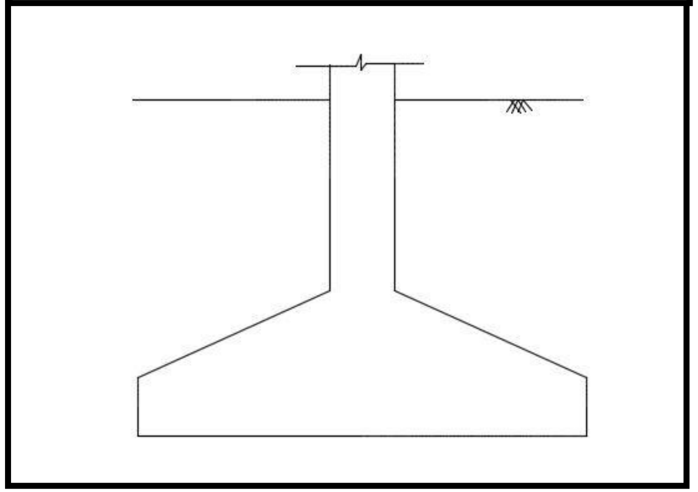
Deep
$D_f / B > 15$

Shallow Foundation

1. Strip Footing or Continuous Footing (L>>B)

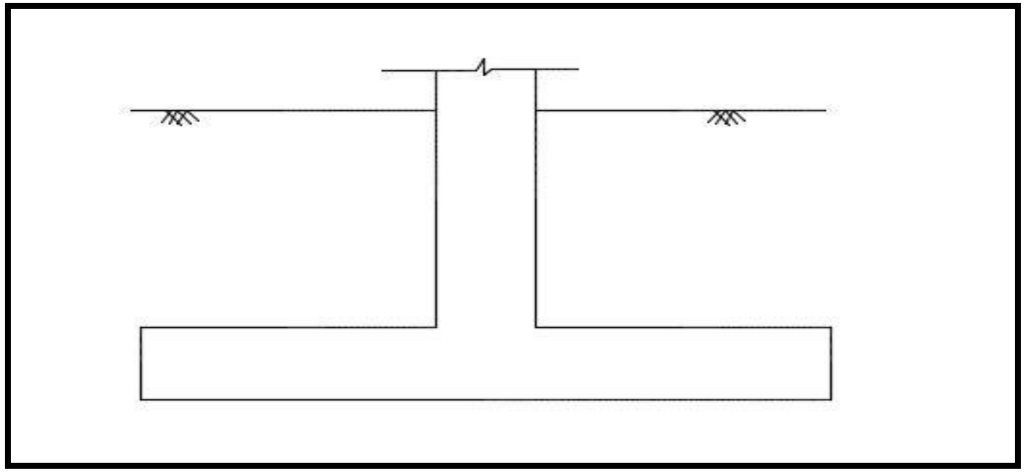
- Provided for load bearing wall
- Provided for a row of columns which are closely spaced that their footings overlap each other.

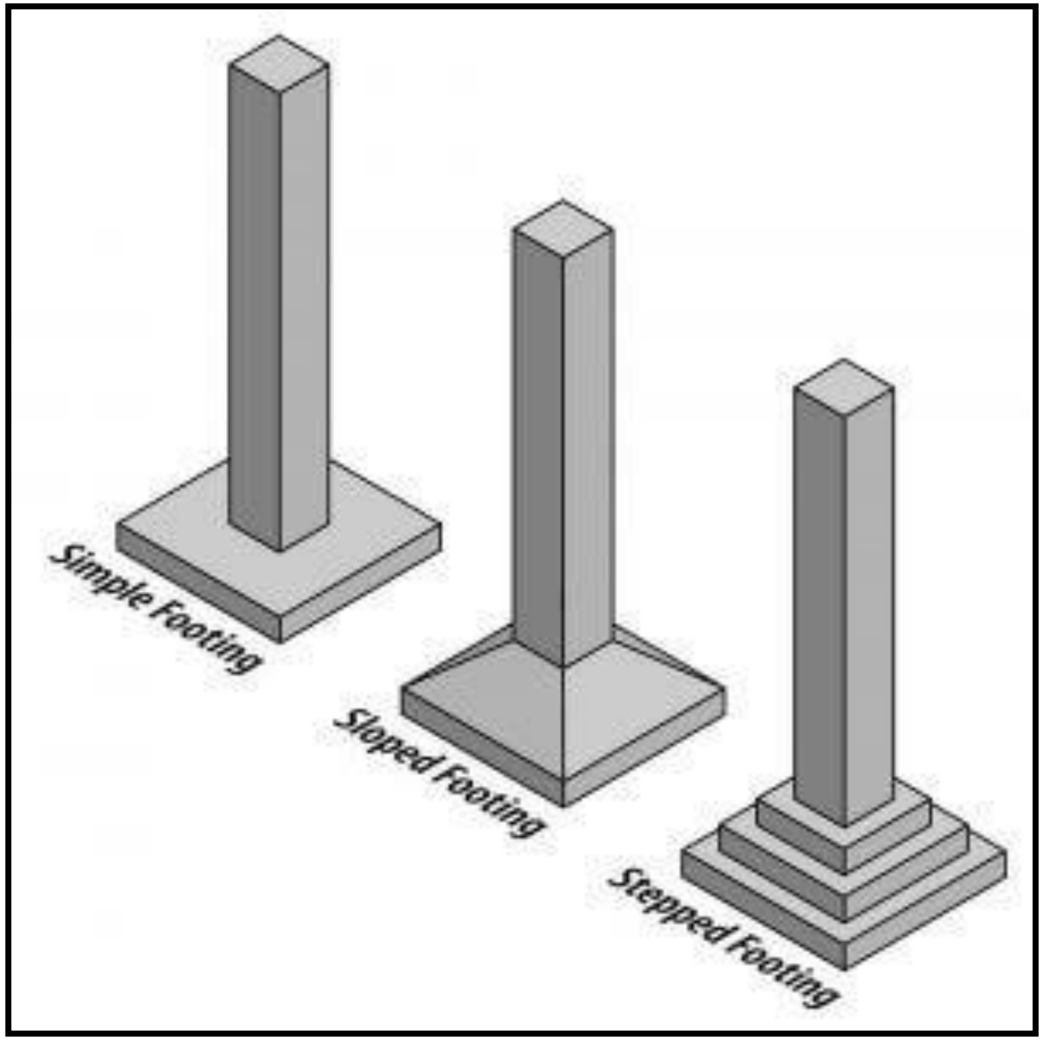




2. **Spread Footing or Isolated Footing**

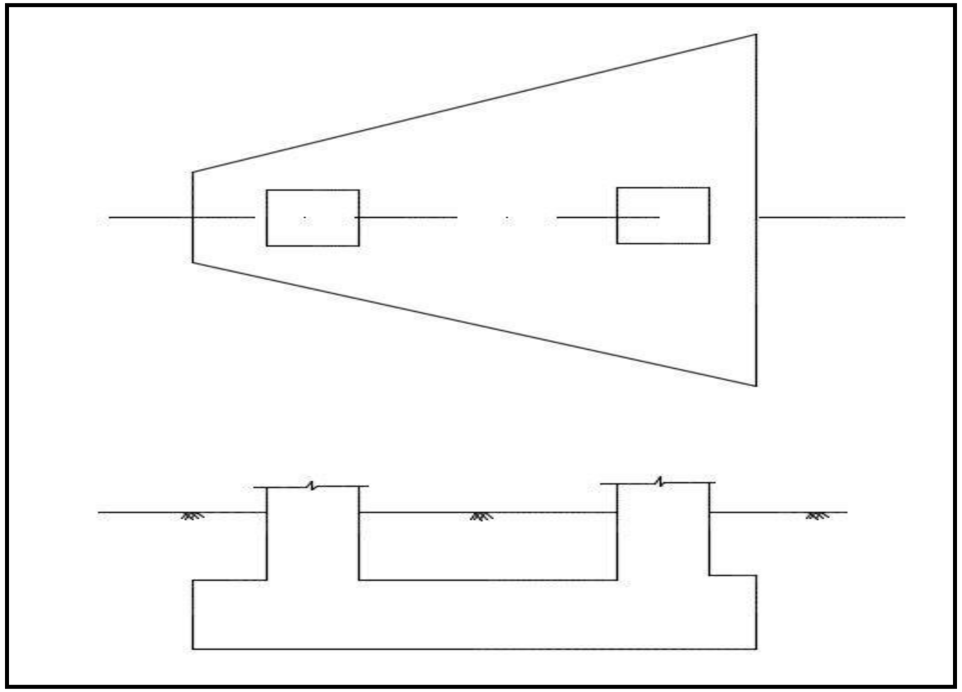
- Provided to support an individual column
- Circular, Square and rectangular

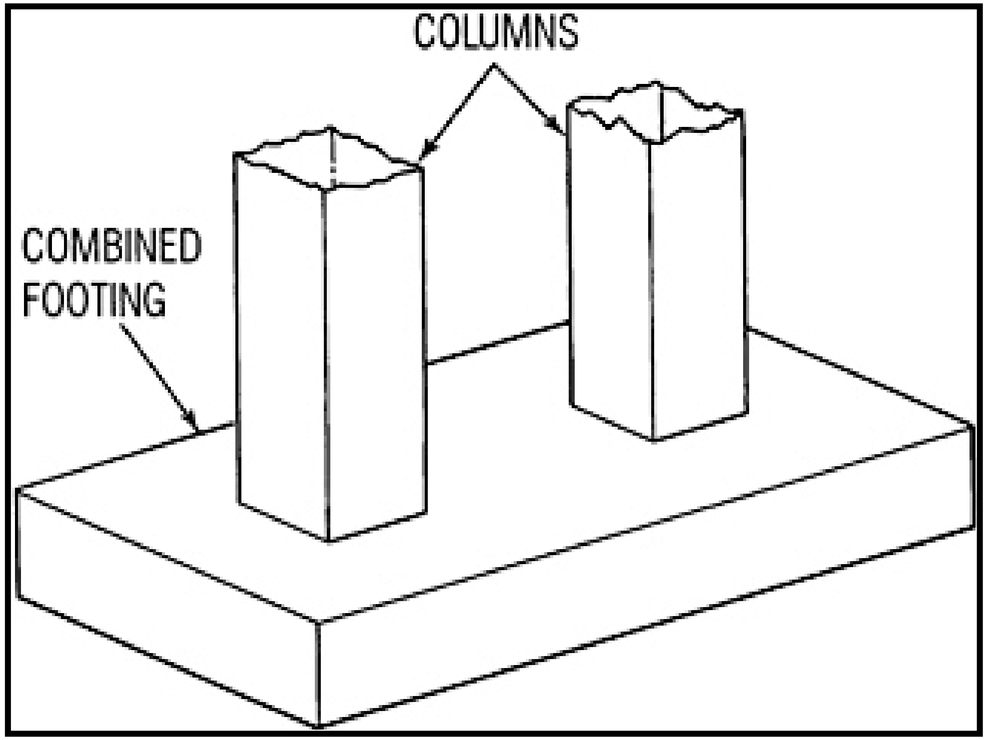




3. Combined Footing

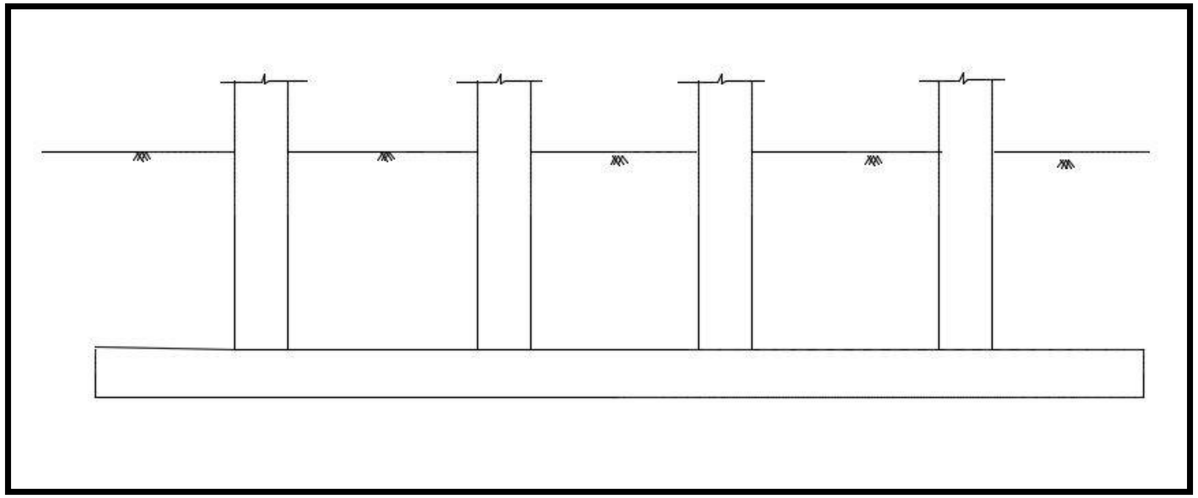
- Provided to support more than one column

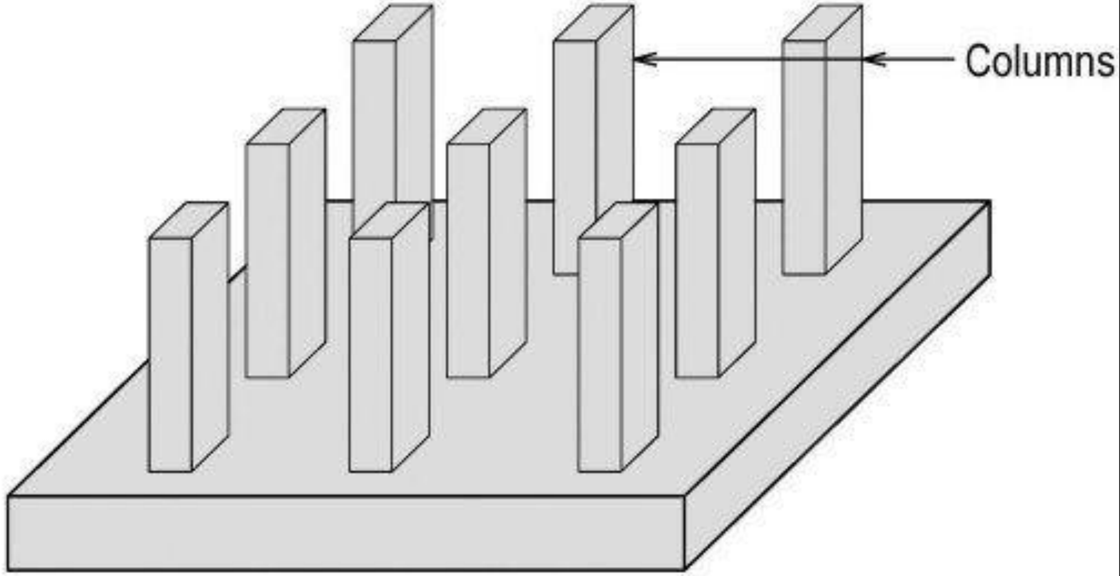




4. Mat or Raft Foundation

- Large slab supporting number of columns and walls under the entire structures





Mat /Raft Foundation

Choice of particular type of foundation depends on the

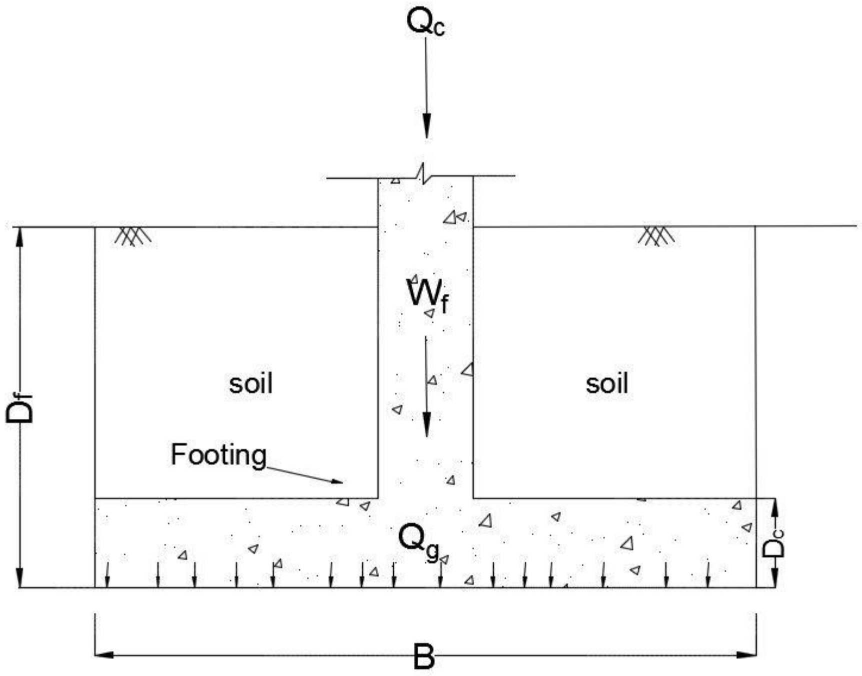
- Magnitude of loads
- Nature of the subsoil strata
- Nature of the superstructure
- Specific requirements

Two basic criteria for **design of foundation**

- Shear failure or Bearing capacity criteria
- Settlement criteria

Shear failure or Bearing Capacity Criteria :

The foundation should be design such that the soil below does not fail in shear



$$Q_g = Q_c + W_f + W_s$$

Q_c = wt. of superstructure

W_f = wt. of footing

W_f = wt. of soil/fill

The gross pressure or the gross load intensity (q_g)

$$q_g = Q_g / A$$

Ultimate bearing capacity (q_u) : The maximum gross intensity of loading that soil can support before it fails in shear.

Net ultimate bearing capacity (q_{nu}) : The maximum net intensity of loading at the base of the foundation that the soil can support before fail in shear.

$$q_{nu} = q_u - \gamma D_f$$

Net safe bearing capacity (q_{ns}) : The maximum net intensity of loading that soil can safely support without the risk of shear failure.

$$q_{ns} = q_{nu} / F$$

Gross safe bearing capacity (q_s) : The maximum gross intensity of loading that soil can carry safely without failing in shear.

$$q_s = \frac{q_{nu}}{F} + \gamma D_f$$

$$q_s = \frac{q_u - \gamma D_f}{F} + \gamma D_f$$

Settlement Criterion

Safe bearing pressure : The maximum net intensity loading that can be allowed on the soil without the settlement exceeding the permissible value.

Allowable bearing pressure ($q_{a\text{-}net}$) : The maximum net intensity of loading that can be imposed on the soil with no possibility of shear failure or the possibility of excessive settlement. It is the smaller of the net safe bearing capacity (shear failure criterion) and safe bearing pressure (settlement criterion)

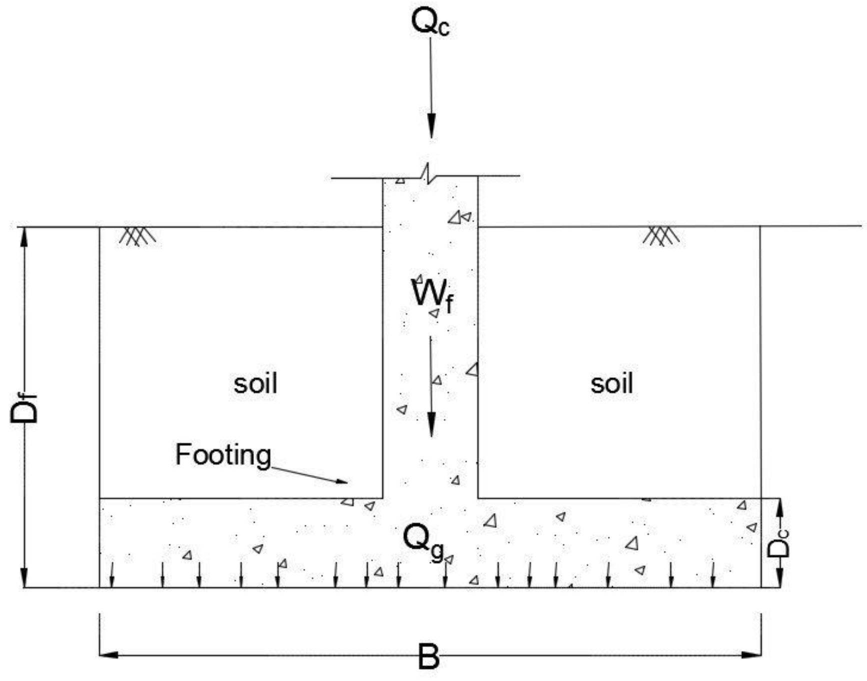
Shallow Foundation: Bearing Capacity II

Two basic criteria for **design of foundation**

- Shear failure or Bearing capacity criteria
- Settlement criteria

Shear failure or Bearing Capacity Criteria :

The foundation should be design such that the soil below does not fail in shear



$$Q_g = Q_c + W_f + W_s$$

Q_c = wt. of superstructure

W_f = wt. of footing

W_f = wt. of soil/fill

The gross pressure or the gross load intensity (q_g)

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Ultimate bearing capacity (q_u) : The maximum gross intensity of loading that soil can support before it fails in shear.

Net ultimate bearing capacity (q_{nu}) **:** The maximum net intensity of loading at the base of the foundation that the soil can support before fail in shear.

$$q_{nu} = q_u - \gamma D_f$$

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$$q_{ns} = q_{nu} / F$$

Gross safe bearing capacity (q_s) : The maximum gross intensity of loading that soil can carry safely without failing in shear.

$$q_s = \frac{q_{nu}}{F} + \gamma D_f$$

$$q_s = \frac{q_u - \gamma D_f}{F} + \gamma D_f$$

Settlement Criterion

Safe bearing pressure : The maximum net intensity loading that can be allowed on the soil without the settlement exceeding the permissible value.

Allowable bearing pressure ($q_{a\text{-}net}$) : The maximum net intensity of loading that can be imposed on the soil with no possibility of shear failure or the possibility of excessive settlement. It is the smaller of the net safe bearing capacity (shear failure criterion) and safe bearing pressure (settlement criterion)

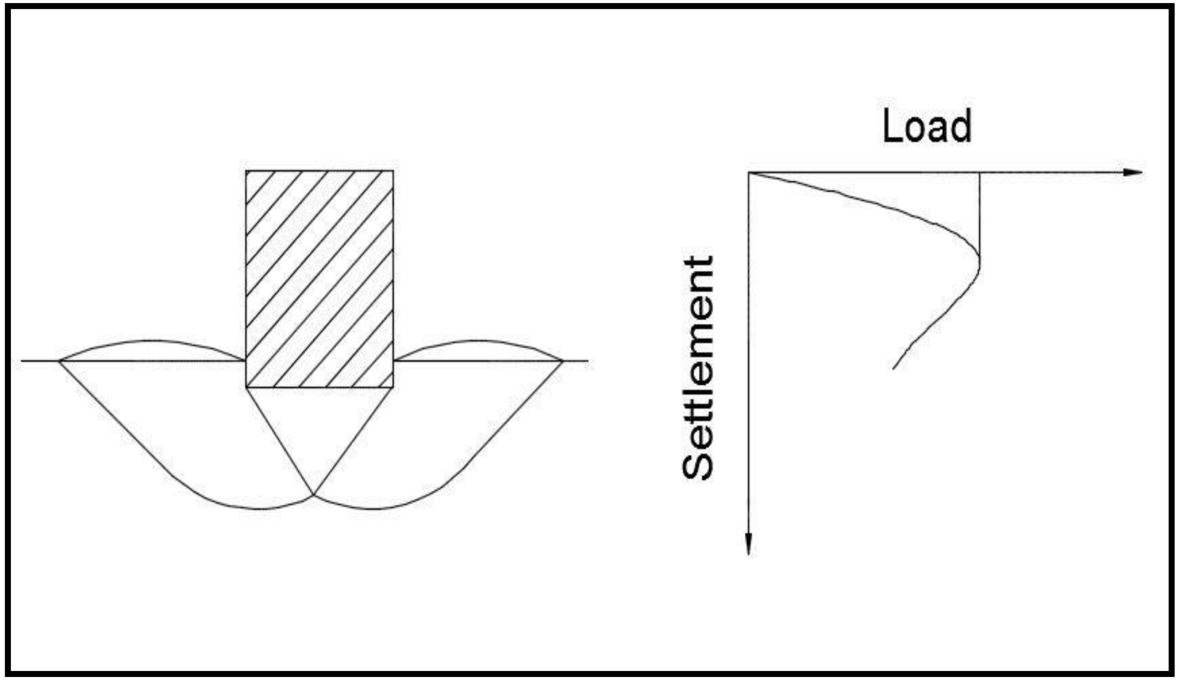
Modes of soil failure

General shear failure (Dense sand / stiff clay)

- A well defined failure surface

- A bulging of ground surface adjacent to the foundation

- The ultimate load can be easily located.



c_u (kPa)	consistency
0 – 12.5	very soft
12.5-25	soft
25-50	medium
50-100	stiff
100-200	very stiff
>200	hard

D_r (%)	consistency
0-15	very loose
15-35	loose
35-65	medium
65-85	dense
85-100	very dense

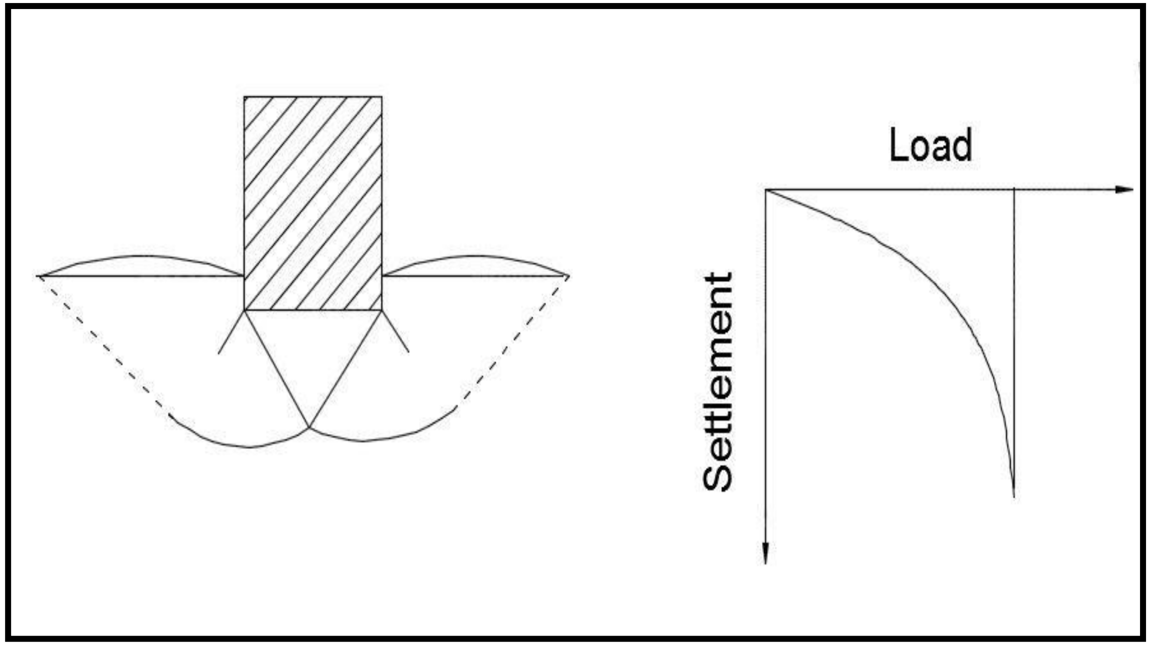
Local shear failure (medium or relatively loose sand /medium and relatively soft consistency clay)

- Well defined wedge and slip surfaces only beneath the foundation

- Slight bulging of the ground surface adjacent to the foundation

- Load settlement curve does not indicate ultimate load clearly

- Significant compression of the soil directly beneath the footing



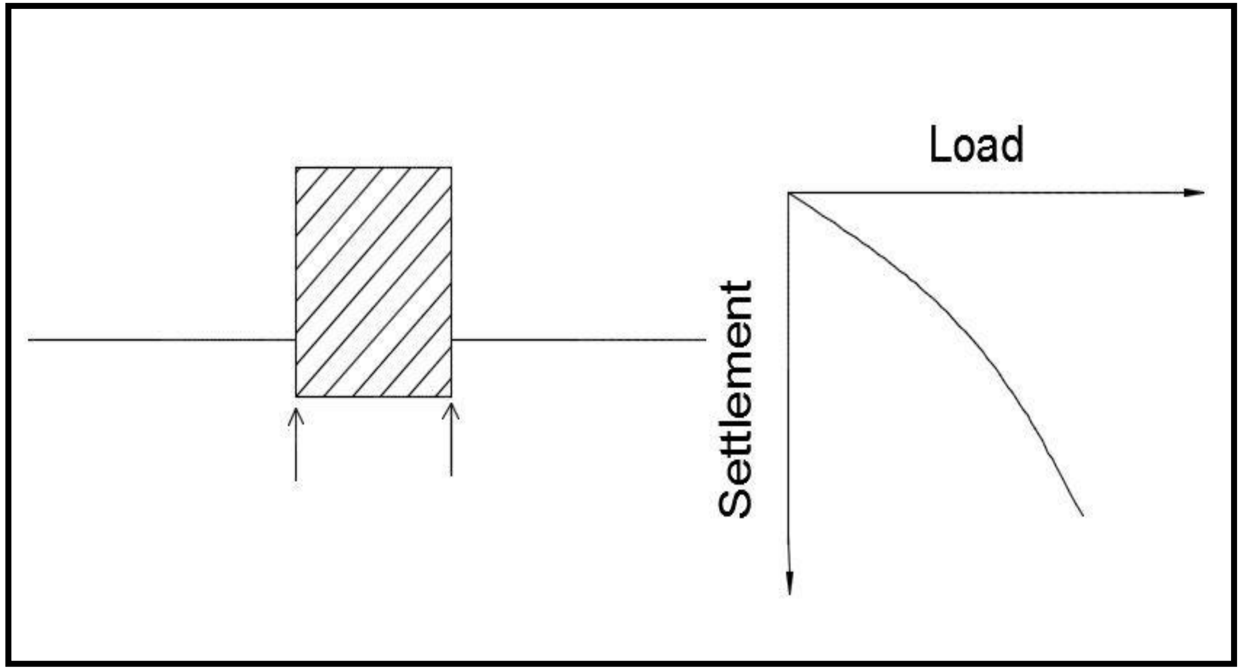
Punching shear failure (very loose sand / very soft clay)

- Poorly defined shear planes

- Soil zones beyond the loaded area being little affected

- Significant penetration of a wedge shaped soil zone beneath the foundation

- Ultimate load can not be clearly recognized



Terzaghi’s bearing capacity theory:

The footing is a **long strip or a continuous** footing resting on a deep **homogeneous** soil having **shear parameter c and ϕ.**

- Analysis is a 2-D condition
- The soil fails in a general shear failure mode
- The load is vertical and concentric

- The ground surface is horizontal.
- The base of the footing is laid at a shallow depth i.e., $\mathbf{D_f \leq B}$.
- The shearing resistance of the soil between the surface and the depth D_f is neglected. The footing is considered as a surface footing with a uniform surcharge equal to $\boldsymbol{\gamma D_f}$ at a level of the footing

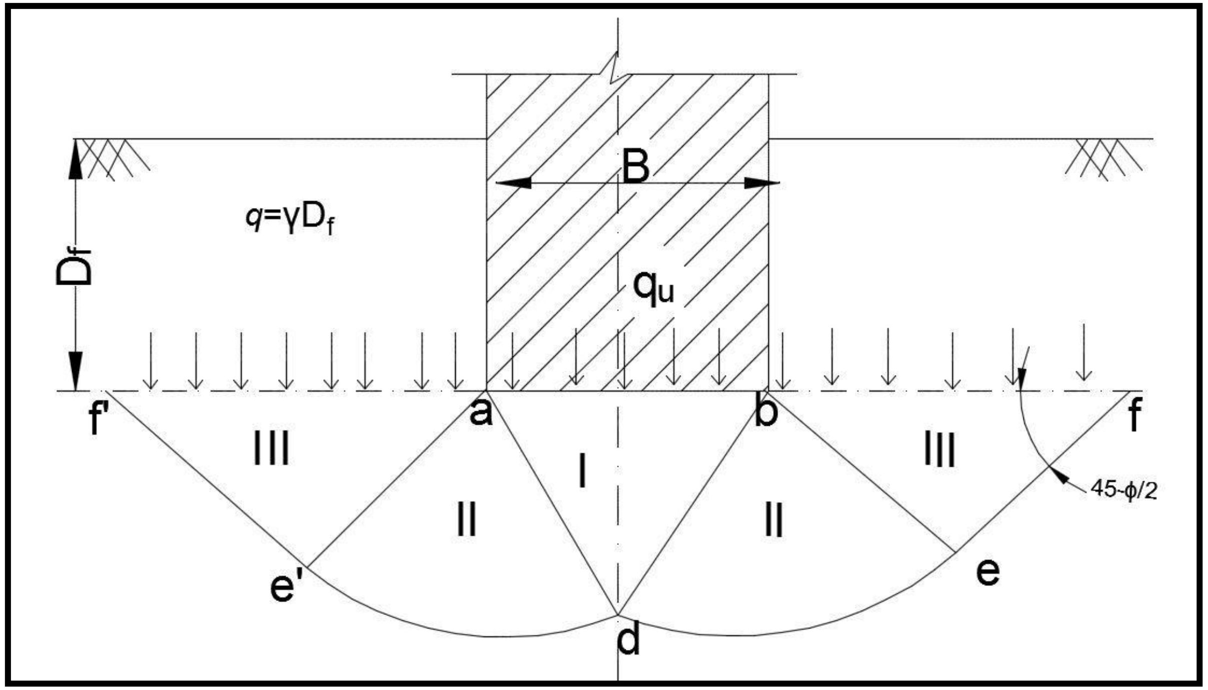
Zone – I (zone abd)

- The soil in this zone remains in a state of elastic equilibrium

- The soil wedge **abd** immediately beneath the footing is prevented from undergoing any lateral movement by the friction and adhesion between the base of footing and soil.

Zone II (bed and ae′d) : Zone of radial shear

Zone III (bef and ae′f) : Rankine passive zone



Shallow Foundation : Bearing Capacity III

Terzaghi’s bearing capacity theory:

The footing is a **long strip or a continuous** footing resting on a deep **homogeneous** soil having **shear parameter c and ϕ.**

- Analysis is a 2 -D condition

- The soil fails in a general shear failure mode

- The load is vertical and concentric

- The ground surface is horizontal.
- The base of the footing is laid at a shallow depth i.e., $\mathbf{D_f \leq B}$.
- The shearing resistance of the soil between the surface and the depth D_f is neglected. The footing is considered as a surface footing with a uniform surcharge equal to $\boldsymbol{\gamma} \mathbf{D_f}$ at a level of the footing

Zone – I (zone abd)

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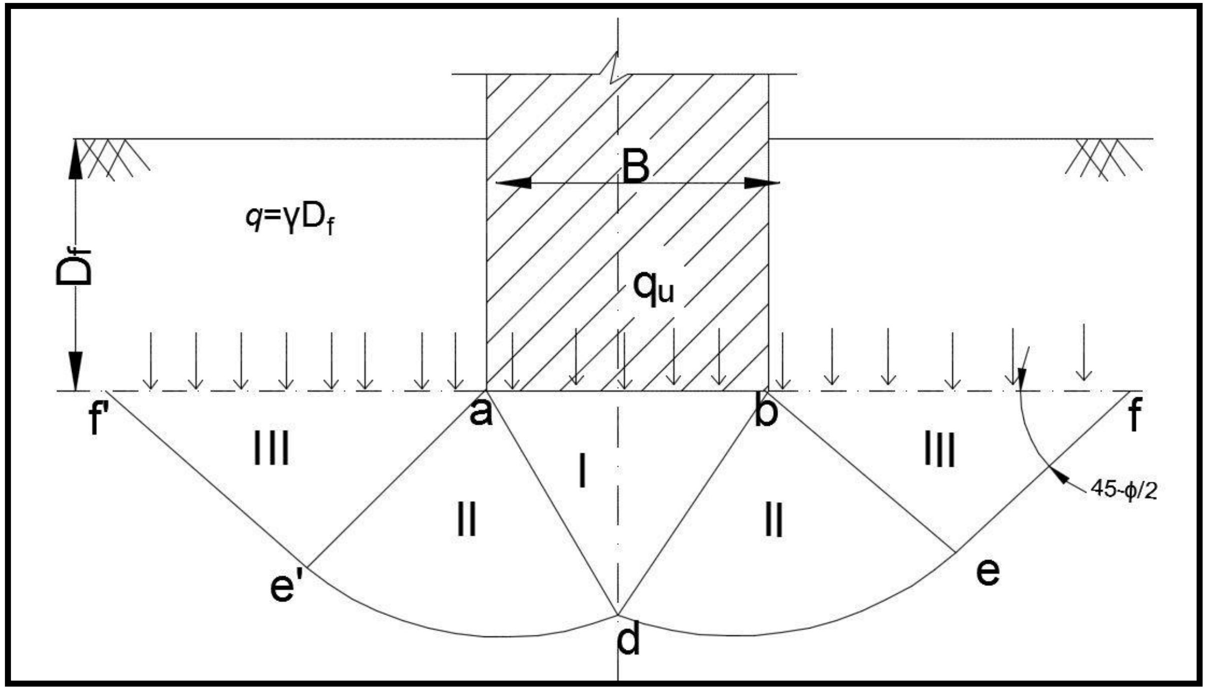
Zone II (bed and ae′d) : Zone of radial shear

Zone III (bef and ae′f) : Rankine passive zone



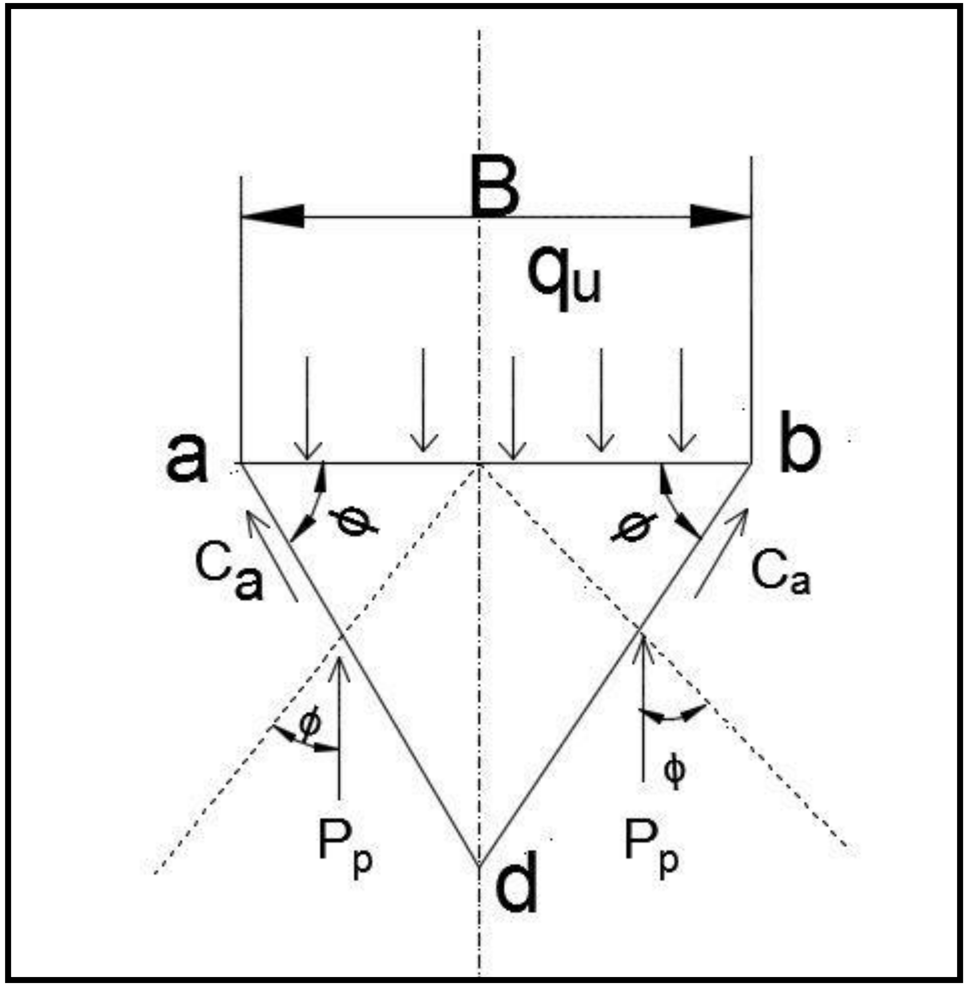
The equation developed for the ultimate bearing capacity is

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

$$N_c = \cot\phi \left[\frac{a^2}{2\cos^2\left(45^\circ + \frac{\phi}{2}\right)} - 1 \right] \qquad N_q = \left[\frac{a^2}{2\cos^2\left(45^\circ + \frac{\phi}{2}\right)} \right]$$

$$N_\gamma = \frac{1}{2}\left[\frac{K_p}{\cos^2\phi} - 1 \right]\tan(\phi)$$

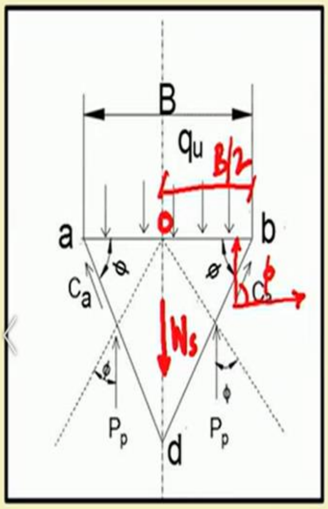
where $a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$



$$q_u \times B = 2P_p + 2C_a\sin\phi - W_s$$

q_u (kN/m²)

C_a = kN/m (Force)



$$W_s = \frac{1}{2}\times B \times \frac{B}{2}\tan\phi \times \gamma = \frac{1}{4}B^2\tan\phi \times \gamma$$

$$C_a = c \times bd = c\,\frac{B/2}{\cos\phi} = \frac{cB}{2\cos\phi}$$

$$Od = \frac{B}{2}\tan\phi$$

γ = unit wt. of the soil

c = cohesion (kN/m²)

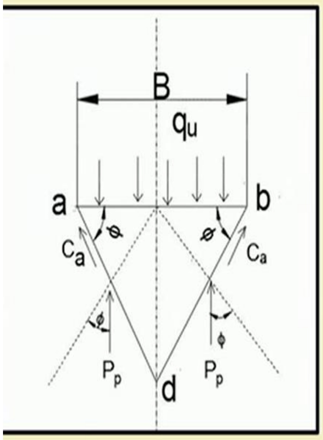
$$bd = \frac{B/2}{\cos\phi}$$

$$q_u B = 2P_p + 2\times\frac{cB}{2\cos\phi}\sin\phi - \frac{1}{4}\gamma B^2\tan\phi$$

$$= 2P_p + cB\tan\phi - \frac{1}{4}\gamma B^2\tan\phi$$

P_p = Passive force (kN/m)

$$q_u B = 2P_p + Bc\tan\phi - \frac{1}{4}\gamma B^2\tan\phi$$



$$P_p = (P_{p\gamma} + P_{pc} + P_{pq})$$

- $P_{p\gamma}$: $c=0, q=0$ — Due to the soil wt.
- P_{pc}: Due to the cohesion ($q=0, \gamma=0$)
- P_{pq}: Due to the surcharge ($\gamma=0, c=0$)

$$q_u B = 2(P_{p\gamma} + P_{pc} + P_{pq}) + Bc\tan\phi - \frac{1}{4}\gamma B^2\tan\phi$$

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

$$2P_{pc} + Bc\tan\phi = B\times cN_c$$

$$q = \gamma D_f$$

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

Bearing capacity factors (N_c, N_q, N_γ)

ϕ	Terzaghi's Bearing Capacity Factor		
	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
50	347.5	415.1	1153.2

Ultimate bearing capacity for local shear failure

Mobilized cohesion: $c_m = \frac{2}{3} c$

Mobilized angle of shearing resistance: $\phi_m = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$

$$q_u = \frac{2}{3} c N'_c + \gamma D_f N'_q + \frac{1}{2} \gamma B N'_\gamma$$

For sandy soil (c’=0)

- $\phi \geq 36°$ - **Purely general shear failure**, $\phi \leq 29°$ - **Purely local shear failure**

  ϕ between this range represents the **mixed state of general and local shear failure**

For c-ϕ soil

- Failure of soil specimen occur at a relatively small strain (less than 5%) - **General shear failure**
- If stress – strain curve does not show peak and has a continuously rising pattern upto a strain of 10- 20% - **Local shear failure**

Ultimate bearing capacity of strip, square, circular and rectangular footing

$$q_u = \alpha_1 c N_c + \gamma D_f N_q + \alpha_2 \gamma B N_\gamma$$

For strip footing : $\alpha_1 = 1.0$, $\alpha_2 = 0.5$

For square footing : $\alpha_1 = 1.3$, $\alpha_2 = 0.4$

For circular footing : $\alpha_1 = 1.3$, $\alpha_2 = 0.3$

For Rectangular Footing:

Ultimate bearing capacity in purely cohesionless soil (c = 0)

$$q_u = \gamma D_f N_q + \alpha_2 \gamma B N_\gamma$$

Ultimate bearing capacity in purely cohesive soil (ϕ = 0)

$$q_u = \alpha_1 c N_c + \gamma D_f$$

Effect of water table :

$$q_u = cN_c + qN_q + 0.5\gamma BN_\gamma$$

For φ = 0 (**saturated clay**) , **q_{nu} = 5.7 c_u**

The effect of **submergence** is to reduce the **undrained shearing strength c_u** due to a **softening effect**. The shear strength parameter should be determined in the laboratory under saturated condition .

Water table located above the base of footing:

The effective surcharge is reduced as the effective weight below the water table is equal to the submerged unit weight .

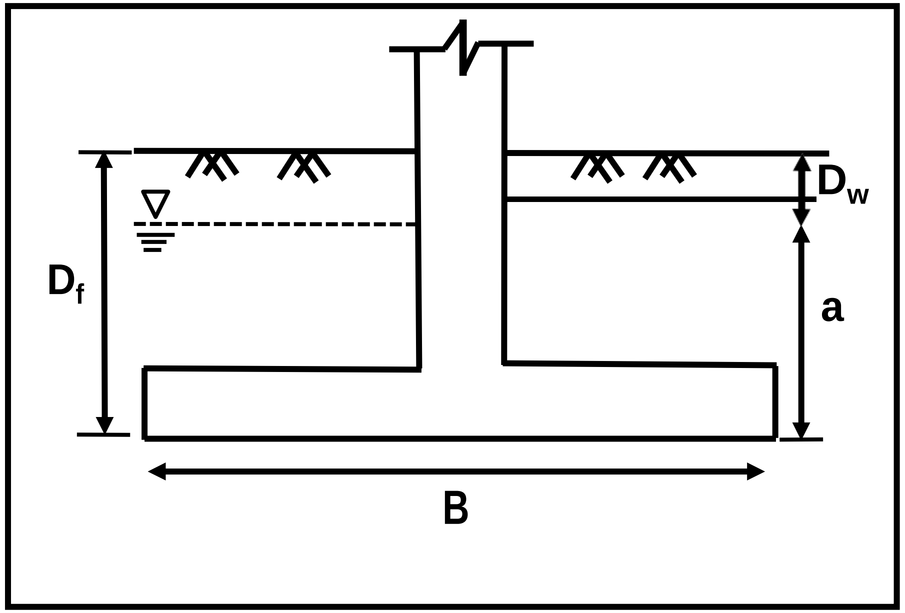
$$q = D_w\gamma + a\gamma'$$

As, **a = D$_f$ - D$_w$** $q = \gamma' D_f + (\gamma - \gamma')D_w$

$$q_u = c_u N_c + \left[\gamma' D_f + (\gamma - \gamma')D_w\right]N_q + \frac{1}{2}\gamma' B N_\gamma$$

If $D_w = 0$ (i.e., $a = D_f$) $q_u = c_u N_c + \gamma' D_f N_q + \frac{1}{2}\gamma' B N_\gamma$

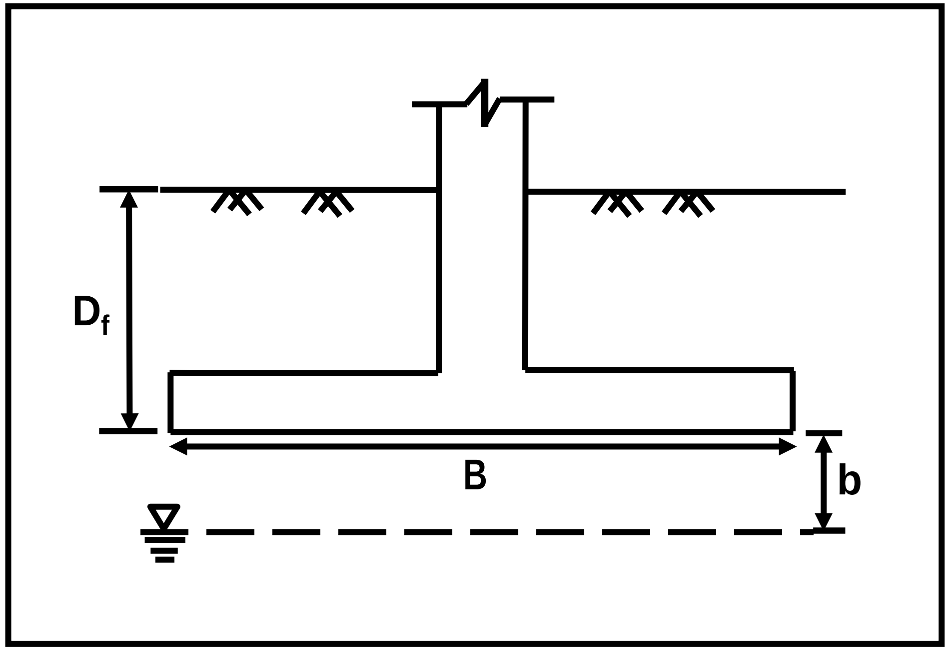
If $a = 0$ (i.e., $D_f = D_w$) $q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2}\gamma' B N_\gamma$



Water table located at a depth b below the base of footing

In this case, the **surcharge term is not affected**. However, the unit weight in the third term of bearing capacity equation is modified as

$$\bar{\gamma} = \gamma' + \frac{b}{B}(\gamma - \gamma')$$



$$q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B\left[\gamma' + \frac{b}{B}(\gamma - \gamma')\right] N_\gamma$$

If b = 0, i.e., W/T at the base, $q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B\gamma' N_\gamma$

If b = B, i.e., W/T at depth below B, $q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B\gamma N_\gamma$

Shallow Foundation : Bearing Capacity IV

Terzaghi's bearing capacity theory:

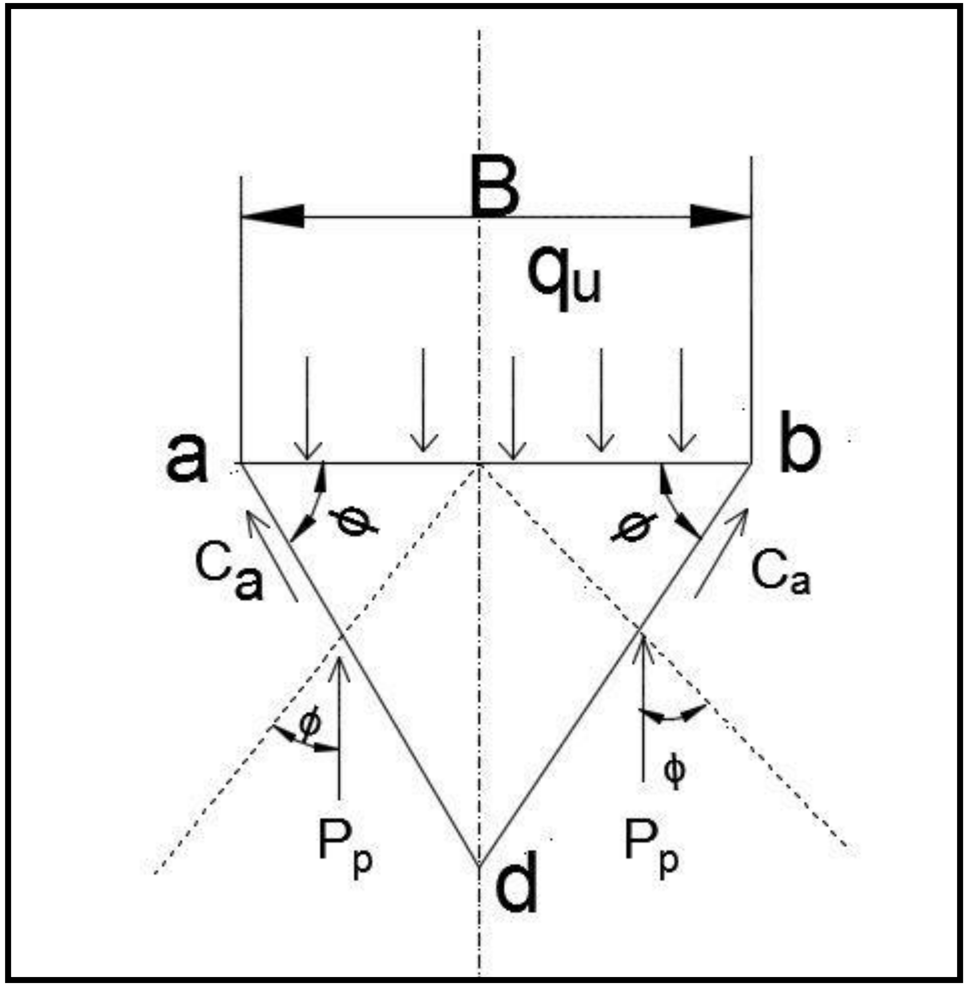
The equation developed for the ultimate bearing capacity is

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

$$N_c = \cot\phi\left[\frac{a^2}{2\cos^2\left(45^\circ + \frac{\phi}{2}\right)} - 1\right] \qquad N_q = \left[\frac{a^2}{2\cos^2\left(45^\circ + \frac{\phi}{2}\right)}\right]$$

$$N_\gamma = \frac{1}{2}\left[\frac{K_p}{\cos^2\phi} - 1\right]\tan(\phi)$$

where $a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$



ϕ	Terzaghi's Bearing Capacity Factor		
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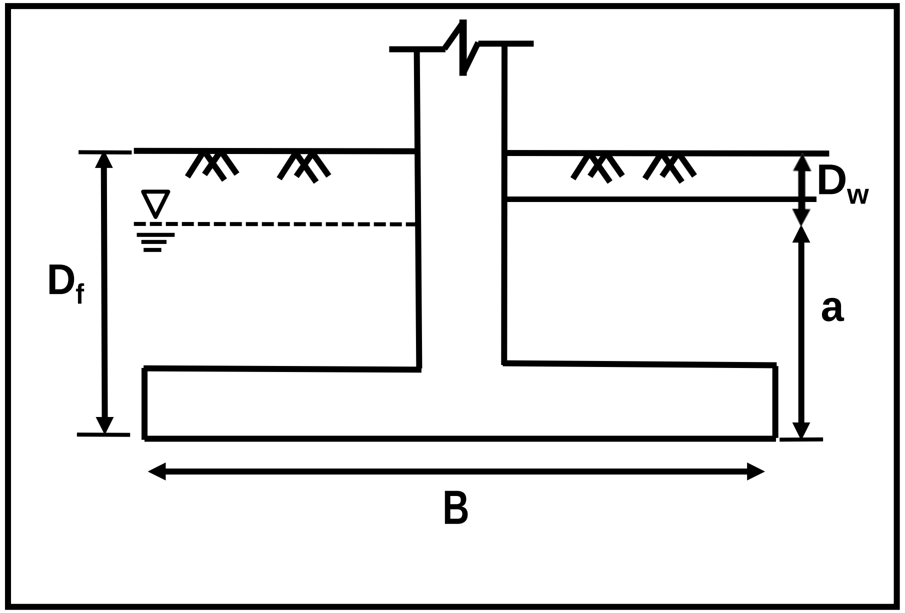
Ranjan and Rao, 1991

Water table located above the base of footing:

The effective surcharge is reduced as the effective weight below the water table is equal to the submerged unit weight .



$$q = D_w\gamma + a\gamma'$$

As, **a = D_f - D_w** $\quad q = \gamma' D_f + (\gamma - \gamma')D_w$

$$q_u = c_u N_c + \left[\gamma' D_f + (\gamma - \gamma')D_w\right]N_q + \frac{1}{2}\gamma' B N_\gamma$$

If D_w = 0 (i.e., a = D_f) $\quad q_u = c_u N_c + \gamma' D_f N_q + \frac{1}{2}\gamma' B N_\gamma$

If a = 0 (i.e., D_f = D_w) $\quad q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2}\gamma' B N_\gamma$

Water table located at a depth b below the base of footing

In this case, the **surcharge term is not affected**. However, the unit weight in the third term of bearing capacity equation is modified as

$$\gamma = \gamma' + \frac{b}{B}(\gamma - \gamma')$$

If b = 0, i.e., W/T at the base, $q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B\gamma' N_\gamma$

If b = B, i.e., W/T at depth below B, $q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B\gamma N_\gamma$

Ultimate bearing capacity analysis for clay soil (Skempton,1951):

For $\phi = 0$, $q_{nu} = c_u N_c$

For strip footing : $N_c = 5\left(1 + 0.2\frac{D_f}{B}\right)$ The maximum value of **N_c** is 7.50

For square and circular footing : $N_c = 6\left(1 + 0.2\frac{D_f}{B}\right)$

The maximum value of **N_c** is 9

For rectangular footing :

$$N_c = 5.0\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right)$$ For $D_f/B \leq 2.5$

$$N_c = 7.5\left(1 + 0.2\frac{B}{L}\right)$$ For $D_f/B > 2.5$

The analysis is valid for **any value** of D_f/B

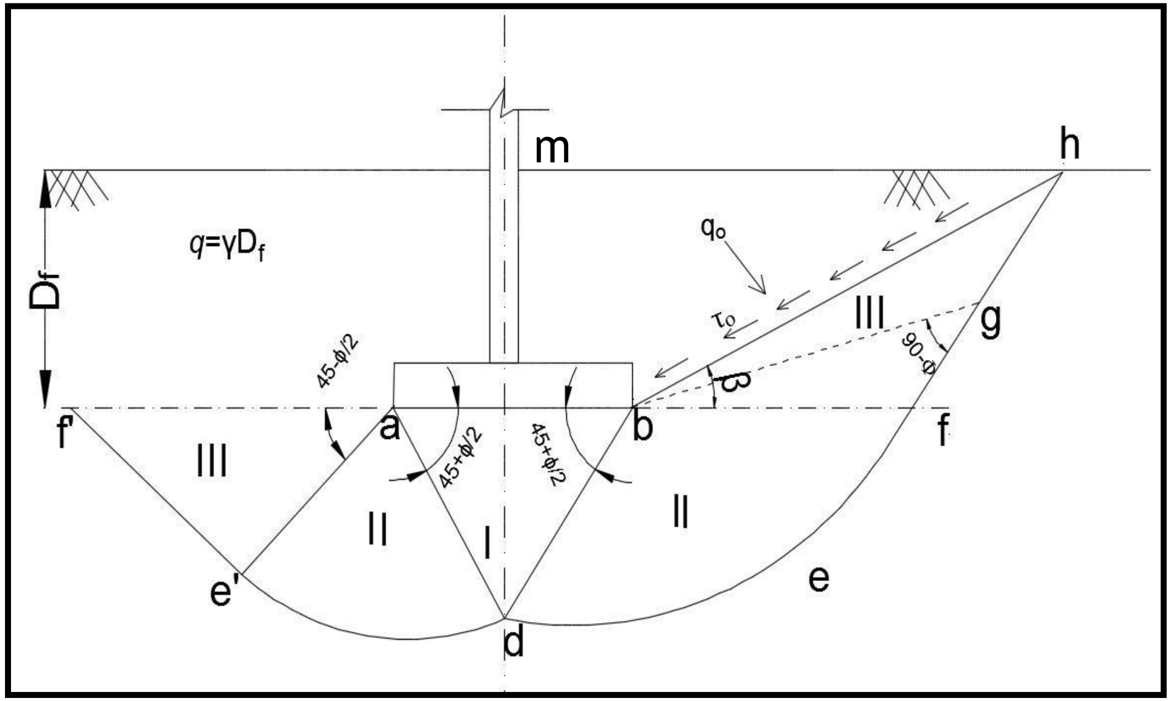
Meyerhof's Analysis :

- Bearing capacity for a strip footing at **any depth**.
- For shallow footing, $q_0 = \gamma D_f$

$$q_u = cN_c + q_0 N_q + \frac{1}{2} B\gamma N_\gamma$$

N_c, N_q, N_γ depends on roughness of base, depth of footing, and the shape of footing, in addition to the angle of shearing resistance ϕ'

β increases with an increase in depth D_f and is equal to 90° for deep foundation



Zone I – abd, elastic zone
Zone II – bgd, zone of radial shear
Zone III – bghm, zone of mixed shear in which shear varies between radial shear and plane shear

$$q_u = cN_c s_c d_c i_c + q_0 N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

s, d, and i stand for shape factor, depth factor, inclination factor

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

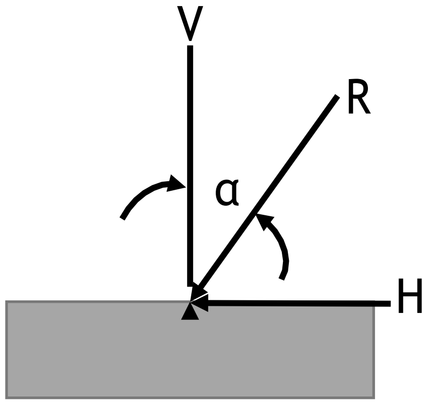
S_c , S_q , S_γ= 1 for strip footing

Shape, depth, inclination factor for the Meyerhof's bearing capacity equation:

Factors	Value	For
Shape	$s_c = 1 + 0.2K_p\left(\frac{B}{L}\right)$	Any ϕ
	$s_q = s_\gamma = 1 + 0.1K_p\left(\frac{B}{L}\right)$	ϕ > 10°
	$s_q = s_\gamma = 1$	ϕ = 0°
Depth	$d_c = 1 + 0.2\sqrt{K_p}\left(\frac{D_f}{B}\right)$	Any ϕ
	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\left(\frac{D_f}{B}\right)$	ϕ > 10°
	$d_q = d_\gamma = 1$	ϕ = 0°

Bowles, 1997

Shape, depth, inclination factor for the Meyerhof's bearing capacity equation:

Factors	Value	For
Inclination 	$i_c = i_q = \left(1 - \frac{\alpha^\circ}{90^\circ}\right)^2$	Any ϕ
	$i_\gamma = \left(1 - \frac{\alpha^\circ}{\phi^\circ}\right)^2$	$\phi > 0^\circ$
	$i_\gamma = 0$ For $\alpha > 0$	$\phi = 0^\circ$

$$K_p = \tan^2\left(45^\circ + \frac{\phi}{2}\right)$$

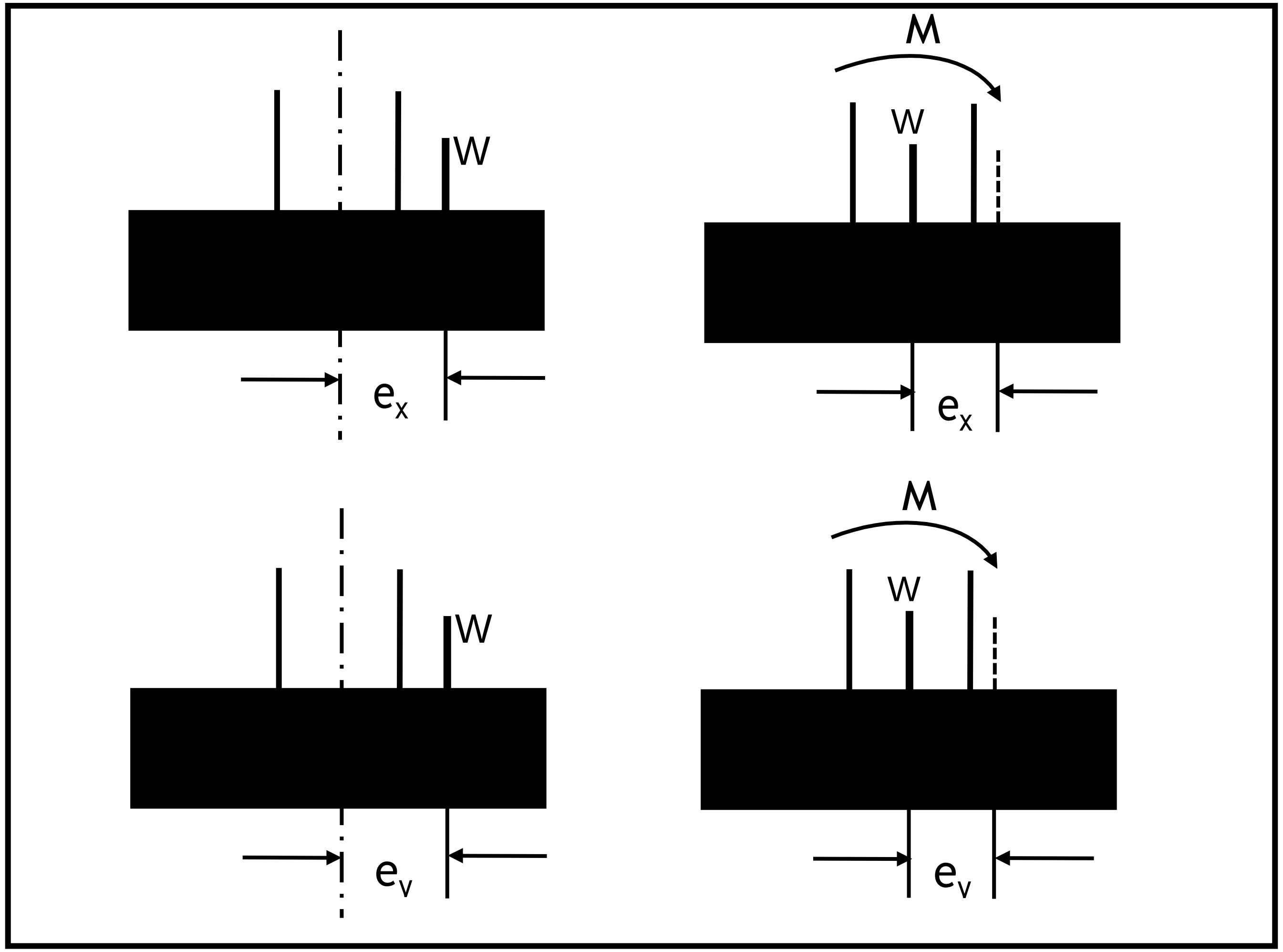
α angle of resultant R measured from vertical

Bowles, 1997

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.5	1.6	0.07
10	8.3	2.5	0.37
15	11	3.9	1.2
20	14.8	6.4	2.9
25	20.7	10.7	6.8
30	30.1	18.4	16.7
32	35.5	23.2	22.0
34	42.2	29.4	31.1
36	50.6	37.8	44.5
38	61.4	48.9	64.0

ϕ	N_c	N_q	N_γ
40	75.3	64.1	93.7
45	133.9	134.9	262.8
50	266.9	319.1	874.0

Eccentrically of loaded foundation:



For **strip footing**: $B' = B - 2e_x$

For **rectangular footing**: $B' = B - 2e_x$

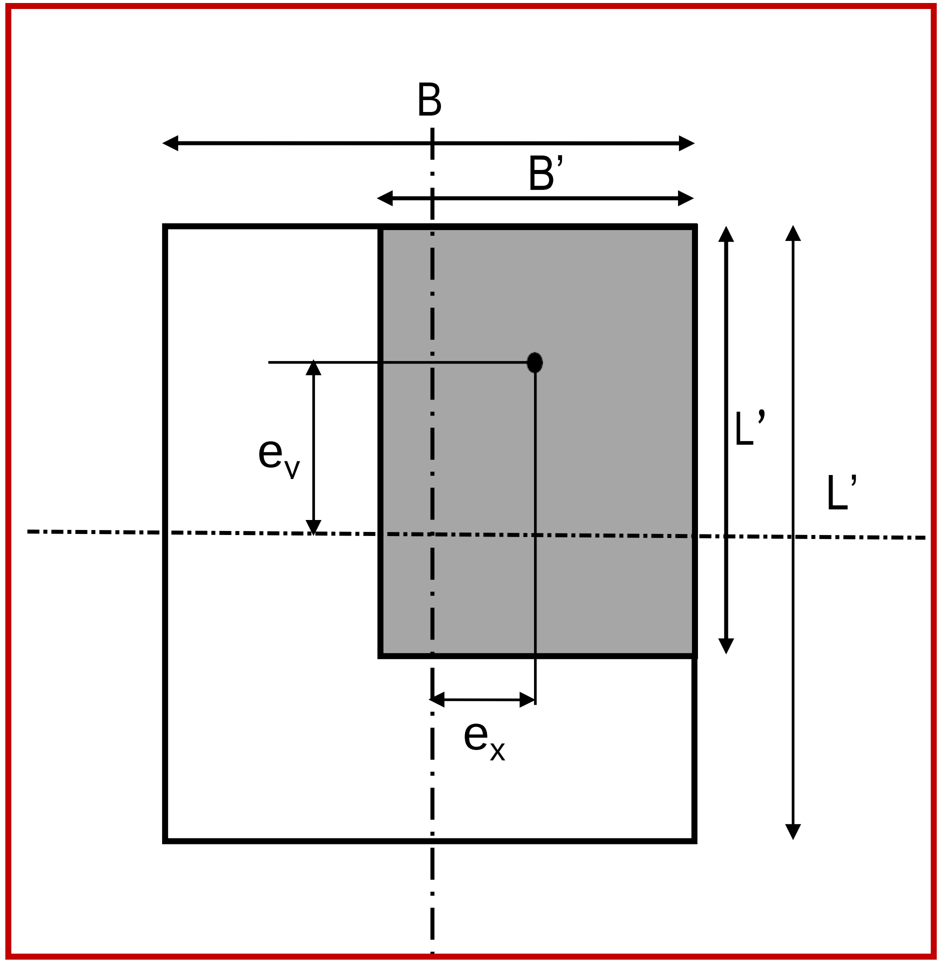
$L' = L - 2e_y$

The **effective area** of footing $A' = B' \times L'$

The ultimate load bearing capacity of footing can be expressed as

$$Q_u = q_u \times A'$$

$$q_u = cN_c s_c d_c i_c + q\,N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$



Hansen's bearing capacity Theory:

For **cohesive soil**, Hansen's theory gives better correlation than the Terzaghi equation

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

For ϕ =0 $q_u = cN_c(1 + s_c + d_c - i_c) + q$

$N_c = (N_q - 1)\cot(\phi)$ Same as Meyerhof

$N_q = e^{\pi \tan(\phi)} \tan^2\left(45 + \frac{\phi}{2}\right)$ Same as Meyerhof

$N_\gamma = 1.5(N_q - 1)\tan(\phi)$

ϕ	N_γ
0	0
5	0.1
10	0.4
15	1.2
20	2.9
25	6.8
30	15.1
32	20.8
34	28.8
36	40.1
38	56.2
40	79.5
45	200.8
50	568.5

<u>Hansen's bearing capacity factors</u>

Shape, depth, inclination factor for the Hansen’s bearing capacity equation:

Factors	Value
Shape	$s_c = 1 + \frac{N_q}{N_c}\left(\frac{B}{L}\right) \quad for\ \phi \neq 0$
	$s_c = 0.2\frac{B}{L} \quad for\ \phi = 0$
	$s_q = 1 + \sin(\phi)\left(\frac{B}{L}\right)$
	$s_\gamma = (1 - 0.4\frac{B}{L}) \geq 0.6$
Depth	$d_c = 1 + 0.4k$ $\quad k = \frac{D_f}{B}$ For $D_f/B \leq 1$ and $k = \tan^{-1}(D_f/B)$ For $D_f/B > 1$, k in radian
	$d_q = 1 + 2(\tan\phi)(1 - \sin\phi)^2\left(\frac{D_f}{B}\right)$
	$d_\gamma = 1$ For all ϕ

$s_c = 1$ for strip footing

Bowles, 19 7

Factors	Value
Inclination	$i_c = i_q - \frac{1 - i_q}{N_q - 1}$ For $\phi \neq 0°$
	$i_c = 0.5 - 0.5\sqrt{1 - \frac{H}{A'c_a}}$ For $\phi = 0°$
	$i_q = \left(1 - \frac{0.5H}{V + A'c_a \cot\phi}\right)^5$
	$i_\gamma = \left(1 - \frac{0.7H}{V + A'c_a \cot\phi}\right)^5$

H = horizontal component of inclined load, V = vertical component of inclined load

c_a= base adhesion, 0.6 to 1 X Base cohesion

Bowles, 1997

Shallow Foundation : Bearing Capacity V

Vesic's bearing capacity theory:

The bearing capacity equation is similar in form to Hansen's equation

$$N_c = (N_q - 1)\cot(\phi)$$

Same as Meyerhof

$$N_q = e^{\pi \tan(\phi)} \tan^2\left(45 + \frac{\phi}{2}\right)$$

Same as Meyerhof

$$N_\gamma = 2(N_q + 1)\tan(\phi)$$

ϕ	N_γ
0	0
5	0.4
10	1.2
15	2.6
20	5.4
25	10.9
30	22.4
32	30.2
34	41
36	56.2
38	77.9

ϕ	N_γ
40	109.4
45	271.3
50	762.84

Vesic's bearing capacity factors

Shape, depth, inclination factor for the Vesic's bearing capacity equation:

Factors	Value
Shape	$s_c = 1 + \frac{N_q}{N_c}\left(\frac{B}{L}\right)$
	s_c = 1 for strip footing
	$s_q = 1 + \tan(\phi)\left(\frac{B}{L}\right)$ For all ϕ
	$s_\gamma = \left(1 - 0.4\frac{B}{L}\right) \ge 0.6$
Depth	$d_c = 1 + 0.4k$ $k = \frac{D_f}{B}$ For $D_f/B \le 1$ and $k = \tan^{-1}(D_f/B)$ For $D_f/B > 1$, k in radian
	$d_q = 1 + 2(\tan\phi)(1 - \sin\phi)^2 k$
	$d_\gamma = 1$ For all ϕ

Bowles, 1997

Factors	Value
Inclination	$i_c = i_q - \frac{1 - i_q}{N_q - 1}$
	$i_c = 1 - \frac{mH}{A' c_a N_c}$ For $\phi = 0°$
	$i_q = \left(1 - \frac{H}{V + A' c_a \cot \phi}\right)^m$
	$i_\gamma = \left(1 - \frac{H}{V + A' c_a \cot \phi}\right)^{m+1}$

$m = m_B = \frac{2 + B/L}{1 + B/L}$ When H parallel to B, $m = m_L = \frac{2 + L/B}{1 + L/B}$ When H parallel to L, If you have both H_B and H_L use $m = \sqrt{m_B^2 + m_L^2}$

Note: Use B and L not B' and L'

Bowles, 1997

IS code method (6403 -1981)

$$q_{nu} = cN_c s_c d_c i_c + q(N_q - 1)s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma W'$$

N_c, N_q, N_γ, are the same as those given by Vesic

W' – factor for water table

W' = 1, when water table is at or **below a depth of (D_f + B)** measured from the GL

W' = 0.5, when water table is located at a depth **D_f** or **likely to rise to the base of footing** or above

W' can be linearly interpolated **when $D_f < D_w < D_f + B$**

q = effective pressure at base

Shape Factor:

S_c	$\left(1+0.2\frac{B}{L}\right)$	Rectangular footing
	1.3	Square and Circular
S_q	$\left(1+0.2\frac{B}{L}\right)$	Rectangular footing
	1.2	Square and Circular
S_γ	$\left(1-0.4\frac{B}{L}\right)$	Rectangular footing
	0.8	Square
	0.6	Circular

Depth Factor:

d_c	$1+0.2\frac{D_f}{B}\tan\left(45^\circ+\frac{\phi}{2}\right)$	For any ϕ
d_q	$1+0.1\frac{D_f}{B}\tan\left(45^\circ+\frac{\phi}{2}\right)$	$\phi > 10^\circ$
	1	$\phi < 10^\circ$
d_γ	$1+0.1\frac{D_f}{B}\tan\left(45^\circ+\frac{\phi}{2}\right)$	$\phi > 10^\circ$
	1	$\phi < 10^\circ$

Inclination Factor:

i_c	$i_c = i_q = \left(1 - \frac{\alpha^{\circ}}{90^{\circ}}\right)^2$
$i_q = i_\gamma$	$i_\gamma = \left(1 - \frac{\alpha^{\circ}}{\phi^{\circ}}\right)^2$

Bearing capacity of granular soils based on SPT (Standard Penetration Test)

Teng (1962)

$$q_{nu} = \frac{1}{6}\left[3N^2BR_w' + 5(100+N^2)D_f R_w\right]$$

For strip footing

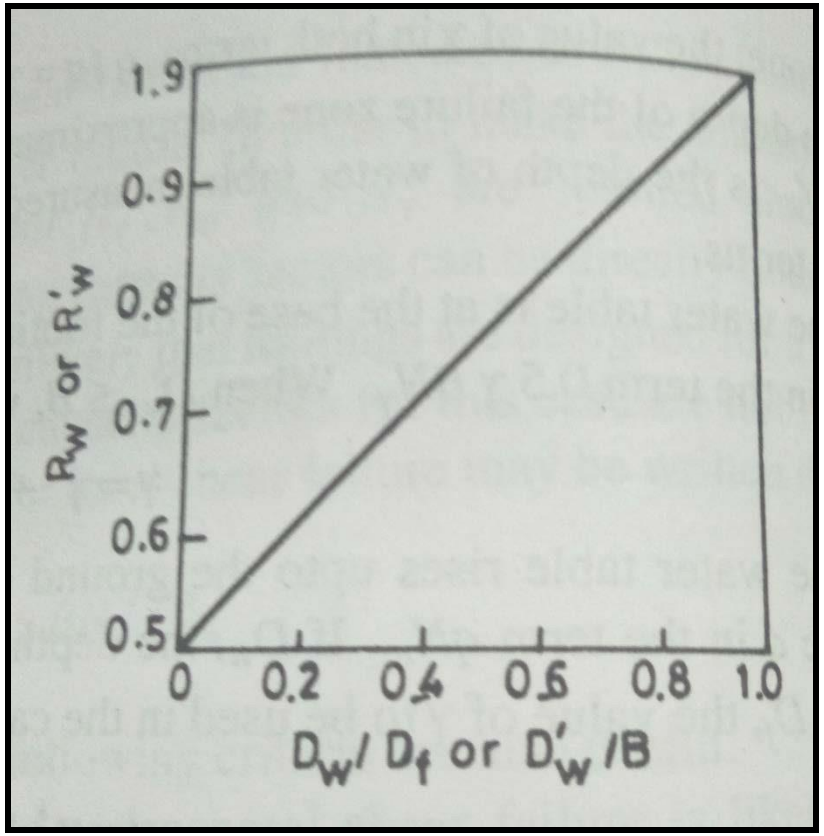
$$q_{nu} = \frac{1}{3}\left[N^2BR_w' + 3(100+N^2)D_f R_w\right]$$

For square and circular footing

q_{nu} = net ultimate bearing capacity in kN/m^2

N = average N value corrected for overburden pressure

D_f = depth of footing in m; if $D_f > B$ take $D_f = B$



Ranjan and Rao,1991

D_w = depth of water table below the ground surface limited to the depth equal to D_f
D'_w = depth of water table measured from base level of the footing with a limiting value equal to the width of footing B

Bearing capacity of footings on layered soils:

$$c_{avg} = \frac{c_1 H_1 + c_2 H_2 + \cdots + c_n H_n}{\Sigma H_i}$$

$$\phi_{avg} = \tan^{-1}\left(\frac{H_1 \tan \phi_1 + H_2 \tan \phi_2 + \dots\dots\dots + H_n \tan \phi_n}{\Sigma H_i} \right)$$

Factors influencing bearing capacity :

i) For $c_u = 0$

$$q_u = qN_q + 0.5\gamma BN_\gamma$$

a) Relative density or ϕ

b) Width of the footing

c) Depth of the footing

d) Unit weight of the soil

e) Position of ground water

ii) For ϕ = 0

$$q_u = c_u N_c + q$$

a) The bearing capacity of footing on a cohesive soil is unaffected by the **width of footing**

b) The **net ultimate bearing capacity** ($q_{nu} = N_c c_u$) is not affected by the **depth of foundation**.

c) **For ϕ = 0, N_c = 5.14 (smooth base) and 5.7 (rough base**)

Ex.1: A rectangular footing of size 3m X 6m is founded at a depth of 1m in a homogeneous sandy soil. The water table is at a great depth. The unit wt of soil 18 kN/m^3. Determine net ultimate bearing capacity c= 0 and ϕ = 40°

Using Terzaghi's theory

$$q_{nu} = q_u - \gamma D_f = \gamma D_f (N_q - 1) + \frac{1}{2}\gamma B N_\gamma \left(1 - 0.2\frac{B}{L}\right)$$

From table N_q = 81.3, N_γ = 100.4 for ϕ = 40°

B = 3m and L = 6m

$$q_{nu} = 18 \times 1 \times (81.3 - 1) + \frac{1}{2} \times 18 \times 3 \times 100.4 \times \left(1 - 0.2 \times \frac{3}{6}\right) = 3885.12 kN/m^2$$

Using Meyerhof's theory

$$q_{nu} = q_{ult} - \gamma D_f = \gamma D_f N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma - \gamma D_f$$

$$s_q = s_\gamma = 1 + 0.1\tan^2\left(45^\circ + \frac{\phi}{2}\right)\left(\frac{B}{L}\right) = 1.23$$

$$d_q = d_\gamma = 1 + 0.1\tan\left(45^\circ + \frac{\phi}{2}\right)\left(\frac{D_f}{B}\right) = 1.07$$

From table $N_q = 64.1$, $N_\gamma = 93.7$ for $\phi = 40^\circ$

$$q_{nu} = 18 \times 1 \times 64.1 \times 1.23 \times 1.07 + 0.5 \times 18 \times 3 \times 93.7 \times 1.23 \times 1.07 - 18 \times 1 = 4830.11 kN/m^2$$

Using Hansen's theory

$$q_{nu} = q_{ult} - \gamma D_f = \gamma D_f N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma - \gamma D_f$$

$$s_q = 1 + \sin(\phi)\left(\frac{B}{L}\right) = 1.32 \qquad s_\gamma = (1 - 0.4\frac{B}{L}) = 0.8$$

$$d_q = 1 + 2(\tan\phi)(1 - \sin\phi)^2\left(\frac{D_f}{B}\right) = 1.07 \qquad d_\gamma = 1$$

From table $N_q = 64.1$, $N_\gamma = 79.5$ for $\phi = 40°$

$$q_{nu} = 18 \times 1 \times 64.1 \times 1.32 \times 1.07 + 0.5 \times 18 \times 3 \times 79.5 \times 0.8 \times 1 - 18 \times 1 = 3328.82 kN/m^2$$

Using Vesic's theory

$$q_{nu} = q_{ult} - \gamma D_f = \gamma D_f N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma - \gamma D_f$$

$$s_q = 1 + \tan(\phi)\left(\frac{B}{L}\right) = 1.41$$

$$s_\gamma = (1 - 0.4\frac{B}{L}) = 0.8$$

$$d_q = 1 + 2(\tan\phi)(1-\sin\phi)^2\left(\frac{D_f}{B}\right) = 1.07$$

$$d_\gamma = 1$$

From table $N_q = 64.1$, $N_\gamma = 109.4$ for $\phi = 40°$

$$q_{nu} = 18\times1\times64.1\times1.41\times1.07 + 0.5\times18\times3\times109.4\times0.8\times1 - 18\times1 = 4085.77 kN/m^2$$

Using IS Code Method

$$q_{nu} = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

$$s_q = 1 + 0.2\left(\frac{B}{L}\right) = 1.10 \qquad s_\gamma = (1 - 0.4\frac{B}{L}) = 0.8$$

$$d_q = 1 + 0.1(\frac{D_f}{B})\tan(45 + \frac{\phi}{2}) = 1.07 \qquad d_\gamma = 1$$

$N_q = 64.1$, $N_\gamma = 109.4$ for $\phi = 40°$ (same as Vesic)

$$q_{nu} = 18 \times 1 \times (64.1 - 1) \times 1.10 \times 1.07 + 0.5 \times 18 \times 3 \times 109.4 \times 0.8 \times 1 = 3699.87 kN / m^2$$

Author	q_{nu} kN/m^2
Terzaghi	3885.12
Meyerhof	4830.11
Hansen	3328.82
Vesic	4085.77
Is code	3699.87

Meyerhof 's method gives higher value of q_{nu} than all other methods

Ex.2: A rectangular footing of size 3m X 6m is founded at a depth of 1m in a homogeneous sandy soil. The water table is at a great depth. The unit wt of soil 18 kN/m³. Determine net ultimate bearing capacity. c= 0 and $\phi = 22°$.

Using Terzaghi's theory

$$q_{nu} = q_u - \gamma D_f = \gamma D_f (N'_q - 1) + \frac{1}{2}\gamma B N'_\gamma \left(1 - 0.2\frac{B}{L}\right)$$

$$\phi' = \tan^{-1} 0.67(\tan(22^0)) = 15^0$$

From table $N_q = 4.4$, $N_\gamma = 2.5$ for $\phi' = 15°$ (local shear failure)

B = 3m and L = 6m

$$q_{nu} = 18\times1\times(4.4-1) + \frac{1}{2}\times18\times3\times2.5\times\left(1-0.2\frac{3}{6}\right) = 121.95 kN/m^2$$

ϕ	Terzaghi's Bearing Capacity Factor		
	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
50	347.5	415.1	1153.2

Ranjan and Rao, 1991

Ex.3: A rectangular footing of size 3m X 6m is founded at a depth of 1m in a homogeneous sandy soil. The water table is at a great depth. The unit wt of soil 18 kN/m ^{3}. c= 0 and $\phi = 35°$. Determine net ultimate bearing capacity.

N_q = 41.4, N'_q = 12.7 for ϕ_m = 25°. Hence a ctual,

$$\bar{N}_q = 12.7 + (41.4 - 12.7) \times \left(\frac{35-29}{36-29} \right) = 37.3$$

N_γ = 42.4, N'_γ = 9.7. Hence actual,

$$\bar{N}_\gamma = 9.7 + (42.4 - 9.7) \times \left(\frac{35-29}{36-29} \right) = 37.72$$

$$\boxed{q_{nu} = q_u - \gamma D_f = \gamma D_f (\bar{N}_q - 1) + \frac{1}{2} \gamma B \bar{N}_\gamma \left(1 - 0.2 \frac{B}{L} \right)}$$

$$q_{nu} = 18 \times 1 \times (37.3 - 1) + \frac{1}{2} \times 18 \times 3 \times 37.72 \times (1 - 0.2 \times \frac{3}{6}) = 1569.99 kN / m^2$$

Ex.4: A rectangular footing of size 3m X 6m is founded at a depth of 1m in a homogeneous c-ϕ soil. The water table is at a great depth. The unit wt of soil 18 kN/m^3. Determine net ultimate bearing capacity. c= 50 kPa and $\phi = 20°$.

$$q_{nu} = q_u - \gamma D_f = cN_c\left(1+0.3\frac{B}{L}\right) + \gamma D_f(N_q - 1) + \frac{1}{2}\gamma B N_\gamma\left(1-0.2\frac{B}{L}\right)$$

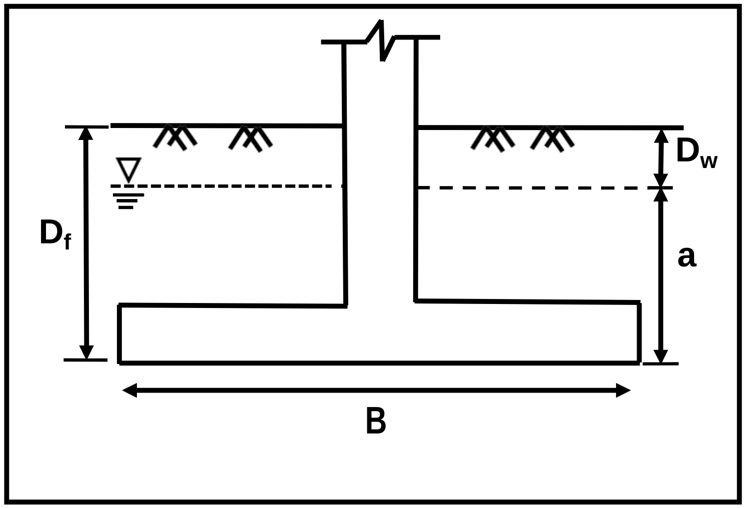
From table N_c = 17.7, N_q = 7.4, N_γ = 5 for ϕ = 20°
B = 3m and L = 6m

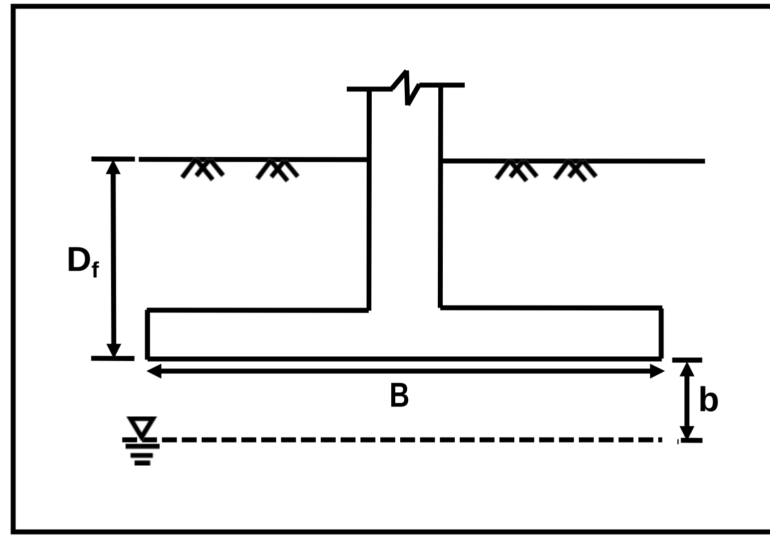
$$q_{nu} = 50\times17.7\times\left(1+0.3\times\frac{3}{6}\right) + 18\times1\times(7.4-1) + \frac{1}{2}\times18\times3\times5\times\left(1-0.2\times\frac{3}{6}\right) = 1254.45 kN/m^2$$

Shallow Foundation : Settlement-I

Ex.:

$$q_u = c_u N_c + \left[\gamma' D_f + (\gamma - \gamma')D_w\right] N_q + \frac{1}{2}\gamma' B N_\gamma$$

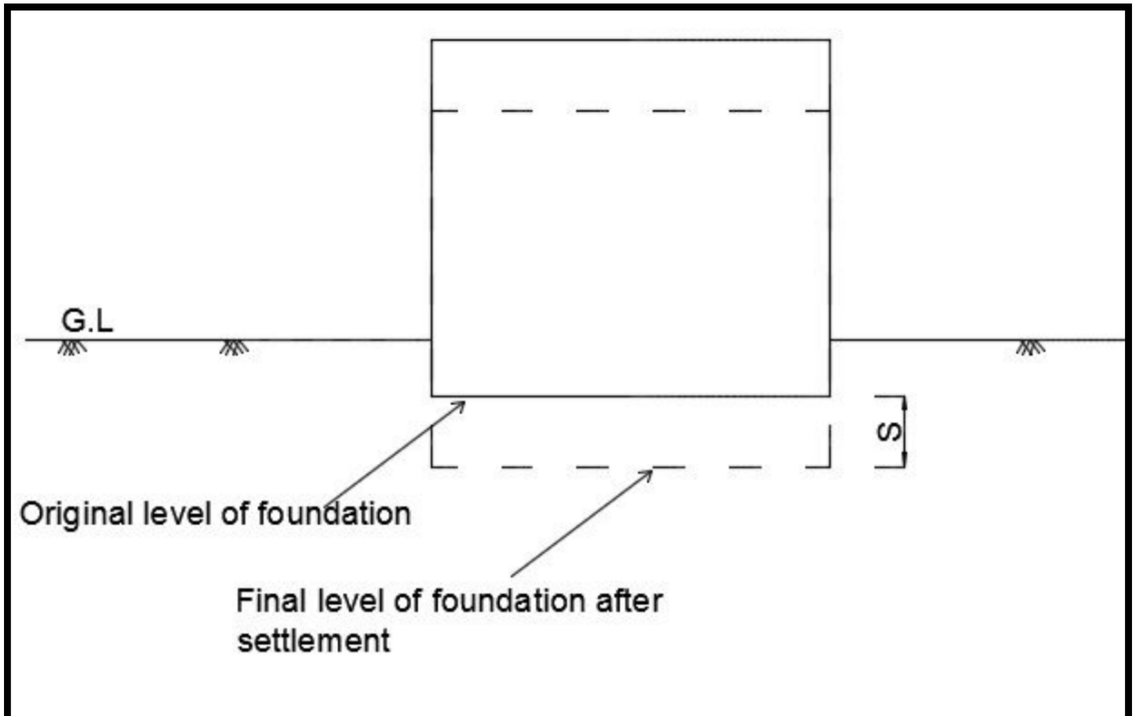




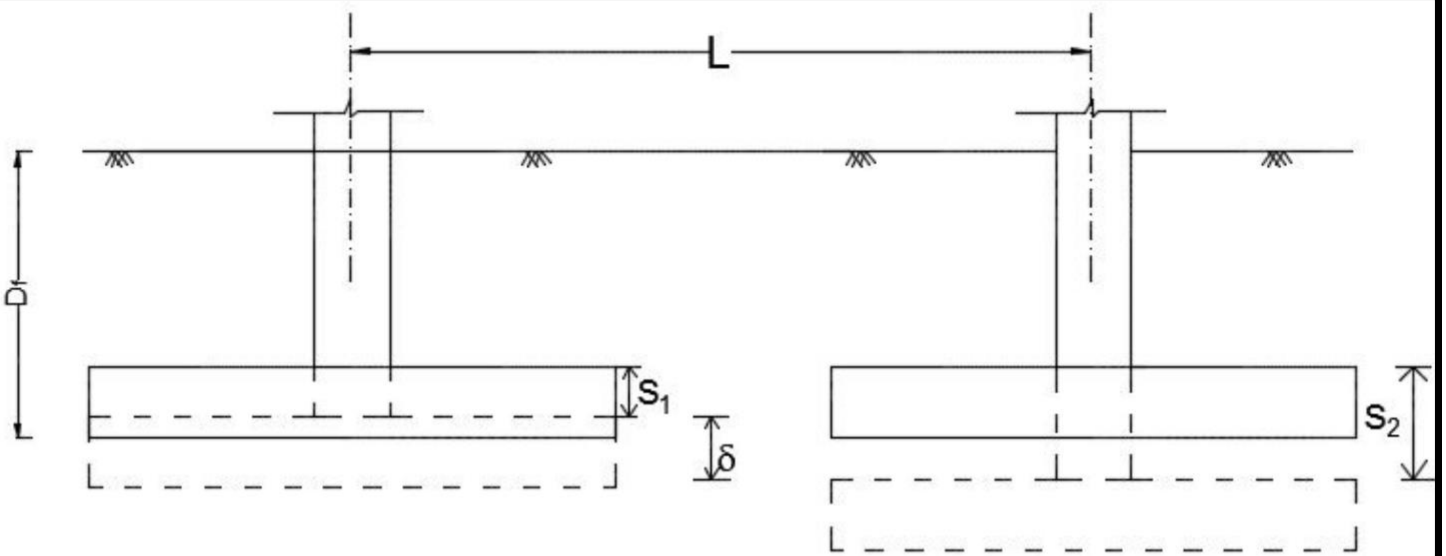
$$\bar{\gamma} = \gamma' + \frac{b}{B}(\gamma - \gamma')$$

$$q_u = c_u N_c + \gamma D_f N_q + \frac{1}{2} B \left[\gamma' + \frac{b}{B}(\gamma - \gamma') \right] N_\gamma$$

Types of Settlement found in shallow foundation

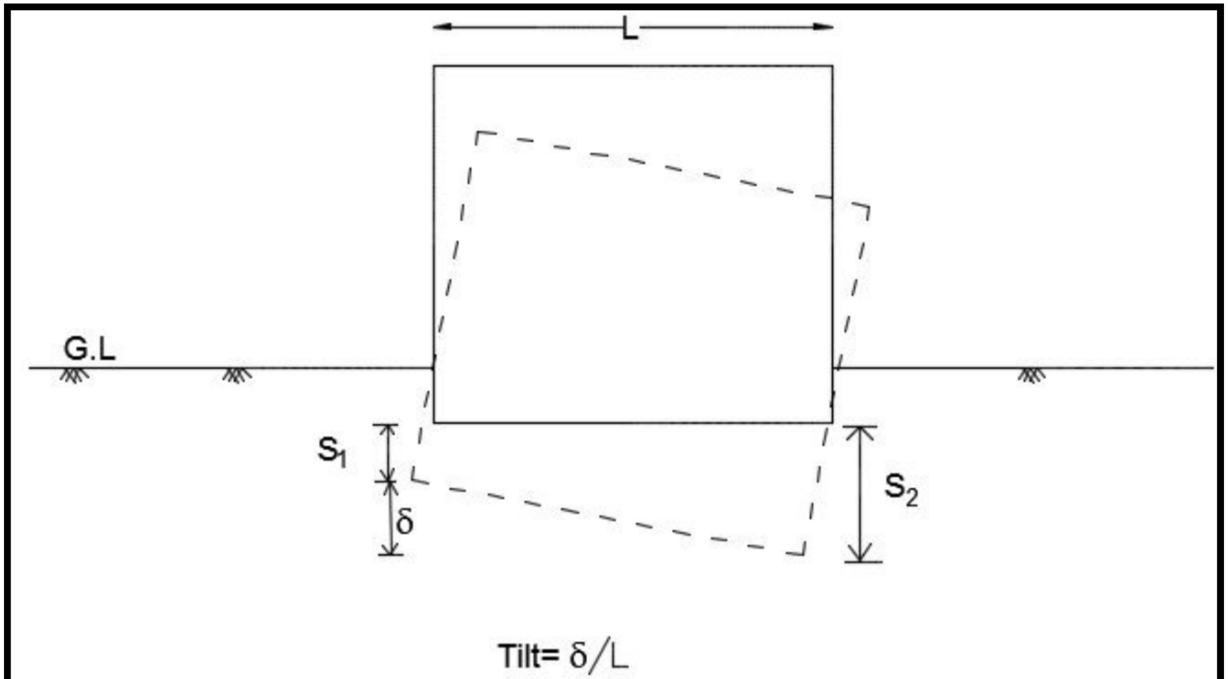


(a) Uniform settlement



Angular distortion= δ / L

(b) Angular distortion



(c) Tilt

Settlement of shallow foundation

Total Settlement

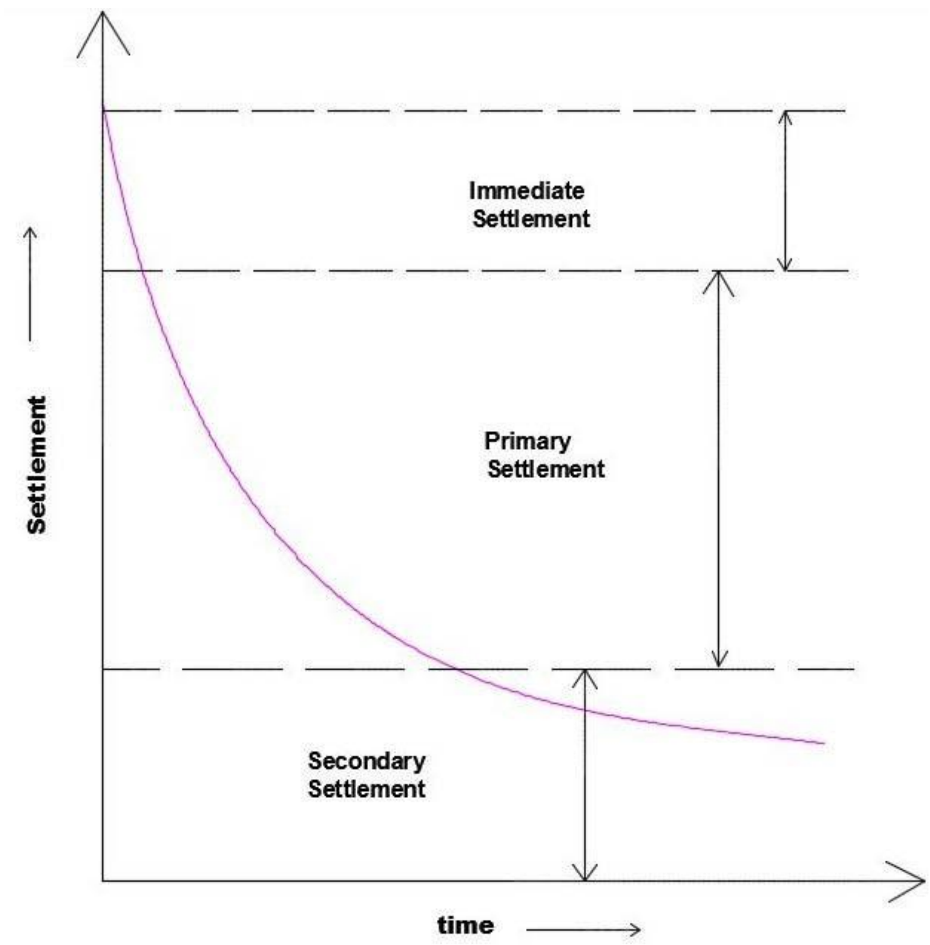
$$S_t = S_i + S_c + S_s$$

S_i= Immediate or elastic settlement (<7 days). It takes place during the application of loading.
In clays, the settlement is due to the change in the shape of the soil without a change in volume or water content. It is neglected as compared to long term settlement.

S_c= Primary consolidation settlement. It is due to the consolidation.

S_s= Secondary Compression Settlement. It occurs because of volume change occurring due to rearrangement of soil particles.

- Immediate settlement is not time dependant settlement.
- Primary consolidation and secondary settlement are time dependant.
- For granular soils, immediate settlement is the entire settlement.
- In inorganic clays, Primary consolidation accounts major part of the settlement.
- In organic clays, secondary compression accounts major part of the settlement .



1. Immediate or elastic settlement

$$S_i = qB\left(\frac{1-\mu^2}{E}\right)I_f$$

where q= Net foundation pressure

μ= Poisson's ratio

E= Elastic Modulus of soil

I_f= Influence factor

Types of corrections: 1. Depth correction
2. Rigidity correction for raft foundation

Shape	I_f for Flexible Foundation			I_f for Rigid Foundation
	Centre	Corner	Average	
Circle	1.0	0.64	0.85	0.86
Square	1.12	0.56	0.95	0.82
Rectangle				
L/B= 1.5	1.36	0.68	1.2	1.06
L/B= 2	1.52	0.76	1.3	1.2
L/B= 5	2.10	1.05	1.83	1.70
L/B= 10	2.52	1.26	2.25	2.10
L/B= 100	3.38	1.69	2.96	3.40

Ranjan and Rao, 1991

Types of soil	μ
1. Clay , saturated	0.4-0.5
2. Clay, unsaturated	0.1-0.3
3. Sandy clay	0.2-0.3
4. Silt	0.3-0.35
5. Sand(dense)	
5.1 Coarse(e=0.4-0.7)	0.15
5.2 Fine grained	0.25
6. Rock	0.1-0.4

Ranjan and Rao, 1991

Young's Modulus Calculation

Type of soil	SPT (N) or CPT(q_c)
Sand (NC)	E= 500(N+15)
Sand (OC)	E= 250(N+15)
Sand(Saturated)	E= 250(N+15)
Gravely Sand	E= 1200(N+6)
Clayey sand	E= 320(N+15)
Silty sand	E= 300(N+6)
Soft clay	E= 5 to 8 q_c

Ranjan and Rao , 1991

* E is in kN/m2.

Elastic Modulus Calculation

- Normally consolidate clay, E_u= (750 to 1200) S_u
- Heavily overconsolidated clay, E_u= (1500 to 2000) S_u
- Normally consolidated sensitive clay, E_u= (200 to 600)S_u

Elastic Modulus Calculation

Soil type	E kg/ cm²)
Clay	
1. Very soft	20-150
2. soft	50-250
3. medium	150-500
4. Hard	500-1000
5. Sandy	250-2500

Soil type	E (kg/ cm²)
Sand	
1. silty	70-210
2. loose	100-240
3.dense	480-800

Soil type	E (kg/ cm²)
Sand and gravel	
1. Loose	500-1450
2. Dense	1000-1900

Ranjan and Rao, 1991

2. Consolidation settlement

Consolidation settlement

$$S_c = \sum \frac{C_c}{1+e_0} H \log_{10}\left(\frac{p_0 + \Delta p}{p_0}\right)$$

$$or\ S_c = \sum m_v H_0 \Delta p$$

Where p_0 = initial effective overburden pressure before applying foundation load

Δp= vertical stress at the centre due to application of load

C_c= Compression index

e_0= initial void ratio

m_v= coefficient of volume compressibility

Types of corrections: 1. Depth correction
2. Rigidity correction for raft foundation
3. Pore water pressure correction

Corrections

1. Corrections for the effect of 3-D consolidation

$$S_{c(3D)} = \eta S_{c\,(1D)}$$

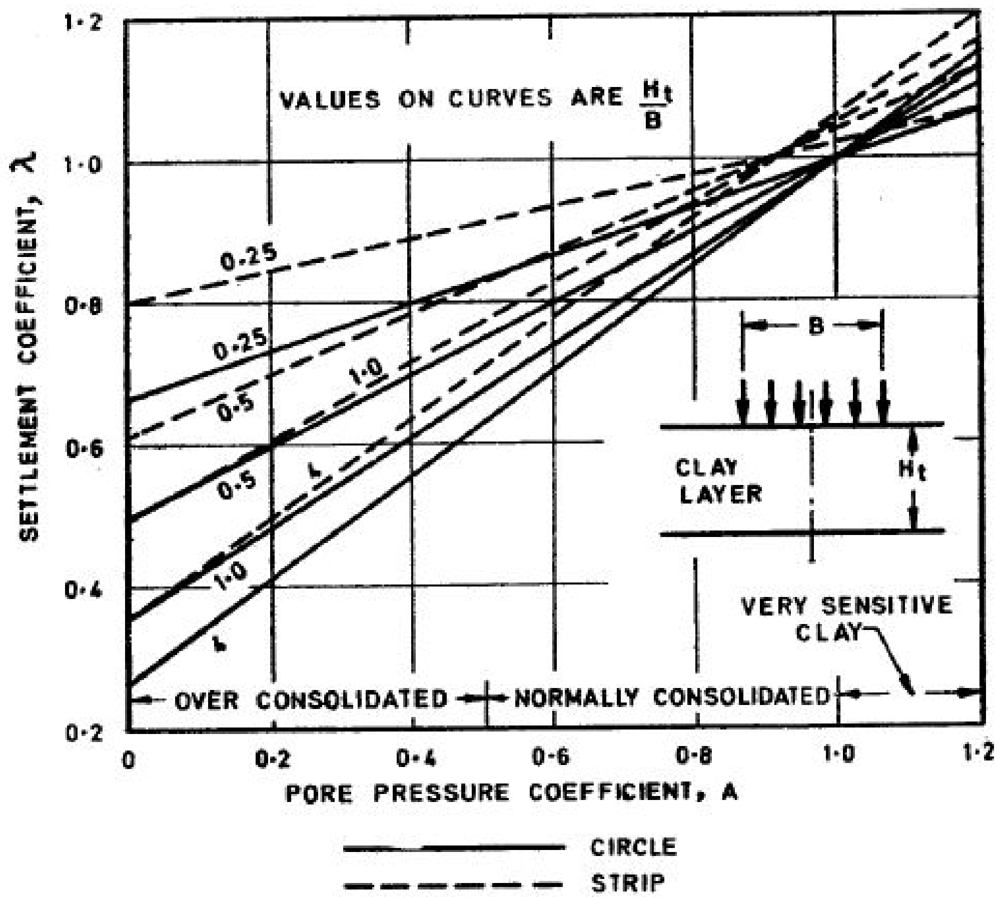
where η= correction factor. In absence of data regarding pore water pressure parameter A, following values can be taken:

η = 1-1.2 very sensitive clay

=0.7-1.0 Normally consolidated clay

=0.5-0.7 Over consolidated clay

=0.3-0.5 Heavily over consolidated clay



IS :8009 (Part I) -1976

Shallow Foundation : Settlement-II

2. Consolidation settlement

Consolidation settlement

$$S_c = \sum \frac{C_c}{1+e_0} H \log_{10}\left(\frac{p_0 + \Delta p}{p_0}\right)$$

$$or \;\; S_c = \sum m_v H_0 \Delta p$$

where p_0 = initial effective overburden pressure before applying foundation load

Δp= vertical stress at the centre due to application of load

C_c= Compression index

e_0= initial void ratio

m_v= coefficient of volume compressibility

Types of corrections: 1. Depth correction
2. Rigidity correction for raft foundation
3. Pore water pressure correction

Corrections

1. Corrections for the effect of 3-D consolidation

$$S_{c(3D)} = \eta S_{c\,(1D)}$$

where μ= correction factor. In absence of data regarding pore water pressure parameter A, following values can be taken:

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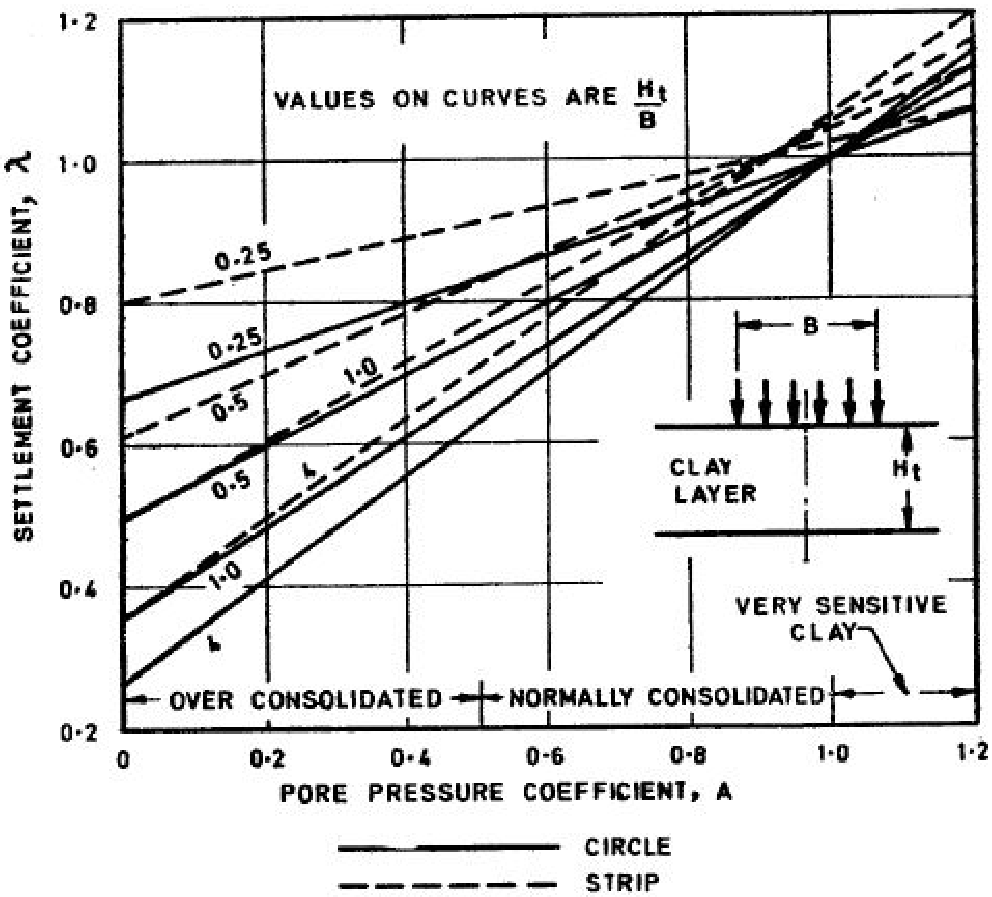

IS :8009 (Part I) -1976

2. Corrections for the rigidity of foundation

$$Rigidity\ factor = \frac{Total\ settlement\ of\ rigid\ foundation}{Total\ settlement\ at\ centre\ of\ flexible\ foundation}$$

Correction factor= 0.8 for rigid foundation

3. Corrections for the depth of the embedment

$$Depth\ factor = \frac{S_{embedded}}{S_{surface}}$$

Fox's correction for settlement of flexible rectangular footing of L x B at a depth D

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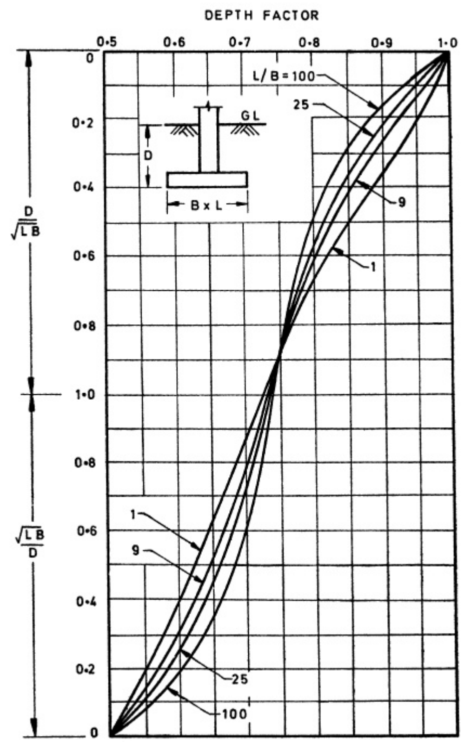


FIG. 12 Fox's Correction Curves for Settlements of Flexible Rectangular Footings of $L \times B$ at Depth D

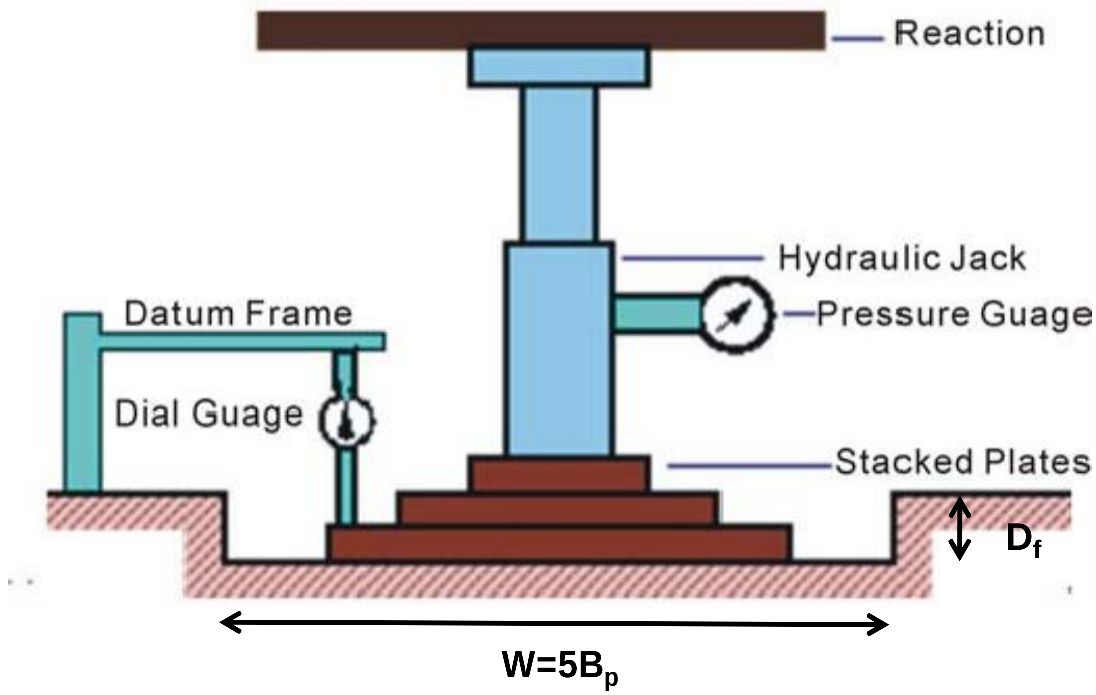
22

IS :8009 (Part I) -1976

Settlement of Foundations on Granular Soils

- Due to consolidation, short term field tests are not suitable to determine the settlement of cohesive soil.

a) Plate load test method (IS-1888-1982)




DRILLING

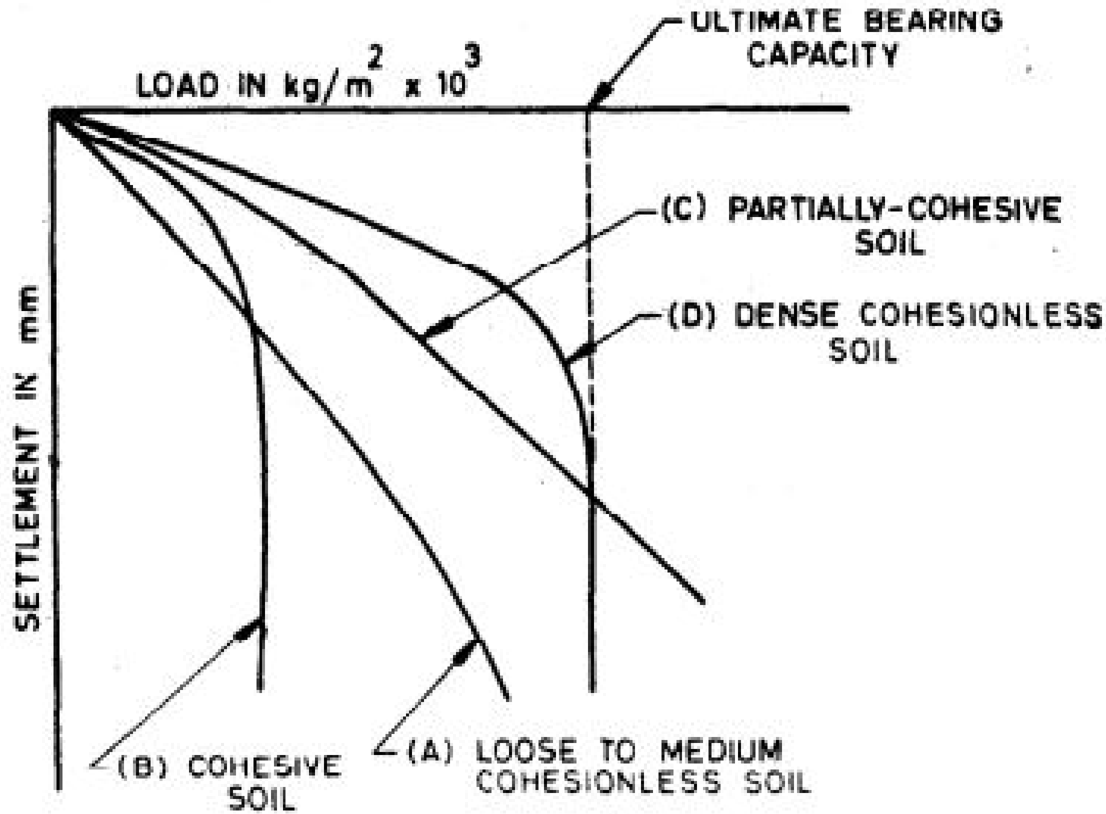
Procedure

- Rough mild steel plates of size 30cm,45 cm,60cm,or 75 cm , square or circular in shape are generally used.
  - 5mm (maximum thickness) fine sand is placed before placing the plate.
  - Smaller sizes are used for dense or stiff soil.
  - larger size are used for loose or soft soil.
  - Water is removed by pumping out.
- Loads on the test plate may be applied by gravity loading or reaction loading.
- Seating load of $70 kg/cm^2$ is first applied and released after sometimes.

- Load is applied at 1/5th the estimated safe load up to failure or at least 25mm settlement, whichever is earlier.
- At each load, settlement is recorded at time intervals of 1, 2.25, 4, 6.25, 9, 16 and 25 mins and thereafter at hourly interval.
  - For clayey soils, the load is increased when the time-settlement curve indicates that settlement has exceeded 70-80 % of the probable ultimate settlement or at the end of 24 hours.
  - For other soils, the load is increased when the rate of settlement drops to a value less than 0.02 mm/min.

IS:1888-1982

- Settlement are recorded through a minimum of two dial gauges mounted on independent datum and resting diametrically opposite ends of the plates.
- The load settlement curve for the test plate can be plotted from the test data.


ULTIMATE BEARING CAPACITY
LOAD IN kg/m² x 10³
(C) PARTIALLY-COHESIVE SOIL
(D) DENSE COHESIONLESS SOIL
SETTLEMENT IN mm
(B) COHESIVE SOIL
(A) LOOSE TO MEDIUM COHESIONLESS SOIL

IS 1888-1982

Settlement Calculation from plate load test

- Terzaghi and Peck (1948):

$$\frac{S_f}{S_p} = \left[\frac{B_f\left(B_p + 30\right)}{B_p\left(B_f + 30\right)}\right]^2$$

(For granular soil)

Where S_f = settlement of a foundation of width B_f (cm)
S_p = settlement of a foundation of width B_p (cm) at the same load intensity as on the foundation

- Bjerrum and Eggestad (1963):

$$\frac{S_f}{S_p} = \frac{4}{\left(1 + \frac{D_p}{D_f}\right)^2}$$

where D_p = diameter of plate
D_f = diameter of footing

Important Considerations

- Plate size smaller than 30 cm should never be used in any case.
- It may lead to misleading results, if the soil at site is not homogenous.
- Capillarity in sand bed increases its effective vertical stress or its stiffness. The test will result in a severe underestimate of actual settlement.
- For clayey soil, immediate settlement is not the main settlement. However, plate load test gives the immediate test.

$$\frac{S_f}{S_p} = \frac{B_f}{B_p}$$

Ultimate Bearing capacity Calculation from plate load test

- For cohesionless soil

$$\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}$$

- For cohesive soil

$$q_{uf} = q_{up}$$

Where, q_{uf}= ultimate bearing capacity of footing

q_{up}= ultimate bearing capacity of plate

Safe Bearing capacity Calculation from plate load test

- The safe bearing capacity of a footing can be determined from the load-settlement curve of the test plate.
- If the permissible settlement of foundation of width B_f is S_p, corresponding settlement S_p of test plate B_p can be found from equation given earlier. Then the load intensity corresponding to S_p is read from load settlement curve and taken as safe bearing capacity of foundation.

Safe Bearing capacity Calculation from plate load test

- If the load test is carried out above the natural water table, the settlement computed from the curve will have to be corrected if there is a likelihood of rise in water table in future.

$$\text{Actual settlement} = \frac{\text{Settlement computed from plateload test}}{\text{Correction factor } (C_w)}$$

Peck, Hanson, and Thornburn (1974)

$$C_w = 0.5 + 0.5\left(\frac{D_w}{D_f + B}\right)$$

D_w= depth of water table below the ground level
D_f= depth of foundation
B= width of foundation

IS:8009 method

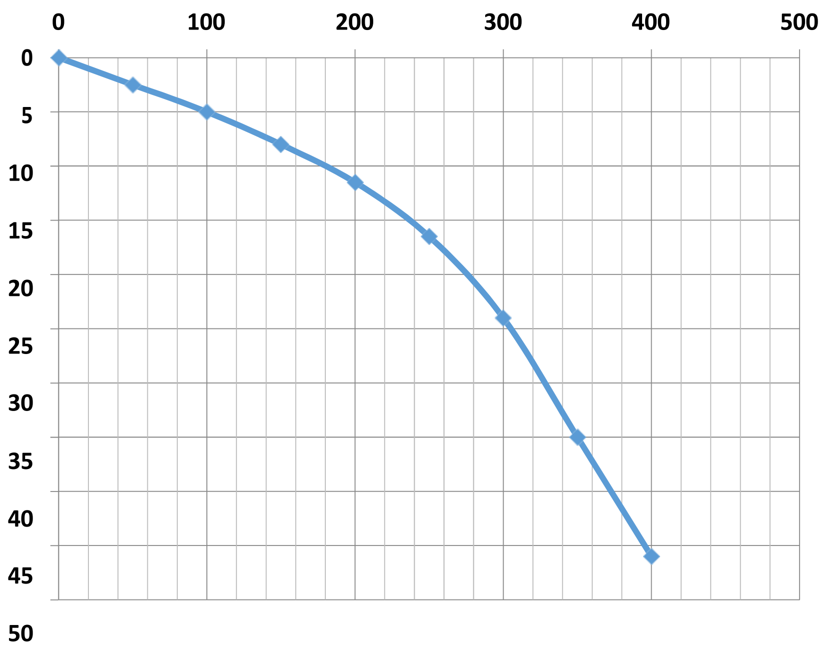
$$C_w = 0.5 + 0.5\left(\frac{D'_w}{B}\right) \leq 1$$

D'_w= depth of water table from base of footing

Shallow Foundation : Settlement-III

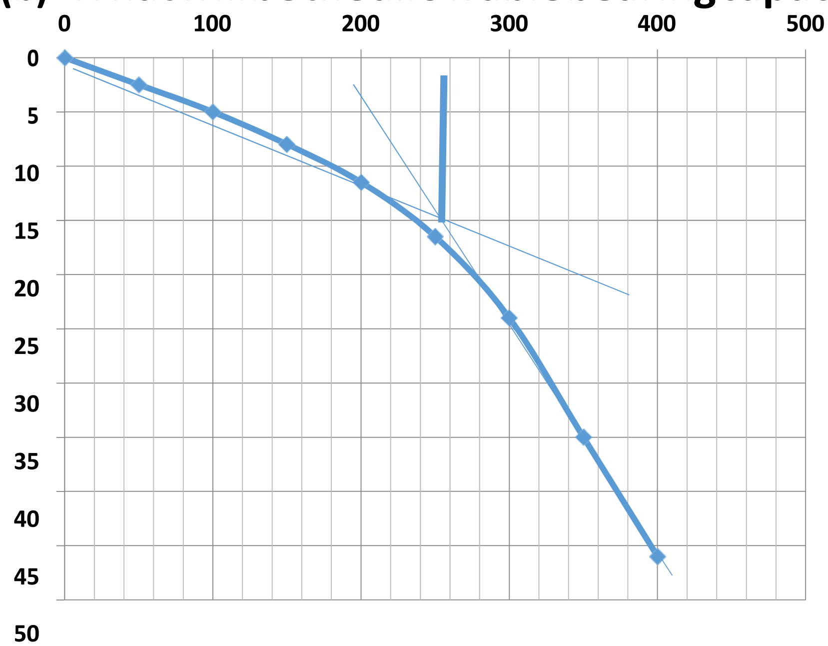
Example (a) :The following data was obtained from a plate load test conducted on 60 cm square test plate at a depth of 2 m below the ground level on a sandy soil which extends up to large depth.
Determine the settlement of a foundation 4 m x 4 m carrying a load of 1200 kN placed at a depth of 2 m below ground surface on the same soil.
(b) What will be the actual settlement if water table is raised at the base of the footing. Load test data:

Load intensity (kN/m^2)	Settlement (mm)
50	2.5
100	5.0
150	8.0
200	11.5
250	16.5
300	24.0
350	35.0
400	46.0


0
100
200
300
400
500
0
5
10
15
20
25
30
35
40
45
50

Example (a): Using the same plate load test data determine the allowable bearing capacity of a foundation 3m x 3m placed at a depth of 2 m below ground surface on the same soil. Permissible settlement of the foundation is 50 mm and factor of safety against bearing is 2.5. Unit weight of the soil is 19 kN/m³.

(b) What will be the allowable bearing capacity if water table is raised at the base of the footing.

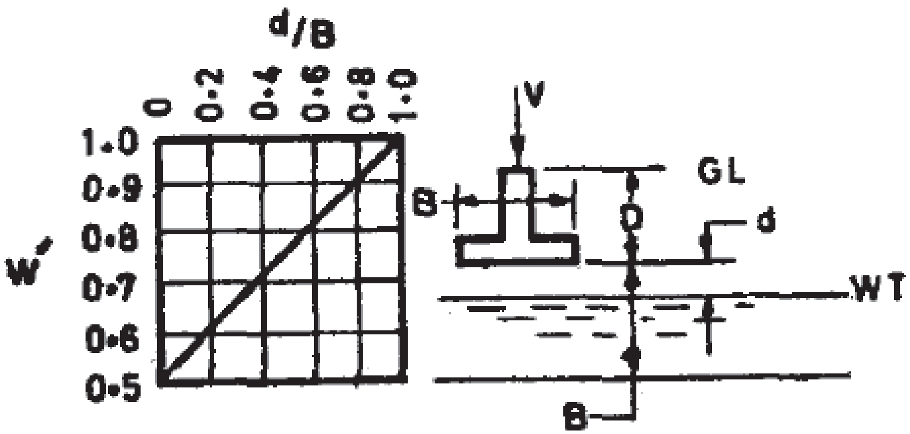


ϕ	Terzaghi's Bearing Capacity Factor		
	N_c	N_q	N_γ
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5

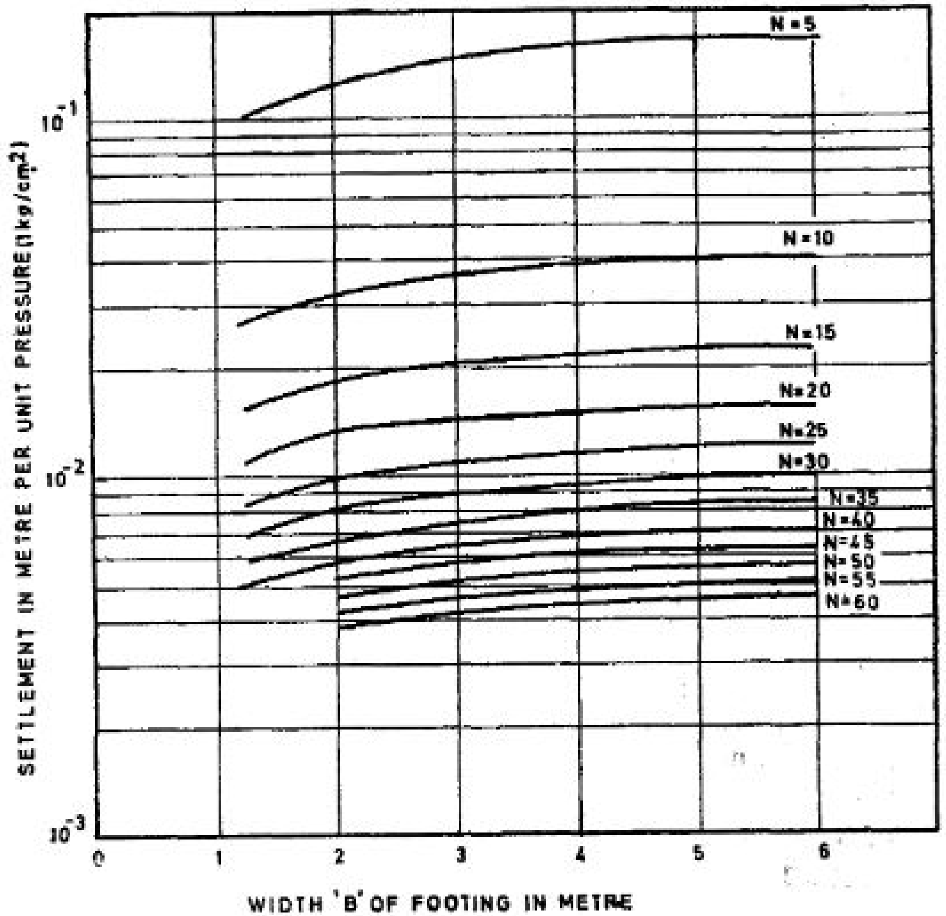
Ranjan and Rao, 1991

Shallow Foundation : Settlement-IV

(b) Method based on SPT (IS 8009-Part 1-1976)



$$\text{Actual settlement} = \frac{\text{Settlement computed from SPT value}}{\text{Correction factor } (W')}$$



(c) Method based on SCPT

- De Beer and Martens (1957) used the static cone penetration resistance diagram to predict the settlement of a structure on sands

First

- Sand stratum is divided into no. of layers such that each layer has same value of CPT resistance.

Second

- The average value for each layer is chosen.

Third

- Compressibility coefficient, C is related to static cone resistance, q_c and effective overburden pressure $\bar{\sigma}_0$, at depth at which test is carried out.

The relationships suggested are:

$$C = 1.5\left(\frac{q_c}{\sigma_0}\right)$$

The settlement for each layer is given by :

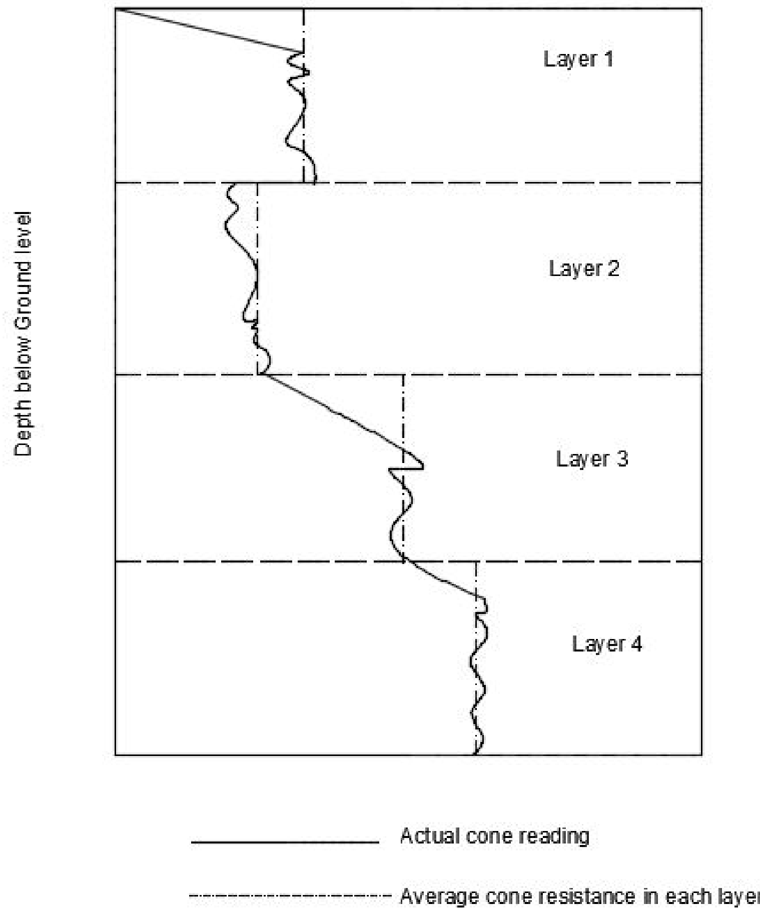
$$S = 2.3\frac{H}{C}\log\left(\frac{\bar{\sigma}_0 + \Delta\sigma}{\bar{\sigma}_0}\right)$$

where H= thickness of layer

Δσ= increase in vertical stress at middle of the layer

<u>Meyerhof(1965)</u>

$$C = 1.9\left(\frac{q_c}{\sigma_0}\right)$$



(d) Semi-empirical Method (Buisman, 1948)

$$S = \sum 2.3 \frac{\overline{\sigma}_0}{E} H \log\left(\frac{\overline{\sigma}_0 + \Delta\sigma}{\overline{\sigma}_0} \right)$$

where H= thickness of layer

Δσ= increase in vertical stress at middle of the layer

E = Elastic Modulus of each soil layer

Settlement Calculation

Immediate Settlement (for clay)

$$S_i = qB\left(\frac{1-\mu^2}{E}\right)I_f$$

Consolidation Settlement (for clay)

$$S_c = \sum \frac{C_c}{1+e_0} H \log_{10}\left(\frac{p_0+\Delta p}{p_0}\right)$$

$$or\ \ S_c = \sum m_v H_0 \Delta p$$

Settlement (granular soil or sand) (all Immediate Settlement)

(a) Plate load test method (IS-1888-1982)

(b) Method based on SPT (IS 8009-Part 1-1976)

(c) Method based on SCPT $S = 2.3\frac{H}{C}\log\left(\frac{\bar{\sigma}_0+\Delta\sigma}{\bar{\sigma}_0}\right)$ where $C = 1.5\left(\frac{q_c}{\sigma_0}\right)$ (De Beer and Martens (1957)) or $C = 1.9\left(\frac{q_c}{\sigma_0}\right)$ (Meyerhof(1965))

(d) Semi-empirical Method (Buisman, 1948) $S = \sum 2.3\frac{\bar{\sigma}_0}{E} H\log\left(\frac{\bar{\sigma}_0+\Delta\sigma}{\bar{\sigma}_0}\right)$

Shallow Foundation : Settlement-V

Settlement Calculation

Immediate Settlement (for clay)

$$S_i = qB\left(\frac{1-\mu^2}{E_f}\right)I_f$$

Consolidation Settlement (for clay)

$$S_c = \sum \frac{C_c}{1+e_0} H \log_{10}\left(\frac{p_0+\Delta p}{p_0}\right)$$

$$or \; S_c = \sum m_v H_0 \Delta p$$

Settlement (granular soil or sand) (all Immediate Settlement)

(a) Plate load test method (IS-1888-1982)

(b) Method based on SPT (IS 8009-Part 1-1976)

(c) Method based on SCPT

$$S = \sum 2.3 \frac{H}{C} \log\left(\frac{\bar{\sigma}_0+\Delta\sigma}{\bar{\sigma}_0}\right)$$

where

De Beer and Martens (1957): $C = 1.5\left(\frac{q_c}{\sigma_0}\right)$

or

Meyerhof(1965): $C = 1.9\left(\frac{q_c}{\sigma_0}\right)$

(d) Semi-empirical Method (Buisman, 1948)

$$S = \sum 2.3 \frac{\bar{\sigma}_0}{E} H \log\left(\frac{\bar{\sigma}_0+\Delta\sigma}{\bar{\sigma}_0}\right)$$