Bearing Capacity Of Shallow Foundations

Bearing Capacity of Shallow Foundation

- **A foundation is required for distributing the loads of the superstructure on a larger area. The foundation should be designed such that**

  a) The soil below does not fail in shear &

  b) Settlement is within the safe limits.

Basic Definitions :

1) Gross Ultimate Bearing Capacity (q_u):

The ultimate bearing capacity is the gross pressure at the base of the foundation at which soil fails in shear.

2) Net Ultimate Bearing Capacity (q_{nu}) :

It is the net increase in pressure at the base of foundation that cause shear failure of the soil.

Thus, $q_{nu} = q_u - \gamma D_f$ (ovrbruden pressure)

Basic Definitions :

3) Gross Safe Bearing Capacity (q_s) :

It is the maximum pressure which the soil can carry safely without shear failure at the base of foundation.

$$q_s = q_{nu} / FOS + \gamma D_f$$

4) Net Safe Bearing Capacity (q_{ns}) :

It is the net soil pressure which can be safely applied to the soil considering only shear failure.

Thus, $q_{ns} = q_{nu} / FOS$

FOS - Factor of safety usually taken as 2.0 - 3.0

Basic Definitions :

5) Safe Settlement Pressure (q_{sp}) :

It is the net pressure which the soil can carry without exceeding allowable/permissible settlement.

6) **Net Allowable Bearing Pressure (qna):**

It is the net bearing pressure which can be used for design of foundation satisfying both bearing capacity and settlement criteria.

Thus,

$$q_{na} = q_{ns} \quad ; \text{if } q_{sp} > q_{ns}$$

$$q_{na} = q_{sp} \quad ; \text{if } q_{ns} > q_{sp}$$

It is also known as Allowable Soil Pressure (ASP) or Allowable bearing Capacity (ABC)

Modes of Bearing Capacity Failure :

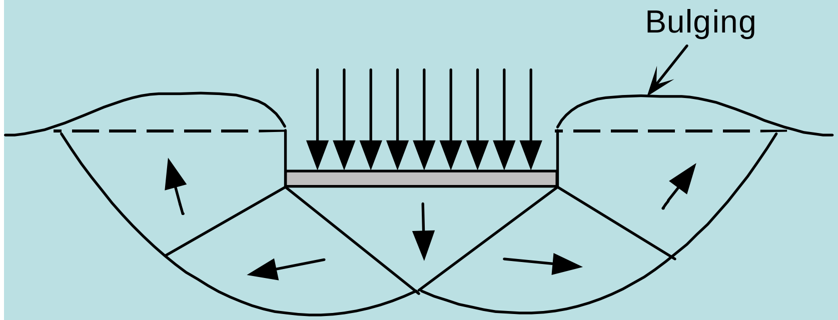
- **Terzaghi (1943) classified shear failure of soil under a foundation base into following two modes 1 & 2 and then Vesic (1963) added the mode 3 depending on the type of soil & location of foundation.**

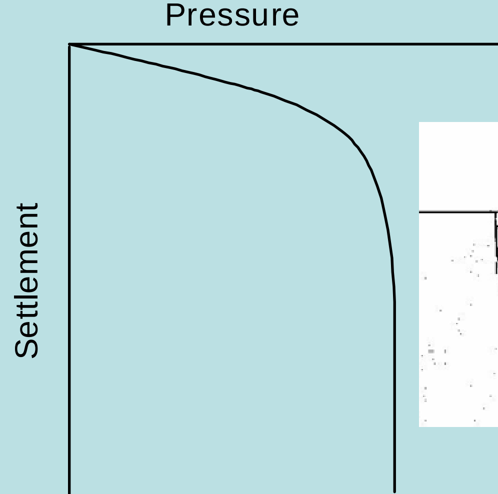
  1) General Shear failure.

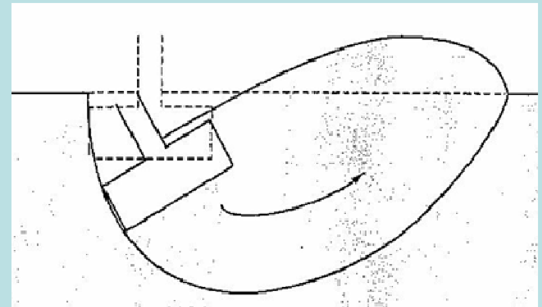
  2) Local Shear failure.

  3) Punching Shear failure **{Vesic (1963) added}**

General Shear failure

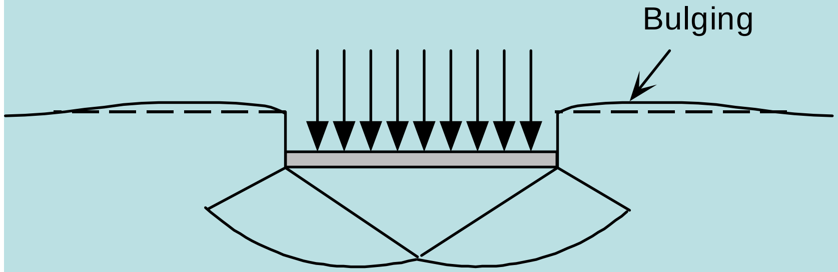


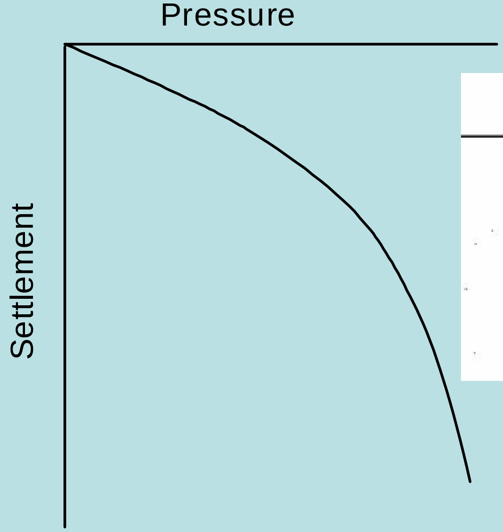


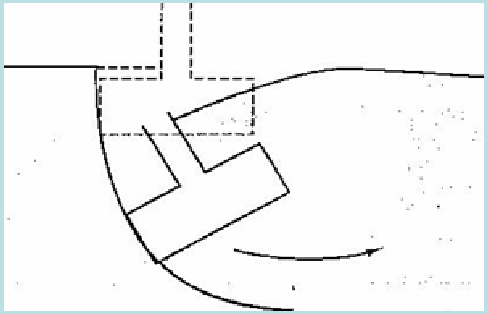


- Applicable to narrow footings placed at shallow depth on dense or over-consolidated cohesive soils of low compressibility.
- Continuous failure surfaces are developed between footing edges and ground surface.
- Soil around the footing bulges out
- Failure is sudden accompanied by tilting of footing
- GSF is common under undrained conditions
- UBC is well defined from the pressure-settlement curve.

Local Shear Failure

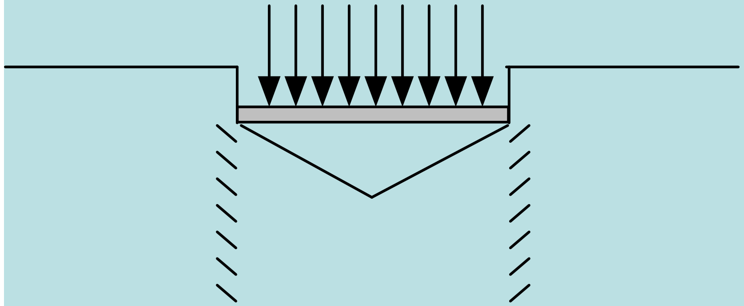


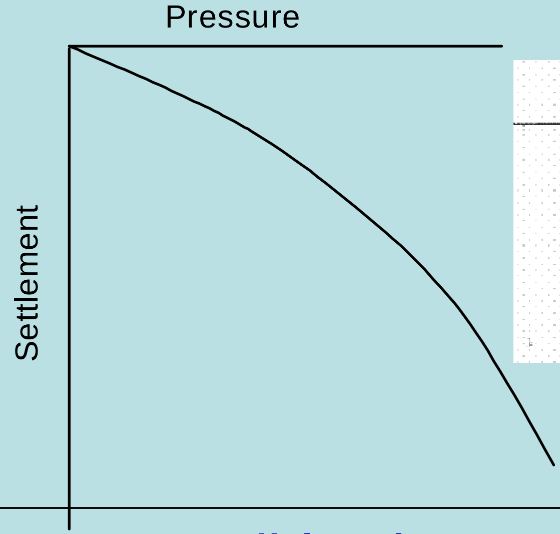


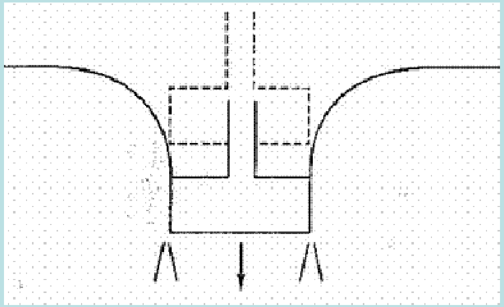


- Occurs in soils of high compressibility
- Slip surfaces/lines well defined below the footing only
- Slip lines extends only a short distance into the soil mass
- Slight heaving occurs
- Little tilting of the foundation at relatively large settlement
- UBC in not well defined from the settlement-pressure graph
- Usually settlement is the main design criterion.

Punching Shear Failure







- This failure occurs in highly compressible clays, silts and in loose sands when footing is placed at a considerable depth
- Failure by considerable vertical downward movement i.e. shearing in the vertical direction around the edges of the footing
- Slip surface restricted to vertical planes adjacent to the sides of the footing
- No bulging usually, no tilting.
- Failure is usually slow and time consuming (conditions are drained)
- Stress-strain curve is not well defined.

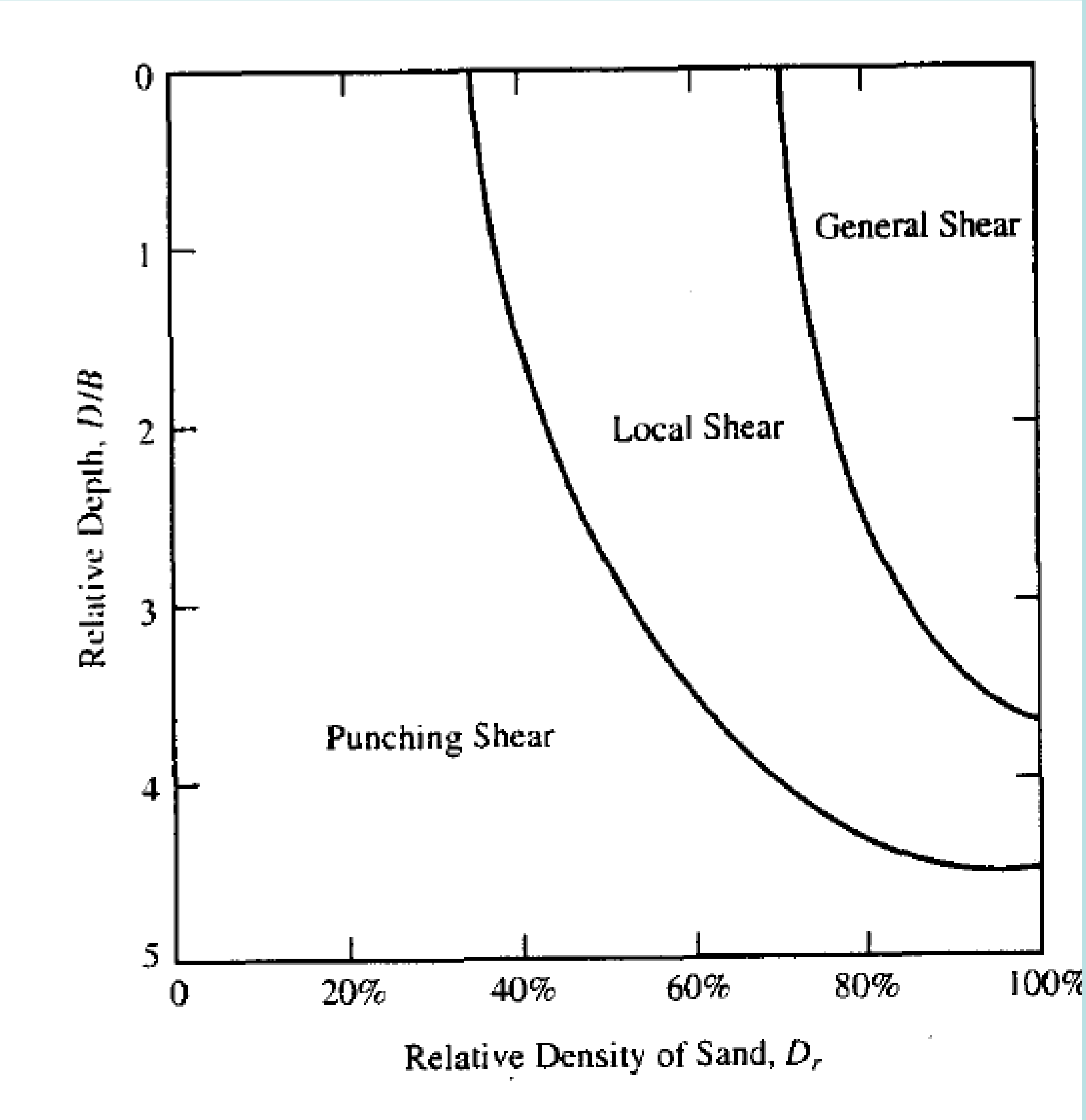
Guidelines for different Failures

- If the failure of a c-ϕ soil occurs at a small strain in a shear test (e.g. less than 5%), failure would probably occur in the field by general shear.
- If, on the other hand, a c-ϕ soil fails in a shear test at strains of over 10 %, local shear failure in the field would seem more probable.
- In a fairly soft, or loose and compressible soil that would undergo large deformations under the foundation before the failure zone develops, failure by punching shear is most probable.
- For cohesionless soils, if the angle of internal friction ϕ is more than 36 degrees, general shear failure is probable; and when ϕ is less than 29 degrees, local shear failure may be assumed.

Comments on Shear Failure

- Usually only necessary to analyze General Shear Failure.
- Local and Punching shear failure can usually be anticipated by settlement analysis.
- Failure in shallow foundations is generally settlement failure; bearing capacity failure must be analyzed, but in practical terms is usually secondary to settlement analysis.

Modes of BC Failure



Development of Bearing Capacity Theory

- **Application of limit equilibrium method was first employed by Prandtl on punching of thick masses of metal. He proposed the BC equation for shear failure of soil as given below:**

$$q_{ult} = \frac{c}{\tan\phi}\{\tan^2(45 + \frac{\phi}{2})e^{\pi \tan\phi} - 1\}$$

- **Prandtl's equation shows that if the cohesion of the soil is zero, the bearing capacity would also be equal to zero. This is quite contrary to the actual conditions. For cohesion less soil, the equation is indeterminate**

- **The limitations of Prandtl approach were recognized and accounted to some extent by Terzaghi and others. Terzaghi proposed bearing capacity equation for shallow foundations.**

- **Meyerhof, Hanson, Vesic and others improved on Terzaghi's original theory and added other factors for a more complete analysis**

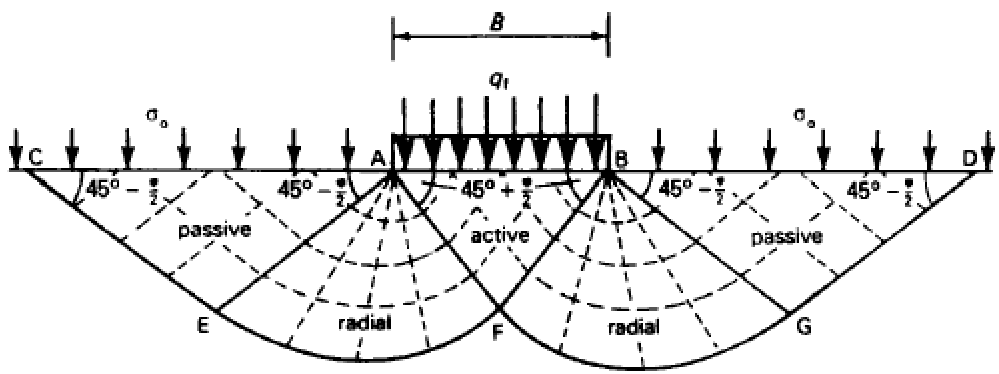
Assumptions for Terzaghi's Method

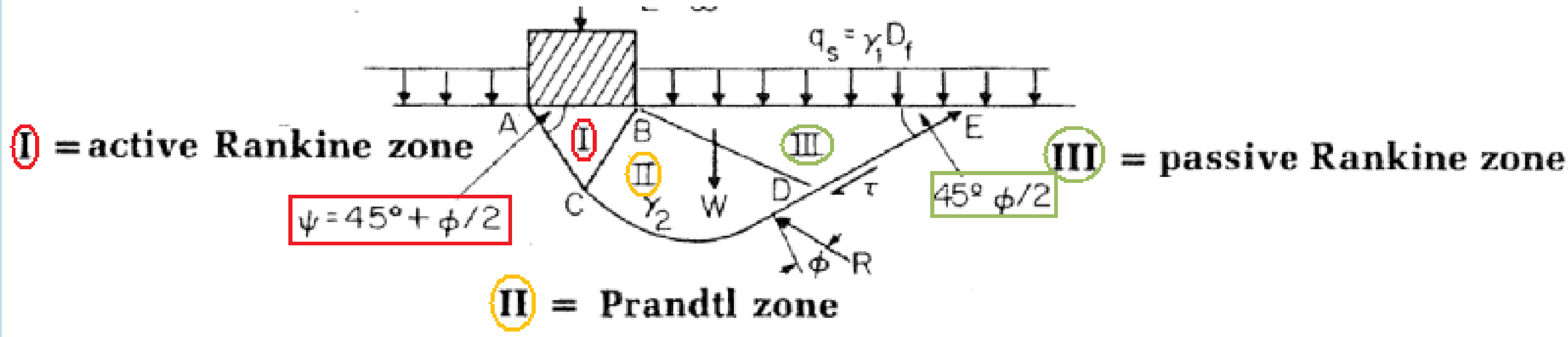
- Depth of foundation is less than or equal to its width
- No sliding occurs between foundation and soil (rough foundation)
- Soil beneath foundation is homogeneous semi infinite mass
- Mohr-Coulomb model for soil applies
- General shear failure mode is the governing mode (but not the only mode)

Assumptions for Terzaghi's Method

- No soil consolidation occurs; undrained condition
- Foundation is very rigid relative to the soil
- Soil above bottom of foundation has no shear strength; it provided only a surcharge load against the overturning load
- Applied load is compressive and applied vertically to the centroid of the foundation
- No applied moments present

Failure Geometry for Terzaghi's Method





$$q_u = q_c + q_q + q_\gamma$$

Terzaghi Bearing capacity equation

$$q_{ult} = cN_c Sc + \gamma D_f N_q + 0.5\gamma B N_\gamma S_\gamma$$

Shape Factors

	strip	circular	square	rectangle
*S*c =	1	1.3	1.3	$1 + 0.3\frac{B}{L}$
S_γ =	1	0.6	0.8	$(1 - 0.2\frac{B}{L})$

Nc, Nq, Nγ are bearing capacity factor

Terzaghi's BC Equations for different footings

$$q_{ult} = cN_c + \gamma D_f N_q + 0.5\gamma B N_\gamma$$

Strip footing

$$q_{ult} = 1.3cN_c + \gamma D_f N_q + 0.3\gamma B N_\gamma$$

Circular footing

$$q_{ult} = 1.3cN_c + \gamma D_f N_q + 0.4\gamma B N_\gamma$$

Square footing

$$q_{ult} = cN_c(1 + 0.3\frac{B}{L}) + \gamma D_f N_q + 0.5\gamma B N_\gamma (1 - 0.2\frac{B}{L})$$

Rectangular footing

Terzaghi's BC Equations

Few comments on Terzaghi equation:

1- The ultimate B.C increases with depth of footing.

2- The ultimate B.C of a cohesive soil ($\varphi = 0$) is independent of footing size, i.e. at the ground surface ($D_f = 0$) $q_u = 5.7c$.

3- The ultimate B.C of a cohesion less soil ($c = 0$) is directly dependent on footing size, but the depth of footing is more significant than size.

4- The above equations given by Terzaghi are for General Shear Failure case. For Local Shear Failure condition, following soil parameters were proposed by Terzaghi:

$$c' = 2/3\ c$$

$$\tan \varphi' = 2/3 \tan \varphi$$

BC factors for use in Terzaghi's bearing capacity equation.

$$N_q = \frac{a^2}{a\cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

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

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

ϕ	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°
N_c	5.7	7.3	9.6	12.9	17.7	25.1	37.2	57.8	95.7	172
N_q	1.0	1.6	2.7	4.4	7.4	12.7	22.5	41.4	81.3	173
N_γ	0.0	0.5	1.2	2.5	5.0	9.7	19.7	42.4	100	298

Fig. 8.6 Terzaghi's bearing capacity coefficients.

Effect of GWT on Bearing Capacity

$$q_{ult} = cN_c + \gamma D_f N_q + 0.5\gamma B N_\gamma$$

Method-I

For Case-I: **(D_w<D) , calculate 2nd term as {$\gamma D_w + \gamma'(D - D_w)$}$N_q$ and use γ' in the 3rd term of BC equation**

For Case 2 ($D \leq D_w < D + B$):

$$\gamma' = \gamma - \gamma_w \left(1 - \left(\frac{D_w - D}{B}\right)\right)$$

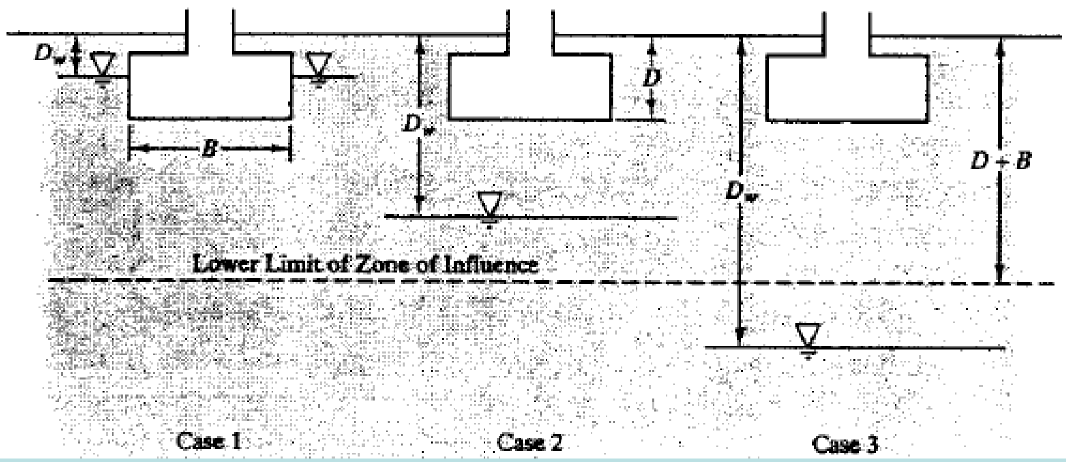
Use this γ' in 3rd term of BC Eq.

For Case 3 ($D + B \leq D_w$; no groundwater correction is necessary):

$$\gamma' = \gamma$$

No correction to BC Eq.

In Case 1, the second term in the bearing capacity formulas also is affected, but the appropriate correction is implicit in the computation of σ_D'.



Effect of GWT on Bearing Capacity

$$q_{ult} = cN_c + \gamma D_f N_q R'_w + 0.5\gamma B N_\gamma R_w$$

Method-2

(Approximate)

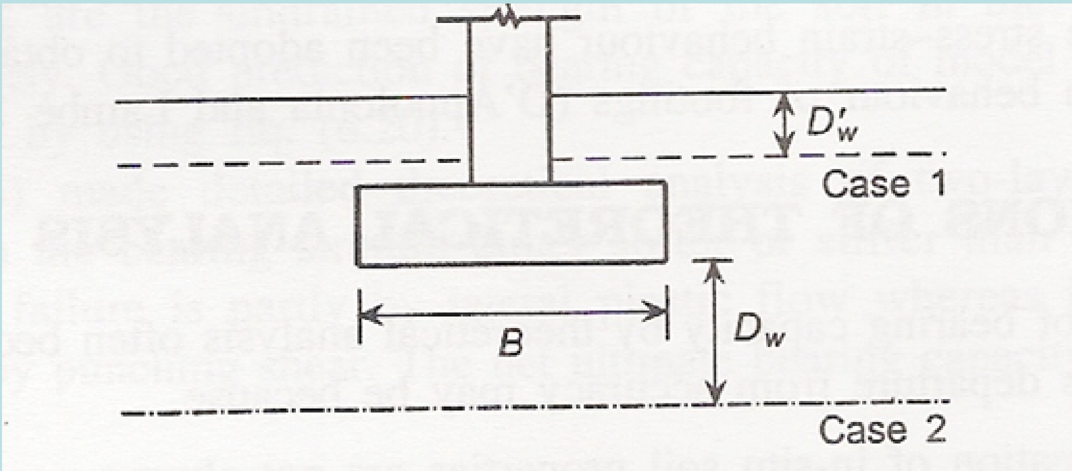


Fig. 6.12 Effect of ground water table.

$$R'_w = 0.5\left[1 + \frac{D'_w}{D_f}\right]$$

$$R_w = 0.5\left[1 + \frac{D_w}{B}\right]$$

Maximum value of Rw’ & Rw is 1

General form of the bearing capacity equation

Meherhof, Hansen & Vesic proposed the following general BC equation

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \overline{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

s_c, s_q and s_γ are shape factors
d_c, d_q and d_γ are depth factors
i_c, i_q and i_γ are inclination factors
g_c, g_q and g_γ are ground factors (base on slope)
b_c, b_q and b_γ are base factors (inclination of base)

Bearing Capacity Factors
for General BC Equation

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

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

(common for all)

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

(Meyerhof, 1963)

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

(Hansen, 1970)

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

(Vesic, 1973)

Table 2: *BC* factors for use in Meyerhof's, Hansen's and Vesic's equations. Subscripts identify author for *N*

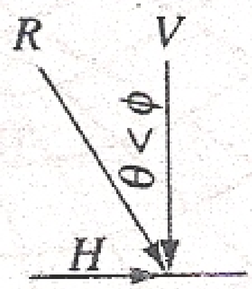
(degree)	*Nc*	*Nq*	$N\gamma$(M)	$N\gamma$(H)	$N\gamma$(V)
0	5.14	1.0	0	0	0
5	6.5	1.6	0.1	0.1	0.4
10	8.4	2.5	0.4	0.4	1.2
15	11.0	3.9	1.1	1.2	2.7
20	14.9	6.4	2.9	3.0	5.4
22	16.9	7.8	4.1	4.1	7.1
24	19.4	9.6	5.7	5.8	9.5
25	20.8	10.7	6.8	6.8	10.9
26	22.3	11.9	8.0	8.0	12.6
27	24.0	13.2	9.5	9.4	14.5
28	25.9	14.8	11.2	11.0	16.8
29	27.9	16.5	13.3	12.9	19.4
30	30.2	18.5	15.7	15.1	22.5
31	32.8	20.7	18.6	17.8	26.1
32	35.6	23.3	22.1	20.9	30.3
33	38.8	26.2	26.3	24.6	35.3
34	42.3	29.6	31.3	28.9	41.2
35	46.3	33.4	37.3	34.1	48.2

Meherhof Method

Shape, Depth and Inclination Factors

$$q_{ult} = cN_c s_c d_c i_c + \overline{q} N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

Factors	Value	For
Shape:	$s_c = 1 + 0.2K_p \frac{B}{L}$	Any ϕ
	$s_q = s_\gamma = 1 + 0.1K_p \frac{B}{L}$	$\phi > 10°$
	$s_q = s_\gamma = 1$	$\phi = 0$
Depth:	$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	Any ϕ
	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$	$\phi > 10$
	$d_q = d_\gamma = 1$	$\phi = 0$
Inclination:	$i_c = i_q = \left(1 - \frac{\theta°}{90°}\right)^2$	Any ϕ
	$i_\gamma = \left(1 - \frac{\theta°}{\phi°}\right)^2$	$\phi > 0$
	$i_\gamma = 0$ for $\theta > 0$	$\phi = 0$



Where $K_p = \tan^2(45 + \phi/2)$ as in Fig. 4-2

θ = angle of resultant R measured from vertical without a sign; if $\theta = 0$ all $i_i = 1.0$.

Hansen Method

$$q_{ult} = cN_c s_c d_c i_c + \overline{q} N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

Nc, Nq & Nγ as under

N_q = same as Meyerhof above

N_c = same as Meyerhof above

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

SHAPE FACTORS

$$s_c = 1 + \frac{B}{L}\frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L}\tan\phi$$

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

(Hansen Method)

Depth factors

Depth factor	$D_f/B \le 1.0$	$D_f/B > 1.0$
d_c	$1+0.4\,(D_f/B)$	$1+0.4\,\arctan(D_f/B)$
d_q	$1+2\tan\phi\,(1-\sin\phi)^2\,(D_f/B)$	$1+2\tan\phi\,(1-\sin\phi)^2\,\arctan(D_f/B)$
d_γ	1.0	1.0

Note: The arctan values must be expressed in radians, e.g. if D_f = 1.5 and B = 1.0 m then arctan (D_f/B) = arctan (1.5) = 56.3° = 0.983 radians.

Vesic Method

$$q_{ult} = cN_c s_c d_c i_c + \overline{q} N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

Nc, Nq & Nγ as under

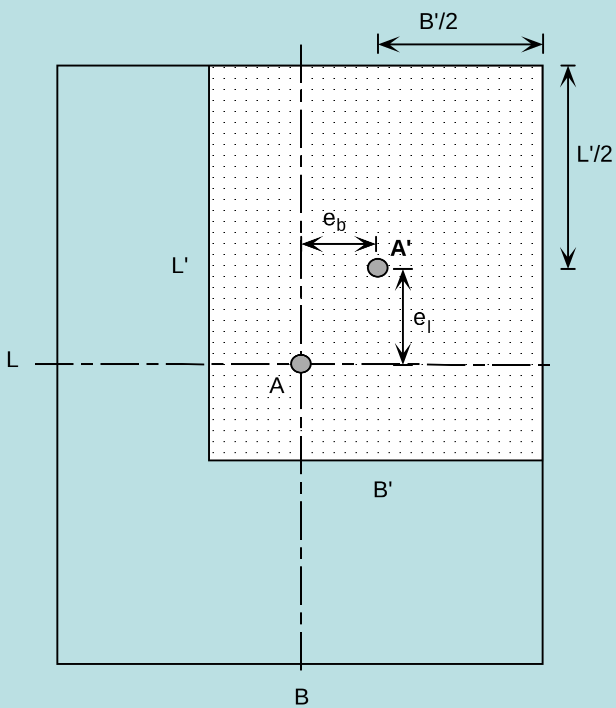
$$N_q = \text{same as Meyerhof above}$$
$$N_c = \text{same as Meyerhof above}$$
$$N_\gamma = 2(N_q + 1)\tan\phi$$

Shape and depth factors same as that of Hansen Method

Inclination factors **for both Hansen and Vesic methods are different (see Bowles book), however, for simplicity use Meherhof's Inclination factors at this level**

Eccentrically Loaded Footings

- If a foundation is subjected to lateral loads and moments in addition to vertical loads, eccentricity in loading results.
- The point of application of the resultant of all loads would lie outside the geometric centre of the foundation
- The eccentricity is measured from the centre of the footing to the point of application normal to the axis of the foundation
- The maximum eccentricity allowed is B/6, (B being the width of the footing) to avoid negative pressure at the footing base.

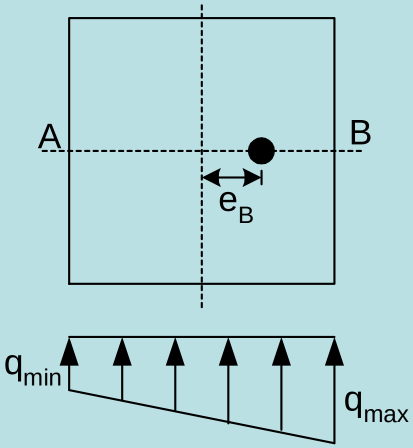


- *Determine effective dimensions as*
  $B' = B - 2e_B$
  $L' = L - 2e_L$
- *Effective footing Area,* $A' = B' \times L'$
- $Q = Q/A'$
- *To calculate BC, use B' & L' in BC equation*

Maximum and Minimum Base Pressure under Eccentric Loadings

When footing is eccentrically loaded, the soil experiences a maximum or minimum pressure at one of the edges/corners of the footing.

1. One Way Eccentricity

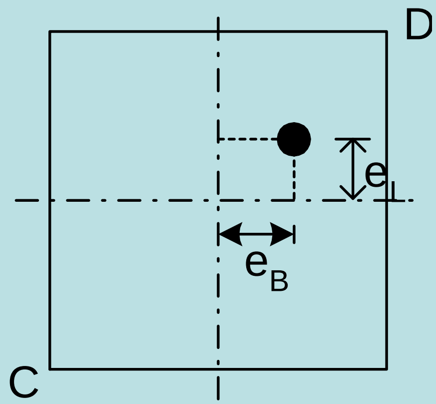


$$q = \frac{Q}{A}\{1 \pm 6\frac{e}{B}\}$$

$$q_{max} = \frac{Q}{A}\{1 + 6\frac{e}{B}\}$$ At Point B

$$q_{min} = \frac{Q}{A}\{1 - 6\frac{e}{B}\}$$ At Point A

2. Two Way Eccentricity



$$q = \frac{Q}{A}\{1 \pm 6\frac{e_B}{B} \pm 6\frac{e_L}{L}\}$$

$$q_{max} = \frac{Q}{A}\{1 + 6\frac{e_B}{B} + 6\frac{e_L}{L}\}$$ At Point D

$$q_{min} = \frac{Q}{A}\{1 - 6\frac{e_B}{B} - 6\frac{e_L}{L}\}$$ At Point C

e_B or e_L should be less than B/6 or L/6, respectively, to avoid negation pressure under the footing in case of one/two way eccentricity.