Unit 2: BEARING CAPACITY OF SHALLOW FOUNDATION

THE BEARING CAPACITY: The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure.

Types of Bearing Capacity of Soil

Following are some types of bearing capacity of soil:

1. Ultimate bearing capacity (q_u)

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

2. Net ultimate bearing capacity (q_{nu})

By neglecting the overburden pressure from ultimate bearing capacity we will get net ultimate bearing capacity.

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

Where γ = unit weight of soil, D_f = depth of foundation

3. Net safe bearing capacity (q_{ns})

By considering only shear failure, net ultimate bearing capacity is divided by certain factor of safety will give the net safe bearing capacity.

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

Where F = factor of safety = 3 (usual value)

4. Gross safe bearing capacity (q_s)

When ultimate bearing capacity is divided by factor of safety it will give gross safe bearing capacity.

$$q_s = q_u / F$$

5. Net safe settlement pressure (q_{np})

The pressure with which the soil can carry without exceeding the allowable settlement is called net safe settlement pressure.

6. Net allowable bearing pressure (q_{na})

This is the pressure we can used for the design of foundations. This is equal to net safe bearing pressure if $q_{np} > q_{ns}$. In the reverse case it is equal to net safe settlement pressure.

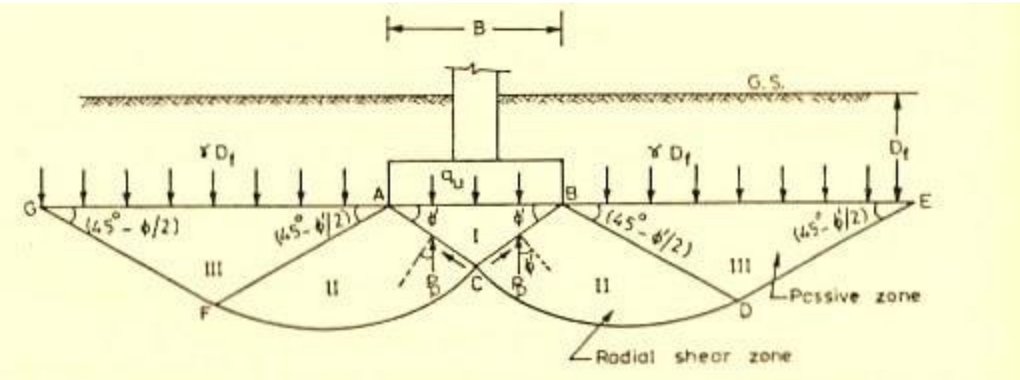
Calculation of Bearing Capacity

For the calculation of bearing capacity of soil, there are so many theories. But all the theories are superseded by Terzaghi's bearing capacity theory.

1. Terzaghi's bearing capacity theory

Terzaghi's bearing capacity theory is useful to determine the bearing capacity of soils under a strip footing. This theory is only applicable to shallow foundations. He considered some assumptions which are as follows.

1. The base of the strip footing is rough.
2. The depth of footing is less than or equal to its breadth i.e., shallow footing.
3. He neglected the shear strength of soil above the base of footing and replaced it with uniform surcharge. (γD_f)
4. The load acting on the footing is uniformly distributed and is acting in vertical direction.
5. He assumed that the length of the footing is infinite.
6. He considered Mohr-coulomb equation as a governing factor for the shear strength of soil.



As shown in above figure, AB is base of the footing. He divided the shear zones into 3 categories. Zone -1 (ABC) which is under the base is acts as if it were a part of the footing itself. Zone -2 (CAF and CBD) acts as radial shear zones which is bear by the sloping edges AC and BC. Zone -3 (AFG and BDE) is named as Rankine's passive zones which are taking surcharge (y D_f) coming from its top layer of soil. From the equation of equilibrium, Downward forces = upward forces

Load from footing x weight of wedge = passive pressure + cohesion x CB sinϕ

$$q_u \times B + \left(\frac{1}{4} \Upsilon B^2 \sin \phi\right) = 2P_p + 2c' \times \left(\frac{B}{2 \cos\phi} . \sin\phi\right)$$

Where P_p = resultant passive pressure = $(P_p)_y + (P_p)_c + (P_p)_q$ $(P_p)_y$ is derived by considering weight of wedge BCDE and by making cohesion and surcharge zero. $(P_p)_c$ is derived by considering cohesion and by neglecting weight and surcharge. $(P_p)_q$ is derived by considering surcharge and by neglecting weight and cohesion.

Therefore,

$$q_u \times B = 2((P_p)_y + (P_p)_c + (P_p)_q) + \left(\frac{Bc'}{\cos\phi} . \sin\phi\right) - \left(\frac{1}{4} \Upsilon B^2 \sin \phi\right)$$

By substituting,

$$2(P_p)_y - \frac{1}{4} \Upsilon B^2 \sin \phi = B \times 0.5 \Upsilon B N_y$$

$$2(P_p)_y + Bc' \tan \phi = B c' N_c$$

$$2(P_p)_q = B \Upsilon D_f N_q$$

So, finally we get **$q_u = c'N_c + y D_f N_q + 0.5 y B N_y$** The above equation is called as Terzaghi's bearing capacity equation. Where q_u is the ultimate bearing capacity and N_c, N_q, N_y are the Terzaghi's bearing capacity factors. These dimensionless factors are dependents of angle of shearing resistance (). **Equations to find the bearing capacity factors are:**

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

$$N_q = \left[\frac{a^2}{2\cos^2(45 + \phi/2)}\right] \text{ and}$$

$$N_y = 0.5 \tan \phi \left[\frac{Kp}{\cos^2\phi} - 1\right]$$

Where $a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$ Kp = coefficient of passive earth pressure. For different values of ϕ , bearing capacity factors under general shear failure are arranged in the below table.

ϕ	Nc	Nq	Ny
0	5.7	1	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

Finally, to determine bearing capacity under strip footing we can use

$$q_u = c'N_c + \gamma D_f N_q + 0.5 \gamma B N_\gamma$$

By the modification of above equation, equations for square and circular footings are also given and they are. For square footing

$$q_u = 1.2 c'N_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma$$

For circular footing

$$q_u = 1.2 c'N_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma$$

2. Hansen's bearing capacity theory

For cohesive soils, Values obtained by Terzaghi's bearing capacity theory are more than the experimental values. But however it is showing same values for cohesionless soils. So Hansen modified the equation by considering shape, depth and inclination factors. According to Hansen's

$$q_u = c'N_c Sc\ dc\ ic + \gamma D_f N_q Sq\ dq\ iq + 0.5 \gamma B N_\gamma Sy\ dy\ iy$$

Where Nc, Nq, Ny = Hansen's bearing capacity factors Sc, Sq, Sy = shape factors dc, dq, dy = depth factors ic, iq, iy = inclination factors Bearing capacity factors are calculated by following equations.

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

$$Nc = (Nq - 1) \cot \phi$$

$$Ny = 1.8(Nq - 1) \tan \phi$$

For different values of ϕ Hansen bearing capacity factors are calculated in the below table.

ϕ	Nc	Nq	Ny
0	5.14	1	0
5	6.48	1.57	0.09
10	8.34	2.47	0.09
15	10.97	3.94	1.42
20	14.83	6.4	3.54
25	20.72	10.66	8.11
30	30.14	18.40	18.08
35	46.13	33.29	40.69
40	75.32	64.18	95.41
45	133.89	134.85	240.85
50	266.89	318.96	681.84

Shape factors for different shapes of footing are given in below table.

Shape of footing	Sc	Sq	Sy
Continuous	1	1	1
Rectangular	1+0.2B/L	1+0.2B/L	1-0.4B/L
Square	1.3	1.2	0.8

Circular	1.3	1.2	0.6

Depth factors are considered according to the following table.

Depth factors	**Values**
dc	1+0.35(D/B)
dq	1+0.35(D/B)
dy	1.0

Similarly inclination factors are considered from below table.

Inclination factors	**Values**
ic	1 – [H/(2 c B L)]
iq	1 – 1.5 (H/V)
iy	$(iq)^2$

Where H = horizontal component of inclined load B = width of footing L = length of footing.

Terzaghi (1943)

$$q_{ult} = c \cdot N_c \cdot s_c + \gamma \cdot D \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma$$

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

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

$$N_c = (N_q - 1) \cdot \cot\varphi$$

$$N_q = \frac{\tan\varphi}{2} \cdot \left(\frac{N_{py}}{\cos^2\varphi} - 1\right)$$

For:	strip	round	square
S_c =	1.0	1.3	1.3
S_γ =	1.0	0.6	0.8

Meyerhof (1963)

Vertical load:

$$q_{ult} = c \cdot N_c \cdot s_c \cdot d_c + \gamma \cdot D \cdot N_q \cdot d_q + 0.5 \cdot \gamma \cdot B' N_\gamma \cdot s_\gamma \cdot d_\gamma$$

Inclined load:

$$q_{ult} = c \cdot N_c \cdot d_c \cdot i_c + \gamma \cdot D \cdot N_q \cdot d_q \cdot i_q + 0.5 \cdot \gamma \cdot B' N_\gamma \cdot d_\gamma \cdot i_\gamma$$

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

$$N_c = (N_q - 1) \cdot \cot\varphi$$

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

Hansen (1970)

$$q_{ult} = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + \gamma \cdot D \cdot N_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot g_\gamma \cdot b_\gamma$$

$\varphi = 0$

$$q_{ult} = 5.14 \cdot c_u \cdot (1 + s'_c + d'_c - i'_c - g'_c - b'_c) + \gamma \cdot D$$

N_q= same as Meyerhof above

N_c= same as Meyerhof above

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

Vesic (1975)

Use Hansen equation above

$\varphi = 0$

$$q_{ult} = 5.14 \cdot c_u \cdot (1 + s'_c + d'_c - i'_c - g'_c - b'_c) + \gamma \cdot D$$

N_q= same as Meyerhof above

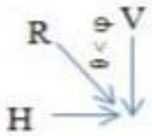
N_c= same as Meyerhof above

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

Bearing-capacity equations by the several authors indicated

Meyerhof (1963) proposed a formula for calculation of bearing capacity similar to the one proposed by Terzaghi but introducing further foundation shape coefficients. He introduced a coefficient s_q that multiplies the Nq factor, depth factors di and inclination factors i_i depth factors di and inclination factors ii for the cases where the load line is inclined to the vertical.

Meyerhof obtained the N factors by making trials on several BF arcs *(see Prandtl mechanism)* whilst shear along AF was given approximate values.

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

Shape, depth, and inclination factors for the Meyerhof bearing-capacity

Hansen's (1970) formula is a further extension on Meyerhof's; the additions consists in the introduction of bi that accounts for the possible inclination of the footing to the horizontal and a factor gi for inclined soil surface
Hansen's formula is valid for whatever ratio D/B and therefore for both surface and deep foundations, however the author introduces coefficients to compensate for the otherwise excessive increment in limit load with increased depth.

Vesic (1975) proposes a formula that is analogous to **Hansen's** with N_q ed N_c as per Meyerhof and N_γ as below:

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

Shape and depth factors are the same as Hansen's but there are differences in load inclination, ground inclination and footing inclination factors.

Shape factors	Depth factors
$s'_{c(H)} = 0.2 \cdot \frac{B'}{L'} \quad (\varphi = 0)$	$d'_c = 0.4 \cdot k \quad (\varphi = 0^\circ)$
$s_{c(H)} = 1 + \frac{N_q}{N_c} \cdot \frac{B'}{L'}$	$d_c = 1 + 0.4 \cdot k$
$s_{c(V)} = 1 + \frac{N_q}{N_c} \cdot \frac{B}{L}$	$k = D/B$ for $D/B \le 1$ $k = \tan^{-1}(D/B)$ for $D/B > 1$
$s_c = 1$ for strip	k in radians
$s_{q(H)} = 1 + \frac{B'}{L'} \cdot \sin\varphi$	$d_q = 1 + 2 \cdot \tan\varphi \cdot (1 - \sin\varphi)^2 \cdot k$
$s_{q(V)} = 1 + \frac{B}{L} \cdot \tan\varphi$	k defined above
for all φ	
$s_{\gamma(H)} = 1 - 0.4 \cdot \frac{B'}{L'} \quad \ge 0.6$	$d_\gamma = 1$ for all φ
$s_{\gamma(V)} = 1 - 0.4 \cdot \frac{B}{L} \quad \ge 0.6$	

Shape and depth factors for use in either the Hansen (1970) or Vesic (1975) bearing-capacity equations.

Inclination factors	Ground factors (base on slope)
$i'_c = 0.5 - \sqrt{1 - \frac{H_i}{A_f \cdot C_a}}$	$g'_c = \beta^\circ / 147^\circ$ $g_c = 1 - \beta^\circ / 147^\circ$
$i_c = i_q - \frac{1 - i_q}{N_q - 1}$	$g_q = g_\gamma = (1 - 0.5 \cdot \tan\beta)^5$
$i_q = \left[1 - \frac{0.5 \cdot H_i}{V + A_f \cdot c_a \cdot \cot\varphi}\right]^{\alpha_1}$ $2 \le \alpha_1 \le 5$	**Base factors (tilted base)**
$i_\gamma = \left[1 - \frac{0.7 \cdot H_i}{V + A_f \cdot c_a \cdot \cot\varphi}\right]^{\alpha_2}$	$b'_c = \beta^\circ / 147^\circ \quad (\varphi = 0)$ $b_c = 1 - \beta^\circ / 147^\circ \quad (\varphi > 0)$
$i_\gamma = \left[1 - \frac{(0.7 - \eta^\circ / 450^\circ) \cdot H_i}{V + A_f \cdot c_a \cdot \cot\varphi}\right]^{\alpha_2}$ $2 \le \alpha_2 \le 5$	$b_q = \exp(-2\eta \tan\varphi)$ $b_\gamma = \exp(-2.7\eta \tan\varphi)$ η in radians

Table of inclination, ground, and base factors for the Hansen (1970) equations

Inclination factors	Ground factors (base on slope)
$i'_c = 1 - \frac{m \cdot H_i}{A_f \cdot c_a \cdot N_c} \quad (\varphi = 0)$	$g'_c = \beta / 5.14 \quad \beta$ in radians $g_c = 1 - \frac{1 - i_q}{5.14 \cdot \tan\varphi} \quad \varphi > 0$ i_q defined with i_c
$i_c = i_q - \frac{1 - i_q}{N_q - 1} \quad (\varphi > 0)$ i_q, and m defined below	$g_q = g_\gamma = (1 - 0.5 \cdot \tan\beta)^2$
$i_q = \left[1 - \frac{H_i}{V + A_f \cdot c_a \cdot \cot\varphi}\right]^m$	**Base factors (tilted base)**
$i_\gamma = \left[1 - \frac{H_i}{V + A_f \cdot c_a \cdot \cot\varphi}\right]^{m+1}$	$b'_c = g'_c \quad (\varphi = 0)$ $b_c = 1 - 2 \cdot \beta / (5.14 \cdot \tan\varphi)$
$m = m_B = \frac{2 + B/L}{1 + B/L}$ $m = m_L = \frac{2 + L/B}{1 + L/B}$	$b_q = b_\gamma = (1 - \eta \tan\varphi)^2$

Table of inclination, ground, and base factors for the Vesic (1975) equations

Brich-Hansen **(EC7-EC8)** *"In order that a foundation may safely sustain the projected load in regard to general failure for all combinations of load relative to the ultimate limit state, the following must be satisfied:*

$$V_d \leq R_d$$

Where V_d the design load at ultimate limit state normal to the footing, including the weight of the foundation itself and R_d is the foundation design bearing capacity for normal loads, also taking into account eccentric and inclined loads. When estimating R_d for fine grained soils short- and long-term situations should be considered."

Bearing capacity **in drained conditions** is calculated by:

$$R/A' = (2+\pi) \cdot c_u \cdot s_c \cdot i_c + q$$

Where:

A’ = B’·L’ design effective foundation area. Where eccentric loads are involved, use the reduced area at whose center the load is applied.

c_u undrained cohesion
q total lithostatic pressure on bearing surface
s_c foundation shape factor
s_c 1 + 0,2 (B’/L’) rectangular shapes
s_c 1,2 square or circular shapes
i_c correction factor for inclination due to a load H

$$i_c=0.5\cdot[1+(1-H/(A'\cdot c_u)\text{^}0.5]$$

Design bearing capacity in **drained conditions** is calculated as follows:

$$R/A' = c' \cdot N_c \cdot s_c \cdot i_c + q'\cdot N_q \cdot s_q \cdot i_q + 0,5\cdot g' \cdot B' \cdot N_g \cdot s_g \cdot i_g$$

Where:

N_c= same as Meyerhof (1963) above
N_q= same as Meyerhof (1963) above
$N_\gamma=2\cdot(N_q-1)\cdot\tan\phi$

Shape factors
s_q = 1+(B’/L’) ·sinϕ’ rectangular shape
s_q = 1+sinϕ’ square or circular shape
s_γ =1-0.3·(B’/L’) rectangular shape
s_γ =0.7 square or circular shape
s_c= (s_q ·N_q-1)/(N_q-1) rectangular, square, or circular shape.

In addition to the correction factors reported in the table above will also be considered the ones complementary to the depth of the bearing surface and to the inclination of the bearing surface and ground surface (Hansen).

Sliding considerations
The stability of a foundation should be verified with reference to collapse due to sliding as well as to general failure. For collapse due to sliding, the resistance is calculated as the sum of the adhesion component and the soil-foundation friction component. Lateral resistance arising from passive thrust of the soil can be taken into account using a percentage supplied by the user. Resistance due to friction and adhesion is calculated with the expression:

$$F_{Rd} = N_{sd} \cdot\tan\delta+c_a \cdot A'$$

In which N_{sd} is the value of the vertical force, δ is the angle of shearing resistance at the base of the foundation, c_a is the foundation-soil adhesion, and A’ is the effective foundation area. There where eccentric loads are involved, use the reduced area at whose centre the load is applied.

Bearing capacity for foundations on rock
Where foundations rest on rock, it is appropriate to take into consideration certain other significant parameters such as the geologic characteristics, type of rock and its quality measured as **RQD.** It is the practice to use very high values of safety factor for bearing capacity of rock and correlated in some way with the value of **RQD** *(Rock quality designator).* For example for a rock whose **RQD** is up to a maximum of 0.75 the safety factor oscillates between 6 and 10. Terzaghi’s formula can be used in calculation of rock bearing capacity using friction angle and cohesion of the rock or those proposed by Stagg and **Zienkiewicz (1968)** according to which the coefficients of the bearing capacity are:

$$N_q = \tan^6(45+\varphi/2)$$
$$N_c = 5 \cdot \tan^4(45+\varphi/2)$$
$$N_\gamma = N_q + 1$$

These coefficients should be used with form factors from the formula of Terzaghi. Ultimate bearing capacity is a function of RQD as follows:

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

If rock coring does not render whole pieces (RQD tends to 0) the rock is treated as a soil estimating as best the factors: c and φ.

Types of shear failure of foundation soils

Depending on the stiffness of foundation soil and depth of foundation, the following are the modes of shear failure experienced by the foundation soil.

1. General shear failure (Fig.1(a))
2. Local shear failure (Fig.1(b))
3. Punching shear failure (Fig.1(c))
4.

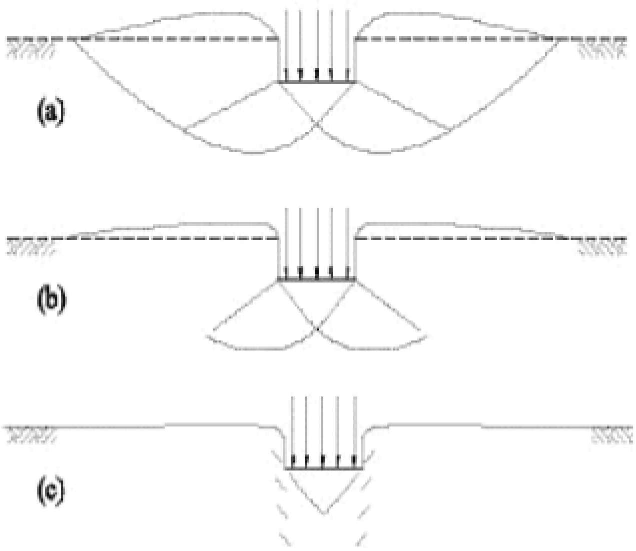


Fig.1: Shear failure in foundation soil

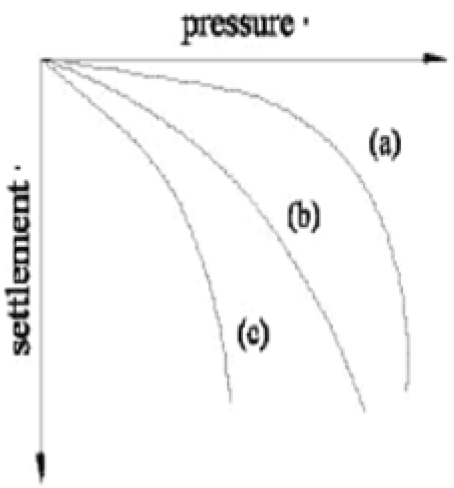


Fig: P – Δ Curve in different foundation soils

General Shear Failure

This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure.

5. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.
6. Dense or stiff soil that undergoes low compressibility experiences this failure.
7. Continuous bulging of shear mass adjacent to footing is visible.
8. Failure is accompanied by tilting of footing.
9. Failure is sudden and catastrophic with pronounced peak in $p-\Delta$ curve.
10. The length of disturbance beyond the edge of footing is large.
11. State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.
12. General shear failure is accompanied by low strain (<5%) in a soil with considerable ϕ (ϕ >36°) and large N (N > 30) having high relative density (I_D> 70%).

Local Shear Failure

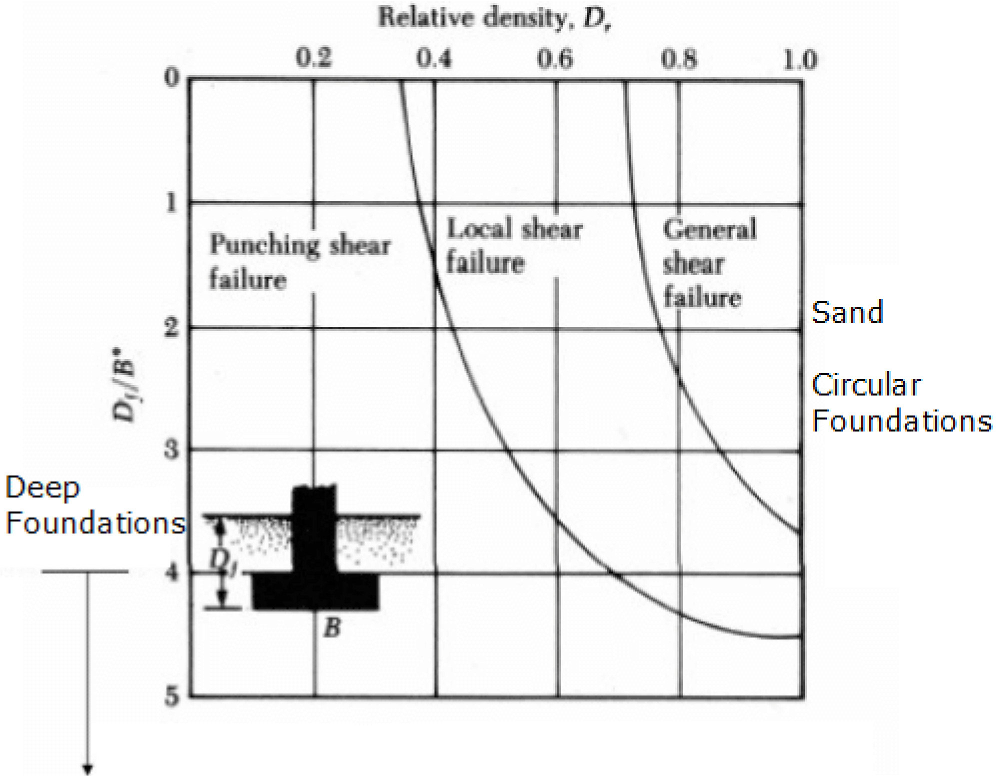
This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure.

13. A significant compression of soil below the footing and partial development of plastic equilibrium is observed.
14. Failure is not sudden and there is no tilting of footing.
15. Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.
16. Failure surface is not well defined.
17. Failure is characterized by considerable settlement.\
18. Well defined peak is absent in $p-\Delta$ curve.
19. Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low ϕ (ϕ<28o) and low N (N < 5) having low relative density (I_D> 20%).

Punching Shear Failure of foundation soils

This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure.

20. This type of failure occurs in a soil of very high compressibility.
21. Failure pattern is not observed.
22. Bulging of soil around the footing is absent.
23. Failure is characterized by very large settlement.
24. Continuous settlement with no increase in P is observed in $p-\Delta$ curve.



EFFECTE OF SIZE OF FOOTING ON BOTH ULTIMATE BEARING CAPACITY AND SATTLEMENT :

Fig. 2.4.1 gives typical load-settlement relationships for footings of different widths on the surface of a homogeneous sand deposit.

It can be seen that the ultimate bearing capacities of the footings per unit area increase with the increase in the widths of the footings.

However, for a given settlement s, such as 25 mm, the soil pressure is greater for a footing of intermediate width B_b than for a large footing with BC.

The pressures corresponding to the three widths intermediate, large and narrow, are indicated by points b, c and a respectively.

The same data is used to plot Fig. 2.4.1 which shows the pressure per unit area corresponding to a given settlement s_1, as a function of the width of the footing.

The soil pressure for settlement s_i increases for increasing width of the footing, if the footings are relatively small, reaches a maximum at an intermediate width, and then decreases gradually with increasing width.

Although the relation shown in Fig. 2.4.1 is generally valid for the behavior of footings on sand, it is influenced by several factors including the relative density of sand, the depth at which the foundation is established, and the position of the water table.

Furthermore, the shape of the curve suggests that for narrow footings small variations in the actual pressure, Fig. 2.4.1 may lead to large variation in settlement and in some instances to settlements so large that the movement would be considered a bearing capacity failure.

On the other hand, a small change in pressure on a wide footing has little influence on settlements as small as s_1, and besides, the value of q_i corresponding to s_1 is far below that which produces a bearing capacity failure of the wide footing.

For all practical purposes, the actual curve given in Fig. 2.4.1 can be replaced by an equivalent curve omn where om is the inclined part and mn the horizontal part.

The horizontal portion of the curve indicates that the soil pressure corresponding to a settlement s_1 is independent of the size of the footing.

The inclined portion *om* indicates the pressure increasing with width for the same given settlement s_1 up to the point *m* on the curve which is the limit for a bearing capacity failure.

This means that the footings up to size *B* in Fig. 2.4.1 should be checked for bearing capacity failure also while selecting a safe bearing pressure by settlement consideration.

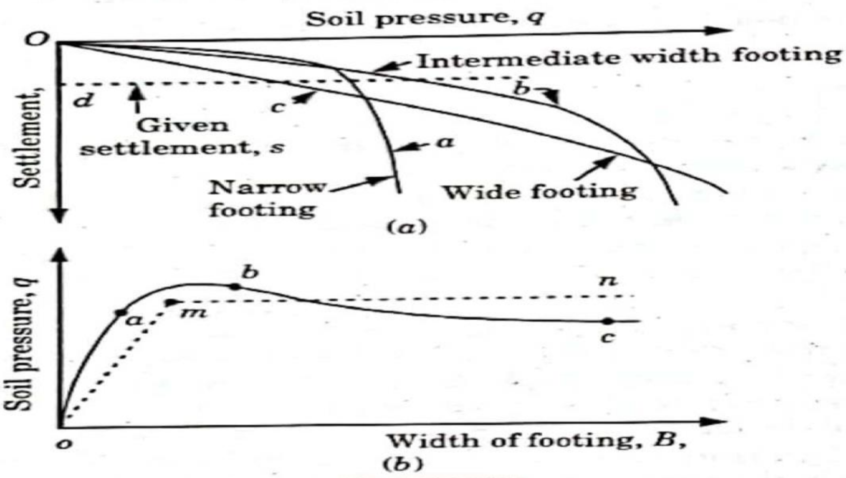


Fig. 2.4.1.

The position of the broken lines *omn* differs for different sand densities or in other words for different SPT *N* values.

The soil pressure that produces a given settlement s_i on loose sand is obviously smaller than the soil pressure that produces the same settlement on a dense sand.

Since *N*-value increases with density of sand, q_s therefore increases with an increase in the value of *N*.

Que 2.11. **A square footing of 3.0 m × 3.0 m size has been founded at 1.2 m below the ground level in a cohesive soil having a bulk density of 1.8 t/m³ and an unconfined compressive strength of 5.5 t/m². Determine the safe bearing capacity of the footing for a factor of safety of 2.5 by Skempton's method.** **AKTU 2015-16, Marks 05**

Answer

Given : Size of footing = 3 m × 3 m, Depth of footing, $D_f = 1.2$ m
Density of clay, $\rho = 1.8$ t/m³, Unconfined compressive strength, $q_u = 5.5$ t/m², Factor of safety = 2.5
To Find : Capacity of footing by Skempton's method.

1. Cohesion, $$C_u = \frac{q_u}{2} = \frac{5.5}{2} \times 10 = 27.5 \text{ kN/m}^2$$

   For $$\frac{D_f}{B} = \frac{1.2}{3} = 0.4 < 2.5$$

2. Bearing capacity factor,
$$N_c = 6\left[1 + 0.2 \times \frac{D_f}{B}\right] = 6\left[1 + 0.2 \times \frac{1.2}{3}\right] = 6.48$$

3. Net bearing capacity,
$$q_{nu} = C_u N_c = 27.5 \times 6.48 = 178.2 \text{ kN/m}^2$$

4. Safe bearing capacity,
$$q_{ns} = \frac{178.2}{2.5} = 71.28 \text{ kN/m}^2$$

Que 2.12. **A foundation in sand will be 5 metres wide and 1.5 meters deep. Adopting a factor of safety of 2.5, what will be safe bearing capacity if the unit weight of the sand is 1.9 gm/cc and angle of internal friction is 35°. How does it compare with safe bearing capacity for surface loading ?**

$N_c = 57$, $N_q = 44$, $N_y = 42$.

AKTU 2013-14, Marks 06

Answer

Given : Angle of internal friction, $\phi = 35°$, Unit weight of sand, = 19 kN/m³, Width of footing, $B = 5$ m, Depth of footing, $D_f = 1.5$ m, Factor of safety, $F_s = 2.5$, For sand (non-cohesive soil), $C = 0$,
For $\phi = 35°$, $N_c = 57$, $N_q = 44$, $N_\gamma = 42$
To Find : Safe bearing capacity.

1. Safe bearing capacity (q_s) for rectangular (square) footing,
   Ultimate bearing capacity,

$$q_u = \bar{\sigma}N_q + 0.4\gamma BN_\gamma = \gamma DN_q + 0.4\gamma BN_\gamma$$
$$= (19 \times 1.5 \times 44) + (0.4 \times 19 \times 5 \times 42)$$
$$= 1254 + 1596 = 2850 \text{ kN/m}^2$$

2. Net bearing capacity, $q_{nu} = q_u - \gamma D$

$$= 2850 - 19 \times 1.5 = 2821.5 \text{ kN/m}^2$$

3. Safe bearing capacity, $q_s = \frac{q_{nu}}{F_s} + \gamma D = \frac{2821.5}{2.5} + (19 \times 1.5)$

$$q_s = 1128.6 + 28.5 = 1157.1 \text{ kN/m}^2$$

Que 2.13. **Determine the ultimate bearing capacity of the footing, 1.5 m wide and its base at a depth of 1 m, if the ground water table is located :**

i. At a depth of 0.5 m below the ground surface.
ii. At a depth of 0.5 m below the base of the footing.

$\gamma_{sat} = 20$ kN/m³.
$\gamma_d = 17$ kN/m³, $\phi = 38°$ and $C = 0$. Use Terzaghi's theory.
$N_q = 60$ and $N_\gamma = 75$.

AKTU 2013-14, Marks 06

Answer

Given : Width of footing, $B = 1.5$ m, Depth of footing, $D = 1.0$ m
Unit weight of saturated soil, $\gamma_{sat} = 20$ kN/m³, Unit weight of dry soil, $\gamma_d = 17$ kN/m³, Angle of internal friction, $\phi = 38°$, $N_q = 60$, $N_\gamma = 75$
To Find : Ultimate bearing capacity.

1. When Water Table is above Footing :

i. Correction factor, $R_w = 0.5$

ii. Ultimate bearing capacity is given by,

$$q_u = \bar{\sigma} N_q + \frac{1}{2} B \gamma N_\gamma R_w$$

Net stress, $\bar{\sigma} = 0.5 \times 17 + 0.5 \times (20 - 9.81) = 13.595 \text{ kN/m}^2$

$$q_u = 13.595 \times 60 + (1/2) \times 1.5 \times 20 \times 75 \times 0.5$$
$$= 1378.2 \text{ kN/m}^2$$

2. When Water Table in below the Footing Base :

i. $z_w = 1.5 - 1 = 0.5$

ii. Correction factor, $R_w = 0.5\left[1 + \frac{z_w}{B}\right]$

$$R_\omega = 0.5\left[1 + \frac{0.5}{1.5}\right] = 0.667$$

iii. Average density, $\gamma_{av} = \frac{0.5 \times 17 + 0.5 \times 20}{0.5 + 0.5} = 18.5 \text{ kN/m}^3$

iv. Ultimate bearing capacity,

$$q_u = \gamma_d D N_q + \frac{1}{2} B \gamma_{av} N_\gamma R_w$$
$$q_u = 17 \times 1 \times 60 + (1/2) \times 1.5 \times 18.5 \times 75 \times 0.667$$
$$= 1714 \text{ kN/m}^2$$

Que 2.17. A square footing 1.5 m × 1.5 m is located at a depth of 1 m. The soil has the following properties, $\gamma = 17.5$ kN/m^3, $C = 0$ and $\phi = 35°$. Use Hansen's method to compute the ultimate bearing capacity of the soil. The footing base and ground are horizontal.

AKTU 2016-17, Marks 10

Answer

Given: Size of footing, $L \times B = 1.5 \times 1.5$ m
Depth of footing, $D_f = 1$ m, Unit weight of soil, $\gamma = 17.5$ kN/m^3
Cohesion of soil, $C = 0$, Angle of internal friction, $\phi = 35°$
To Find : Ultimate bearing capacity.

1. For $\phi = 35°$, bearing capacity factor is calculated as :

i.
$$N_q = e^{(\pi \tan \phi)} \tan^2\left(45° + \frac{\phi}{2}\right)$$
$$N_q = e^{(\pi \tan 35°)} \tan^2\left(45° + \frac{35°}{2}\right) = 33.296$$

ii.
$$N_c = (N_q - 1) \cot \phi = (33.296 - 1) \cot 35° = 46.12$$

iii.
$$N_\gamma = 1.8\,(N_q - 1) \tan \phi$$
$$= 1.8 \times (33.296 - 1) \tan 35° = 40.7$$

2. Bearing capacity equation is given by,
$$q_u = CN_c\, s_c\, d_c\, i_c + qN_q\, s_q\, d_q\, i_q + 0.5\gamma\, BN_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$$

3. Bearing capacity correction factors :

i.
$$d_q = 1 + 2 \times \tan \phi \times (1 - \sin \phi)^2\, \frac{D_f}{B}$$
$$= 1 + 2 \times \tan 35° \times (1 - \sin 35°)^2 \times \frac{1}{1.5} = 1.17$$

ii.
$$d_\gamma = 1 \text{ (for square footing)}$$

iii.
$$i_q = \left[1 - \left\{\frac{0.5H}{V + BLC \cot\phi}\right\}\right]^5 = \left[1 - \frac{0.5H}{H + 0}\right]^5 \qquad [\because C = 0]$$
$$= 0.03125 \qquad [\because H = V]$$

iv.
$$i_\gamma = \left[1 - \left\{\frac{0.7H}{V + B\,LC \cot\phi}\right\}\right]^5 = [1 - 0.7]^5 = 0.00243$$

v.
$$s_q = 1 + i_q\left(\frac{B}{L}\right)\sin\phi = 1 + 0.03125 \times \left(\frac{1.5}{1.5}\right) \sin 35° = 1.018$$

vi.
$$s_\gamma = 1 - 0.4\, i_\gamma \left(\frac{B}{L}\right) = 1 - 0.4 \times 0.00\,243 \times \frac{1.5}{1.5} = 0.999$$

4.
$$q_u = \gamma D_f N_q\, s_q\, d_q\, i_q + 0.5\, B\gamma\, N_\gamma\, s_\gamma\, d_\gamma i_\gamma \qquad [\because C = 0]$$

Que 2.19. A rectangle footing of 2.5 m × 4.0 m size is to be constructed at 1.8 m below the ground level in a c-ϕ soil having the following properties : $c = 1.0$ t/m², $\phi = 20°$ and $\gamma = 1.75$ t/m³.
The footing has to carry a gross vertical load of 80 t, inclusive of its self-weight. In addition, the column is subjected to a horizontal load of 10 t applied at a height of 3.3 m above the base of footing. Determine the factor of safety of the footing against shear failure as per IS : 6403-1981.

AKTU 2015-16, Marks 15

Answer

Given : Height of horizontal load above the base = 3.3 m
Length of footing, $L = 4$ m, Width of footing, $B = 2.5$ m,
For $\phi = 20°$, $N_c = 14.8$, $N_q = 6.4$, $N_\gamma = 5.4$
To Find : Factor of safety.

Assume water table is situated at well below the base of footing ($\because W' = 1$)

1. $$\tan \alpha = \frac{\text{Horizontal force}}{\text{Vertical force}}$$
$$\tan \alpha = \frac{10}{80} = 0.125$$
$$\alpha = 7.12° \approx 7°$$
2. Eccentricity (e) of the resultant load can be calculated as,
$$e / 3.3 = \tan \alpha = 0.125$$
3. Eccentricity, $e = 3.3 \times 0.125 = 0.41$ m
4. Reduced dimension B' an account of eccentricity of loading is given by,
$$B' = B - 2e = 2.5 - 2 \times 0.41 = 1.68 \text{ m}$$
5. Corrected Area,
$$A' = B'L = 1.68 \times 4 = 6.72 \text{ m}^2$$
6. Shape factors :
$$s_c = 1 + 0.2\frac{B'}{L} = 1 + 0.2\left(\frac{1.68}{4}\right) = 1.084$$
$$s_q = 1 + 0.2\frac{B'}{L} = 1 + 0.2\left(\frac{1.68}{4}\right) = 1.084$$
$$s_\gamma = 1 - 0.4\frac{B'}{L} = 1 - 0.4\left(\frac{1.68}{4}\right) = 0.832$$
7. Depth factors : $d_q = d_\gamma = 1 + 0.1\frac{D_f}{B'}\tan\left(45 + \frac{\phi}{2}\right)$
$$= 1 + 0.1 \times \frac{1.8}{1.68}\tan\left(45 + \frac{20}{2}\right) = 1.153$$

$$i_c = i_q = i_r = 1$$

3. Shape factors : $s_c = 1 + 0.2\,B/L = 1 + 0.2 \times \frac{1.8}{3} = 1.12$

$$s_q = 1 + 0.2 \times B/L = 1 + 0.2 \times \frac{1.8}{3} = 1.12$$

$$s_\gamma = 1 - 0.4 \times B/L = 1 - 0.4 \times \frac{1.8}{3} = 0.76$$

4. Depth factors : $d_c = 1 + 0.2\,(D/B)\tan(45° + \phi/2)$

$$= 1 + 0.2 \times \frac{1.5}{1.8} \times \tan\left(45° + \frac{32°}{2}\right) = 1.3$$

$$d_q = d_\gamma = 1 + 0.1\,(D/B)\tan(45° + \phi/2)$$

$$= 1 + 0.1 \times \left(\frac{1.5}{1.8}\right)\tan\left(45° + \frac{32°}{2}\right) = 1.15$$

5. Saturated weight,

$$\gamma_{sat} = \frac{(1+w)G\,\gamma_w}{1+wG} = \frac{(1+0.15)\times 2.67 \times 10}{1+0.15\times 2.67}$$

$$\gamma_{sat} = 21.92\ \text{kN/m}^3$$

6. $q_{nu} = 8 \times 36 \times 1.12 \times 1.3 \times 1 + (21.92 \times 1.5) \times (23 - 1) \times 1.12 \times 1.15 \times 1 + 0.5 \times 1.8 \times 21.92 \times 30 \times 0.76 \times 1.15 \times 1 \times 1$

$$q_{nu} = 1868.3\ \text{kN/m}^2$$

7. Safe load bearing capacity $= \frac{1868.3}{3} = 622.77\ \text{kN/m}^2$

8. Safe load $= 622.77 \times (1.8 \times 3) = 3362.958\ \text{kN} \approx 3363\ \text{kN}$