Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.1 : Introduction]

Objectives

In this section you will learn the following

- introduction
- Basic definitions
- Presumptive bearing capacity

17. Bearing capacity : It is the load carrying capacity of the soil.

Basic definitions

Ultimate bearing capacity or Gross bearing capacity (q_u): It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

Net ultimate bearing capacity (q_{un}): It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil (γ) are close enough to be considered equal, then

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

where,

D_f is the depth of the footing, Ref. fig. 4.7

Safe bearing capacity: It is the bearing capacity after applying the factor of safety (FS). These are of two types,

- Safe net bearing capacity (q_{ns}) : It is the net soil pressure which can be safety applied to the soil considering only shear failure. It is given by,

$$q_{ns} = \frac{q_{nu}}{FS}$$

- Safe gross bearing capacity (q_s): It is the maximum gross pressure which the soil can carry safely without shear failure. It is given by,

$$q_s = q_{ns} + \gamma D_f$$

Allowable Bearing Pressure: It is the maximum soil pressure without any shear failure or settlement failure.

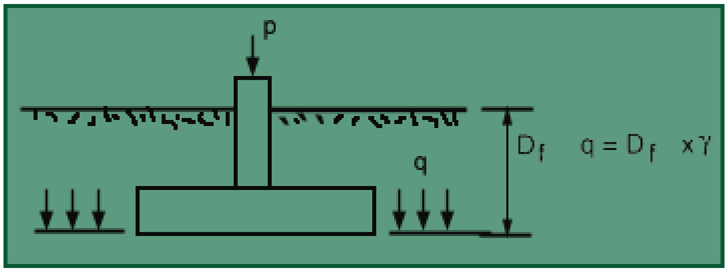


Fig. 4.7 Bearing capacity of footing

Presumptive bearing capacity : Building codes of various organizations in different countries gives the allowable bearing capacity that can be used for proportioning footings. These are “Presumptive bearing capacity values based on experience with other structures already built. As presumptive values are based only on visual classification of surface soils, they are not reliable. These values don't consider important factors affecting the bearing capacity such as the shape, width, depth of footing, location of water table, strength and compressibility of the soil. Generally these values are conservative and can be used for preliminary design or even for final design of small unimportant structure. IS1904-1978 recommends that the safe bearing capacity should be calculated on the basis of the soil test data. But, in absence of such data, the values of safe bearing capacity can be taken equal to the presumptive bearing capacity values given in table 4.1, for different types of soils and rocks. It is further recommended that for non-cohesive soils, the values should be reduced by 50% if the water table is above or near base of footing.

Table 4.1 Presumptive bearing capacity values as per IS1904-1978.

Type of soil / rock	Safe / allowable bearing capacity (KN/ m^2)
Rock	3240
Soft rock	440
Coarse sand	440
Medium sand	245
Fine sand	440
Soft shell / stiff clay	100
Soft clay	100
Very soft caly	50

Recap

In this section you have learnt the following

- Introduction
- Basic definitions
- Presumptive bearing capacity

Objectives

In this section you will learn the following

- Various methods of determining bearing capacity
- Presumptive Analysis

Methods of determining bearing capacity

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

- Presumptive Analysis
- Analytical Methods
- Plate Bearing Test
- Penetration Test
- Modern Testing Methods
- Centrifuge Test

1. Presumptive analysis

This is based on experiments and experiences.

For different types of soils, IS1904 (1978) has recommends the following bearing capacity values.

Table 4.2 Bearing Capacity Based on Presumptive Analysis

Types	Safe /allowable bearing capacity(kN/m^2)
Rocks	3240
Soft rocks	440
Coarse sand	440
Medium sand	245
Fine sand	100
Soft shale/stiff clay	440
Soft clay	100
Very soft clay	50

Recap

In this section you have learnt the following

- Various methods of determining bearing capacity
- Presumptive Analysis

Objectives

In this section you will learn the following

- Prandtl's Analysis
- Terzaghi's Bearing Capacity Theory
- Skempton's Analysis for Cohesive soils
- Meyerhof's Bearing Capacity Theory
- Hansen's Bearing Capacity Theory
- Vesic's Bearing Capacity Theory
- IS code method

Analytical methods

The different analytical approaches developed by various investigators are briefly discussed in this section.

- **Prandtl's Analysis**

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

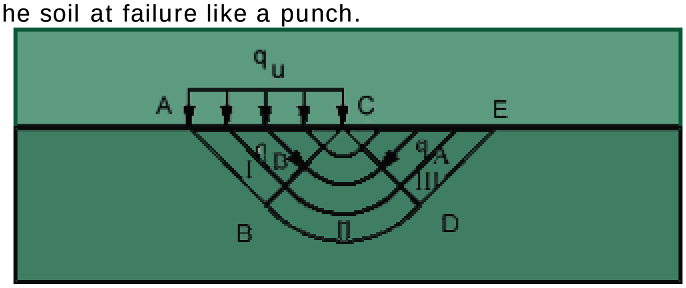


Fig 4.8 Prandtl's Analysis

Prandtl analysed the problem of the penetration of a punch into a weightless material. The punch was assumed rigid with a frictionless base. Three failure zones were considered.

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

Zone1 consist of a triangular zone and its boundaries rise at an angle $45 + \phi/2$ with the horizontal two zones on either side represent passive Rankine zones. The boundaries of the passive Rankine zone rise at angle of $45 - \phi/2$ with the horizontal. Zones 2 located between 1 and 3 are the radial shear zones. The bearing capacity is given by (Prandtl 1921) as

$$q_d = cN_c$$

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

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

Reissner (1924) extended Prandtl's analysis for uniform load q per unit area acting on the ground surface. He assumed that the shear pattern is unaltered and gave the bearing capacity expression as follows.

$$q_d = cN_c + qN_q$$

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

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

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

$$q_d = (\pi + 2)c + q$$

=5.14c+q

if q=0,

we get $q_d = 5.14c$ =2.57q_u

where q_u is the unconfined compressive strength.

- **Terzaghi's Bearing Capacity Theory**

  Assumptions in Terzaghi's Bearing Capacity Theory

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

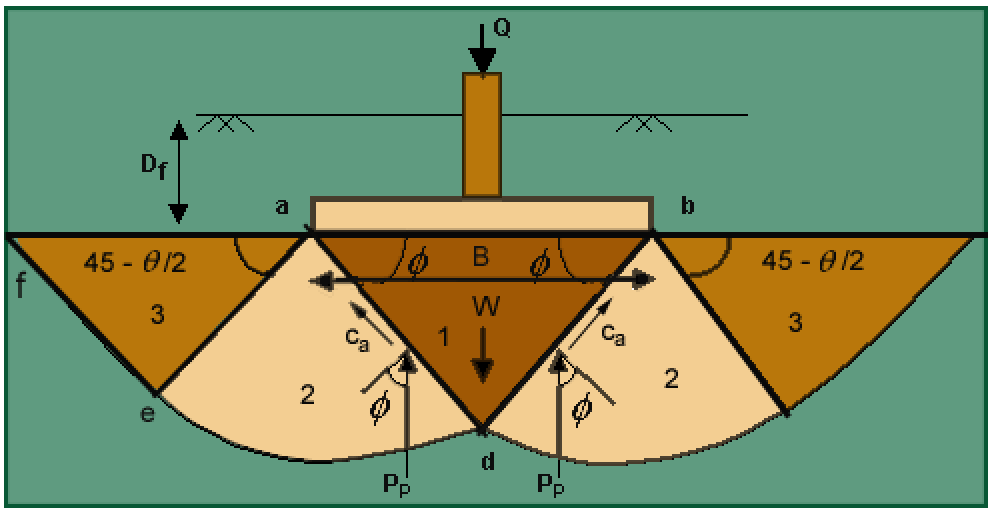


Fig. 4.9 Terzaghi's Bearing Capacity Theory

Consider a footing of width B and depth D_f loaded with Q and resting on a soil of unit weight γ. The failure of the zones is divided into three zones as shown below. The zone1 represents an active Rankine zone, and the zones 3 are passive zones.the boundaries of the active Rankine zone rise at an angle of $45 + \phi/2$, and those of the passive zones at $45 - \phi/2$ with the horizontal. The zones 2 are known as zones of radial shear, because the lines that constitute one set in the shear pattern in these zones radiate from the outer edge of the base of the footing. Since the base of the footings is rough, the soil located between it and the two surfaces of sliding remains in a state of equilibrium and acts as if it formed part of the footing. The surfaces ad and bd rise at ϕ to the horizontal. At the instant of failure, the pressure on each of the surfaces ad and bd is equal to the resultant of the passive earth pressure P_P and the cohesion force C_a. since slip occurs along these faces, the resultant earth pressure acts at angle ϕ to the normal on each face and as a consequence in a vertical direction. If the weight of the soil adb is disregarded, the equilibrium of the footing requires that

$$Q_d = 2P_p + 2C_a \sin\phi = 2P_p + Bc\tan\phi \qquad \text{------- (1)}$$

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

The value of the bearing capacity may be calculated as :

$$Q_d = 2(P_p + P_C + Pq + \frac{1}{2}Bc\tan\phi) \qquad \text{------- (2)}$$

by introducing into eqn(2) the following values: $N_C = \frac{2P_C}{Bc} + \tan\phi$

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

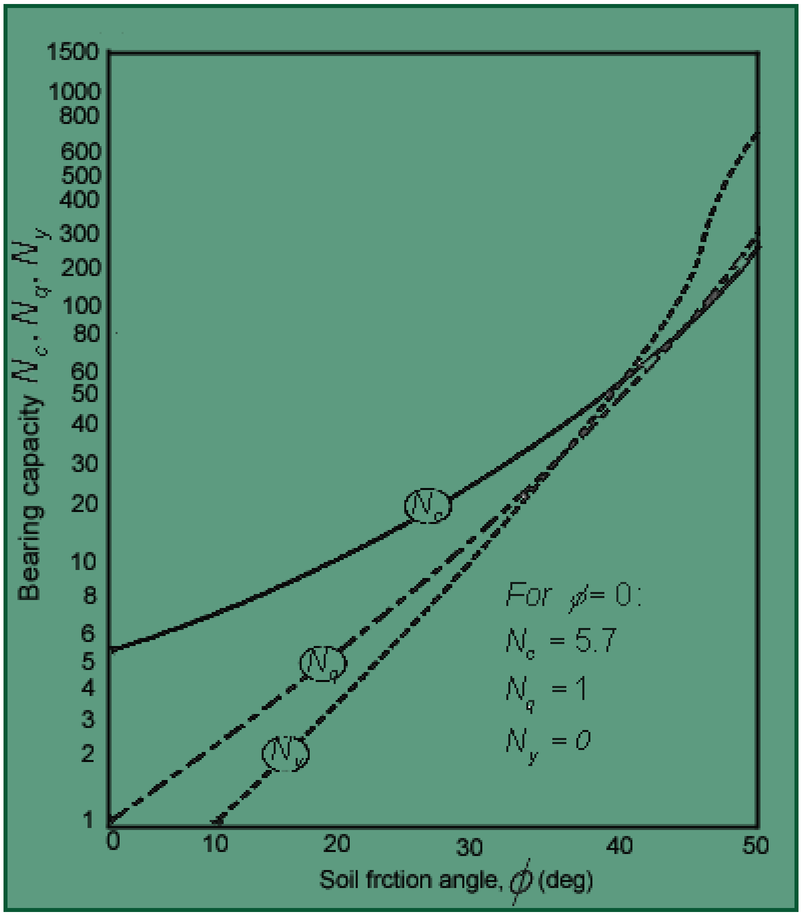


Fig.4.10 Variation of bearing capacity factors with ϕ

$$N_\gamma = \frac{4P'_p}{B^2\gamma}$$,

$$Q_d = B(cN_c + \gamma D_f N_q + \frac{1}{2}\gamma BN_\gamma)$$

the quantities N_q , N_c , N_γ are called bearing capacity factors.

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

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

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

$$a = \exp\left[\left(\frac{3\pi}{4}-\frac{\phi}{2}\right)\tan\phi\right]$$

where K_p = passive earth pressure coefficient, dependent on ϕ.

The use of chart figure (4.11) facilitates the computation of the bearing capacity. The results obtained by this chart are approximate.

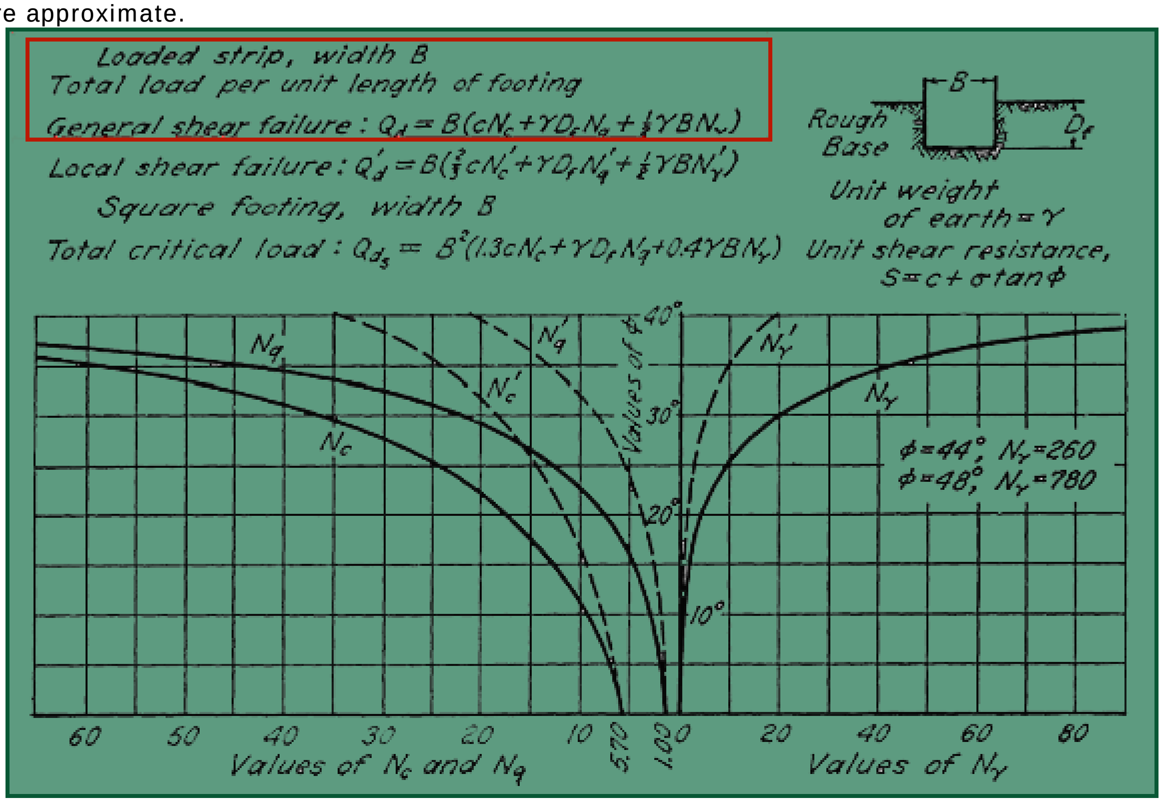


Fig 4.11 Chart Showing Relation between Angle of Internal Friction and Terzaghi's Bearing Capacity Factors

Table 4.3 : Terzaghi's bearing capacity factors

ϕ_f	N_c	N_q	N_γ		ϕ_f	N_c	N_q	N_γ
28	17.81	31.61	15.7		0	1.00	5.70	0.0
30	22.46	37.16	19.7		2	1.22	6.30	0.2
32	28.52	44.04	27.9		4	1.49	6.97	0.4
34	36.50	52.64	36.0		6	1.81	7.73	0.6
35	41.44	57.75	42.4		8	2.21	8.60	0.9
36	47.16	63.53	52.0		10	2.69	9.60	1.2
38	61.55	77.50	80.0		12	3.29	10.76	1.7
40	81.27	95.66	100.4		14	4.02	12.11	2.3
42	108.75	119.67	180.0		16	4.92	13.68	3.0
44	147.74	151.95	257.0		18	6.04	15.52	3.9
45	173.29	172.29	297.5		20	7.44	17.69	4.9
46	204.19	196.22	420.0		22	9.19	20.27	5.8
48	207.85	258.29	780.1		24	11.40	23.36	7.8
50	415.15	347.51	1153.2		26	14.21	27.06	11.7

Bearing capacity of square and circular footings

If the soil support of a continuous footing yields due to the imposed loads on the footings, all the soil particles move parallel to the plane which is perpendicular to the centre line of the footing. Therefore the problem of computing the bearing capacity of such footing is a plane strain deformation problem. On the other hand if the soil support of the square and circular footing yields, the soil particles move in radial and not in parallel planes. Terzaghi has proposed certain shape factors to take care of the effect of the shape on the bearing capacity. The equation can be written as, $q_u = nN_cS_c + \overline{\gamma}DN_qs_q + \frac{1}{2}\gamma BN\gamma s_\gamma$
where,

s_q , s_c , s_γ are the shape factors whose values for the square and circular footings are as follows, For long footings: s_c = 1, s_q = 1, s_γ = 1,

For square footings: s_c = 1.3, s_q = 1, s_γ = 0.8,

For circular footings: s_c = 1.3, s_q = 1, s_γ = 0.6,

For rectangular footing of length L and width B : s_c = $\left(1+0.3\frac{B}{L}\right)$, s_q = 1, s_γ = $\left(1-0.2\frac{B}{L}\right)$.

- **Skempton's Analysis for Cohesive soils**

Skempton (1951) has showed that the bearing capacity factors N_c in Terzaghi's equation tends to increase with depth for a cohesive soil.

For (D_f/B) < 2.5, (where D_f is the depth of footing and B is the base width).

(N_c) for rectangular footing = $5\left(1+\frac{0.2D_f}{B}\right)\left(1+\frac{0.2B}{L}\right) \le 9$

(N_c) for circular and rectangular footing = $6\left(1+\frac{0.2D_f}{B}\right)\left(1+\frac{0.2B}{L}\right) \le 9$

For (D_f/B) >= 2.5, (N_c) for rectangular footing = $7.5\left(1+\frac{0.2B}{L}\right) \le 9$

Ultimate bearing capacity

For $\phi_u = 0, N_q = 1, N_\gamma = 0$,

$q_u = c_u N_c + \gamma D_f$, where c_u is the undrained cohesion of the soil.

- **Meyerhof's Bearing Capacity Theory**

The form of equation used by Meyerhof (1951) for determining ultimate bearing capacity of symmetrically loaded strip footings is the same as that of Terzaghi but his approach to solve the problem is different. He assumed that the logarithmic failure surface ends at the ground surface, and as such took into account the resistance offered by the soil and surface of the footing above the base level of the foundation. The different zones considered are shown in fig. 4.12

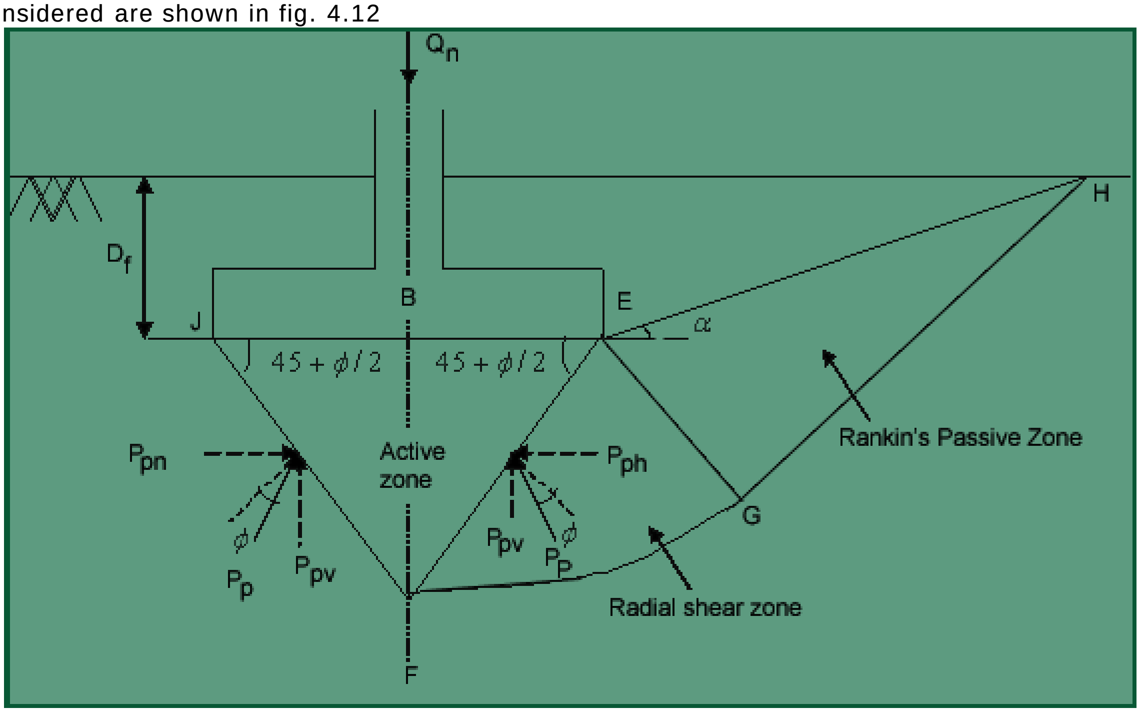


Fig. 4.12 Failure zones considered by Meyerhof

In this, EF failure surface is considered to be inclined at an angle of ($45 - \phi/2$) with the horizontal followed by FG which is logspiral curve and then the failure surface extends to the ground surface (GH).

EF is considered as a imaginary retaining wall face with failure surface as FGH. This problem is same as the retaining wall with the inclined backfill at an angle of a. For this case the passive earth pressure acting on the retaining wall Pp is given by Caqnot and Kerisel (1856). Considering the equilibrium of the failure zone,

$$\sum F_v = 0$$

$$Q_u + W = 2P_{pv}$$

where,

Q_u is the load on the footing,

W is the weight of the active zone and,

P_{pv} is the vertical component of the passive pressure acting on walls JF and EF.

Then the ultimate bearing capacity (qu) is given as,

$$q_u = \frac{Q_u}{Bx1} = \frac{2P_{pv} - W}{B} = \frac{2\left(P_{pcv} + P_{pqv} + P_{p\gamma v}\right) - W}{B}$$

Where, B is the width of the footing.

Comparing the above equation with,

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

We get ,

$$N_c = \frac{2P_{pcv}}{cB}$$

$$N_\gamma = \frac{2P_{p\gamma v} - W}{\frac{1}{2}\gamma B^2}$$

$$N_q = \frac{2P_{qv}}{qB}$$

The form of equation proposed by Meyerhof (1963) is,

$q_u = cN_c + d_c s_c i_c + \bar{\gamma} D N_q d_q s_q i_q + \frac{1}{2}\gamma B N_\gamma d_\gamma s_\gamma i_\gamma$ where, N_q , N_c , N_γ = Bearing capacity factors for strip foundation, c = unit cohesion,

s_q , s_c , s_γ = Shape factors,

i_q , i_c , i_γ = inclination factors for the load inclined at an angle a 0 to the vertical,

d_q , d_c , d_γ = Depth factors,

Table 4.4 shows the shape factors given by Meyerhof.

$\bar{\gamma}$ = effective unit weight of soil above base level of foundation,

γ = effective unit weight of soil below foundation base,

D = depth of the foundation.

In table 4.4,

$K_p = \tan^2(45 + \phi/2)$,

θ = angle of resultant measured from vertical without sign,

B = width of footing,

L = length of footing,

D = depth of footing.

- **Hansen's Bearing Capacity Theory**

For cohesive soils, Hansen (1961) gives the values of ultimate bearing capacity which are in better with experimental values.
According to Hansen, the ultimate bearing capacity is given by

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

where N_c, N_q,N_γ, are Hansen's bearing capacity factors and q is the effective surcharge at the base level, s_q , s_c , s_γ = Shape factors, i_q , i_c , i_γ = inclination factors for the load inclined at an angle a 0 to the vertical, d_q , d_c , d_γ Depth factors,

are the shape factors, d_q , d_c , d_γ are the depth factors and i_q , i_c , i_γ are inclination factors.

The bearing factors are given by the following equations.

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

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

$$N_\gamma = 1.8(N_q - 1)\tan\phi$$

- **Vesic's Bearing Capacity Theory**

Vesic(1973) confirmed that the basic nature of failure surfaces in soil as suggested by Terzaghi as incorrect. However, the angle which the inclined surfaces AC and BC make with the horizontal was found to be closer to $45 + \phi/2$ instead of ϕ. The values of the bearing capacity factors N_q , N_c , N_γ for a given angle of shearing resistance change if above modification is incorporated in the analysis as under

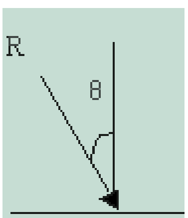
$$N_q = e^{\pi \tan\phi} \tan^2(45 + \phi/2) \quad \text{------(1)}$$

$$N_c = (N_q - 1)\cot\phi \quad \text{------(2)}$$

$$N_\gamma = 2(N_q + 1)\tan\phi \quad \text{------(3)}$$

eqns(1)was proposed by Prandtl(1921),and eqn(2) was given by Reissner (1924). Caquot and Keisner (1953) and Vesic (1973) gave eqn (3). The values of bearing capacity factors are given in table (4.5).

Table 4.4 Mayerhof bearing capacity factors

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

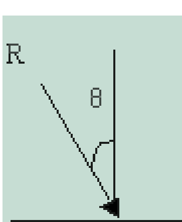
Factors	Value	For
Shape	$s_c = \left(1 + 0.2K_p \frac{B}{L}\right)$	Any ϕ
	$s_q = s_\gamma = \left(1 + 0.1K_p \frac{B}{L}\right)$	ϕ >10
	$s_q = s_\gamma = 1$	ϕ =0
Depth	$d_c = \left(1 + 0.2\sqrt{K_p} \frac{D}{B}\right)$	Any ϕ
	$d_q = d_\gamma = \left(1 + 0.1\sqrt{K_p} \frac{D}{B}\right)$	ϕ >10
	$d_q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

Table 4.5 Vesic's Bearing Capacity Factors

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
20^0	14.83	6.40	5.39	28^0	25.80	14.72	16.72
22^0	16.88	7.82	7.13	30^0	30.14	18.40	22.40
24^0	19.32	9.60	9.44	32^0	35.49	23.18	30.22
26^0	22.25	11.85	12.54	34^0	42.16	29.44	41.06

Table 4.6 Shape Factors Given By Vesic

Shape of footing	s_c	s_q	s_γ
Strip	1	1	1
Rectangle	$1+(B/L)(N_q/N_c)$	$1+(B/L)\tan\phi$	$1-0.4(B/L)$
Circle and square	$1+(N_q/N_c)$	$1+\tan\phi$	0.6

Bearing capacity is similar to that given by Hansen.

But the depth factors are taken as:

$d_c = 1+0.4(D_f/B)$, $d_q = 1+2\tan\phi(1-\sin\phi)^2(D_f/B)$, $d_\gamma = 1$

Inclination factors $i_c = i_q = (1-\alpha/90^0)^2$

$i_\gamma = (1-\alpha/90^0)^2$ where α is the inclination of the load with the vertical.

Bearing Capacity Factors

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.00	0.00	15	11.0	3.94	1.42	30	30.1	18.4	18.1
1	5.38	1.09	0.00	16	11.6	4.34	1.72	31	32.7	20.6	21.2
2	5.63	1.20	0.01	17	12.3	4.77	2.08	32	35.5	23.2	24.9
3	5.90	1.31	0.03	18	13.1	5.26	2.49	33	38.6	26.1	29.3
4	6.19	1.43	0.05	19	13.9	5.80	2.97	34	42.2	29.4	34.5
5	6.49	1.57	0.09	20	14.8	6.40	3.54	35	46.1	33.3	40.7
6	6.81	1.72	0.14	21	15.8	7.07	4.19	36	50.6	37.8	48.1
7	7.16	1.88	0.19	22	16.9	7.82	4.96	37	55.6	42.9	56.9
8	7.53	2.06	0.27	23	18.0	8.66	5.85	38	61.4	48.9	67.4
9	7.92	2.25	0.36	24	19.3	9.60	6.89	39	67.9	56.0	80.1
10	8.34	2.47	0.47	25	20.7	10.7	8.11	40	75.3	64.2	95.4
11	8.80	2.71	0.60	26	22.3	11.9	9.53	41	83.9	73.9	114
12	9.28	2.97	0.76	27	23.9	13.2	11.2	42	93.7	85.4	137
13	9.81	3.26	0.94	28	25.8	14.7	13.1	43	105	99.0	165
14	10.4	3.59	1.16	29	27.9	16.4	15.4	44	118	115	199
								45	134	135	241
								46	152	159	294
								47	174	187	359
								48	199	222	442
								49	230	265	548
								50	267	319	682

Values of N_c **after Prandtl (1921)**
N_q **after Reissner (1924)**
N_γ **after Hansen (1961)**

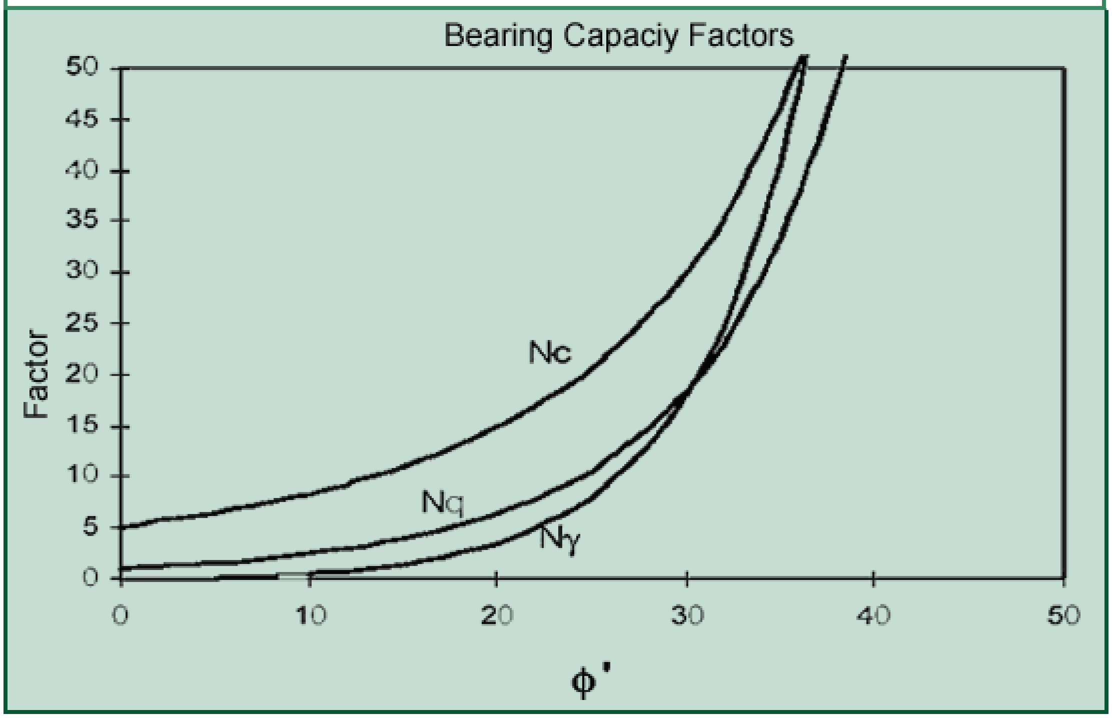


Figure 4.13 Bearing Capacity Factors Given by Prandtl, Hansen and Reissner

- **IS code method**

IS: 6403-1981 gives the equation for the net ultimate bearing capacity as

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

The factor W' takes into account, the effect of the water table. If the water table is at or below a depth of D_f +B, measured from the ground surface, W'=1. If the water table rises to the base of the footing or above, W'=0.5. If the water table lies in between then the value is obtained bylinear interpolation. The shape factors given by Hansen and inclination factors as given by Vesic are used. The depth factors are given below.

$$d_c = 1 + 0.2(D_f / B)\tan(45 + \phi/2)$$

$$d_q = d_\gamma = 1 for \phi' < 10^0$$

$$d_\gamma = d_q = 1 + 0.1(D_f / B)\tan(45^0 + \phi/2) for \phi > 10^0$$

For cohesive soils:

$$q_{nu} = cN_c s_c d_c i_c$$

where N_c=5.14 and s_c , d_c and i_c are respectively the shape, depth and inclination factors.

Recap

In this section you have learnt the following

- Prandtl's Analysis
- Terzaghi's Bearing Capacity Theory
- Skempton's Analysis for Cohesive soils
- Meyerhof's Bearing Capacity Theory
- Hansen's Bearing Capacity Theory
- Vesic's Bearing Capacity Theory
- IS code method

Objectives

In this section you will learn the following

- Test Procedure
- Report
- Calculation

Plate Bearing Test

Plate bearing test is an important field test for determining the bearing capacity of the foundation. In this a compressive stress is applied to the soil pavement layer through rigid plates of relatively large size and the deflections are measured for various stress values. The coefficient of sub-grade reaction is a very useful parameter in the design of rigid highway and airfield pavements. The modulus of sub-grade reaction K is used in rigid pavement analysis for determining the radius of relative stiffness ‘l' using the relation:

$$l = \left[\frac{Eh^3}{12K(l - H^2)}\right]^{\frac{1}{4}}$$

The exact load deflection behavior of the soil or the pavement layer in-situ for static loads is obtained by the plate bearing test. The supporting power of the soil sub-grade or a pavement layer may be found in pavement evaluation work. Repeated plate bearing test is carried out to find the sub-grade support in flexible pavement design by Mc Leod method.

Objective

To determine the modulus of sub-grade reaction (K) of the sub-grade soil by conducting the in-situ plate bearing test.

Apparatus

- Bearing Plates: Consist of mild steel 75 cm in diameter and 0.5 to 2.5 cm thickness and few other plates of smaller diameters (usually 60, 45, 30 and 22.5 cm) used as stiffeners.
- Loading equipment: Consists of a reaction frame and a hydraulic jack. The reaction frame may suitably be loaded to give the needed reaction load on the plate.
- Settlement Measurement: Three or four dial gauges fixed on the periphery of the bearing plate. The datum frame should be supplied for from the loading area.

Test Procedure

The plate load test shall be carried out in accordance to BS5930 or ASTM D1194 with the following additional requirement:

- Test pit should be at least 4 times as wide as the plate and to the foundation depth to be placed.
- The test shall be carried out at the same level of the proposed foundation level or as directed by the Engineer while the same conditions to which the proposed foundation will be subjected should be prepared if possible.
- At least three (3) test locations are required for calibration on size effect of test plates, and the distance between test locations shall not be less than five (5) times the diameter of the largest plate used in the tests
- The test surface should be undisturbed, planar and free from any crumbs and loose debris. When the test surface is excavated by machinery, the excavation should be terminated at 200mm to 300mm above the test surface and the test surface should be trimmed manually.
- To ensure even transference of the test load on to the test surface, the steel plate should be leveled and have full contact with the ground. Sand filling or cement mortar or plaster of Paris could level small uneven ground surface.
- If the test is carried out below the groundwater level, it is essential to lower the groundwater level by a system of wells or other measures outside and below the test position.
- The preparation of the test surface may cause an unavoidable change in the ground stress which may result in irreversible changes to the subsoil properties. It is essential that the exposure time of the test surface and the delay between setting up and testing should be minimized. The time lag shall be reported with the test result.
- Support the loading platforms or bins by cribbing or other suitable means, at points as far removed from the test area, preferably not less than 2.4m. The total load required for the test shall be available at the site before the test is started.
- The support for the beam with dial gauges or other settlement-recording devices shall not less than 2.4m from the center of the loaded area.
- Mackintosh Probe Test to be carried out at load test location (center of plate) at testing level before the test for calibration purpose.
- Loading shall be applied in 3 cycles. The time interval of each stage of loading should not less than 15 minute. Longer time interval is required at certain specified loading stages.
- The settlement at each stage of loading should be taken at the interval of every 15 minutes before and after each load increment. If the required time interval is more than 60 minutes, the reading shall be taken at every 15 minutes interval.
- In the load measurement, the test record sheet should include the targeted load schedule, load cell readings (primary measurement) & pressure gauges readings (secondary measurement).
- The testing contractor shall control the loading using load cell readings to achieve the targeted load in each stage of loading & record the actual readings in the load cell & the pressure gauge simultaneously.
- Continue each test until a peak load is reached or until the ratio of load increment to settlement increment reaches a minimum, steady magnitude. If sufficient load is available, continue the test until the total settlement reaches at least 10 percent of the plate diameter, unless a well-defined failure load is observed.

The test shall be discontinued if any of the following occurs:

1. Faulty jack or gauges,
2. Instability of the Kent ledge,
3. Improper setting of datum,
4. Unstable reference bench mark or reference beam,
5. Measuring instruments used are found to have been tempered.

Report

In addition to the continuous listing of all time, load, and settlement data for each test, the report shall include at least the followings:

- General information such as date, weather conditions, temperature, location of test, test surface soil description and others.
- Measured data. All data shall be checked for misreporting or miscalculation.
- Notes or abnormal phenomenon during the test shall be described.
- Load settlement relationship shall be plotted and presented in the report.
- Evaluation of the yielding load, elastic modulus, sub grade reaction and allowable bearing pressure.

Calculation

A graph is plotted with the mean settlement in mm on x axis and load kN/mm^2 y-axis. The pressure P corresponding to a settlement of A = 1.25 mm is obtained from the graph. The modulus of sub-grade reaction K is calculated from the relation

$K = \frac{P}{125} kN/mm^2/m$ or kN/mm ^{3}

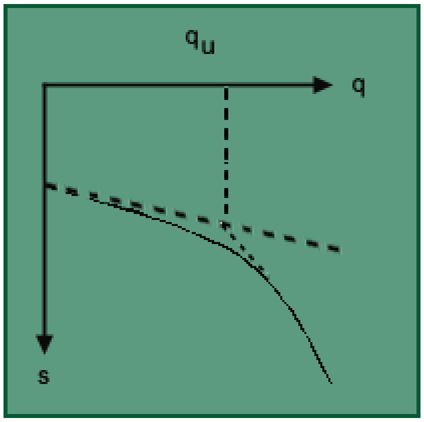


Fig. 4.14 Load settlement graph

Recap

In this section you have learnt the following

- Test Procedure
- Report
- Calculation

Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.5 : Standard Penetration Test]

Objectives

In this section you will learn the following

- Introduction
- Procedure
- Limitations

Standard Penetration Test

Method 1 .The ultimate bearing capacity of cohesion less soil is determined from the standard penetration number N. The standard penetration test is conducted at a number of selected points in the vertical direction below the foundation level at intervals of 75 cm or at point where there is a change of strata. An average value of N is obtained between the level of the base of footing and the depth equal to 1.5 to 2 times the width of the foundation. The value ϕ is obtained from the N value and the bearing capacity factors are found. It can also be directly found from figure 4.15.

Method 2 . As the bearing capacity depends upon ϕ and hence on N, it can be related directly to N. Teng (1962) gave the following equation for the net ultimate capacity of a strip footing.

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

$$q_{nu} = 0.5N^2 BW_\gamma + 0.83(100 + N^2)D_f W_q$$

where q_{nu} = net ultimate bearing capacity(kN/m^2),

B=width of footing, N=average SPT number, D_f=depth of footing. If D_f>B, use D_f=B.

W_γ and W_q are water table correction factor.

For square or circular footings,

$$q_{nu} = \frac{1}{3}\left[N^2 BW_\gamma + 3(100 + N^2)D_f W_q\right]$$

or $q_{nu} = 0.33N^2BW_\gamma + 1.0(100 + N^2)D_fW_q$

the net allowable bearing capacity can be obtained by applying a factor of safety of 3.0

for strip footings,

$$q_{ns} = 0..167N^2BW_\gamma + 2.77(100 + N^2)D_fW_q$$

for circular footings and square footings,

$$q_{ns} = 0..11N^2BW_\gamma + 0.33(100 + N^2)D_fW_q$$

• The allowable bearing pressure, for a footing on sand can be estimated from the results of an SPT test by means of the relationship between the SPT index, N, and the footing width, as given in Fig.4.15

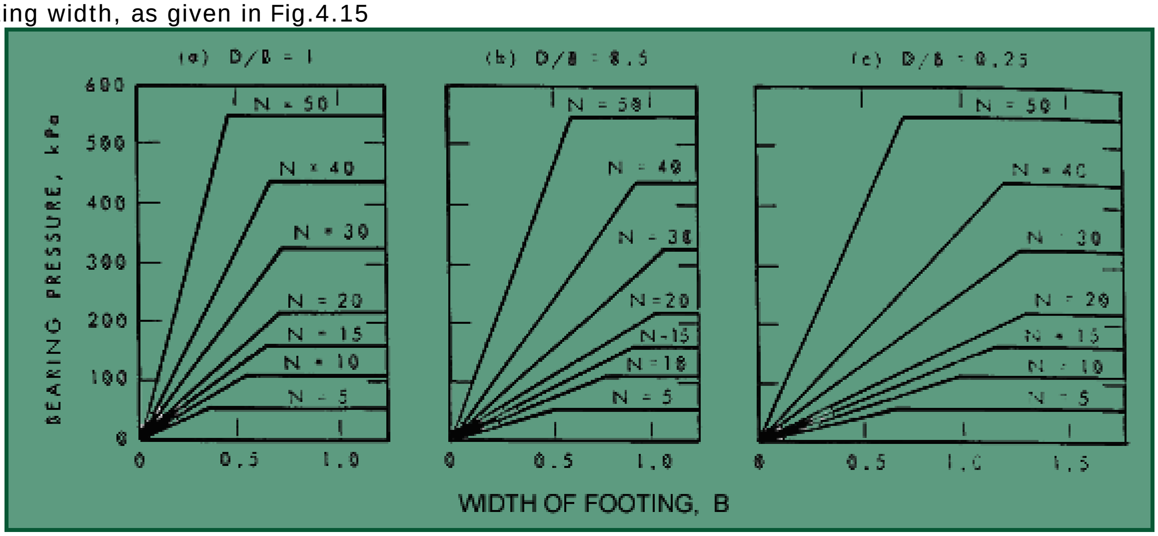


Figure 4.15 Design chart for proportioning footings on sand (after Peck et al., 1974)

- Values determined in this manner correspond to the case where the groundwater table is located deep below the footing foundation elevation.
- If the water table rises to the foundation level, no more than half the pressure values indicated in Fig 4.16 should be used.

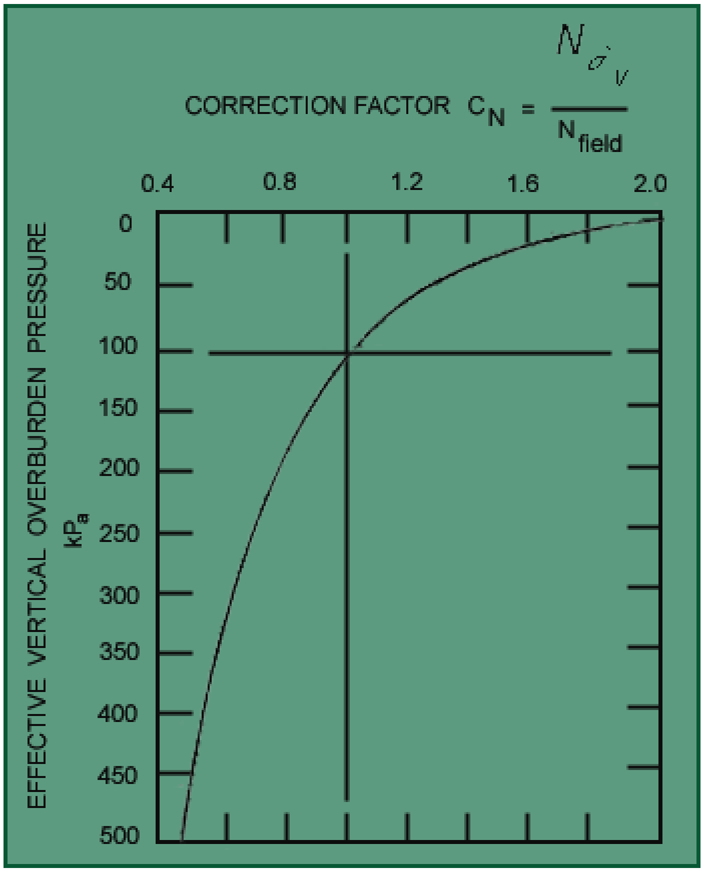


Fig.4.16 Chart for correcting SPT index, N values for depth (after Peck et al., 1974)

The charts are based on SPT indices obtained from a depth where the effective overburden pressure is about 100 KPa (about 5m). Indices obtained from other depths must be adjusted before using the charts. Fig. 4.16 indicates a correction factor, C_N , based on the effective overburden stress at the depth where the actual SPT was performed. The allowable bearing pressure determined from Fig. 4.15 is expected to produce settlements smaller than about 25 mm.

SPT Limitations:

The SPT is subject to many errors which affect the reliability of the SPT index, N. Correlation between the SPT index and the internal friction angle of sand is very poor. Consequently, the calculation of allowable bearing pressure from N values should be considered with caution. The SPT index is not appropriate for determining the bearing pressure in fine-grained cohesive soils.

Recap

In this section you have learnt the following

- Introduction
- Procedure
- Limitations

Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.6 : Modern Methods ; Centrifuge test]

Objectives

In this section you will learn the following

- Centrifuge test

Centrifuge test

Model testing represents a major tool available to the geotechnical engineer since it enables the study and analysis of design problems by using geotechnical materials. A centrifuge is essentially a sophisticated tool frame on which soil samples can be tested. Analogous to this exists in other branches of civil engineering: the hydraulic press in structural engineering, the wind tunnel in aeronautical engineering and the triaxial cell in geotechnical engineering. In all cases, a model is tested and the results are then extrapolated to a prototype situation.

Modeling has a major role to play in geotechnical engineering. Physical modeling is concerned with replicating an event comparable to what might exist in prototype. The model is often a reduced scale version of the prototype and this is particularly true for centrifuge modeling. The two events should obviously be similar and that similarity needs to be related by appropriate scaling laws. These are very standard in areas such as wind tunnel testing where dimensionless groups are used to relate events at different scales.

Modeling of foundation behavior is the main focus of many centrifuge studies. A wide range of foundations have been used in practical situations including spread foundations, pile foundations and caissons. The main objectives of centrifuge modeling for foundation behavior are to investigate:

- Load-settlement curves from which yield and ultimate bearing capacity as well as stiffness of the foundation may be determined.
- Stress distribution around and in foundations, by which the apportionment of the resistance of the foundation to bearing load and the integrity of the foundation may be examined.
- The performance of foundation systems under working loads as well as extreme loading conditions such as earthquakes and storms.

Recap

In this section you have learnt the following

- Centrifuge test

Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.7 : Presence of the Water Table & different modes of failure]

Objectives

In this section you will learn the following

- Presnece of water table
- Modes of Failure
  - General shear failure.
  - Local shear failure.
  - Punching shear failure.

Presence of the Water Table

In granular soils, the presence of water in the soil can substantially reduce the bearing capacity.

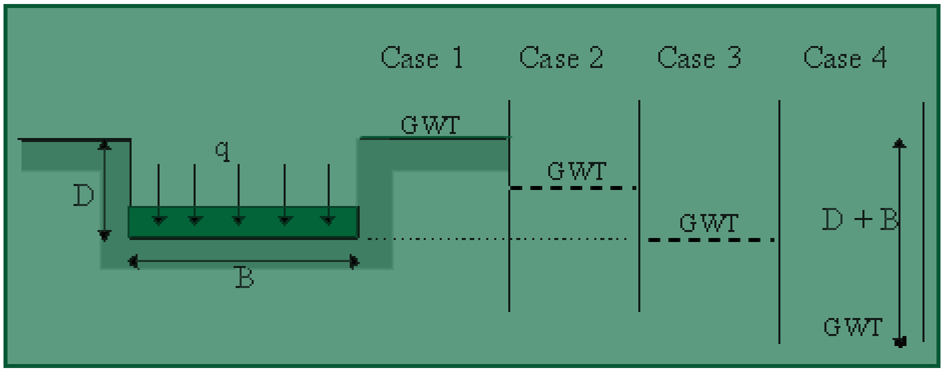
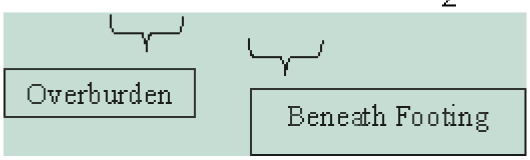


Fig 4.17 footing with various levels of water table

Case 1 : use γ' for the $\gamma D N_q$ and $\frac{1}{2}B\gamma N\gamma$ terms



Case 2 : for the $\gamma D N_q$ = $\sigma' N_q$ term calculate the effective stress at the depth of the footing

$\sigma' = \sigma - u = \gamma D - \gamma_w h_w$, and

for the $\frac{1}{2}B\gamma N$ use γ'.

Case 3 : use γ for the $\gamma D N_q$ term, and

use γ' for the $\frac{1}{2}B\gamma N\gamma$ term.

Case 4 : use γ for the $\gamma D N_q$ and $\frac{1}{2}B\gamma N\gamma$ terms.

In cohesive soils for short-term, end-of-construction conditions use:

$\gamma = \gamma_t$ and $\phi = 0$ N_c = 5.14, N_q = 1, and N_γ = 0

Thus

$$q_u = 5.14c + \gamma_t D$$

Modes of Failure

There are three principal modes of shear failure:

- General shear failure.
- Local shear failure.
- Punching shear failure.

General shear failure results in a clearly defined plastic yield slip surface beneath the footing and spreads out one or both sides, eventually to the ground surface. Failure is sudden and will often be accompanied by severe tilting. Generally associated with heaving. This type of failure occurs in dense sand or stiff clay.

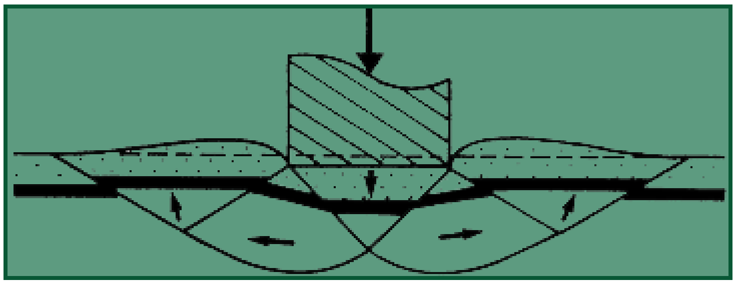


Fig. 4.18 General shear failure

Local shear failure results in considerable vertical displacement prior to the development of noticeable shear planes. These shear planes do not generally extend to the soil surface, but some adjacent bulging may be observed, but little tilting of the structure results. This shear failure occurs for loose sand and soft clay.

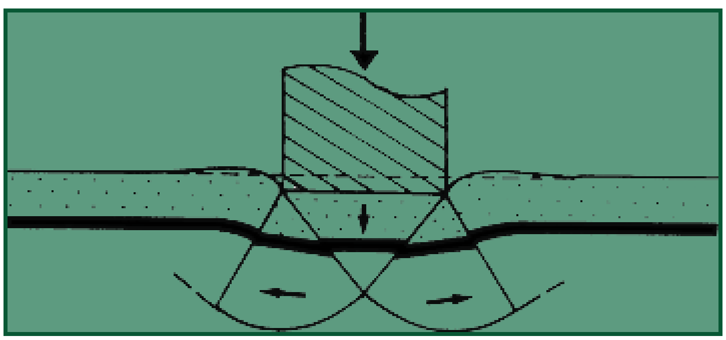


Fig. 4.19 Local shear failure.

Punching shear failure occurs in very loose sands and soft clays and there is little or no development of planes of shear failure in the underlying soil. Slip surfaces are generally restricted to vertical planes adjacent to the footing, and the soil may be dragged down at the surface in this region.

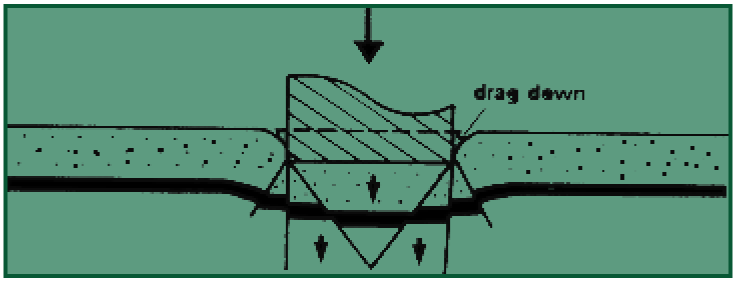


Fig. 4.20 Punching shear failure.

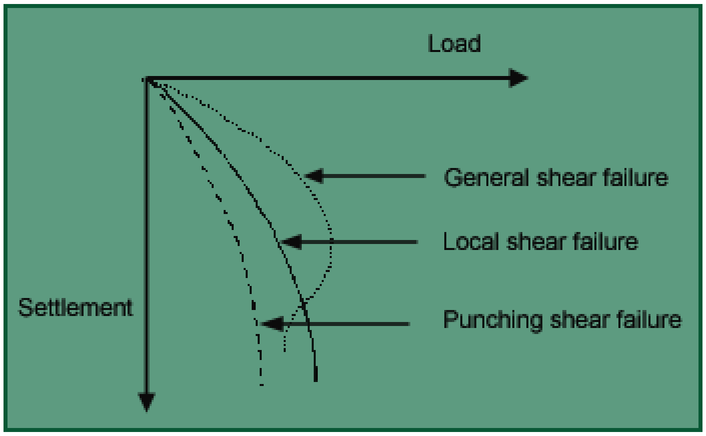


Fig. 4.21 Load settlement curves for different shear

From the curves the different types of shear failures can be predicted :

- For general shear failure there is a pronounced peak after which load decreases with increase in settlement. The load at the peak gives the ultimate stress or load.
- For local shear failure there is no pronounced peak like general shear failure and hence the ultimate load is calculated for a particular settlement.
- For punching shear failure the load goes on increasing with increasing settlement and hence there is no peak resistance.

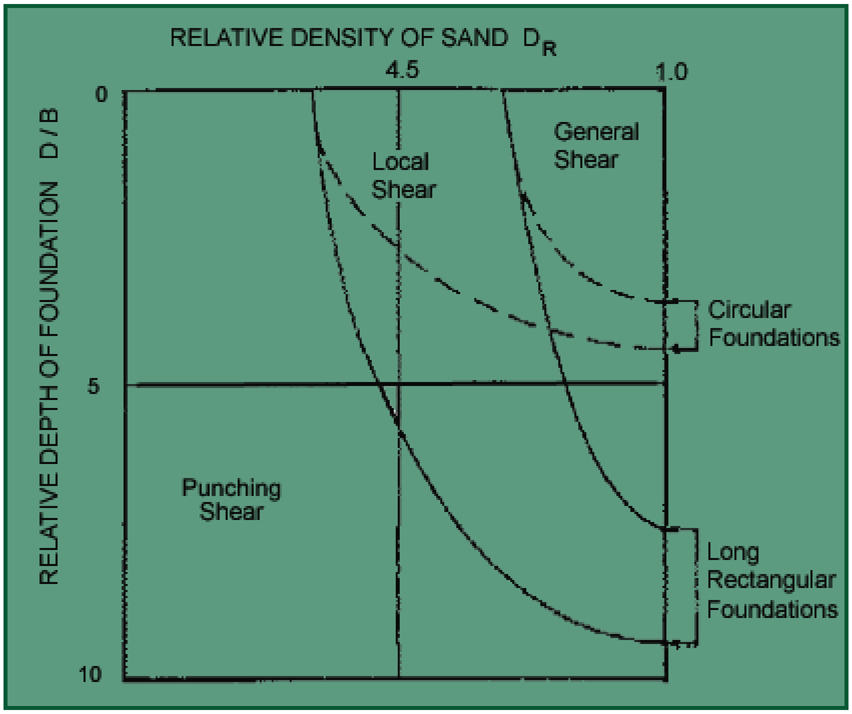


Fig. 4.22 Variation of the nature of bearing capacity failure in sand with Relative density D_R and relative depth D/B (Vesic 1963)

Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.7 : Presence of the Water Table & different modes of failure]

As per Terzaghi the bearing capacity equation is as follows:

$$q_u = cN_c + 0.5\gamma BN\gamma + \gamma DN_q$$

The above equation is valid for general shear failure but with certain modifications also applicable for local shear failure.

If, $\phi < 29^o$ => local shear failure.

$\phi > 36^o$ => general shear failure.

$29^o < \phi < 36^o$ => combined shear failure.

For local shear failure $c' = 2/3\ c$ and $\phi' = \tan^{-1}(2/3 \tan \phi)$

Say, $\phi = 25^o$ this implies that the failure is local shear failure. So for $\phi = 25^o$ refer to the chart of local shear failure, or convert ϕ to ϕ' (= 17.26 ^{o}) and for that angle refer to general shear chart. Also use c' and not c

Table 4.13 Terzaghi's bearing capacity factors

Tarzaghi Dimensionless Bearing Capacity Factors (after Bowles 1988)

ϕ_f	N_q	N_c	N_γ	ϕ_f	N_q	N_c	N_γ
28	17.81	31.61	15.7	0	1.00	5.70	0.0
30	22.46	37.16	19.7	2	1.22	6.30	0.2
32	28.52	44.04	27.9	4	1.49	6.97	0.4
34	36.50	52.64	36.0	6	1.81	7.73	0.6
35	41.44	57.75	42.4	8	2.21	8.60	0.9
36	47.16	63.53	52.0	10	2.69	9.60	1.2
38	61.55	77.50	80.0	12	3.29	10.76	1.7
40	81.27	95.66	100.4	14	4.02	12.11	2.3
42	108.75	119.67	180.0	16	4.92	13.68	3.0
44	147.74	151.95	257.0	18	6.04	15.52	3.9
45	173.29	172.29	297.5	20	7.44	17.69	4.9
46	204.19	196.22	420.0	22	9.19	20.27	5.8
48	207.85	258.29	780.1	24	11.40	23.36	7.8
50	415.15	347.51	1153.2	26	14.21	27.06	11.7

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

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

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

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

Recap

In this section you have learnt the following

- Presnece of water table
- Modes of Failure
  - General shear failure.
  - Local shear failure.
  - Punching shear failure.

Module 4 : Design of Shallow Foundations

Lecture 17 : Bearing capacity [Section17.8 : Bearing capacity of layered soil]

Objectives

In this section you will learn the following

- Bearing capacity of layered soil.
- Bulton Method (1953)
- By Bowle's method
- Bearing Capacity of the Rock (shallow Foundation)
- Depth of shallow foundations

Bearing capacity of layered soil.

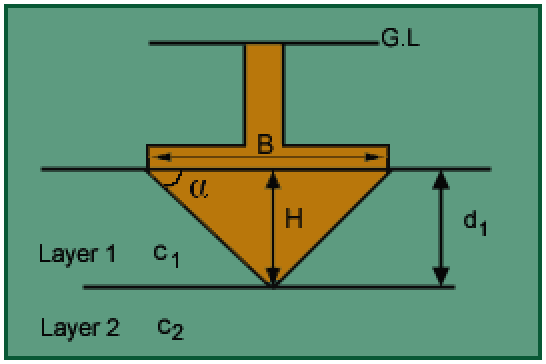


Fig 4.23 Bearing Capacity on Layered Soil

If $d_1 > H$ No effect of layered soil.

If $d_1 < H$ Effect of layered soil considered.

Three general cases of footing on a layered soil may be there :

Case 1 : Footing on layered clays (ϕ =0)

a) Top layer weaker than lower layer ($c_1 < c_2$)

b) Top layered stronger than lower layer ($c_1 > c_2$)

Case 2 : Footing on layer c- ϕ soil a, b same as in case 1.

Case 3 : Footing on layered sand and clay soils

a) Sand overlying clay

b) Clay overlying sand

These cases might be analytically sholved by using a number of methods among which Button's methods (1953) was the first of its kind.

- **Button Method (1953)**

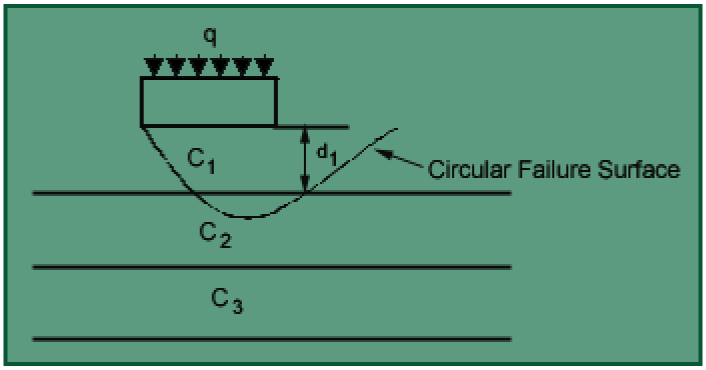


Fig. 4.24 Bearing Capacity on Layered Soil

Applicable when C_r lies between 0.6< C_r <1.3, where C_r = (c_2 / c_1)<1.

When this condition is not satisfied then use the following method to obtain N_c as given by Brown & Meyerhof (1969) based on model tests.

For clays C_r = (c_2 / c_1)<1, bottom layer is weaker soil.

For strip footing

$$N_c = [\frac{1.5.d_1}{B} + 5.14C_r] < 5.14$$

For ϕ = 0 , N_c =5.14.

For circular footing

$$N_c = [\frac{3.d_1}{B} + 6.05C_r] \leq 6.05$$

When c_r >0.7 reduced the above value of N_c by 10%.

When c_r > 1.00

a. For both the layers (For strip footing)

$$N_{c_1} = 4.14 + \frac{0.5B}{d_1}$$

$$N_{c_2} = 4.14 + \frac{1.1.B}{d_1}$$

b. For Circular footing

$$N_{c_1} = 0.05 + \frac{0.35B}{d_1}$$

$$N_{c_2} = 0.05 + \frac{0.66B}{d_1}$$

$$N_{c(avg)} = \frac{d_1.N_{c_1} + N_{c_2}(H - d_1)}{H}$$

- **By Bowels' method**

$$H = \frac{B}{2}\tan\alpha \quad \text{if}(H > d_1)$$

$$\phi_m = \left[d_1.\phi_1 + (H - d_1).\phi_2\right]/H$$

$$C_m = \frac{d_1.c_1 + (H - d_1).C_2}{H}$$

Fig -4.25 Calculation of Avg value of Cohesion by Bowles Method

- **Bearing Capacity of the Rock (shallow Foundation)**

Factor of safety required 3 (for sound rock) to 6 (Weak or Fissured rock).

For sound rock

$$N_q = \tan^6(45 + \frac{\phi}{2})$$

$$N_c = 5\tan^6(45 + \frac{\phi}{2})$$

$$N_\gamma = N_q + 1$$

For fissured rock or any other type of rock

q_{ult} is calculated by the equation given by Terzaghi.

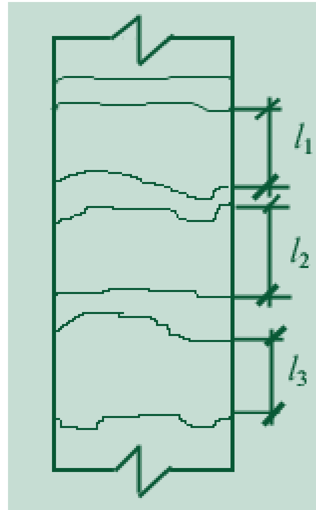
q_{ult} = Q_{ult} of sound rock × (RQD)2 .

RQD means rock quality designations.

For calculation of the RQD value take the pieces of the rock which is having length greater than 10 cm.

$$RQD = \frac{l_1 + l_2 + l_3}{L}$$

l_1, l_2, l_3, are having length greater than 10 cm and L is the length of core advance



- **Depth of shallow foundations**

1. for soft strata.
By Bells equation

$$D_f = \frac{1}{\gamma}\left[q.K_a^2 - 2C\sqrt{K_a}\left(1 + K_a\right)\right]$$

q = Soil pressure at the base of the footing.

K_a = active earth pressure coefficient.

c = Cohesion of the soil.

γ= Unit weight of soil.

D_f = Depth of the foundation.

2. If very hard strata is available even then we provide some depth of foundation according to IS 1904 i.e. min depth 80 cm.

Recap

In this section you have learnt the following

- Bearing capacity of layered soil.
- Bulton Method (1953)
- By Bowle's method
- Bearing Capacity of the Rock (shallow Foundation)
- Depth of shallow foundations

Congratulations, you have finished Lecture 17. To view the next lecture select it from the left hand side menu of the page