BEARING CAPACITY BASED ON IN-SITU AND LABORATORY TESTS

IN-SITU TESTS

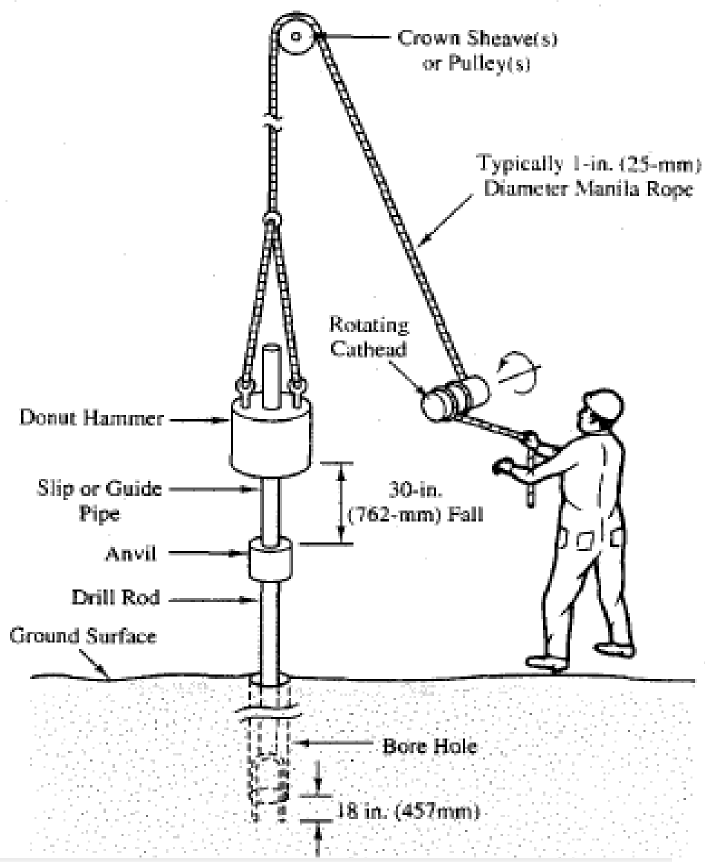
BC can be estimated by in-situ and laboratory tests, the most common in-situ tests for BC evaluation are

- Standard Penetration Test (SPT), very cheap, most common and popular
- Cone Penetration Test (CPT), very sophisticated, costly, not very common as compared with SPT
- Vane shear Test

SPT

SPT-N Value:

Number of blows required for 12 inch penetration of split spoon sampler under the impact of a standard wt. of 140 lbs dropped from a height of 30 inch



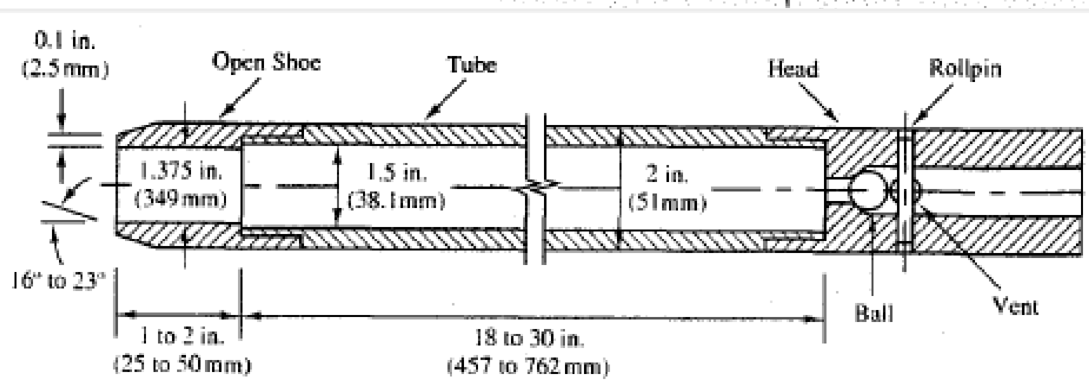


Figure 4.8 The SPT sampler (Adapted from ASTM D1586; Copyright ASTM, used with permission).

- It consists of penetrating a sampler known as split spoon sampler by dropping a standard weight of 140 lbs by 30 inch height. The sampler is penetrating by 18 inches total and for each 6 inches; the number of blows required for each of the penetration are counted separately. The number of blows for first 6 inches is ignored and the total numbers of blows for next two 6 inch penetration (total 12 inch) are taken and known as SPT-N value. (e.g. if the respective blow count for three successive 6 inch penetration are 8, 9, 10 then SPT-N value is 19.)
- As a part of test, the representative but disturbed soil sample is procured at the test depth for laboratory testing.

- SPT is generally performed at every 1 m interval up to 15~20 m and then interval may be increased to 1.5-2 m.

- If SPT is performed below GWT, the SPT-N values is overestimated and a correction to measured N is (dilatancy correction) applied if SPT-N value exceeds 15

$$N_{corr.} = 15 + 0.5(N_{measured} - 15)$$

- The SPT is more reliable for granular soils as compared with fine grained soils.
- In case of gravels, a 60^o cone is used in stead of split spoon samples

- If SPT is performed below GWT, sand boiling causes disturbance leading to erroneous SPT-N values. The borehole casing should be filled with water all the time to avoid sand boiling in case of light percussion technique.

- The SPT-N value has the following correlation with different parameters.

GRANULAR SOILS

Description	**Very Loose**	**Loose**	**Medium**	**Dense**	**Very Dense**
Relative density, D_r	0 – 0.15	0.15 – 0.35	0.35 – 0.65	0.65 – 0.85	0.85 – 1.00
Standard Penetration Test value, *N*	**0 – 4**	**5 – 10**	**11 – 30**	**31 – 50**	**51 – UP**
Approximate angle of internal friction, ϕ (degree)	25 – 28	28 – 30	30 – 35	35 – 40	38 – 43
Approximate range of moist unit weight, γ (pcf)	70 – 100	90 – 115	110 – 130	110 – 140	130 – 150
Submerged unit weight, γ_{sub}	60	55 – 65	60 – 70	65 – 85	75

COHESIVE SOILS

Description	**Very Soft**	**Soft**	**Firm**	**Stiff**	**Very Stiff**	**Hard**
Unconfined compressive strength, q_u (tsf)	0 – 0.25	0.25 – 0.5	0.5 – 1.0	1.0 – 2.0	2.0 – 4.0	4.0 – UP
Standard Penetration Test value, *N*	**0 – 2**	**3 – 4**	**5 – 8**	**9 – 16**	**17 – 32**	**33 – UP**
Approx. range of saturated unit weight, γ_{sat} (pcf)	100 – 120		100 – 130	120 – 140		130^+

1. Bearing Capacity from SPT

a. Terzaghi & Peck (1948) Method

Terzaghi & Peck (1948) were first to propose a correlation between SPT-N value and allowable pressure for a settlement of 25 mm (1 inch). The estimation of q_a is considered to be very conservative and is generally not used by current practitioners. The equation is as inder:

$$q_a = 720(N-3)\left(\frac{B+1}{2B}\right)^2 K_d R'_w \qquad N > 3$$

where q_a = net allowable bearing pressure in psf for 1 inch settlement and B in ft.

or $$q_a = 34.5(N-3)\left(\frac{B+0.305}{2B}\right)^2 K_d R'_w, \qquad q_a \text{ in kPa, } B \text{ in m}$$

$$K_d = (1+0.33\frac{D}{B}) \leq 1.33, \qquad \text{(for R'w see Fig. 1)}$$

This equation can be modified for calculation of settlement for any given pressure

$$s = \frac{2.9}{(N-3)}\left(\frac{B}{B+0.305}\right)^2 qC_d K_w \qquad s \text{ in mm, } B \text{ in m and } q \text{ in kPa}$$

C_d = 1 for D/B = 0 $\quad$ C_d = 0.75 for D/B = 1

K_w = 1 for $D_w > B$ $\quad$ K_w = 2.0 for D_w = 0

b. Meyerhof (1956) method:

(SI units) $\quad q_a = 12\, N\, K_d \quad$ for $B \leq 1.2$ m

(Fps units) $\quad q_a = \frac{N}{4} K_d \quad$ for $B < 4$ ft

(SI units) $\quad q_a = 8N\left(\frac{B+0.3}{B}\right)^2 K_d \quad$ for $B > 1.2$ m

(Fps units) $\quad q_a = \frac{N}{6}\left(\frac{B+1}{B}\right)^2 K_d \quad$ for $B > 4$ ft

Where,

q_a = allowable bearing pressure for a maximum settlement of 25 mm or 1-inch, kPa or ksf.

N = SPT resistance in blows/300 mm = statistical average value for the footing influence zone of about 0.5B above footing base to at least 2B below.

B = footing width in meters or feet.

K_d = depth factor = $(1+0.33\frac{D}{B}) \le 1.33$

For any settlement, $$s_{\text{actual}} = \frac{s}{q_{\text{a}}} q_{\text{actual}}$$

For s = 25 mm, the above equations (in SI units) can be modified to determine settlement under the known contact pressure or vice versa as below:

$$s_{\text{actual}} = \frac{2}{N} C_d q_{\text{actual}} \qquad \text{for } B \le 1.2 \text{ m}$$

$$s_{\text{actual}} = \frac{3.12}{N}\left(\frac{B}{B+0.3}\right)^2 C_d q_{\text{actual}} \qquad \text{for } B > 1.2 \text{ m}$$

Where $C_d = \frac{1}{K_d} = \frac{1}{1+0.33\frac{D}{B}} \ge 0.75$ and ≤ 1.0

c. Meyerhof method modified by Bowles (1977):

$$\left.\begin{array}{lll} \text{(SI units)} & q_a = 20\, N\, K_d & \text{for } B \le 1.2 \text{ m} \\ \text{(Fps units)} & q_a = \dfrac{N}{2.5} K_d & \text{for } B < 4 \text{ ft} \end{array}\right\}$$

$$\left.\begin{array}{lll} \text{(SI units)} & q_a = 12.5N\left(\dfrac{B+0.3}{B}\right)^2 K_d & \text{for } B > 1.2 \text{ m} \\ \text{(Fps units)} & q_a = \dfrac{N}{4}\left(\dfrac{B+1}{B}\right)^2 K_d & \text{for } B > 4 \text{ ft} \end{array}\right\}$$

Where,

q_a = allowable bearing pressure for a maximum settlement of 25 mm or 1-inch, kPa or ksf.

N = SPT resistance in blows/300 mm = statistical average value for the footing influence zone of about 0.5B above footing base to at least 2B below.

B = footing width in meters or feet.

K_d = depth factor = $(1+0.33\dfrac{D}{B}) \le 1.33$

For any settlement, $$s_{\text{actual}} = \frac{s}{q_{\text{a}}} q_{\text{actual}}$$

For s = 25 mm, the above equations (in SI units) can be modified to determine settlement under the known contact pressure or vice versa as below:

$$s_{\text{actual}} = \frac{1.25}{N} C_d q_{\text{actual}} \qquad \text{for } B \le 1.2 \text{ m}$$

$$s_{\text{actual}} = \frac{2}{N}\left(\frac{B}{B+0.3}\right)^2 C_d q_{\text{actual}} \qquad \text{for } B > 1.2 \text{ m}$$

Where $C_d = \dfrac{1}{K_d} = \dfrac{1}{1+0.33\dfrac{D}{B}} \ge 0.75$ and ≤ 1.0

d. Teng (1962) Relations based on shear failure criterion

For strip footing:

$$q_{ult} = 3BN^2R'_w + 5(100+N^2)DR_w \qquad \text{(Fps units)}$$

The above equation may be modified for q_s (FS=3) in SI units

$$q_s = 0.157BN^2R'_w + 0.262(100+N^2)DR_w \qquad \text{(SI units)}$$

For square footing:

$$q_{ult} = 2BN^2R'_w + 6(100+N^2)DR_w \qquad \text{(Fps units)}$$

The above equation may be modified for q_s (FS=3) in SI units

$$q_s = 0.105BN^2R'_w + 0.314(100+N^2)DR_w \qquad \text{(SI units)}$$

Where,

q_s = net safe bearing capacity w.r.t. shear failure alone for FOS of 3 in psf or kPa

B = footing width in ft or meters

N = SPT resistance in blows/300 mm

D = footing depth in ft or meters; if $D > B$ use $D = B$ for computation

R_w & R'_w = water table reduction factor

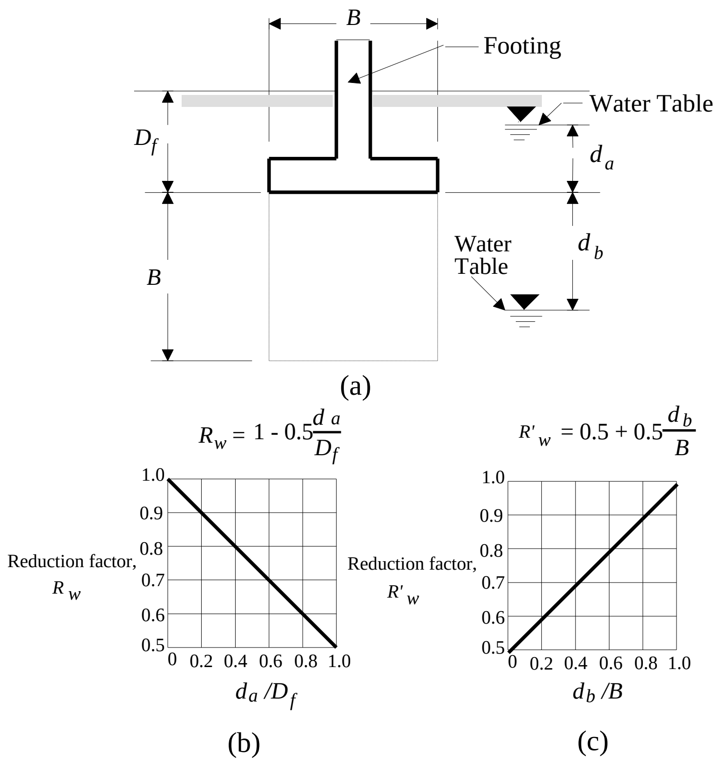


Fig. 1: Correction factors for position of water level: (a) depth of water level with respect to dimension of footing; (b) water level above base of footing; (c) water level below base of footing.

e. Parry (1977) Relation based on shear failure criterion

For cohesionless soils only

$q_{ult} = 30\ N$ (kPa) ($D \leq B$)

N = average SPT value at a depth about $0.75B$ below the proposed base of the footing.

2. Bearing Capacity using CPT

(i) Meyerhof (1956)

For a maximum settlement of 25 mm; for foundations (strip or square) on dry sands:

$q_a = 3.6\ q_c$ kN/m$^2 \cong q_c/30$ kg/cm^2 for $B \leq 1.2$ m

$q_a = 2.1\ q_c\ (1 + 1/B)^2$ kN/m$^2 \cong q_c/50\ (1 + 1/B)^2$ kg/cm^2 for $B > 1.2$ m

For any value of B, an approximate formula is:

$q_a = 2.7\ q_c$ kN/m$^2 = q_c/40$ kg/cm^2

Where,

q_a = allowable pressure for 25 mm

B = footing width in meters.

q_c = CPT cone resistance in kPa.

Notes:

- ☞ above equations are based on the approximate rule that $N = q_c/4$ (in kg/cm^2).
- ☞ q_a is halved if the sand within the stresses zone is submerged.
- ☞ For rafts and pier foundations, double the q_a values determined above.

(ii) Schmertmann (1978)

The bearing capacity factors for use in Terzaghi's bearing capacity equation can be estimated as:

$0.8\ N_q \cong 0.8\ N_\gamma \cong q_c$ $D/B \leq 1.5$.

Where q_c is average cone resistance over depth interval from $B/2$ above to $1.1B$ below footing base.

- ***For Cohesionless Soils***

Strip: $q_{\text{ult}} = 28 - 0.0052(300 - q_c)^{1.5}$ in kg/cm^2 or tons/ft^2

Square: $q_{\text{ult}} = 48 - 0.009(300 - q_c)^{1.5}$ in kg/cm^2 or tons/ft^2

- ***For Cohesive Soils***

Strip: $q_{ult} = 2 + 0.28 q_c$ in kg/cm^2 or tons/ft^2

Square $q_{ult} = 5 + 0.34 q_c$ in kg/cm^2 or tons/ft^2

3. Bearing Capacity Using Vane Shear Test (VST)

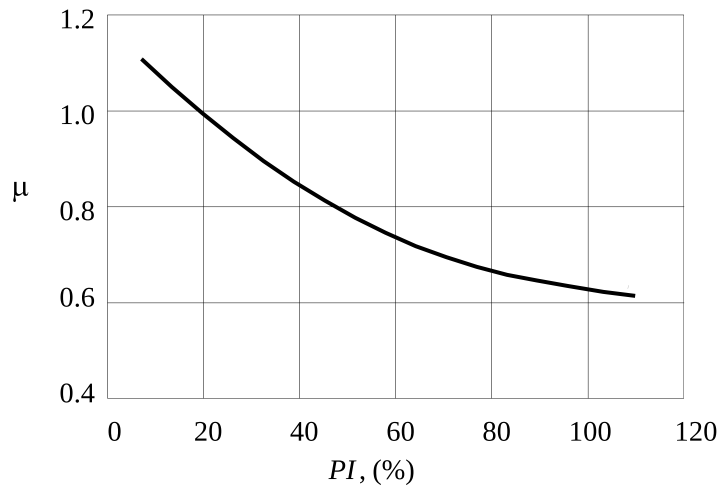
$$q_{ult} = 5\mu\tau_u(1+0.2\,D/B)(1+0.2\,B/L)+\sigma_v$$

Where,

μ = strength reduction factor

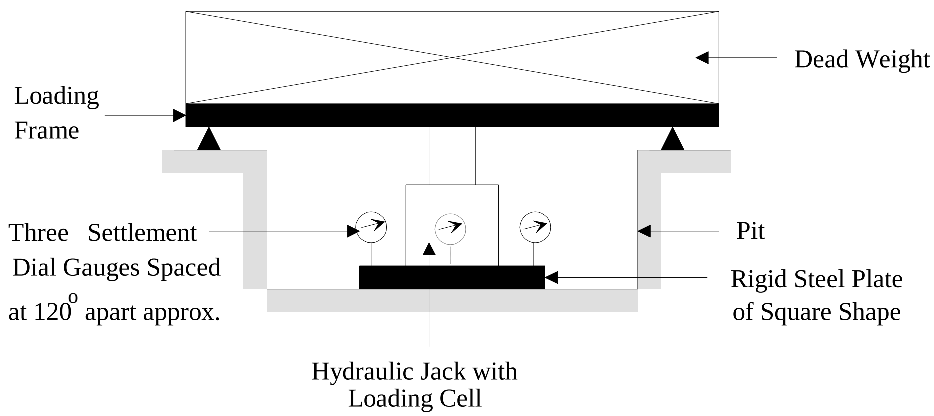
τ_u = undrained shear strength = $\dfrac{T}{3.6D^3}$

T = measured torque
D = blade diameter of vane
σ_v = total overburden pressure at foundation level.



Correction factor for the field vane test as a function of PI, (after Bjerrum, 1972*, and Ladd, et al.*, 1977*).*

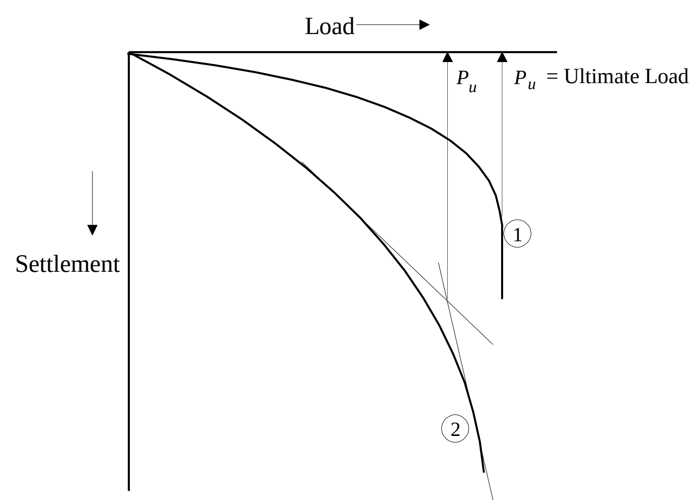
4. BEARING CAPACITY FROM PLATE LOAD TEST



Schematic sketch showing load-test arrangement

For details of equipment and testing procedure, refer to ASTM D 1195

- The load is applied in increments of 25% of the proposed design load.
- Increments are added till the final load is 150 to 200% of the proposed design load or to the failure of the soil underneath the plate.
- Each increment of load is maintained until the settlement is ceased, however the final applied load is maintained for not less than 24 hours.
- Settlement dial readings are recorded for each load increment after 5, 10, 15, 30, 60 minutes and every 1 hour interval thereafter to the first 6 hours and at least once every 12 hours thereafter.



Typical load-settlement plots of a load test

Data Reduction and Analysis

The ultimate load can be obtained:

- directly from the curve (1) or
- using two tangents method, curve (2).

then

$q_{ult,\ \text{foundation}} = q_{ult,\ \text{load test}}$ for clay

$$q_{\text{ult}} = q_{\text{plate}}\left(\frac{B_{\text{foundation}}}{B_{\text{plate}}}\right)$$ for sand

Settlement of prototype footing (Terzaghi and Peck, 1948):

$$s_f = s_p\left(\frac{B_f}{B_p}\right)$$ for clays, and

$$S_f = Sp\left(\frac{\mathrm{B_f}}{\mathrm{B_p}}\right)^2\left(\frac{\mathrm{B_p}+1}{\mathrm{B_f}+1}\right)^2$$ for sands

For a square plate of 1 ft × 1 ft size

$$s_f = s_p\left(\frac{2B_f}{B_f+1}\right)^2$$ for sands

Where s_f & s_p = settlements of prototype foundation and a square plate of 1 ft × 1 ft size respectively.

B_f (or B) & B_p = widths of the prototype foundation and plate respectively.

Above equations are for surface footings i.e. $D = 0$

To estimate the settlement of footings placed at depth D apply the depth correction factor using Fox's (1948) curves.

How to Obtain BC from Plate Load Test Results

The permissible settlement S_f for a prototype foundation should be known. Normally settlement of 2.5 cm (1 inch) is recommended. In above equations, the values of S_f and Bp are therefore known. The unknown are Sp and B_f. The value of Sp for any assumed size B_f may be found out from the above equations and then using the plate load settlement curve, the value of the bearing pressure corresponding to the computed value of Sp is found out.

The bearing pressure is the allowable bearing pressure for a given permissible settlement S_f.

Limitation of the Plate Load Test

1. Since plate load test is of short duration, it will not give consolidation settlement. If the underlying soil is sandy in nature immediate settlement may be taken as the total settlement. If the soil is clayey type, the immediate settlement is only a fraction of the total settlement. Load tests, therefore, do not have much significance in clayey soils to determine allowable pressure on the basis of settlement criterion.

2. Plate load tests should be used with caution and the present practice is not to rely too much on this test. If the soil is not homogeneous to a great depth, plate load tests give very misleading results.

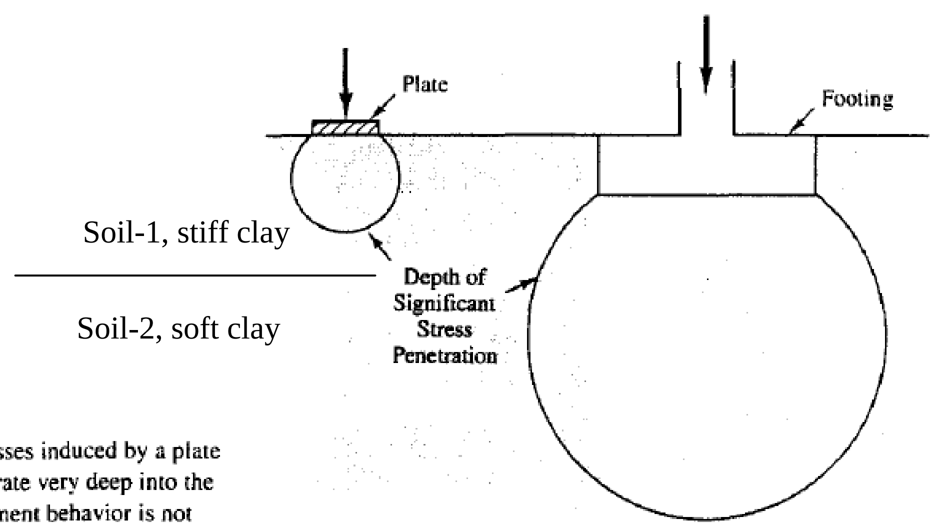


Figure 7.1 The stresses induced by a plate load test do not penetrate very deep into the soil, so its load-settlement behavior is not the same as that of a full-sized footing.

Assume two layers of soil. The top layer is stiff clay where as the bottom layer is soft clay. The load test conducted near the surface of the ground measures the characteristics of stiff clay but does not indicate the nature of the soft clay soil which is below. The actual foundation of a building, however, has a bulb of pressure which extends to a great depth into the poor soil which is highly compressible. Whereas the soil tested by the plate load test is very good leading to unsafe design. Plate load test is, therefore, not at all recommended on soils, which are not homogeneous at least to a depth equal to 1.5 to 2 times the width of the prototype foundation.

3. Plate load tests should not be relied on to determine the ultimate B.C of sandy soils as the scale (size) effect gives very misleading results. However, when the tests are carried on clay soils, the ultimate B.C as determined by the test may be taken as equal to that of the foundation since the bearing of clay is essentially independent of the footing size

5. BY LABORATORY UNCONFINED COMPRESSION TEST

The B.C of a cohesive soil can also be evaluated from the unconfined compressive test on cohesive soils. The failure axial stress in case of unconfined compression test is termed as unconfined compressive stress which is equal to:

$$q_u = 2C$$

and $C = q_u/2$ and $\phi = 0$ (for undrained condition)

By Terzaghi's equation, the BC of cohesive soils for $\phi = 0$ case is

$q_{un} = CNc$ $Nc = 5.7$ or approximately 6

$q_{un} = 6C$

for FS=3

$qns = 2C = q_u$

Therefore, the net safe bearing capacity (q_{ns}) of cohesive soil can be taken approximately equal to unconfined compression strength of cohesive soil.

6. BC BY BUILDING CODES:

In many countries/cities, the local building code stipulates values of allowable soil pressure to use when designing foundations. These values are usually based on years of experience, although in some cases they are simply used from the building construction handbooks.

These arbitrary values of soil pressure are termed as Presumptive Bearing Pressures. The presumptive pressures are generally based on a visual soil classification. Following table summaries the Presumptive Bearing Pressures from the International Building Code.

Table: Presumptive Bearing Pressures from the International Building Code (IBC, 1997)

Soil or Rock Classification	**Allowable Bearing Pressure, q_a**	
	(kPa)	**(lbs/ft^2)**
Crystalline Bedrock	600	12,000
Sedimentary or Foliated Rocks	300	6,000
Sandy gravel, or gravel (GW, GP)	250	5,000
Sand, silty Sand, clayey sand, silty gravel, clayey gravel, (SW, SP, SM, SC, GM and GC)	150	3,000
Clay, sandy clay, silty clay, or clayey silt, (CL, ML, MH, CH)	100	2,000
Mud, organic silt, organic clay, peat or unprepared fill	0	0