Bearing Capacity of Shallow Foundations
INTRODUCTION

Bearing capacity is the power of foundation soil to hold the forces from the superstructure without undergoing shear failure or excessive settlement. Foundation soil is that portion of ground which is subjected to additional stresses when foundation and superstructure are constructed on the ground. The following are a few important terminologies related to bearing capacity of soil.

**Ultimate Bearing Capacity** \((q_f)\) : It is the maximum pressure that a foundation soil can withstand without undergoing shear failure.

**Net ultimate Bearing Capacity** \((q_n)\) : It is the maximum extra pressure (in addition to initial overburden pressure) that a foundation soil can withstand without undergoing shear failure.
Here, $q_o$ represents the overburden pressure at foundation level and is equal to $\rho D$ for level ground without surcharge where $\rho$ is the unit weight of soil and $D$ is the depth to foundation bottom from Ground Level.

**Safe Bearing Capacity ($q_s$)**: It is the safe extra load the foundation soil is subjected to in addition to initial overburden pressure.

\[
q_s = \frac{q_n}{F} + q_o
\]

Here, $F$ represents the factor of safety.

**Allowable Bearing Pressure ($q_a$)**: It is the maximum pressure the foundation soil is subjected to considering both shear failure and settlement.

**Foundation** is that part of the structure which is in direct contact with soil. Foundation transfers the forces and moments from the super structure to the soil below such that the stresses in soil are within permissible limits and it provides stability against sliding and overturning to the super structure. It is a transition between the super structure and foundation soil. The job of a geotechnical engineer is to ensure that both foundation and soil below are safe against failure and do not experience excessive settlement (settlement to remain within permissible limits). Footing and foundation are synonymous.
MODES OF SHEAR FAILURE

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

**General shear failure :**

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

1. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.
2. Dense or stiff soil that undergoes low compressibility experiences this failure.
3. Continuous bulging of shear mass adjacent to footing is visible.
4. Failure is accompanied by tilting of footing.
5. Failure is sudden and catastrophic with pronounced peak in $P – \Delta$ curve.
6. The length of disturbance beyond the edge of footing is large.
7. State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.
8. General shear failure is accompanied by low strain (\(<5\%\)) in a soil with considerable $\Phi$ ($\Phi>36^\circ$) and large $N$ ($N > 30$) having high relative density ($I_D > 70\%$).
9. Failure plane in soil sample, At the peak load a clear failure plane will develop and the deformation will be due to relative movement along the failure plane. (Triaxial or unconfined compression test)
Figure: Schematic representation of General Shear Failure

Figure: Failure plane in soil samples for General Shear Failure
**Local shear failure:**
This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure.

1. A significant compression of soil below the footing and partial development of plastic equilibrium is observed.
2. Failure is not sudden and there is no tilting of footing.
3. Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.
4. Failure surface is not well defined.
5. Failure is characterized by considerable settlement.
6. Well defined peak is absent in $P – \Delta$ curve.
7. Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low $\Phi$ ($\Phi < 28^\circ$) and low $N$ ($N < 5$) having low relative density ($I_D > 20\%$).
8. Failure plane in soil sample: No clear failure plane is developed. The failure is represented by bulging (Triaxial or unconfined compression test).
**Punching shear failure:**
This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure.

1. This type of failure occurs in a soil of very high compressibility.
2. Failure pattern is not observed.
3. Bulging of soil around the footing is absent.
4. Failure is characterized by very large settlement.
5. Continuous settlement with no increase in $P$ is observed in $P - \Delta$ curve.
6. Figure below presents the conditions for different failure modes in sandy soil carrying circular footing based on the contributions from Vesic (1963 & 1973)
7. **Failure plane in soil sample:** The test sample may not be handled or may slump when taken out of the sampler.

![Figure: Schematic representation of Punching Shear Failure](image-url)
# Distinction between General Shear and Local or Punching Shear Failure

<table>
<thead>
<tr>
<th>General Shear Failure</th>
<th>Local/Punching Shear Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occurs in dense/stiff soil</td>
<td>Occurs in loose/soft soil</td>
</tr>
<tr>
<td>$\Phi &gt; 36^\circ$, $N &gt; 30$, $I_D &gt; 70%$, $C_u &gt; 100$ kPa</td>
<td>$\Phi &lt; 28^\circ$, $N &lt; 5$, $I_D &lt; 20%$, $C_u &lt; 50$ kPa</td>
</tr>
<tr>
<td>Results in small strain ($&lt; 5%$)</td>
<td>Results in large strain ($&gt; 20%$)</td>
</tr>
<tr>
<td>Failure pattern well defined &amp; clear</td>
<td>Failure pattern not well defined</td>
</tr>
<tr>
<td>Well defined peak in $P-\Delta$ curve</td>
<td>No peak in $P-\Delta$ curve</td>
</tr>
<tr>
<td>Bulging formed in the neighborhood of footing at the surface</td>
<td>No Bulging observed in the neighborhood of footing</td>
</tr>
<tr>
<td>Extent of horizontal spread of disturbance at the surface large</td>
<td>Extent of horizontal spread of disturbance at the surface very small</td>
</tr>
<tr>
<td>Observed in shallow foundations</td>
<td>Observed in deep foundations</td>
</tr>
<tr>
<td>Failure is sudden &amp; catastrophic</td>
<td>Failure is gradual</td>
</tr>
<tr>
<td>Less settlement, but tilting failure observed</td>
<td>Considerable settlement of footing observed</td>
</tr>
</tbody>
</table>
**Plane of shear failure**

Shear resistance offered by the stable soil below the plane of failure

**Bulging of Soil**

Shear movement of soil above the failure plane

**Footing at ground surface**

**Extra resistance**

Hence will carry a higher load

**Footing below ground surface**
Bulging of Soil

Shear movement of soil above the failure plane

Plane of shear failure

Shear resistance offered by the stable soil below the plane of failure

Clayey Silt

Gravel

Sand
Plane of shear failure

Shear resistance offered by the stable soil below the plane of failure

Bulging of Soil

Shear movement of soil above the failure plane

GWT
Sequence of development of failure plane
Mechanism for development of failure plane
Based on experimental results, Vesic (1973) has proposed a relationship for the mode of bearing capacity failure of foundations resting on sands. This is shown in the following Figure, which uses the following notations:

\[ \text{Dr} = \text{relative density of sand} \]
\[ \text{Df} = \text{depth of foundation measured from the ground surface} \]

\[ B^* = \frac{2BL}{B + L} \]

Where

\[ B = \text{width of foundation} \]
\[ L = \text{length of foundation} \]
For square foundations, \( B = L \); for circular foundations, \( B = L = \text{diameter} \), So \( B^* = B \)

*Relationship between mode of failure and relative density of sand*
For foundations located at a shallow depth (that is, small $D_r/B^*$) the ultimate load may be reached at a foundation settlement of 4—10% of $B$. This is true for general shear failure;

However, in the case of local or punching shear failure, the ultimate load may be reached at settlements of 15—25% of the width of foundation ($B$).
FACTOR AFFECTING BEARING CAPACITY OF SOIL

1. Soil Properties
   a. Type of soil
   b. Moisture content of soil
   c. Density of soil
   d. History of deposit
      i. Recent fill
      ii. Natural deposit
   e. Layered soil
      i. Strong soil layer overlying soft layer
      ii. Soft soil layer overlying strong layer

2. Site Conditions
   a. Sloping ground
   b. Lateral confinement
   c. Unstable ground condition
   d. Mining activities
   e. Frost penetration
i. Natural
ii. Artificial (under refrigeration houses)

3. Footing Characteristics
   a. Size of footing
   b. Shape of footing
   c. Depth of footing
   d. Load distribution
      i. Concentric load + moment
      ii. Eccentric load
      iii. Inclined load

4. Location of GWT and possibility of future fluctuation
   a. Artesian condition
   b. Drainage layer
   c. Impervious layer
   d. Seepage flow
5. Local conditions (time of investigation)
   a. Weather conditions
   b. Seismic activity
   c. Flooding activity

6. Site History
   a. Site developed after removal of wood. As long as the roots are alive, these will act as reinforcement and will result in an increase in bearing capacity. But when the roots decompose, cavities will be produced causing decrease in soil density and allow deep penetration of water causing decrease in shear strength.
   b. Agricultural lands
   c. Reclaimed site
   d. Site developed after demolition of building
   e. Old fill with domestic waste
In general the bearing capacity depends on the following factors:

- Type of soil
- Unit weight of soil
- Surcharge load
- Depth of foundation
- Mode of failure
- Size of footing
- Shape of footing
- Depth of water table
- Eccentricity in footing load
- Inclination of footing load
- Inclination of ground
- Inclination of base of foundation
METHODS OF OBTAINING BEARING CAPACITY

a. Analytical methods
b. Empirical methods
c. In-Situ loading tests
d. Building Codes or Civil Engineering handbooks.

**Analytical Method:**

The soil bearing capacity can be calculated by means of

1. The theory of elasticity
2. The earth pressure theory
3. The theory of plasticity and

1. **Bearing Capacity based on theory of Elasticity**

   Schleicher derived the following expression for bearing capacity for a uniformly distributed surface load.

   \[ q_0 = \frac{sc}{\omega \sqrt{A}} \]
Where,

\( q_o \) = Allowable bearing capacity of soil

\( S \) = Permissible settlement

\( \omega \) = Shape Coefficient

\( C \) = Constant, which depends on soil properties

\[
C = \frac{E}{1 - \mu^2}
\]

Where,

\( E \) = Modules of elasticity

\( \mu \) = Poisson’s ratio

2. **Bearing Capacity based on Earth Pressure Theory**

According to earth pressure theory, whenever applied stress on a soil mass exceeds a certain value, rupture surfaces are formed. The value of stress at the formation of rupture surfaces may be considered as the ultimate bearing capacity. The famous theories are that of Pauker’s, Rankine’s and others.
The soil bearing capacity therefore is determined on the basis of

a. Relationship between the principles stresses which occur upon the formation of rupture surfaces.
b. The shape of rupture surfaces.
c. The mode of expulsion of ruptured soil mass from underneath the footing

a. **Pauker’s Formula:**

Pauker’s formula is good for sandy soil. The equation for ultimate bearing capacity presented by Pauker is as follows:

\[ q_u = \gamma z \tan^4 \left(45^0 + \frac{\phi}{2} \right) \]

b. **Rankine’s Formula:**

The Rankine’s equation for ultimate bearing capacity is as under:

\[ q_u = \gamma z \left( \frac{1 + \sin \phi}{1 - \sin \phi} \right)^2 \]
The above equations have many deficiencies and therefore are not used in practice.

3. **Bearing Capacity based on theory of Plasticity:**

   Based on the theory of plastic equilibrium, Prandtl and Terzaghi presented equations for the ultimate bearing capacity of soil.

   a. **Prandtl’s Equation:**
Assumptions:

1. For soil loaded by a strip footing, soil wedges AFD, ABC and BEG behave as rigid wedges i.e., they slide without deformation.
2. Sectors ACD, BCE deform plasticity.
3. The soil is homogeneous, isotropic and weightless.
4. In the plastic sectors BCE and ACD, the stresses along any radius vector, such as AX are constant but they vary from radius vector to radius vector, i.e., with angle $\omega$.

Considering the equilibrium of the plastic sector, Pandtl gave the ultimate bearing capacity of soil whose cohesion is $C$ and angle of internal friction is $\phi$ by the following Eq.

$$q_u = \frac{C}{\tan \phi} \left[ \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) e^{\pi \tan \phi} - 1 \right]$$
Discussion of Prandtl’s Theory

1. The ultimate bearing capacity of soil depends very much upon the assumed shape of the rupture surfaces.
2. The rupture surface curve is compound, and consists of arcs of a logarithmic spiral and a tangent to the spiral.
3. It is applicable to a long strip foundation.
4. It is applicable to \((c - \phi)\) soil, and to \((c)\) soil loaded on its surface by a long strip foundation with a small base.
5. When \(c = 0\), and \(\phi > 0\), then, \(q_u = 0\), which is not true.
6. When \(\phi = 0\), then the equation transforms into an indeterminate quantity, namely \(q_u\phi = \infty\)
7. It is independent of the width, \(b\), of the strip foundation.
8. Assumed rupture surface is only an approximation
9. No consideration of the reactive stress distribution in soil along the rupture surface at the time of rupture.

b. **Terzaghi's Ultimate Bearing Capacity Equation:**

The method is widely used to find the bearing capacity. 
The method is for shallow strip footing. 
Various factors; like width & depth of foundation, density of soil, cohesion and angle of internal friction are considered
TERZAGHI’S BEARING CAPACITY THEORY

Terzaghi (1943) was the first to propose a comprehensive theory for evaluating the safe bearing capacity of shallow foundation with rough base.

Assumptions:
1. Soil is homogeneous and Isotropic.
2. The shear strength of soil is represented by Mohr Coulombs Criteria.
3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.
4. Elastic zone has straight boundaries inclined at an angle equal to Φ to the horizontal.
5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected.
6. Method of superposition is valid.
7. Passive pressure force has three components ($P_{PC}$ produced by cohesion, $P_{Pq}$ produced by surcharge and $P_{Py}$ produced by weight of shear zone).
8. Effect of water table is neglected.
10. Footing and ground are horizontal.
11. Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.
12. The properties of foundation soil do not change during the shear failure.
Limitations:
1. The theory is applicable to shallow foundations.
2. As the soil compresses, $\Phi$ increases which is not considered. Hence fully plastic zone may not develop at the assumed $\Phi$.
3. All points need not experience limit equilibrium condition at different loads.
4. Method of superstition is not acceptable in plastic conditions as the ground is near failure zone.

Figure 1: Terzaghi’s concept of Footing with five distinct failure zones in foundation soil
Failure mechanism for determining the ultimate bearing capacity (general shear failure) for a rough strip footing located at a depth D is shown in Figure-2.

Zone I- The soil wedge ABJ is an elastic zone. Both AJ and BJ make an angle $\phi$ with the horizontal.

Zones II- The zones AJE and BJD are the radial shear zones,

Zones III- The zones AGE and BFD are the passive zones.

The rupture curves JD and JE are arcs of a logarithmic spiral, and DF and EG are straight lines.

AE, BD, EG, and DF make angles of $45 - \Phi/2$ with the horizontal.

- Pressure $q_u$, is applied to a footing to cause general shear failure
- Passive pressure $Pp$ is acting on each faces of the soil wedge ABJ.
- Imagine AJ and BJ as two walls pushing the soil wedges AJEG & BJDF, to cause passive failure.

- $Pp$ is inclined at an angle $\delta$ (angle of wall friction) to the perpendicular to the wedge faces (AJ and BJ).
- In this case, $\delta = \Phi$, since AJ is a soil surface not wall.
Figure 2: Shallow strip footing

Figure 3: Shallow strip footing
• Since AJ and BJ are inclined at an angle $\Phi$ to the horizontal, direction of $P_p$ should be vertical.
• Consider the free body diagram of the wedge ABJ as shown in Figure C.
• Considering unit length of the strip footing, for equilibrium

$$(q_u)(2b)(1) = - W + 2C \sin \Phi + 2p_p$$  \hspace{1cm} (1)
Where, $b = B/2$
$W = \text{weight of soil wedge } ABJ = \gamma b^2 \tan \Phi$
$C = \text{cohesive force acting along the faces } AJ \text{ and } BJ$, and is equal to the unit cohesion times the length of each face $= c (b/\cos \Phi)$

Thus

$$2bq_u = 2P_p + 2bctan\Phi - \gamma b^2 \tan \Phi$$

(2)

The passive pressure in the above Eq. is the sum of the contribution of,

1- Weight of soil, $\gamma$
2- Cohesion, $c$
3- Surcharge, $q$

Figure D shows the distribution of passive pressure due each of the above components on the wedge face BJ. Thus,

$$P_p = 1/2\gamma (b \tan \Phi)^2 K_\gamma + c(b \tan \Phi).K_c + q(b \tan \Phi)Kq$$

(3)

Where, $K_\gamma$, $K_c$ and $Kq$ are earth pressure coefficients that are functions of friction angle, $\Phi$
Substituting Eq. (3) in Eq. (2) we get

\[ 2bq_u = 2b \cdot c \tan \Phi (K_c + 1) + 2bq[(\tan \Phi)K_q] + b^2 \gamma [\tan \Phi (K_{\gamma} \tan \Phi - 1)] \]  

(4)

Figure 4: Terzaghi's bearing capacity analysis
Figure D: Passive pressure distribution on the wedge face BJ shown in figure C.
Rearranging the terms of Eq. 4 we get

\[ q_u = c\left[\tan \phi (K_c + 1)\right] + q\left[\tan \phi K_q\right] \]

\[ + \gamma \cdot \frac{B}{2} \left[ \frac{1}{2} \tan \phi (K_\gamma \cdot \tan \phi - 1) \right] \]

The terms I, II, and III in Eq. (5) are respectively, the contributions of cohesion, surcharge, and the unit weight of soil to the ultimate bearing capacity.

It is extremely tedious to evaluate the values of \( K_c, K_q \) and \( K_\gamma \). For that reason, Terzaghi used an approximate method to determine the ultimate bearing capacity; \( q_u \).

The principle of this approximation is given below:

1. If \( C = 0 \) and surcharge \( q = 0 \) (that is, \( D = 0 \)), then from Eq. (5)
\[ q_u = q_\gamma = \frac{1}{2} \gamma B \left[ \frac{1}{2} \tan \phi (K_\gamma \cdot \tan \phi - 1) \right] \]

\[ = \frac{1}{2} B \gamma N_\gamma \]  

(6)

2. If \( \gamma = 0 \) (that is, weightless soil) and \( q = 0 \), then from Eq. (5)

\[ q_u = q_c = c \left[ \tan \phi (K_c + 1) \right] \]

\[ = c N_c \]  

(7)

3. If \( \gamma = 0 \) (weightless soil) and \( c = 0 \)

\[ q_u = q_q = q \left[ \tan \phi . K_q \right] \]

\[ = q N_q \]  

(8)
By method of superimposition, when the effects of unit weight of soil, cohesion, and surcharge are taken into consideration,

\[ q_u = q_c + q_q + q_\gamma = cN_c + qN_{q} + \frac{1}{2} \gamma B N_\gamma \]  \hspace{1cm} (9)

- Equation (9) is referred to as Terzaghi's bearing capacity equation.
- The terms \( N_c \), \( N_q \) and \( N_\gamma \) are called the bearing capacity factors.
- The values of these bearing capacity factors are given in Table A.
<table>
<thead>
<tr>
<th>$\phi$</th>
<th>$N_c$</th>
<th>$N_q$</th>
<th>$N_g$</th>
<th>$N'_c$</th>
<th>$N'_q$</th>
<th>$N'_g$</th>
</tr>
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<tbody>
<tr>
<td>0</td>
<td>5.7</td>
<td>1.0</td>
<td>0.0</td>
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<td>5</td>
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<tr>
<td>50</td>
<td>347.6</td>
<td>415.1</td>
<td>1153.2</td>
<td>81.3</td>
<td>65.6</td>
<td>87.1</td>
</tr>
</tbody>
</table>

Table 1: Terzaghi’s Bearing Capacity Factors for different values of $\phi$
Ultimate bearing capacity,

\[ q_u = cN_c + \gamma DN_q + 0.5\gamma BN \]

If the ground is subjected to additional surcharge load \( q \), then

\[ q_u = cN_c + (\gamma D + q)N_q + 0.5\gamma BN \]

Net ultimate bearing capacity,

\[ q_n = cN_c + \gamma DN_q + 0.5\gamma BN - \gamma D \]

\[ q_n = cN_c + \gamma (\mathcal{N}_q - 1) + 0.5\gamma BN \]

Safe bearing capacity,

\[ q_s = \left[ cN_c + \gamma (\mathcal{N}_q - 1) + 0.5\gamma BN \right] \frac{1}{F} + \gamma D \]
Here,  
F = Factor of safety (usually 3)  
c = cohesion  
γ = unit weight of soil  
D = Depth of foundation  
q = Surcharge at the ground level  
B = Width of foundation  
N_c, N_q, N_γ = Bearing Capacity factors

Figure 5: Terzaghi’s Bearing Capacity Factors for different values of φ
**Effects of Shape of Foundation:**
The shape of footing influences the bearing capacity. Terzaghi and other contributors have suggested the correction to the bearing capacity equation for shapes other than strip footing based on their experimental findings. The following are the corrections for circular, square and rectangular footings.

**Circular footing**

\[ q_u = 1.3cN_c + \gamma DN_q + 0.3\gamma BN \gamma \]

**Square footing**

\[ q_u = 1.3cN_c + \gamma DN_q + 0.4\gamma BN \gamma \]

**Rectangular footing**

\[ q_u = \left( 1 + 0.3 \frac{B}{L} \right) cN_c + \gamma DN_q + \left( 1 - 0.2 \frac{B}{L} \right) 0.5\gamma BN \gamma \]
Summary of Shape Factor:
Table below gives the summary of shape factors suggested for strip, square, circular and rectangular footings. B and L represent the width and length respectively of rectangular footing such that B < L.

<table>
<thead>
<tr>
<th>Shape</th>
<th>$s_c$</th>
<th>$s_q$</th>
<th>$s_r$</th>
</tr>
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<tbody>
<tr>
<td>Strip</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Square</td>
<td>1.3</td>
<td>1</td>
<td>0.4</td>
</tr>
<tr>
<td>Round</td>
<td>1.3</td>
<td>1</td>
<td>0.6</td>
</tr>
<tr>
<td>Rectangle</td>
<td>$(1 - 0.2\frac{B}{L})$</td>
<td>1</td>
<td>$(1 + 0.3\frac{B}{L})$</td>
</tr>
</tbody>
</table>

Equation (9) was derived on the assumption that the bearing capacity failure of soil takes place by general shear failure.
Local Shear Failure:
The equation (9) for bearing capacity explained above is applicable for soil experiencing general shear failure. If a soil is relatively loose and soft, it fails in local shear failure. Such a failure is accounted in bearing capacity equation by reducing the magnitudes of strength parameters $c$ and $\phi$ as follows.

\[
c' = \frac{2}{3}c
\]

and

\[
tan\phi' = \frac{2}{3}tan\phi
\]

Ultimate bearing capacity of soil for a strip footing may be given by

\[
q'_u = c'N'_c + qN'_q + \frac{1}{2}\gamma BN'_\gamma
\]

Modified bearing capacity factors $N'_c$, $N'_q$ and $N'_\gamma$ are calculated by using the same general equation as for $N_c$, $N_q$ and $N_\gamma$ but by substituting $\Phi' = \tan^{-1} \left( \frac{2}{3} \tan \Phi \right)$ for $\Phi$. 
Table B summarizes the bearing capacity factors to be used under different situations. If $\phi$ is less than $36^\circ$ and more than $28^\circ$, it is not sure whether the failure is of general or local shear type. In such situations, linear interpolation can be made and the region is called mixed zone.

<table>
<thead>
<tr>
<th>Local Shear Failure</th>
<th>Mixed Zone</th>
<th>General Shear Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi &lt; 28^\circ$</td>
<td>$28^\circ &lt; \phi &lt; 36^\circ$</td>
<td>$\Phi &gt; 36^\circ$</td>
</tr>
<tr>
<td>$N_c^1, N_q^1, N_y^1$</td>
<td>$N_c^m, N_q^m, N_y^m$</td>
<td>$N_c, N_q, N_y$</td>
</tr>
</tbody>
</table>

Table 2: Bearing capacity factors in zones of local, mixed and general shear conditions.
The Equations of ultimate bearing capacity for square and circular footings for local shear failure are given as follows.

Square footing

\[ q'_u = 1.3c'N'_c + qN'_q + 0.4\gamma BN'_{\gamma} \]  \hspace{1cm} (15)

Circular footing

\[ q'_u = 1.3c'N'_c + qN'_q + 0.3\gamma BN'_{\gamma} \]  \hspace{1cm} (16)

Values of the bearing capacity factors for local shear failure are given in Figure 5.
Figure 6: Terzaghi’s bearing capacity factors for local shear failure
Effect of Ground Water Table

- For the above equations, it was assumed that GWT is located at a depth much greater than width, B, of the footing.
- However, if the ground water table is close to the footing (which ultimately reduces the strength of foundation soil), some changes in the second and third terms of Equations are required (in the presence of water table is submerged density and not dry density).
- Three different conditions can arise regarding the location of GWT as shown in Figures for each case.

Case I (Adjoining Figure): If GWT is located at a distance D above the bottom of the foundation, the magnitude of q in the second term of the bearing capacity equation should be calculated as

\[ q = \gamma (D_f - D) + \gamma' D \]  \hspace{1cm} (17)

Also, the unit weight \( \gamma \) of soil appearing in the third term of the equations should be replaced by \( \gamma' \)

Where,

\[ \gamma' = \gamma_{sat} - \gamma_w = \text{effective unit weight of soil} \]
**Case II** (Figure below): If the ground water table coincides with the bottom of the foundation;

then \( q \) is equal to \( \gamma D_f \)

However, the unit weight \( \gamma \) in the third term of the equations should be replaced by \( \gamma' \)

**Case III** (Figure below): When the ground water table is at a depth \( D \) below the bottom of the foundation;

\[ q = \gamma D_f. \]

The magnitude of \( \gamma \) in the third term of the equations should be replaced by \( \gamma_{av} \)
\[ \gamma_{av} = \frac{1}{B} \left[ \gamma D + \gamma (B - D) \right] \quad \text{(for } D \leq B) \quad (18a) \]

\[ \gamma_{av} = \gamma \quad \text{(for } D > B) \quad (18b) \]
Figure 7: Effect of water table when WTL is above foundation level.

Ultimate bearing capacity with the effect of water table is given by,

\[ q_u = cN_c + \gamma DN_q R_{w1} + 0.5\gamma BN \gamma R_{w2} \]

where \( Z_{w1} \) is the depth of water table from ground level.

1. \( 0.5 < R_{w1} < 1 \)
2. When water table is at the ground level \((Z_{w1} = 0)\), \( R_{w1} = 0.5 \)
3. When water table is at the base of foundation \((Z_{w1} = D)\), \( R_{w1} = 1 \)
4. At any other intermediate level, \( R_{w1} \) lies between 0.5 and 1
Figure 8: Effect of water table when WTL is above foundation level

Here

\[ R_{w2} = \frac{1}{2} \left[ 1 + \frac{Z_{w2}}{B} \right] \]

where \( Z_{w2} \) is the depth of water table from foundation level.

1. \( 0.5 < R_{w2} < 1 \)
2. When water table is at the base of foundation \( (Z_{w2} = 0), R_{w2} = 0.5 \)
3. When water table is at a depth \( B \) and beyond from the base of foundation \( (Z_{w2} \geq B), R_{w2} = 1 \)
4. At any other intermediate level, \( R_{w2} \) lies between 0.5 and 1
GENERAL BEARING CAPACITY EQUATION

The soil-bearing capacity equation for a strip footing can be improved by incorporating the following factors:

Depth factor: to account for shearing resistance developed along failure surface in soil above the base of footing

Shape factor: to determine the bearing capacity of rectangular/circular foundations

Inclination factor: to determine the bearing capacity for inclined loads.

\[
q_u = c N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}
\]

Where,

\(c\) = Cohesion

\(q\) = Effective stress at the level of the bottom of foundation

\(\gamma\) = Unit weight of soil

\(B\) = Width of foundation (= diameter for a circular foundation)

\(F_{cs}, F_{qs}, F_{\gamma s}\) = Shape factors

\(F_{cd}, F_{qd}, F_{\gamma d}\) = Depth factors

\(F_{ci}, F_{qi}, F_{\gamma i}\) = Load inclination factors

\(N_c, N_q, N_\gamma\) = Bearing capacity factors
DIFFERENT BEARING CAPACITY EQUATIONS

Since the development of Terzaghi's bearing capacity equation, several investigators refined the solution.

Different solutions show that the bearing capacity factors $N_c$ and $N_q$ do not change very much. However, the values of $N_y$ obtained by different investigators vary over a wide range.

This is due to the variation of assumed shape of the soil wedge located directly below the footing.

Terzaghi assumed angle of the failure wedge $= \phi$

While the others assumed the angle as $45 + \phi/2$,

This results in changes in the values of the bearing capacity factors

The equations/factors are proposed by several authors, e.g., Meyerhof, Hansen and Vesic etc.
The values of the factors by each author are different.
For using equation by any of the authors corresponding values of the factors should be used.
The values of shape factors, depth factors and inclination factors have been given by Different investigators, e.g., De Beer (1970), Hansen (1970) and Meyerhof (1953). These are empirical factors based on experimental observations. These shape, depth, and inclination factors are given in Table.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Relationship</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape$^a$</td>
<td>$F_{cs} = 1 + \frac{B \cdot N_q}{L \cdot N_c}$</td>
<td>De Beer (1970)</td>
</tr>
<tr>
<td></td>
<td>$F_{qs} = 1 + \frac{B \cdot \tan \phi}{L}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{\gamma s} = 1 - 0.4 \frac{B}{L}$</td>
<td></td>
</tr>
</tbody>
</table>

Where $L = \text{length of the foundation (} L > B \)$
<table>
<thead>
<tr>
<th>Factor</th>
<th>Relationship</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth$^b$</td>
<td>Condition (a): $D_f/B \leq 1$</td>
<td>Hansen (1970)</td>
</tr>
<tr>
<td></td>
<td>$F_{cd} = 1 + 0.4 \frac{D_f}{B}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{qd} = 1 + 2 \tan \phi (1 - \sin \phi)^2 \frac{D_f}{B}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{yd} = 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Condition (b): $D_f/B &gt; 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{cd} = 1 + (0.4) \tan^{-1} \frac{D_f}{B}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{qd} = 1 + 2 \tan^{-1} \phi (1 - \sin \phi)^2 \tan^{-1} \frac{D_f}{B}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{yd} = 1$</td>
<td></td>
</tr>
</tbody>
</table>
Where $\beta = \text{inclination of the load on the Foundation with respect to the vertical}$

These shape factors are empirical relations based on extensive laboratory tests.

The factor $\tan^{-1}(D_i/B)$ is in radians.
<table>
<thead>
<tr>
<th>Factor</th>
<th>Relationship</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape(^a)</td>
<td>For $\phi = 0$: $F_{cs} = 1 + 0.2 \frac{B}{L}$</td>
<td>Meyerhof (1953)</td>
</tr>
<tr>
<td></td>
<td>$F_{qs} = 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{\gamma s} = 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>For $\phi \geq = 10^0$: $F_{cs} = 1 + 0.2 \frac{B}{L} \tan^2 \left(45 + \frac{\phi}{2}\right)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{qs} = F_{\gamma s}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$= 1 + 0.1 \frac{B}{L} \tan^2 \left(45 + \frac{\phi}{2}\right)$</td>
<td></td>
</tr>
<tr>
<td>Factor</td>
<td>Relationship</td>
<td>Source</td>
</tr>
<tr>
<td>---------</td>
<td>-------------------------------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>Depth</td>
<td>For $\phi = 0$:</td>
<td>Meyerhof (1963)</td>
</tr>
<tr>
<td></td>
<td>$F_{cd} = 1 + 0.2 \frac{D_f}{B}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{qd} = F_{yd} = 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>For $\phi \geq 10^0$:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$F_{cd} = 1 + 0.2 \frac{D_f}{B}$</td>
<td>tan $45 + \frac{\phi}{2}$</td>
</tr>
<tr>
<td></td>
<td>$F_{qd} = F_{yd}$</td>
<td>$= 1 + 0.1 \frac{D_f}{B}$ tan $45 + \frac{\phi}{2}$</td>
</tr>
</tbody>
</table>
\[ F_{ci} = F_{qi} = \frac{1 - F_{qi}}{N_q - 1} \]

\[ F_{qi} = \left( 1 - \frac{(0.7 \ Q_u \ sin \beta)}{Q_u \ cos \beta + BL_c \ cot \ \phi} \right)^5 \]

\[ F_{\gamma i} = \left( 1 - \frac{(0.7 \ Q_u \ sin \beta)}{Q_u \ cos \beta + BL_c \ cot \ \phi} \right)^5 \]

\[ a \ L = \text{length} \ (\geq B) \]
## Bearing Capacity Factor For The Meyerhof, Hansen And Vesic Bearing Capacity Equations.

<table>
<thead>
<tr>
<th>Ø</th>
<th>Nc</th>
<th>Nq</th>
<th>Nγ(H)</th>
<th>Nγ(M)</th>
<th>Nγ(V)</th>
<th>Nq/Nc</th>
<th>2tan Ø(1-sinØ)^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5.14</td>
<td>1.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.195</td>
<td>0.000</td>
</tr>
<tr>
<td>5</td>
<td>6.49</td>
<td>1.6</td>
<td>0.1</td>
<td>0.1</td>
<td>.4</td>
<td>.242</td>
<td>0.146</td>
</tr>
<tr>
<td>10</td>
<td>8.34</td>
<td>2.5</td>
<td>0.4</td>
<td>0.4</td>
<td>1.2</td>
<td>0.296</td>
<td>0.241</td>
</tr>
<tr>
<td>15</td>
<td>10.97</td>
<td>3.9</td>
<td>1.2</td>
<td>1.1</td>
<td>2.6</td>
<td>0.359</td>
<td>0.294</td>
</tr>
<tr>
<td>20</td>
<td>14.83</td>
<td>6.4</td>
<td>2.9</td>
<td>2.9</td>
<td>5.4</td>
<td>0.431</td>
<td>0.315</td>
</tr>
<tr>
<td>25</td>
<td>20.71</td>
<td>10.7</td>
<td>6.8</td>
<td>6.8</td>
<td>10.9</td>
<td>0.514</td>
<td>0.311</td>
</tr>
<tr>
<td>30</td>
<td>30.13</td>
<td>18.4</td>
<td>15.1</td>
<td>15.7</td>
<td>22.4</td>
<td>0.610</td>
<td>0.289</td>
</tr>
<tr>
<td>34</td>
<td>42.14</td>
<td>29.4</td>
<td>28.7</td>
<td>31.1</td>
<td>41</td>
<td>0.698</td>
<td>0.262</td>
</tr>
<tr>
<td>40</td>
<td>75.25</td>
<td>64.1</td>
<td>79.4</td>
<td>93.6</td>
<td>109.3</td>
<td>0.852</td>
<td>0.214</td>
</tr>
<tr>
<td>45</td>
<td>133.73</td>
<td>134.7</td>
<td>200.5</td>
<td>262.3</td>
<td>271.3</td>
<td>1.007</td>
<td>0.172</td>
</tr>
<tr>
<td>50</td>
<td>266.50</td>
<td>318.5</td>
<td>567.4</td>
<td>871.7</td>
<td>761.3</td>
<td>1.195</td>
<td>0.131</td>
</tr>
</tbody>
</table>
FACTOR OF SAFETY

Factor of safety is to cater for uncertainties. The value depends on the degree of uncertainty. Following factors are worth consideration.

1. **Factors related to soil**:  
   - Complexity of soil behavior  
   - Lack of control over environmental changes after construction  
   - Incomplete knowledge of subsurface conditions  
   - Inability to develop a good mathematical model for the foundation  
   - Inability to determine soil parameters accurately or Reliability of soil data

2. **Factors related to site, structure & design etc**:  
   - Magnitude of damages (loss of life, property & lawsuits) if a failure results  
   - Relative cost of increasing or decreasing SF  
   - Changes in soil properties from construction operations, and later from any other causes  
   - Accuracy of currently used design/analysis methods
The following table can be used as a guide

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Foundation type</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear</td>
<td><strong>Earthworks</strong></td>
<td>1.2-1.6</td>
</tr>
<tr>
<td></td>
<td>Dams, fills, etc.</td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td><strong>Retaining structure</strong></td>
<td>1.5-2.0</td>
</tr>
<tr>
<td></td>
<td>Walls</td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>Sheetpiling cofferdams</td>
<td>1.2-1.6</td>
</tr>
<tr>
<td></td>
<td>Temporary braced</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Excavations</td>
<td>1.2-1.5</td>
</tr>
<tr>
<td>Shear</td>
<td><strong>Footings</strong></td>
<td>2-3</td>
</tr>
<tr>
<td></td>
<td>Spread</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mat</td>
<td>1.7-2.5</td>
</tr>
<tr>
<td></td>
<td>Uplift</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Seepage</strong></td>
<td>1.5-2.5</td>
</tr>
<tr>
<td></td>
<td>Uplift, heaving</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Piping</td>
<td>3-5</td>
</tr>
</tbody>
</table>

Table 3: Values of safety factors usually used
Factor of safety for bearing capacity failure usually = Fs = 3
It is not too conservative.
Soils are neither homogeneous nor isotropic.
Much uncertainty is involved in evaluating shear-strength.

<table>
<thead>
<tr>
<th>Category</th>
<th>Typical Structures</th>
<th>Characteristics of the Category</th>
<th>Design Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Railway bridges, warehouses, blast furnaces, hydraulic, retaining walls, silos</td>
<td>Maximum design load likely to occur often; consequences of failure disastrous</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4.0</td>
</tr>
<tr>
<td>B</td>
<td>Highway bridges, light industrial and public buildings</td>
<td>Maximum design loads may occur occasionally, consequences of failure serious</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.5</td>
</tr>
<tr>
<td>C</td>
<td>Apartment and office buildings</td>
<td>Maximum design load unlikely to occur</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
</tr>
</tbody>
</table>

Figure 9: Typical factors of safety for bearing capacity calculation in different situations
DETERMINATION OF BEARING CAPACITY FROM FIELD TESTS

Field Tests are performed in the field. You have understood the advantages of field tests over laboratory tests for obtaining the desired property of soil. The biggest advantages are that there is no need to extract soil sample and the conditions during testing are identical to the actual situation.

**Major advantages of field tests are**
1. Sampling is not required
2. Soil disturbance is minimum

**Major disadvantages of field tests are**
1. Laborious
2. Time consuming
3. Heavy equipment to be carried to field
4. Short duration behavior

The most commonly tests used for estimating the bearing capacity of soil, are;

I. Plate load test
II. Standard Penetration test
III. Cone penetration test
It is also known as Field load test or Static load test. The test method has been withdrawn by the ASTM committee since December 2000.

It is the most reliable method of obtaining the ultimate bearing capacity if the load test is on a full-size footing; however, this is not possible. The usual practice is to load test plates of diameters 30-cm to 75-cm. These sizes are usually too small to extrapolate to full-size footings. Several factors, such as:
1. The test gives information about soil to a depth of 2D only,
2. Does not fully take into account the time effect which causes the extrapolation to be questionable.

For such reasons ASTM has deleted the test since 2000.
1. **APPARATUS:**

1.1 Loading platform of sufficient size and strength to supply the estimated load.

1.2 Hydraulic or mechanical jack of sufficient capacity to provide the maximum estimated load for the soil conditions involved, but not less than 50-tons.

1.3 Bearing plates: Three circular steel bearing plates, not less than 1-in (25-mm) in thickness and varying in diameter from 12-in (30-cm) to 30-in (75-cm).

1.4 Settlement recording devices such as dial gauges to measure settlement to an accuracy of at least 0.01-in (0.25-mm).

1.5 Equipment required for making test pit & loading arrangement.

2. **PROCEDURE:**

2.1 Selection of test areas: Perform the load test at the proposed footing level and under the same conditions to which the proposed footing will be subjected.
Figure 10: Schematics of plate Load Test
Figure 11: Arrangement for plate load test on the ground surface (basement Bed)
Figure 12: Arrangement and equipment details for the plate load test
2.1 **Test pits:**
   i. Excavate the test pit to the proposed footing level
   ii. The test pit should be at least four times as wide as the plate.
   iii. Prior to loading, protect the test pit and the loading area against moisture changes in soil.
   iv. If wetting is expected during life time of structure, such as in case of hydraulic structures, prewet the soil in the area to the desired extent to a depth not less than twice the diameter of bearing plate.
   v. Carefully level and clean the area to be loaded so that the loads are transmitted over the entire contact area on undisturbed soil.
   vi. For uniform contact of plate use sand or plaster of Paris under the plate.

2.2 **Loading platforms:** Support the loading platform preferably not less than 8-ft (2.4-m) away from test area (i.e., center of plate)
2.3 **Dead load:** Weigh and record as dead weight all the equipment used, such as steel plates, loading column and jack etc,
2.4 **Reference beam:** Independently support the beams supporting the dial gauges not less than 8-ft (2.4-m) from the center of the loaded area
2.5 **Load increments:** Apply the load to the soil in cumulative equal increments of not more than 1.0 ton/ft², or not more than one tenth of the estimated bearing capacity of the area being tested.
Figure 13: Arrangement for plate load test at footing level in test pit
2.6 **Time interval of loading:** After the application of each load increment, maintain the cumulative load for a selected time interval of not less than 15 min. normally each increment of load is applied after 30 minutes. The duration for all the load increments should be the same.

2.8 **Measurement of Settlement:** Measured the settlement using preferably three dial gauges evenly spaced on the circumference of the plate.

2.9 **Termination of test:** Continue the test until a peak load is reached or until the ratio of load increment to settlement increment reaches a minimum, steady magnitude. If sufficient reaction from the loading platform is available, continue the test until the total settlement reaches at least 10 percent of the plate diameter, or 25-mm. or unless a well-defined failure load is observed. After completion of observations for the last load increment, release the applied load in three approximately equal decrements. Continue recording rebound deflections until the deformation ceases or for a period not smaller than the time interval of loading.
3. **REPORT**:

In addition to recording the time, load, and settlement data, report all associated conditions and observations pertaining to the test, including the following:

1. Date
2. List of personnel
3. Weather conditions
4. Air temperature at time of load increments and
5. Irregularity in routine procedure if any.

4. **CALCULATIONS**:

a. Draw a graph showing relation between (stress & time) and (settlement & time).

b. Draw another graph between stress & settlement with stress at ordinate and settlement at abscissa. From the stress-settlement graph, the ultimate bearing capacity ($\sigma_u$) is taken as shown in Fig. 1, and the allowable bearing capacity ($\sigma_a$) is determined by:

\[ \sigma_a = \frac{\sigma_u}{F.O.S} \]
Figure 14: Load ~ time ~ settlement curve for the plate load test
Figure 14: Load ~ settlement curve for the plate load test
Where Factor of safety (F.O.S) normally for building = 3
But that value is taken as allowable bearing capacity for which the settlement is within permissible limits i.e., not more then 1-in. (25-mm).

The test observations are recorded as follows:

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Stress (Ton/ft²)</th>
<th>Load (Ton)</th>
<th>Settlement (mm)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dial (1)</td>
<td>Dial (2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4: Test observations for Plate Load Test
5. LIMITATIONS:

1. The test results reflect the soil character within a depth of twice the plate size. Foundations are much larger, settlement and resistance to shear failure will depend on the properties of a much thicker stratum.
2. It is a short duration test, hence the test does not give the ultimate settlement, particularly in case of cohesive soils which consolidate for much longer duration.
3. For clayey soils, the ultimate bearing capacity is independent of the width of footing, the bearing capacity for a large foundation is the same as that for the test plate. But in sandy soils, the bearing capacity increases with the size of foundation, and the tests on smaller size bearing plates tend to give conservative values.
4. The test is difficult to perform at greater depths due to possibility of buckling of packing between loading platform and the plate.
5. To find ultimate bearing capacity of stronger soils, very heavy loading platform is required.
6. The test data will be unreliable if plate settlement is restricted by presence of a boulder under the plate.
7. The test is somewhat reliable for uniform soil, which however, need to be ascertained through supplement borings, probing or SPT within the anticipated influence zone for the actual foundation.
6. **PRECAUTIONS:**

1. If the type of load application is to be a reaction against piles, they should be driven or installed prior to test pit excavation to avoid excessive vibration and loosening of soil in the excavation where the load test is to be performed.
2. Prior to loading, protect the test pit and the loading area against moisture changes in soil. This can achieved by covering the test pit against sun or rains or the last 6 to 9-in. excavation at the pit bottom be made just before the commencement of the test.
3. The cut excavation faces should be checked for stability or any need for shoring.
4. The duration for all the load increments should be the same.

7. **CALCULATION OF ULTIMATE BEARING CAPACITY OF ACTUAL FOUNDATION**

Extrapolating plate load-test results to full-size footings is not standard. However, the test results has been used to estimate the bearing capacity of full size foundation.
a. **Clayey Soil**:

For clayey soils, $q_u$ is independent of width of footing, therefore the ultimate bearing capacity of proposed foundation is as follows

$$ q_u \text{ (foundation)} = q_u \text{ (plate)} \quad (1) $$

b. **Cohesionless and C-φ Soil**

In cohesion less and ‘C-φ’ soils the bearing capacity depends on width of foundation, bearing capacity of proposed foundation is given by Equation-2 below.

$$ q_u \text{ (foundation)} = M+N \left( \frac{B_f}{B_p} \right) \quad (2) $$

Where, $M$ includes the $N_c$ and $N_q$ terms (which do not have the width $B$) and $N$ is for the $N_γ$ term. By using several sizes of test plates the equation-2 can be solved graphically for $q_u$ foundation.
Practically for extrapolating plate load test for sands or ‘C-φ’ soils the following relationship is used.

\[ q_{u(f)} = q_{u(P)} \times \frac{B_f}{B_p} \]  

(3)

The use of the equation-3 is recommended only when the ratio is up to about 3 or 4.

When the ratio is 6 to 15 or more, the extrapolation from a plate-load test is little more than a guess.

In above equations, \((B_f)\) is width of foundation in meters and \((B_p)\) is width of plate in meters.
8. CALCULATION OF SETTLEMENT OF ACTUAL FOUNDATION

The plate load test can also be used to determine the settlement for a given intensity of loading \( (q_0) \). The relation between settlement of plate \( (S_p) \) and that of the foundation \( (S_f) \) for the same loading intensity are given below:

For clayey soils,

\[
S_f = S_p \times \frac{B_f}{B_p} \quad \text{(4)}
\]

For sandy soils,

\[
S_f = S_p \left[ \frac{B_f (B_p + 0.3)}{B_p (B_f + 0.3)} \right]^2 \quad \text{(5)}
\]
For designing a shallow foundation for an allowable settlement of \((S_f)\), a trial and error procedure is adopted. First of all, a value of \((B_f)\) is assumed and the value of \((q_0)\) is obtained as:

\[
q_0 = \frac{Q}{A_f} \tag{6}
\]

Where,

\((A_f)\) is area of footing and \((Q)\) is the applied structural load. For the computed value of \((q_0)\), the plate settlement \((S_p)\) is determined from the load settlement curve obtained from the plate load test. The value of \((S_f)\) is computed using Eq. (4) if soil is clayey and using Eq. (5), if it is sand. The computed value of \((S_f)\) is compared with the allowable settlement. The procedure is repeated till the computed value is equal to the allowable settlement.
Advantages of Plate Load Test

1. It provides the allowable bearing pressure at the location considering both shear failure and settlement.
2. Being a field test, there is no requirement of extracting soil samples.
3. The loading techniques and other arrangements for field testing are identical to the actual conditions in the field.
4. It is a fast method of estimating ABP and P – Δ behavior of ground.

Disadvantages of Plate Load Test

1. The test results reflect the behavior of soil below the plate (for a distance of ~2Bp), not that of actual footing which is generally very large.
2. It is essentially a short duration test. Hence, it does not reflect the long term consolidation settlement of clayey soil.
3. Size effect is pronounced in granular soil. Correction for size effect is essential in such soils.
4. It is a cumbersome procedure to carry equipment, apply huge load and carry out testing for several days in the tough field environment.
The test is used to determine bearing capacity and relative density of non-cohesive soil, and to some extent consistency and shearing resistance of cohesive soils.  
The standard penetration test is currently the most popular & economical means to get sub-surface information.  
It can be used for all types of soils but on sandy soil it gives accurate results.

Test Specifications

- Size of clean hole = 55 to 150 mm dia.
- Outer dia. of the split spoon sampler = 50.8 mm (2 in)
- Inner dia. of the split spoon sampler = 35.0 mm (1.5 in)
- Length of split spoon sampler = 600 mm (24 in)
- Weight of the hammer = 63.5 Kg (140 lbs)
- Height of fall of hammer = 760 mm (30 in)
Test Procedure

1. A clean bore hole is made in the ground.
2. Casing is used to support the sides of the hole if required.
3. Drilling tools are removed and spoon sampler or cone is lowered to the bottom of hole.
4. The spoon sampler is driven into the soil by 140 lbs hammer falling from a height of 30 in. at the rate of 30 blows per min.
5. Number of blows required to drive the sampler for every 6 in. till a total penetration of 18 in. is achieved.
6. Numbers of blows for last 12in. penetration is taken as SPT-value or N-value.
7. The test is performed in the bore hole at different depth intervals.
8. For economy the depth interval is increased for higher depth.
9. Average N-value within the influence zone is taken, so that the allowable bearing capacity of the whole influence zone is obtained.
10. If the soil contains gravels, the split spoon may be damaged therefore standard cone is used, the advantage of cone is that the gravel slides sideways.
Figure 15: Typical set up for Standard Penetration test assembly
Figure 16: Schematics of Standard Penetration Test
Refusal to penetration of SPT

If the complete penetration (18-in) of the SPT sampler can not be possible the test is halted, and the bore log shows the refusal if any one of the following cases occur:

1. 50-blows are required for any 6-in penetrations.
2. 100-blows are obtained to drive the required 12-in.
3. 10 successive blows produce no advance.

For the above cases the bore log shows the ratio as 50/2”, indicating that 50-blows resulted in penetration of 2-in.

1. A reasonable value of width of the foundation B in ft is assumed.
2. The allowable bearing capacity in K/ft² is given on the ordinate against SPT value on curves.
The following methods are used to determine the allowable bearing capacity.

2. Teng’s Method.

1. Terzaghi & Peck Graphical Method

Terzaghi & Peck (1967) produced curves for the determination of allowable bearing capacity using N values. The procedure is as follows.

- A reasonable value of width of the foundation B in ft is assumed.
- The allowable bearing capacity in K/ft² is given on the ordinate against SPT value on curves.
Figure 17: Chart for estimating allowable bearing pressure for foundations in sand on basis of results of standard penetration test (Terzaghi & Peck).
2. TENG METHOD

Teng (1962) derived the formula for the determination of allowable bearing capacity on the basis of the graphical method. The formula is:

\[ q_a = 0.7 \Phi \ N_{av} - 3 \left( \frac{B + 1}{2B} \right)^2 \]

Where:

- \( B \) = Width of foundation (ft)
- \( N_{av} \) = Average SPT value within influence zone
- \( Q_a \) = Allowable bearing capacity (K/ft²) for a permissible settlement of 1-in.

The above equation is only applicable for FPS system.
3. MEYERHOF METHOD

Meyerhof (1956, 1974) relationship for the allowable bearing capacity is as follows:

\[ q_a = \frac{N}{4} \quad \text{for, } B \leq 4 \text{ ft} \]

\[ q_a = \frac{N}{6} \left( \frac{B + 1}{B} \right)^2 \quad \text{for, } B > 4 \text{ ft} \]

Where:

- \( B \) = Width of foundation (ft).
- \( N \) = Average SPT value.
- \( q_a \) = Allowable bearing capacity (K/ft\(^2\)) for a permissible settlement of 1-in.
Figure 18: Allowable bearing capacity for surface-loaded footings for 2.54-cm (1-in.) settlement, based on Meyerhof’s equation; $k_d = 1.0$. 
LIMITATIONS OF SPT

Following are some of the limitations:

- In loose coarse gravel, the split spoon tends to slide into the large voids and low penetration resistance is observed.
- Excessive large resistance may be expected when the spoon is blocked by a large piece of gravel or when the piece of gravel sedge into the spoon.

Table 4: Suggested expressions for $q_a$ based on SPT values

<table>
<thead>
<tr>
<th>Units</th>
<th>Meyerhof</th>
<th>Bowles</th>
<th>For</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_a = \frac{N}{4} k_d$</td>
<td>$q_a = \frac{N}{2.5} k_d$</td>
<td>$B \leq 4 \text{ ft}$</td>
</tr>
<tr>
<td>FPS</td>
<td>$q_a = \frac{N}{6} \left( \frac{B + 1}{B} \right)^2 k_d$</td>
<td>$q_a = \frac{N}{4} \left( \frac{B + 1}{B} \right)^2 k_d$</td>
<td>$B &gt; 4 \text{ ft}$</td>
</tr>
<tr>
<td></td>
<td>$q_a = 12 N k_d$</td>
<td>$q_a = 20 N k_d$</td>
<td>$B \leq 1.22 \text{ m}$</td>
</tr>
<tr>
<td>SI</td>
<td>$q_a = 8 N \left( \frac{B + 0.305}{B} \right)^2 k_d$</td>
<td>$q_a = 12.5 N \left( \frac{B + 0.305}{B} \right)^2 k_d$</td>
<td>$B &gt; 1.22 \text{ m}$</td>
</tr>
</tbody>
</table>

Note: $k_d = 1 + 0.33 \left( \frac{D}{B} \right) \leq 1.33$ suggested by Meyerhof

$q_a$ units: kips/ft$^2$ @ FPS system
kN/m$^2$ @ SI system
FACTORS AFFECTING REPRODUCIBILITY OF SPT

1. Effect of overburden pressure. Soils of same density will give smaller count near ground surface.
2. Use of a drive shoe that is badly damaged or worn from too many driving to refusal.
3. Failure to properly seat the sampler on undisturbed material in the bottom of the boring.
4. Inadequate cleaning of loosened material from the bottom of the boring.
5. Driving a stone ahead of the sampler.
6. Variations in the 30-in drop height of the drive weight, since this is often done by eye.
7. Interference with the free fall of the drive weight by guides and/or the rope used to hoist the drive weight for successive blows.
8. Failure to maintain sufficient hydrostatic pressure in the boring so that the test zone becomes “quick.” Too large a hydrostatic pressure as with use of drilling mud and/or head greater than static ground level may also influence N.
9. Use of too light or too long a string of drill rods.
10. Careless work on the part of the drill crew.
Advantages Of SPT

1. The test is too economical in terms of cost per unit of information.
2. The test has been used for estimating the stress and strain modulus Es.
3. The test has been used for estimating the bearing capacity of foundations.
4. A tube (split spoon) recovery length of 18-in. produces a visual profile of bearing strata.
5. Although the split spoon samples are disturbed but they can still be tested for strength properties.
7. The accumulation of large SPT data base which is continuously expanding.
8. The equipment is very simple.
9. Other methods can be readily used for supplement the SPT, when the borings indicate more refinement in sample or data.
10. Standard penetration test are not only useful in granular soil, they are also used in other types of soils.
Disadvantages Of SPT

1. Requires the preparation of bore hole.
2. Dynamic effort is related to mostly static performance.
3. SPT is abused, standards regarding energy are not uniform.
4. If hard stone is encountered, difficult to obtain reliable result.
5. Test procedure is tedious and requires heavy equipment.
6. Not possible to obtain properties continuously with depth.
CONE PENETRATION TEST

- It is a method for in-situ soil exploration.
- No samples are taken in this test.
- The test cone has a standard tip as shown in the figure.
- The cone is pushed into the soil by jacking instead of driving by hammer.
- It is therefore known as static cone penetration test.
- The rate of penetration varies from 15-20-mm/sec.
- Some cones are driven by blows of hammer and the test is therefore known as dynamic cone penetration test.
- Resistance of soil against cone penetration is measured and recorded as ‘q_c’.
- The cone penetration resistance (q_c) is then related to different soil properties.
- Different types of cone penetration equipment have been developed.
- The Dutch cone is one of the most widely used.
- The specification of Dutch cone penetrometer is as follows.
  - The diameter of conical point of the cone = 1.4-in. (3.6-cm)
  - The area of the base = 10-cm²
  - The apex angle of the cone = 60°.
Figure 19: Different types of cones used for CPT

(a) Mechanical friction cone penetrometer tip
(b) Electric-cone penetrometer tip
(c) Cone resistance versus depth. Note corresponding soil profile.
Figure 20: Typical set up for Static Cone Penetration test assembly
Figure 21: Cone penetration test equipment and arrangement
Various cone configuration exist in the field. Some of them are as follows:

1. **Dutch Cone** - the simplest cone penetrometer measures only the tip resistance.
2. **Friction Cone** - some cones measure the tip resistance as well as the skin resistance through a friction sleeve provided above the cone point. The sleeve friction is used to identify the soil type. The friction sleeve has a cylindrical area of 15-cm². The side or skin friction \( f_s \) is measured as the frictional resistance per unit area on the friction sleeve.
3. **Electric Cone** - the recent development is the electric cone penetrometer which enables a continuous record of the penetration resistance through a data logger.
4. **Piezocone** - Piezocones also measure the excess pore water pressure which is developed in cohesive soil in the vicinity of the cone tip during driving.
ADVANTAGES OF CPT
1. It is very fast – particularly when electronic data acquisition equipment is used to record the tip resistance as well as the skin resistance.
2. It allows continuous record of resistance in the stratum of interest.
3. It is useful in very soft soil where undisturbed sampling is difficult.
4. A number of correlations between cone resistance and desired engineering properties are available.
5. The method is applicable for both cohesionless soil and cohesive soil.
6. The results are most reliable for sand and silt with a degree of saturation of less than 85%.

DISADVANTAGES OF CPT
1. The method is only applicable to fine grained soils (clay, silt, fine sand) which do not have massive resistance to penetration.
2. As no sampling is made during the test therefore soil type can not be known. Additional boring need to be performed to determine the soil type.
3. Since the cone is pushed by jacking, therefore for stronger soil heavy rigs may be required to advance the cone into the soil. If light-weight equipment is used the test will be only possible for soft soil otherwise the rig will be lifted up.
According to Meyerhof (1956) the cone penetration resistance $q_c$ has been related to the allowable bearing capacity.

\[
q_a = \frac{q_c}{30} \quad B \leq F_4
\]

\[
q_a = \frac{q_c}{50} \left( \frac{B + F_3}{B} \right)^2 \quad B > F_4
\]

Where,

$q_c =$ average cone resistance within the influence zone, kPa or kip/ft$^2$
$q_a =$ allowable bearing capacity kip/ft$^2$
$B =$ least lateral dimension of the footing, ft. or m.
$F_i =$ constant which depends on the unit used, the values are given in the table-1.

<table>
<thead>
<tr>
<th>$F$</th>
<th>SI, m</th>
<th>FPS, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.05</td>
<td>2.5</td>
</tr>
<tr>
<td>2</td>
<td>0.08</td>
<td>4.0</td>
</tr>
<tr>
<td>3</td>
<td>0.30</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>1.20</td>
<td>4.0</td>
</tr>
</tbody>
</table>
According to Begemann (1974) the undrained shear strength ($s_u$) of cohesive soil has been related to the cone resistance as follows:

$$s_u = \frac{q_c - p_0}{N'_c}$$

Where,

- $p_0$ = Effective overburden pressure at CPT depth
- $N'_c$ = constant ranging from 5 to 70 depending on deposit and OCR (values ranging from 9 to 15 are most common.)

A compressibility coefficient (similar to the compression index of cohesive soil) is suggested by de Beer and martens (1951):

$$C = \frac{1.5q_c}{\sigma'_{v0}}$$

Where

- $q_c$ = cone resistance (MN/m$^2$)
- $\sigma'_{v0}$ = effective overburden pressure (MN/m$^2$)
The settlement $S_i$ at the centre of a layer of thickness $H$ is then given by

\[
S_i = \frac{H}{C} \log \left( \frac{\sigma_{v0}' + \Delta q}{\sigma_{v0}'} \right)
\]

Where

$\Delta q$ = increase in stress at the centre of the layer due to a foundation pressure of $q$

Above method is considered to overestimate the value of $S_i$.

A rapid conservative method was suggested by Meyerhof (1974):

\[
S_i = \frac{q_n B}{2q_c}
\]

Where

$q_n$ = net applied loading = $q - \sigma_{v0}'$
$q_c$ = average cone resistance over a depth below the footing equal to the breadth $B$
The cone penetration resistance has been correlated with the equivalent Young’s modulus (E) of soils by various investigators.

Schmertmann (1970) has given a simple correlation for sand as

\[ E = 2q_c \]

Trofimenkov (1974) has given the following correlations for the stress–strain modulus in sand and clay.

\[ E = 3qc \]  \hspace{1cm} \text{(for sands)}
\[ E = 7qc \]  \hspace{1cm} \text{(for clays)}

Above correlations can be used in the calculation of elastic settlement of foundations.