Settlement of Foundation

- Immediate Settlement: Occurs immediately after the construction. This is computed using elasticity theory (Important for Granular soils).
- Primary Consolidation: Due to gradual dissipation of pore pressure induced by external loading and consequently expulsion of water from the soil mass, hence volume change. (Important for Inorganic clays)
- Secondary Consolidation: Occurs at constant effective stress with volume change due to rearrangement of particles. (Important for Organic soils)

For any of the above mentioned settlement calculations, we first need vertical stress increase in soil mass due to net load applied on the foundation.

Elasticity

\[
\begin{align*}
\varepsilon_{xx} &= -\frac{1}{E} \left[ \sigma_{xx} - \nu (\sigma_{yy} + \sigma_{zz}) \right], \\
\varepsilon_{yy} &= -\frac{1}{E} \left[ \sigma_{yy} - \nu (\sigma_{xx} + \sigma_{zz}) \right], \\
\varepsilon_{zz} &= -\frac{1}{E} \left[ \sigma_{zz} - \nu (\sigma_{xx} + \sigma_{yy}) \right], \\
\varepsilon_{xy} &= -\frac{1 + \nu}{E} \sigma_{xy}, \\
\varepsilon_{yz} &= -\frac{1 + \nu}{E} \sigma_{yz}, \\
\varepsilon_{xz} &= -\frac{1 + \nu}{E} \sigma_{xz}.
\end{align*}
\]

\( \nu \) is Poisson’s ratio
\( E \) is the modulus of elasticity (Young’s modulus)
Stress Distribution: Concentrated load

Boussinesq Analysis

\[ u_x = \frac{P(1+\nu)}{2\pi ER} \left( \frac{z}{R^2} - \frac{(1-2\nu)(1-\frac{z}{R})}{R^4} \right) \]
\[ u_y = 0, \]
\[ u_z = \frac{P(1+\nu)}{2\pi ER} \left( \frac{z}{R} \right) \left( \frac{1}{R} \right) \]

\[ z = 0 : \quad u_z = \frac{P(1-\nu^2)}{\pi ER} \]

Where,

\[ r = \sqrt{x^2 + y^2} \]
\[ R = \sqrt{x^2 + y^2 + z^2} \]

Influence Factor for General solution of vertical stress

\[ \sigma = \frac{P}{\pi z} I \]
Vertical Stress: Uniformly Distributed Circular Load

**Uniformly Distributed Circular Load**

![Diagram of Uniformly Distributed Circular Load]

\[ r = 0 : \quad \sigma_{zz} = p(1 - \frac{z^2}{b^2}), \]
\[ r = 0 : \quad \sigma_{rr} = p\left(1 + \frac{z^2}{b^2}\right) - \frac{1}{2}\left(1 - \frac{z^2}{b^2}\right) \]
\[ b = \sqrt{z^2 + a^2} \]

---

Vertical Stress: Uniformly Distributed Circular Load

**Rigid Plate on half Space**

![Diagram of Rigid Plate on half Space]

\[ P = \pi a^2 b \]
\[ z = 0, \ 0 < r < a : \quad \sigma_{zz} = \frac{1}{2} \sqrt{1 - \frac{r^2}{a^2}} \]
\[ z = 0, \ 0 < r < a : \quad u_r = \frac{E}{2} \left(1 - \frac{r^2}{a^2}\right) \]

---

Vertical Stress: Rectangular Area

**Rectangular Area**

![Diagram of Rectangular Area]

\[ P = b \left( B' + L' + \frac{b}{2} \right) \]
\[ L_x = \frac{1}{2} \left( \frac{2bL_x \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]
\[ = \frac{1}{2} \left( \frac{2bL_x \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]
\[ L_y = \frac{1}{2} \left( \frac{2bL_y \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]
\[ = \frac{1}{2} \left( \frac{2bL_y \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]

\[ \text{Other terms:} \]
\[ L_x = \frac{1}{2} \left( \frac{2bL_x \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]
\[ = \frac{1}{2} \left( \frac{2bL_x \sqrt{B'^2 + L'^2} + \frac{b}{2}}{\sqrt{B'^2 + L'^2} + \frac{b}{2}} \right) \]
Vertical Stress: Rectangular Area

Pressure Bulb

Pressure Bulb for Square Foundation
Influence Value

This Model is good for normally-consolidated, lightly overconsolidated clays, and variable deposits.
Westergaard's Method

- Provided solution for layered soils
- Point Loads
- Assumption:
  Elastic soil mass is laterally reinforced by numerous, closely spaced, horizontal sheets of negligible thickness but infinite rigidity, that allow only vertical movement but prevent the mass as a whole from undergoing any lateral strain.

\[ \sigma_z = \frac{P}{2\pi e} \left[ \frac{1}{C^2 + (r/z)^2} \right] \]

\[ C = \left[ \frac{1-2v}{2(1-v)} \right] \]

This Model is specially good for pre-compressed or overconsolidated clays.

Westergaard’s influence Chart

\[ \sigma_{z,2} = n \times 0.001 \]

Fröhlich Chart with concentration factor \( m' = 4 \)

\[ \Delta \sigma_z = n(0.005)q \]

This Model is specially good for Sands.
**Simplified Methods (Poulos and Davis, 1974)**

**Circular Foundation:**
\[
\Delta \sigma_z = \left[ 1 - \left( 1 + \left( \frac{B}{2z_f} \right)^2 \right)^{-1.5} \right] (q - \sigma'_{zd})
\]

**Square Foundation:**
\[
\Delta \sigma_z = \left[ 1 - \left( 1 + \left( \frac{B}{2z_f} \right)^2 \right)^{-1.76} \right] (q - \sigma'_{zd})
\]

**Strip Foundation:**
\[
\Delta \sigma_z = \left[ 1 - \left( 1 + \left( \frac{B}{2z_f} \right)^2 \right)^{-1.60} \right] (q - \sigma'_{zd})
\]

**Rectangular Foundation:**
\[
\Delta \sigma_z = \left[ 1 - \left( 1 + \left( \frac{B}{2z_f} \right)^{1.38 + 0.62B/L} \right)^{-2.60 - 0.44B/L} \right] (q - \sigma'_{zd})
\]

**Approximate Methods**

**Rectangular Foundation:**
\[
\Delta \sigma_z = \frac{B L}{(B + z)(L + z)}
\]

**Square/Circular Foundation:**
\[
\Delta \sigma_z = \frac{B^2}{(B + z)^2}
\]

**Strip Foundation:**
\[
\Delta \sigma_z = \frac{B}{(B + z)}
\]
Contact Pressure and Settlement distribution

Cohesive Soil - Flexible Footing
Granular Soil Flexible Footing

Cohesive Soil - Rigid Footing
Granular Soil Rigid Footing

Elastic settlement of Foundation

Elastic settlement:

\[ S_e = \frac{1}{E_s} \int \varepsilon_e dz = \frac{1}{E_s} \int \left( \Delta \sigma_z - \mu_s \Delta \varepsilon_z - \mu_s \Delta \sigma_y \right) dz \]

- \( E_s \): Modulus of elasticity
- \( H \): Thickness of soil layer
- \( \mu_s \): Poisson’s ratio of soil

Elastic settlement for Flexible Foundation:

\[ S_e = \frac{qB}{E_s} \left( 1 - \mu_s^2 \right) I_f \]

- \( I_f \): Influence factor; depends on the rigidity and shape of the foundation
- \( E_s \): Avg elasticity modulus of the soil for (4B) depth below foundn level

Elastic settlement of Foundation

<table>
<thead>
<tr>
<th>Soil</th>
<th>( E_s ), kg/cm²</th>
<th>Soil</th>
<th>( E_s ), kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very clay</td>
<td>20 - 150</td>
<td>Sand</td>
<td>140 - 600</td>
</tr>
<tr>
<td>Soft</td>
<td>50 - 250</td>
<td>Silty</td>
<td>70 - 210</td>
</tr>
<tr>
<td>Medium</td>
<td>150 - 500</td>
<td>Loose</td>
<td>100 - 240</td>
</tr>
<tr>
<td>Medium</td>
<td>500 - 1000</td>
<td>Dense</td>
<td>300 - 800</td>
</tr>
<tr>
<td>Sandy</td>
<td>250 - 2500</td>
<td>Sand and gravel</td>
<td>450 - 14500</td>
</tr>
<tr>
<td>Glaciol till</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>100 - 1500</td>
<td>Silt</td>
<td>20 - 200</td>
</tr>
<tr>
<td>Dense</td>
<td>1500 - 2700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very dense</td>
<td>4800 - 14500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Elastic settlement of Foundation

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>SPT</th>
<th>CPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (normally consolidated)</td>
<td>$E = 500 \left( N + 15 \right)$</td>
<td>$E = 2 \text{ to } 4 \cdot \gamma'$</td>
</tr>
<tr>
<td>Sand (saturated)</td>
<td>$E = 250 \left( N + 15 \right)$</td>
<td>$E = 6 \text{ to } 30 \cdot \gamma'$</td>
</tr>
<tr>
<td>Gravelly sand</td>
<td>$E = 1200 \left( N + 6 \right)$</td>
<td>$E = 3 \text{ to } 6 \cdot \gamma'$</td>
</tr>
<tr>
<td>Clayey sand</td>
<td>$E = 320 \left( N + 15 \right)$</td>
<td>$E = 1 \text{ to } 2 \cdot \gamma'$</td>
</tr>
<tr>
<td>Silty sand</td>
<td>$E = 200 \left( N + 6 \right)$</td>
<td>$E = 5 \text{ to } 8 \cdot \gamma'$</td>
</tr>
</tbody>
</table>

Soil Strata with Semi-infinite depth

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, saturated</td>
<td>0.4 - 0.5</td>
</tr>
<tr>
<td>Clay, unsaturated</td>
<td>0.1 - 0.3</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>0.2 - 0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.3 - 0.35</td>
</tr>
<tr>
<td>Gravelly sand (oc-sand)</td>
<td>0.15</td>
</tr>
<tr>
<td>Fine grained (void ratio = 0.4 - 0.7)</td>
<td>0.25</td>
</tr>
<tr>
<td>Rock</td>
<td>0.1 - 0.45 (depends somewhat on type of rock)</td>
</tr>
<tr>
<td>Loess</td>
<td>0.1 - 0.3</td>
</tr>
<tr>
<td>Ice</td>
<td>0.36</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shape</th>
<th>Flexible Foundation</th>
<th>Rigid Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centre</td>
<td>Corner</td>
<td>Average</td>
</tr>
<tr>
<td>Circle*</td>
<td>1/2π</td>
<td>0.95</td>
</tr>
<tr>
<td>Square</td>
<td>1.12</td>
<td>0.56</td>
</tr>
<tr>
<td>Rectangle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$L/B = 1.5$</td>
<td>1.36</td>
<td>0.68</td>
</tr>
<tr>
<td>$L/B = 2$</td>
<td>1.52</td>
<td>0.76</td>
</tr>
<tr>
<td>$L/B = 5$</td>
<td>2.10</td>
<td>1.03</td>
</tr>
<tr>
<td>$L/B = 10$</td>
<td>2.52</td>
<td>1.26</td>
</tr>
<tr>
<td>$L/B = 100$</td>
<td>3.38</td>
<td>1.69</td>
</tr>
</tbody>
</table>

*Use diameter for $B$
Steinbrenner’s Influence Factors for Settlement of the Corners of loaded Area LxB on Compressible Stratus of μ = 0.5, and Thickness H,

\[ S = C_1 C_2 \left( q - \gamma D_f \right) \sum_{0}^{z_f} \frac{I_z}{E_z} \Delta z \]

- \( C_1 \) = Correction factor for foundation depth
- \( C_2 \) = Correction factor for creep effects

For square and circular foundations:
- \( I_1 = 0.1 \) at \( z = 0 \)
- \( I_2 = 0.5 \) at \( z = z_1 = 0.5B \)
- \( I_3 = 0 \) at \( z = z_2 = 2B \)

For foundation with L/B >10:
- \( I_1 = 0.2 \) at \( z = 0 \)
- \( I_2 = 0.5 \) at \( z = z_1 = B \)
- \( I_3 = 0 \) at \( z = z_2 = 4B \)

Interpolate the values for 1 < L/B < 10

Example

\[ S = C_1 C_2 \left( q - \gamma D_f \right) \sum_{0}^{z_f} \frac{I_z}{E_z} \Delta z \]

For square and circular foundations
- \( E_c \approx 2.5q_f \)

For rectangular foundations
- \( E_r \approx 3.5q_f \)

Correlation with SPT data:
- \( E_r = 800N/m^2 \) in kPa
Burland and Burbidge’s Method for Sandy Soils

Depth of Stress Influence (z’):

\[ z' = 2B \text{ and } z' = z'' \text{ - Thickness of soft layer below foundation} \]

Elastic Settlement (S_e):

\[ S_e = \alpha_1 \frac{\sigma_v \sigma_o}{B} \left[ \frac{1.25 (L/B)}{0.25 + (L/B)} \right]^2 \]

where B is in meters and \( \sigma' \) is in kPa

Compressibility index: \( \alpha_1 = 1.7 \left( \text{parameter} \right) \) for NC sand

\[ \alpha_1 = 0.57 \left( \text{parameter} \right) \text{ for OC sand} \]

\[ \alpha_1 = 0.0047 \text{ for NC sand} \]

\[ \alpha_1 = 0.0016 \text{ for OC sand with } q_{oa} \leq p_{oa} \]

\[ \alpha_1 = 0.0047 \text{ for OC sand with } q_{oa} \leq p_{oa} \]

\[ \frac{z'}{z''} < 1 \]

\[ q' = q_{oa} \text{ for NC sand and for OC sand with } q_{oa} \leq p_{oa} \]

\[ q' = q_{oa} - 0.67 p_{oa}' \text{ for OC sand with } q_{oa} > p_{oa} \]

Settlement due to Primary Consolidation

For NC clay (\( \sigma'_{c} < \sigma'_{p} < \sigma'_{c} + \Delta \sigma'_{c} \))

\[ S_c = \frac{C_h}{1 + e_0} \log \left( \frac{\sigma'_{c} + \Delta \sigma'_{c}}{\sigma'_{c}} \right) \]

For OC clay (\( \sigma'_{p} < \sigma'_{c} < \sigma'_{c} + \Delta \sigma'_{c} \))

\[ S_c = \frac{C_o}{1 + e_0} \log \left( \frac{\sigma'_{c} + \Delta \sigma'_{c}}{\sigma'_{p}} \right) \]

For OC clay (\( \sigma'_{c} < \sigma'_{p} < \sigma'_{c} + \Delta \sigma'_{c} \))

\[ S_c = \frac{C_h}{1 + e_0} \log \left( \frac{\sigma'_{p} + \Delta \sigma'_{c}}{\sigma'_{p}} \right) + \frac{C_o}{1 + e_0} \log \left( \frac{\sigma'_{c} + \Delta \sigma'_{c}}{\sigma'_{p}} \right) \]

\( \sigma'_{c} = \) Average effective vertical stress before construction

\( \Delta \sigma'_{c} = \) Average increase in effective vertical stress

\( \sigma'_{p} = \) Effective pre-consolidation pressure

\( e_0 = \) Initial void ratio of the clay layer

\( C_h = \) Compression Index

\( C_o = \) Swelling Index

\( H_c = \) Thickness of the clay layer

\( \Delta \sigma'_{p} = \frac{1}{3} \left( 4 \Delta \sigma'_{c} + \Delta \sigma'_{c} \right) \)

Settlement Correction for Effect of 3-D Consolidation

\[ (S_c)_{3D} = \lambda (S_c)_{1D} \]

where \( \lambda \) is the 3-D settlement coefficient.

\( \lambda = 1 - \frac{H_c}{B} \)

\( \lambda = 1 - \frac{H_c}{B} \)  for 3-D consolidation.

\( \lambda = 1 - \frac{H_c}{B} \)  for very sensitive clay.

\( \lambda = 1 - \frac{H_c}{B} \)  for normally consolidated clay.
Fox’s Depth Correction Factor for Rectangular Footings of (L)x(B) at Depth (D)

\[ \frac{(S_d)}{(S_f)} = \text{Depth factor} \]

Rigidity Factor as per IS:8009-1976

Total settlement of rigid foundation
Total settlement at the center of flexible foundation

Rigidity factor = 0.8

Time Rate of Settlement

\[ S_t = S_0 + US \quad T = \frac{c}{H} \]

Assumption of pore pressure distribution under the given stress conditions

For open clay layer with two-way drainage use curve for \( V = 1 \)

Settlement Due to Secondary Consolidation

\[ S_p = \frac{C_p H_p \log\left(\frac{t_2}{t_1}\right)}{1 + e_p} \]

\( C_p = \) Secondary Compression Index = \( \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)} \)
\( e_p = \) Void ratio at the end of primary consolidation
\( H_p = \) Thickness of Clay Layer

Secondary consolidation settlement is more important in the case of organic and highly-compressible inorganic clays
Total Settlement from SPT Data for Cohesionless soil

Multiply the settlement by factor \( W' \).

Total Settlement from CPT Data for Cohesionless soil

\[ S = \frac{H \ln \left( \frac{\sigma_v + \Delta \sigma}{\sigma_v} \right)}{C} \]

- Depth profile of cone resistance can be divided in several segments of average cone resistance.
- Average cone resistance can be used to calculate constant of compressibility.
- Settlement of each layer is calculated separately due to foundation loading and then added together.

Plate Load Test – IS:1888-1982

Ball and socket arrangement: Head room for person to sit and observe dial gauge.

Dial gauge:
- Loaded platform
- Dial gauge fixture
- Test plate or block
- Pit, strutted if necessary

- AS required
- Head room for person to sit and observe dial gauge.
- Load and socket arrangement.
Plate Load Test – IS:1888-1982

Bearing Plate:
- Rough mild steel bearing plate in circular or square shape
- Dimension: 30 cm, 45 cm, 60 cm, or 75 cm.
  - Thickness > 25 mm
- Smaller size for stiff or dense soil. Larger size for soft or loose soil
- Bottom of the plate is grooved for increased roughness.
- Concrete blocks may be used to replace bearing plates.

Test Pit:
- Usually to the depth of foundation level.
- Width equal to five times the test plate
- Carefully leveled and cleaned bottom.
- Protected against disturbance or change in natural formation

Procedure:
- Selection of Location
  - Based on the exploratory boring.
  - Test is carried out at the level of proposed foundation. If water table is below the foundation level but the depth is less than width of plate then the test is carried out at the level of water table. If the water table is above the foundation level then the water level is reduced to proposed foundation level by pumping out the water during the test; however, in case of high permeability material perform the test at the level of water table.
  - In case the soil is expected to have significant capillary action and the water table is within 1 m below the foundation, it becomes necessary to perform the test at the level of water table in order to avoid the effect of higher effective stresses due to capillary action resulting in lower values of interpreted settlements.
- Reaction supports should be at least (3.5 x width of plate) away from the test plate location, and loading arrangement should provide sufficient working space.
- Test plate should be placed over a 5 mm thick sand layer and it should be centered with the loading arrangement.
A seating pressure of 7 kPa is applied and then released after some time before the test.

Loads are applied in the increments of approximately 1/5th of the estimated ultimate safe load. (Or, one may choose to increase the load at an increment of 0.5 kN.)

At each load settlement is recorded at time intervals of 1, 2, 4, 6, 9, 16, 25 min and thereafter at intervals of one hour.

For clayey soil, the load is increased when time settlement curve shows that the settlement has exceeded 70-80% of the probable ultimate settlement or a duration of 24 Hrs.

For the other soils, the load is increased when the settlement rate drops below 0.02 mm/min.

The minimum duration for any load should, however, be at least 60 min.

Dial gauges used for testing should have at least 25 mm travel and 0.01 mm accuracy.

The load settlement curve can then be plotted from settlement data.

Zero Correction:

The intersection of the early straight line or nearly straight line with zero load line shall be determined and subtracted from the settlement readings to allow for the perfect seating of the bearing plate.
Plate Load Test: Some Considerations

- The width of test plate should not be less than 30 cm. It is experimentally shown that the load settlement behavior of soil is qualitatively different for smaller widths.
- The settlement influence zone is much larger for the real foundation sizes than that for test plate, which may lead to gross misinterpretation of expected settlement for proposed foundation.
- The foundation settlements in loose sands are usually much larger than what is predicted by plate load test.
- Plate load test is relatively short duration test and gives mostly the immediate settlements. In case of granular soils the immediate settlement is close to total settlements. However, due to considerable consolidation settlement in case of cohesive soils, the plate load test becomes irrelevant in such case. Although the following relationship is suggested for interpreting the settlements in cohesive soils, it can not be used seriously for design.

\[ \frac{S_1}{S_2} = \frac{B_1}{B_2} \]

Plate Load Test: Bearing Capacity

- In case of dense cohesionless soil and highly cohesive soils ultimate bearing capacity may be estimated from the peak load in load-settlement curve.
- In case of partially cohesive soils and loose to medium dense soils the ultimate bearing capacity load may be estimated by assuming the load settlement curve so as to be a bilinear relationship.

A more precise determination of bearing capacity load is possible if the load-settlement curve is plotted in log-log scale and the relationship is assume to be bilinear. The intersection point is taken as the yield point of the bearing capacity load.

For cohesionless soil \[ q_{uc} = \frac{B_1}{B_2} q_{u0} \]

For cohesive soil \[ q_{uc} = q_{u0} \]
Differential Settlement

Terzaghi’s recommendation:
Differential settlement should not exceed 50% of the total settlement calculated for the foundation.
Considering the sizes of different footing, the following criteria is suggested for buildings:

- For raft foundation the requirements shall be more stringent and they may designed for the following criteria:
  
  1. Differential settlement of footing \( \leq 75\% \) of max calculated settlement of footing
  
  2. Differential settlement of raft footing \( \leq 37\% \) of max calculated settlement of raft footing

\[ \Delta = \text{maximum settlement} \]
\[ \delta = \text{differential settlement} \]
\[ \delta / \Delta = \text{angular distortion} \]

Allowable maximum and differential settlements as prescribed by IS:1904-1986 are given on the next slide.
Rotation of Footings Subjected to Moment

- Footings subjected to moment will have the tendency to rotate and the amount of rotation can be estimated by assuming that the footing is supported on a bed of springs and using the modulus of sub-grade reaction theory.

Modulus of sub-grade reaction:

\[ k = \frac{E_s}{B(1-v^2)} \]

Moment about the base due to soil reaction:

\[ M = 2 \int_0^L k (\theta) dx = \frac{LB^2 k \theta}{12} \]

Rotation of Footings Subjected to Moment

\[ \theta = \frac{12M}{LB^2 k} = \frac{12M}{LB^2 E_s} \left( 1-v^2 \right) \left( \frac{E_s}{LB^2 k} \right) \]

<table>
<thead>
<tr>
<th>k values</th>
<th>Flexible</th>
<th>Rigid</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1.04</td>
<td>1.59</td>
</tr>
<tr>
<td>1</td>
<td>3.15</td>
<td>4.17</td>
</tr>
<tr>
<td>2</td>
<td>3.57</td>
<td>4.59</td>
</tr>
<tr>
<td>5</td>
<td>3.77</td>
<td>4.87</td>
</tr>
<tr>
<td>20</td>
<td>3.81</td>
<td>4.98</td>
</tr>
<tr>
<td>300</td>
<td>3.82</td>
<td>5.06</td>
</tr>
</tbody>
</table>

Allowable Bearing Pressure

- Maximum bearing pressure that can be applied on the soil satisfying two fundamental requirements
  - Bearing capacity with adequate factor of safety
    - net safe bearing capacity
  - Settlement within permissible limits (critical in most cases)
    - net safe bearing pressure
**Allowable Bearing Pressure**

**Terzaghi and Peck (1967):**

\[ q_{aw} = 1.37(N^* - 3)(\frac{B + 0.3}{2B})^2 \]

\[ R' = \left( \frac{D_0 - D}{D} \right) \left[ R'_{\infty} \right] \]

\[ R'_{\infty} = 0.5 \left( 1 + \frac{D_0 - D}{D} \right) \]

\[ R_{in} = \text{depth correction factor} \]

\[ = 1 + 0.2 \left( \frac{D_0}{D} \right) \leq 1.2 \]

**Peck, Hanson, and Thornburn (1974):**

\[ q_{aw} = 0.44C_n N^* N \]

**Teng's (1962) Correlation:**

**Net safe bearing pressure**

\[ q_{aw} = 1.4(N_{aw} - 3)(\frac{B + 0.3}{2B})^2 \]

\[ R' C_n S_n \]

\[ S_n = \text{Permissible settlement in mm. (25 mm)} \]

\[ C_n = \text{water table correction} \]

\[ = 0.5 + 0.5 \left( \frac{D_0}{D} \right) \]

**Effective Overburden stress**
Allowable Bearing Pressure

Meyerhof’s (1974) Correlation:
Net safe bearing pressure
\[ q_{s,n} = 0.49 N' \rho_0 S_n \text{ kN/m}^2 \text{ for } B \leq 1.2 \text{ m} \]
\[ q_{s,n} = 0.32 N' \rho_0 \left( \frac{B + 0.3}{B} \right)^3 S_n \text{ kN/m}^2 \text{ for } B > 1.2 \text{ m} \]
\[ R_n = \text{depth correction factor} \]
\[ R_{nit} = \text{depth correction factor} \]
\[ = 1 + 0.5 \frac{D}{B} \leq 1.2 \]
\[ = 1 + 0.15 \frac{D}{B} \leq 1.33 \]

Bowel’s (1982) Correlation:
\[ q_{s,n} = 0.75 N' \rho_0 S_n \text{ kN/m}^2 \text{ for } B \leq 1.2 \text{ m} \]
\[ q_{s,n} = 0.48 N' \rho_0 \left( \frac{B + 0.3}{B} \right)^3 S_n \text{ kN/m}^2 \text{ for } B > 1.2 \text{ m} \]

N-value corrected for overburden using Bazaraa’s equation, but the N-value must not exceed field value.

IS Code recommendation: Use total settlement correlations with SPT data to determine safe bearing pressure.

Correlations for raft foundations:
Rafts are mostly safe in bearing capacity and they do not show much differential settlements as compared to isolated foundations.

Teng’s Correlation: \[ q_{s,n} = 0.7 (N^* - 3) R' C' \gamma_{p,0} \text{ kN/m}^2 \]

Peck, Hanson, and Thornburn (1974): \[ q_{s,n} = 0.88 C' N^* \gamma_{p,0} \text{ kN/m}^2 \]

Correlations using CPT data:
Meyerhof’s correlations may be used by substituting \( q_c/2 \) for \( N \), where \( q_c \) is in kg/cm².

Net vs. Gross Allowable Bearing Pressure

<table>
<thead>
<tr>
<th>Gross load</th>
<th>( q_{s,n} = \frac{Q}{B} + D_T (T - n_T) \gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross load</td>
<td>( q_{s,n} = \frac{Q}{B} + D_T (T - n_T) \gamma )</td>
</tr>
<tr>
<td>Gross load</td>
<td>( q_{s,n} = \frac{Q}{B} + D_T (T - n_T) \gamma )</td>
</tr>
</tbody>
</table>

Usually \( D_T \gamma \) is much smaller than \( D_T \gamma \)