Foundation Engineering
Shallow and Deep Foundations

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Bearing Capacity of
Shallow Foundations
Footing and Foundation

- Loads/Stresses and Deformations/Strains transmitted from the superstructure to the substructure and the influenced soil

Broad Classification

- Shallow Foundation: $D_f < B$
  - Soil at shallow depth possess adequate strength to bear the superstructure loads

- Deep Foundation: $D_f > B$
  - Upper stratum of the soil is weak, and the loads are to be transmitted to larger depths
Broad Classification

Foundation

Technical classification

Shallow Foundation
Foundation for which the bearing failure lines reaches the ground surface and manifests ground heaving or subsidence

Deep Foundation
Foundation for which the bearing failure lines do not reach the ground surface

Hybrid Foundation

Piled Raft

Types of Shallow Foundations

- Spread / Pad / Strip footing
- Combined footing
- Strap footing
- Raft footing
Types of Deep Foundations

- Crude assumption
  - Depth of foundation greater than the width
- Types
  - Piles
  - Piers
  - Caissons
  - Drilled Shafts
  - Well

Hybrid Foundations

- Piled-Raft Foundations
  - Combination of shallow and deep foundations
- Compensated Mat foundations
  - Mat foundations supporting basement levels
Stable Foundation

- Proper location in regard to future influence which can adversely affect its performance
  - Many different contributory factors
  - Some evaluated analytically, some by engineering judgment

- Foundation must be safe from failure
  - Can be evaluated analytically considering various factors

- Foundation must not settle or deflect sufficiently to damage the structure and impair its usefulness
  - Should not deflect or settle objectionably
    - Level of deflection with regard to objection is not clearly defined
Causes of Foundation Instability

- Local erosion of soil due to flowing water
- Underground defects
- Unconsolidated fill and marshy lands
- Expansive soils – Swelling and Shrinkage
- Adjacent properties
- Ground water, Frosting and Dessication

Failure Modes of Shallow Foundations

- Three prominent failure modes of shallow foundations
  - General Shear Failure
    - Footing resting on dense sand or stiff clay
    - Three distinct failure zones are obtained
      - Rankine’s Active Zone – Zone I
      - Zone of radial shear – Zone II
      - Rankine’s Passive zone – Zone III
Failure Modes of Shallow Foundations

- Three prominent failure modes of shallow foundations

  - **Local Shear Failure**
    - Footing resting on medium dense sand or medium stiff clay
    - Three distinct failure zones are obtained
      - Rankine’s Active Zone – Zone I
      - Zone of radial shear – Zone II
      - Imperfect Rankine’s Passive zone – Zone III
        - associated with ground heaving

![Diagram of Failure Modes of Shallow Foundations](image)

Punching Shear Failure

- Three prominent failure modes of shallow foundations

  - **Failure zones**
    - One distinct failure zone is obtained
      - Rankine’s Active Zone – Zone I
    - Imperfect failure zones
      - Zone of radial shear – Zone II
      - Rankine’s Passive zone – Zone III
        - The Zones II and III are not developed at all
        - No formation of heave

![Diagram of Punching Shear Failure](image)
Bearing Capacity Failures in Sand

- Nature of failure in sand at ultimate load depends upon
  - Depth of foundation ($D_f$)
  - Width of foundation ($B$)
  - Aspect ratio of foundation ($L/B$)
  - Relative density of sand

\[ R = \frac{A}{P} \]

\[ R_{\text{Rectangular}} = \frac{(BL)}{2(B+L)} \]

\[ R_{\text{Square}} = \frac{B}{4} \]

Beyond $D_f/R ≥ 18$, Punching shear failure occurs irrespective of the relative density of sand

Same relative density of sand
- A needle will punch through
- A raft foundation will show general shear failure mode

Bearing Capacities of Shallow Foundations

- Gross allowable bearing capacity
  \[ q_{all} = \frac{q_u}{\text{FoS}} \quad \text{FoS} \rightarrow 1.5-3 \]

- Net ultimate bearing capacity
  \[ q_{nu} = q_u - q = q_u - \gamma D_f \]

- Net allowable bearing capacity
  \[ q_{na} = \frac{q_{nu}}{\text{FoS}} = \left(\frac{q_u - q}{\text{FoS}}\right) \]

- Safe bearing capacity ($q_s$)
  - The bearing stress at which the settlement of the foundation increases beyond 10% of the width of the foundation which is governed by
    - Immediate or elastic settlement (for sandy soil)
    - Primary or secondary consolidation settlement (for clayey soil)
    - Depth of water table below the foundation

- Allowable bearing capacity ($q_{all}$)
  - Lower of the magnitudes of $q_u$ and $q_s$
Terzaghi’s Bearing Capacity Theory

• Bearing capacity of shallow strip foundation
  - General Shear Failure condition

\[ q_u = cN_c + qN_q + 0.5\gamma BN_\gamma \]

\[ \Rightarrow q_u = cN_c + \gamma D_f N_q + 0.5\gamma BN_\gamma \]

- Assumptions
  - Footing
    - Shallow foundation system \( (D_f \leq B) \)
    - Rough interface between footing and foundation soil
  - Footing is rigid – does not undergo differential settlement
  - Footing considered is a continuous, strip footing
  - Foundations soil
    - Homogeneous
    - Extends to a great depth i.e. footing rests on a single stratum
    - Elastic
    - Isotropic
    - Incompressible
  - Applied Load
    - Vertical
    - Centric – is in perfect symmetry with the geometry in cross-section

Meyerhof’s Bearing Capacity Theory

• Bearing capacity of shallow strip foundation – Depth effect
  - General Shear Failure condition

\[ \phi = 0: \quad F_{cd} = 1 + 0.2 \left( \frac{D_f}{B} \right) \]

\[ F_{qd} = F_{\gamma cd} = 1 \]

\[ \phi \geq 10^\circ: \quad F_{cd} = 1 + 0.2 \left( \frac{D_f}{B} \right) \tan (45 + \phi/2) \]

\[ F_{qs} = F_{\gamma s} = 1 + 0.1 \left( \frac{D_f}{B} \right) \tan (45 + \phi/2) \]
Shape Modifications

- Shape modification factors for square and circular footings (Terzaghi 1948)
  - Square footing of side $B$ $q_u = 1.3cN_c + \gamma D_f N_q + 0.4\gamma BN_{\gamma}$
  - Circular footing of diameter $B$ $q_u = 1.3cN_c + \gamma D_f N_q + 0.3\gamma BN_{\gamma}$

- One-dimensional load dispersion vs Two-dimensional load dispersion

Terzaghi’s Proposition for Local Shear Failure

- Bearing capacity equations ($\phi < 27^\circ$, very low cohesion)
  - Strip foundation $q_u = c'N_c + \gamma D_f N_q + 0.5\gamma BN_{\gamma}$
  - Square foundation $q_u = 1.3c'N_c + \gamma D_f N_q + 0.4\gamma BN_{\gamma}$
  - Circular foundation $q_u = 1.3c'N_c + \gamma D_f N_q + 0.3\gamma BN_{\gamma}$

- $N_{c'}, N_{q'}, N_{\gamma'} \rightarrow$ Modified bearing capacity factors for local shear failure

- Reduced Strength parameters
  - To account for lesser inter-particle contact stress
  
  $c' = 0.67c, \quad \phi' = \tan^{-1}(0.67 \tan \phi)$
Effect of Water Table

- Terzaghi’s basic theory does not consider effect of water table
  - **CASE I**: Soil Fully Submerged ($d = 0$)
    \[ q_s = \gamma D_f + 0.5\gamma N_y \]
    
  - **CASE II**: Water table above the depth of embedment of footing ($0 < d \leq D_f$)
    \[ q_s = \gamma D_f + 0.5\gamma N_y \]
    
  - **CASE III**: Water table below the depth of embedment but within a depth of $B$ from the base of footing ($D_f < d \leq D_f + B$)
    \[ q_s = \gamma (D_f + B) + 0.5\gamma N_y \]

What about strength parameters $c$ and $\phi$?

Effect of Soil Compressibility

- Determination of compressibility factors
  - Incorporate the compressibility effect of soil
    - Rigidity index of the soil ($I_r$)
    - Determine the critical rigidity index of the soil [$I_{rcr}$]
  - Determination of compressibility factors
    - If $I_r \geq I_{rcr}$
      - Soil is considered incompressible
    - If $I_r < I_{rcr}$
      - Soil is considered to be compressible

\[ F_{cc} = F_{qc} = F_{cr} = 1.0 \]

\[ F_{cc} = 0.32 + 0.12 \left( \frac{B}{L} \right) + 0.6 \log I_r \quad \forall \phi = 0 \]

\[ F_{qc} = \frac{1 - F_{qc}}{N_{qc} \tan \phi} \quad \forall \phi \neq 0 \]

\[ F_{qc} = F_{qc} = \exp \left\{ -4.4 + 0.6 \left( \frac{B}{L} \right) \tan \phi + \left( \frac{3.07 \sin \phi \log (2L)}{1 + \sin \phi} \right) \right\} \]
General Bearing Capacity Equation

- Developed considering all the modification factors
  \[ q_u = cN_c F_{cs} F_{cd} F_{ci} F_{cc} + qN_q F_{qs} F_{qd} F_{qi} F_{qc} + 0.5\gamma_B N_{\gamma} F_{\gamma_d} F_{\gamma_d} F_{\gamma_i} F_{\gamma_c} \]
  - \( F_{cs}, F_{qs}, F_{s} \rightarrow \text{Shape factors} \)
    - In order to incorporate the modification on continuous, strip footing
  - \( F_{cd}, F_{qd}, F_{d} \rightarrow \text{Depth factors} \)
    - In order to incorporate the effect of embedment of footing
  - \( F_{ci}, F_{qi}, F_{i} \rightarrow \text{Inclination factors} \)
    - In order to incorporate the effect of inclination of load due to the presence of lateral thrust and imposed moments from the superstructure
  - \( F_{cc}, F_{qc}, F_{c} \rightarrow \text{Compressibility factors} \)
    - In order to incorporate the effect of compressibility characteristics of the soil

Eccentrically Loaded Continuous Footing

- Effect of eccentric loads or moments

Tension in Footing
  - May result in loss of contact with underlying soil

Zone of footing effective in providing resisting stress
Effective Footing Dimension for Eccentric Loading

- Rectangular footing of size $L \times B$
  - Ultimate bearing capacity (Meyerhof, 1953)
    \[ q_u = c N_c F_{cs} F_{cd} + q N_q F_{qs} F_{qd} + 0.5 \gamma B' N_y F_{yd} \]
  - Ultimate Load
    \[ Q_u = q_u A' = q_u (L' B') \]
  - One-way eccentricity along width ($e_L = 0$)
    \[ B' = B - 2 e_B \]
    \[ L' = L \]
    \[ A' = B' L \]
**Two Way Eccentricities**

Two-way eccentricities in foundation analysis are crucial for understanding the behavior of buildings and structures. The eccentricity ratio, \( e_{p/B} \), should be considered in the context of the dimensions of the foundation, \( L_1 \) and \( B_1 \), as well as the overall structure dimensions, \( L \) and \( B \), to ensure stability and safety.

**Bearing Capacity of Interfering Footings**

The bearing capacity of interfering footings is calculated using the formula:

\[
q_u = qN_q\zeta_q + 0.5\gamma BN\zeta\gamma
\]

Where:
- \( q_u \): bearing capacity
- \( q \): soil pressure
- \( N_q \): pile factor
- \( \zeta_q \): efficiency ratio
- \( \gamma \): unit weight
- \( B \): foundation width
- \( N \): pile
- \( \zeta\gamma \): efficiency ratio

\( \zeta_q, \zeta\gamma \) are efficiency ratios that account for the interaction between footings and their impact on the overall stability of the structure.
Bearing Capacity of Interfering Footings

- Influence of depositional activity during soil formation
  - Cohesion vary differently in different directions

\[
q_{ut} = c_V \left( N_{c(i)} + qN_q(i) \right) + 0.5\gamma BN_{y(i)}
\]

- C-soil

\[
q_{ut} = N_{c(i)} \left( c_{uV} + c_{uH} \right)
\]

- C-f soil

\[
\beta_c = \frac{\alpha' I}{c_{V(z=0)}} \quad K = \frac{c_{V(z)}}{c_{H(z)}}
\]
Bearing Capacity of Anisotropic Foundation

- Continuous strip footing resting on anisotropic sand deposit

* Equivalent soil friction angle (Meyerhof, 1978)
  - $n \rightarrow$ Friction ratio ($0 < n < 1$)
  - $\phi_1$ can be easily identified using drained triaxial test

* Ultimate bearing capacity of rectangular footing
  - $F_{qs}, F_{\gamma s} \rightarrow$ Shape factor (using $\phi_{eq}$)

\[ \phi_{eq} = \frac{2\phi_1 + \phi_2}{3} = \frac{(2 + n)\phi_1}{3}, \quad n = \frac{\phi_2}{\phi_1} \]

\[ q_u = q_N q_{eq} F_{qs} + 0.5 \gamma B N_{\gamma_s} q_{eq} F_{\gamma s} \]

Bearing Capacity: Oblique Load

- Combined action of eccentric and oblique loading
Foundations on Slopes

- Footing on the crest or face of a slope

Bearing Capacity Estimation

- Bearing capacity theories do not address deformation
  - Loading causes deformation
  - Deformation leads to new stress
  - Coupled Stress-deformation problem
Stress Distribution under External Loading

- Commonly encountered external loading
  - **Point Load**
    - Idealistic, rare occurrence, can be used to analyze any other load forms
  - **Rectangular Load**
    - Loading on rectangular footings
  - **Circular Load**
    - Loading on circular or ring foundations
  - **Line Load**
    - Loading under a railway track
  - **Strip Load**
    - Foundations of building walls, retaining walls etc.
  - **Trapezoidal and Triangular Loading**
    - Surcharge loading of an embankment
Stress Distribution at a point within the soil medium
Boussinesq Point Load Analysis: Schematic

• Assumptions
  - Point load acting at the surface
  - Soil
    - Elastic,
    - Isotropic
    - Homogeneous
    - Half-space
  - Infinitesimally small soil element

Stress Components

\[
\sigma_x = \frac{3Q yz^2}{2\pi R^5} \left[ \frac{3}{R^3} + \frac{1}{(R+z)^3} \right]
\]
\[
\tau_{xy} = \frac{3Q xz^2}{2\pi R^5} \left[ \frac{1-2v}{3} \frac{(2R+z)^2}{R^3} \right]
\]
\[
\tau_{yz} = \frac{3Q xz^2}{2\pi R^5} \left[ \frac{1-2v}{3} \frac{(2R+z)^2}{(R+z)^3} \right]
\]
\[
\tau_{xz} = \frac{3Q yz^2}{2\pi R^5} \left[ \frac{1-2v}{3} \frac{(2R+z)^2}{R^3} \right]
\]

Normal Stress Components

\[
\sigma_z = \frac{3Q yz^2}{2\pi R^5} \left[ \frac{3}{R^3} + \frac{1}{(R+z)^3} \right]
\]
\[
\sigma_x = \frac{3Q xz^2}{2\pi R^5} \left[ \frac{(2R+z)^2}{R^3} \right] - \frac{1}{R(R+z)} \left[ \frac{(2R+z)^2}{R^3} \right]
\]
\[
\sigma_y = \frac{3Q xz^2}{2\pi R^5} \left[ \frac{(2R+z)^2}{(R+z)^3} \right] - \frac{1}{R(R+z)} \left[ \frac{(2R+z)^2}{(R+z)^3} \right]
\]

Vertical stress generated at any point in the subsurface due to the point load applied at the surface

\[
\Delta \sigma_z = \frac{3Q yz^2}{2\pi R^5} - \frac{Q}{z^2} \left[ \frac{3}{2\pi} \frac{1}{\left(1 + \left( \frac{z}{R} \right)^2 \right)^{3/2}} \right]
\]

At any depth z

\[
I_g \rightarrow \text{Boussinesq Influence factor for point load surcharge at the surface}
\]
Boussinesq Multiple Point Load Analysis: 3D Schematic

• By principle of superposition

\[ \Delta \sigma_z = \frac{Q_1}{z^2} I_{B1} + \frac{Q_2}{z^2} I_{B2} + \frac{Q_3}{z^2} I_{B3} \]

Stress at any Point due to a Uniformly Loaded Rectangular Area: Schematic
Stress Beneath the Centre of a Uniformly Loaded Circular Area: Schematic

\[ \Delta \sigma_z = q \left[ 1 - \frac{1}{\left[1 + \left(\frac{R_0}{z}\right)^2\right]^{3/2}} \right] = q \times I_{\text{circular}} \]

Stress Beneath a Uniform Line Load: Schematic

\[ I_{\text{line}} = \frac{2/\pi}{\left[1 + (x/z)^2\right]^2} \]

\[ \Delta \sigma_z = \frac{q}{z} \times I_{\text{line}} \]

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

\[ \cos \theta = \frac{z}{\sqrt{x^2 + z^2}} \]
Stress Beneath a Uniform Strip Load: Schematic

- Strip load is a congregation of line loads
  - A *plane strain load*, since it extends towards infinity in longitudinal direction
  - **Difference with rectangular load**
    - Rectangular load has a definite aspect ratio (length-to-breadth ratio)
    - The aspect ratio of a strip load is infinite

\[
\Delta \sigma_z = \frac{q}{\pi} \left[ \tan^{-1} \left( \frac{z}{x-b} \right) - \tan^{-1} \left( \frac{z}{x+b} \right) \right] - \frac{2q(e^{x^2-b^2-z^2})}{(x^2-b^2+z^2)^2 + 4b^2z^2} = q \times I_{strip}
\]

Stress Beneath a Linearly Increasing Triangular Load: Schematic

- Embankment loading whose top width tends to zero

\[
\Delta \sigma_z = \frac{q}{\pi a} \left[ \tan^{-1} \left( \frac{z}{x-a} \right) - \tan^{-1} \left( \frac{z}{x} \right) \right] = q \times I_{tri}
\]
Embankment Loading: Stress at a Point Beneath Point B

\[ \Delta \sigma_z = \frac{q_0}{\pi} \left[ \frac{B_1 + B_2}{B_2} (\alpha_1 + \alpha_2) - \frac{B_1}{B_2} \alpha_2 \right] = q_0 \times I_{emb} \]

\[ \alpha_1 = \left( \tan^{-1} \frac{B_1}{z} + \tan^{-1} \frac{B_1}{z} \right) \text{ rad} \]

\[ \alpha_2 = \tan^{-1} \frac{B_1}{z} \]

- \( q_0 \) → Maximum overburden pressure on the ground surface
  \[ q_0 = \gamma H \]
  - \( \gamma \) → unit weight of the embankment soil
  - \( H \) → Height of the embankment
- \( I_{emb} \) → Stress influence coefficient for embankment loading
  \[ I_{emb} = f \left( \frac{B_1}{z}, \frac{B_2}{z} \right) \]

Embankment Loading: Stress at any Point

Method of Superposition

- Embankment loading is considered as a combination of two linearly increasing triangular loads and a central strip load
  - Stress at a point is determined by the principle of superposition of the effect of combined loads
Approximate Method of Stress Estimation: Boussinesq Point Load Estimation

- Applicable for irregularly loaded profile in magnitude

Isobars and Pressure Bulbs

- Isobars
  - Line joining points of equal stress
    - Alternative can be termed as a ‘stress contour’

  - Any number of isobars can be constructed for a given system
    - Depends upon the stress level for which the points are to be identified

- Pressure bulbs
  - Congregation of isobars
  - Extremely important with regard to settlement analysis of foundations
Pressure Bulb for Point Load and Significant Depth

- Correlation between the parameters ‘r’ and ‘z’ to obtain a particular ratio of excess stress

\[
\Delta \sigma_z = \frac{Q}{z^2} \left[ \frac{3}{2\pi \left[ 1 + (r/z)^2 \right]^{5/2}} \right]
\]

\[
\frac{\Delta \sigma_z}{Q} = p_{iso} = \frac{1}{z^2} \left[ \frac{3}{2\pi \left[ 1 + (r/z)^2 \right]^{5/2}} \right]
\]

\[r = \pm \sqrt[5/2]{\frac{3}{2\pi \cdot p_{iso}}} - \frac{z^2}{5/2}\]

Circular Load: Isobar and Significant Depth

- Depending on the importance of settlement calculation on the response of the structure, the significant depth will vary

<table>
<thead>
<tr>
<th>(\Delta \sigma_z/q)</th>
<th>(z/R_0)</th>
<th>(z_{sig})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>(\approx 2.5)</td>
<td>1.25 (x (2R_0))</td>
</tr>
<tr>
<td>0.1</td>
<td>(\approx 3.8)</td>
<td>1.9 (x (2R_0))</td>
</tr>
<tr>
<td>0.05</td>
<td>(\approx 5.5)</td>
<td>2.75 (x (2R_0))</td>
</tr>
</tbody>
</table>
Square and Strip Load: Isobar and Significant Depth

- Depending on the importance of settlement calculation on the response of the structure, the significant depth will vary

  - **Square footing**

<table>
<thead>
<tr>
<th>( \Delta \sigma/q )</th>
<th>( z/b )</th>
<th>( Z_{sig} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>( \approx 3 )</td>
<td>1.5 x (2b)</td>
</tr>
<tr>
<td>0.1</td>
<td>( \approx 4.5 )</td>
<td>2.25 x (2b)</td>
</tr>
<tr>
<td>0.05</td>
<td>( \approx 6.5 )</td>
<td>3.25 x (2b)</td>
</tr>
</tbody>
</table>

  - **Strip Footing**

<table>
<thead>
<tr>
<th>( \Delta \sigma/q )</th>
<th>( z/B )</th>
<th>( Z_{sig} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>( \approx 2.5 )</td>
<td>2.5 x (B)</td>
</tr>
<tr>
<td>0.1</td>
<td>( \approx 3.75 )</td>
<td>3.75 x (B)</td>
</tr>
<tr>
<td>0.05</td>
<td>( \approx 5.5 )</td>
<td>5.5 x (B)</td>
</tr>
</tbody>
</table>

Pressure Bulb for Point Load and Significant Depth

- **Significant depth**

  - The depth of foundation soil beneath the load which is most effective in contributing towards the settlement of the superstructure
  - Related to the vertical influence zone of the pressure bulb

- **Quantification of significant depth**

  - The depth beyond which the excess stress generated due to the external load becomes lower than a specific value
  - Magnitude of the limit of the excess stress
    - Depends on the necessity of the problem and importance of structure
      - Structures meant for nuclear plants have to be considered for low magnitude of excess stress
    - Depends on the type of footing
Pressure Bulb for Point Load and Significant Depth

- Terzaghi’s proposition for significant depth
  - For square and circular footings
    - Excess stress level considered \( \Delta \sigma_z/q = 0.2 \)
  - Significant depth generally considered under such condition
    - \( z_{slg} \approx 1.5B \)
      - \( B \) → Width of the footing
  - Similar observation is made even for point load systems

---

Terzaghi’s Proposition of Significant Depth

- Significant depth considering excess stress ratio of 0.2
  - Single isolated footings \( \Rightarrow 1.5B \)
    - \( B \) → Width of the single footing
  - Multiple footings placed very close to each other \( \Rightarrow 1.5B \)
    - \( B \) → Width of the footing system taking into account all the adjacent footings
Isobars and Interference of Footing

- Lateral influence of isobars
  - *Governed by the prohibition of interference of footings when placed adjacent to each other*
    - Accounts for the determination of the clear space that should be provided between the footings placed adjacent to each other in order to minimize interference.

- Circular footing
  - \(\frac{\Delta \sigma_z}{q} = 0.1\)
    - \(\frac{r}{R_0} \approx 1.75\), or \(r \approx 1.75R_0\)
    - Clear space required between two closely spaced circular footings to prevent interference:
      - \(2 \ast (1.75 - 1)R_0) = 1.75R_0\)

- Square footing
  - \(\frac{\Delta \sigma_z}{q} = 0.1\)
    - \(\frac{x}{b} \approx 2\), or \(x \approx 2b (=B)\)
    - Clear space required between two closely spaced square footings to prevent interference:
      - \(2 \ast \left(\frac{B-B}{2}\right) = B\)

- Strip footing
  - \(\frac{\Delta \sigma_z}{q} = 0.1\)
    - \(\frac{x}{B} \approx 1.5\), or \(x \approx 1.5B\)
    - Clear space required between two closely spaced strip footings to prevent interference:
      - \(2 \ast \left(1.5B-B/2\right) = 2B\)
Interfering Footings

Contact Stresses
Rigid and Flexible Foundations
Contact Stress below Rigid and Flexible Foundations

- Rigid and Flexible foundations
  - Basic difference rest in the contact pressure and settlement profiles

- Rigid Foundations/Footing
  - High resistance to bending deformation
  - Uniform settlement
  - Non-uniform contact stress distribution

- Flexible Foundations/Footing
  - Can bend i.e. have bending stiffness
  - Footing settlement is non-uniform
  - Contact stress profile is uniform

Uniformly Loaded Rigid Footing on Sand

- The rigid footing is acted upon by uniformly distributed load
  - The settlement profile is uniform
    - The footing is sufficiently rigid to resist bending
  - Contact stress profile is non-uniform
    - Effective contact stress profile
    - At the very instant of loading, the pore pressure rises (if the soil is saturated)
      - Due to high permeability of the material, the pore pressure dissipates immediately, and the stresses are taken up by the soil particles
      - As per the stress distribution profiles, the central part of the footing experiences maximum stress while the edges experience minimum stress
    - Each cross-section of the footing suffers stress imbalance
Uniformly Loaded Rigid Footing on Clay

- A rigid footing resting on the semi-infinite soil mass is acted upon by uniformly distributed load
  - **Saturated cohesive soil**
    - Pore pressure conditions just after loading
      - Maximum pore pressure at the centre and minimum pore pressure at the edges of the footing
  - To determine the effective contact stress profile
    - Subtract the pore pressures from the total pressures generated
  - The effective contact stress profile
    - Maximum contact stress at the edges
    - Minimum contact stress at the centre
      - Each c/s of footing experiences force imbalance

Uniformly Loaded Flexible Footing on Sand

- A flexible footing resting on the semi-infinite soil mass is acted upon by uniformly distributed load
  - **Cohesionless soil**
  - The footing is allowed to bend affecting the surface settlement of footing
    - The flexible footing will continue to bend till the force equilibrium is reached at each of the c/s
      - Maximum positive imbalance is at the edges, while the centre suffer maximum negative imbalance
      - Settlement profile of the flexible footing
        - Maximum at the edges
        - Minimum at the centre
      - Contact stress profile of the footing
        - Uniform along the length of the footing
Uniformly Loaded Flexible Footing on Clay

- A flexible footing resting on the semi-infinite soil mass is acted upon by uniformly distributed load
  - Saturated cohesive soil
  - The footing is allowed to bend affecting the surface settlement of footing
    - The flexible footing will continue to bend till the force equilibrium is reached at each of the c/s
      - Maximum positive imbalance is at the centre, while the edges suffer maximum negative imbalance
    - Settlement profile of the flexible footing
      - Maximum at the centre
      - Minimum at the edges
    - Contact stress profile of the footing
      - Uniform along the length of the footing

Factors Affecting Contact Stress Distributions

- External load distribution
  - Depending on the external load distribution, the contact stress distribution will change
Contact Stress and Deformation Pattern in Combined Footing

- Both transverse and longitudinal bending

- Moments in transverse moments
  - Only sagging

- Moments in longitudinal direction
  - Sagging beneath columns
  - Hogging between columns

Contact Stress beneath Combined Trapezoidal Footing

- Net upward soil pressure
  \[ p_0 = \frac{P_{eq}}{0.5 (B_1 + B_2) L} \]

- To determine the depth
  - Proceed in the same way as done for rectangular footing

- Points to be noted
  - Longitudinal direction
    - Uniformly distributed load is no more linear
      - Trapezoidal
  - Transverse direction
    - Each of the column have to be checked for moments and shear
Strap Footing: Proportioning

- Size of columns
  - Column A: $a_1 \times b_1$
  - Column B: $a_2 \times b_2$

- Column Loads $P_1, P_2$

- Weight of footing + strap beam
  - $W_f = 0.1(P_1 + P_2)$

- Area of footing required
  - $A_1 + A_2 = \frac{P_1 + P_2 + W_f}{q_{na}}$
  - $B(L_1 + L_2) = \frac{P_1 + P_2 + W_f}{q_{na}}$

Strap Beam: Design

- Depth of strap beam
  - Critical moment
    - Maximum moment???
    - Critical section lies at the edge of the footing
      - T-beam action gets converted to a normal hanging beam
      - Check the maximum moment at the footing edges
  - Thickness of strap beam
    - $d_s = \sqrt{\frac{M_{\text{max-strap}}}{R_c \times b_s}}$
    - $b_s \rightarrow$ width of strap beam
    - For stringent restriction on $d$, use larger width of strap beam
Contact Stress beneath Flexible Mats

Beams on Elastic Foundations

- Varying Load
- Varying Subgrade Reaction
Types of Settlement

- Uniform settlement
  - Structure built over very rigid mat and structural loads evenly distributed

- Uniform tilt / Rigid rotation
  - Structure built over rigid mat and structural load distribution is skewed

- Differential non-uniform settlement
  - Structure built on flexible foundation

- Angular distortion ($\Delta$)
  \[ \Delta = \frac{(S_{\text{max}} - S_{\text{min}})}{L} \]
### Classification of Settlement based on Time

- **Immediate or Elastic Settlement**
  - Effective for any type of soil
  - Instantaneous with the application of load

- **Time-dependent Consolidation Settlement**
  - Effective for low permeable soils
  - Due to gradual dissipation of excess pore-pressures developed due to the imposed load
    - Results in the decrease in voids and permanent settlement of the structure

- **Permanent settlement in sand**
  - Aging of sand – Similar to consolidation in clays
  - Reconstruction of voids and rearrangement of the soil particles in a more denser state

### Elastic Parameters of Soil – Immediate Settlement

- **Elastic parameters of soil**
  - Helps in the estimation of immediate settlement of structures
  - Modulus of elasticity \((E_s)\)
    - Range for sand: 5 – 50 MPa
    - Range for clay: 0.1 – 200 MPa
    - Compressible clays: 0.1 – 5 MPa
  - Poisson’s Ratio \((\nu)\)
    - Range for various soils: 0.1 – 0.5

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Poisson’s ratio, (\nu)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Course sand</td>
<td>0.15 – 0.20</td>
</tr>
<tr>
<td>Medium loose sand</td>
<td>0.20 – 0.25</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.25 – 0.30</td>
</tr>
<tr>
<td>Sandy silt and silt</td>
<td>0.36 – 0.35</td>
</tr>
<tr>
<td>Saturated clay (undrained)</td>
<td>0.50</td>
</tr>
<tr>
<td>Saturated clay—lightly overconsolidated (drained)</td>
<td>0.2 – 0.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type</th>
<th>(E_s) (kN / m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse and medium course sand</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>25,000 – 35,000</td>
</tr>
<tr>
<td>Medium dense</td>
<td>30,000 – 40,000</td>
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<tr>
<td>Dense</td>
<td>40,000 – 45,000</td>
</tr>
<tr>
<td>Fine sand</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>20,000 – 25,000</td>
</tr>
<tr>
<td>Medium dense</td>
<td>25,000 – 35,000</td>
</tr>
<tr>
<td>Dense</td>
<td>35,000 – 40,000</td>
</tr>
<tr>
<td>Sandy silt</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>8,000 – 12,000</td>
</tr>
<tr>
<td>Medium dense</td>
<td>10,000 – 12,000</td>
</tr>
<tr>
<td>Dense</td>
<td>12,000 – 15,000</td>
</tr>
</tbody>
</table>
Steinbrenner Analysis: Settlement of Circular Footing

- Immediate settlement beneath a uniformly loaded flexible surface circular footing \((q\) per unit area)
  - At the surface contact beneath the footing \((z/R=0)\)
    \[ S_{e|z/R=0} = q \left( \frac{1-v^2}{E_s} \right) R \]
  - At the centre of footing \((r/R=0)\): \(I_z = 2\)
  - At the edge of footing \((r/R=0)\): \(I_z = 1.273\)

\[
S_{e|\text{centre}, z/R=0} = 2q \frac{1-v^2}{E_s} \frac{R}{E_s} = qB \left(1-v^2\right) \\
S_{e|\text{edge}, z/R=0} = 1.273q \frac{1-v^2}{E_s} \frac{R}{E_s} = 0.636qB \left(1-v^2\right)
\]

- \(B\) ➔ Diameter of the loaded area (=2R)

Steinbrenner Analysis: Circular Footing

- Immediate settlement beneath a uniformly loaded flexible surface circular footing \((q\) per unit area)
  - Average settlement at surface for flexible footing
    \[
    S_{e|\text{avg}, z/R=0} = \frac{1}{B} \int_{-B/2}^{+B/2} S_e dx \approx 0.85 S_{e|\text{centre}, z/R=0} \\
    \Rightarrow S_{e|\text{avg}, z/R=0} = 0.85qB \left(1-v^2\right) / E_s
    \]
  - Considering the effect of rigidity of the surface footing
    \[
    S_{e|\text{avg}, \text{rigid}} \approx 0.93 S_{e|\text{avg}, \text{flexible}} \\
    \Rightarrow S_{e|\text{avg}, \text{rigid}} = \frac{0.79qB \left(1-v^2\right)}{E_s}
    \]
Steinbrenner Analysis: Rectangular Footing

- Immediate settlement beneath a uniformly loaded flexible surface rectangular footing \((q \text{ per unit area})\)
  - Obtained from the integration of vertical deformations calculated using the vertical strains developed at each point
  - Settlement at the corner of the rectangular footing at a point located at any depth ‘\(z\)’ beneath the footing

\[
S_{\text{corner}} = \frac{qB}{2E_s} \left(1 - \nu^2\right) \left[I_3 - \frac{1 - 2\nu}{1 - \nu} I_4\right]
\]

- \(I_3, I_4\) → Steinbrenner’s Influence factor for uniformly loaded rectangular footing

\[
I_3, I_4 = f\left(m', n'\right), \quad m' = L/B, \quad n' = z/B
\]

- Tables to be provided for \(I_3\) and \(I_4\)
  - \(L, B\) → Length and breadth of the footing

---

Steinbrenner Analysis: Rectangular Footing

- Immediate settlement beneath a uniformly loaded flexible surface rectangular footing \((q \text{ per unit area})\)
  - At the surface contact beneath the footing \((n' = z/B = 0)\), \(I_4 = 0\)

\[
S_{\text{corner, } n'=0} = \frac{qB}{2E_s} \left(1 - \nu^2\right) I_3
\]

\[
S_{\text{centre, } n'=0} = \frac{qB}{E_s} \left(1 - \nu^2\right) I_3
\]

- Average settlement at any depth for flexible footing

\[
S_{\text{avg}} \approx 0.85 S_{\text{centre}}
\]

- Average settlement of a rigid footing

\[
S_{\text{avg, rigid}} \approx 0.93 S_{\text{avg, flex}}
\]
Effect of Rigid Base within Significant Depth

- Egorov’s solution for rectangular footing \((L \times B)\)
  - Consideration of the presence of a rigid base within the zone of significant depth of the footing
    - \(a \rightarrow\) Egorov’s influence factors \([f(L/B, h/B)]\):
      - \(h \rightarrow\) Depth of rigid base from the base of the footing
  - Circular footing
    \[
    S_e|_{centre, flexible} = \frac{Rq}{E_s} \left(1 - \nu^2\right) a_1 \\
    S_e|_{centre, rigid} = \frac{Rq}{E_s} \left(1 - \nu^2\right) a_2
    \]
  - Rectangular footing
    \[
    S_e|_{centre, flexible} = \frac{Bq}{E_s} \left(1 - \nu^2\right) a_3 \\
    S_e|_{centre, rigid} = \frac{Bq}{E_s} \left(1 - \nu^2\right) a_4
    \]

Effect of Depth of Embedment of Foundation

- Fox’s solution for rectangular footing \((L \times B)\)
  - Consideration of the depth of embedment of the footing
    - \(I_f \rightarrow\) Fox’s influence factors \([f(D_f/B, \nu, h/B)]\):
      - \(D_f \rightarrow\) Depth of embedment of the footing
    - Estimation of \(S_e|_{D_f=0}\)
      - Use Steinbrenner’s solution if the soil is extending to a great depth
      - Use Egorov’s solution if affected by a rigid base with significant depth
Effect of Stratified Soil Deposit

- Bowles’ proposition
  - **Concept of Equivalent Elastic Modulus** \( E_{sq} \)
    \[
    E_{sq} = \frac{\sum (E_z \Delta z)}{z}
    \]
    - \( E_z \) → Modulus of elasticity of each layer
    - \( \Delta z \) → Thickness of each stratum
    - \( \frac{1}{z} \) → \( h \) (Depth of rigid base) or \( 2B \) (Significant depth of footing), whichever is smaller

- For bearing capacity of stratified deposits
  - **Equivalent cohesion and angle of internal friction**
    \[
    c_{eq} = \frac{\sum (c_z \Delta z)}{z} \quad \phi_{eq} = \tan^{-1} \left( \frac{\sum (\Delta z \tan \phi_z)}{z} \right)
    \]

Consolidation Settlement

- Consolidation settlement
  - Related to the permeability and compressibility characteristics of the soil

- Components of consolidation settlement
  - **Primary consolidation and settlement**
    - Expulsion of water present in the voids of the saturated soil mass
      - Continual change of effective stress with time
      - Accounts for nearly 90% of consolidation settlement
  - **Secondary consolidation and settlement**
    - Mostly due to rearrangement of particles under nearly constant effective stress (Conversion from Flocculated to Dispersed structures)
      - Occurs in moderate to highly-plastic clays
  - **Tertiary consolidation and settlement**
    - Mostly due to re-structuring of the particles (Negligibly low magnitude)
      - Occurs in clayey soils whose particles are elongated and flaky
**Flocculated and Dispersed Structures**

- **Flocculated Structure**
  - Random orientation of particles
  - Edge-to-Face contact of particles
  - Cross-permeability or Anisotropic permeability is nearly of same value
  - Can be created by random remolding of soils

- **Dispersed Structure**
  - Particles oriented in a bedding place
  - Face-to-Face contact of particles
  - In-plane permeability is always greater than the cross-plane permeability
  - Can be created by uniform compaction of soil in very thin layers

- **Identification of flocculated and dispersed structure**
  - Carry out anisotropic permeability test
    - Determine if the cross-permeabilities are significantly different

---

**Oedometer Test and Preconsolidation Pressure**

![Oedometer Test Diagram](image)

- Effective stress ($\sigma'$)
- Void Ratio ($\varepsilon$)
- Point of maximum curvature
- Horizontal line from point of maximum curvature
- Bisector at the point of maximum curvature
- Tangent line from point of maximum curvature
- Compression Index $\Rightarrow$ Tangent to the compression line (Slope in log scale=$C'_c$)
Compression and Swelling Index of Soils

- Normally consolidated clay
  - The present state of stress is the maximum in its stress history \( \sigma' \geq \sigma_c' \)
  - \( \sigma' \rightarrow \) Preconsolidation stress

- Compression index \( (C_c) \)
  - Slope of \( e - \log \sigma' \) curve

\[
C_c = \frac{e_1 - e_2}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right) - \log\left(\frac{\sigma'_1}{\sigma'_1}\right)} = \frac{\Delta e}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right) - \log\left(\frac{\sigma'_1}{\sigma'_1}\right)}
\]

- Overconsolidated clay
  - The present state of stress is the lower than the maximum pressure noted in its stress history

\( \sigma < \sigma_c' \)

- Swelling index \( (C_s) \)
  - Slope of \( e - \log \sigma' \) curve

\[
C_s = \frac{e_3 - e_4}{\log\left(\frac{\sigma'_4}{\sigma'_3}\right) - \log\left(\frac{\sigma'_3}{\sigma'_3}\right)} = \frac{\Delta e}{\log\left(\frac{\sigma'_4}{\sigma'_3}\right) - \log\left(\frac{\sigma'_3}{\sigma'_3}\right)}
\]
Compression and Swelling Index: Empirical Relations

- In absence with proper consolidation data
  - Compression Index based on regression analysis of experimental data
    - For clays with sensitivity ratio greater than 10: \( C_c = 0.009(LL - 10) \)
    - For remolded clays: \( C_c = 0.007(LL - 10) \)
    - Chicago clays: \( C_c = 0.01w_n \)
    - Brazilian clay: \( C_c = 0.0046(LL - 9) \)
    - Motley clay: \( C_c = 1.21 + 1.055(e_0 - 1.87) \)
      - \( LL \rightarrow \) Liquid limit of the soil
      - \( w_n \rightarrow \) Natural moisture content of the soil
      - \( e_0 \rightarrow \) Initial void ratio of the soil specimen
  - Swelling Index: \( C_s = \left(\frac{1}{4} \text{ to } \frac{1}{3}\right)C_c \)

Estimation of Primary Consolidation Settlement

- Normally Consolidated (NC) clays
  \[
  S_c = \frac{C_c H_c}{1 + e_0} \log \left(\frac{\sigma_0 + \Delta \sigma_{av}}{\sigma_0}\right) \quad \forall \sigma_0 > \sigma_c
  \]

- Overconsolidated (OC) clays
  \[
  S_c = \frac{C_s H_c}{1 + e_0} \log \left(\frac{\sigma_0 + \Delta \sigma_{av}}{\sigma_0}\right) \quad \forall \left(\sigma_0 + \Delta \sigma_{av}\right) < \sigma_c
  \]
  \[
  S_c = \frac{C_s H_c}{1 + e_0} \log \left(\frac{\sigma_0 + \Delta \sigma_{av}}{\sigma_c}\right) \quad \forall \sigma_0 < \sigma_c < \left(\sigma_0 + \Delta \sigma_{av}\right)
  \]

- \( H_c \rightarrow \) Thickness of the clay layer
- \( e_0 \rightarrow \) Initial void ratio
- \( \sigma_0 \rightarrow \) Average effective in-situ stress at the centre of clay layer
- \( \Delta \sigma_{av} \rightarrow \) Increase in average effective stress layer due to external load
**Estimation of Δσ_{av}: Simpson’s Method**

- Should not be estimated directly at the centre of the clay layer
  - Variation of the increase in effective stress due to external load is not linear along a thick clay layer

- Simpson’s method to determine Δσ_{av}
  \[ \Delta \sigma_{av} = \frac{1}{6} [\Delta \sigma_t + 4 \Delta \sigma_m + \Delta \sigma_b] \]
  - Δσ_t, Δσ_m, Δσ_b → Increase in effective stress at the top, middle and bottom of clay layer
    - To be calculated based on the methods of stress distribution
      - For quick results, 2:1 distribution method can be suitably followed

**Approximate Method of Stress Estimation: 2:1 Distribution of Stress**

- Stress distributions by 2:1 method
  1. *Distribution in case of plane-strain loads*
    - Any load bounded by a single lateral dimension
      \[ \Delta \sigma_z = \frac{Bq}{(B + z)} = \frac{Q}{(B + z)} \]
  2. *Distribution in case of two-dimensional loads*
    - Any load bounded by two lateral dimensions
      \[ \Delta \sigma_z = \frac{B.Lq}{(B + z)(L + z)} = \frac{Q}{(B + z)(L + z)} \]
Estimation of $\Delta \sigma_{av}$: Method of Thin Strata

- Divide the entire clay layer into several thin strata
  - Preferably 1m or 0.5m
  - Determine the in-situ overburden stress and average increase of stress at the centre of each strata
    - Average increase of stress variation is now considered to be piecewise linear for each thin layer
  - Sum up the settlements of each thin layers to obtain the settlement of the total layer

$$S_c = \sum_{j=1}^{n} C_i H_{ci} \log \left( \frac{\sigma_{0j} + \Delta \sigma_{avi}}{\sigma_{0i}} \right)$$

Secondary Consolidation Settlement (Creep)

- Secondary compression index
  $$C_\alpha = \frac{\Delta e}{\log \left( t_2/t_1 \right)}$$
  - $C_\alpha > 0.001$ (Clays with OCR>2)
  - $C_\alpha > 0.025$ (Organic clays)
  - $C_\alpha = (0.004-0.025)$ (NC clays)

- Secondary Consolidation Settlement
  $$S_S = \frac{C_\alpha H_C \log \left( \frac{t_2}{t_1} \right)}{1 + e_p}$$
  - $e_p \rightarrow$ Void ratio at the end of primary consolidation
  - $t_1, t_2 \rightarrow$ Rests on the straight line portion of the $e$-$\log t$ curve
Total Settlement

- Total settlement \( S = S_e + S_c + S_s \)
  - Comprises the individual contribution of
    - Immediate or Elastic settlement
    - Primary consolidation settlement
    - Secondary consolidation settlement

- Actual consolidation settlement in the lab/field in a specific time will be less than that estimated – Why??
  - Void ratio, permeability and compressibility decreases with the increase in pressure during consolidation

Pile Foundations
Pile Foundations

- Superstructure is located on a top soil which is loose, soft or swelling
  - Resistance is obtained by Bearing and Friction/Adhesion

- The underlying hard stratum has a very high bearing strength
  - Resistance of mainly attained by Bearing
  - Relative contribution of adhesion/friction of overlying soil is sufficiently low

- The rigid stratum is located at a very large depth
  - Resistance is primarily obtained from the Friction/Adhesion of the overlying soil

Piles are long slender structures that transfer the load to a hard bearing stratum – Are they?

Necessity of Pile Foundations

- Resist against horizontal forces due to wind or earthquake (e.g. high retaining walls)
- Resist uplift of transmission towers and offshore platforms
- Avoid zone of erosion for bridge piers and abutments
- Avoid zones of possible liquefaction
- Avoid zones of swelling or collapsible soils
Classification of Piles: Slenderness Ratio

- Slenderness Ratio
  - Classifies into Long and Short piles
  - The ratio of length-to-diameter of pile (L/d)

  - Long/Flexible piles (L/d > 10)
    - Undergoes bending and behaves as a beam buckled at a hinge
      - Beyond a particular depth, the length of a pile loses its significance under lateral loads
      - The frictional resistance generated at the sides of the pile provides a significant contribution to the total load resistance of the pile

  - Short/Rigid piles (L/d ≤ 10)
    - Behaves as a rigid body and rotates as a unit under lateral load
      - Total resistance is primarily contributed by the bearing resistance at the tip of the pile

Classification of Piles

- Orientation
  - Vertical or Inclined Piles

- Composition
  - Timber, Concrete or Steel

- Installation type
  - Driven or Cast-in-situ

- Special Piles
  - Compaction piles
  - Underreamed piles
  - Franki Piles

- Load carrying technique
  - End-bearing or Friction

Shallow and Deep Foundations
Compaction Piles

- Compaction Piles in cohesionless soil

  - A special case of driven piles
    - Compaction pile in cohesionless soil
      - When driven in sand or granular soil, volume of soil equal to that of the soil gets displaced and enters the voids of the adjacent mass
      - Leads to the densification of the soil
        - In turn, increases the bearing capacity of the soil
  - Cylindrical water tank to be rested on piles in cohesionless soil
    - Driving technique: ‘Driving outwards’ or ‘Driving inwards’
      - Driving of each pile leads to the densification of adjacent soil mass
      - Moving from periphery towards inner core results in progressing densification of the inner core
        - Driving the central piles require huge amount of energy impact
        - Use of very high energy may result in the undesirable uplift of the already driven adjacent piles
    - DRIVE OUTWARDS

- Compaction pile in cohesive soil

  - Poor drainage quality of cohesive soil does not allow free displacement of the driven soil during pile driving
    - Results in the rise of the pore pressure, reduction of the effective stress and the bearing capacity of the soil
      - Use of excessive energy during driving may even result in local liquefaction or dynamic mobilization of soil
    - Immediate effect of driving in cohesive soil results in the temporary loss of strength of the soil
      - Soil gets remolded and loses its structural strength to a certain extent
  - Driving in cohesive soil
    - Plan the pile driving operation
      - After each pile is driven, give ample time to allow the pore water to dissipate
        - Time taking process
      - Alternate solution
        - Application of geosynthetic encased stone columns for release of immediate pore-water pressure
Ultimate Bearing Capacity of a Single Vertical Pile

- Ultimate load on a pile
  - Single circular pile of uniform diameter ‘D’ and length ‘L’ driven into homogeneous soil of known physical properties
  - Static vertical load
  - Ultimate load on pile

\[ Q_u = Q_b + Q_f = q_b A_b + f_s A_s \]

- \( Q_b \rightarrow \) Ultimate base load or tip load
- \( Q_f \rightarrow \) Ultimate friction load or skin load
- \( q_b \rightarrow \) ultimate bearing capacity of the pile tip
- \( A_b \rightarrow \) Bearing area of the pile tip
- \( A_s \rightarrow \) surface area of pile embedded below surface
- \( f_s \rightarrow \) ultimate unit skin friction

Load Settlement Curve

- Pile is loaded to failure by gradually increasing the vertical load at the pile head
  - Settlement at the head is measured after each load increment and attainment of subsequent equilibrium
**Load Transfer Mechanism in a Pile**

- Skin and tip do not mobilize simultaneously
  - Gradual mobilization of skin resistance
  - Skin resistance gets completely mobilized before tip resistance

**Failure Load, Working Load and FoS**

- Failure load
  - In bearing → Obtained from the load-settlement curves
  - In settlement → 10% of the pile width or pile diameter

- Allowable or Working load
  - Normally a FoS=2.5 is used
  - If the tip resistance and skin friction can be determined independently

- FoS of bearing resistance is higher than skin friction since much higher settlement is necessary to fully mobilize the tip resistance
Types of Pile Failure Conditions

- Governed by relative stiffness of pile and soil
  - Buckling failure
  - General shear failure
  - Local shear failure
  - Punching shear failure
  - Uplift failure

Assumed Failure Surfaces

- Terzaghi’s assumption – Similar to shallow foundations
- Meyerhof’s assumption – Failure surfaces revert back to pile shaft
- Vesic’s assumption – Failure surfaces do not revert back to pile shaft (Experimentally verified)
Bearing Capacity of Pile Foundations

Estimation of Ultimate Bearing Capacity

- Various techniques available to estimate ultimate bearing capacity of pile foundations
  
  - Estimation from Static bearing capacity equations
  
  - Estimation from SPT and CPT correlations
  
  - Estimation from Field load tests
  
  - Estimation from Dynamic methods
Vesic’s Proposition of Ultimate Load Capacity of Pile

- Pile in cohesionless soil
  \[ Q_b = \left[ cN_c + q_0N_q + 0.5\gamma dN_f \right]A_b \]
  - \( c = 0 \); and the third term becomes insignificant in comparison to the overburden pressure for deep foundations

- Ultimate Base resistance
  \[ Q_b = q_0N_qA_b = q_bA_b \]

- Ultimate Skin Friction
  \[ Q_f = A_s q_0 \bar{K}_s \tan \delta \]
  - \( A_s \rightarrow \) Surface area of the embedded length of the pile
  - \( q_0 \rightarrow \) Average effective overburden pressure over the embedded depth of the pile
  - \( \bar{K}_s \rightarrow \) Average lateral earth pressure coefficient
  - \( \delta \rightarrow \) angle of wall friction

- Ultimate load capacity of pile
  \[ Q_u = Q_b + Q_f = q_0N_qA_b + A_s q_0 \bar{K}_s \tan \delta \]

---

Vesic’s Proposition of Ultimate Load Capacity of Pile

- Pile in cohesive soil
  \[ Q_b = \left[ cN_c + q_0N_q + 0.5\gamma dN_f \right]A_b \]
  - \( \phi = 0 \rightarrow N_c=9, N_q=1, N_f=0 \)
  - The third term becomes insignificant in comparison to the overburden pressure for deep foundations

- Ultimate Base resistance
  \[ Q_b = c_bN_c + q_0A_b \]

- Ultimate Skin Friction
  \[ Q_f = A_\alpha c_u \]
  - \( c_b \rightarrow \) Undrained shear strength of clay at the base level
  - \( c_u \rightarrow \) Average undrained shear strength of clay over the embedded depth of the pile
  - \( \alpha \rightarrow \) Adhesion Factor

- Ultimate load of pile
  \[ Q_u = Q_b + Q_f = 9c_b + q_0A_b + A_\alpha c_u \]
Net Ultimate Load Capacity of Pile

- Piles in cohesionless soils
  \[ Q_{nu} = Q_u - W_p = q_0 N_q A_b + A_q q_0 K_z \tan \delta - W_p \]
  
  \[ W_p \approx q_0 A_b \]
  
  \[ Q_{nu} = q_0 \left(N_q - 1\right) A_b + A_q q_0 K_z \tan \delta \]

- Piles in cohesive soils
  \[ Q_{nu} = Q_u - W_p = \left[q_c b + q_0 A_b + A_q \alpha \sigma_u \right] A_b - W_p \]
  
  \[ Q_{nu} = q_c b A_b + A_q \alpha \sigma_u \]

Effect of Pile Installation on \( \phi \)

- Pile driven into loose sand
  - Densification of soil due to compaction
  - Width of compacted zone – 6 to 8 times pile diameter

  - Kishida’s Proposition
    - \( \phi \) decreases from a maximum value \( \phi_2 \) at the centre of the pile to a value of \( \phi_1 \) at a distance of 3.5 times the pile diameter from the centre
    - \( \phi_2 \rightarrow \) Angle of internal friction before the installation of pile
    - \( \phi_2 \rightarrow \) Angle of internal friction after the installation of the pile at pile tip
    - \[ \phi_2 = \frac{\phi_1 + 40^\circ}{2} \]
      
      - No extra densification if \( \phi_2 = 40^\circ \)
Effect of Pile Installation on $\phi$

- Pile driven into loose sand
  - Relationship between SPT value and $\phi_1$
    \[ \phi_1 = \sqrt{20(N_1)_{60}} + 15^\circ \]
  - $N_{60}$ value corrected for overburden pressure
- Pile bored into dense sand
  - Loosening of the sand due to boring and subsequent extraction of the soil
    - Reduction of the effective angle of internal friction
      \[ \phi_2 = \phi_1 - 3 \]

NB: Always use $\phi_i$ in absence of any specific instruction

Effect of Pile Installation

- Influence on excess pore pressure generated
  - Driven pile shows higher pore pressure due to impact loading
**Effect of Pile Installation**

- Influence on relative shear stress generated
  - Higher shear stress indicate more compaction and densification

**Pile Bearing Capacity from SPT Results**

- Meyerhof’s proposition for piles in cohesionless soil
  - **Piles driven in sand**
    
    \[ Q_u = Q_b + Q_f = \min \left( 40 \frac{N_1}{60} \frac{L}{d}, 400 \frac{N_1}{60} \right) A_b + 2 \frac{N_1}{60} A_s \]
  
  - **Piles bored in sand**
    
    \[ Q_u = Q_b + Q_f = 133 \frac{N_1}{60} A_b + 0.67 \frac{N_1}{60} A_s \]
  
    - \( Q_u \) → Ultimate load in kN
    - \( (N_1)_{60} \) → Corrected SPT value below pile tip
    - \( (N_1)_{60} \) → Average corrected SPT value along the pile shaft
    - \( A_b, A_s \) → Area of pile base and shaft in m²

- **Allowable load capacity of pile**
  
  \[ Q_a = \frac{Q_u}{FoS_{\text{min}}} = 4 \]
Pile Bearing Capacity from CPT Results

- Vander Veen’s proposition for piles in cohesionless soil
  - **Ultimate end bearing resistance of the pile is equal to the point resistance of cone**
    - Considering the variation of cone resistance during driving
      - Average of the cone resistance over a depth equal to 3 times the pile diameter above pile tip and 1 time the pile diameter below pile tip
        \[ Q_b = q_b A_b = q_p A_p \]
      - \( q_b \) → Unit base resistance of the pile tip
      - \( Q_b \) → Base resistance of the pile tip
      - \( A_b \) → Area of the pile base
      - \( Q_a \) → Allowable bearing resistance of pile tip
  - **Ultimate skin resistance**
    - \( f_s \) → Unit skin friction
    - \( q_c \) → Average skin friction along the pile shaft

\[ Q_s = f_s A_s = \left[ \min \left( \frac{q_c}{2}, 54 \text{kPa} \right) \right] A_s \]

- Schmertmann’s proposition for piles in cohesive soil
  - **Unit end bearing resistance of the pile**
    \[ q_p = \left( \frac{q_{c1} + q_{c2}}{2} \right) \]
    - \( q_{c1} \) → average cone resistance below the tip of the pile within a depth of 0.7 to 4 times the pile diameter
    - \( q_{c2} \) → minimum cone resistance below the tip of the pile within a depth of 0.7 to 4 times the pile diameter
    - \( q_{c3} \) → average cone resistance recorded above the tip of pile to a height of 8 times the pile diameter
  - **Ultimate end bearing resistance of the pile**
    \[ Q_b = q_p A_p \]
Pile Bearing Capacity from CPT Results

- Schmertmann’s proposition for piles in cohesive soil
  - *Ultimate skin resistance of the pile*
    \[ Q_f = \alpha' \bar{f} c A_s \]
  - $\alpha'$ → Ratio of pile to penetrometer sleeve friction (From chart)
  - $f_c$ → Average sleeve friction
  - $A_s$ → Area of the pile shaft

- *Ultimate bearing load of the pile*
  \[ Q_u = Q_h + Q_f \]
  - FoS → 2.5

Pile Bearing Capacity from Pile Load Test

- **Pile Load Test**
  - *In situ method of determination of pile bearing capacity*
  - *Can be used to determine*
    - Vertical load bearing capacity – Most commonly practiced
    - Lateral load bearing capacity
    - Uplift load capacity
      - Last two are done if it is known that the piles have to resist huge lateral or uplift loads

- **Pile load capacity**
  - *Estimated from the site conditions using the theoretical methods*
  - *Load tests are carried out at the beginning of the construction to check the adequacy of the estimated magnitude*
    - Carried out as stage loading (in fractions of the estimated pile load)
Pile Bearing Capacity from Pile Load Test

- Pile Load Test carried out on
  - **Working Pile**
    - This pile will be utilized to carry the load from the superstructure even after the testing
    - Cannot be tested to its full failure load
    - Max load to be applied on the pile → 1½ the design load
  - **Test Pile**
    - This pile is constructed separately and is not meant to carry load coming from the superstructure
    - Can be tested to its full failure condition
    - Max load on such pile
      - 2 ½ times the design load, or
      - Load imposed which gives a settlement greater than $\frac{1}{10}$th the pile diameter

Vertical Pile Load Test: Assembly

- Test assembly
  - A reaction frame (supported by Anchor Piles) to resist the reaction of the load applied on the pile head
  - Hydraulic jack of sufficient capacity to apply the pile load
  - Three dial gauges to measure the settlement of the pile head
Vertical Pile Load Test: Assembly

Vertical Pile Load Test: Type of Test

- Depends on load application
  - **Continuous load Test**
    - Continuous increment of load is applied at the pile head in stages
    - Settlement is recorded at the end of each load level
    - Step loading
      - 1/4th of the working load
Vertical Pile Load Test: Type of Test

- Depends on load application
  - Cyclic Load Test
    - Load is increased at a particular level and then reduced to zero
    - Next cycle undergoes a higher load level than the previous cycle
    - Settlement is recorded at each increment or decrement of load
      - Net settlement under any load $Q_2$
        $$S_{net(1)} = S_{t(1)} - S_{e(1)}$$

Vertical Pile Load Test: Allowable Load

- Minimum of the four criteria (wherever applicable)
  - Criterion 1
    - Ultimate load (as determined from the load-settlement curve using single or double tangent method) used with a suitable FoS
  - Criterion 2
    - 50% of the load at which the total settlement of the pile 0.1 times the pile diameter
  - Criterion 3
    - $2/3$rd of the load that causes a total settlement of the pile of 12mm
  - Criterion 4
    - $2/3$rd of the load which causes a net settlement of 6mm
Pile Bearing Capacity from Pile Driving Formulae

- Basic principle
  - **Resistance offered by pile during driving is the indicator of its bearing capacity**
    - Pile experiencing greater resistance during driving is capable of carrying a greater load

- **Energy equilibrium relationship**  \( Wh = Q_u s \)
  - \( Wh \rightarrow \) Energy of hammer blow
  - \( Q_u s \rightarrow \) Resisting energy of the pile to penetration
  - \( W \rightarrow \) Weight of the driving hammer
  - \( h \rightarrow \) Height of fall of hammer
  - \( Q_u \rightarrow \) Ultimate resistance of pile to penetration
  - \( s \rightarrow \) Average penetration of pile under one hammer blow
    - Considered for 100% efficiency of the system with no energy losses

Pile Bearing Capacity from Pile Driving Formulae

- Various types of hammers used for installation
  - **Drop Hammer**
  - **Single acting air or steam hammer**
  - **Double acting and differential air or steam hammer**
  - **Diesel hammer**
  - **Vibratory driver**
Pile Bearing Capacity from Pile Driving Formulae

- Hiley formula
  - Consideration of energy losses
  - Energy Input = Energy Used + Energy Loss
    - Energy used \( Q_u \)
    - Energy input \( \eta_h W_h \)
      - \( \eta_h \rightarrow \) Efficiency of the hammer

- Energy Losses
  - Energy loss \( (E_1) \) due to the elastic compression of the pile cap \( (C_1) \), pile material \( (C_2) \) and the soil surrounding the pile \( (C_3) \)
    \[
    E_1 = \frac{Q_u}{2}(c_1 + c_2 + c_3) = Q_u C
    \]

- Hiley's expression of ultimate load
  \[
  Q_u = \eta_h W_h \frac{1 + RC^2}{s + C} \quad R = \frac{W_p}{W}
  \]
Pile Bearing Capacity from Pile Driving Formulae

- Parameters of Hiley’s expression
  - Elastic compression ($c_1$) of cap and pile head
    - $c_1 = \frac{Q_u L}{AE}$
  - Elastic compression ($c_2$) of pile material
    - $c_2 = \frac{Q_u L}{AE}$
  - Elastic compression ($c_3$) of soil around pile
    - $0.1$ is taken as average
    - $0$ for hard soils and $0.2$ for resilient soils

<table>
<thead>
<tr>
<th>Pile Material</th>
<th>Range of Driving Stress kg/cm²</th>
<th>Range of $c_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precoat concrete pile with packing inside cap</td>
<td>30-150</td>
<td>0.12-0.50</td>
</tr>
<tr>
<td>Timber pile without cap</td>
<td>30-150</td>
<td>0.05-0.20</td>
</tr>
<tr>
<td>Steel H-pile</td>
<td>30-150</td>
<td>0.04-0.16</td>
</tr>
</tbody>
</table>

- Pile hammer efficiency ($\eta_h$)

<table>
<thead>
<tr>
<th>Hammer Type</th>
<th>$\eta_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drop</td>
<td>1.00</td>
</tr>
<tr>
<td>Single acting</td>
<td>0.75–0.85</td>
</tr>
<tr>
<td>Double acting</td>
<td>0.85</td>
</tr>
<tr>
<td>Diesel</td>
<td>1.00</td>
</tr>
</tbody>
</table>

- Coefficient of restitution ($C_r$)

<table>
<thead>
<tr>
<th>Material</th>
<th>$C_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood pile</td>
<td>0.25</td>
</tr>
<tr>
<td>Compact wood cushion on steel pile</td>
<td>0.32</td>
</tr>
<tr>
<td>Cast iron hammer on concrete pile without cap</td>
<td>0.40</td>
</tr>
<tr>
<td>Cast iron hammer on steel pipe without cushion</td>
<td>0.55</td>
</tr>
</tbody>
</table>
Pile Bearing Capacity from Pile Driving Formulae

- Engineering News Record (ENR) expression
  - Modified Hiley’s expression
    - The hammer is considered to be 100% efficient ($\eta_h = 1$)
    - The impact of hammer and pile is assumed to be perfectly elastic ($C_r = 1$)
    
    $$Q_u = \frac{Wh}{s + C}$$

- $s \rightarrow$ Final penetration per blow (SET of the pile). Considered as the average penetration per blow for the last 5 blows of drop hammer or 20 blows of a steam hammer
- $C \rightarrow$ An empirical constant
  - 2.5 cm for drop hammer
  - 0.25 cm for single and double acting steam hammers

Pile Bearing Capacity from Pile Driving Formulae

- General comments on the application of dynamic formulae to determine static bearing capacity
  - Inconsistency in the predicted values of bearing capacity
    - Higher or lower than that of the observed values
    - In reality, dynamic resistance of soil does not represent static resistance
      - Effect of impact loading is largely different from sustained loading
  - Should be used for freely draining materials
    - Piles driven into loose fine sand and silt has every possibility of liquefaction
      - Reduces the bearing capacity of pile
  - Should not be used for piles driven into cohesive soils
    - Sudden increase in pore pressure aids in offering resistance
    - Decrease in effective value of internal friction aids in penetration
      - Two counteracting phenomena
        - Not subjected to the analytical treatment through the dynamic formulae
      - Thixotropic behavior remains unaccounted
Pile Bearing Capacity on a Rocky Bed

- Presence of hard rocky stratum beneath the soft layer
  - Piles meeting the rocky bed are driven to refusal to obtain the maximum bearing capacity
    - Soil surrounding the shaft is quite weak
    - Pile is considered as a column without any lateral support
    - Pile bearing capacity is defined in terms of the buckling strength of the pile
    - Surrounding soil is strong enough to provide lateral confinement
    - The general procedures are used to determine the bearing capacity

- Point bearing resistance of piles resting on rocks
  \[ q_b = 2N\phi q_{ur}, \quad N\phi = \tan^2 (45 + \phi/2) \]
  - \( q_{ur} \rightarrow \) Unconfined compressive strength of rock
  - \( \phi \rightarrow \) Angle of internal friction of rock

Pile Bearing Capacity under Uplift

- Uplift resistance of piles
  - Tension piles, Uplift piles or Anchor piles
    \[ P_{ul} = W_p + A_s f_r \]
    - \( P_u \rightarrow \) Uplift resistance of pile
    - \( W_p \rightarrow \) Weight of pile
    - \( f_r \rightarrow \) Unit skin resistance
    - \( A_s \rightarrow \) Effective area of the embedded length of the pile

- Piles in cohesive soils
  \[ P_{ul} = W_p + A_s \alpha e_u \]

- Piles in cohesionless soils
  \[ P_{ul} = W_p + A_s K_s q_0 \tan \delta \]
Pile Groups

- Construction of pile foundation with a single pile is rare
  - A minimum of three piles are used beneath a column or a foundation
- Basic arrangements in a group
  - Triangular and square
  - Any other arrangement can be derived using the basic arrangements
Spacing Recommendations

<table>
<thead>
<tr>
<th>Type of Pile</th>
<th>Spacing (with respect to diameter ‘d’ of pile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight uniform diameter pile</td>
<td>2.6 <em>d</em></td>
</tr>
<tr>
<td>Friction piles</td>
<td>≥ 3 <em>d</em></td>
</tr>
<tr>
<td>End bearing pile passing through compressible stratum</td>
<td>≥ 2.5 <em>d</em></td>
</tr>
<tr>
<td>End bearing pile passing through compressible stratum and resting on stiff clay</td>
<td>≥ 3.5 <em>d</em></td>
</tr>
<tr>
<td>Compaction pile</td>
<td>2 <em>d</em></td>
</tr>
</tbody>
</table>
Efficiency of a Pile Group

- Pile groups need to be checked for
  - Ultimate load bearing capacity of the group \( Q_{gu} \)
  - Settlement of the group \( S_g \) under allowable load \( Q_{ga} \)

- Pile group efficiency
  - Ratio of the group ultimate load to the sum of individual ultimate loads
  - Pile group efficiency factor is governed by
    - Properties of the embedding soil
    - Method of installation of piles
    - Whether all the piles are of different or same properties in the group

\[
E_g = \frac{Q_{gu}}{\sum Q_a} \text{ or } \frac{Q_{gu}}{nQ_a}
\]

- \( n \rightarrow \) Number of piles in a group

Pile Group Efficiency in Cohesionless Soil

- Piles embedded in cohesionless soil
  - Significant amount of load is carried by the skin friction
  - No acceptable theoretical expression to determine \( E_g \)
    - Pile group efficiency is generally less than 1 (using empirical expression)

- Converse-Labarre expression

\[
E_g = 1 - \frac{\theta(n-1)m + (m-1)n}{90mn}
\]

\[
Q_{gu} = E_g \left[ nQ_a \right] \text{ or } E_g \left[ \sum Q_a \right]
\]

- \( m \) and \( n \rightarrow \) Number of rows and columns in a pile group \( m < n \)
- \( \theta \rightarrow \tan^{-1}(d/s) \) in degrees
- \( d \rightarrow \) Diameter of pile
- \( s \rightarrow \) Spacing of pile
  - Inherent assumption: The group ultimate capacity of piles driven in cohesionless soil is governed by the failure of the each of the individual piles
**Group Failure of Piles in Cohesive Soils**

- Ultimate bearing capacity of pile group
  - **Block failure of pile in cohesive soil**
    \[ Q_{gu} = c_b N_c A_g + P_g L_c \]
    - \( Q_{gu} \) → ultimate bearing capacity of the pile block
    - \( c_b \) → Cohesive strength of the clay beneath the pile group
    - \( N_c \) → Bearing capacity factor for deep foundations
    - \( A_g \) → Cross-sectional area of the pile group
    - \( P_g \) → Perimeter of the pile group

- **Bearing capacity based on individual pile failure**
  \[ Q_{gu}^i = nQ_i \text{ or } \sum Q_{gu} \]

- **Ultimate bearing capacity of pile group**
  \[ \min \left[ Q_{gu}, Q_{gu}^i \right] \]

---

**Settlement of Pile Foundations**
Vesic’s Method: Settlement of Single Pile in Cohesionless Soil

- Total settlement of a single pile \( S = S_p + S_f \)
  - \( S_p \rightarrow \text{Settlement of the pile tip} \)
  - \( S_f \rightarrow \text{Settlement due to axial deformation of pile shaft} \)
  - \( Q_f \rightarrow \text{Skin friction load} \)
  - \( \alpha' \rightarrow \text{Coefficient that describes the distribution of skin friction along the shaft (Usually } \alpha' = 0.6-0.7) \)
  - \( L \rightarrow \text{Length of the pile} \)
  - \( A \rightarrow \text{Area of cross section of the pile} \)
  - \( E \rightarrow \text{Elastic Modulus of the pile shaft} \)
  - \( D_r \rightarrow \text{Relative density of sand} \)
  - \( Q_b \rightarrow \text{End bearing load under ultimate condition} \)
  - \( q_{pu} \rightarrow \text{Point resistance per unit area under allowable condition} \)
  - \( C_w \rightarrow \text{Settlement coefficient} \)
  - \( \text{Driven piles} - 0.04, \text{Jacketed piles} - 0.05, \text{Bored piles} - 0.18 \)
  - \( B \text{ or } D \rightarrow \text{Width or Diameter of the piles} \)

\[
S_p = \frac{C_w Q_b}{(1 + D_r^2) q_{pu} (B \text{ or } D)}
\]

\[
S_f = \left( Q_b + \alpha' Q_f \right) \frac{L}{AE}
\]

\[
Q_{pu} = \frac{Q_b / A_b}{FoS}
\]

Settlement of Pile Group in Cohesionless Soil

- Settlement of pile group \( S_g = F_g S \)
  - \( F_g \rightarrow \text{Group settlement factor} \)
    - \( B \rightarrow \text{Distance between outer piles} \)
      - Only square pile group is considered
      - Piles are embedded in medium dense sand
      - \( d \rightarrow \text{Diameter of a single pile} \)

\[
F_g = \frac{\text{Group influence factor}}{A_B}
\]

\[
\text{Group influence factor} = 1 + \frac{B}{d}
\]
**Settlement of Pile Group in Cohesive Soils**

- Piles in cohesive soil
  - *Both immediate and consolidation settlement are considered*

- Load distribution of a pile group resting in cohesive soil
  - *Depending on the stiffness of the pile-soil system, the load distribution remains unaffected up to a certain depth*
  - *A fictitious raft footing is considered at that depth*
    - Formation of a composite footing system comprising of a ‘Shallow foundation resting at a considerable depth’
  - *Load distribution is considered beyond the point of rest of the fictitious raft*
    - The zone of soil within the pressure bulb is then utilized to estimate the immediate and consolidation settlement

**Development of Fictitious Raft Footings**

- Pile group resting on homogeneous clay
  - *The fictitious footing is assumed to be at a depth of 2L/3*
    - \( L \rightarrow \) Length of the pile
  - *2:1 distribution is followed for the load dissipation*

- If pile cap is embedded at a depth of \( D_{pc} \)
  \[ D_f = D_{pc} + 2L/3 \]
**Development of Fictitious Raft Footings**

- Pile group passing through a weaker layer into a stronger layer

  - Only the contribution of the bearing layer is considered for the estimation of the location of the fictitious footing
    - The weaker stratum of thickness $L_1$ does not contribute in any form to the settlement estimation
      - It is assumed that the weaker layer is significantly less stiffer than the pile group

  - Location of the fictitious footing
    - $2/3L_2$ below the depth of the stronger layer
      - $L_2 \rightarrow$ Depth of pile embedded in the stronger layer

**Development of Fictitious Raft Footings**

- Point bearing piles passing through a weaker layer and resting on a very firm bearing stratum

  - The fictitious raft is located on the firm stratum
    - The load distribution entirely occurs in the firm stratum
Estimation of Immediate Settlement

- Equivalent method

\[ S_e = \frac{q_n B}{E_{eq}} \left( 1 - \nu_{eq}^2 \right) I_p I_f \]
\[ q_n = \frac{Q_g}{B B'} \]

- \( q_n \) → Net pressure on the equivalent raft
- \( Q_g \) → Ultimate load capacity of the pile group
- \( B, B' \) → Dimensions of the equivalent raft
- \( \nu_{eq} \) → Equivalent Poisson’s ratio of the soil within the influence zone of the equivalent raft
- \( E_{eq} \) → Equivalent elastic modulus of the soil within the influence zone of the equivalent raft
- \( I_p \) → Steinbrenner’s influence factor \((I_2)\) for the settlement of rectangular footing \((n' = 0)\)
- \( I_f \) → Fox’s depth correction factor

Estimation of Immediate Settlement

- Method of higher accuracy

\[ S_e^i = \frac{q_n^i B_i}{E_i} \left( 1 - \nu_{eq}^2 \right) I_p I_f \]
\[ q_n^i = \frac{Q_g}{B_i B_i} \]

- Estimate the immediate settlement at the centre of each of the stratum
  - Find the excess stress generated at the centre of the layer following the 2:1 load distribution
  - Estimate the settlement at the centre of each of the stratum using the above expression
  - Add all the settlement of the soil within the zone of influence of the equivalent raft and determine the total immediate settlement

- Effect of depth of embedment of the pile cap

  - In many instance, the pile cap is embedded in the soil
    - Depth of embedment = \( D_{pc} \)
      - Depth of embedment of the equivalent raft
        \[ D_f = D_{pc} + 2L/3 \]
Estimation of Consolidation Settlement

- Similar to as estimated for a shallow foundation

\[ S_c = I_f I_p \left[ \sum \left( \frac{C_c H \log \frac{\sigma_0 + \Delta \sigma_v}{\sigma_0}}{1 + e_0} \right) \right] \]

- \( C_c \rightarrow \) Coefficient of consolidation
- \( e_0 \rightarrow \) Initial void ratio
- \( H \rightarrow \) Thickness of the stratum
- \( \sigma_0 \rightarrow \) In-situ overburden stress at the centre of the stratum
- \( \Delta \sigma \rightarrow \) Excess vertical effective stress generated at the centre of the stratum due to the external load
- \( I_f \rightarrow \) Fox’s depth correction factor
- \( I_p \rightarrow \) Pore pressure correction factor (To be provided in the problem)

Negative Skin Friction
Negative Skin Friction

- Occurrence of Negative Skin Friction
  - Single pile or group of piles passing through recently constructed cohesive soil fill
  - Single or group of piles passing through a fill material comprising of loose cohesionless soil
  - Possibility of occurrence when the fill is placed on a soft clay soil such as peat
    - A drag is generated when the fill starts consolidating subjected to its own overburden pressure
      - The drag is termed as **Negative Skin Friction**
  - May also result due to sudden lowering of ground water table thus increasing the effective weight of the consolidating layer and generation of additional frictional drag in the pile

---

Negative Skin Friction

- Single and Group of piles passing through a fill and embedded in a stiff soil
Negative Skin Friction

- Factor of Safety ($F_{s,ng}$) considering negative skin friction
  
  $F_{s,ng} = \frac{\text{Ultimate bearing capacity} (Q_u)}{\text{Working load + Negative skin friction load}}$
  
  $\Rightarrow \text{Working or Allowable Load} = \frac{Q_u}{F_{s,ng}} \cdot \text{Negative skin friction load}$

Negative Skin Friction Load on a Single Pile ($F_n$)

- Single pile embedded in cohesive soils $F_n = PL_c \bar{c}_u$
  
  - $P \rightarrow \text{Perimeter of the pile}$
  - $\bar{c}_u \rightarrow \text{Average undrained cohesion along the pile shaft}$

- Single pile embedded in cohesionless soil $F_n = 0.5PL_c^2K_0 \tan \delta$
  
  - $K_0 \rightarrow \text{Earth pressure at rest}$
  - $\delta \rightarrow \text{Angle of wall friction}$
    
    $K_0 = 1 - \sin \phi$
    
    $\delta = \begin{cases} \frac{1}{3} \text{ to } \frac{2}{3} \phi \end{cases}$

Negative Skin Friction Load on Pile Group ($F_{ng}$)

- Pile group embedded in cohesionless soil $F_{ng} = nF_n$
  
  - Inherent assumption: Full negative skin friction is mobilized for all the piles of the group

- Pile group embedded in cohesive soil
  
  - Considering the negative skin friction developed on a single pile $F_{ng} \text{ (single)} = nF_n$
  
  - Considering the negative skin friction developed on the pile group
    
    $F_{ng} \text{ (group)} = c_uL_nP_g + \gamma L_nA_g$
    
    - $A_g, P_g \rightarrow \text{Area and Perimeter of the pile group}$

  - Design value of negative skin friction load $F_{ng}^{\text{des}} = \max \left[ F_{ng} \text{ (single)}, F_{ng} \text{ (group)} \right]$
Negative Skin Friction Load on Pile Group ($F_{ng}$)

- Pile group embedded in peat and soft clay
  - Considering individual action
    \[ F_{ng} \text{ (single)} = n(F_{n1} + F_{n2}) \]
  - Considering group action
    \[ F_{ng} \text{ (group)} = s_1 L_4 P_g + s_2 L_2 P_h + \gamma_1 L_4 A_g + \gamma_2 L_2 A_g \]
    \[ = P_h (s_1 L_4 + s_2 L_2) + A_g (\gamma_1 L_4 + \gamma_2 L_2) \]
  - Design Load
    \[ F_{ng}^{des} = \max [F_{ng} \text{ (single)}, F_{ng} \text{ (group)}] \]

Lateral Load Behavior of Pile
Laterally Loaded Pile

- Typical deflection profile, soil reaction curve, and generated stresses around a laterally loaded pile in homogeneous soil

Fixed Headed Laterally Loaded Pile

- Example of pile supporting bridge pier and embedded in multi-layered soil
Fixed Headed Laterally Loaded Pile

- 3-D Finite Element modelling of single and group of piles in multilayered soil and subjected to lateral loading

Fixed Headed Laterally Loaded Pile

- Flexural Characteristics along the length of pile
  - Deflection, Bending moment, Shear force and Contact stress profiles
Fixed Headed Laterally Loaded Pile

- Load-deformation behaviour and serviceability states to defined the extent of lateral deformation

![Graph showing lateral stress and pile deflection](image)

Fixed Headed Laterally Loaded Pile

- Soil deformation isosurface due to lateral displacement of pile

![Soil deformation isosurfaces](image)
Fixed Headed Laterally Loaded Pile

- Soil deformation vectors due to lateral displacement of pile
  - 3D horizontal stress dispersion

State of earth pressure during pile displacement
- Transition from active state behind the pile to passive state in front of the pile
Fixed Headed Laterally Loaded Pile Group

- Higher lateral capacity of the pile group
  - Block failure action
    - Strain all around the piles
    - No strain in the soil block within the piles

Hybrid Foundation
Piled Raft System
Finite Element Modeling of Piled Raft System

- Foundation of multi-storied building at Surfer’s Paradise, Gold Coast Australia
Finite Element Modeling of Piled Raft System

- Foundation of multi-storied building at Surfer’s Paradise, Gold Coast Australia
  - Deformation of piled-raft system

FE Modeling of Disconnected Piled Raft System

- Foundation of Petronas Twin Tower, Malaysia
FE Modeling of Disconnected Piled Raft System

- Foundation of Petronas Twin Tower, Malaysia

- Comparison of connected and disconnected piled raft
  - Response of the raft
FE Modeling of Disconnected Piled Raft System

- Foundation of Petronas Twin Tower, Malaysia
  - Comparison of connected and disconnected piled raft
    - Response of the pile

![Graphs showing comparison of axial force versus depth for inner and outer piles.]

Thank You for Patient Hearing