5) Eccentrically Loaded Foundations

For Q + M loading condition,

\[ q_{\text{max, min}} = \frac{Q}{BL} \pm \frac{6M}{B^2 L} \quad \text{for } e \leq B/6 \]

For Q with eccentricity e,

\[ q_{\text{max, min}} = \frac{Q}{BL} (1 \pm \frac{6e}{B}) \quad \text{for } e \leq B/6 \]

\[ q_{\text{min}} = 0, \quad q_{\text{max}} = \frac{4Q}{3L(B-2e)} \quad \text{for } e > B/6 \]

Note:
* Effective area method (Meyerhof(1953))

\[ B' = B - 2e \]
(For Q+M loading condition, \( e = M/Q \))

\[ q_u = cN_c F'_{cs} F_{cd} + qN_q F'_{qs} F_{qd} + \gamma B' N_f F_{pd} F_{pf} + 1/2 \gamma B' N_f F_{pd} F_{pf} \]

To get \( F'_{cs}, \) \( F'_{qs}, \) and \( F_f, \) use \( B' \) instead of \( B \). (Use \( B \) for \( F'_{cd}, \) \( F'_{qd}, \) and \( F'_{pf} \))

\[ Q_u = q_u (B')(L) \]

\[ \Rightarrow \quad \text{F.S.} = Q_u / Q \]

- To reduce the effects of eccentricity, use foundation of columns with off-center loading.
6) Effect of Layering

i) Stronger soil underlain by weaker soil

* Simplified approach

\[ q_0 = \frac{Q}{B + H} \text{ for strip ft.} \]

Bearing Capacity check here
With \[ q = \gamma (D_f + H) \] and footing width of \[ B + H \]
* General Approach
Layer 1: Punching shear

For cohesion($c_a$) effect, $q_{u1} = 2c_a H / B$.

$\leftarrow c_a$ given by relative cohesion based on $c_1^*$, with $q_2 / q_1$.

where $q_1 = c_1^* N_{c_1} + 1/2 \gamma_1 BN_{\gamma_1}$, and $q_2 = c_2^* N_{c_2} + 1/2 \gamma_2 BN_{\gamma_2}$.
For passive pressure effect,
\[ q_{u1} = 2P_p \sin \delta / B \]
\[ P_p = 0.5\gamma H^2 (1 + \frac{2D_f}{H}) \frac{K_{plh}}{\cos \delta}, \]
where \( K_{plh} \) is the coefficient of passive earth pressure
\[ (= (1 + \sin \phi')/(1 - \sin \phi')) \].

Meyerhof recommends \( K_s \tan \phi' = K_{plh} \tan \delta \), where \( K_s \) is punching shear resistance, and for sand layer over clay, \( K_s = f(q_2/q_1, \phi'_1) \) as shown below.

Thus,
\[ q_{u1}(= 2P_p \sin \delta / B) = \gamma H^2 (1 + \frac{2D_f}{H}) K_s \frac{\tan \phi'}{B} \]

Layer 2: General shear failure
\[ q_{u2} = c_2'N_{c2} + 1/2\gamma_2BN_{q2} + \gamma_1(H + D_f)N_{q2} \]
- Ultimate bearing capacity

\[ q_u = q_{u1} + (q_{u2} - \gamma_1 H) \]

\[ q_u = \frac{2c_u H}{B} + \gamma_1 H^2 (1 + \frac{2D_f}{H}) K_s \tan \phi_s \]

\[ + \frac{c_2' N_{c2} + 1/2 \gamma_2 BN_{\gamma_2} + \gamma_1 (H + D_f)N_q}{B} - \gamma_1 H \]

For strong clay over soft clay,

\[ q_u = \frac{2c_u H}{B} + c_2' N_{c2} + \gamma_1 D_f \leq \text{general shear failure of top soil} \]

\[ = c_1' N_{c1} + \gamma_1 D_f \]

For dense sand over soft clay,

\[ q_u = \gamma_1 H^2 (1 + \frac{2D_f}{H}) K_s \tan \phi_s \]

\[ + \frac{c_2' N_{c2} + \gamma_1 D_f}{B} \leq \text{general shear failure of top soil} \]

\[ = \frac{1}{2} \gamma_1 BN_{\gamma_1} + \gamma_1 D_f N_{q1} \]

To consider shape effect for layered soil, Meyerhof recommends use of factor \((1+B/L)\) for punching shear terms and values as below for remained terms.

For \(\phi = 0\):

\[ F_{c_v} = 1 + 0.2B/L, \quad F_{q_v} = 1, \quad F_w = 1 \]

For \(\phi \geq 0\):

\[ F_{c_v} = 1 + 0.2(B/L) \tan^2 (45 + \phi/2), \quad F_{q_v} = F_w = 1 + 0.1(B/L) \tan^2 (45 + \phi/2) \]
ii) Weak soil over strong soil.

\[
q_u = q_t + (q_b - q_t) \left(1 - \frac{H}{H_f}\right)^2 \geq q_t
\]

where, 
- \(q_t\): Bearing capacity of top soil.
- \(q_b\): Bearing capacity of bottom soil.
- \(H_f\): Failure depth (from bottom of footing) (=B).

For soft clay over stiff clay,

\[
q_t = c_1'N_{c1} + \gamma_1D_f = q_{u\min} \quad \text{(for } H \geq B) \\
q_b = c_2'N_{c2} + \gamma_1D_f = q_{u\max} \quad \text{(for } H = 0)
\]
iii) Foundation Supported by a Soil with a Rigid Base at Shallow Depth
(Mandel & Salencon (1972))

* The extent of the failure zone below the bottom of foundation, $D$

![Diagram](image1.png)

- The failure surface in case that depth to a rigid, rough base $H$ is smaller than $D$,

![Diagram](image2.png)
- The ultimate bearing capacity of a rough continuous foundation with a rigid, rough base at a shallow depth

\[ q_u = c' N_c^* + q N_q^* + \frac{1}{2} \gamma B N_r^* \]

where, \( N_c^*, N_q^*, N_r^* \) = modified bearing capacity factors

(Mandel and Sclencon (1972))
For $H \geq D, \quad N_c^* = N_c, \quad N_q^* = N_q$ and $N_\gamma^* = N_\gamma$
7) Bearing Capacity of Foundations on Top of Slope

*Meyerhof recommends:

\[ q_u = c' N_{cq} + \frac{1}{2} \gamma B N_{\gamma q} \]

where \( N_{cq} \) and \( N_{\gamma q} \) are given in Figures on next page.

For \( N_{cq} \) in Figure 4.12,

i) \( N_s \) (= stability number) = \( \gamma H / c' \)

ii) If \( B < H \), use the curves for \( N_s = 0 \)

If \( B \geq H \), use the curves for the calculated \( N_s \)
8) Selection of Soil Strength Parameters

i) Saturation
   Generally
   Saturated strength < Unsaturated strength
   To design for the worst-case conditions, the saturated strength is nearly always used.

ii) The fundamental bearing capacity formulas are based on continuous footings (Plane strain conditions)
    ➞ Formulas for other shapes are derived from the continuous footing using empirical adjustments.
    ➞ Plane strain strength should be used for bearing capacity analysis regardless of footing shape.
    ➞ However, engineers rarely consider the differences between plane strain and axisymmetric strengths from tests, and plane strain testing device is more complicated and need experienced skill to handle.
    ➞ So, axisymmetric strength is generally used.

iii) Drained vs. Undrained Strength (Saturated soils)
    - Sands : Drained strength
    - Clays
      - Normally consolidated or lightly overconsolidated conditions (positive pore water pressure)
        Undrained strength < Drained strength
      - Heavily overconsolidated conditions (negative pore pressure)
        Undrained strength > Drained strength
    - Intermediate soils : More conservative approach ⇒ undrained strength.
9) Bearing Capacity on Rocks

* Problems:

* The allowable bearing pressure may be determined in at least four ways (Kulhawy and Goodman, 1980):
  - Presumptive values found in building codes (Table 6.5)
  - Empirical rules
  - Rational methods based on bearing capacity and settlement analyses
  - Full scale load tests

* Semi-empirical approach for bearing capacity (Carter and Kulhawy, 1988)

\[ q_u = JcN_{cr} \]

where:

- \( q_u \) = net ultimate bearing capacity
- \( J \) = correction factor (Figure 6.17)
- \( c \) = cohesive strength of the rock mass
- \( \phi \) = friction angle of the rock mass
- \( N_{cr} \) = bearing capacity factor (Figure 6.18)
- \( H \) = vertical spacing of discontinuities
- \( S \) = horizontal spacing of discontinuities
- \( B \) = width of footing

* Modification of \( c' \) and \( \phi' \) from lab test results.

\[ \phi' = (0.5 - 0.75)\phi'_{lab}, \quad c = a_Ec'_{lab} \]

\[ a_E = 0.1 \text{ for } RQD < 70\% \]

\[ a_E = 0.6 \text{ for } RQD = 100\% \]

<\( RQD : \text{Rock Quality Designation}>\)

* If rock mass is very strong, the strength of the footing concrete may governs the bearing capacity.

SNU Geotechnical Engineering Lab.
<Typical allowable bearing pressures for foundation on bedrock>

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Rock Consistency</th>
<th>Allowable Bearing Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive crystalline igneous and metamorphic rock: Granite, diorite, basalt, gneiss, thoroughly cemented conglomerate</td>
<td>Hard and sound (minor cracks OK)</td>
<td>120,000 ~ 200,000</td>
</tr>
<tr>
<td>Foliated metamorphic rock: Slate, schist</td>
<td>Medium hard, sound (minor cracks OK)</td>
<td>60,000 ~ 80,000</td>
</tr>
<tr>
<td>Sedimentary rock: Hard cemented shales, siltstone, sandstone, limestone without cavities</td>
<td>Medium hard, sound</td>
<td>30,000 ~ 50,000</td>
</tr>
<tr>
<td>Weathered or broken bedrock of any kind: compaction shale or other argillaceous rock in sound condition</td>
<td>soft</td>
<td>16,000 ~ 24,000</td>
</tr>
</tbody>
</table>
Figure 6.17 Correction factor, J, for Equation 6.54 (Adapted from Carter and Kullaway, 1988. Copyright ©1988 Electric Power Research Institute, reprinted with permission).

Figure 6.18 Bearing capacity factor, \( N_{qe} \), for Equation 6.54 (Adapted from Carter and Kullaway, 1988. Copyright ©1988 Electric Power Research Institute, reprinted with permission).