OVERVIEW

REVIEW OF 14.431 FOUNDATIONS & SOILS ENG.

- Soil Borings.
- Geotechnical Report (not covered).
- Bearing Pressure Calculations.
- Settlement Calculations.
- Lateral Earth Pressure Calculations.
- Retaining Wall Design Review.
STANDARD PENETRATION TEST (SPT) (ASTM D1586-11)

• Very common test worldwide
• 1902 - Colonel Gow of Raymond Pile Co.
• Split-barrel sample driven in borehole.
• Conducted on 2½ to 5 ft depth intervals
• ASTM D1586 guidelines
• Drop Hammer (140 lbs falling 30 inches)
• 3 or 4 increments of 6 inches each
  – Three (3) Increments: Sum of last two increments = “SPT N value” (blows/ft)
  – Four (4) Increments: Sum of last two increments = “SPT N value” (blows/ft)
• Correlations available with all types of soil engineering properties
• Disturbed Soil Samples Collected
STANDARD PENETRATION TEST (SPT) (ASTM D1586-11)

Typical Setup

Split Spoon Dimensions (after ASTM D1586)
STANDARD PENETRATION TEST (SPT) (ASTM D1586-11)

- **63.5 kg Drop Hammer**
  - Repeatedly Falling 0.76 m
- **Anvil**
- **Borehole**
  - **Drill Rod**
    - "N" or "A" Type
  - **Split-Barrel (Drive) Sampler**
    - **Thick Hollow Tube**
    - **O.D.** = 50 mm
    - **I.D.** = 35 mm
    - **L** = 760 mm
- **Need to Correct to a Reference Energy Efficiency of 60% (ASTM D 4633)**
- **Note:** Occasional Fourth Increment Used to provide additional soil material
- **SPT Resistance (N-value) or “Blow Counts”**
  - is total number of blows to drive sampler last 300 mm (or blows per foot).
STANDARD PENETRATION TEST (SPT) (ASTM D1586-11)
### STANDARD PENETRATION TEST (SPT)

Factors Affecting SPT (after Kulhawy & Mayne, 1990 & Table 8. FHWA IF-02-034 )

<table>
<thead>
<tr>
<th>Cause</th>
<th>Effects</th>
<th>Influence on N Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inadequate Cleaning of Borehole</td>
<td>SPT not made in insitu soil, soil trapped, recovery reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Failure to Maintain Adequate Head in Borehole</td>
<td>Bottom of borehole may become quick</td>
<td>Decreases</td>
</tr>
<tr>
<td>Careless Measure of Drop</td>
<td>Hammer Energy varies</td>
<td>Increases</td>
</tr>
<tr>
<td>Hammer Weight Inaccurate</td>
<td>Hammer Energy varies</td>
<td>Increases</td>
</tr>
<tr>
<td>Hammer Strikes Drill Rod Collar Eccentrically</td>
<td>Hammer Energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Lack of Hammer Free (ungreased sleeves, stiff rope, more than 2 turns on cathead, incomplete release of drop, etc.)</td>
<td>Hammer Energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Sampler Driven Above Bottom of Casing</td>
<td>Sampler driven in disturbed soil</td>
<td>Inc. Greatly</td>
</tr>
<tr>
<td>Careless Blow Count Recording</td>
<td>Inaccurate Results</td>
<td>Inc. or Dec.</td>
</tr>
<tr>
<td>Use of Non-Standard Sampler</td>
<td>Correlations with Std. Sampler Invalid</td>
<td>Inc. or Dec.</td>
</tr>
<tr>
<td>Coarse Gravel or Cobbles in soil</td>
<td>Sampler becomes clogged or impeded</td>
<td>Increases</td>
</tr>
<tr>
<td>Use of Bent Drill Rods</td>
<td>Inhibited transfer of energy to sampler</td>
<td>Increases</td>
</tr>
</tbody>
</table>
CARE & PRESERVATION OF SOIL SAMPLES

- Mark and Log samples upon retrieval (ID, type, number, depth, recovery, soil, moisture).
- Place jar samples in wood or cardboard box.
- Should be protected from extreme conditions (heat, freezing, drying).
- Sealed to minimize moisture loss.
- Packed and protected against excessive vibrations and shock.
14.485 CAPSTONE DESIGN
Module 4 – Geotechnical Engineering

STANDARD PENETRATION TEST (SPT)

- $c_u = \text{undrained strength}$
- $\gamma_T = \text{unit weight}$
- $I_R = \text{rigidity index}$
- $' = \text{friction angle}$
- $OCR = \text{overconsolidation}$
- $K_0 = \text{lateral stress state}$
- $e_o = \text{void ratio}$
- $V_s = \text{shear wave}$
- $E' = \text{Young's modulus}$
- $C_c = \text{compression index}$
- $q_b = \text{pile end bearing}$
- $f_s = \text{pile skin friction}$
- $k = \text{permeability}$
- $q_a = \text{bearing stress}$

- $D_R = \text{relative density}$
- $\gamma_T = \text{unit weight}$
- $LI = \text{liquefaction index}$
- $' = \text{friction angle}$
- $c' = \text{cohesion intercept}$
- $e_o = \text{void ratio}$
- $q_a = \text{bearing capacity}$
- $\sigma_p' = \text{preconsolidation}$
- $V_s = \text{shear wave}$
- $E' = \text{Young's modulus}$
- $\Psi = \text{dilatancy angle}$
- $q_b = \text{pile end bearing}$
- $f_s = \text{pile skin friction}$

What Do We Need? How Do We Get It?
CORRECTIONS TO SPT N VALUE

\[ N_{60} = \text{Corrections to SPT N Value} \]

\[ N_{measured} = \text{Raw SPT Value from Field Test (ASTM D1586-11)} \]

\[ N_{60} = \text{Corrected N values corresponding to 60\% Energy Efficiency} \]

(i.e. The Energy Ratio (ER) = 60\% (ASTM D4633-10))

Note: 30\% < ER < 100\% with average ER = 60\% in the U.S.

\[ N_{60} = C_E C_B C_S C_R N_{measured} \]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Term</th>
<th>Equipment Variable</th>
<th>Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy Ratio</td>
<td>( C_E = \frac{\text{ER}}{60} )</td>
<td>Donut Hammer</td>
<td>0.5 to 1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Safety Hammer</td>
<td>0.7 to 1.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Automatic Hammer</td>
<td>0.8 to 1.5</td>
</tr>
<tr>
<td>Borehole Diameter</td>
<td>( C_B )</td>
<td>65 – 155 mm</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150 mm</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 mm</td>
<td>1.15</td>
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<tr>
<td>Sampling Method</td>
<td>( C_S )</td>
<td>Standard Sampler</td>
<td>1.0</td>
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<tr>
<td></td>
<td></td>
<td>Non-Standard Sampler</td>
<td>1.1 to 1.3</td>
</tr>
<tr>
<td>Rod Length</td>
<td>( C_R )</td>
<td>3 – 4 m</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 – 6 m</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 – 10 m</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 10 m</td>
<td>1.00</td>
</tr>
</tbody>
</table>

SPT Corrections

(From Table 9, FHWA IF-02-034)

For Guidance Only. Actual ER values should be measured per ASTM D4633
Corrections to SPT N Value

Two Borings/One Site Example:

Data from Robertson, et al. (1983), Courtesy of FHWA NHI Course 132031 Subsurface Investigations
Normalized SPT N Value \((N_1)_{60}\)

\((N_1)_{60} = N_{60}\) values normalized to 1 atmosphere overburden stress.

\((N_1)_{60} = C_N N_{60}\)

Where:

\[ C_N = \left( \frac{P_a}{\sigma'_{vo}} \right)^n \]

- \(P_a\) = Atmospheric Pressure (1 atm = 14.7 psi = 2116 psf)
- \(\sigma'_{vo}\) = Insitu Vertical Effective Stress
- \(n = 1\) (clays) and 0.5 to 0.6 (sands)
Effective Friction Angle ($\phi'$) for Sands - SPT

\[ \phi' = [15.4(N_1)_{60}]^{0.5} + 20^\circ \]

Figure 9-12. FHWA NHI Course 132031 Subsurface Investigations
### Soil Shear Strength Correlations from In-Situ Testing

<table>
<thead>
<tr>
<th>Shear Strength Parameter</th>
<th>Insitu Testing Method</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Effective Soil Friction Angle (φ')</strong></td>
<td>SPT: See Slide 15, CPT: arctan[0.1+0.38log(qt/σ_vo')], DMT: 28°+14.6°log(K_D)-2.1°log²K_D</td>
</tr>
<tr>
<td><strong>Undrained Shear Strength (S_u)</strong></td>
<td>NO ACCEPTABLE CORRELATIONS, SPT: (qt-σ_vo)/N_kt (N_kt = 15 for CHS), CPT: 0.22σ_vo'(0.5K_D)^{1.25}, DMT: Aas et al. (1986), Marchetti et al. (2001) ISSMGE TC 16 Report</td>
</tr>
</tbody>
</table>

**NOTES:**
1. (Nt)_60 = N_60(P_a/σ_vo')^{0.5} for sands. P_a = Atmospheric Pressure = 1 bar = 1 tsf.
2. σ_vo' = In-situ Effective Overburden Pressure = In-situ Vertical Effective Stress.
3. σ_vo = Total Overburden Pressure = In-situ Vertical Total Stress.
### SOIL SHEAR STRENGTH CORRELATIONS FROM IN-SITU TESTING

**Effective Soil Friction Angle ($\phi'$) Summary from NCHRP Report 651 (2010)**

<table>
<thead>
<tr>
<th>Equation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi' = 54^\circ - 27.6034 \times \exp(-0.014(N_1)_{60})$</td>
<td>Peck, Hanson, &amp; Thorton (1974) from Kulhawy &amp; Mayne (1990)</td>
</tr>
<tr>
<td>$\phi' = [20(N_1)<em>{60}]^{0.5} + 20^\circ$ for $3.5 \leq (N_1)</em>{60} \leq 30$</td>
<td>Hatanaka &amp; Uchida (1996)</td>
</tr>
<tr>
<td>$\phi' = 27.1^\circ + 0.3(N_1)<em>{60} - 0.00054(N_1)</em>{60}^2$</td>
<td>Peck, Hanson, &amp; Thorton (1974) from Wolff (1989)</td>
</tr>
<tr>
<td>$\phi' = [15.4(N_1)_{60}]^{0.5} + 20^\circ$</td>
<td>Mayne et al. (2001) based on Hatanaka &amp; Uchida (1996)</td>
</tr>
<tr>
<td>$\phi' = [15(N_1)<em>{60}]^{0.5} + 15^\circ$ for $(N_1)</em>{60} &gt; 5$ and $\phi \leq 45^\circ$</td>
<td>JRA (1996)</td>
</tr>
</tbody>
</table>
### Soil Engineering Property Correlations from In-situ Testing (Table 1)

<table>
<thead>
<tr>
<th>Soil</th>
<th>Density/Consistency</th>
<th>$N$</th>
<th>$q_t$ (MPa)</th>
<th>$\gamma_t$ (pcf)</th>
<th>$\phi'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SANDS</td>
<td>V. Loose</td>
<td>0-4</td>
<td>0-2</td>
<td>90-105</td>
<td>&lt;30</td>
</tr>
<tr>
<td></td>
<td>Loose</td>
<td>5-10</td>
<td>2-5</td>
<td>95-110</td>
<td>30-35</td>
</tr>
<tr>
<td></td>
<td>Medium Dense</td>
<td>11-30</td>
<td>5-15</td>
<td>105-120</td>
<td>35-38</td>
</tr>
<tr>
<td></td>
<td>Dense</td>
<td>31-50</td>
<td>15-25</td>
<td>115-130</td>
<td>38-41</td>
</tr>
<tr>
<td></td>
<td>Very Dense</td>
<td>&gt;50</td>
<td>&gt;25</td>
<td>125-140</td>
<td>41-44</td>
</tr>
<tr>
<td>COHESIVE SOILS</td>
<td>Very Soft</td>
<td>0-2</td>
<td>0-0.5</td>
<td>90-100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Firm</td>
<td>2-8</td>
<td>0.5-1.5</td>
<td>90-110</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stiff</td>
<td>9-15</td>
<td>1.5-3</td>
<td>105-125</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>Very Stiff</td>
<td>15-30</td>
<td>3-6</td>
<td>115-135</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Hard</td>
<td>&gt;30</td>
<td>&gt;6</td>
<td>120-140</td>
<td></td>
</tr>
</tbody>
</table>

after Fang et al. (1991) and EM 1110-1-1905.

**NOTE:** 1 MPa = 10.44 tsf
NORMALIZED SPT N VALUE N₁₆₀
(AASHTO 2010)

\[ N₁₆₀ = Cₙ N₆₀ \]

Where:

- \( Cₙ = [0.77 \log_{10}(40/σ'_{vo})] \)
  \((Cₙ < 2.0)\)
- \( σ'_{vo} = \text{Insitu Vertical Effective Stress (ksf)} \)
- \( N₆₀ = \text{SPT Blow Count corrected for hammer eff.} \)

* Assume ER = 60% (Cathead) & 80% (Automatic)
AASHTO (2010) 10.6 – SPREAD FOOTINGS
10.6.1.3 – Effective Footing Dimensions

\[ B' = B - 2e_b \]
\[ L' = L - 2e_L \]

Where:
\[ e_b = \text{Eccentricity parallel to Dimension B.} \]
\[ e_L = \text{Eccentricity parallel to Dimension L.} \]

Figure C10.6.1.3.1. – Reduced Footing Dimensions.
AASHTO (2010) 10.6 – SPREAD FOOTINGS
10.6.3.1 – Bearing Resistance of Soil

\[ q_R = \phi_b \cdot q_n \]

Where:
- \( q_R \) = Factored Resistance
- \( \phi_b \) = Resistance Factor
  (see Article 10.5.5.2.2)
- \( q_n \) = Nominal Bearing Resistance
  \( = q_{ult} \) in 14.431
**REVIEW – BEARING CAPACITY EQUATION**
*(after Meyerhof, 1963)*

\[ q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + 0.5\gamma BN_{\gamma}F_{\gamma s}F_{\gamma d}F_{\gamma i} \]

Where:
- \( c' = \) Soil Cohesion
- \( N_c = \) Bearing Capacity Factor - Cohesion
- \( q = \) Surcharge = \( D_f\gamma \)
- \( N_q = \) Bearing Capacity Factor - Surcharge
- \( \gamma = \) Soil Unit Weight
- \( N_{\gamma} = \) Bearing Capacity Factor – Soil
- \( B = \) Footing Width
- \( F_{cs}, F_{qs}, F_{\gamma s} = \) Shape Factors
- \( F_{cd}, F_{qd}, F_{\gamma d} = \) Depth Factors
- \( F_{ci}, F_{qi}, F_{\gamma i} = \) Inclination Factors
Bearing Capacity Factors (Dimensionless, based on $\phi'$)

\[ N_q = \tan^2 \left( 45^\circ + \frac{\phi'}{2} \right) e^{\pi \tan \phi'} \]

\[ N_c = (N_q - 1) \cot \phi' \]

\[ N_\gamma = 2(N_q + 1) \tan \phi' \]

Also see Table 12.1, Das FGE (2006) for Tabular Data
### Bearing Capacity Factors (Table 12.1 Das FGE 2006)

<table>
<thead>
<tr>
<th>$\phi'$</th>
<th>$N_c$</th>
<th>$N_q$</th>
<th>$N_y$</th>
<th>$\phi'$</th>
<th>$N_c$</th>
<th>$N_q$</th>
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<td>0</td>
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<td>1.00</td>
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<td>25.80</td>
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<td>6</td>
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<td>27.86</td>
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<td>30.14</td>
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<td>32.67</td>
<td>20.63</td>
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<td>2.25</td>
<td>1.03</td>
<td>32</td>
<td>35.49</td>
<td>23.18</td>
<td>30.22</td>
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<td>2.47</td>
<td>1.22</td>
<td>33</td>
<td>38.64</td>
<td>26.09</td>
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<td>8.80</td>
<td>2.71</td>
<td>1.44</td>
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<td>9.28</td>
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<td>10.37</td>
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<td>38</td>
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<td>12.34</td>
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<td>75.31</td>
<td>64.20</td>
<td>109.41</td>
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<td>13.10</td>
<td>5.26</td>
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<td>83.86</td>
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<td>5.80</td>
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<td>20</td>
<td>14.83</td>
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<td>5.39</td>
<td>43</td>
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<td>186.54</td>
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<td>21</td>
<td>15.82</td>
<td>7.07</td>
<td>6.20</td>
<td>44</td>
<td>118.37</td>
<td>115.31</td>
<td>224.64</td>
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<tr>
<td>22</td>
<td>16.88</td>
<td>7.82</td>
<td>7.13</td>
<td>45</td>
<td>133.88</td>
<td>134.88</td>
<td>271.76</td>
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</table>
### Bearing Capacity Factors

<table>
<thead>
<tr>
<th>Factor</th>
<th>Cohesion</th>
<th>Surcharge</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shape</strong> (DeBeer, 1970)</td>
<td>$F_{cs} = 1 + \frac{B}{L} \left( \frac{N_q}{N_c} \right)$</td>
<td>$F_{qs} = 1 + \frac{B}{L} \tan \phi'$</td>
<td>$F_{\gamma_s} = 1 - 0.4 \left( \frac{B}{L} \right)$</td>
</tr>
<tr>
<td><strong>Depth</strong> (D_f/B ≤ 1) (Hanson, 1970)</td>
<td>$F_{cd} = 1 + 0.4 \left( \frac{D_f}{B} \right)$</td>
<td>$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \frac{D_f}{B}$</td>
<td>$F_{\gamma_d} = 1$</td>
</tr>
<tr>
<td><strong>Depth</strong> (D_f/B &gt; 1) (Hanson, 1970)</td>
<td>$F_{cd} = 1 + 0.4 \tan^{-1} \left( \frac{D_f}{B} \right)$</td>
<td>$F_{q_i} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \tan^{-1} \left( \frac{D_f}{B} \right)$</td>
<td>$F_{\gamma_d} = 1$</td>
</tr>
<tr>
<td><strong>Inclination</strong> Hanna &amp; Meyerhof (1981)</td>
<td>$F_{ci} = \left( 1 - \frac{\beta^\circ}{90^\circ} \right)^2$</td>
<td>$F_{qi} = \left( 1 - \frac{\beta^\circ}{90^\circ} \right)^2$</td>
<td>$F_{\gamma_i} = \left( 1 - \frac{\beta}{\phi'} \right)$</td>
</tr>
</tbody>
</table>

$\beta = \text{Inclination of Load with respect to vertical}$

The factor $\tan^{-1}(D_f/B)$ is in radians.
**BEARING CAPACITY EQUATION**

(AASHTO LRFD Design Specifications, 5th Ed., 2010)

\[ q_n = \frac{cN_{cm}}{\gamma} + \frac{\gamma D_f N_{qm} C_{wq}}{\gamma} + 0.5\frac{\gamma B N_{\gamma m} C_{w\gamma}}{\gamma} \]

Where:

- \( c \) = Undrained Shear Strength
- \( N_{cm} = N_c s_c i_c \)
- \( N_c \) = Cohesion BCF for undrained loading (see Table 10.6.3.1.2a-1)
- \( s_c \) = Footing Shape Correction Factor (see Table 10.6.3.1.2a-3)
- \( i_c \) = Load Inclination Factor (Eq. 10.6.3.1.2a-5 or 6)

- \( \gamma \) = Moist Unit Weight of soil above footing
- \( D_f \) = Footing Embedment Depth
- \( N_{qm} = N_q s_q d_q i_q \)
- \( N_q \) = Surcharge BCF (see Table 10.6.3.1.2a-1)
- \( s_q \) = Footing Shape Correction Factor (see Table 10.6.3.1.2a-3)
- \( d_q \) = Depth Correction Factor (see Table 10.6.3.1.2a-4)
- \( i_q \) = Load Inclination Factor (Eq. 10.6.3.1.2a-7)
- \( C_{wq} \) = Groundwater CF (Table 10.6.3.1.2a-2)

- \( \gamma \) = Moist Unit Weight of soil below footing
- \( B \) = Footing Width
- \( N_{\gamma m} = N_s i \gamma \)
- \( N_\gamma \) = Unit Weight BCF (see Table 10.6.3.1.2a-1)
- \( s_\gamma \) = Footing Shape Correction Factor (see Table 10.6.3.1.2a-3)
- \( i_\gamma \) = Load Inclination Factor (Eq. 10.6.3.1.2a-8)
- \( C_{w\gamma} \) = Groundwater CF (Table 10.6.3.1.2a-2)

This is Munfakh et al. (2001) for determining \( \phi_b \)
## Presumptive Maximum Allowable Bearing Pressures
(from Table 1804.2, IBC 2006)

<table>
<thead>
<tr>
<th>Material</th>
<th>USCS</th>
<th>$q_{all}$ (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crystalline Bedrock</td>
<td></td>
<td>12</td>
</tr>
<tr>
<td>Sedimentary and Foliated Rock</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Sandy Gravel and/or Gravel</td>
<td>GW &amp; GP</td>
<td>3</td>
</tr>
<tr>
<td>Sand, Silty Sand, Clayey Sand, Silty Gravel, &amp; Clayey Gravel</td>
<td>SW, SP, SM, SC, GM, GC</td>
<td>2</td>
</tr>
<tr>
<td>Clay, Sandy Clay, Silty Clay, Clayey Silt, Silt, and Sandy Silt</td>
<td>CL, CH, ML, MH</td>
<td>1.5</td>
</tr>
<tr>
<td>Mud, Organic Silt, Organic Clays, Peat, and Unprepared Fill</td>
<td></td>
<td>TBD</td>
</tr>
</tbody>
</table>

See also Table 1 NAVFAC DM7.02 (p. 142) and Table C10.6.2.6.1-1 AASHTO (2010).
AASHTO (2010) 10.6 – SPREAD FOOTINGS

Other Bearing Capacity Considerations

- Figure C10.6.1.3.2b-1. Punching Failure (Reduction in Shear Strength)
- Figure C10.6.1.3.2c-1 (& c-2). Footing on Slopes (Reduction in Bearing Factors)
- Figure C10.6.1.3.2e-2. Footing on Two Layer Soil Systems (Dependent)
AASHTO (2010) 10.6 – Spread Footings

10.6.2.4.2 – Settlement Analysis

\[ S_t = S_e + S_c + S_s \]

Where:
- \( S_t \) = Total Settlement
- \( S_e \) = Elastic Settlement
- \( S_p \) = Primary Consolidation Settlement
- \( S_s \) = Secondary Consolidation Settlement
AASHTO (2010) 10.6 – SPREAD FOOTINGS
10.6.2.4.2 – Settlement (Cohesionless Soils)

Elastic Half-Space Method:

\[ S_e = \frac{q_o (1 - \nu^2) \sqrt{A'}}{144 E_s \beta_z} \]

Where:

- \( S_e \) = Elastic Settlement
- \( q_o \) = Applied Vertical Stress (ksf)
- \( A' \) = Effective Area of Footing (ft^2)
- \( E_s \) = Young’s Modulus of Soil (ksi)
- \( \nu \) = Soil Poisson’s Ratio
- \( \beta_z \) = Shape Factor (dimensionless)

Unless \( E_s \) varies significantly with depth, \( E_s \) should be determined at a depth of about \( \frac{1}{2} \) to \( \frac{2}{3} \) of \( B \) below the footing. If the soil modulus varies significantly with depth, a weighted average value of \( E_s \) should be used.
ELASTIC SETTLEMENT OF SOIL (DMT)

DMT USING A DRILL RIG

Uses Direct Measurement of Soil to Calculate Settlement

Figure courtesy of Marchetti (1999) - The Flat Dilatometer (DMT) and It's Applications to Geotechnical Design
14.485 CAPSTONE DESIGN
Module 4 – Geotechnical Engineering

ELASTIC SETTLEMENT OF SOIL (DMT)

\[ S_e = \sum \frac{\Delta \sigma}{M_{DMT}} \Delta Z \]

Where:
- \( S_e \) = Elastic Settlement
- \( \Delta \sigma \) = Change in Stress
- \( M_{DMT} \) = Constrained Modulus
- \( Z \) = Depth

Figure courtesy of Marchetti (1999) - The Flat Dilatometer (DMT) and It’s Applications to Geotechnical Design
LATERAL EARTH PRESSURES
Coulomb or Rankine Review

Coulomb “Wedge” Theory:
• Accounts for wall friction.
• Unique failure angle for each design.
• Used by National Masonry Concrete Association (NCMA) & USACE.
• Inaccurate passive earth pressures w/large wall angles or friction angle (particularly for $\delta' > \phi' / 2$).
• Decreased accuracy w/ depth.
• Calculates lower active earth pressure than Rankine for level backslope.

Rankine “State of Stress” Theory:
• Does not account for wall friction.
• Requires vertical wall.
• Conservative relative to other methods.
• Fixed plane of failure.
• Favored by the transportation agencies (AASHTO and FHWA).
See AASHTO Standard Specifications for Highway Bridges.
LATERAL EARTH PRESSURES
Coulomb

\[ k_a = \frac{\sin^2 (\theta + \phi'_r)}{\Gamma \left[ \sin^2 \theta \sin(\theta - \delta) \right]} \]

\[ \Gamma = \left[ 1 + \frac{\sin (\phi'_r + \delta) \sin (\phi'_r - \beta)}{\sin (\theta - \delta) \sin (\theta + \beta)} \right]^2 \]

Coulomb

Rankine

\[ K_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} \]

\[ K_a = \tan^2 \left( 45 - \frac{\phi'}{2} \right) \]

\[ K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} \]

\[ K_p = \tan^2 \left( 45 + \frac{\phi'}{2} \right) \]

\[ K_p = \frac{1}{K_a} \]
LATERAL EARTH PRESSURES COULOMB OR RANKINE?

Figure C3.11.5.3-1 – Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design (AASHTO 2012).
REVIEW: RETAINING WALL DESIGN ANALYSES

Overturning

Sliding
(Strength)

Bearing Capacity
(Strength)

Overall Stability
(Service)

Figure 13.4. Das FGE (2005) and Figure C11.6.2.3-1 (AASHTO 2012).

Revised 02/2013
AASHTO (2010) SECTION 11 – ABUTMENTS
11.5.2 – Service Limit States

Abutments, piers, and wall shall be investigated for:

- Excessive vertical and lateral displacements
  - Vertical: Dependent on wall
  - Lateral: < 1.5 inches (C11.5.2)

- Overall Stability
  - Can use Modified Bishop, Simplified Janbu, and Spencer Analysis Methods (11.6.2.3)
Abutments and walls shall be investigated at the strength limit states using Eq. 1.3.2.1-1 for:

- **Bearing Resistance Failure** (i.e. bearing capacity)
- **Lateral Sliding**
- Excessive Loss of Base Contact
- Pullout Failure of anchors and soils reinforcements
- Structural Failure
AASHTO (2010) SECTION 11 – ABUTMENTS

11.5.5 – Load Combinations & Load Factors

Where:

- **DC** = Dead Load of Structural Components
- **DW** = Dead Load of Wearing Surfaces and Utilities
- **EH** = Horizontal Earth Pressure Load
- **ES** = Earth Surcharge Load
- **EV** = Vertical Pressure from Dead Load of Earth Fill
- **WA** = Water Load and Stream Pressure

Figure C11.5.5-2 – Typical Application of Load Factors for Sliding and Eccentricity.
AASHTO (2010) SECTION 11 – ABUTMENTS

11.5.5 – Load Combinations & Load Factors

Where:

- **DC** = Dead Load of Structural Components
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**Figure C11.5.5-2** – Typical Application of Load Factors for Sliding and Eccentricity.
AASHTO (2010) 11 – ABUTMENTS
11.5.5 – Load Combinations & Load Factors

Where:

\[ \text{LS} = \text{Live Load Surcharge} \]

Figure C11.5.5-3 – Typical Application of Live Load Surcharge.
AASHTO (2010)

Section 11 – Abutments

11.6.3.2 – Bearing Resistance (Soil)

\[
\sigma_v = \frac{\Sigma V}{B - 2e}
\]

Where:

- \(V\) = Sum of Vertical Forces
- \(B\) = Footing Width
- \(e\) = Eccentricity

**Figure 11.6.3.2.-1** – Bearing Stress Criteria for Conventional Wall Foundations on Soil.
AASHTO (2010) Section 11 – Abutments
11.6.3.6 – Sliding (refer to 10.6.3.4 – Failure by Sliding)

\[ R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep} \]

Where:
- \( R_R \) = Factored Resistance against Failure by Sliding
- \( R_n \) = Nominal Sliding Resistance
- \( \phi_\tau \) = Shear Resistance Factor (see Table 10.5.5.2.2-1)
- \( R_\tau \) = Nominal Shear Resistance (=\( V \cdot \tan \delta \))
- \( \phi_{ep} \) = Passive Resistance Factor (see Table 10.5.5.2.2-1)
- \( R_{ep} \) = Nominal Passive Resistance
11.6.3.5 – Passive Resistance

• Neglected in Stability Calculations
  • Unless base of the wall extends below the depth of maximum scour, freeze-thaw, or other disturbances

• Neglected is soil providing passive resistance is, or is likely to become, soft, loose, or disturbed, or if contact between the soil and ground is not tight.
INTERFACIAL FRICTION ANGLES (NAVFAC DM7.02)

NOTE:
$\frac{1}{3}\phi' < \delta' < \frac{2}{3}\phi'$

Section 10.6.3.4:
Concrete Cast against Soil:
$\tan \delta = \tan \phi_f$

Precast Concrete Footing:
$\tan \delta = 0.8 \times \tan \phi_f$

### INTERFACE MATERIALS

<table>
<thead>
<tr>
<th>INTERFACE MATERIALS</th>
<th>FRICTION ANGLE, $\delta$ DEGREES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel sheet piles against the following soils:</td>
<td></td>
</tr>
<tr>
<td>clean gravel, gravel-sand mixtures, well-graded</td>
<td>22</td>
</tr>
<tr>
<td>rock fill with spalls</td>
<td></td>
</tr>
<tr>
<td>clean sand, silty sand-gravel mixture, single size</td>
<td>17</td>
</tr>
<tr>
<td>hard rock fill</td>
<td></td>
</tr>
<tr>
<td>Silty sand, gravel or sand mixed with silt or clay...</td>
<td>14</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt</td>
<td>11</td>
</tr>
</tbody>
</table>

Formed concrete or concrete sheet piling against the following soils:
- Clean gravel, gravel-sand mixture, well-graded
- Rock fill with spalls
- Clean sand, silty sand-gravel mixture, single size
- Hard rock fill
- Silty sand, gravel or sand mixed with silt or clay...
- Fine sandy silt, nonplastic silt

Mass concrete on the following materials:
- Clean sound rock
- Clean gravel, gravel-sand mixtures, coarse sand
- Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel
- Clean fine sand, silty or clayey fine to medium sand
- Fine sandy silt, nonplastic silt
- Very stiff and hard residual or preconsolidated clay
- Medium stiff and stiff clay and silty clay

(Masonry on foundation materials has same friction factors.)

Various structural materials:
- Masonry on masonry, igneous and metamorphic rocks:
  - Dressed soft rock on dressed soft rock
  - Dressed hard rock on dressed soft rock
  - Dressed hard rock on dressed hard rock
  - Masonry on wood (cross grain)
  - Steel on steel at sheet pile interlocks