ELASTIC SETTLEMENT OF SHALLOW FOUNDATIONS ON GRANULAR SOIL—A CRITICAL REVIEW

BRAJA M. DAS
Settlement, $S$

- Elastic settlement, $S_e$
- Consolidation settlement
  - Primary, $S_p$
  - Secondary, $S_s$

$$S = S_e + S_p + S_s$$
In his landmark paper in 1927 entitled *The Science of Foundations*, Karl Terzaghi wrote:

“Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists of finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results. By systematically accumulating such knowledge, the empirical data being well defined by the results of adequate soil investigations, foundation engineering could be developed into a semi-empirical science. . . .”

“The bulk of the work—the systematic accumulation of empirical data—remains to be done.”
To evaluate the current state of the art for settlement predictions of shallow foundations in sand, in an attempt to promote the use of shallow foundations.

— A FHWA initiative

1. Federal Highway Administration (FHWA)
2. Texas A & M University
3. Geotest Engineering
4. American Society of Civil Engineers (ASCE)
Texas A&M University
National Geotech Experiment Site
Approximately 12m × 28m

5 Square Footings:
1m × 1m
1.5m × 1.5 m
2.5m × 2.5m
3m × 3m (North)
3m × 3m (South)
PROBLEM: —
Predict the load at 25 mm settlement

**In Situ Test Summary**

- Bore hole shear test — 3
- Cross hole test — 4
- Cone penetration test — 7
- Dilatometer test — 4
- Pressuremeter test — 4
- Step blade test — 1
- Standard penetration test — 6
Number of participants: — 31
15 consultants
16 academics

- Israel – 1
- Japan – 1
- Canada – 2
- Hong Kong – 1
- USA – 21

- Brazil – 1
- France – 1
- Italy – 1
- Australia – 2
Methods Used for Settlement Prediction

—22 different methods—
Schmertmann (1970, 1978), Burland and Burbidge (1985) and FEM being popular

<table>
<thead>
<tr>
<th>Methods Used</th>
<th>Methods Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alpan (3 times)</td>
<td>Menard/Briaud (5)</td>
</tr>
<tr>
<td>Bowles (4)</td>
<td>Meyerhof (4)</td>
</tr>
<tr>
<td>Buisman, DeBeer (3)</td>
<td>NAVFAC (4)</td>
</tr>
<tr>
<td>Burland &amp; Burbidge (9)</td>
<td>Oweis (4)</td>
</tr>
<tr>
<td>Canada Found. Manual (1)</td>
<td>Parry (1)</td>
</tr>
<tr>
<td>D’Appolonia (4)</td>
<td>Peck (2)</td>
</tr>
<tr>
<td>DeBeer (1)</td>
<td>Robertson &amp; Campanella (1)</td>
</tr>
<tr>
<td>Decourt (1)</td>
<td>Schmertmann (18)</td>
</tr>
<tr>
<td>FEM (1)</td>
<td>Schulze &amp; Sherif (3)</td>
</tr>
<tr>
<td>Hanson (1)</td>
<td>Terzaghi &amp; Peck (5)</td>
</tr>
<tr>
<td>Leonard &amp; Frost (4)</td>
<td>Vesic (6)</td>
</tr>
<tr>
<td>Item</td>
<td>Footing (m)</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td></td>
<td>1×1</td>
</tr>
<tr>
<td>Measured</td>
<td>850</td>
</tr>
</tbody>
</table>
Elastic Settlement, $S_e$

Existing methods for predicting settlement may be grouped into three categories:

A — Methods in which observed settlement of structures are linked to *in situ* test results (standard penetration test, cone penetration test, Pressuremeter tests)

B — Semi-empirical method

C — Use of theory of elasticity and modulus of elasticity, $E_s$
CATEGORY A

- Terzaghi and Peck (1948, 1967)
- Meyerhof (1956, 1965)
- DeBeer and Martens (1957)
- Hough (1969)
- Peck and Bazaraa (1969)
- Burland and Burbidge (1985)
Terzaghi and Peck (1948, 1967)

\[
\frac{S_e}{S_{e(1)}} = \frac{4}{\left[1 + \left(\frac{B_1}{B}\right)\right]^2}
\]

*Se* = settlement of prototype foundation measuring *B* × *B*

*Se(1) = settlement of a test plate measuring *B*_1 × *B*_1

➢ *B*_1 is usually of the order of 0.3m to 1m
Data from Bjerrum and Eggestad and Bazaraa

$S_e / S_{e(1)}$

Terzaghi and Peck correlation

$B / B_1$
\[ S_e = C_w C_D \frac{3q}{N_{60}} \left( \frac{B}{B + 0.3} \right)^2 \]

where \( q \) is in kN/m²; \( B \) is in m; \( S \) is in mm

\( C_w \) = ground water table correction  
= 1 if depth of water table is greater than 2\( B \) below foundation  
= 2 if depth of water table is less than or equal to \( B \)

\( C_D \) = correction for depth of embedment  
= 1 – (\( D_f/4B \))
\[
\frac{S_e}{S_{e(1)}} = 4 \left( \frac{B}{B + 0.3} \right)^2
\]
\[
\left( \frac{B}{B + 0.3} \right)^2 = \frac{1}{4} \left( \frac{S_e}{S_{e(1)}} \right)
\]
\[
S_e = \frac{3q}{N_{60}} \left( \frac{B}{B + 0.3} \right)^2
\]
\[
S_e = \frac{3q}{N_{60}} \left( \frac{1}{4} \right) \left( \frac{S_e}{S_{e(1)}} \right)
\]
\[
\frac{q}{S_{e(1)}} = \frac{N_{60}}{0.75}
\]
\[ \frac{q}{S_{e(1)}} = \frac{N_{60}}{0.75} \]

\[ \frac{q}{S_{e(1)}} = \frac{N_{60}}{0.5} \]

Bazaraa (1967)
Meyerhof

1956

\[
S_e (\text{mm}) = \frac{2q(\text{kN/m}^2)}{N_{60}} \quad (B \leq 1.22\text{m})
\]

\[
S_e (\text{mm}) = \frac{3q(\text{kN/m}^2)}{N_{60}} \left( \frac{B}{B + 0.3} \right)^2 \quad (B > 1.22\text{m})
\]

1965

\[
S_e (\text{mm}) = \frac{1.25q(\text{kN/m}^2)}{N_{60}} \quad (B \leq 1.22\text{m})
\]

\[
S_e (\text{mm}) = \frac{2q(\text{kN/m}^2)}{N_{60}} \left( \frac{B}{B + 0.3} \right)^2 \quad (B > 1.22\text{m})
\]

Note: $q$ increased by about 50%
\[ S_e (mm) = C_W C_D \frac{1.25q}{N_{60}} \quad (B \leq 1.22m) \]

and

\[ S_e (mm) = C_W C_D \frac{2q(kN/m^2)}{N_{60}} \left( \frac{B}{B + 0.3} \right)^2 \quad (B > 1.22m) \]

\[ C_W = 1.0 \]

and

\[ C_D = 1.0 - \frac{D_f}{4B} \]
### Meyerhof’s Analysis (1965)

<table>
<thead>
<tr>
<th>Structure</th>
<th>( B ) (m)</th>
<th>( N_{60} )</th>
<th>( q ) (kN/m²)</th>
<th>( \frac{S_e(\text{predicted})}{S_e(\text{observed})} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T. Edison, Sao Paulo</td>
<td>18.3</td>
<td>15</td>
<td>229.8</td>
<td>1.95</td>
</tr>
<tr>
<td>Banco do Brasil, Sao Paulo</td>
<td>22.9</td>
<td>18</td>
<td>239.4</td>
<td>0.99</td>
</tr>
<tr>
<td>Iparanga, Sao Paulo</td>
<td>9.15</td>
<td>9</td>
<td>220.2</td>
<td>1.29</td>
</tr>
<tr>
<td>C.B.I. Esplanada, Sao Paulo</td>
<td>14.6</td>
<td>22</td>
<td>383.0</td>
<td>1.20</td>
</tr>
<tr>
<td>Riscal, Sao Paulo</td>
<td>3.96</td>
<td>20</td>
<td>229.8</td>
<td>1.56</td>
</tr>
<tr>
<td>Thyssen, Dusseldorf</td>
<td>22.6</td>
<td>25</td>
<td>239.4</td>
<td>0.77</td>
</tr>
<tr>
<td>Ministry, Dusseldorf</td>
<td>15.9</td>
<td>20</td>
<td>220.4</td>
<td>0.98</td>
</tr>
<tr>
<td>Chimney, Cologne</td>
<td>20.4</td>
<td>10</td>
<td>172.4</td>
<td>3.30</td>
</tr>
</tbody>
</table>

Average \( \approx 1.50 \)
DeBeer and Martens (1957)

\[ S_e = \frac{2.3}{C} \log_{10} \left( \frac{\sigma'_o + \Delta \sigma}{\sigma'_o} \right) H \]

\( \sigma'_o \) = effective overburden pressure at a depth
\( \Delta \sigma \) = increase in pressure due to foundation loading
\( H \) = thickness of layer considered

\[ C \approx 1.5 \frac{q_c}{\sigma'_o} \]

Field Test Results: \[ \frac{S_{e-predicted}}{S_{e-observed}} \approx 1.9 \]
DeBeer (1965)

- Method applied to normally consolidated sand
- Reduction factor needed for over-consolidated sand
Hough (1969)

\[ S_e = \frac{C_c}{1 + e_o} H \log_{10} \left( \frac{\sigma'_o + \Delta \sigma}{\sigma'_o} \right) \]

\[ C_c = a(e_o - b) \]
<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Value of constant</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$a$</td>
</tr>
<tr>
<td>Uniform cohesionless material (uniformity coefficient $C_u \leq 2$)</td>
<td></td>
</tr>
<tr>
<td>Clean gravel</td>
<td>0.05</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.06</td>
</tr>
<tr>
<td>Medium sand</td>
<td>0.07</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.08</td>
</tr>
<tr>
<td>Inorganic silt</td>
<td>1.00</td>
</tr>
<tr>
<td>Well-graded cohesionless soil</td>
<td></td>
</tr>
<tr>
<td>Silty sand and gravel</td>
<td>0.09</td>
</tr>
<tr>
<td>Clean, coarse to fine sand</td>
<td>0.12</td>
</tr>
<tr>
<td>Coarse to fine silty sand</td>
<td>0.15</td>
</tr>
<tr>
<td>Sandy silt (inorganic)</td>
<td>0.18</td>
</tr>
</tbody>
</table>
Peck and Bazaraa (1969)

\[ S_e (\text{mm}) = C_W C_D \frac{2q (kN/m^2)}{(N_1)_{60}} \left( \frac{B}{B + 0.3} \right)^2 \]

where \( B \) is in m

\((N_1)_{60} = \text{corrected standard penetration number}\)

\(C_W = \sigma'_o / \sigma_o\) at 0.5\(B\) below the bottom of foundation

\(\sigma_o = \text{total overburden pressure}\)

\(\sigma'_o = \text{effective overburden pressure}\)

\(C_D = 1.0 - 0.4(\gamma D/q)^{0.5}\)

\(\gamma = \text{unit weight of soil}\)
Peck and Bazaraa (1969)

\[
(N_1)_{60} = \frac{4N_{60}}{3.25 + 0.01\sigma'_o} \quad (\sigma'_o > 75\text{kN/m}^2)
\]

\[
(N_1)_{60} = \frac{4N_{60}}{1 + 0.04\sigma'_o} \quad (\sigma'_o \leq 75\text{kN/m}^2)
\]
Peck and Bazaraa’s Method
(after D’Appolonia et al. 1970)
GRANULAR SOIL
Burland and Burbidge (1985)

For gravel or sandy gravel
\[ N_{60(a)} \approx 1.25N_{60} \]

For fine sand or silty sand below the ground water and \( N_{60} > 15 \)
\[ N_{60(a)} \approx 15 + 0.6(N_{60} - 15) \]

where \( N_{60(a)} = \) adjusted \( N_{60} \) value
Depth of Stress Influence, $z'$

If $N_{60(a)}$ [or $N_{60(a)}$] is approximately constant (or increasing) with depth,

$$
\frac{z'}{B_R} = 1.4 \left( \frac{B}{B_R} \right)^{0.75}
$$

where

$B_R = \text{reference width} = 0.3 \text{m}$

$B = \text{width of the actual foundation (m)}$
Depth of Stress Influence, $z'$

If $N_{60(a)}$ [or $N_{60(a)}$] is decreasing with depth, calculate $z' = 2B$ and $z' = \text{distance from the bottom of the foundation to the bottom of the soft soil layer (} z'' \text{)}$.

Use $z' = 2B$ or $z' = z''$, whichever is smaller.
Depth of Influence

Correction factor, \( \alpha = \frac{H}{z'} \left( 2 - \frac{H}{z'} \right) \leq 1 \)

\( H = \) thickness of compressible layer
For Normally Consolidated Soil

\[
\frac{S_e}{B_R} = 0.14 \alpha \left\{ \frac{1.71}{[\bar{N}_{60} \text{ or } \bar{N}_{60(a)}]^{1.4}} \right\} \times \left[ \frac{1.25 \left( \frac{L}{B} \right)}{0.25 + \left( \frac{L}{B} \right)} \right]^2 \left( \frac{B}{B_R} \right)^{0.7} \left( \frac{q}{p_a} \right)
\]

where \( L = \) length of the foundation  
\( p_a = \) atmospheric pressure (\( = 100 \text{ kN/m}^2 \))
For Overconsolidated Soil

\( q \leq \sigma'_c; \quad \sigma'_c = \text{overconsolidation pressure} \)

\[
\frac{S_e}{B_R} = 0.447\alpha \left\{ \frac{0.57}{[N_{60} \text{ or } \bar{N}_{60(a)}]^{1.4}} \right\} \times 
\left[ 1.25 \left( \frac{L}{B} \right) \right]^2 \left( \frac{L}{B} \right) \left( \frac{B}{B_R} \right)^{0.7} \left( \frac{q}{p_a} \right)
\]
For Overconsolidated Soil

\((q > \sigma'_c)\):

\[
\frac{S_e}{B_R} = 0.14 \alpha \left\{ \frac{0.57}{[N_{60 \text{ or } N_{60(a)}}]^{1.4}} \right\} \times \\
\left[ \frac{1.25 \left( \frac{L}{B} \right)}{0.25 + \left( \frac{L}{B} \right)} \right]^2 \left( \frac{B}{B_R} \right)^{0.7} \left( \frac{q - 0.67\sigma'_c}{\rho_a} \right)
\]
# Probability of Exceeding 25-mm Settlement in the Field

(After Sivakugan and Johnson 2004)

<table>
<thead>
<tr>
<th>Predicted settlement (mm)</th>
<th>Predicted methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Terzaghi &amp; Peck</td>
</tr>
<tr>
<td>1</td>
<td>0.00</td>
</tr>
<tr>
<td>5</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>0.00</td>
</tr>
<tr>
<td>15</td>
<td>0.09</td>
</tr>
<tr>
<td>20</td>
<td>0.20</td>
</tr>
<tr>
<td>25</td>
<td>0.26</td>
</tr>
<tr>
<td>30</td>
<td>0.31</td>
</tr>
<tr>
<td>35</td>
<td>0.35</td>
</tr>
<tr>
<td>40</td>
<td>0.387</td>
</tr>
</tbody>
</table>
CATEGORY B

- Schmertmann (1970), Schmertmann et al. (1978)
- Briaud (2007)
- Terzaghi, Peck and Mesri (1996)
- Akbas and Kulhawy (2009)
Schmertmann (1970)

\[ \varepsilon_z = \frac{q(1 - \mu_s)}{E_s}[(1 - 2\mu_s)A' + B'] \]

\[ l_z = \frac{\varepsilon_z E_s}{q} = (1 - \mu_s)[(1 - 2\mu_s)A' + B'] \]
\[ S_e = C_1 C_2 q \sum \frac{l_z}{E_s} \Delta z \]

\( q \) = net stress at the level of the foundation

\( C_1 \) = correction factor for the depth of the foundation

\[ = 1 - 0.5 \left( \frac{q_o}{q} \right) \]

\( q_o \) = effective overburden pressure at the level of the foundation

\( C_2 \) = correction factor to account for creep in soil

\[ = 1 + 0.2 \log \left( \frac{t}{0.1} \right) \]

\( E_s \) = 2\( q_c \)
The same 79 foundations records given by Jeypalan and Boehm (1986) and Papadopoulos (1992) were analyzed by Sivakugan et al. (1998).
Schmertmann method
79 points
$R^2 = 0.345$
Schmertmann et al. (1978)

\[
I_z(\text{peak}) = 0.5 + 0.1 \left( \frac{q}{\sigma'_o} \right)^{0.5}
\]

<table>
<thead>
<tr>
<th>Item</th>
<th>$L/B = 1$</th>
<th>$L/B \geq 10$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_z$ at $z = 0$</td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>$z_p / B$</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>$z_o / B$</td>
<td>2.0</td>
<td>4.0</td>
</tr>
<tr>
<td>$E_s$</td>
<td>$2.5q_c$</td>
<td>$3.5q_c$</td>
</tr>
</tbody>
</table>
Salgado (2008)

\[ I_z (\text{at } z=0) = 0.1 + 0.0111 \left( \frac{L}{B} \right) \leq 2 \]

\[ \frac{z_p}{B} = 0.5 + 0.0555 \left( \frac{L}{B} - 1 \right) \leq 1 \]

\[ \frac{z_o}{B} = 2 + 0.222 \left( \frac{L}{B} - 1 \right) \leq 4 \]
Lee et al. (2008)
FEM Analysis

\[ I_{z(peak)} \approx 0.5 \]

\[ \frac{Z_p}{B} = 0.5 + 0.11 \left[ \left( \frac{L}{B} \right) - 1 \right] \leq 1 \]
with a maximum of 1 at \( \frac{L}{B} = 6 \)

\[ \frac{Z_o}{B} = 0.95 \cos \left\{ \frac{\pi}{5} \left( \frac{L}{B} - 1 \right) \right\} - \pi \right\} + 3 \leq 4 \]
with a maximum of 4 at \( \frac{L}{B} = 6 \)
Terzaghi et al. (1996)

\[ z_o = 2 \left[ 1 + \log \left( \frac{L}{B} \right) \right] \leq 4 \]
\[ S_e = q \sum_{z=0}^{z=z_o} \frac{I'_z}{E_s} \Delta z \]

\[
\frac{E_s(L/B)}{E_s(L/B=1)} = 1 + 0.4 \log \left( \frac{L}{B} \right) \leq 1.4
\]

\[ E_s(L/B=1) = 3.5q_c \]

\[ S_e(\text{creep}) = \left( \frac{0.1}{q_c} \right) z_o \log \left( \frac{t \text{ days}}{1 \text{ day}} \right) \]

\[ \bar{q}_c = \text{weighted mean value } q_c \text{ (in MN/m}^2) \]
81 Foundation and 92 Plate Load Tests

\[ E_s (MN/m^2) = 3.5 q_c \]

Terzaghi et al. (1996)

- Sands
- Gravelly soils
Load-Settlement Curve
Based on Pressuremeter Test
Briaud (2007)
Pressuremeter Test
\[ p_{\text{mean}} = \frac{A_1}{A} p_{p(1)} + \frac{A_2}{A} p_{p(2)} + \frac{A_3}{A} p_{p(3)} + \ldots \]
\[ q = \Gamma[f_{(L/B)}, f_e, f_\delta, f_{\beta_d}]p_{p(\text{mean})} \]

\[ \frac{S_e}{B} = 0.24 \frac{\Delta R}{R} \]

\[ \Gamma = \text{gamma function} \]
SHAPE FACTOR

\[ f_{(L/B)} = 0.8 + 0.2 \frac{B}{L} \]

ECCENTRICITY FACTOR

\[ f_e = 1 - 0.33 \left( \frac{e}{B} \right) \quad \text{……Center} \]

\[ f_e = 1 - \left( \frac{e}{B} \right)^{0.5} \quad \text{……Edge} \]
LOAD INCLINATION FACTOR

\[ f_\delta = 1 - \left( \frac{\delta \text{ (deg)}}{90} \right)^2 \]  \ldots\ldots\text{Center}

\[ f_\delta = 1 - \left( \frac{\delta \text{ (deg)}}{360} \right)^{0.5} \]  \ldots\ldots\text{Edge}

SLOPE FACTOR

\[ f_{\beta,d} = 0.8 \left( 1 + \frac{d}{B} \right)^{0.1} \]  \ldots\ldots3:1\text{ slope}

\[ f_{\beta,d} = 0.7 \left( 1 + \frac{d}{B} \right)^{0.15} \]  \ldots\ldots2:1\text{ slope}
Long-term settlement, including creep =

\[ S_e \left( \frac{t}{t_1} \right)^{0.03} \]

\[ t = \text{design life (in hours)} \]
\[ t_1 = 1 \text{ hour} \]
Akbas and Kulhawy (2009)

$L_1 - L_2$ Method

37 Sites

167 Axial compression field load tests
Mean $S_{e(L1)} = (0.23\%)B$

Mean $S_{e(L2)} = (5.39\%)B$
\[
\frac{Q}{Q_{l2}} = \frac{\frac{S_e}{B}}{0.69 \left( \frac{S_e}{B} \right) + 1.68}
\]
\[ B > 1 \text{ m} \]

\[ Q_{L2} = \text{ultimate bearing capacity } Q_u \text{ (Vesic 1973, 1975)} \]

\[ B \leq 1 \text{ m} \]

\[
Q_{L2} = \left( \frac{Q_u^\gamma}{B} \right) + Q_u^q
\]

\[ Q_u^\gamma = N_\gamma \text{ portion of Vesic's theory} \]

\[ Q_u^q = N_q \text{ portion of Vesic's theory} \]
Note: Vesic’s theory includes compressibility factor. So

\[ Q_u = f(\mu, E_s, \phi', \gamma, B) \]

Proper assumption of \( E_s \) and \( \phi' \) is needed.
Use of Theory of Elasticity and Modulus of Elasticity

\[ S_e = q_o (\alpha B') \left( 1 - \frac{\mu_s^2}{E_s} \right) I_s I_f \]

- \( E_s \) = average modulus of elasticity (\( z = 0 \) to \( z = 4B \))
- \( B' = B/2 \)
- \( \mu_s \) = Poisson’s ratio
Steinbrenner (1934)

\( l_s = \text{shape factor} = f(m, n) \)

For \( S_e \) at the \text{center} : \( \alpha = 4 \)

\[ m = \frac{L}{B} \]

\[ n = \frac{H}{\left( \frac{B}{2} \right)} \]
Fox (1948)

\[ I_f = \text{depth factor} = f \left( \frac{D_f}{B}, \frac{L}{B} \text{ and } \mu_s \right) \]

\[ S_{e(\text{rigid})} \approx 0.93 S_{e(\text{flexible, center})} \]
# Variation of $I_f$

<table>
<thead>
<tr>
<th>$D_f / B$</th>
<th>$L / B$</th>
<th>( \mu_s = 0.30 )</th>
<th>( \mu_s = 0.40 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.902</td>
<td>0.930</td>
<td>0.951</td>
</tr>
<tr>
<td>0.40</td>
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<td>0.857</td>
<td>0.899</td>
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<td>0.796</td>
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<tr>
<td>2.00</td>
<td>0.596</td>
<td>0.640</td>
<td>0.714</td>
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</table>
Bowles (1987)

Weighted average, \( E_s = \frac{\sum E_{s(i)} \Delta z}{z} \)

\( z = H \) or \( 4B \), whichever is smaller

\( E_s = 500(N_{60} + 15) \) kN/m\(^2\)
Mayne and Poulos (1999)

Compressible soil layer $E_s, \mu_s$

$E_s = E_o + kz$

Rigid layer

Depth $z$
\[ E_s = E_o + kz \]

\[ S_e = \frac{q B_e I_G I_R I_E}{E_o} (1 - \mu_s^2) \]

\[ I_G = \text{influence factor} = f \left( \beta = \frac{E_o}{kB_e}, \frac{H}{B_e} \right) \]

\[ I_R = \text{foundation rigidity correction factor} \]

\[ I_E = \text{foundation embedment correction factor} \]

\[ B_e = \left( \frac{4BL}{\pi} \right)^{0.5} \]
\[ l_R = \frac{\pi}{4} + \frac{1}{4.6 + 10 \left( \frac{E_f}{E_o + \frac{B_e}{2} k} \right) \left( \frac{2t}{B_e} \right)^3} \]

\[ l_E = 1 - \frac{1}{3.5 \exp(1.22 \mu_s - 0.4) \left( \frac{B_e}{D_f} + 1.6 \right)} \]
\[ K_F = \left( \frac{E_f}{E_o + \frac{B_0}{2} k} \right) \left( \frac{2t}{B_e} \right)^3 = \text{Flexibility factor} \]
Berardi and Lancellotta (1991)

\[ S_e = I_s \frac{qB}{E_s} \]

\( I_s = \) influence factor for a rigid foundation \( (\mu_s = 0.15) \)

\( (\text{Tsytovich, 1951}) \)

<table>
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<tr>
<th>( \frac{L}{B} )</th>
<th>( H_1/B )</th>
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<td>5</td>
<td>0.41</td>
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<td>10</td>
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</table>
Berardi and Lancellotta (1991) re-analyzed field performance of 130 structures on predominantly silica sand as reported by Burland and Burbidge

\[ E_s = K_E \rho_a \left( \frac{\sigma'_o + 0.5 \Delta \sigma'}{\rho_a} \right)^{0.5} \]  

(Janbu, 1963)

\( \rho_a = \) atmospheric pressure

\( \sigma'_o \) and \( \Delta \sigma' \) at a depth \( B/2 \) below the foundation
$K_E = f\left(\bar{N}_1\right)_{60}$

$\left(\bar{N}_1\right)_{60} = \text{average corrected standard penetration number in the influence zone}$

Influence zone for square foundation:

$H_{15} = (1.2 \text{ to } 2.8)B$

$H_{25} = (0.8 \text{ to } 1.3)B$

Influence zone for $L/B \geq 10$:

$H_{15} \approx (1.8 \text{ to } 2.4)B$

$H_{25} \approx (1.2 \text{ to } 2.0)B$
Skempton (1986)

\[(N_1)_{60} = N_{60} \left( \frac{2}{1 + 0.01\sigma'_o} \right) \]

\(\sigma'_o\) is in kN/m²

\[\frac{(N_1)_{60}}{D_r^2} = 60\]
Procedure for Calculating $S_E$
Berardi and Lancellotta (1991)

1. Obtain the variation of $N_{60}$ within the influence zone (i.e., $H_{25}$).
2. Obtain $(N_1)_{60}$ within the influence zone.
3. Obtain $(\overline{N}_1)_{60}$.
4. Obtain $K_E$ at $S_e/B = 0.1\%$.
5. Calculate $E_s = K_E \rho_a \left( \frac{\sigma_o + 0.5\Delta \sigma'}{\rho_a} \right)^{0.5}$. 
Procedure for Calculating $S_E$ (continued)

6. Determine $I_s$.
7. Use an equation from theory of elasticity to calculate $S_e$.
8. Calculate $S_e/B$. Is it equal to assumed $S_e/B$?
9. If so, the calculated $S_e$ in Step 7 is the answer.
10. If not, use $S_e/B$ from Step 8 to obtain the new $K_E$.
11. Repeat Steps 5, 7 and 8 until the assumed and calculated $S_e/B$ are equal.
Settlement Prediction in Granular Soils — A Probabilistic Approach

<table>
<thead>
<tr>
<th>Method</th>
<th>Probability of exceeding 25 mm in the field</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi &amp; Peck (1948)</td>
<td>0.26 (26%)</td>
</tr>
<tr>
<td>Schmertmann (1970)</td>
<td>0.27 (27%)</td>
</tr>
<tr>
<td>Burland &amp; Burbidge (1985)</td>
<td>0.42 (42%)</td>
</tr>
<tr>
<td>Berardi &amp; Lancellotta (1991)</td>
<td>0.52 (52%)</td>
</tr>
</tbody>
</table>
COMMENTS AND CONCLUSIONS

1. Meyerhof’s relations (1965) simple to use. On the average, will give \( \frac{S_e(\text{predicted})}{S_e(\text{observed})} \approx 1.5 \) to 2.0.


Difficult to estimate overconsolidation pressure from field exploration.
4. Modified strain influence factor methods of Schmertmann et al. (1978), Terzaghi et al. (1996), Salgado (2008) and Lee et al. (2008) will give reasonable results with proper values of $E_s$.

5. Suggested $E_s$ relations:

$$\frac{E_s(L/B)}{E_s(L/B=1)} = 1 + 0.4\log\left(\frac{L}{B}\right) \leq 1.4$$

$$E_s(L/B=1) = 3.5q_c$$

6. The $E_s(L/B = 1)$ relationship can be related to $N_{60}$ via $D_{50}$. 
7. Pressuremeter method of developing load-settlement relationship is very effective, but may not be cost effective.

8. \( L_1 - L_2 \) (Akbas and Kulhawy) is a good method. However proper assumption of \( E \) and \( \phi' \) needed to estimate \( Q_{L2} \).

9. Relationships for settlement developed using theory of elasticity will give equally good results provided a realistic \( E_s \) is used.
   - Use of iteration method is suggested.
   - If not, used Terzaghi et al.’s relationship (1996).
Anagnostopoulos et al. (2003)

Robertson et al. (1983)

\[
\frac{q_c / p_a}{N_{60}} = 7.6429 D_{50}^{0.26}
\]
What we have seen is a systematic accumulation of knowledge over 60 years. The parameters for comparing settlement prediction methods are **accuracy** and **reliability**.

Reliability is the probability that the actual settlement would be less than that computed by a specific method.
In choosing a method for design, it all comes down to keeping a critical balance between reliability and accuracy, which can be difficult at times, knowing the non-homogeneous nature of soil in general. We cannot be over-conservative but, at the same time, not be accurate.

We need to keep in mind what Karl Terzaghi said in the 45th James Forrest Lecture at the Institute of Civil Engineers in London: “Foundation failures that occur are no longer ‘an act of God’.”