06b Settlement in Sand and Bearing Capacity

Ref:  
Principles of Geotechnical Engineering, Braja M. Das, 1994  
Dr. D. Bloomquist Notes

Topics:  
Settlement in Sand  
Total Settlement & Change of Settlement with Time  
Mechanical Properties  
Shear Strength  
Degree of Permeability  
Elastic Constants  
Blow Count  
Examples  
Bearing Capacity Summary

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Settlement in Sand

- **Considerations:**
  - Settlement occurs very rapidly (no primary consolidation)
  - Settlement is typically small
  - Creep normally not a factor (no secondary compression)
  - Sand stiffness strongly dependent on stress, both vertical and horizontal
  - Sand stiffness increases with depth (stress) and classical linear elastic models do not perform well.
  - In situ properties can vary dramatically (violent deposition environment)
  - Sands difficult to sample and test undisturbed in the lab
  - Need method which measures stiffness in situ

\[ \rho = \rho_i + \rho_c + \rho_s = \rho_i \]

- **Possible Settlement Methods for Sands:**
  - Use immediate settlement method from clays - relatively poor results
  - Plate Load Tests - must transfer to larger footing area
  - Standard Penetration Methods (SPT) - lots of variability, unreliable precision of field data, need correlation of modulus to \( N_{SPT} \) (blow count)
  - Dilatometer Test (DMT) - flat plate, direct measurement of modulus, Boussinesq stress increase, calculate strain - good method, relatively new
  - Cone Penetrometer Test (CPT) - measurement of bearing on tip of cone, correlate with modulus, use strain factor method, can be conservative but good field data precision
• Schmertmann Strain Factor Method (CPT):
  o Widely used in practice today to estimate settlement under center of footing.
  o Mixture of rational & empirical basis
  o Typically conservative (overestimate settlement)
  o Backed by field and model observations
  o Originally proposed 1970, based on UF research
  o Revised 1978 for spread & strip footings, time effects, and updated E vs. q_c correlation
  o Method uses linear-elastic theory for a half-space
  o Effect is usually negated by the disturbance of the soil by the construction process

\[
\rho = C_1 C_2 \Delta \sigma \sum_{i=1}^{n} \left( \frac{I_{z_i}}{E_i} \right) \Delta z_i
\]

\[
\rho_i = \text{(immediate) settlement}
\]

\[
C_1 = \text{correction factor for depth of embedment} = 1 - 0.5 \left( \frac{\sigma'_0}{\Delta \sigma'_Z} \right) \geq 0.5
\]

\[
C_2 = \text{correction factor for secondary creep settlement} = 1 + 0.2 \log_{10} \left( \frac{t_{yr}}{0.1} \right)
\]

\[
\Delta \sigma'_Z = \text{net foundation pressure increase at footing bottom} = (q - \sigma'_0)
\]

\[
\sigma'_0 = \text{effective stress at footing bottom before any excavation}
\]

\[
I_{z_i} = \text{Strain influence factor at mid-height of each sub-layer from idealized distribution shown below (non-dimensional)}
\]

\[
E_i = \text{Young’s modulus for each sub-layer (estimated from CPT)}
\]

\[
= 2.5 q_{CI} \text{ for axisymmetric footings (L/B = 1)}
\]

\[
= 3.5 q_{CI} \text{ for strip footings (L/B > 10)}
\]

\[
q_{CI} = \text{cone tip resistance (average assigned to each sub-layer)}
\]

\[
\Delta z_i = \text{height of each sub-layer}
\]

\[
t_{yr} = \text{time in years after placement of footing}
\]
Idealized $I_Z$ Distribution

$I_{zp} = \text{maximum } I_Z = 0.5 + 0.1(\Delta \sigma_z/\sigma_0')^{1/2}$

Axisymmetric ($L/B = 1$)
- $I_Z = 0.1$ at $z = 0$
- $I_Z = I_{zp}$ at $z = 0.5B$
- $I_Z = 0$ at $z = 2B$

Plane Strain ($L/B > 10$)
- $I_Z = 0.2$ at $z = 0$
- $I_Z = I_{zp}$ at $z = B$
- $I_Z = 0$ at $z = 4B$

B/2 (axisym.)

$\Delta \sigma_z = q - \sigma_0'$

$\sigma_0'$

$\sigma_0$

Procedure:

Given: $L$, $B$, $q$, $\sigma_0'$

**Step 1** Develop Strain Influence Distribution

a. determine $I_z$ at $z = 0$ → interpolate between $L/B = 1$ and 10

$L/B = 1 \rightarrow I_{z1} = 0.1$

$L/B = 10 \rightarrow I_{z10} = 0.2$

$$I_z = I_{z1} + \left(\frac{L}{B} - 1\right) \frac{I_{z10} - I_{z1}}{10 - 1} = 0.1 + \left(\frac{L}{B} - 1\right) \frac{0.1}{9}$$

b. determine depth of $I_{zp} \rightarrow$ interpolate between $L/B = 1$ and 10

$L/B = 1 \rightarrow z_1 = 0.5B$

$L/B = 10 \rightarrow z_{10} = 1.0B$

$$z = z_1 + \left(\frac{L}{B} - 1\right) \frac{z_{10} - z_1}{10 - 1} = B \left[0.5 + \left(\frac{L}{B} - 1\right) \frac{0.5}{9}\right]$$

c. determine $I_{zp}$

- calculate $\sigma'_{zp} = \sigma_{zp} - u$,
  - where $\sigma_{zp} = \gamma_{sat}ZB$ (fully saturated), $u = \gamma_wZB$

- calculate $\Delta \sigma_z' = q - \sigma_0'$

$$I_{zp} = 0.5 + 0.1 \sqrt{\frac{\Delta \sigma_z'}{\sigma_{zp}'}}$$

d. determine depth of influence $\rightarrow$ interpolate between $L/B = 1$ and 10

$L/B = 1 \rightarrow z_1 = 2B$

$L/B = 10 \rightarrow z_{10} = 4B$

$$z = z_1 + \left(\frac{L}{B} - 1\right) \frac{z_{10} - z_1}{10 - 1} = B \left[0.5 + \left(\frac{L}{B} - 1\right) \frac{2}{9}\right]$$
Step 2  Define layers with uniform \( q_c \)

Step 3  Determine \( I_z \) and \( E \) for each layer, calculate settlement at each layer and sum. Do calculation for each layer at mid-height. \( E \) is determined from \( q_c \) (cone resistance \( \rightarrow \) lab test)

Notes on Schmertmann Method:
- Interpolate idealized \( I_z \) distribution for footings with \( 1 < L/B < 10 \):
  - initial \( I_z \) 0.1 \( \rightarrow \) 0.2
  - depth of \( I_{ZP} \) 0.5B \( \rightarrow \) B
  - maximum depth of \( I_z \) influence 2B \( \rightarrow \) 4B
- Interpolate modulus, \( E \), for footings with \( 1 < L/B < 10 \):
  - \( L/B = 1 \rightarrow E = 2.5q_c \)
  - \( L/B = 10 \rightarrow E = 3.5q_c \)
  - \( \frac{E}{q_c} = 2.5 + \left( \frac{L}{B} - 1 \right) \frac{3.5 - 2.5}{10 - 1} = 2.5 + \left( \frac{L}{B} - 1 \right) \frac{1}{9} \)
- Developed for NC sands - generally conservative if sand is preloaded, compacted, etc.
- Applies to static loading only, dynamic loads can create large pore pressures and significantly decrease effective stresses leading to large deflections
- Can use \( N_{SPT} \) to estimate \( q_c \) if no CPT data available. This is a common and generally conservative procedure, but there is significant uncertainty due to the variability of the SPT (some prefer to use \( q_c \) to estimate \( N \)). The ratio \( (q_c/N_{SPT}) \) increases as the mean grain size increases.

\( N_{60} \) blows/ft or blows/0.3 m (60% energy), \( q_c \) in bars (1 bar = 1.044 tsf = 100 kPa)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>( q_c / N_{60} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sensitive Clays</td>
<td>2</td>
</tr>
<tr>
<td>Organic Soils</td>
<td>1</td>
</tr>
<tr>
<td>Clay</td>
<td>1</td>
</tr>
<tr>
<td>Clay to Silty Clay</td>
<td>1.5</td>
</tr>
<tr>
<td>Silty Clay to Clayey Silt</td>
<td>2</td>
</tr>
<tr>
<td>and Clay-Silt-Sand mix</td>
<td></td>
</tr>
<tr>
<td>Clayey Silt to Sandy Silt</td>
<td>2.5</td>
</tr>
<tr>
<td>Sandy Silt to Silty Sand</td>
<td>3</td>
</tr>
<tr>
<td>Silty Sand to Sand</td>
<td>4</td>
</tr>
<tr>
<td>Sand</td>
<td>5</td>
</tr>
<tr>
<td>Sand to Gravelly Sand</td>
<td>6</td>
</tr>
</tbody>
</table>

Note that cemented sands (common in FL) can have \( (q_c/N_{60}) > 10 \)
Mechanical Properties of Soil

Shear Strength - obtained from soil tests, formula for saturated soil
\[ \tau = c + \sigma' \tan \phi \]

\( \tau \) = shear strength
\( c \) = cohesion strength (undrained shear strength)
\( \sigma' \) = effective stress, \( \sigma' = \sigma - u = \gamma' H \)
\( \phi \) = angle of internal friction (angle of repose)

from Das (1994)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>( \phi ) (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>27-35</td>
</tr>
<tr>
<td>Medium sand</td>
<td>30-40</td>
</tr>
<tr>
<td>Dense sand</td>
<td>35-45</td>
</tr>
<tr>
<td>Gravel with some sand</td>
<td>34-48</td>
</tr>
<tr>
<td>silt</td>
<td>26-35</td>
</tr>
</tbody>
</table>

Cohesionless soil (80% or more sand), \( c = 0 \) \( \Rightarrow \) \( \tau_f = \sigma' \tan \phi \)

Cohesive soil, assume \( \phi = 0 \),
Cohesion strength \( (c) \) for clays from unconfined compression strength,
(Das 1994)

<table>
<thead>
<tr>
<th>Consistency</th>
<th>ton/ft(^2)</th>
<th>kN/m(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>0 - 0.5</td>
<td>0 - 48</td>
</tr>
<tr>
<td>Soft</td>
<td>0.5 - 1</td>
<td>48 - 96</td>
</tr>
<tr>
<td>Medium</td>
<td>1 - 2</td>
<td>96 - 192</td>
</tr>
<tr>
<td>Stiff</td>
<td>2 - 4</td>
<td>192 - 384</td>
</tr>
<tr>
<td>Very stiff</td>
<td>4 - 8</td>
<td>384 - 766</td>
</tr>
<tr>
<td>hard</td>
<td>&gt; 8</td>
<td>&gt; 766</td>
</tr>
</tbody>
</table>

For normally consolidated clays: \( c_u/p_o = 0.11 + 0.0037(LL - PL) \)

Modulus of Elasticity \( (E) \) (Das, 1994)

<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Modulus of Elasticity ( (E) ) (psi)</th>
<th>Modulus of Elasticity ( (E) ) (kN/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>250-500</td>
<td>1380-3450</td>
</tr>
<tr>
<td>Hard clay</td>
<td>850-2000</td>
<td>5865-13,800</td>
</tr>
<tr>
<td>Loose sand</td>
<td>1500-4000</td>
<td>10,350-27,600</td>
</tr>
<tr>
<td>Dense sand</td>
<td>5000-10,000</td>
<td>34,500-69,000</td>
</tr>
</tbody>
</table>
Degree of Permeability

<table>
<thead>
<tr>
<th>Degree of permeability</th>
<th>k (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>&gt; 10^{-1}</td>
</tr>
<tr>
<td>Medium</td>
<td>10^{-3} - 10^{-1}</td>
</tr>
<tr>
<td>Low</td>
<td>10^{-5} - 10^{-3}</td>
</tr>
<tr>
<td>Very low</td>
<td>10^{-7} - 10^{-5}</td>
</tr>
<tr>
<td>Practically impermeable</td>
<td>&lt; 10^{-7}</td>
</tr>
</tbody>
</table>

Blow Count (N, blows/ft or blows/30 cm)

N is the average blows per foot in the stratum, number of blows of a 140-pound hammer falling 30 inches to drive a standard sampler (1.42" I. D., 2.00" O. D.) one foot. The sampler is driven 18 inches and blows counted the last 12 inches.

Indicates soil strength***

<table>
<thead>
<tr>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>N</td>
</tr>
<tr>
<td>Very loose</td>
<td>0-4</td>
</tr>
<tr>
<td>Loose</td>
<td>4-10</td>
</tr>
<tr>
<td>Medium</td>
<td>10-30</td>
</tr>
<tr>
<td>Dense</td>
<td>30-50</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt;50</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 T/m² = 0.1 tons/ft²

*** Blow Count or the Standard Penetration Test is standard in the U.S., but should only be used in sandy soil (NOT clays). Clays tend to have erroneously high blow counts when tested in place due to the inability of water to drain out (i.e. the test is on water pressure, not soil strength.

A better test is the Static Cone Test in which an instrumented sensor is continuously driven into the soil and sends data back to a computer.
Examples

Units: 1 t ~ 1000 kgf ~ 10 kN, (t = metric ton)
   1 lb/ft² = 47.88 N/m²
   1 psi = 6.9 kN/m²
   1 lb/ft² = 0.016 t/m³

for unsaturated soil with S=40%, \( \gamma' = 1.8 \) t/m³
for saturated soils, \( \gamma = 2.0 \) t/m³
for quartz sand, \( G = 2.65, n = 38\% = 0.38 \)
for water, \( FW \gamma_w = 62.4 \) lb/ft³ = 0.998 t/m³
   \( SW \gamma_w = 64 \) lb/ft³ = 1.02 t/m³
** can use \( \gamma_w = 1.0 \) t/m³ for most calculations
Ultimate Bearing Capacity of the Soil

Two aspects for design:
- Settlement
- Bearing Capacity

**Bearing Capacity** (definition): ability of the soil to safely carry the pressure placed on the soil by any engineered structure without undergoing a shear failure with accompanying large settlements. A safe bearing pressure with respect to failure does not ensure that settlement will be within acceptable limits. Must conduct settlement analysis.

**Procedure (USACE)**
1. Evaluate the ultimate bearing capacity pressure $q_u$
2. Determine a reasonable factor of safety (FS) based on available subsurface surface information, variability of the soil, soil layering and strengths, type and importance of the structure and past experience. FS will typically be between 2 and 4. (marine applications 1.5-2.5)
3. Evaluate allowable bearing capacity $q_a$ by dividing $q_u$ by FS; i.e., $q_a = q_u / FS$
4. Perform settlement analysis when possible and adjust the bearing pressure until settlements are within tolerable limits. The resulting design bearing pressure $q_d$ may be less than $q_a$. Settlement analysis is particularly needed when compressible layers are present beneath the depth of the zone of a potential bearing failure. Settlement analysis must be performed on important structures and those sensitive to settlement.

**Bearing Capacity Design Criteria:**
- **Bearing Stress** ($q_s$) - the bearing stress actually applied to the soil by a foundation, force per unit area ($Q_s/A$).
- **Allowable Bearing Stress** ($q_a$) - the bearing stress used as a design limit after consideration of stability, failure criteria, soil layering and variability, influence of other structures/footings, and risk tolerance - typically divide the ultimate bearing capacity by a factor of safety (F.S. $\approx$ 3).
- **Local Shear Bearing Capacity** ($q_{ls}$) - the bearing stress at which local shear failure occurs, typically where the bearing stress vs. movement plot becomes significantly nonlinear.
- **Ultimate Bearing Capacity** ($q_u$) - the bearing stress at which there is catastrophic movement, usually a general shear failure.
Factors Affecting Mode of BC Failure:
- Depth of embedment, $D_r$
- Stiffness or relative density, $D_r = \left( \frac{e_{\max} - e}{e_{\max} - e_{\min}} \right) \times 100\%$
- Geometry of foundation (B/L), shape
- Inclination or eccentricity of applied load

BC Failure Modes:
- *General shear failure* ($q_u$) - abrupt, sudden. Failure surface extends to ground surface (dense sand).

- *Local shear failure* ($q_{ls}$) - occurs slowly, with substantial settlement. Failure surface does not extend to ground surface. Progresses to general shear failure (medium compacted sand, clayey soil)

- *Punching shear failure* ($q_{ps}$) - continuous punching failure and settlement with gradual increase in $q_s$ due to compaction (loose sands), more likely to occur at depth.
Estimate type of failure from geometry & $D_f$:
- general shear failure, $q_u$ will occur at
  $\Delta = (4\text{–}10\%) \times B$
- local shear failure (or punching), $q_u$ will occur at $\Delta = (15\text{–}25\%) \times B$

(figure by Vesic, 1963)

**Terzaghi Bearing Capacity (1943):** B.C. still a real problem in Terzaghi’s era, Prandtl B.C. (1920) for metals assumed weight forces small compared to material strength
- Depth of foundation $\leq$ width (i.e. $D_f \leq B$)
- Rough bottom, foundation does not slide
- homogeneous, semi-infinite, isotropic soil mass
- Mohr-Coulomb failure criteria, $\tau = c + \sigma \tan \phi$, (usu. effective stress analysis w/ $c'$ & $\phi'$)
- General shear failure mode
- Movement due only to shear, no settlement
- Rigid foundation in comparison to soil stiffness
- Soil above bottom of footing acts as surcharge only and has no strength
- Applied load vertical, in compression, through footing centroid, no moment
- Radial shear zone, governed by passive pressure
- Started with plane strain (strip footing), then extended to square & round footings

Superimpose effects of $c$, $\gamma$, $q$: $q_u = q_c + q_q + q_\gamma$

Ultimate Bearing Capacity ($q_u$) Calculation (Terzaghi’s Ultimate Bearing Capacity Equation)
For saturated, submerged soils
- strip foundations: $q_u = q_c + q_q + q_\gamma = cN_c + qN_q + 0.5\gamma'BN_\gamma$
- circular foundations $q_u = q_c + q_q + q_\gamma = 1.3cN_c + qN_q + 0.3\gamma'BN_\gamma$
- square foundations $q_u = q_c + q_q + q_\gamma = 1.3cN_c + qN_q + 0.4\gamma'BN_\gamma$
\( q_c, q_{\phi}, q_{\gamma} \) = load contributions from cohesion, soil weight and surcharge
\( N_c, N_q, N_{\gamma} \) = bearing capacity factors for cohesion, soil weight and surcharge
\( c \) = cohesion strength of soil
\( q \) = soil weight, \( q = \gamma' D \)

\( \gamma' \) = effective bulk density of soil (\( \gamma' = \gamma - \gamma_w = \frac{G - 1}{1 + e} \gamma_w \))

\( B \) = width of the foundation
\( D \) = the depth of penetration of the foundation

From *Handbook of Coastal Engineering*, vol 2, ch 7 (A. G. Young), 1991


\[
N_q = e^{\pi \tan \phi} \tan^2 (45 + \phi / 2)
\]
\[
N_{\gamma} = 2(N_q + 1) \tan \phi
\]
\[
N_c = (N_q - 1) \cot \phi, \phi > 0
\]
\[
N_c = \pi + 2 = 5.14, \phi = 0, \text{clay}
\]
for deep foundations \( N_c \approx 9 \)

alternate bearing factors (Dr. Bloomquist notes)

\[
N_q = \frac{a_0^2}{2 \cos^2 (45 + \phi / 2)}, \quad a_0 = e^{(0.75 \pi - \phi) \tan \phi} \Leftrightarrow (\phi \text{ in radians})
\]
\[
N_{\gamma} \approx \frac{2(N_q + 1) \tan \phi}{1 + 0.4 \sin (4\phi)} \text{ (Coduto)}
\]
\[
N_c = \frac{N_q - 1}{\tan \phi}, \phi > 0
\]
\[
N_c = 5.70 \text{ for } \phi = 0
\]

alternate bearing factors: See table

Allowable bearing capacity \( (q_a) \)

\[
q_a = \frac{q_u}{FS}, \text{ essentially the allowable load of the structure}
\]

The above assumes homogeneous soil. Layered profiles will require more complex analysis.
| Friction Angle $\phi^\circ$, degrees | Terzaghi | | | Meyerhof | | | |
| | $N_c$ | $N_q$ | $N_r$ | $N_c$ | $N_q$ | $N_r$ |
| 0 | 5.70 | 1.00 | 0.00 | 5.14 | 1.00 | 0.00 |
| 1 | 6.00 | 1.10 | 0.01 | 5.38 | 1.09 | 0.00 |
| 2 | 6.30 | 1.22 | 0.04 | 5.63 | 1.20 | 0.01 |
| 3 | 6.62 | 1.35 | 0.06 | 5.90 | 1.31 | 0.02 |
| 4 | 6.97 | 1.49 | 0.10 | 6.19 | 1.43 | 0.04 |
| 5 | 7.34 | 1.64 | 0.14 | 6.49 | 1.57 | 0.07 |
| 6 | 7.73 | 1.81 | 0.20 | 6.81 | 1.72 | 0.10 |
| 7 | 8.15 | 2.00 | 0.27 | 7.16 | 1.88 | 0.15 |
| 8 | 8.60 | 2.21 | 0.35 | 7.53 | 2.06 | 0.20 |
| 9 | 9.09 | 2.44 | 0.44 | 7.92 | 2.25 | 0.28 |
| 10 | 9.60 | 2.69 | 0.56 | 8.34 | 2.47 | 0.37 |
| 11 | 10.16 | 2.98 | 0.69 | 8.80 | 2.71 | 0.47 |
| 12 | 10.76 | 3.29 | 0.85 | 9.28 | 2.97 | 0.59 |
| 13 | 11.41 | 3.63 | 1.04 | 9.81 | 3.26 | 0.74 |
| 14 | 12.11 | 4.02 | 1.26 | 10.37 | 3.59 | 0.92 |
| 15 | 12.86 | 4.45 | 1.52 | 10.98 | 3.94 | 1.13 |
| 16 | 13.68 | 4.92 | 1.82 | 11.63 | 4.34 | 1.37 |
| 17 | 14.56 | 5.45 | 2.18 | 12.34 | 4.77 | 1.66 |
| 18 | 15.52 | 6.04 | 2.59 | 13.10 | 5.26 | 2.00 |
| 19 | 16.56 | 6.70 | 3.07 | 13.93 | 5.80 | 2.40 |
| 20 | 17.69 | 7.44 | 3.64 | 14.83 | 6.40 | 2.87 |
| 21 | 18.92 | 8.26 | 4.31 | 15.81 | 7.07 | 3.42 |
| 22 | 20.27 | 9.19 | 5.09 | 16.88 | 7.82 | 4.06 |
| 23 | 21.75 | 10.23 | 6.00 | 18.05 | 8.66 | 4.82 |
| 24 | 23.36 | 11.40 | 7.08 | 19.32 | 9.60 | 5.71 |
| 25 | 25.13 | 12.72 | 8.34 | 20.72 | 10.66 | 6.76 |
| 26 | 27.09 | 14.21 | 9.84 | 22.25 | 11.85 | 8.00 |
| 27 | 29.24 | 15.90 | 11.60 | 23.94 | 13.20 | 9.46 |
| 28 | 31.61 | 17.81 | 13.70 | 25.80 | 14.72 | 11.19 |
| 29 | 34.24 | 19.98 | 16.18 | 27.86 | 16.44 | 13.23 |
| 30 | 37.16 | 22.46 | 19.13 | 30.14 | 18.40 | 15.66 |
| 31 | 40.41 | 25.28 | 22.65 | 32.67 | 20.63 | 18.56 |
| 32 | 44.04 | 28.52 | 26.87 | 35.49 | 23.18 | 22.02 |
| 33 | 48.09 | 32.23 | 31.94 | 38.64 | 26.09 | 26.16 |
| 34 | 52.64 | 36.50 | 38.04 | 42.16 | 29.44 | 31.14 |
| 35 | 57.75 | 41.44 | 45.41 | 46.12 | 33.30 | 37.15 |
| 36 | 63.53 | 47.16 | 54.36 | 50.59 | 37.75 | 44.26 |
| 37 | 70.07 | 53.80 | 65.27 | 55.63 | 42.92 | 53.21 |
| 38 | 77.50 | 61.55 | 78.61 | 61.35 | 48.93 | 64.07 |
| 39 | 85.97 | 70.61 | 95.03 | 67.87 | 55.96 | 77.33 |
| 40 | 95.66 | 81.27 | 115.31 | 75.31 | 64.20 | 93.69 |
| 41 | 106.81 | 93.85 | 140.51 | 83.86 | 73.90 | 113.98 |
| 42 | 119.67 | 108.75 | 171.99 | 93.71 | 85.37 | 139.31 |
| 43 | 134.58 | 126.50 | 211.56 | 105.11 | 99.01 | 171.14 |
| 44 | 151.95 | 147.74 | 261.60 | 118.37 | 115.31 | 211.40 |
| 45 | 172.29 | 173.29 | 325.34 | 133.87 | 134.87 | 262.74 |
| 46 | 196.22 | 204.19 | 407.11 | 152.10 | 158.50 | 328.73 |
| 47 | 224.55 | 241.80 | 512.84 | 173.64 | 187.21 | 414.32 |
| 48 | 258.29 | 287.85 | 650.67 | 199.26 | 222.30 | 526.45 |
| 49 | 298.72 | 344.64 | 831.99 | 229.92 | 265.50 | 674.91 |
| 50 | 347.51 | 415.15 | 1072.80 | 266.88 | 319.06 | 873.85 |