An Introduction to Settlement and Volume Expansion in Soils

J. Paul Guyer, P.E., R.A.

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5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone & Fax: 703-988-0088
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CONTENTS

1. INTRODUCTION
2. ANALYSIS OF STRESS CONDITIONS
3. INSTANTANEOUS SETTLEMENT
4. PRIMARY AND SECONDARY SETTLEMENTS
5. TOLERABLE AND DIFFERENTIAL SETTLEMENT
6. METHODS OF REDUCING OR ACCELERATING SETTLEMENT
7. ANALYSIS OF VOLUME EXPANSION
8. REFERENCES

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(The figures, tables and formulas in this publication may at times be a little difficult to read, but they are the best available. DO NOT PURCHASE THIS PUBLICATION IF THIS LIMITATION IS NOT ACCEPTABLE TO YOU.)
1. INTRODUCTION

1.1 SCOPE. This publication concerns (a) immediate settlements, (b) long-term settlements, (c) rate of settlement, (d) criteria for tolerable settlement, (e) methods of reducing or accelerating settlements for saturated fine-grained soils and (f) methods for controlling and/or estimating heave in swelling soils. Procedures given are for fine-grained compressible soils as well as for coarse-grained soils.

1.2 OCCURRENCE OF SETTLEMENTS. The settlement of saturated cohesive soil consists of the sum of three components; (1) immediate settlement occurring as the load is applied, (2) consolidation settlement occurring gradually as excess pore pressures generated by loads are dissipated, and (3) secondary compression essentially controlled by the composition and structure of the soil skeleton. The settlement of coarse-grained granular soils subjected to foundation loads occurs primarily from the compression of the soil skeleton due to rearrangement of particles. The permeability of coarse-grained soil is large enough to justify the assumption of immediate excess pore pressure dissipation upon application of load. Settlement of coarse-grained soil can also be induced by vibratory ground motion due to earthquakes, blasting or machinery, or by soaking and submergence.

1.3 APPLICABILITY. Settlement estimates discussed in this publication are applicable to cases where shear stresses are well below the shear strength of the soil.
2. ANALYSIS OF STRESS CONDITIONS

2.1 MECHANICS OF CONSOLIDATION. See Figure 1. Superimposed loads develop pore pressures in compressible strata exceeding the original hydrostatic pressures. As pore pressure gradients force water from a compressible stratum, its volume decreases, causing settlement.

2.2 INITIAL STRESSES. See Figure 2 for profiles of vertical stress in a compressible stratum prior to construction. For equilibrium conditions with no excess hydrostatic pressures, compute vertical effective stress as shown in Case 1, Figure 2.
Figure 1
Consolidation Settlement Analysis
<table>
<thead>
<tr>
<th>STRESS CONDITION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. SIMPLE OVERBURDEN PRESSURE</td>
<td>TOTAL STRESS ( \sigma ), both above and below the G.W. PORE WATER PRESSURE UNTIL G.W.L. EFFECTIVE STRESS ( \sigma' ).</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>STRESS CONDITION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>2. LOWERING OF GROUND WATER LEVEL</td>
<td>IMMEDIATELY AFTER LOWERING OF THE GROUNDWATER TOTAL STRESS UNCHANGED, BUT THE EFFECTIVE STRESSES SLOWLY INCREASE FROM THE CLAY LAYER THE EFFECTIVE STRESSES OVR TIME TO REACH THE NEW EQUILIBRIUM VALUE.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>STRESS CONDITION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. PARTIAL CONSOLIDATION UNDER WEIGHT OF INITIAL FILL</td>
<td>TOTAL STRESSES ON A CLAY LAYER INCREASED BY THE ADDITION OF OVERBURDEN LOAD. WATER IN THE FORM OF EXCESS PORE PRESSURE AS THE SETTLEMENT PROGRESSES IN THE CLAY LAYER THE EFFECTIVE STRESSES INCREASES TO CORRESPOND TO THE STRESSES FROM SURFACE LOAD.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>STRESS CONDITION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. RISE OF GROUND WATER LEVEL</td>
<td>EFFECTIVE STRESSES PRESSED OVERBURDEN EFFECTIVE STRESSES EQUALS ORIGINAL EFFECTIVE STRESSES ( \sigma' ).</td>
</tr>
</tbody>
</table>

**Figure 2**

Profiles of Vertical Stresses Before Compaction
Figure 2 (continued)
Profiles of Vertical Stresses Before Compaction
2.2.1 PRECONSOLIDATION. Stresses exceeding the present effective vertical pressure of overburden produce preconsolidation (1) by the weight of material that existed above the present ground surface and that has been removed by erosion, excavation, or recession of glaciers, (2) by capillary stresses from desiccation, and (3) by lower groundwater levels at some time in the past.

2.2.2 UNDERCONSOLIDATION. Compressible strata may be incompletely consolidated under existing loads as a result of recent lowering of groundwater or recent addition of fills or structural loads. Residual hydrostatic excess pore pressure existing in the compressible stratum will dissipate with time, causing settlements.

2.2.3 EVALUATION OF EXISTING CONDITIONS. Determine consolidation condition at start of construction by the following steps:

2.2.3.1 REVIEW THE DATA AVAILABLE on site history and geology to estimate probable preconsolidation or underconsolidation.

2.2.3.2 COMPARE PROFILE of preconsolidation stress determined from laboratory consolidation tests with the profile of effective over-burden pressures.

2.2.3.3 ESTIMATE PRECONSOLIDATION from $c/P_c$ ratio, where $c$ is the cohesion ($q_u/2$) and $P_c$ is the preconsolidation stress, using laboratory data from unconfined compression test and Atterberg limits.

2.2.3.4 IF UNDERCONSOLIDATION IS INDICATED, install piezometers to measure the magnitude of hydrostatic excess pore water pressures.

2.2.4 COMPUTATION OF ADDED STRESSES. Use the elastic solutions to determine the vertical stress increment from applied loads. On vertical lines beneath selected points in the loaded area, plot profiles of estimated preconsolidation and effective overburden stress plus the increment of applied stress. See Figure 3 for
typical profiles. Lowering of groundwater during construction or regional drawdown increases effective stress at the boundaries of the compressible stratum and initiates consolidation. Stress applied by drawdown equals the reduction in buoyancy of overburden corresponding to decrease in boundary water pressure. In developed locations, settlement of surrounding areas from drawdown must be carefully evaluated before undertaking dewatering or well pumping.
3. INSTANTANEOUS SETTLEMENT

3.1 IMMEDIATE SETTLEMENT OF FINE-GRAINED SOILS. Generally, the instantaneous settlement results from elastic compression of clayey soil. For foundations on unsaturated clay or highly overconsolidated clay, the elastic settlement constitutes a significant portion of the total settlement. Immediate settlement $\Delta_V$ is estimated as:

$$\Delta_V = q \times B \times \left[\frac{1 - \gamma_2}{E_U}\right] \times I$$

$q$ is applied uniform pressure; $B$ is width of loaded area; $I$ is combined shape and rigidity factor; $\gamma$ is Poisson's ratio - ranges between 0.3 and 0.5, the higher value being for saturated soil with no volume change during loading; and $E_U$ is undrained modulus obtained from laboratory or field (pressuremeter) tests. Table 1 (refer to Stresses and Deflections in Foundations and Pavements, by Department of Civil Engineering, University of California, Berkeley, CA) provides values of $I$. Empirical relationship derived from field measurement may be used to determine $E_U$ when actual test values are not available; see Table 2 (refer to An Engineering Manual For Settlement Studies, by Duncan and Buchignani). Empirical correlations for estimation of OCR (Over Consolidation Ratio) are available in the technical literature. If the factor of safety against bearing failure is less than about 3, then the immediate settlement $\Delta_V$ is modified as follows:

$$\Delta_C = \Delta_{SR}$$

$\Delta_C$ = immediate settlement corrected to allow for partial yield condition

$SR = $ Settlement Ratio

Determine $SR$ from Figure 4 (refer to Initial Settlement of Structures on Clay, by D'Appolonia, et al.). See Figure 5 for an example.
Figure 3
Computation of Total Settlement for Various Loading Conditions
3.2. SETTLEMENT OF COARSE-GRAINED SOILS. This immediate settlement is a function of the width and depth of footing, elevation of the water table, and the modulus of vertical subgrade reaction ($K_{VI}$) within the depth affected by the footing. Figure 6 may be used to estimate $K_{VI}$ from the soil boring log, and to compute anticipated settlement. For large footings where soil deformation properties vary significantly with depth or where the thickness of granular soil is only a fraction of the width of the loaded area, the method in Figure 6 may underestimate settlement.

3.3 TOTAL SETTLEMENT IN GRANULAR SOILS. Total settlement is the combined effect of immediate and long-term settlements. A usually conservative estimate of settlement can be made utilizing the method in Figure 7 (Refer to Static Cone to Compute Static Settlement Over Sand, by Schmertmann). A review of methods dealing with settlement of sands utilizing the standard penetration test results can be found in Equivalent Linear Model for Predicting Settlements of Sand Bases, by Oweis.
<table>
<thead>
<tr>
<th>Shape and Rigidity</th>
<th>Center</th>
<th>Corner</th>
<th>Edge/Middle of Long Side</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circle (flexible)</td>
<td>1.00</td>
<td></td>
<td></td>
<td>0.85</td>
</tr>
<tr>
<td>Circle (rigid)</td>
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<td></td>
<td>0.79</td>
</tr>
<tr>
<td>Square (flexible)</td>
<td>1.12</td>
<td>0.56</td>
<td>0.76</td>
<td>0.95</td>
</tr>
<tr>
<td>Square (rigid)</td>
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<td>0.82</td>
<td>0.82</td>
</tr>
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<td>Rectangle (flexible)</td>
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<td></td>
</tr>
<tr>
<td>Length/width</td>
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<td></td>
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<td>1.53</td>
<td>0.76</td>
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<td>1.30</td>
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<td>2.56</td>
<td>1.28</td>
<td>2.10</td>
<td>2.04</td>
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<tr>
<td>Rectangle (rigid)</td>
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<td>Length/width</td>
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</tr>
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<td>1.12</td>
<td>1.12</td>
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<td>1.12</td>
</tr>
<tr>
<td>5</td>
<td>1.60</td>
<td>1.60</td>
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<td>1.60</td>
</tr>
<tr>
<td>10</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
</tr>
</tbody>
</table>

Table 1

Shape and Rigidity Factors I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space
### Table 1 (continued)
Shape and Rigidity Factors $I$ for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space

<table>
<thead>
<tr>
<th>$H/B$</th>
<th>Center of Rigid Circular Area Diameter = $B$</th>
<th>Corner of Flexible Rectangular Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$L/B = 1$</td>
<td>$L/B = 2$</td>
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<tr>
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<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.5</td>
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</tr>
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<td>3.0</td>
<td>0.62</td>
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<td>5.0</td>
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<tr>
<td>10.0</td>
<td>0.74</td>
<td>0.48</td>
</tr>
</tbody>
</table>

For $\nu = 0.50$

<table>
<thead>
<tr>
<th>$H/B$</th>
<th>Center of Rigid Circular Area Diameter = $B$</th>
<th>Corner of Flexible Rectangular Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.5</td>
<td>0.20</td>
<td>0.09</td>
</tr>
<tr>
<td>1.0</td>
<td>0.40</td>
<td>0.19</td>
</tr>
<tr>
<td>1.5</td>
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<td>2.0</td>
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<td>0.32</td>
</tr>
<tr>
<td>3.0</td>
<td>0.64</td>
<td>0.38</td>
</tr>
<tr>
<td>5.0</td>
<td>0.70</td>
<td>0.46</td>
</tr>
<tr>
<td>10.0</td>
<td>0.74</td>
<td>0.49</td>
</tr>
</tbody>
</table>

For $\nu = 0.33$

**Table 1 (continued)**
Shape and Rigidity Factors $I$ for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space

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Example:

Compute immediate settlement at center of uniformly loaded area (flexible) measuring 20' by 20'.

Calculate as the sum of the influence values at the corners of four equal-sided rectangles.

\[ S_u = q \frac{1 - \nu^2}{E_u} I \]

- \[ q = 4 \text{ KSF}, B = 10' \]
- \[ \nu = 0.3, E_u = 20 \text{ KSF} \]
- \[ H/B = 1, L/B = 1, I = 0.15 \]
- \[ \delta = 4 \times 10 \times \left[ \frac{1 - 0.3^2}{20} \times 0.15 \right] \]
- \[ = 0.225' \]

Table 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space
FIGURE 4a
Relationship Between Settlement Ratio and Applied Stress Ratio for Strip Foundation on Homogeneous Isotropic Layer

Figure 4a
Relationship Between Settlement Ratio and Applied Stress Ratio For Strip Foundation on Homogeneous Isotropic Layer
Figure 4b

Relationship Between Initial Shear Stress and Overconsolidation Ratio
Example:

Given LL = 58% PI = 25% c = 1 KSF
Moderately consolidated clay, OCR <3
Depth to rigid layer (H) = 10.5 ft
γ = 0.5
Rigid strip footing, width = 7 ft  
q+appl, = 2.5 KSF  q+ult, = 6 KSF

Find immediate settlement.

\[ \Delta V = q \times B \times \left[ \frac{1 - \gamma^2}{E_U} \right] \times I \]

I = 2.0 (Table 1) assume length/width [approximately] 10

From Table 2, \( E_U = 600 \)

\( E_U = 600 \times 1 = 600 \) KSF

\[ \Delta V = 2.5 \times 7 \times \left[ \frac{1 - 0.5^2}{600} \right] \times 2.0 \times 12 = 0.52 \) inches

Find factor of safety against bearing failure.

\( F_S = \frac{6.0}{2.5} = 2.4 < 3.0 \)

Correct for yield.

\( f = 0.7 \) (Figure 4b)

\( q_{appl} \div q_{ult} = 0.42, H/B = 1.5 \)

SR = 0.60 (Figure 4a)

Figure 5

Example of Immediate Settlement Computations in Clay
Figure 6

Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils

DEFINITIONS

\[ \Delta H_i^* \text{ Immediate settlement of footing} \]
\[ q \text{ Footing unit load in psf} \]
\[ B \text{ Footing width} \]
\[ D \text{ Depth of footing below ground surface} \]
\[ K_{v1} \text{ Modulus of vertical subgrade reaction} \]

COARSE-GRAINED SOILS

\[ K_{v1} = \frac{K_0 (B/D)^2}{1 + (B/D)^2} \]

Shallow footings \( D \leq B \)

- For \( B \leq 20 \) ft:
  \[ \Delta H_i^* = \frac{q \cdot B}{2} \]
- For \( B \geq 40 \) ft:
  \[ \Delta H_i^* = \frac{q \cdot B^2}{2} \]

Interpolate for intermediate values of \( B \).

Deep foundation \( D \geq 5B \)

- For \( B \leq 20 \) ft:
  \[ \Delta H_i = \frac{F_v}{2} \left( B - \frac{1}{2} \right) \]

NOTES:

1. Nonplastic silt is analyzed as coarse-grained soil with modulus of elasticity increasing linearly with depth.

2. Values of \( K_{v1} \) shown for coarse-grained soils apply to dry or moist material with the groundwater level at a depth of at least 1.5B below base of footing. If groundwater is at base of footing, use \( K_{v1}/2 \) in computing settlement.

3. For continuous footings, multiply the settlement computed for width \( "B" \) by 2.
DATA REQUIRED:

1. A profile of standard penetration resistance $N$ (blows/ft) versus depth, from the proposed foundation level to a depth of $2B$, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus $E_S$ is established using the following relationships.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$E_S/N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silts, sands silts, slightly cohesive silt-sand mixtures</td>
<td>4</td>
</tr>
<tr>
<td>Clean, fine to med, sands &amp; slightly silty sands</td>
<td>7</td>
</tr>
<tr>
<td>Coarse sands &amp; sands with little gravel</td>
<td>10</td>
</tr>
<tr>
<td>Sandy gravels and gravel</td>
<td>12</td>
</tr>
</tbody>
</table>

2. Least width of foundation = $B$, depth of embedment = $D$, and proposed average contact pressure = $P$.

3. Approximate unit weights of surcharge soils, and position of water table if within $D$.

4. If the static cone bearing value $q_C$ measured compute $E_S$ based on $E_S = 2q_C$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant $N$ over the depth interval $0$ to $2B$ below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply $N$ values in column 3 by the appropriate factor $E_S/N$ (col. 4) to obtain values of $E_S$; place values in column 5.

4. Draw an assumed $2B-0.6$ triangular distribution for the strain influence factor $I_Z$ along a scaled depth of $0$ to $2B$ below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the $I_Z$ value at the mid-height of each layer, and place in column 7.

Figure 7

Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann's Method
5. Calculate \( \left( \frac{I_z}{E_s} \right) \Delta Z \), and place in column 8. Determine the sum of all values in column 8.

6. Total settlement:

\[
\Delta H = C_1 C_2 \Delta p \sum_{0}^{n} \left( \frac{1}{E_s} \right) \Delta Z,
\]

where:
- \( C_1 = 1 - 0.5 \left( \frac{p_0}{\Delta p} \right) \) embedment correction factor
- \( C_2 = 1 + 0.2 \log (10t) \) creep correction factor
- \( p_0 \) = overburden pressure at foundation level
- \( \Delta p \) = net foundation pressure increase
- \( t \) = elapsed time in years

**EXAMPLE PROBLEM:**

**GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:**

**FIGURE 7 (continued)**

Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method
Figure 3

Computation of Total Settlement for Various Loading Conditions
3.2. SETTLEMENT OF COARSE-GRAINED SOILS. This immediate settlement is a function of the width and depth of footing, elevation of the water table, and the modulus of vertical subgrade reaction ($K_{VI}$) within the depth affected by the footing. Figure 6 may be used to estimate $K_{VI}$ from the soil boring log, and to compute anticipated settlement. For large footings where soil deformation properties vary significantly with depth or where the thickness of granular soil is only a fraction of the width of the loaded area, the method in Figure 6 may underestimate settlement.

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<td>0.64</td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>Circle (rigid)</td>
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<td>0.79</td>
<td></td>
</tr>
<tr>
<td>Square (flexible)</td>
<td>1.12</td>
<td>0.56</td>
<td>0.76</td>
<td>0.95</td>
</tr>
<tr>
<td>Square (rigid)</td>
<td>0.85</td>
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<tr>
<td>Rectangle (flexible)</td>
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<tr>
<td>Length/width</td>
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</tbody>
</table>

Table 1

Shape and Rigidity Factors I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space
Table 1 (continued)
Shape and Rigidity Factors $I$ for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space

Shape and Rigidity Factor $I$ for Loaded Areas on an Elastic Half-Space of Limited Depth Over a Rigid Base

<table>
<thead>
<tr>
<th>$H/B$</th>
<th>Center of Rigid Circular Area</th>
<th>Corner of Flexible Rectangular Area</th>
</tr>
</thead>
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<td></td>
<td>$L/B = 1$</td>
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<td>1.5</td>
<td>0.48</td>
<td>0.23</td>
</tr>
<tr>
<td>2.0</td>
<td>0.54</td>
<td>0.29</td>
</tr>
<tr>
<td>3.0</td>
<td>0.62</td>
<td>0.36</td>
</tr>
<tr>
<td>5.0</td>
<td>0.69</td>
<td>0.44</td>
</tr>
<tr>
<td>10.0</td>
<td>0.74</td>
<td>0.48</td>
</tr>
</tbody>
</table>

Table 1 (continued)
Shape and Rigidity Factors $I$ for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space
Table 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space

Example:

Compute immediate settlement at center of uniformly loaded area (flexible) measuring 20' by 20'.

Calculate as the sum of the influence values at the corners of four equal-sided rectangles.

\[ S_u = qI - \frac{1}{E_u} I \]

\[ q = 4 \text{ KSF}, B = 10' \]

\[ \nu = 0.5, E_u = 20 \text{ KSF} \]

\[ H/B = 1, L/B = 1, I = 0.15 \]

\[ \delta = 4 \times 10 \times \left[ 1 - \frac{0.25^2}{20} \right] \times 0.15 \]

\[ = 0.225' \]

Table 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space
Figure 4a

Relationship Between Settlement Ratio and Applied Stress Ratio
For Strip Foundation on Homogeneous Isotropic Layer
Figure 4b

Relationship Between Initial Shear Stress and Overconsolidation Ratio

Figure 4b

Relationship Between Initial Shear Stress
And Overconsolidation Ratio
Example:

Given LL = 58% PI = 25% c = 1 KSF
Moderately consolidated clay, OCR <3
Depth to rigid layer (H) = 10.5 ft
γ = 0.5
Rigid strip footing, width = 7 ft

Find immediate settlement.
\[ \Delta V = q \times B \times \left[ \frac{(1 - \gamma^2)}{E_U} \right] \times I \]

I = 2.0 (Table 1) assume length/width [approximately] 10

From Table 2, \( E_U = 600 \)
\( E_U = 600 \times 1 = 600 \) KSF
\[ \Delta V = 2.5 \times 7 \times \frac{(1 - 0.5^2)}{600} \times 2.0 \times 12 = 0.52 \text{ inches} \]

Find factor of safety against bearing failure.
\[ F_S = \frac{6.0}{2.5} = 2.4 < 3.0 \]

Correct for yield.

\( f = 0.7 \) (Figure 4b)
\[ \frac{q_{appl}}{q_{ult}} = 0.42, \frac{H}{B} = 1.5 \]

SR = 0.60 (Figure 4a)

---

Figure 5

Example of Immediate Settlement Computations in Clay
**Figure 6**

Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils
DATA REQUIRED:

1. A profile of standard penetration resistance N (blows/ft) versus depth, from the proposed foundation level to a depth of 2B, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus $E_S$ is established using the following relationships.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$E_S/N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silts, sands silts, slightly cohesive silt-sand mixtures</td>
<td>4</td>
</tr>
<tr>
<td>Clean, fine to med, sands &amp; slightly silty sands</td>
<td>7</td>
</tr>
<tr>
<td>Coarse sands &amp; sands with little gravel</td>
<td>10</td>
</tr>
<tr>
<td>Sandy gravels and gravel</td>
<td>12</td>
</tr>
</tbody>
</table>

2. Least width of foundation = B, depth of embedment = D, and proposed average contact pressure = P.

3. Approximate unit weights of surcharge soils, and position of water table if within D.

4. If the static cone bearing value $q_C$ measured compute $E_S$ based on $E_S = 2q_C$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant N over the depth interval 0 to 2B below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply N values in column 3 by the appropriate factor $E_S/N$ (col. 4) to obtain values of $E_S$; place values in column 5.

4. Draw an assumed 2B-0.6 triangular distribution for the strain influence factor $I_z$ along a scaled depth of 0 to 2B below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the $I_z$ value at the mid-height of each layer, and place in column 7.

Figure 7

Settlement of Footings Over Granular Soils: Example Computation
Using Schmertmann’s Method
5. Calculate \( (I_z/E_s) \Delta Z \) and place in column 8. Determine the sum of all values in column 8.

6. Total settlement \( \Delta H = C_1 C_2 \Delta p \sum_{0}^{28} \left( \frac{I_z}{E_s} \right) \Delta Z \),

where \( C_1 = 1 - 0.5 \left( \frac{p_0}{\Delta p} \right) \); \( C_1 \geq 0.5 \) embedment correction factor

\( C_2 = 1 + 0.2 \log (10t) \) creep correction factor

\( p_0 = \) overburden pressure at foundation level

\( \Delta p = \) net foundation pressure increase

\( t = \) elapsed time in years.

**EXAMPLE PROBLEM:**

**GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:**

**FIGURE 7 (continued)***

Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Nomograph for Consolidation with Vertical Drainage
5. Calculate \((I_x/E_s) \Delta z\), and place in column 8. Determine the sum of all values in column 8.

6. Total settlement = \(\Delta H = C_1 C_2 \Delta P \sum \frac{(I_x)}{E_s} \Delta z\),

where

- \(C_1 = 1 - 0.5 (P_0/\Delta P)\)
- \(C_2 = 1 + 0.2 \log (10t)\)

\(P_0 = \) overburden pressure at foundation level

\(\Delta P = \) net foundation pressure increase

\(t = \) elapsed time in years.

**EXAMPLE PROBLEM:**

Given the following soil system and corresponding standard penetration test (SPT) data, determine the amount of ultimate settlement under a given footing and footing load:

**FIGURE 7 (continued)**

Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann’s Method

Figure 7 (continued)

Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann’s Method
4. PRIMARY AND SECONDARY SETTLEMENTS.

4.1 PRIMARY CONSOLIDATION.

4.1.1 CONSOLIDATION SETTLEMENT. For conditions where excess pore pressures are developed during the application of load and if preconsolidation stress is determined reliably, total settlement can be predicted with reasonable accuracy. The percentage error is greatest for settlement from recompression only. In this case an overestimate may result unless high quality undisturbed samples are used for consolidation tests.

4.1.1.1. TYPICAL LOADING CYCLE. See Figure 3 for loading sequence in building construction. Foundation excavation can cause swell and heave. Application of a structural load recompresses subsoil and may extend consolidation into the virgin range. Stress changes are plotted on a semi-logarithmic pressure-void ratio $e$-$\log p$ curve similar to that shown in Figure 3.

4.1.1.2 PRESSURE-VOID RATIO DIAGRAM. Determine the appropriate $e$-$\log p$ curve to represent average properties of compressible stratum from consolidation tests. The $e$-$\log p$ curve may be interpreted from straight line virgin compression and recompression slopes intersecting at the preconsolidation stress. Draw $e$-$\log p$ curve to conform to these straight lines as shown in Figure 3.

4.1.1.3 MAGNITUDE OF CONSOLIDATION SETTLEMENT. Compute settlement magnitude from change in void ratio corresponding to change in stress from initial to final conditions, obtained from the $e$-$\log p$ curve (Figure 3). To improve the accuracy of computations divide the clay layer into a number of sublayers for computing settlement. Changes in compressibility of the stratum and existing and applied stresses can be dealt with more accurately by considering each sublayer independently and then finding their combined effect.
4.1.1.4 PRELIMINARY ESTIMATES of $C_C$ can be made using the correlations in Table 3.

4.1.2 CORRECTIONS TO MAGNITUDE OF CONSOLIDATION SETTLEMENTS. Settlements computed for overconsolidated clays by the above procedures may give an overestimate of the settlement. Correct consolidation settlement estimate as follows:

$$H_C = \alpha (W - \Delta H)_{OC}$$

$H_C =$ corrected consolidation settlement

$\alpha =$ function of overconsolidation ratio (OCR)

$OCR = \frac{P_C}{P_O}$

$\Delta H = \text{calculated settlement resulting from stress increment of } P_O \text{ to } P_C$

and the width of loaded area and thickness of compressible stratum (See Figure 8 for values and refer to Estimating Consolidation Settlements of Shallow Foundation on Overconsolidated Clay, by Leonards.)

$C_C = 0.009 \text{ (LL - 10%) inorganic soils, with sensitivity less than 4}$

$C_C = 0.0115 \text{ water content at liquid limit and } e_0$ is initial void ratio.

$C_C = 1.15 \text{ all clays }^* \text{ with sensitivity less than 4}$

$C_C = 1.15 \text{ all clays }^* \text{ with sensitivity less than 4}$

$C_C = (1 + e_0)(0.1 + [w_n - 25] 0.006) \text{ varved clays }^*$

$w_n$ is natural moisture content, LL is water content at liquid limit and $e_0$ is initial void ratio.
FIGURE 8
Relation Between Settlement Ratio and Overconsolidation Ratio

Figure 8
Relation Between Settlement Ratio and Overconsolidation Ratio
4.2 TIME RATE OF PRIMARY CONSOLIDATION.

4.2.1 APPLICATION. Settlement time rate must be determined for foundation treatment involving either acceleration of consolidation or preconsolidation before construction of structure. Knowledge of settlement rate or percent consolidation completed at a particular time is important in planning remedial measures on a structure damaged by settlement.

4.2.2 TIME RATE OF CONSOLIDATION. Where pore water drainage is essentially vertical, the ordinary one dimensional theory of consolidation defines the time rate of settlement. Using the coefficient of consolidation \( c_v \) compute percent consolidation completed at specific elapsed times by the time factor \( T_v \) curves of Figure 9 (upper panel), refer to, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), DOD. For vertical sand drains use Figure 10 (upper panel, same reference). For preliminary estimates, the empirical correlation for \( c_v \) may be used.

4.2.2.1 EFFECT OF PRESSURE DISTRIBUTION. Rate of consolidation is influenced by the distribution of the pressures which occur throughout the depth of the compressible layer. For cases where the pressures are uniform or vary linearly with depth, use Figure 9 which includes the most common pressure distribution. The nomograph in Figure 11 may be used for this case. For nonlinear pressure distribution, refer to Soil Mechanics in Engineering Practice, by Terzaghi and Peck, to obtain the time factor.

4.2.2.2 ACCURACY OF PREDICTION. Frequently the predicted settlement time is longer than that observed in the field for the following reasons:

4.2.2.2.1 THEORETICAL CONDITIONS ASSUMED for the consolidation analysis frequently do not hold in situ because of intermediate lateral drainage, anisotropy in
permeability, time dependency of real loading, and the variation of soil properties with effective stress. Two or three dimensional loading increases the time rate of consolidation. Figure 12 gives examples of how the width of the loaded area and anisotropy in permeability can affect the consolidation rate substantially. As the ratio of the thickness of the compressible layer to the width of the loaded area increases, the theory tends to overestimate the time factor. For deposits such as some horizontal varved clays where continuous seams of high permeability are present, consolidation can be expected to be considerably faster than settlement rates computed based on the assumption of no lateral drainage.

4.2.2.2.2 THE COEFFICIENT OF CONSOLIDATION, as determined in the laboratory, decreases with sample disturbance. Predicted settlement time tends to be greater than actual time.
**FIGURE 9**

Time Rate of Consolidation for Vertical Drainage
Due to Instantaneous Loading

Figure 9

Time Rate of Consolidation for Vertical Drainage
Due to Instantaneous Loading
Figure 10
Vertical Sand Drains and Settlement Time Rate

7.1-228

Figure 10
Vertical Sand Drains and Settlement Time Rate
Figure 11
Nomograph for Consolidation with Vertical Drainage
4.2.2.2.3 GRADUAL LOAD APPLICATION. If construction time is appreciable compared to time required for primary consolidation, use the time factors of Figure 13 to determine consolidation rate during and following construction.
4.2.2.2.4 COEFFICIENT OF CONSOLIDATION FROM FIELD MEASUREMENTS. 
Where piezometers are installed to measure pore water pressure under the applied loads, $c_v$ is computed as shown in Figure 14.

4.2.3 TIME RATE OF MULTI-LAYER CONSOLIDATION. If a compressible stratum contains layers of different overall properties, use the procedure of Figure 15 to determine overall settlement time rate.

4.3 SECONDARY COMPRESSION.

4.3.1 LABORATORY e-LOG p CURVE. A laboratory e-log p curve includes an amount of secondary compression that depends on duration of test loads. Secondary compression continues exponentially with time without definite termination. Thus, total or ultimate settlement includes secondary compression to a specific time following completion of primary consolidation.

4.3.2 SETTLEMENT COMPUTATION. Compute settlement from secondary compression following primary consolidation as follows:

$$H_{sec} = C_\alpha \left(H_t\right) \left(\log \left[t_{sec}/t_P\right]\right)$$

where:

$H_{sec}$ = settlement from secondary compression 
$C_\alpha$ = coefficient of secondary compression expressed by the strain per log cycle of time 
$H_t$ = thickness of the compressible stratum 
$t_{sec}$ = useful life of structure or time for which settlement is significant 
$t_P$ = time of completion of primary consolidation
See example in Figure 9 for calculating the secondary settlement. The parameter C can be determined from laboratory consolidation tests; for preliminary estimates, the correlations in Figure 16 may be used. This relationship is applicable to a wide range of soils such as inorganic plastic clays, organic silts, peats, etc.

4.3.3 COMBINING SECONDARY AND PRIMARY CONSOLIDATION. If secondary compression is important, compute the settlement from primary consolidation separately, using an e-log p curve that includes only compression from primary consolidation. For each load increment in the consolidation test, compression is plotted versus time (log scale). The compression at the end of the primary portion (rather than 24 hours) may be used to establish e-log p curve.
Figure 13
Time Rate of Consolidation for Gradual Load Application
Figure 14

Coefficient of Consolidation from Field Measurements

\[ U_z = 1 - \frac{u_c}{u_0} \] (consolidation ratio)

\[ u_c = \text{Excess pore pressure at some time } t \]

\[ u_0 = \text{Excess pore pressure at time } t = 0 \text{ (due to external loading)} \]
Example:

Thickness of clay layer H = 66 ft, Drainage - top & bottom

H = 66/2 = 33 ft

Depth of piezometer below top of compressible layer = 21 ft

Applied external load [W-Δp] = 1.5 KSF

Initial excess pore water pressure = u₀ = [W-Δp] = 1.5 KSF

Excess pore pressure after time t₁ = 15 days, uₑ(15) = 20 ft = Uₑ₁

Excess pore pressure after time t₂ = 100 days, uₑ(100) = 14 ft = Uₑ₂

Piezometer measure U₀ = 24 feet of water +21 ft (initial static head) for a total of 45 ft.

Z/H = 0.21/0.33 = 0.64

Consolidation ratio at time t₁ = 15 days = (uₑ₁)t₁ = 1 - 20/24 = 0.17

Consolidation ratio at time t₂ = 100 days = (uₑ₂)t₂ = 1 - 14/24 = 0.47

From above graph T₁ = 0.11 (point A), T₂ = 0.29 (point B)

Cᵥ = [(0.29 - 0.11)/(100-15)] x (33)² = 231 ft²/day

FIGURE 14 (continued)

Coefficient of Consolidation from Field Measurements
For a soil system containing n layers with properties $C_{vi}$ (coefficient of consolidation) and $H_i$ (layer thickness), convert the system to one equivalent layer with equivalent properties, using the following procedure:

1. Select any layer $i$, with properties $c_v = c_{vi}$, $H = H_i$

2. Transform the thickness of every other layer to an equivalent thickness of a layer possessing the soil properties of layer $i$, as follows:
   
   \[
   H'_1 = \frac{[(H_1)(c_{vi})^{1/2}]}{c_{vi}} \\
   H'_2 = \frac{[(H_2)(c_{v2})^{1/2}]}{c_{v2}} \\
   H'_n = \frac{[(H_n)(c_{vn})^{1/2}]}{c_{vn}}
   \]

3. Calculate the total thickness of the equivalent layer:
   
   \[
   H'_T = H'_1 + H'_2 + ... + H'_i + ... + H'_n
   \]

4. Treat the system as a single layer of thickness $H'_T$, possessing a coefficient of consolidation $c_v = c_{vi}$

5. Determine values of percent consolidation ($U$) at various times ($t$) for total thickness ($H'_T$) using nomograph in Figure 11.

Figure 15

Procedure for Determining the Rate of Consolidation for All Soil Systems Containing "N" layers
Figure 15 (continued)
Procedure for Determining the Rate of Consolidation for All Soil Systems Containing "N" Layers

Figure 15 (continued)
Procedure for Determining the Rate of Consolidation for All Soil Systems Containing "N" Layers
Figure 16

Coefficient of Secondary Compression as Related to Natural Water Content

Coefficient of Secondary Compression as Related to Natural Water Content
4.4 SANITARY LANDFILL. Foundations on sanitary landfills will undergo extensive settlements, both total and distortional, which are extremely difficult to predict. Settlements result not only from compression of the underlying materials, but also from the decomposition of organic matter. Gases in landfill areas are health and fire hazards. A thorough study is necessary when utilizing sanitary landfill areas for foundations.

4.5 PEAT AND ORGANIC SOILS. Settlements in these soils are computed in a similar manner as for fine-grained soils. However, the primary consolidation takes place rapidly and the secondary compression continues for a long period of time and contributes much more to the total settlement.
5. TOLERABLE AND DIFFERENTIAL SETTLEMENT

5.1 APPLICATIONS. For an important structure, compute total settlement at a sufficient number of points to establish the overall settlement pattern. From this pattern, determine the maximum scope of the settlement profile or the greatest difference in settlement between adjacent foundation units.

5.2. APPROXIMATE VALUES. Because of natural variation of soil properties and uncertainty on the rigidity of structure and thus actual loads transmitted to foundation units, empirical relationships have been suggested to estimate the differential settlements (or angular distortion) in terms of total settlement (refer to Structure Soil Interaction, by Institution of Civil Engineers). Terzaghi and Peck, page 489) suggested that for footings on sand, differential settlement is unlikely to exceed 75% of the total settlement. For clays, differential settlement may in some cases approach the total settlement.

5.3 TOLERABLE SETTLEMENT.

5.3.1 CRITERIA. Differential settlements and associated rotations and tilt may cause structural damage and could impair the serviceability and function of a given structure. Under certain conditions, differential settlements could undermine the stability of the structure and cause structural failure. Table 4 (Allowable Settlements of Structures, by Bjerrum) provides some guidelines to evaluate the effect of settlement on most structures. Table 5 provides guidelines for tanks and other facilities.

5.3.2 REDUCTION OF DIFFERENTIAL SETTLEMENT EFFECTS. Settlement that can be completed during the early stages of construction, before placing sensitive finishes, generally will not contribute to structural distress. In buildings with light frames where large differential settlements may not harm the frame, make special provisions to avoid damage to utilities or operating equipment. Isolate sensitive equipment, such
as motor-generator sets within the structure, on separate rigidly supported foundations. Provide flexible couplings for utility lines at critical locations.
Table 4
Tolerable Settlements for Building
### TABLE 5

Tolerable Differential Settlement for Miscellaneous Structures

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>TOLERABLE DISTORTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. UNREINFORCED LOAD BEARING WALLS</strong></td>
<td>( \frac{\Delta_{\text{max}}}{L} )</td>
</tr>
<tr>
<td>(L and ( h ) are respectively length and height of the wall from top of footing)</td>
<td>( \frac{\Delta_{\text{max}}}{L} = \frac{1}{3500} ) to ( \frac{1}{2500} ) for ( L/h &lt; 3 )</td>
</tr>
<tr>
<td>(L and ( h ) are respectively length and height of the wall from top of footing)</td>
<td>( \frac{\Delta_{\text{max}}}{L} = \frac{1}{2000} ) to ( \frac{1}{1250} ) for ( L/h &gt; 5 )</td>
</tr>
<tr>
<td>(L and ( h ) are respectively length and height of the wall from top of footing)</td>
<td>( \frac{\Delta_{\text{max}}}{L} = \frac{1}{5000} ) for ( L/h = 1 )</td>
</tr>
<tr>
<td>(L and ( h ) are respectively length and height of the wall from top of footing)</td>
<td>( \frac{\Delta_{\text{max}}}{L} = \frac{1}{2500} ) for ( L/h = 5 )</td>
</tr>
<tr>
<td><strong>B. JOINTED RIGID CONCRETE PRESSURE CONDUTS</strong></td>
<td>( \frac{\Delta_{\text{max}}}{L} = \frac{1}{125} )</td>
</tr>
<tr>
<td>(maximum angle change at joint 2 to 4 times average slope of settlement profile, longitudinal extension affects damage)</td>
<td>( \beta &lt; \frac{1}{300} )</td>
</tr>
<tr>
<td><strong>C. CIRCULAR STEEL PETROLEUM OR FLUID STORAGE TANKS.</strong></td>
<td>( \beta' = \frac{1}{500} ) to ( \frac{1}{300} )</td>
</tr>
</tbody>
</table>

\[ \beta = \frac{s_i - s_j}{L} \]

\[ \beta' = \frac{(s_i - s_j')}{2} + \frac{s_k}{L} \]

---

Table 5

Tolerable Differential Settlement for Miscellaneous Structures
5.4 EFFECT OF STRUCTURE RIGIDITY. Computed differential settlement is less accurate than computed total or average settlement because the interaction between the foundation elements and the supporting soil is difficult to predict. Complete rigidity implies uniform settlement and thus no differential settlement. Complete flexibility implies uniform contact pressure between the mat and the soil. Actual conditions are always in between the two extreme conditions. However, depending on the magnitude of relative stiffness as defined below, mats can be defined as rigid or flexible for practical purposes.

5.4.1 UNIFORMLY LOADED CIRCULAR RAFT. In the case where the raft has a frictionless contact with an elastic half space (as soil is generally assumed to represent), the relative stiffness is defined as:

\[ R = \text{radius of the raft, } t = \text{thickness of raft, subscripts } r \text{ and } s \text{ refer to raft and soil, } \nu = \text{Poisson's ratio and } E = \text{Young's modulus.} \]

For \( K_r \leq 0.08 \), raft is considered flexible and for \( K_r \leq 5.0 \) raft is considered rigid. For intermediate stiffness values see Numerical Analyses of Uniformly Loaded Circular Rafts on Elastic Layers of Finite Depth, by Brown.

5.4.2 UNIFORMLY LOADED RECTANGULAR RAFT. For frictionless contact between the raft and soil, the stiffness factor is defined as:

\[ B = \text{width of the foundation. Other symbols are defined in 5.4.1.} \]

For \( K_r \leq 0.05 \), raft is considered flexible and for \( K+r, \geq 10 \), raft is considered rigid. For intermediate stiffness values see Numerical Analysis of Rectangular Raft on Layered Foundations, by Frazer and Wardle.
6. METHODS OF REDUCING OR ACCELERATING SETTLEMENT

6.1 GENERAL. See Table 6 for methods of minimizing consolidation settlements. These include removal or displacement of compressible material and preconsolidation in advance of final construction.

6.2 REMOVAL OF COMPRESSIBLE SOILS. Consider excavation or displacement of compressible materials for stabilization of fills that must be placed over soft strata.
Procedures for linear fills on swamps or compressible surface stratum:

<table>
<thead>
<tr>
<th>Method</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation of soft material</td>
<td>When compressible foundation soils extend to depth of about 10 to 15 ft, it may be practicable to remove entirely. Partial removal is combined with various methods of displacing remaining soft material.</td>
</tr>
<tr>
<td>Displacement by weight of fill</td>
<td>Complete displacement is obtained only when compressible foundation is thin and very soft. Weight displacement is combined with excavation of shallow material.</td>
</tr>
<tr>
<td>Jetting to facilitate displacement</td>
<td>For a sand or gravel fill, jetting within the fill reduces its rigidity and promotes shear failure to displace soft foundation. Jetting within soft foundation weakens it to assist in displacement.</td>
</tr>
<tr>
<td>Blasting by trench or shooting methods</td>
<td>Charge is placed directly in front of advancing fill to blast out a trench into which the fill is forced by the weight of surcharge built up at its point. Limited to depths not exceeding about 20 ft.</td>
</tr>
<tr>
<td>Blasting by relief method</td>
<td>Used for building up fill on an old roadway or for fills of plastic soil. Trenches are blasted at both toes of the fill slopes, relieving confining pressure and allowing fill to settle and displace underlying soft materials.</td>
</tr>
<tr>
<td>Blasting by underfill method</td>
<td>Charge is placed in soft soil underlying fill by jetting through the fill at a preliminary stage of its buildup. Blasting loosens compressible material, accelerating settlement and facilitating displacement to the sides. In some cases relief ditches are cut or blasted at toe of the fill slopes. Procedure is used in swamp deposits up to 30 ft thick.</td>
</tr>
</tbody>
</table>

Table 6

Methods of Reducing or Accelerating Settlement or Coping with Settlement
### Method

<table>
<thead>
<tr>
<th>Procedures for preconsolidation of soft foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Surcharge fill</strong></td>
</tr>
<tr>
<td><strong>Accelerating consolidation by vertical drains</strong></td>
</tr>
<tr>
<td><strong>Vertical sand drains with or without surcharge fill</strong></td>
</tr>
<tr>
<td><strong>Wellpoints placed in vertical sand drains</strong></td>
</tr>
<tr>
<td><strong>Vacuum method</strong></td>
</tr>
<tr>
<td><strong>Balancing load of structure by excavation</strong></td>
</tr>
</tbody>
</table>

**Table 6 (continued)**

Methods of Reducing or Accelerating Settlement or Coping with Settlement
6.2.1 REMOVAL BY EXCAVATION. Organic swamp deposits with low shear strength and high compressibility should be removed by excavation and replaced by controlled fill. Frequently these organic soils are underlain by very loose fine sands or silt or soft clayey silts which may be adequate for the embankment foundation and not require replacement. Topsoil is usually stripped prior to placement of fills; however, stripping may not be required for embankments higher than 6 feet as the settlement from the upper 1/2 foot of topsoil is generally small and takes place rapidly during construction period. However, if the topsoil is left in place, the overall stability of the embankment should be checked assuming a failure plane through the topsoil.

6.2.2 DISPLACEMENT. Partial excavation may be accompanied by displacement of the soft foundation by the weight of fill. The advancing fill should have a steep front face. The displacement method is usually used for peat and muck deposits. This method has been used successfully in a few cases for soft soils up to 65 feet deep. Jetting in the fill and various blasting methods are used to facilitate displacement. Fibrous organic materials tend to resist displacement resulting in trapped pockets which may cause differential settlement.

6.3 BALANCING LOAD BY EXCAVATION. To decrease final settlement, within an excavation that is carried to a depth at which the weight of overburden, removed partially or completely, balances the applied load.

6.3.1 COMPUTATION OF TOTAL SETTLEMENT. In this case, settlement is derived largely from recompression. The amount of recompression is influenced by magnitude of heave and magnitude of swell in the unloading stage.

6.3.2 EFFECT OF DEWATERING. If drawdown for dewatering extends well below the planned subgrade, heave and consequent recompression are decreased by the application of capillary stresses. If groundwater level is restored after construction, the load removed equals the depth of excavation times total unit weight of the soil. If groundwater pressures are to be permanently relieved, the load removed equals the
total weight of soil above the original water table plus the submerged weight of soil below the original water table. Calculate effective stresses as described in Figure 2, and consolidation under structural loads as shown in Figure 3.

6.4 PRECONSOLIDATION BY SURCHARGE. This procedure causes a portion of the total settlement to occur before construction. It is used primarily for fill beneath paved areas or structures with comparatively light column loads. For heavier structures, a compacted fill of high rigidity may be required to reduce stresses in compressible foundation soil.

6.4.1 ELIMINATION OF PRIMARY CONSOLIDATION. Use Figure 17 to determine surcharge load and percent consolidation under surcharge necessary to eliminate primary consolidation under final load. This computation assumes that the rate of consolidation under the surcharge is equal to that under final load.
Figure 17

Surcharge Load Required to Eliminate Settlement Under Final Load

Figure 17

Surcharge Load Required to Eliminate Settlement Under Final Load
6.4.2 ELIMINATION OF SECONDARY CONSOLIDATION. Use the formula in the bottom panel of Figure 17 to determine surcharge load and percent consolidation under surcharge required to eliminate primary consolidation plus a specific secondary compression under final load.

6.4.3 LIMITATIONS ON SURCHARGE. In addition to considerations of time available and cost, the surcharge load may induce shear failure of the soft foundation soil. Analyze stability under surcharge.

6.5 VERTICAL DRAINS. These consist of a column of pervious material placed in cylindrical vertical holes in the compressible stratum at sufficiently close spaces so that the horizontal drainage path is less than the vertical drainage path. All drains should be connected at the ground surface to a drainage blanket. Vertical drains are utilized in connection with fills supporting pavements or low- to moderate-load structures and storage tanks. Common types of vertical drains are shown in Table 7 (refer to Use of Precompression and Vertical Sand Drains for Stabilization of Foundation Soils, by Ladd). Sand drains driven with a closed-end pipe produce the largest displacement and disturbance in the surrounding soil and thus their effectiveness is reduced.

6.5.1 CHARACTERISTICS. Vertical drains accelerate consolidation by facilitating drainage of pore water but do not change total compression of the stratum subjected to a specific load. Vertical drains are laid out in rows, staggered, or aligned to form patterns of equilateral triangles or squares. See Figure 18 for cross-section and design data for typical installation for sand drains.

6.5.2 CONSOLIDATION RATE. Time rate of consolidation by radial drainage of pore water to vertical drains is defined by time factor curves in upper panel of Figure 10. For convenience, use the nomograph of Figure 19 to determine consolidation time rate. Determine the combined effect of vertical and radial drainage on consolidation time rate as shown in the example in Figure 10.
6.5.3 VERTICAL DRAIN DESIGN. See Figure 20 for an example of design. For a trial selection of drain diameter and spacing, combine percent consolidation at a specific time from vertical drainage with percent consolidation for radial drainage to the drain. This combined percent consolidation $U_C$ is plotted versus elapsed time for different drain spacing in the center panel of Figure 20. Selection of drain spacing depends on the percent consolidation required prior to start of structure, the time available for consolidation, and economic considerations.

6.5.4 ALLOWANCE FOR SMEAR AND DISTURBANCE. In cases where sand drain holes are driven with a closed-end pipe, soil in a surrounding annular space one-third to one-half the drain diameter in width is remolded and its stratification is distorted by smear. Smear tends to reduce the horizontal permeability coefficient, and a correction should be made in accordance with Figure 21.
### Table 7
Common Types of Vertical Drains

<table>
<thead>
<tr>
<th>General Type</th>
<th>Sub-type</th>
<th>Typical Installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Driven Sand Drain</td>
<td>Closed end mandrel</td>
<td>(d_\omega) 18+ in</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(s) 5 - 20 ft</td>
</tr>
<tr>
<td>2. Augered Sand Drain</td>
<td>(a) Screw type auger</td>
<td>(d_\omega) 6 - 30 in</td>
</tr>
<tr>
<td></td>
<td>(b) Continuous flight hollow stem auger</td>
<td>(s) -</td>
</tr>
<tr>
<td>3. Jetted Sand Drain</td>
<td>(a) Internal jetting</td>
<td>(d_\omega) 18 in</td>
</tr>
<tr>
<td></td>
<td>(b) Rotary jet</td>
<td>(s) 5 - 20 ft</td>
</tr>
<tr>
<td></td>
<td>(c) Dutch jet-bailer</td>
<td>(d_\omega) 12 in</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(s) 4 - 16 ft</td>
</tr>
<tr>
<td>4. &quot;Paper&quot; Drain</td>
<td>(a) Kjellman cardboard wick</td>
<td>(d_\omega) 0.1+ in by 4+ in</td>
</tr>
<tr>
<td></td>
<td>(b) Cardboard coated plastic wick</td>
<td>(s) 1.5+ - 4+ ft</td>
</tr>
<tr>
<td>5. Fabric Encased Sand Drain</td>
<td>(a) Sandwich</td>
<td>(d_\omega) 2.5 - 3 in</td>
</tr>
<tr>
<td></td>
<td>(b) Fabridrain</td>
<td>(s) 4 - 12 ft</td>
</tr>
</tbody>
</table>

\(d_\omega\) = diameter of drain, \(s\) = drain spacing
Figure 18

Data for Typical Sand Drain Installation

**Figure 18**

Data for Typical Sand Drain Installation
Nomograph for Consolidation with Radial Drainage to Vertical and Drain

Figure 19

DIRECTIONS
1. LOCATE POINT 1 CORRESPONDING TO ASSUMED DIAMETER AND SPACING OF SAND DRAINS, ARRANGED IN AN EQUILATERAL TRIANGULAR ARRAY (IF SAND DRAINS ARE ARRANGED IN A SQUARE ARRAY, MULTIPLY SPACING BY 0.72 TO CONVERT TO AN EQUIVALENT TRIANGULAR ARRAY).
2. FROM POINT 1, EXTEND A LINE HORIZONTALLY TO REFERENCE LINE TO LOCATE POINT 2.
3. CONNECT POINT 2 AND ASSUMED VALUE OF THE COEFFICIENT OF CONSOLIDATION FOR RADIAL DRAINAGE, (POINT 3) WITH A STRAIGHT LINE TO LOCATE POINT 4 ON THE SUPPORT LINE.
4. PASS A STRAIGHT LINE FROM POINT 4 THROUGH POINT 5 THE ELAPSED TIME AFTER INSTANTANEOUS LOADING, TO DETERMINE POINT 6 CORRESPONDING TO THE DESIRED VALUES OF THE AVERAGE DEGREE OF CONSOLIDATION OR AVERAGE EXCESS PORE WATER PRESSURE RATIO.

NOTES
- NOMOGRAPH APPLIES TO RADIAL FLOW TO VERTICAL SAND DRAINS ASSUMING EQUAL STRAIN CONDITIONS THROUGHOUT COMPRESSIBLE STRATUM.

Nomograph for Consolidation with Radial Drainage to Vertical Sand Drain
**Figure 20**

Example of Surcharge and Sand Drain Design

---

#### Example of Surcharge and Sand Drain Design

<table>
<thead>
<tr>
<th>Fill</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_T = 120 \text{ pcf} )</td>
<td>( \gamma_T = 125 \text{ pcf} )</td>
</tr>
<tr>
<td>Height = 35 ft</td>
<td>( \gamma_{SUB} = 95 \text{ pcf} )</td>
</tr>
<tr>
<td>( P_f = 2.10 \text{ TSF} )</td>
<td>( P_f = 2.00 \text{ TSF} )</td>
</tr>
</tbody>
</table>

**Given Soil Properties**

- **Clay:**
  - \( \gamma_{SUB} = 40 \text{ PSF} \)
  - \( C_h = 0.50 \text{ ft}^2/\text{day} \)
  - \( C_0 = 1.80 \text{ ft}^2/\text{ft/cycle of time} \)
  - \( C_w = 0.05 \text{ ft}^2/\text{ft/cycle of time} \)

- **Sand:**
  - \( K_s = 6 \times 10^{-8} \text{ in/sec} \)
  - \( K_h = 1 \times 10^{-8} \text{ in/sec} \)

**Computation of Total Settlement**

- \( P_0 = 0.63 \text{ TSF} \)
- \( P_1 = P_0 + P_f = 2.73 \text{ TSF} \)

**Primary Consolidation**

- \( \Delta H_f = (C_0)(H_f) \log \frac{P_1}{P_0} = 0.60 \times 20 \log \frac{2.73}{0.63} = 3.07 \text{ ft} \)

**Secondary Compression**

- \( \Delta H_{SEC} = \frac{C_A(H_f)}{T_p} \log \frac{1}{T_p} \text{ ft/cycle of time} \)
- \( \Delta H_{SEC} = 0.015 \times 20 = 0.30 \text{ ft} \)

**Total Settlement**

- \( \Delta H = 3.37 \text{ ft} \)

---

**Design of Sand Drains (No Surcharge Included)**

- Select 18" drain and consider spacing of 10 ft, 16 ft & 20 ft.
- Allowance for smear effect: \( K_h/K_s = 5 \) assume \( n = 1.2 \) for \( d_e = 10 \) ft.
- \( T_e = 6.67 \text{ ft} \)

- From **Figure 21**, equivalent \( r_w = 6.3^2 \text{ ft} \), equivalent \( d_w = 12.6^2 \text{ ft} \).

- Time required for consolidation to obtain time vs. \( U_r \) for vertical drainage only, use nomograph of **Figure 11** to obtain time vs. \( U_r \) for radial drainage to sand drain, use nomograph of **Figure 19**.

- Combined \( U_r \) at any time is \( U_r = 100 \left[ \frac{U_{90} - U_{60} - U_{30}}{100 - U_{30}} \right] \) in \%.
Selection of Surcharge Height

\[ \Delta H = 3.07, \quad \Delta H_{sec} = 0.30, \quad P_{f}/P_{0} = 3.58 \]

To eliminate settlement under \( P_{f} \), \( u_{c} \) is taken equal to \( u_{f+g} \).

\[ u_{f+g} = \frac{\Delta H}{\Delta H_{f+g}} \]

Relation of \( u_{f+g} \) and time is given above for various drain spacings.

Surcharge \( P_{s} \) for values of \( u_{f+g} = \frac{\Delta H}{\Delta H_{f+g}} \) is shown in Fig. 17.

Surcharge \( P_{s} \) for values of \( u_{f+g} = \frac{\Delta H_{f+g} + \Delta H_{sec}}{\Delta H_{f+g}} \) is given by formula in Fig. 17.

Using these relationships, \( P_{s} \) (expressed as height of surcharge) replaces \( u_{f+g} \) in Figure 17.

Combination of sand drain and surcharge is selected based on time available and comparative costs.

FIGURE 20 (continued)
Example of Surcharge and Sand Drain Design

Figure 20 (continued)
Example of Surcharge and Sand Drain Design
Figure 21

Allowance for Smear Effect in Sand Drain Design
6.5.5 SAND DRAINS PLUS SURCHARGE. A surcharge load is normally placed above the final fill level to accelerate the required settlement. Surcharge is especially necessary when the compressible foundation contains material in which secondary compression predominates over primary consolidation. The percent consolidation under the surcharge fill necessary to eliminate a specific amount of settlement under final load is determined as shown in the lowest panel of Figure 20.

6.5.6 GENERAL DESIGN REQUIREMENTS. Analyze stability against foundation failure, including the effect of pore pressures on the failure plane. Determine allowable buildup of pore pressure in the compressible stratum as height of fill is increased.

6.5.6.1 HORIZONTAL DRAINAGE. For major installation investigate in detail the horizontal coefficient of consolidation by laboratory tests with drainage in the horizontal direction, or field permeability tests to determine horizontal permeability.

6.5.6.2 CONSOLIDATION TESTS. Evaluate the importance of smear or disturbance by consolidation tests on remolded samples. For sensitive soils and highly stratified soils, consider nondisplacement methods for forming drain holes.

6.5.6.3 DRAINAGE MATERIAL. Determine drainage material and arrangement to handle maximum flow of water squeezed from the compressible stratum.

6.5.7 CONSTRUCTION CONTROL REQUIREMENTS. Control the rate of fill rise by installing piezometer and observing pore pressure increase for comparison with pore pressure values compatible with stability. Check anticipated rate of consolidation by pore pressure dissipation and settlement measurements.
7. ANALYSIS OF VOLUME EXPANSION.

7.1 CAUSES OF VOLUME EXPANSION. Volume expansion is caused by (a) reduction of effective stresses, (b) mineral changes, and (c) formation and growth of ice lenses. Swell with decrease of effective stress is a reverse of the consolidation process. For description of swelling problems and suggested treatment, see Table 8. Where highly preconsolidated plastic clays are present at the ground surface, seasonal cycles of rainfall and desiccation produce volume changes. The most severe swelling occurs with montmorillinite clays although, in an appropriate climate, any surface clay of medium to high plasticity with relatively low moisture content can heave.

7.2 MAGNITUDE OF VOLUME EXPANSION. Figure 22 outlines a procedure for estimating the magnitude of swelling that may occur when footings are built on expansive clay soils. This figure also indicates a method of determining the necessary undercut to reduce the heave to an acceptable value.
<table>
<thead>
<tr>
<th>Conditions and materials</th>
<th>Mechanism of heave</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reduction of effective stress of overburden:</td>
<td>Soil swells in accordance with laboratory e-p curves. Heave is maximum at center of excavation. Total potential heave may not have occurred by time the load is reapplied. Final structural load will recompress foundation materials.</td>
<td>Provide drainage for rapid collection of surface water. Avoid disturbance to subgrade by placing 4-in-thick working mat of lean concrete immediately after exposing subgrade. Heave is minimized if the groundwater is drawn down 1 or 4 ft below base of excavation at its center to maintain capillary stresses. Protect shale from wetting and drying during excavation by limiting area opened at subgrade and with concrete working mat. Pour concrete floors and foundations directly on protected shale with no underfloor drainage course. Backfill around walls with impervious soils to prevent access of water. Provide proper surface drainage and paving if necessary to avoid infiltration. Where an increase in water content is probable, special structural designs must be considered. These include (1) anchoring or rock bolting the floor to a depth in shale that provides suitable load bearing against swelling pressures; (2) a floor supported on heavily loaded column footings with an opening or compressible fill beneath floors; and (3) a mat foundation designed to resist potential swelling pressures. In any case, excavation is the shale should be protected by sealing costs or working mat immediately after exposure at subgrade.</td>
</tr>
<tr>
<td>Temporary reduction of effective stress by excavation for structure foundation in preconsolidated clays.</td>
<td>In sand shale where water cannot obtain access to the shale, swelling may be insignificant. For hydraulic structures or construction below the ground water table, reduction of effective stresses will cause permanent heave in accordance with laboratory e-p curves. Alternate wetting and drying during excavation increases swelling potential.</td>
<td></td>
</tr>
<tr>
<td>Permanent reduction of effective stress by excavation in chemically inert, uncemented clay-shale or shale.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduction of effective stress of overburden and release of capillary stress: Construction of earth dams of heavily compacted plastic clays.</td>
<td>Intrusion of airage from reservoir releases capillary pressures and reduces effective stress of overburden and may produce swelling leading to sloughing of the slopes. Most critical material are CH-clays with swelling index exceeding 0.07. Compaction at relatively low water contents, where the water deficiency in the clay mineral lattice is high and the degree of saturation is low, will accentuate swelling.</td>
<td>Avoid placing highly plastic fill on or near embankment slopes. Compact clays at a relatively high moisture content consistent with strength and compressibility requirements. Avoid overcompaction to an unnecessarily high dry unit weight.</td>
</tr>
<tr>
<td>Construction of structural fill for light buildings of compacted plastic clay.</td>
<td>Rise of groundwater, seepage, leakage, or elimination of surface evaporation increases degree of saturation and reduces effective stress, leading to expansion.</td>
<td>Compact clays as well as practicable consistent with compressibility requirements. Avoid overcompaction of general fill and undercompaction of backfill at column footings or in utility trenches which would accentuate differential movements. Stabilization of compacted fills with various soil admixures reduces swelling potential by increasing ion concentration in pore water.</td>
</tr>
</tbody>
</table>
### Table 8 (continued)
**Heave From Volume Change**

<table>
<thead>
<tr>
<th>Conditions and materials</th>
<th>Mechanism of heave</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Changes of capillary stresses:</td>
<td>Seasonal movements 1 or 2 in. upwards and downwards occur within the upper 3 to 5 ft. Settlement occurs in early summer and expansion in the fall. Caused by change of capillary stresses produced by transpiration to nearby trees, plant, or grass cover surrounding the structure. Movements are maximum at edge of building. Groundwater is shallow. Change of capillary stresses by evaporation is not of prime importance. Even in the absence of vegetation, seasonal cycles of settlement and heave occur because of the alternate increase and release of capillary stresses. Buildings constructed during wet season may undergo small but nonuniform settlement beneath exterior footings. Buildings constructed in the dry season undergo uneven heave up to 3 or 4 in. maximum, distributed irregularly over the structure. Permanent moisture deficiency exists in the ground. Construction eliminates evaporation over building area, reducing capillary stresses and causing movement of moisture to beneath building. This leads to continuing heave with minor seasonal fluctuations. Thermocouple gradients directed toward cooled subsurface beneath structure contribute to increase in moisture, which may extend to depths of 10 to 15 ft. Exposure to air and water causes oxidation and hydration of pyrites with a volumetric expansion of as much as ten times their original volume, or hydration of anhydrite to gypsum.</td>
<td>Light reinforcing or stiffening minimize effects in small houses. Basements carried to usual depths usually eliminate movements. Support light footings and slabs on compacted, course-grained fill about 4 to 6 ft thick. Place peripheral areas to minimize subsurface moisture content change. Consider the use of belled caissons with supported floor. Open block wall foundations have been utilized for small houses. Collect rainwater falling on structure and surrounding areas and convey runoff away from structures. Damage is minimized by use of slab or raft foundation, dry wall construction, steel or reinforced concrete framing, reinforced foundation beams, and provision for jacking. Heave is eliminated by removal of dehydrated material to a depth of 8 to 12 ft, and replacement by granular fill, or belled caissons, founded near the water table and reinforced to resist tensile forces, supporting floor between caissons with opening or compressible filler beneath floors. Divert rainwater and surface runoff away from structure. Rough excavate no closer than one-half foot to final subgrade and protect exposed shale with a spray or mop coat of bitumen. When ready for foundations, excavate to final grade and pour concrete immediately over a spray or mop coat of bitumen.</td>
</tr>
<tr>
<td>Construction of light buildings on clays of high activity, highly preconsolidated with fractures and slickensides, in climate where hot summers alternate with wet winters. (Southern England, as an example.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction of light to medium load structures in hot, arid climate where the free surface evaporation is several times larger than annual rainfall. Difficulties are greatest in fractured and slickensided clay of high activity, with low water table and maximum deficiency of evaporation over rainfall. (South Africa, as an example.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chemical changes:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavation and exposure of clayey shales or shales containing pyrite (iron sulphide) or anhydrite (calcium sulphate).</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8 (continued) Heave from Volume Change
PROCEDURE FOR ESTIMATING TOTAL SWELL UNDER STRUCTURE LOAD.
1. Obtain representative undisturbed samples of the shallow clay stratum at a time when capillary stresses are effective; i.e., when not flooded or subjected to heavy rain.
2. Load specimens (at natural water content) in consolidometer under a pressure equal to the ultimate value of overburden for high ground water, plus weight of structure. Add water to saturate and measure swell.
3. Compute final swell in terms of percent of original sample height and plot swell versus depth, as in the left panel.
4. Compute total swell which is equal to the area under the percent swell versus depth curve. For the above example:
   \[ \text{Total Swell} = 0.10 \times \left( \frac{8.2 - 1.0}{2.8/100} \right) = 0.10 \text{ft.} \]

PROCEDURE FOR ESTIMATING UNDERCUT NECESSARY TO REDUCE SWELL TO AN ALLOWABLE VALUE.
1. From percent swell versus depth curve plot relationship of total swell versus depth at the right. Total swell at any depth equals area under the curve at left, integrated upward from the depth of zero swell.
2. For a given allowable value of swell, read the amount of undercut necessary from the total swell versus depth curve. For example, for an allowable swell of 0.03 ft, undercut required = 4.6 ft. Undercut clay is replaced by an equal thickness of nonswelling compacted fill.

FIGURE 22
Computation of Swell of Desiccated Clays

Figure 22
Computation of Swell of Desiccated Clay
8. REFERENCES

1. Department of Civil Engineering, University of California, Berkeley, CA, Stresses and Deflections in Foundations and Pavements, Fall, 1965.


