SETTLEMENT ANALYSES
(Saturated Cohesive Soils)

1. Review of 1-D Loading ($S=100\% \rightarrow \varepsilon_0 = \varepsilon_S$)
   1.1 Stress Paths: Low Initial OCR
   1.2 " " : High Initial OCR
   1.3 Settlement Analysis

2. Overview of 2-D \& 3-D Loading ($S=100\%$)
   2.1 Field Condition
   2.2 Components of Settlement
   2.3 Stress Path for Centerline Element with Local Yielding

3. Estimation of Final Consolidation Settlement (2-D Loading)
   3.1 Stress Paths for Moderate OCR Clay: No Yielding (Fig. II-5-1)
   3.2 Lambe's Stress Path Method
   3.3 Conventional "1-D" Analysis ($p_{cf} = p_{cfr}$)
   3.4 Skempton-Bjerrum (1957) \& Ladd's Modification (Fig. II-5-2)

4. Initial Settlement
   4.1 Approaches
   4.2 D'Appolonia et al. (1971) Method

5. Summary and Recommendations
   5.1 Conventional Practice
   5.2 "Stiff" Ground Condition
   5.3 "Soft" Ground Condition

Sheets A \& B: Charts for estimating $p_{cf} \& \varepsilon_0$ D'Appolonia et al. (1971)
C: Skempton-Bjerrum et al. $\mu = A = f(H/B \& L/B)$
D: Information on PPG \& DSM Litigation (CCD = Expert Witness)
Part V-5  SETTLEMENT ANALYSES

1. REVIEW 1-D LOADING (Sot. Clay, \( B = 100 \), \( \nu_0 = \Delta \sigma_v \))

1.1 Stress Paths: Low Initial OCR

1.2 Stress Paths: High Initial OCR

1.3 Settlement Analysis

\[ P_{ef} = Z H_0 \cdot e_{ef} \]

\[ e_{ef} = RL \log \left( \frac{\sigma_{vo}' + Cr(\sigma_{vo}' - \sigma_{ho}' - \Delta \sigma_v)}{\sigma_{vo}' - \sigma_{ho}'} \right) \]

\[ P_c = U \cdot P_{ef} \]

\[ U = f(T_v = t C_v / H_d^2) \]

\[ C_v = k_v / m_v \cdot k_w \]

<table>
<thead>
<tr>
<th>OCR</th>
<th>Recompression</th>
<th>Combined</th>
<th>Virgin</th>
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<tbody>
<tr>
<td>Low</td>
<td>High</td>
<td>High+Low</td>
<td>Low (DM-2 CV vs. GW)</td>
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\[ H' = H \cdot \sqrt{C_v(NC) / C_v(OC)} \]

\( \Delta \sigma_v \)

\( \sigma_{vo}' \)

\( \sigma_{ho}' \)

\( \sigma_{hf}' \)

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Part IV-5 SETTLEMENT ANALYSES

2. OVERVIEW 2.3 3-D LOADING (S=100%)

2.1 Field Condition

2.2 Components of Settlement

2.3 Stress Path For 3 Element with Local Yielding
3. ESTIMATION OF FINAL CONSOLIDATION SETTLEMENT
(For 2D & 3D Loadings)

3.1 Stress Paths for Moderate OCR Clay: No Yielding

See Fig 5-1 (ps) \( \sigma_c = \Sigma (H_i \cdot \epsilon_i) \)

3.2 Laub's Stress Path Method

1. Procedure & corresponding ESP: Duplicates actual ESP & E
   via lab triaxial compression with measurement of \( E_{\text{vertical}} \)

2. Practical problems in applying:
   1) Sample disturbance
   2) Costly sophisticated lab testing (+ should reduce \( \sigma_c \) during consolidation)
   3) What to do if local yielding occurs?

3.3 Conventional "1-D" Analysis \( \sigma_c = \sigma_{\text{est}} \)

1. Procedure (via Elastic Stress Distribution)
   \[
   \sigma_{\text{est}} = \Sigma H_i \cdot \epsilon_{\text{est}} \quad \sigma_{\text{est}} = R R \log \frac{\sigma_c}{\sigma_{\text{vo}}} + C R \log \frac{\sigma_{\text{vo}}}{\sigma_p}
   \]

2. Why approach should overestimate \( \sigma_c \) when \( \sigma_c < \sigma_p \), i.e.,
   during recompression? Since \( \Delta u < \Delta \sigma_v \rightarrow \) starting from \( \sigma_{\text{vu}} > \sigma_v \)

3.4 Skempton-Bjerrum (1957): Ladd's Modification

1. Procedure recognizes that \( \Delta u \neq \Delta \sigma_v \), but assumes 1-D compression during consolidation: \( \therefore \) ESP =

2. \[
\sigma_{\text{vu}} = \sigma_{\text{vo}} + \Delta \sigma_v - \Delta u = \sigma_{\text{vo}} + (1-A)(\Delta \sigma_v - \Delta \sigma_h)
\]
   \( A = \frac{A}{A+1} \)

3. Chart solution (attached sheet) for \( m_v \) constant \( \rightarrow \sigma_c = \sigma_{\text{est}} \)
   But if all compression i.e., \( \sigma_{\text{vo}} \leq \sigma_p \), Why unsafe if \( \sigma_{\text{vu}}' \) significantly > \( \sigma_p \)?
   Because most f \( \sigma_c \) due to virgin compression
   \( \epsilon_m \) is applied to \( \sigma_{\text{est}} \) as in Fig 5-2 (p9)

4. CCL alternative approach:
   Replace \( \sigma_{\text{vo}} \) by \( \sigma_{\text{vu}} \), \( \sigma_{\text{est}} = R R \log \frac{\sigma_{\text{vu}}}{\sigma_{\text{vo}}} + C R \log \frac{\sigma_{\text{vu}}}{\sigma_p} \)

Example

Strip: H/L = 4
\( A = 0.5 \rightarrow \mu = 0.6 \)
Fig. IV5-1  Stress Paths: 3-D Undrained Loading and Consolidation
(ε for moderately OCR clay with NO YIELDING)
Fig. 7.5-2 Illustration of Why Skempton-Bjerrum (1957) Procedure Can Underestimate the Final Consolidation Settlement When Loading O.C. Clay Well Beyond In Situ $\sigma_p'$

Note: Skempton-Bjerrum $\varepsilon_{cf} = \mu \varepsilon_{oad}$ illustrated for $\mu = \Delta u / \Delta \sigma_v = 2/3$
4. INITIAL SETTLEMENT

4.1 Approaches:

1. Ignore
2. Non-linear finite element
3. "Semi-rational" chart solution

4.2 D'Appolonia et al. (1971) Method

D'Appolonia, Poulos & Ladd (1971) ASCE JSMFD V97 SM10
Foolth & Ladd (1981) ASCE TGEO V107 GT8

(i) General methodology (Bi-Linear model)

\[ P_e = \frac{q_B B}{E_u} I_p \]

- Influence factor (Fig. 6- Sheet A)

* How obtain \( E_u \)? Difficult to measure reliably since affected by many variables (1.322)
  - Usually empirically approach via \( E_u / S_u = c_u \)

* Values of \( E_u / S_u = 100 \):
  - Plastic-organic soil of low \( F \)
  - Lean-brittle clay of high \( F \)

\[ \text{Tank } B = 100', H/8 = 1, \quad g = 3000 \text{ psf}, \quad F = 1.5 \]

\[ P_e (\text{in}) = \frac{2100}{E_u / S_u} = 400 \quad \text{for } E_u / S_u = 2000 \]

\[ (q_{ult} = 62 \text{ pcf}) \quad I_p = 0.42 \quad \frac{5}{2}(3000)(100) \quad (0.42)(12) \]
4.2 Continued

(3) Adjustment for local yielding (See attached Fig. 7 & 8 Sheet B)

- \( P_i = \frac{P_e}{S_R} \)

- \( S_R \) (Fig. 8)

- \( f = \frac{\sigma_{vo} - \sigma_{ho}}{2S_u} = \frac{1}{2S_u} (1-K_0) = \frac{(1-K_0)}{2S_u} \frac{\sigma_0}{\sigma_{vo}} \) \( + \frac{3}{5} \frac{Q_i}{\sigma_{vo}} \) definition \( \times \) Mode of failure

(Fig. 7)

- 1st Yielding at \( F \) = factor \( F = 4-6 \) for NC (\( t_0 < 0.7 \))

- Clay \( F \) = factor \( F_1 = L \) & \( t_0 \) (\( t_0 \) = few inches)

- For NC \( t_0 > 0.7 \)

5. SUMMARY & RECOMMENDATIONS

5.1 Conventional Practice

Although \( P_T = P_i + P_{cf} \), usually assumed that \( P_T = P_{load} \)

5.2 "Stiff" Ground Condition (When \( \overline{\sigma}_{vo} < \sigma_p^f \), All recompresion)

- \( \frac{P_i}{P_{cf}} \rightarrow \frac{1}{2} \pm \frac{1}{4} \), i.e. \( P_i \) is large portion of \( P_T \)

- \( P_{load} \geq P_{cf} \), because \( \Delta u \ll \Delta \sigma_v \), i.e. \( \Delta \sigma_u > 1 \)

- \( P_{cf} \) usually small (< few inches)

- "Using \( P_T = P_{load} \) is usually reasonable due to compensating errors"!

5.3 "Soft" Ground Condition (When \( \overline{\sigma}_{vo} > \sigma_p^f \), All recompression & Virgin)

- \( P_{cf} \approx P_{load} \) if recompression small compared to virgin

- Magnitude of \( P_i \) usually can be ignored except when:

  1) Low \( E_u / S_u \) (high Ip and/or highly organic)

  2) Low Ip and large \( t_p \) → undrained creep

- PPG vs DFM (CCL 1st experience as expert witness) Sheet D
Undrained Elastic Settlement Computation for Uniform Loading on Elastic Layer

Adapted from D’Appolonia et al. (1971), Foot & Ladd (1981)

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### Table: Applied Shear Stress Ratio $t_h/c_u$

<table>
<thead>
<tr>
<th>No.</th>
<th>Description</th>
<th>$c_u/s_{vc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Portsmouth Sensitive CL Clay</td>
<td>0.20</td>
</tr>
<tr>
<td>2</td>
<td>Boston CL Clay LL = 65, $t_p = 41$</td>
<td>0.20</td>
</tr>
<tr>
<td>3</td>
<td>Bangkok CH Clay LL = 65, $t_p = 41$</td>
<td>0.27</td>
</tr>
<tr>
<td>4</td>
<td>Mane Organic CH - CH CLay LL = 65, $t_p = 38$</td>
<td>0.285</td>
</tr>
<tr>
<td>5</td>
<td>AGS CH Clay LL = 71, $t_p = 40$</td>
<td>0.255</td>
</tr>
<tr>
<td>6</td>
<td>Atchafalaya CH Clay LL = 95, $t_p = 75$</td>
<td>0.24</td>
</tr>
<tr>
<td>7</td>
<td>Taylor River Peat $W_u = 500$</td>
<td>0.46</td>
</tr>
</tbody>
</table>

*From Ladd & Edgers (1972) MIT for Dames & Moore MIT for Haley & Aldrich*

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(a) Normalized Secant Modulus vs. Stress Level for Normally Consolidated Soils

(b) Normalized Secant Modulus vs. Overconsolidation Ratio
Relationship between settlement ratio and applied stress ratio for strip foundation on homogenous isotropic elastic layer.

Note:

- $c_u = 0.5(\sigma_1 - \sigma_3)_f$ from $C_{ku}$ Triaxial or Plane Strain Tests, $K_u$ from Brooker and Ireland (1965) for ME. Organic Clay.
Fig 5-9 INFLUENCE OF A PARAMETER AND GEOMETRY ON THREE DIMENSIONAL VS. ONE DIMENSIONAL FINAL CONSOLIDATION SETTLEMENT
A. Site Conditions

- PPG Tank Farm (7M)
  - 9 in-house geot. engs.
  - Two VCM tanks = problem

- Diagram: Sand (20') - Replacement Fill - Organic Clay (40') - 3-5' Sand

B. History of Events

1) Initial Design (after geot. consulting firms refused to bid on work)
   - Driller takes samples →盆地 testing lab; data → local perf: \[ \sigma_v = 4000 \text{ psf} \]
   - Pressurization starts

2) During Construction, Sep 1970 (DM called in to evaluate replacement fill; notb. stopped)
   - DM involved in some deep boring, found voids, \( \sigma_v - \sigma_v' = 1200 \text{ psf} \)
   - Redesign \( \sigma_v' = 3300 \), \( F = 1.5 \), let. \( P = 18'' \) with accuracy of ±25%
   - Worried about \( \sigma_v \) due to plastic flow at \( q > 3000 \) (Before D'Appo et al. method)

3) Water Testing (late 1970) → \( P = 2' \)
   - PPG needs DM → computer analyses & lab tests data →
     Conclusion that must have rapid \( \sigma_v' \) much lower \( \sigma_v \)
   - 2nd WT (after reducing of tanks) failed DM prediction. (So DM terminated)

4) Early 1975 (after 4 yr. operation at \( q < 2000 \) + \( P \) for awhile)
   - PPG tank full capacity, but worried about tank safety
   - PPG decides against hiring any geot. consultant to look at problem
   - PPG adds third tank at price of $2M
   - PPG sued DM for $2M for professional negligence "±25%" guarantee

5) Federal Court trial 6 June 1979
   - CCL vs. PPG E&W v. E&W in PPG → Good lawyer most important (a la OT)

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