12/97
12/98

SETTLEMENT ANALYSES
(Saturated Cohesive Soils)

Sheets A & B : Charts for estimating ρ_i à la D'Appolonia et al. (1971)

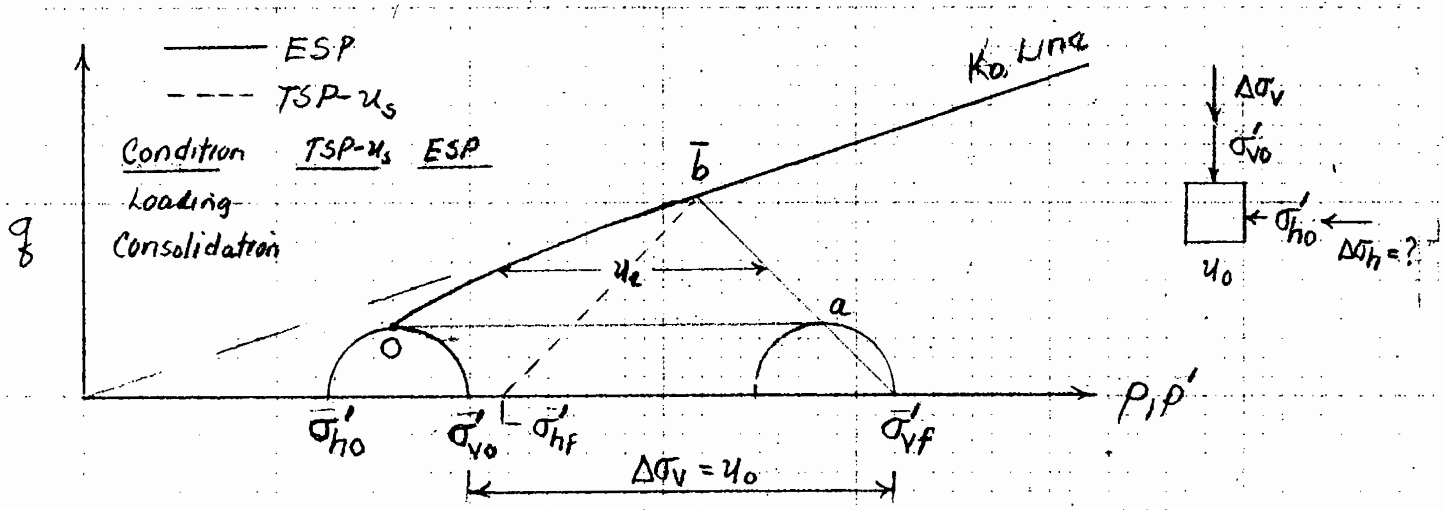
C : Skempton-Bjerrum et al. μ vs A as f (H/B & L/B)

D : Information on PPG vs D&M Litigation (CCL = expert witness)

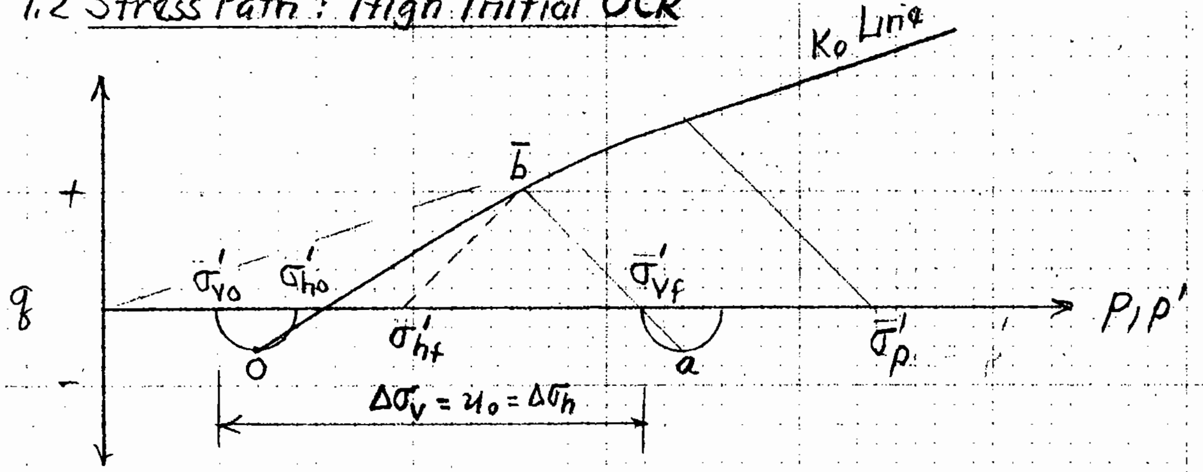
Part V-5 SETTLEMENT ANALYSES

1, REVIEW 1-D LOADING (Sat. Clay, $B = 1.00$, $u_o = \Delta\sigma_v$)

1.1 Stress Paths : Low Initial OCR



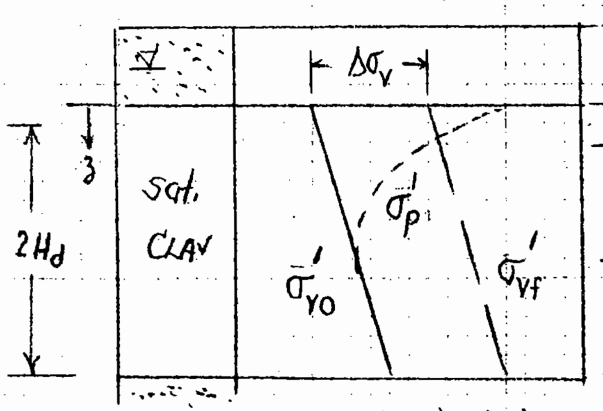
1.2 Stress Path : High Initial OCR



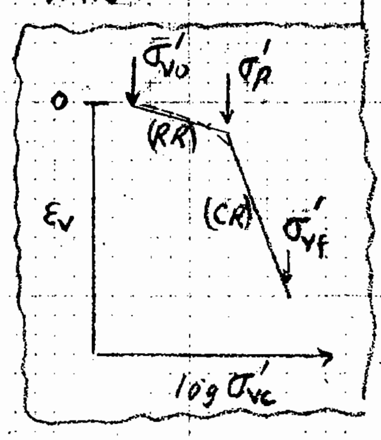
1.3 Settlement Analysis

$$\rho_{cf} = \Sigma H_i \varepsilon_{cf} \; ; \quad \varepsilon_{cf} = RR \log \frac{\bar{\sigma}'_p}{\bar{\sigma}'_{vo}} + CR \log \frac{\bar{\sigma}'_{vf}}{\bar{\sigma}'_p}$$

$$\rho_c = \bar{U}_v \, \rho_{cf} \qquad \bar{U}_v = f\left(T_v = t c_v / H_d^2\right)$$



	$c_v = k_v / m_v \gamma_w$
All recompression	High*
Combined	High→Low
All virgin	Low (DM-7 c_v vs w_L)

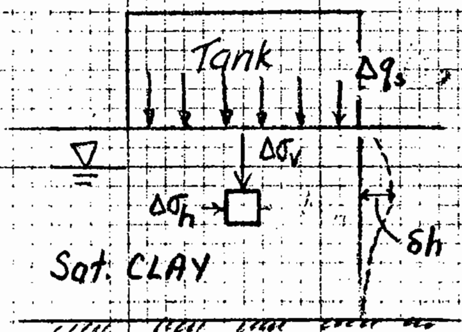


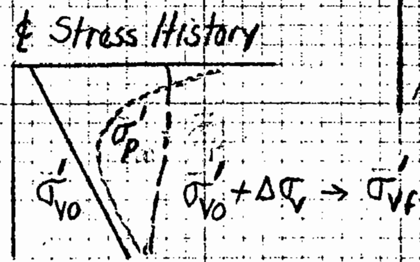
* $c_v(OC) \approx (5\text{–}10)\, c_v(NC) \rightarrow H' = H\sqrt{c_v(NC)/c_v(OC)}$

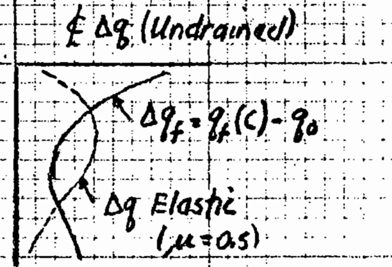
Part V-5 SETTLEMENT ANALYSES

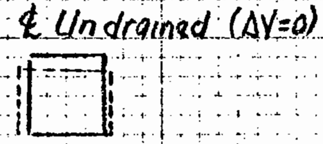
2. OVERVIEW 2 & 3-D LOADING (S = 100%)

2.1 Field Condition

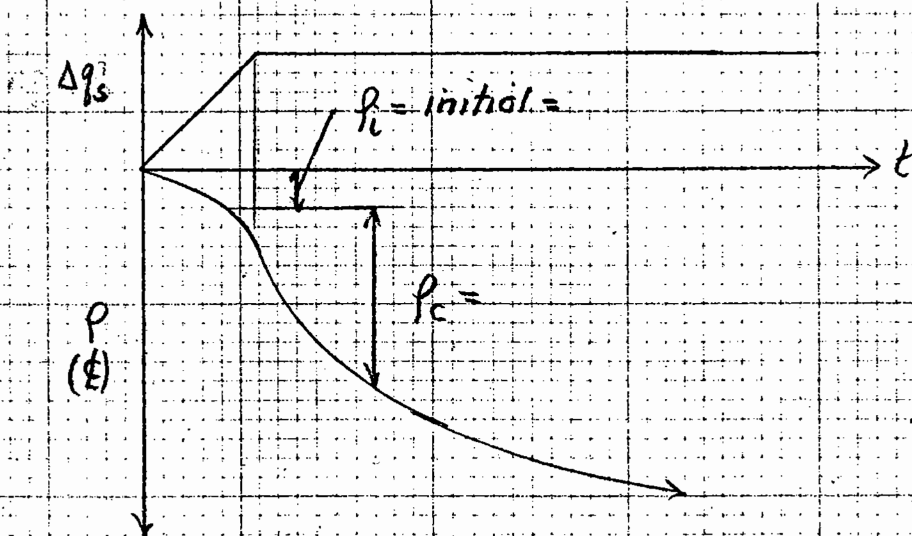




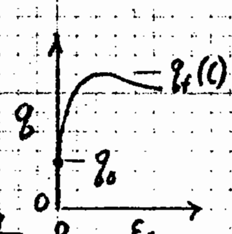


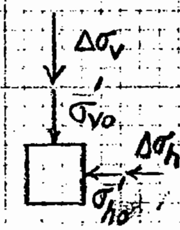


2.2 Components of Settlement

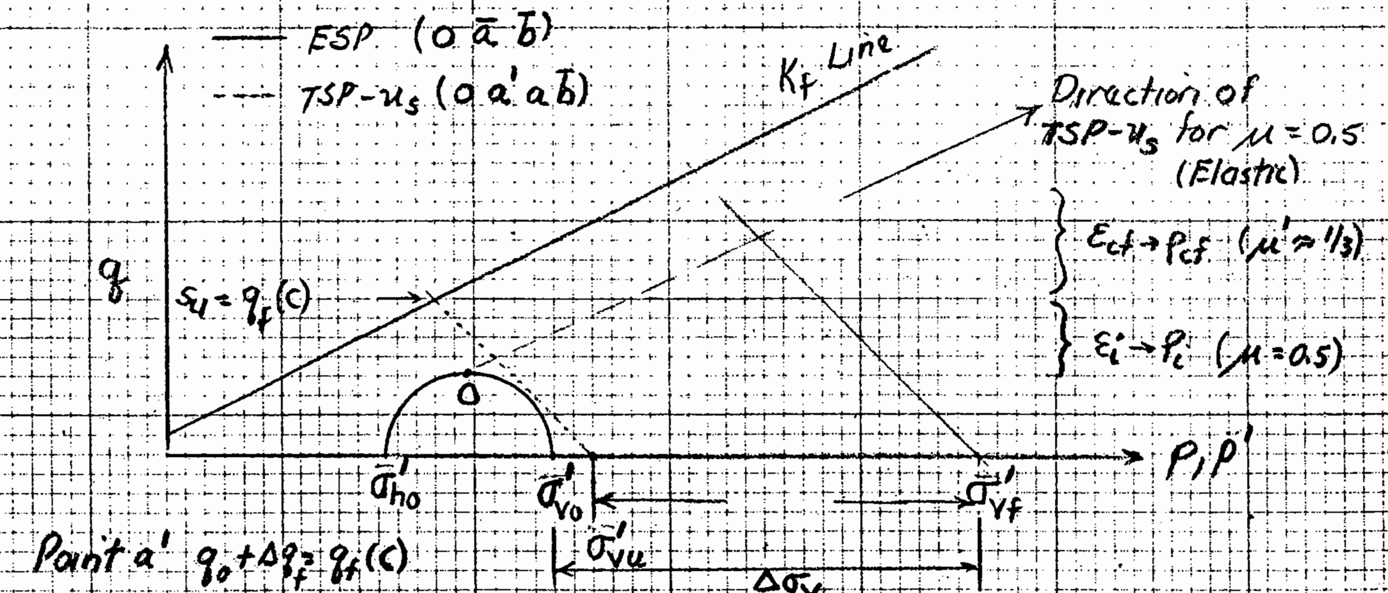


$$\rho_{Total} = \rho_i + \rho_{cf} = \rho_T$$





2.3 Stress Path For ℄ Element with Local Yielding



Point a' $q_o + \Delta q_f = q_f(C)$

Part V-5 SETTLEMENT ANALYSES

3. ESTIMATION OF FINAL CONSOLIDATION SETTLEMENT (For 2 & 3-D Loadings)

3.1 Stress Paths for Moderate OC Clay: No Yielding

See Fig V5-1 (p5) $\rho_{cf} = \Sigma(H_i \cdot \varepsilon_{cf})$

3.2. Lambe's Stress Path Method

(1) Procedure & corresponding ESP: Duplicates actual ESP c̄ ū via lab triaxial compression with measurement of $\varepsilon_{vertical}$

(2) Practical problems in applying:

1) Sample disturbance
2) Costly sophisticated lab testing (+ should reduce σ_h during consolidation)
3) What to do if local yielding occurs?

3.3 Conventional "1-D" Analysis $\boxed{\rho_{cf} = \rho_{oed}}$

(1) Procedure

* • Compute $\Delta\sigma_v$ (via Elastic Stress Distribution) vs depth and assume initial $\Delta u = u_o = \Delta\sigma_v$ *

• $\rho_{oed} = \Sigma H_i \cdot \varepsilon_{oed}$; $\varepsilon_{oed} = RR \log \frac{\sigma'_p}{\sigma'_{vo}} + CR \log \frac{\sigma'_{vf}}{\sigma'_p}$

• Corresponding ESP = ____

(2) Why approach should overestimate ρ_{cf} when $\sigma'_{vf} < \sigma'_p$, i.e. during recompression? Since $\Delta u < \Delta\sigma_v \rightarrow$ starting from $\sigma'_{vu} > \sigma'_{vo}$

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

(1) Procedure recognized that $\Delta u \neq \Delta\sigma_v$, but assumes 1-D compression during consolidation. ∴ ESP = ____

(2) $\sigma'_{vu} = \sigma'_{vo} + \Delta\sigma_v - \Delta u = \sigma'_{vo} + (1-A)(\Delta\sigma_v - \Delta\sigma_h)$
& $u = \Delta\sigma_h + A(\Delta\sigma_v - \Delta\sigma_h)$

(3) Chart solution (attached sheet C) for m_v = constant $\rightarrow \rho_{cf} = \mu \rho_{oed}$

Why unsafe if σ'_{vf} significantly $> \sigma'_p$?
Because most of ρ_{cf} due to virgin compression & μ is applied to ρ_{oed} à la Fig. V5-2 (p6)

But ok if all recompression i.e. $\sigma'_{vf} \leq \sigma'_p$

EXAMPLE
Strip H/B = 4
$A = 0.5 \rightarrow \mu = 0.6$

(4) CCL alternative approach:

Replace σ'_{vo} by $\sigma'_{vu} \rightarrow \varepsilon_{cf} = RR \log \frac{\sigma'_p}{\sigma'_{vo} \rightarrow \sigma'_{vu}} + CR \log \frac{\sigma'_{vf}}{\sigma'_p}$

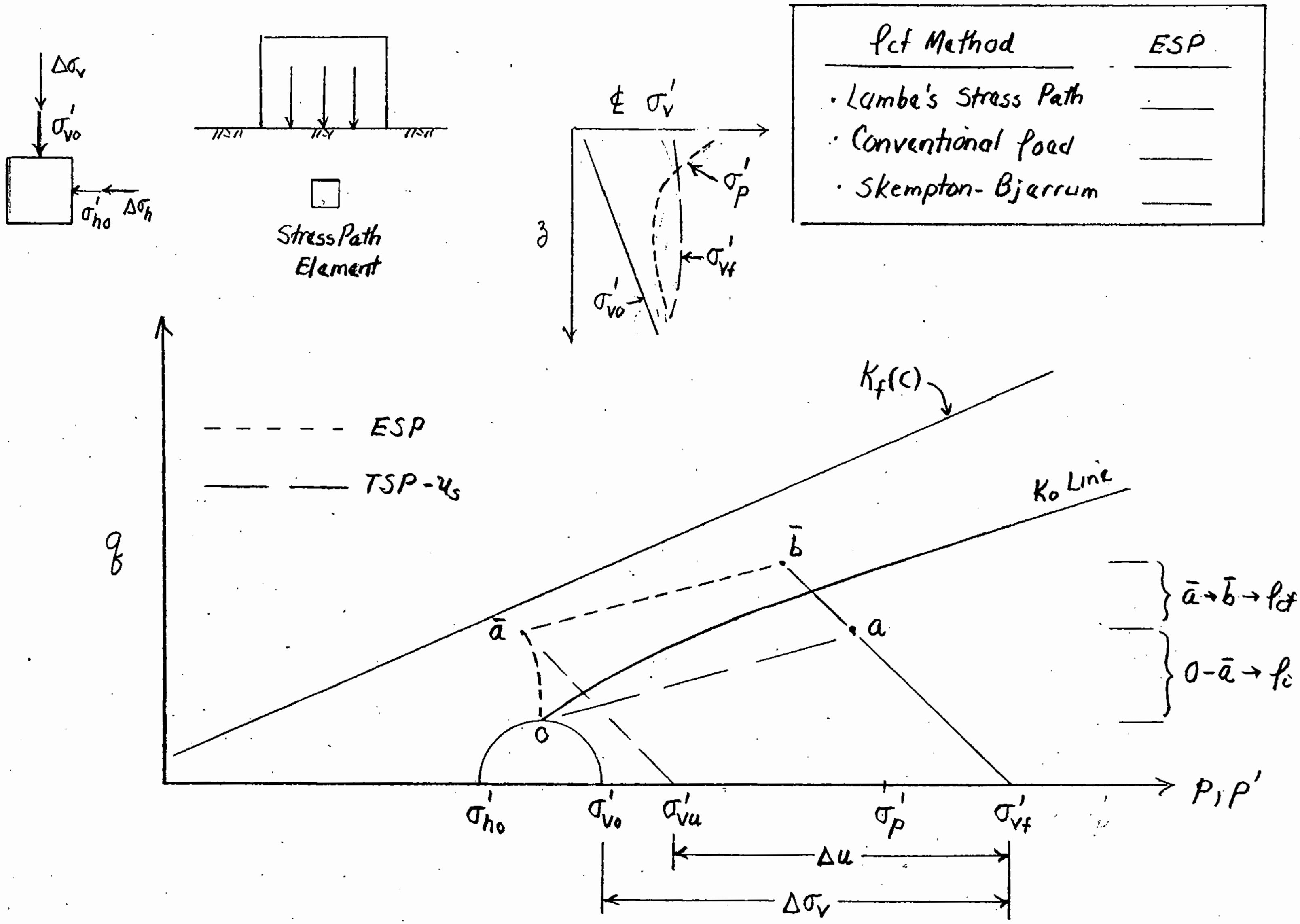
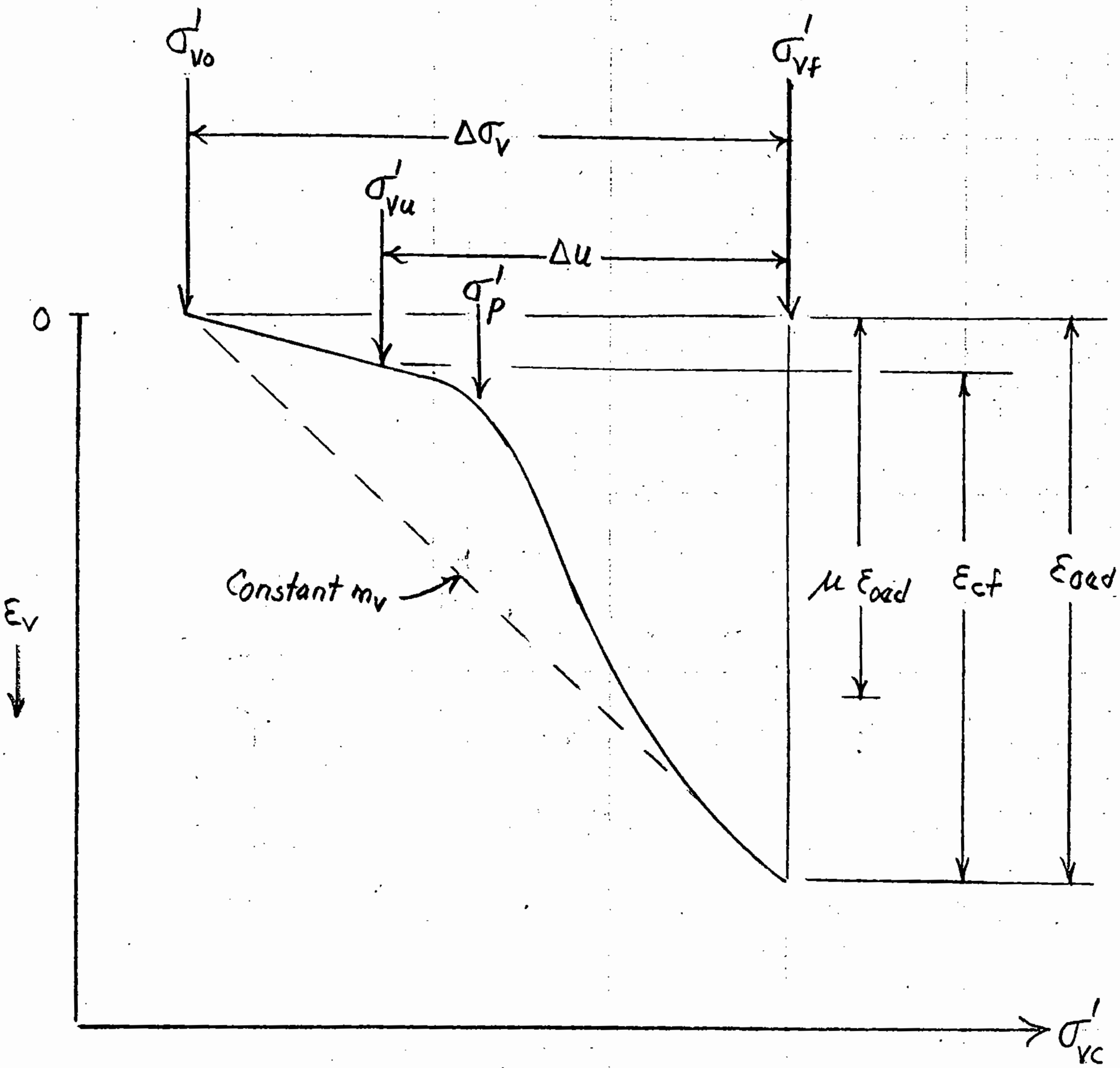


Fig. V 5-1 Stress Paths: 3-D Undrained Loading and Consolidation
(℄ for moderately OC clay with NO YIELDING)

Fig. V5-2 Illustration of Why Skempton-Bjerrum (1957) Procedure Can Underestimate the Final Consolidation Settlement When Loading O.C. Clay Well Beyond In Situ σ_p'



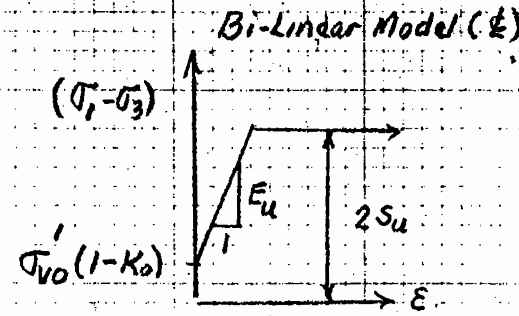
NOTE: Skempton-Bjerrum: $\varepsilon_{cf} = \mu\,\varepsilon_{oed}$ illustrated for $\mu = \Delta u / \Delta\sigma_v = 2/3$

Part V-5 SETTLEMENT ANALYSES

4. INITIAL SETTLEMENT

4.1 Approaches

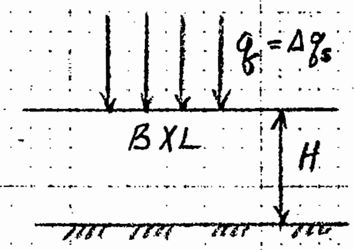
(1) Ignore
(2) Non-linear finite element
(3) "semi-rational" chart solution

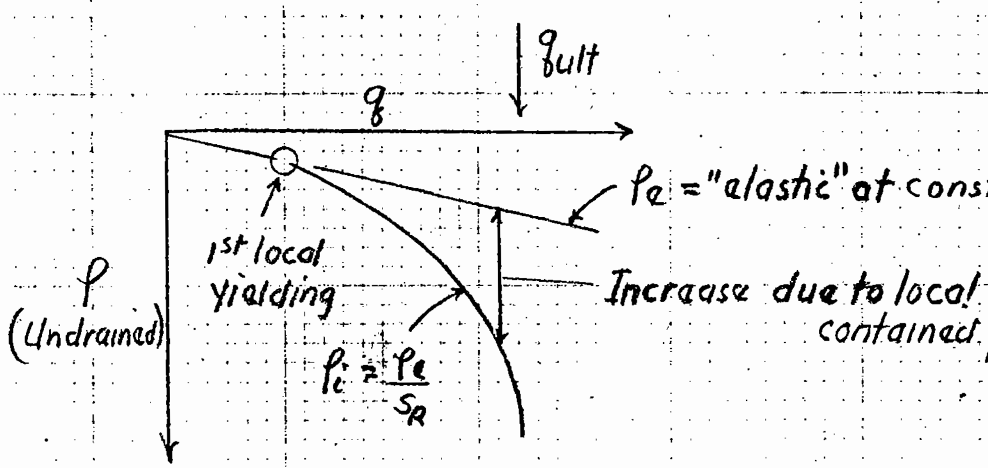


4.2 D'Appolonia et al. (1971) Method

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

(1) General methodology (Bi-Linear model)





(2) $\rho_e = \frac{qB}{E_u} I_p$ ← 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 $E_u/s_u \equiv c_u$

- Values of E_u/s_u = 100 : Plastic-organic soil at low F
  (Fig. 5 - Sheet A) ↓
  2000 : Lean-brittle clay at high F
  } Usually $E_u/s_u \geq 400 \pm 100$

Tank $B = 100'$ $H/B = 1$, $q = 3000$ psf, $F = 1.5$, $s_u = 725$ psf, $(q_{ult} = 6.2\, s_u)$ } → $\rho_e(\text{in.}) \approx \frac{2100^*}{E_u/s_u}$ = ___ for $E_u/s_u = 2000$
= ___ " " = 100

$I_p = 0.42$ $\quad *\frac{1}{2}\frac{(3000)(100)}{725}(0.42)(12)$

4.2 Continued

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

- $\rho_i = \rho_e / S_R$
- S_R (Fig 8)
- $f = \frac{\sigma_{vo} - \sigma_{ho}}{2 s_u} = \frac{\sigma'_{vo}(1-K_o)}{2 s_u} = \frac{(1-K_o)}{2 s_u/\sigma'_{vo}} = \frac{q_o}{q_f}$ (Fig 7)
  - * s_u/σ'_{vo} definition
  - * Mode of failure
- 1st yielding at
  - F = 4-6 ℄ NC ($f \approx 0.7$)
  - $F \geq 1.5$ Edge ($K_o = 1 \rightarrow f = 0$)

F = factor of safety

- For NC Clay: $F = 2$, $F \approx 1.1$; $\rho_c/\rho_e \approx 2$, ≈ 4 } From Fig. 8

5. SUMMARY & RECOMMENDATIONS

5.1 Conventional Practice

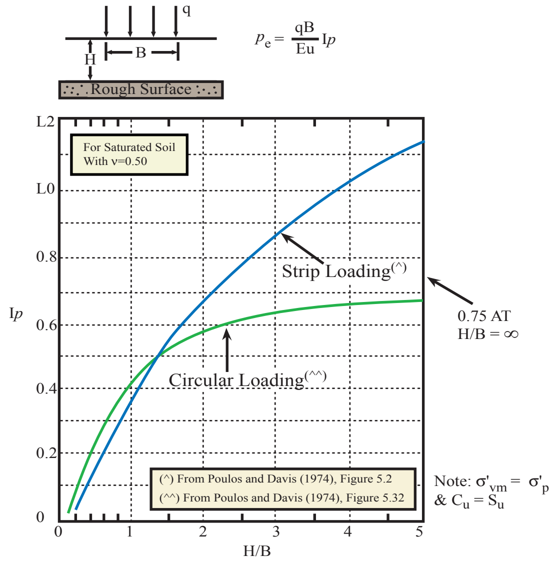
Although $\rho_T = \rho_i + \rho_{cf}$, usually assumed that $\rho_T = \rho_{oed}$

5.2 "Stiff" Ground Condition (When $\sigma'_{vf} < \sigma'_p$; All recompression)

- $\rho_i / \rho_{cf} \rightarrow 1/2 \pm 1/4$, i.e. ρ_i is large portion of ρ_T
- $\rho_{oed} > \rho_{cf}$, because $\Delta u \ll \Delta\sigma_v$, i.e. $\sigma'_{vu} \gg \sigma'_{vo}$
- ρ_{cf} usually small (< few inches)
- ∴ Using $\rho_T = \rho_{oed}$ is: Usually reasonable due to compensating errors!

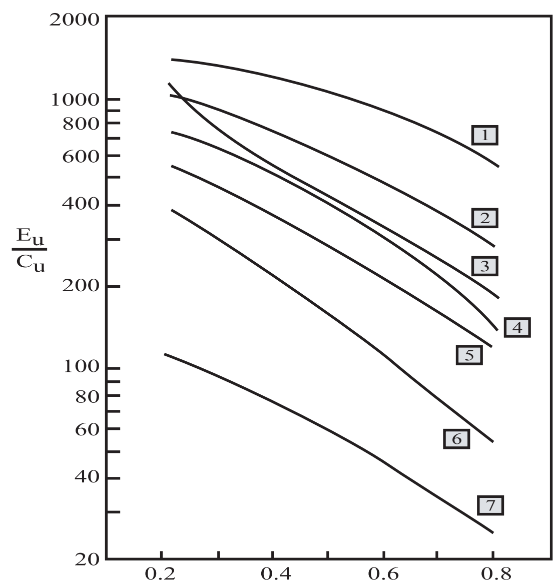
5.3 "Soft" Ground Condition (When $\sigma'_{vf} > \sigma'_p$; Recompr. & Virgin)

- $\rho_{cf} \approx \rho_{oed}$ if recompression small compared to virgin
- Magnitude of ρ_i usually can be ignored except when:
  1) low E_u/s_u (high I_p and/or highly organic)
  PLUS
  2) low F and large t_p → undrained creep
- PPG vs D&M (CCL 1st experience as an expert witness) Sheet D



Undrained Elastic Settlement Computation for Uniform Loading on Elastic Layer

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



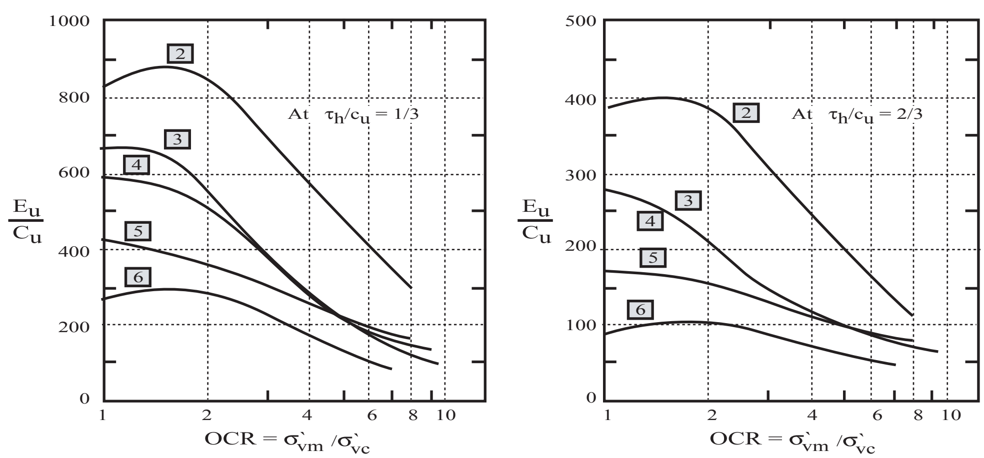
No.	Decription	C_u/σ'_{vc}
1	Portsmouth Sensitive CL Clay $S_1 \geq 10$, LL = 35, $I_p = 15$	0.20
2	Boston CL Clay LL = 41, $I_p = 22$	0.20
3	Bangkok CH Clay LL = 65, $I_p = 41$	0.27
4	Mane Organic CH - CH Clay LL = 65, $I_p = 38$	0.285
5	AGS CH Clay LL = 71, $I_p = 40$	0.255
6	Atchafalaya CH Clay LL = 95, $I_p = 75$	0.24
7	Taylor River Peat $W_N = 500$ %	0.46

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

Applied Shear Stress Ratio τ_h/c_u
(1/Factor of Safety)

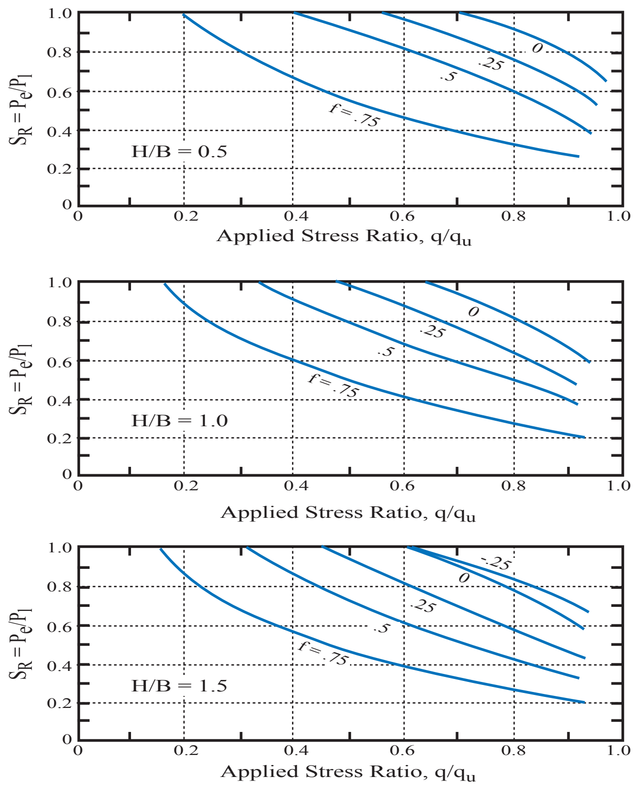
$E_u = 3\tau_h/\gamma$ τ_h = Applied Horizontal Shear Stress
$C_u = (\tau_h)_{max}$ γ = Shear Strain

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

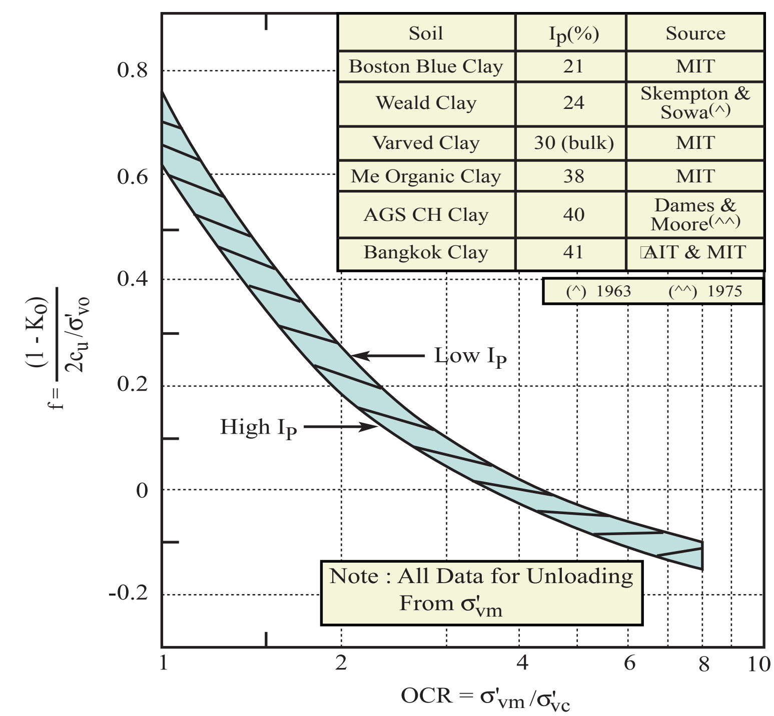


(b) Normalized Secant Modulus vs. Overconsolidation Ratio

A



Relationship between settlement ratio and applied stress ratio for strip foundation on homogenous isotropic elastic layer.



Soil	I_P(%)	Source
Boston Blue Clay	21	MIT
Weald Clay	24	Skempton & Sowa(^)
Varved Clay	30 (bulk)	MIT
Me Organic Clay	38	MIT
AGS CH Clay	40	Dames & Moore(^^)
Bangkok Clay	41	AIT & MIT

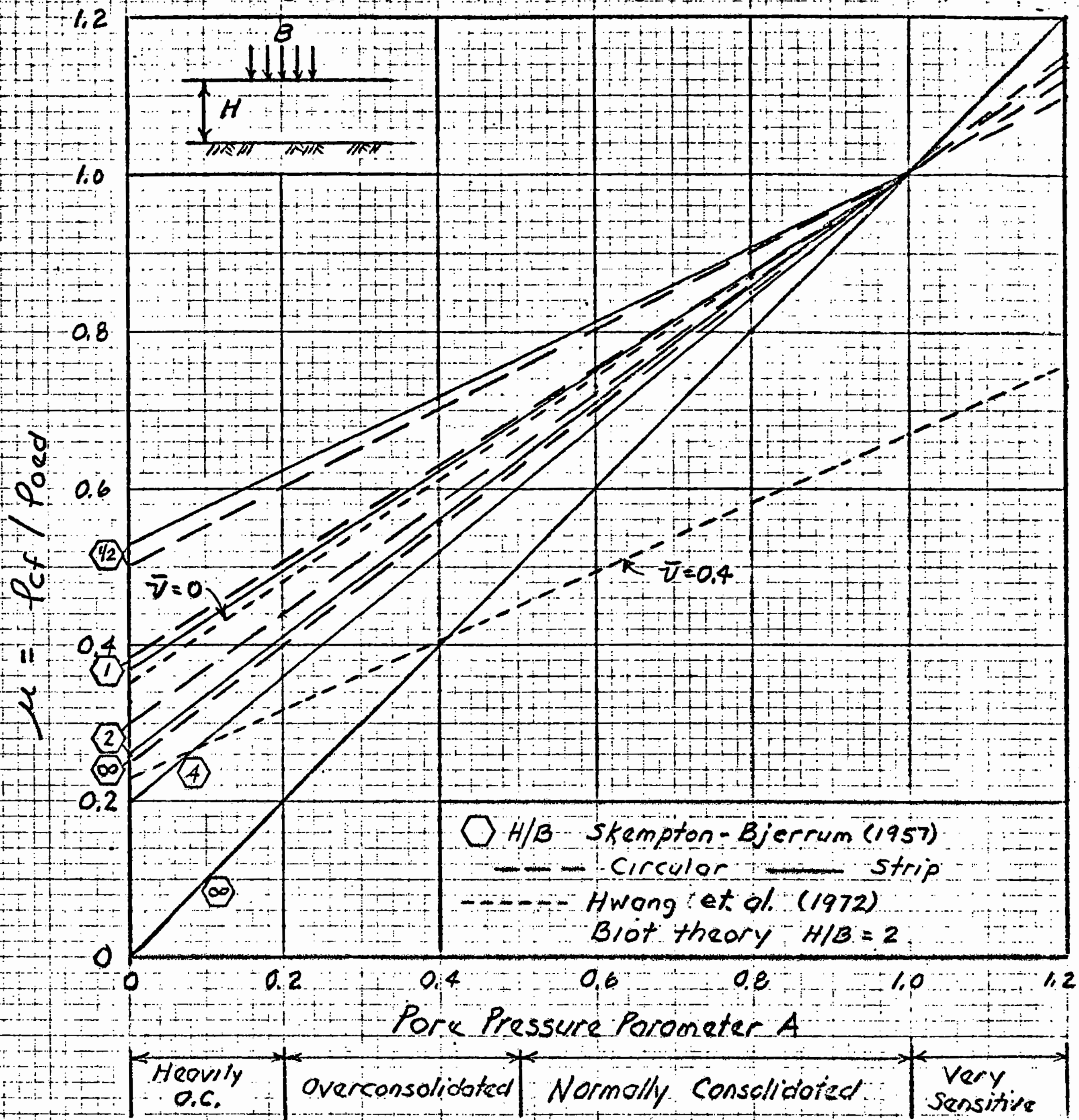
(^) 1963 (^^) 1975

Intial Settlement (p 2/2)

Note : $c_u = 0.5(\sigma_1 - \sigma_2)_f$ from CK_oU Triaxial or Plane Strain Tests, K_o from Brooker and Ireland (1965) for ME. Organic Clay.

B

CCL 1973 Settlement Notes



Meyerhoff (1958) Geot. Vol. 8 No. 2 p 101

$$\mu = 1 - \frac{3(1-A)}{4 + B/H}$$ for circular or square areas

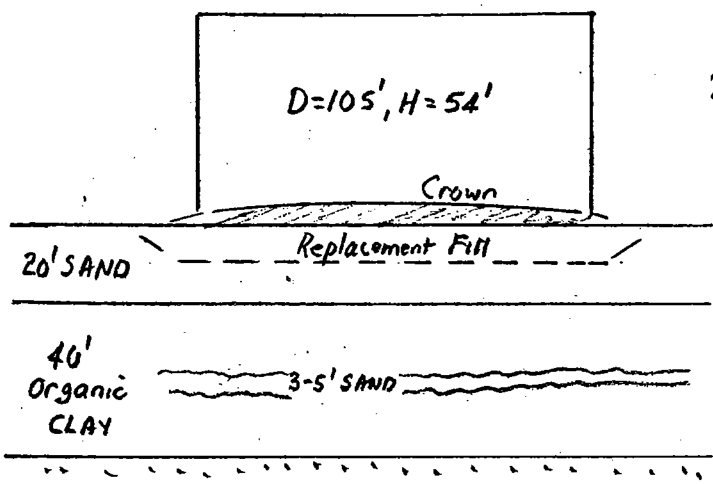
Fig 5-9 INFLUENCE OF A PARAMETER AND GEOMETRY ON THREE DIMENSIONAL VS. ONE DIMENSIONAL FINAL CONSOLIDATION SETTLEMENT

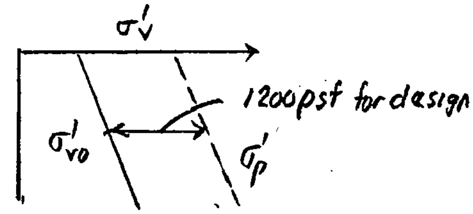
(C)

A. Site Conditions

PPG Tank Farm ($9M)
(no inhouse geot. engr.)

Two VCM Tanks → problem





B. History of Events

1) Initial Design (after geot. consulting firms refused to bid on work)
   - Driller takes samples → materials testing lab; data → local prof. → $q_a = 4000$ psf, $\rho = 3"$

2) During Construction, Sp. 1970 (D&M called in to evaluate replacement fill; const. stopped)
   - D&M insisted on some deep borings → few lab tests → $\sigma'_p - \sigma'_{vo} \approx 1200$ psf
   - Redesign $q_a = 3300$, $F \approx 1.5$, est. $\rho \approx 18"$ w/ est. accuracy of ±25%
   - Worried about ρ_i due to plastic flow at $q > 3000$ (Before D'App et al. method).
   - ∴ Recommended instrumentation during water testing (on approved plans) BUT PPG didn't install & terminated D&M

3) Water Testing (Late 1970) → $\rho = 2'$
   - PPG rehired D&M → computer analyses of lousy ρ vs t data → conclusion that must have rapid ρ_c & much lower σ'_p
   - 2nd WT (after relevelling of tanks) followed D&M predictions. (so D&M terminated)

4) Early 1975 (after 4 yr. operation at $q < 2000$ → ρ few inches)
   - PPG wants full capacity but worried about tank safety
   - PPG decides against hiring any geot. consultant to look at problem and puts both tanks on piles at cost of $2M
   - PPG sues D&M for $2M for professional negligence "±25%" = guarantee

5) Federal Court Trial w/ 6 Jurors 1979
   - CCL & HPA E.W. for D&M • E.D. E.W. for PPG → GOOD LAWYER MOST IMPORTANT (à la OJ)

(D)