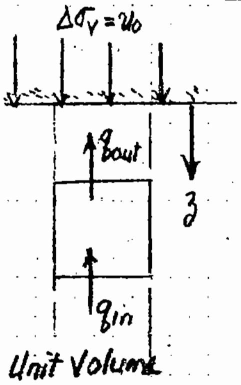
V-2 CONSOLIDATION AND SECONDARY COMPRESSION

Sheet A: $C_{\alpha\varepsilon}$ vs. C_c relationships for three soils

Part V-2 CONSOLIDATION (Chap. 27)

1. TERZAGHI THEORY (1-D loading, saturated soil)

1.1 Assumptions & Basic Eqn (Class Notes IV-2A p1)



From 1-D flow eqn. Chap. 18 with $B = 1.00$, constant σ_v $(\partial\sigma_v/\partial t = 0)$, Small strains ($1+e \approx$ constant) and constant soil properties (k, m_v & k/m_v)

$h = h_e + (h_p = h_{ss} + h_{ex})$

$\frac{\partial^2 h}{\partial z^2} \rightarrow \frac{\partial^2 h_{ex}}{\partial z^2} = \frac{1}{\gamma_w}\frac{\partial^2 u_e}{\partial z^2}$

$\Delta V\ H_2O = \Delta V\ Soil$

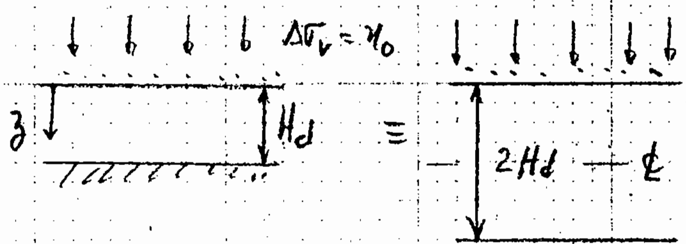
$$k_v \frac{\partial^2 h}{\partial z^2} = \frac{1}{(1+e)}\frac{\partial e}{\partial t} = \frac{\partial \varepsilon_v}{\partial t} \longrightarrow \begin{cases} \partial\varepsilon_v = -m_v\,\partial\sigma_v' = m_v\,\partial u_e \\ h = u_e/\gamma_w \quad \text{where } u_e = \text{EXCESS } u \end{cases}$$

$$\frac{k_v}{\gamma_w}\frac{\partial^2 u_e}{\partial z^2} = m_v \frac{\partial u_e}{\partial t} \longrightarrow c_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t}, \quad \text{where } c_v = \frac{k_v}{m_v \gamma_w} = \frac{k_v(1+e)}{a_v \gamma_w}$$

(m_v = coef. vol. change = $d\varepsilon_v/d\sigma_v'$)

c_v = coef. of consolidation (units of L^2/t; lab data usually reported in cm^2/sec)

1.2 Solution in Dimensionless Form



$Z = z/H_d \qquad U_e = u_e/u_0$

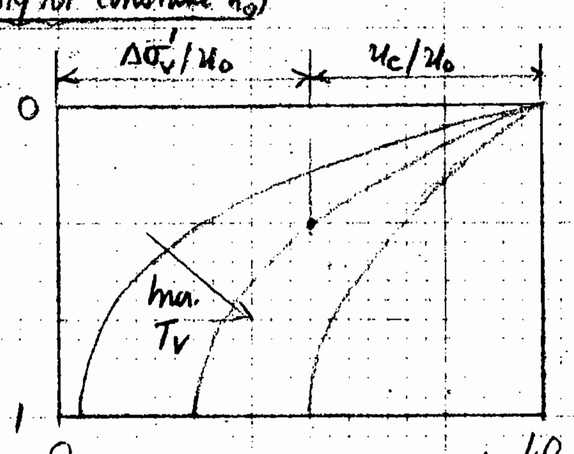
$T_v = c_v t / H_d^2$ = Time factor

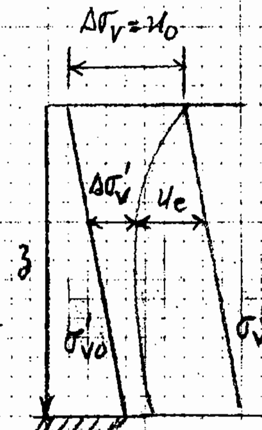
$$\therefore \frac{\partial^2 U_e}{\partial Z^2} = \frac{\partial U_e}{\partial T_v} \qquad (T_v = T \text{ in } L \& W)$$

(1) Degree of Consolidation (U_z) vs depth at varying time {For predicting & evaluating piezometer data}

Fig. 27.2 (Only for constant u_0)

$Z = \frac{z}{H_d}$





Predict: $t = \frac{T_v H_d^2}{c_v}$

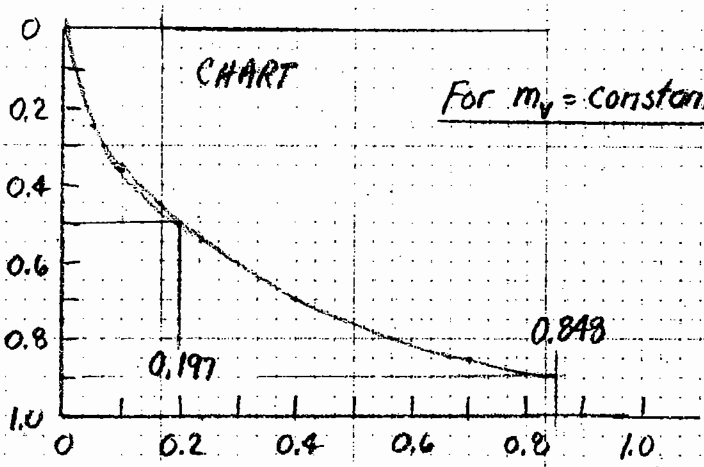
Evaluate: $c_v = \frac{\Delta T_v}{\Delta t} H_d^2$

$$U_z = 1 - \frac{u_e}{u_0} = \frac{\Delta\sigma_v'}{u_0}$$

- How get gradients? $i = \frac{dh}{dz} = \frac{d(u_e/\gamma_w)}{dz}$ ← From dU_z / From dZ

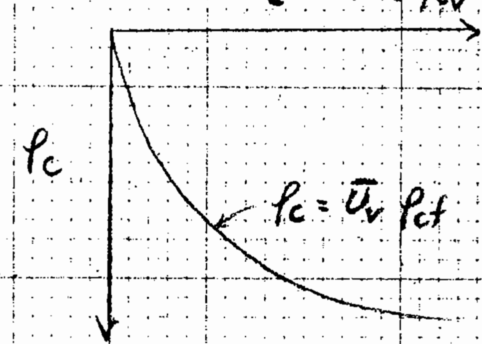
(2) Average Degree of Consolidation ($\bar{U}_v$)

$$\bar{U}_v = \underbrace{1 - \frac{u_e(\text{ave})}{u_o}}_{\text{Ave. } \Delta\sigma_v'/\Delta\sigma_v} = \rho_c/\rho_{cf} \quad \text{vs Time} \qquad \text{Fig. 27.3} \quad \text{(Any linear } u_o\text{)}$$



$$T_v = \frac{c_v t}{H_d^2}$$

Predicted settlement



Eval. ρ_c pt: $c_v = \frac{\Delta T_v}{\Delta t} H_d^2$

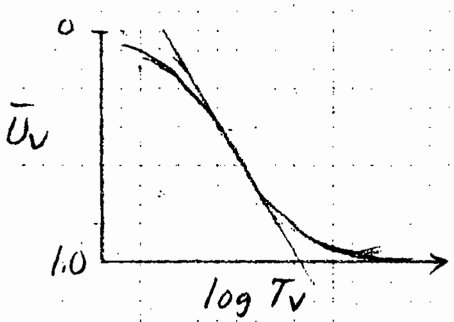
See Fig V2-1 (p3a) for $\bar{U}_v$ vs log T_v & Eqn.

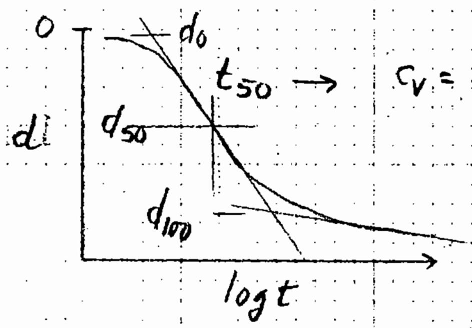
(3) Superposition $(1-\bar{U}) = (1-\bar{U}_v)(1-\bar{U}_h)$ ← Radial drainage with vertical drains

1.3 Coef. of Consolidation from Incremental Oedometer Tests

(1) Theory - Curve fitting à la Fig. 27.4

Casagrande log t



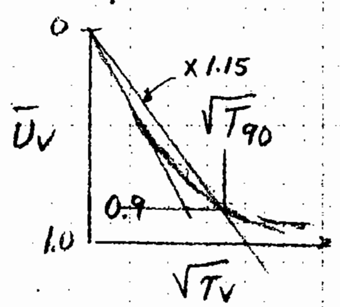


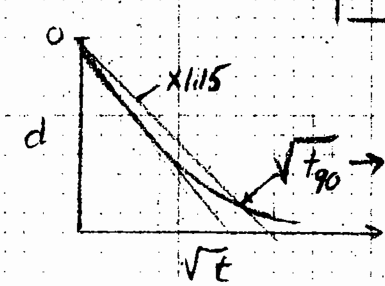
$t_{50} \rightarrow \quad c_v = \frac{\overset{0.197}{T_v} H_d^2}{t_{50}}$

NOTE: For $T_v > 0.2$: $\bar{U}_v = 1 - \frac{8}{\pi^2} e^{-\left(\frac{\pi^2 T_v}{4}\right)}$ ($\bar{U}_v > 50\%$)

Taylor $\sqrt{t}$ $\quad \bar{U}_v = \sqrt{\frac{4T_v}{\pi}}$ for $\bar{U}_v < 0.6$

NOTE: Use AVE. H_d and plot vs. AVE. σ_{vc}'





$\rightarrow \quad c_v = \frac{\overset{0.848}{T_v} H_d^2}{t_{90}}$

Note: Plotted from Table 1 of Scott (1961), JSMFD, ASCE, 87(SM1)

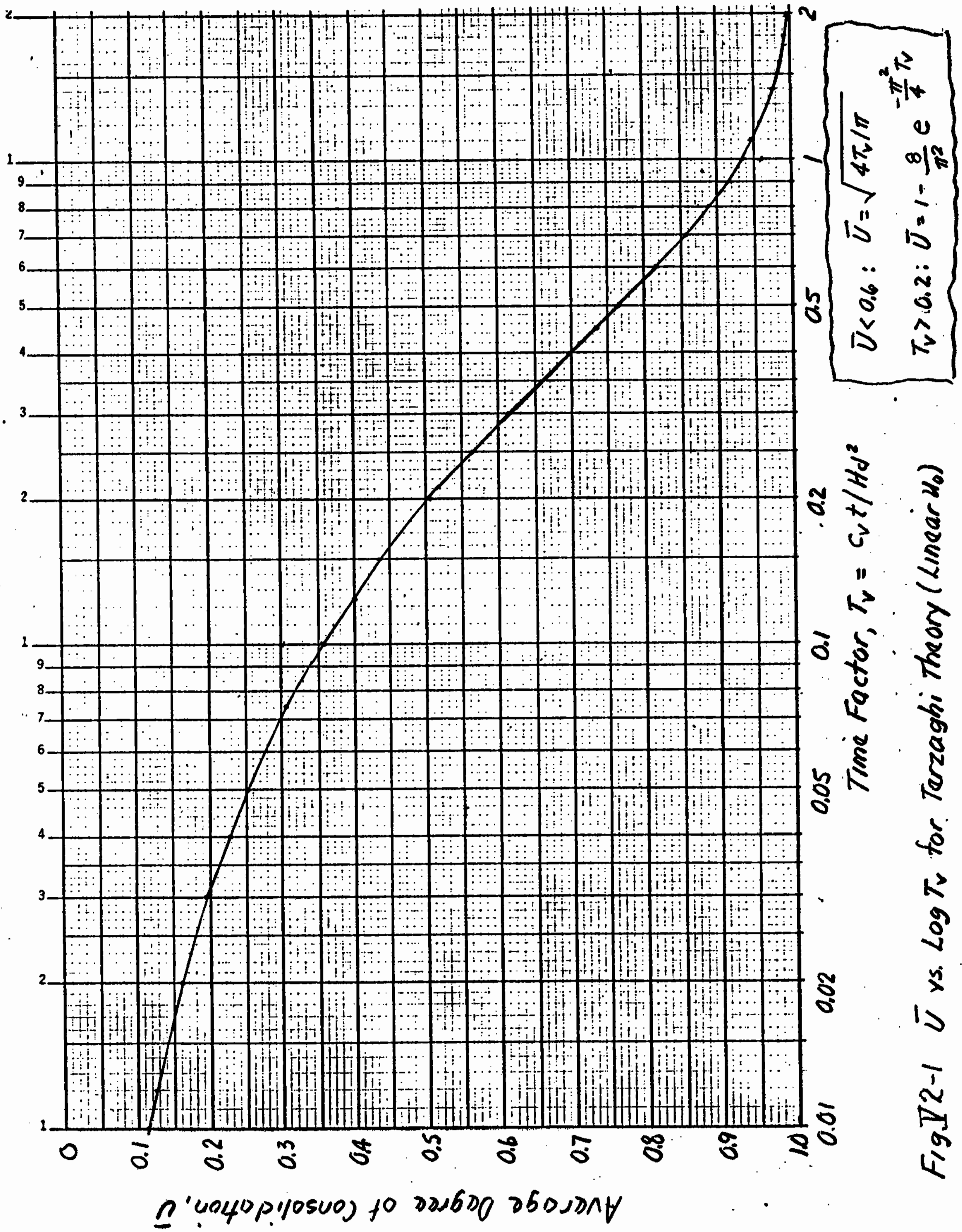


Fig V2-1 $\bar{U}$ vs. Log T_v for Terzaghi Theory (Linear u_0)

CCL 3/24/96

(2) Actual behavior for std. $t_c = 1$ day increments

Soil doesn't follow theory – $S < 100\%$

– $m_v \neq$ constant (esp. OC → NC) ↙ See Section 3

– creep effects

↓

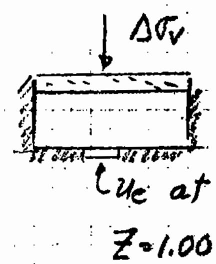
$c_v(\sqrt{t}) \approx (2 \pm 0.5)\, c_v(\log t)$

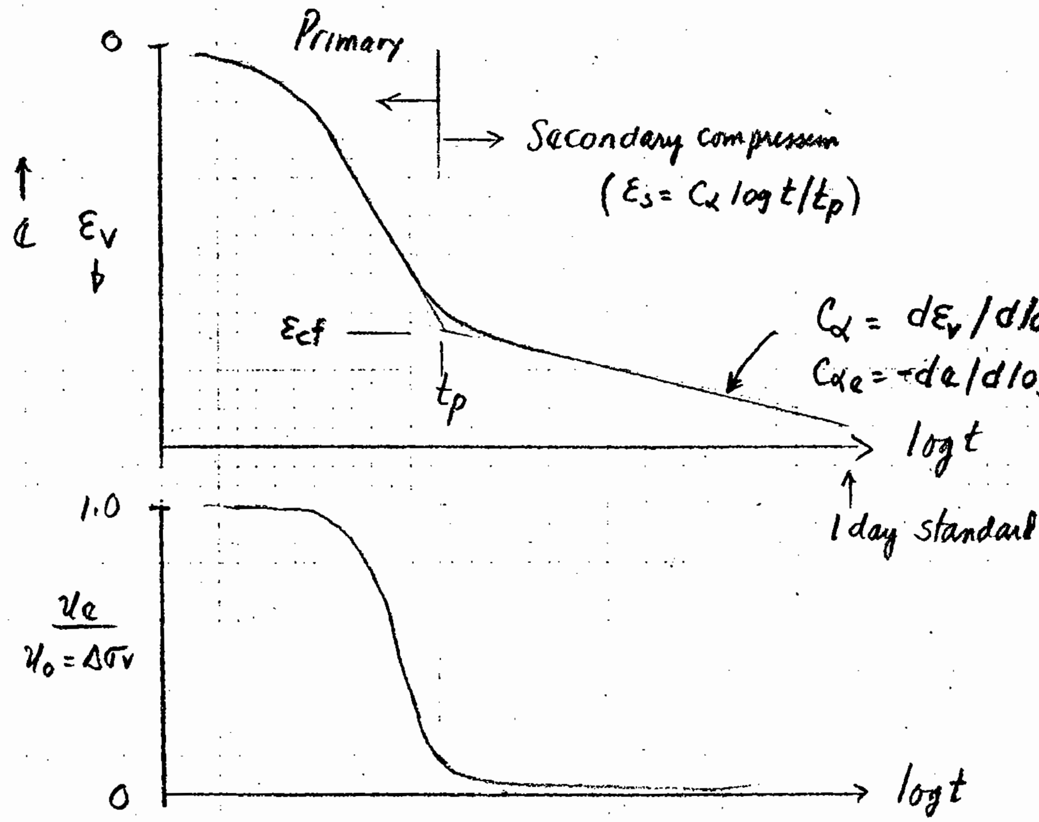
Canada for Sensitive clays $c_v = \dfrac{k_v}{m_v \cdot \gamma_w}$ Measure k_v & m_v

2. MISCELLANEOUS TOPICS

2.1 Primary Consolidation vs Secondary Compression

Test results sat. clay with $\Delta\sigma_v / \sigma'_{vc} \approx 1$, N.C.





Primary consolidation: rate of volume change (drainage) controlled mainly by rate of dissipation of excess pore pressures = hydrodynamic time lag. Terzaghi theory works well if $m_v \approx$ constant

Secondary compression = drained creep for 1-D loading. Mechanisms controversial (CCL = contact slippage, not viscous H_2O). Usually linear vs $\log t$ → constant rate of second. comp. C_α

- $\rho_s = H\,\varepsilon_s = H \cdot C_\alpha \log(t/t_p)$

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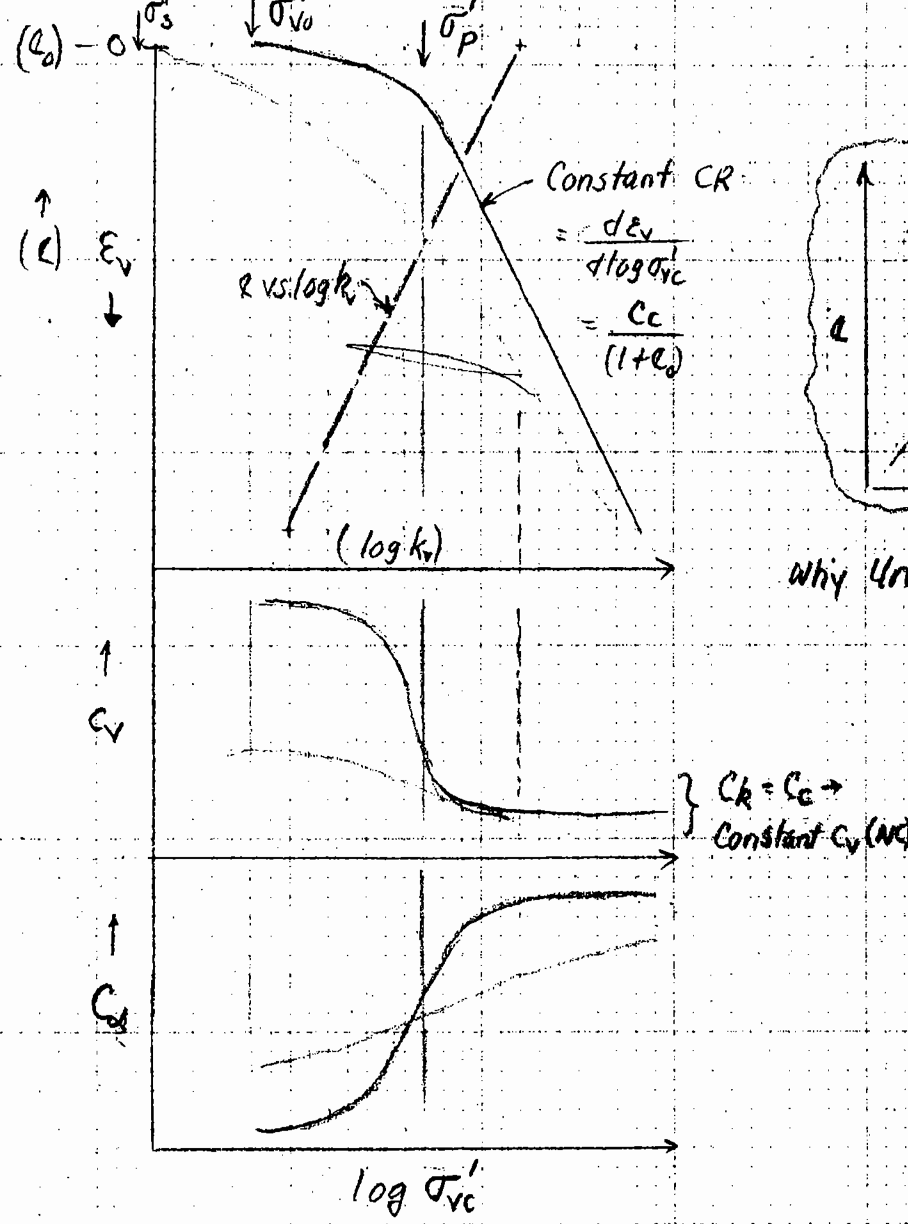
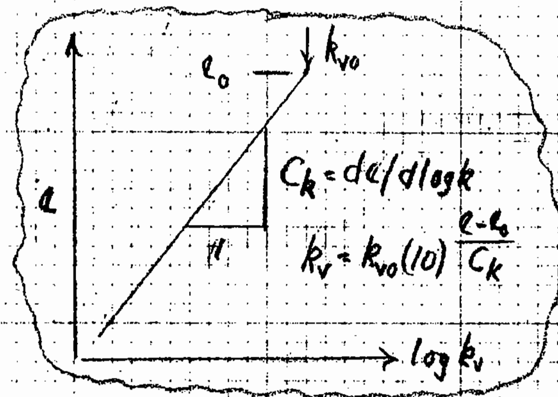
Part V-2 ps

2.2 Influence of Stress History & Sample Disturbance

—— In situ (Undisturbed)

- - - "Disturbed" Lab





Why Undisturbed $c_v(OC) \geq (5\text{-}10)\, c_v(NC)$?

$c_v = \dfrac{k_v}{m_v \cdot \gamma_w}$; $m_v \approx \dfrac{0.434\, C}{\sigma'_{v\,ave}}$

Important observation that C_α / C = constant for given clay (Section 3.1)

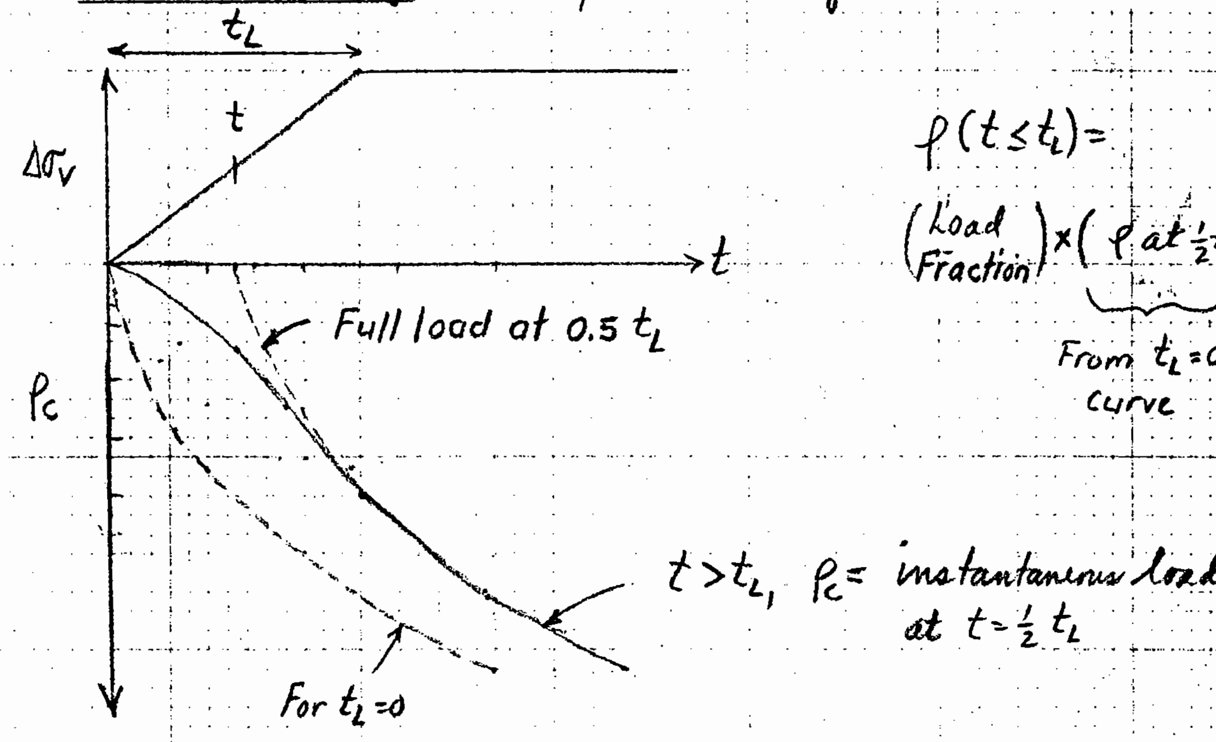
(0.045 ± 0.02)

$C = d\varepsilon_v / d\log\sigma'_{vc}$

Effects of disturbance (Above for excessive disturbance)

- Compressibility : Incr. RR (use U/R cycle); decr. CR, esp. CR_{max} for S-shaped VCL
- c_v : Decr. c_v(OC) due to ____________ (use U/R cycle)
- C_α : Follow C_α / C criterion
- σ'_p : May decrease, esp. S-shaped VCL

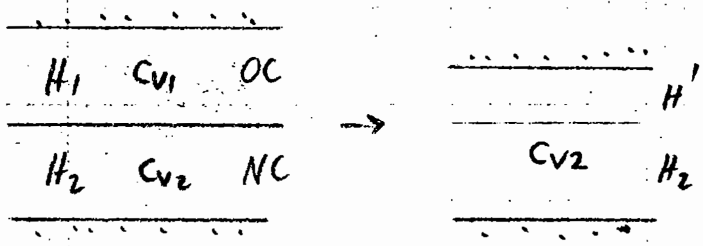
2.3 Rate of Loading (L&W p414 à la Taylor)



$\rho(t \le t_L) = \begin{pmatrix}\text{Load}\\ \text{Fraction}\end{pmatrix} \times (\underbrace{\rho \text{ at } \tfrac{1}{2}t}_{\text{From } t_L = 0 \text{ curve}})$

2.4 Layered System (Approximate, not theoretically correct)

For $c_{v1} > c_{v2}$



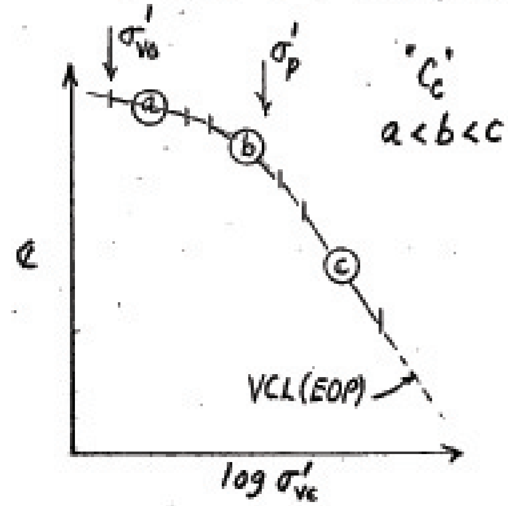
$H' = H_1 \sqrt{c_{v2}/c_{v1}}$

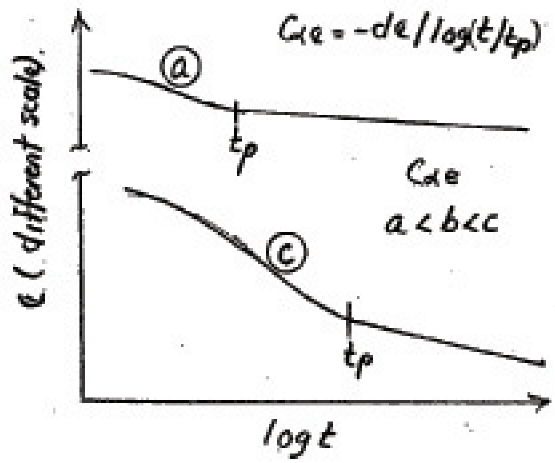
$\boxed{\text{Use reduced } H_d \text{ with lower } c_v}$

3. SECONDARY COMPRESSION (Drained 1-D creep at constant σ'_{vc})

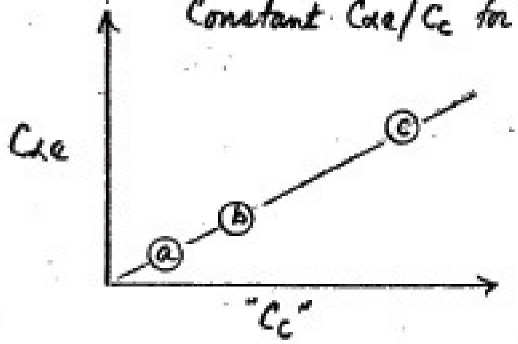
3.1 Rate of Secondary Compression vs. Slope of 1-D Compression Curve

1) Experimental correlations between $C_{\alpha e} = -de/d\log t$ and slope of EOP (end-of-primary) compression curve, "C_c" $= -de/d\log \sigma'_{vc}$





Constant $C_{\alpha e}/C_c$ for both recompression & virgin compression!



Hence drained creep is uniquely related to compressibility of soil during primary consolidation → same physical mechanisms (CCL opinion)

(See Sheet A for actual data)

2) Values of $C_{\alpha e}/C_c = C_{\alpha \varepsilon}/(d\varepsilon_v/d\log \sigma'_{vc})$

Material	C_α/C_e
Granular soils including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

Values of C_α/C_e for Geotechnical Materials

Adapted from Mesri et. al. (1994) ASCE, GSP No. 40. Also see Mesri et. al. (1984), Gest. 34(3) & (1987) ASCE, JGE, 113(3).

Most cohesive soils: NC/OC

$$\frac{C_{\alpha e}}{C_c} = \frac{C_{\alpha \varepsilon}}{CR} \approx 0.045 \pm 0.015$$

$\therefore C_\alpha(OC) \ll C_\alpha(NC)$

3.2 Some Practical Results of Secondary Compression

3.2.1 Increased settlement after end of primary consolidation

1) Basic equations for NC soil ($C_\alpha \equiv C_{\alpha\varepsilon}$)

- Primary consolidation settlement, $\rho_{cf} = H \cdot CR \cdot \log \sigma'_{vf}/\sigma'_p$
- Secondary compression settlement, $\rho_s = H \cdot C_\alpha \cdot \log t/t_p$

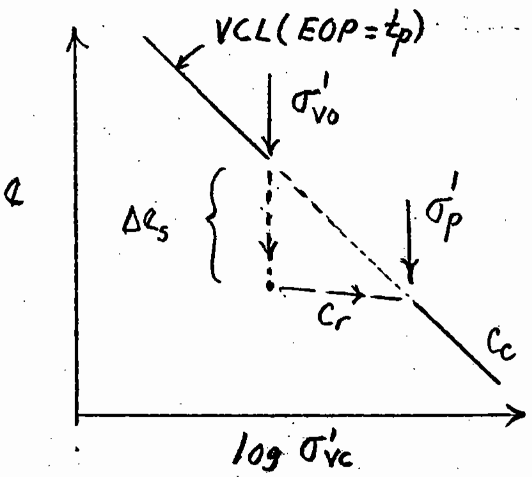
$$\therefore \frac{\rho_s}{\rho_{cf}} = \frac{C_\alpha \log t/t_p}{CR \log \sigma'_{vf}/\sigma'_p}$$

2) Examples for $C_\alpha/CR = 0.045$

σ'_{vf}/σ'_p	$\log \sigma'_{vf}/\sigma'_p$	$\log t/t_p$	ρ_s/ρ_{cf} (%)
2	0.30	1	15
	"	2	30
1.5	0.175	1	≈25
		2	≈50

Most important when have:
- Low σ'_{vf}/σ'_p
- Relatively small t_p
  - Small H_d
  - Installation of vertical drains

3.2.2 Aging of "normally consolidated" deposits (Increase in OCR)



Sec. compr + Recomp = Virgin compr.

- $C_{\alpha e} \log t/t_p + C_r \log \sigma'_p/\sigma'_{vo} = C_c \log \sigma'_p/\sigma'_{vo}$
- $\log \sigma'_p/\sigma'_{vo} = \log OCR = \dfrac{C_{\alpha e}}{C_c - C_r} \log t/t_p$

$$\therefore OCR = (t/t_p)^{\left[\frac{C_{\alpha e}}{C_c - C_r}\right]}$$

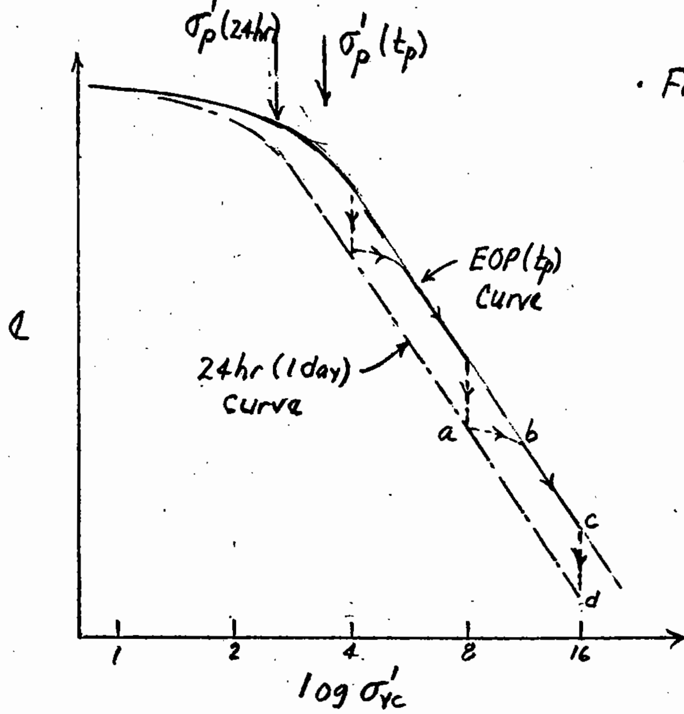
$$= (t/t_p)^{\left[\frac{C_{\alpha e}/C_c}{1 - C_r/C_c}\right]}$$

$$\uparrow = \frac{C_\alpha/CR}{1 - RR/CR}$$

For $C_\alpha/CR = 0.045$ & $RR/CR = 0.15$ (expondent $= 0.045/0.85 = 0.053$):

t/t_p	OCR
= 10 →	= 1.13
= 100	= 1.275
= 1000	= 1.44

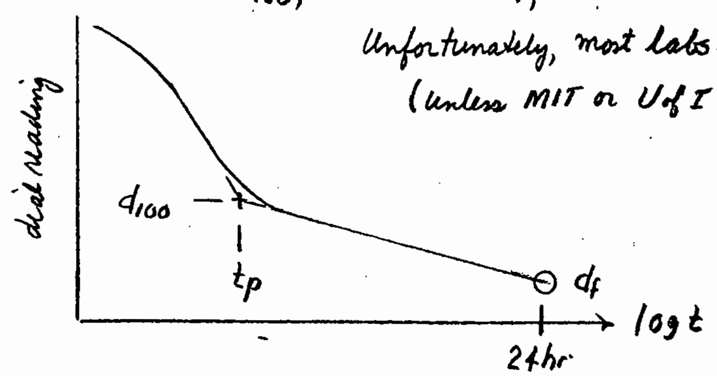
3.2.3 <u>Behavior during standard (24hr.) incremental oedometer tests</u>



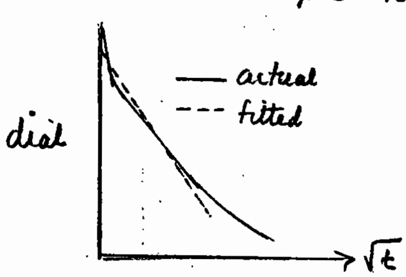
- For typical $H_d \le 1$ cm and $c_v \approx 10^{-3}$ cm²/s, get $t_p \le 15$ min $\rightarrow t_{1day}/t_p \gtrsim 100$
- Path during increment:
  - a-b = recompression
  - b-c = virgin compression
  - c-d = secondary compression

1) Use of 24hr (1day) curve, which is allowed by ASTM D2435-90, will $\rightarrow$ <u>lower estimated σ'_p than from EOP(t_p) curve.</u> Difference is typically about 15±5%. Therefore should use d_{100}, rather that d_f, in order to plot EOP curve.

Unfortunately, most labs plot 1day curve (unless MIT or U of I graduates)



2) Use of 24 hr. increments affects computed c_v, especially from $\sqrt{t}$ method.
   - Sample initially exhibits OC behavior with high $c_v \rightarrow$ curved dial vs. $\sqrt{t}$ plot $\rightarrow$ too steep fitted line.
   - Probably explains why $c_v(\sqrt{t}) > c_v(\log t)$



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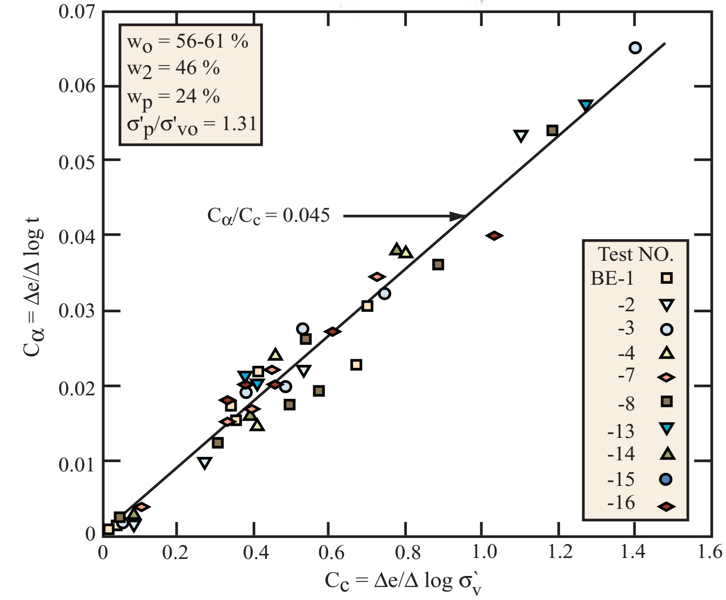
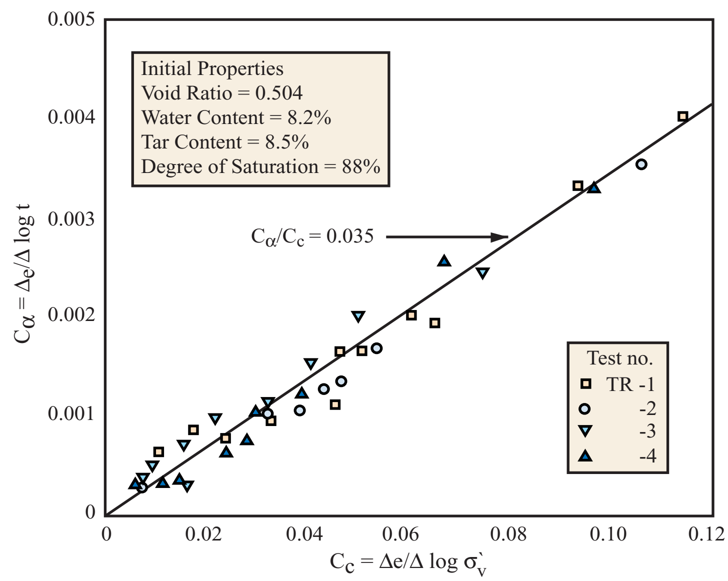
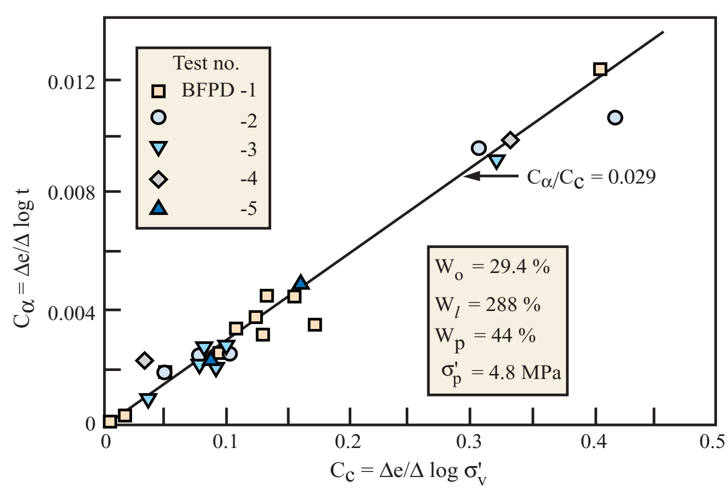


Canadian Quick Clay

$C_\alpha \equiv C_{\alpha e}$



Tar Sand



Shale

$C_{\alpha e}$ vs. C_c Relationships for three soils

Adapted from Mesri & Castro (1987), "C_α / C_c Concept and K_o During Secondary Compression". ASCE, JGE, 113(3), 230-247.