IV-4 STRESS-STRAIN-STRENGTH BEHAVIOR OF SATURATED CLAYS (for drained conditions)

Page No.

1. Introduction

2. Mechanisms of Volume Change & Shear Strain in Clays

3. Conditions Causing Settlement & Heave (1-D Case)

4. 1-D Stress-Strain Behavior

5. Estimating 1-D Settlement in Practice

6. Drained Shear Behavior of Clays

Sheets A1-A5: Preconsolidation Pressure Mechanisms & Illustration of Cemented-Structured-Sensitive Clay

CCL 10/27/83 10/84 10/96 10/99 1.361-1.366

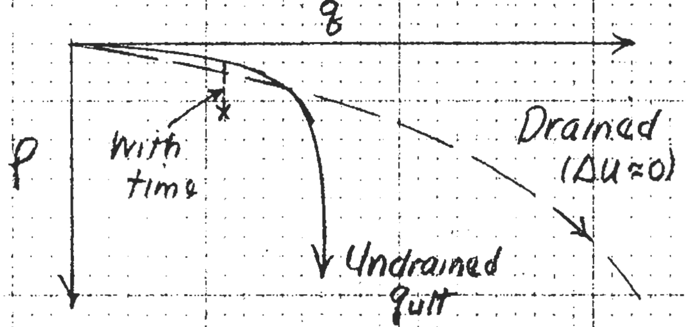
Part IV-4 STRESS-STRAIN-STRENGTH BEHAVIOR OF SATURATED CLAYS

1. INTRODUCTION

1.1 Comparison Clay vs Sand

	Sands	Clays
• Compressibility	Medium → v. low (< few cm)	Medium → v. high (up to ≥ 1m)
• Permeability	High → drainage during construction	V. Low → nearly undrained during construction
• σ-ε-Str. Behavior	Essentially same basic principles	

1.2 Footing on Clay Examples



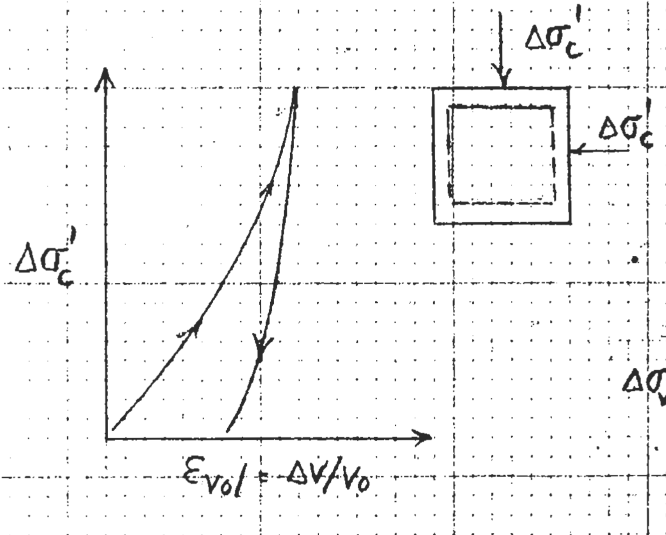
1.3 Coverage

- Mechanisms (wrt soil structure) → Δ volume & shear strain
- Drained σ-ε behavior
  - 1-D
  - ~~During shear~~

2. MECHANISMS OF VOLUME CHANGE & SHEAR STRAIN IN CLAYS

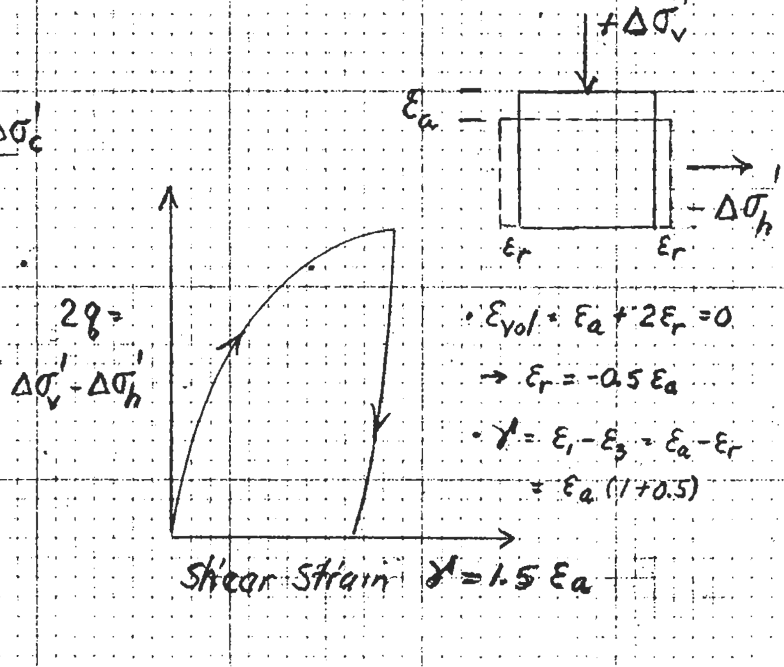
2.1 Volumetric & Shear Strain Behavior (Triaxial cell)

Hydrostatic (Isotropic) Compression



Both: Non-linear & inelastic

Shear So That ΔV=0



- $\varepsilon_{vol} = \varepsilon_a + 2\varepsilon_r = 0$
  $\rightarrow \varepsilon_r = -0.5\,\varepsilon_a$
- $\gamma = \varepsilon_1 - \varepsilon_3 = \varepsilon_a - \varepsilon_r$
  $= \varepsilon_a (1 + 0.5)$

CCL 10/27/83 10/88 10/96 10/27/98 1.361-1.366

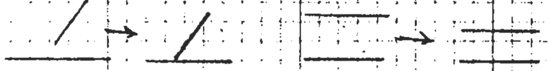
Part IV-4 CLAY BEHAVIOR

2.2 Mechanisms Causing Strain (Soil structure)

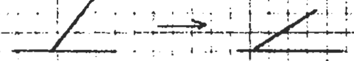
Volumetric Compression	(Swelling) Vol. Rebound	Shear Strain

1. Elastic deformation of particles, esp. bending

2. Changes in closest spacing

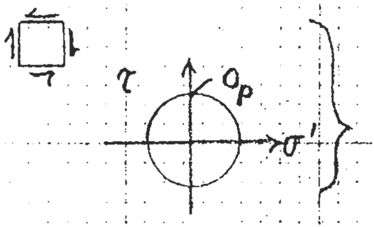


3. Sliding at contacts & reorientation

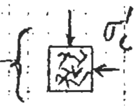


4. Particle crushing (only granular) { Esp. calcareous. Also all sands at very high stresses
   Floc "crushing"

Conclusions: (1) All mechanisms → volumetric strains can also → shear strains, but relative importance varies



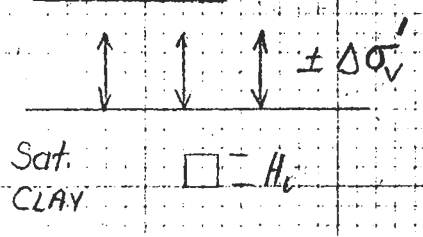
(2) Isotropic stresses → $\overline{\overline{\tau}}$ at contacts



(3) Shear at constant σ'_{oct} → volumetric strains

3. CONDITIONS CAUSING SETTLEMENT & HEAVE (1-D case)

3.1 Overview (S=100%)



**

$$\boxed{\cdot \text{ For 1-D case, any } \pm\Delta\sigma'_v \rightarrow \pm\Delta \text{ volume}}$$

$$\cdot\ \rho_{\pm} = \Sigma\left[H_i\left(\varepsilon_{vi} = \frac{\Delta e}{1+e_0} = m_v \cdot \underset{\overset{\uparrow}{\pm}}{\Delta\sigma'_v}\right)\right]$$

$$\cdot\ \Delta\sigma'_v = \Delta\sigma_v - \Delta u$$

(1) $\Delta\sigma_v$ due to change in overburden stress < Thickness { Deposition, Erosion; $\Delta\gamma_t$ - drying & wetting

(2) " " " " " applied loads (Man + nature) { Ice, waves, etc.

(3) Δu " " " " WT elevation

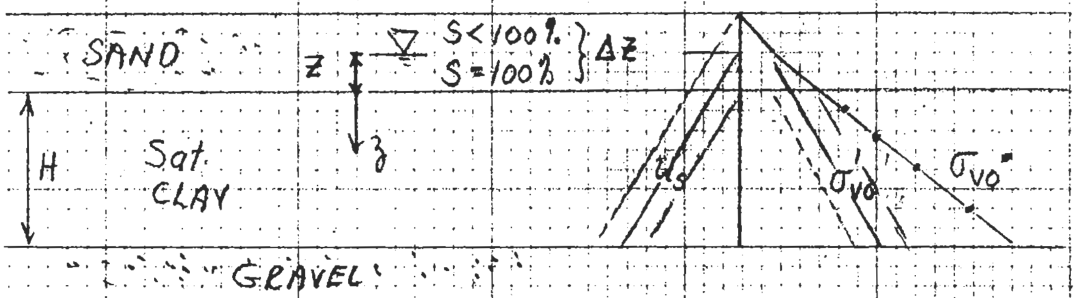
(4) Δu " " " " seepage conditions - pumping
- artesian

** Therefore, if $\Delta V = 0$, then must have $\Delta\sigma'_v = 0$ (for 1-D, saturated condition)

Part IV-4 CLAY BEHAVIOR

3.2 Examples of Changing Pore Pressures

(1) Initial condition (u_s = steady state u drawn for hydrostatic)



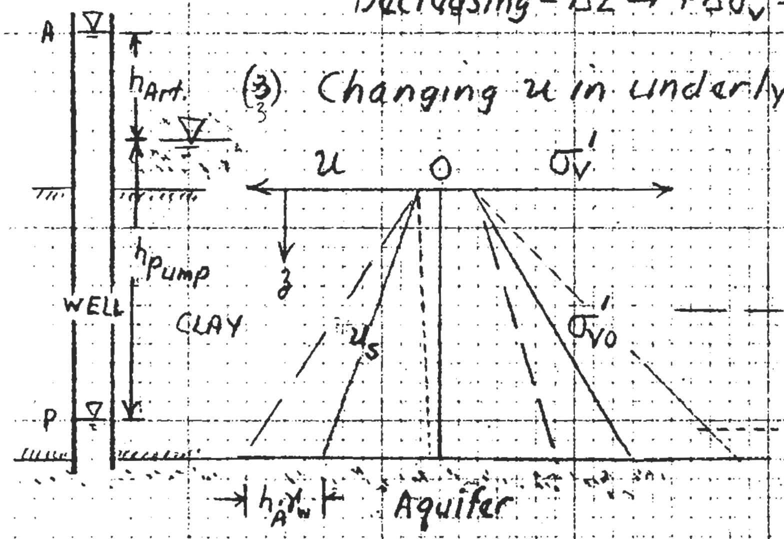
(2) Changing W.T. Elevation

$\Delta W.T. = \pm\Delta Z$ $\quad\Delta\sigma_v' = \overbrace{+\Delta Z\,\Delta\gamma_t}^{\Delta\sigma_v} - \overbrace{\Delta Z\,\gamma_w}^{\Delta u} = \Delta Z\,(\overbrace{\Delta\gamma_t-\gamma_w}^{\text{Always negative}})$

∴ Increasing $+\Delta Z \rightarrow -\Delta\sigma_v' \rightarrow$ heave, since $+\Delta u > +\Delta\sigma_v$

Decreasing $-\Delta Z \rightarrow +\Delta\sigma_v' \rightarrow$ settlement, since $-\Delta u < -\Delta\sigma_v$

(3) Changing u in underlying Gravel



—— = Reference hydrostatic condition

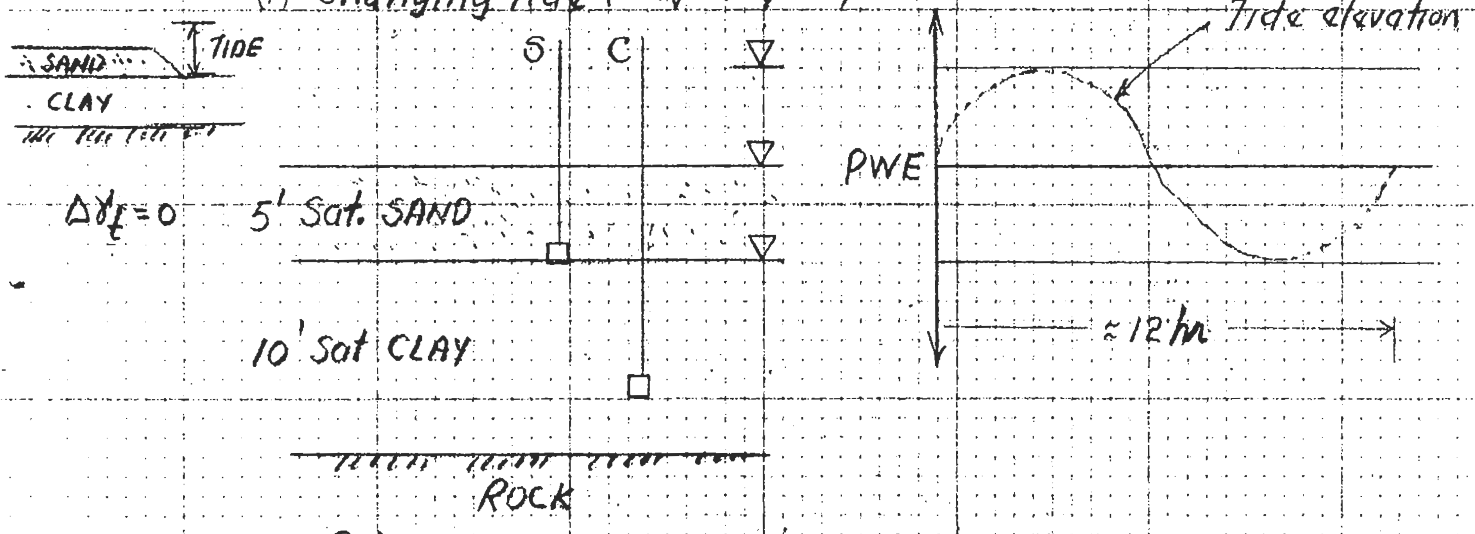
$$\Delta\sigma_v' \text{ vs } z = z\gamma_b \pm z\,i \left\{ i = \frac{h}{H}\gamma_w \right\}$$

— — Artesian condition →

- - - Pumping condition →

EXAMPLES
- Mexico City
- Bangkok
- Taipei
- Houston
- Long Beach, CA (oil)

(4) Changing Tide ($\Delta\sigma_v' = \Delta\sigma_v - \Delta u$)



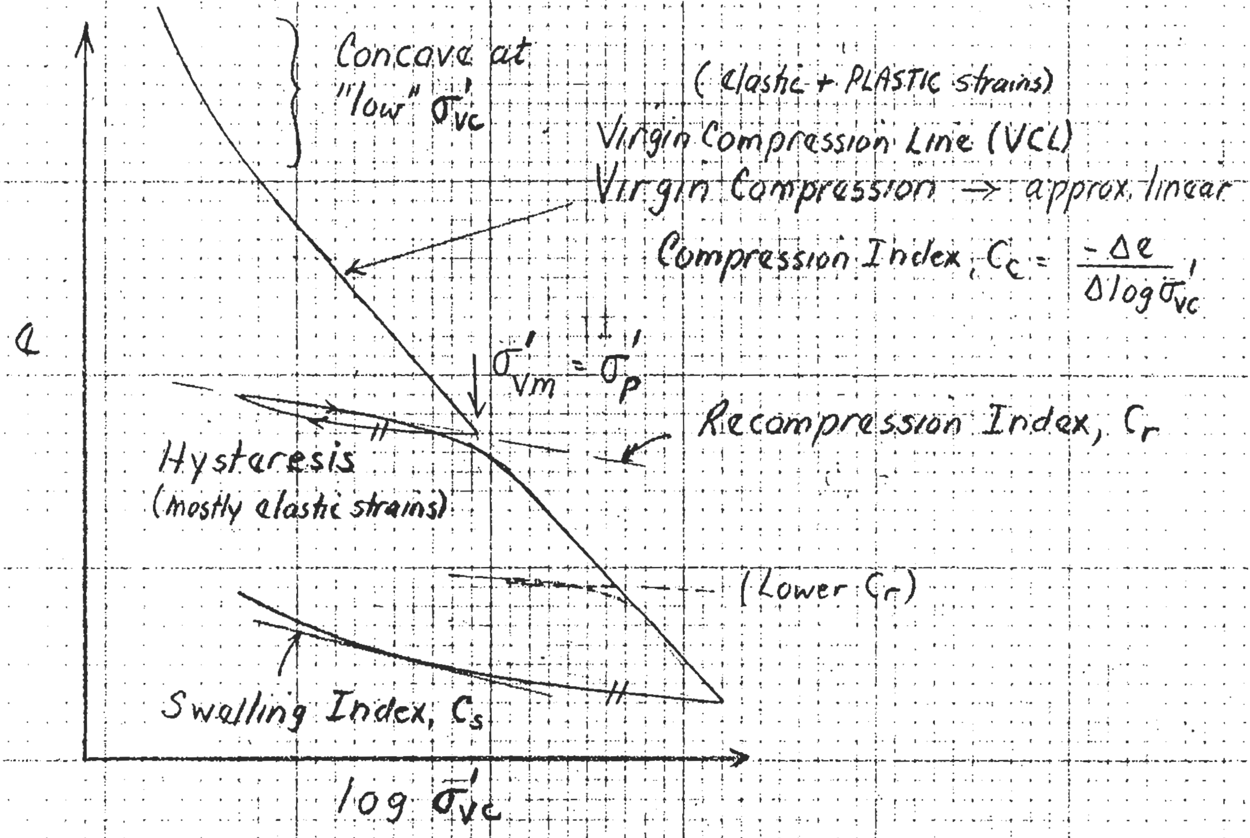
PWE = Piezometric Water Elevation = TOTAL HEAD = $h_p + h_e$

CCL 10/29/83 10/85 10/96 10/98 1.361-1.366

Part IV - 4 CLAY BEHAVIOR

4. 1-D STRESS-STRAIN BEHAVIOR

4.1 "Typical" Sedimentary Clays (With LINEAR VCL)



(1) $C_c \approx$ constant, except: V. low or V. high stresses
: "structured" → S-shaped VCL (e.g. see 4.2.3 D)

(2) Swelling (rebound) curves are approx. parallel, with C_s increasing with incr. $OCR = \sigma'_p / \sigma'_{vc}$

(3) Recompression Index approx. independent of σ'_p, but increases with incr. Δ OCR

(4) Compressibility OC << N.C with $C_r / C_c \leq \frac{1}{5}$ to $\frac{1}{10}$

NOTE: $\sigma'_{vm} \equiv \sigma'_p \equiv P_c$

- Max. past pressure
- Preconsolidation pressure
- Critical pressure

} Very poor terminology by founders of soil mechanics

10/97 10/98

4,2 Mechanisms Causing Preconsolidation Pressure

4.2.1 Physical Significance of σ'_p

Really YIELD STRESS (σ'_{vy}) for 1-D drained loading that separates elastic behavior (small, recoverable) from plastic behavior (includes large, irrecoverable strains)

4.2.2 Four Basic Mechanisms (Sheets A1-A4, especially A3)

A. Mechanical: $\Delta\sigma'_v = \Delta\sigma_v - \Delta u$ à la Section 3.1; constant $\sigma'_p - \sigma'_{vo}$ (for overburden erosion)

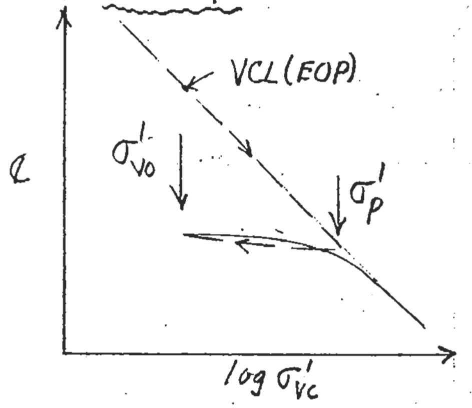
B. Desiccation: drying due to evaporation or freezing; erratic σ'_p

C. Aging = Secondary Compression = 1-D Drained Creep: ≈ constant σ'_p/σ'_{vo} (Covered in IV-2)

D. Physico-Chemical: e.g. natural cementation due to carbonates, Al/Fe oxides, etc.; erratic σ'_p

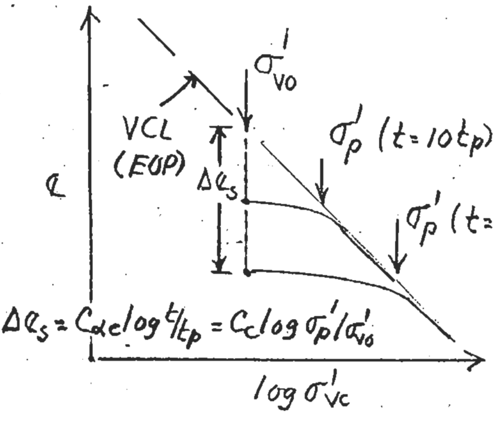
4.2.3 Illustration of Four Mechanisms (Note: EOP = end of primary consolidation)

Mach. A & B



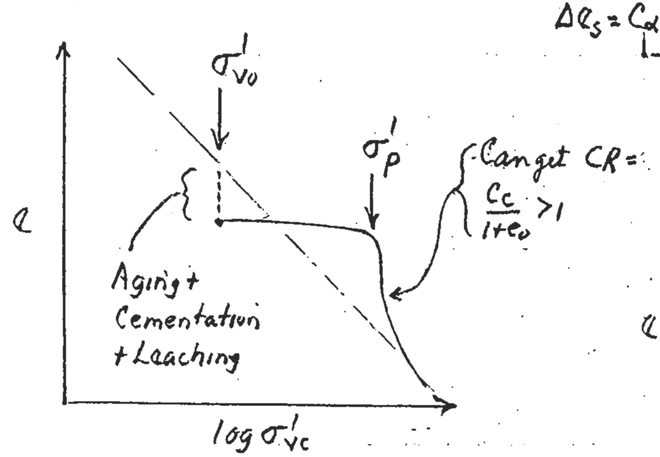
Mach. C.

- $\sigma'_p/\sigma'_{vo} = (t/t_p)^{C_{\alpha e}/C_c}$
- $C_{\alpha e} = de/\log(t/t_p)$

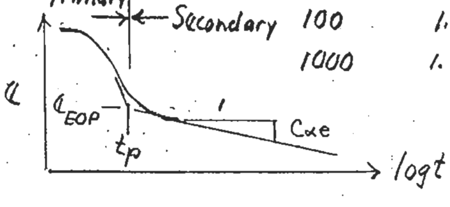


$$\Delta e_s = C_{\alpha e}\log t/t_p = C_c \log \sigma'_p/\sigma'_{vo}$$

Mach D (Discuss Sheets A4,5)



t/t_p	OCR for $C_{\alpha e}/C_c = 0.045$
10	1.1
100	1.2⁵
1000	1.3⁵

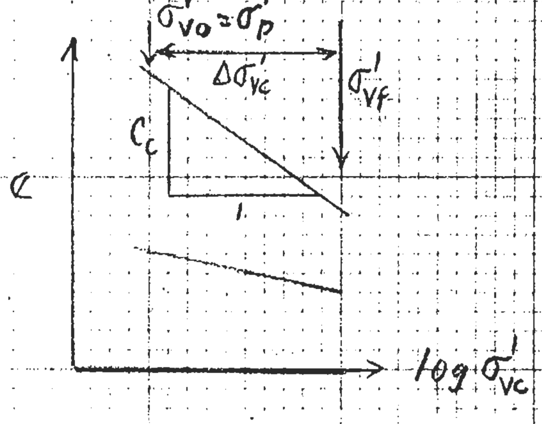


CCL 10/29/83 10/89 10/96 10/98 10/00 1.361-1.366

Part IV - 4 CLAY BEHAVIOR ( )

4.3 Virgin Compressibility

(1) Empirical correlations (Virgin compression)



T & P (1967): $C_c \simeq 0.009\,(w_L - 10\%)$

Nishida (1965) $C_c \approx 0.54\,(e_0 - 0.35)$

w_N	e_0	C_c	CR
150	4.2	2.1	0.40
100	2.8	1.3^{2}	0.35
50	1.4	0.57	0.24
30	0.84	0.26^{5}	0.14^{5}
	Range =	× 7.9	× 2.8

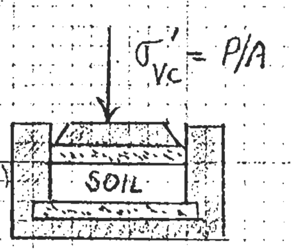
(2) Void ratio vs strain

- $\varepsilon_v = \frac{\Delta e}{1+e_0} = m_v\,\Delta\sigma'_{vc} = \overbrace{\frac{C_c}{1+e_0}}^{CR} \log \frac{\sigma'_{vf}}{\sigma'_{vo}}$
- Typical sedimentary clays with $LI = I_L < 1$ (i.e. low-moderate $S_t = \frac{s_u(U)}{s_u(R)}$)

  $CR \approx 0.25 \pm 0.1$
- $e - \log \sigma'_{vc}$ better for research

(3) "Structured" clays (see p6a)

4.4 Role of Oedometer Test



$H = 0.75 - 1.0"$

$D = 2.5 - 2.75"$

ASTM D 2435

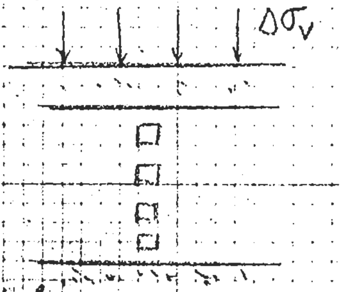
(1) General

- Representative "undisturbed" samples vs depth (use mudded holes for deep low OCR clay) ?!

Excessive stress relief can → failure of clay at bottom of boring in extension BEFORE tube sampling

- Oedometer cell - careful trimming

  $H \approx 2$ cm $D \approx 6-7$ cm (compatible with 3" φ Shelby tube)



- Initial seating (≈ 0.1 atm), then double loads ($LIR = \Delta P/P = 1.0$)

  (Canadian sensitive clays $LIR = 0.5$ to better define S-shaped curves
- Std. practice = 1 day / increment. ~~Hence many~~ engineers plot e or ε at 24hr, whereas they should plot EOP curves → higher (more accurate) values of σ'_p.

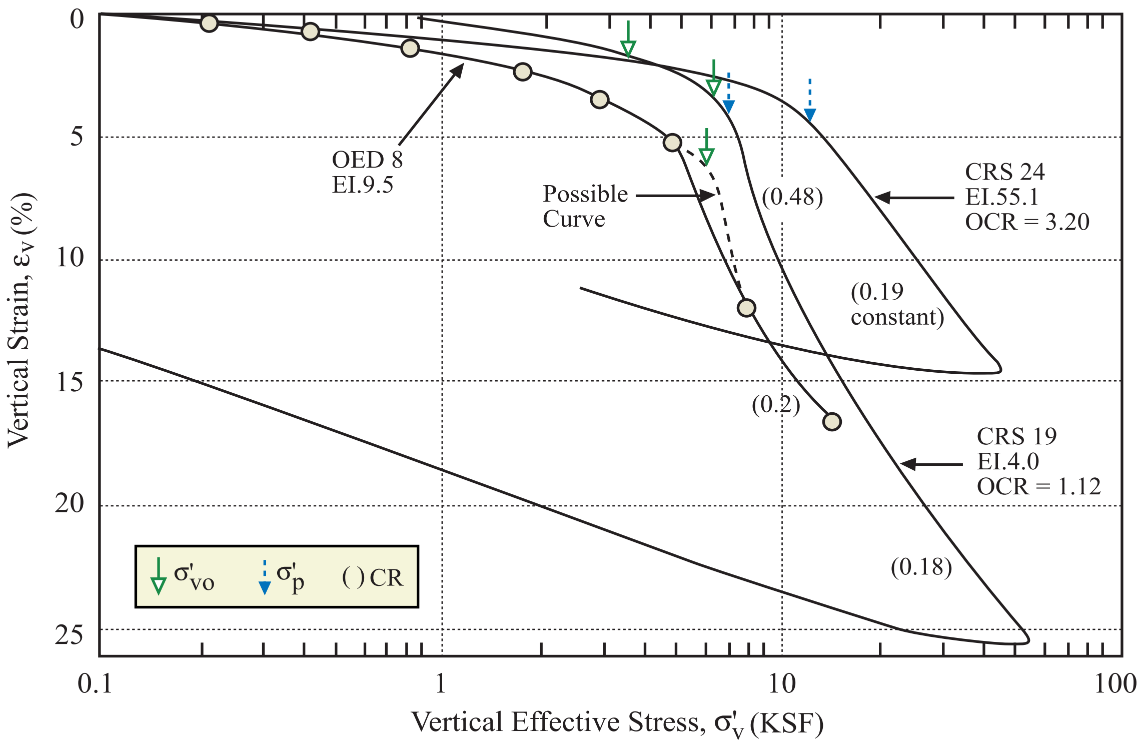
CRS = Constant Rate of Strain

4.3 (3) Virgin Compressibility of "structured" clays

a) Data on natural BBC (Fig 4 below from Ladd et al. (1998) Boston Geo Congress

- CRS 24 on clay from the desiccated crust → linear VCL with CR = 0.19. For this type of behavior, incremental oedometer and CRSC tests → same curves & hence σ'_p & CR.
- CRS 19 on "soft" clay below the crust → distinctly S-shaped VCL ∴ sharp break in curve at σ'_p & large decrease in CR
- Oed 8 on similar clay cannot define highly S-shaped curves (especially with LIR ≈ 1) → significant errors in σ'_p & CR_{max}

b) Experience at MIT since ~~early~~ 1990s with improved sampling & testing techniques (+ radiography) indicate that many (if not most) soft clays exhibit S-shaped curves to varying degrees. Hence profession should switch from increm. oed to CRS testing for more reliable estimates of σ'_p & CR.

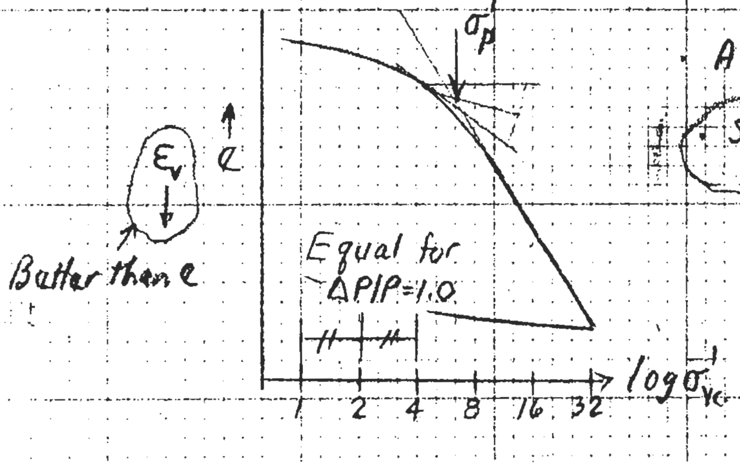


Typical I-D Compression Curves for Blue Clay at SB Site

Note: See section 3.1 of part IV - 3 for description of CRS consolidation test

4.4 *(Continued)*

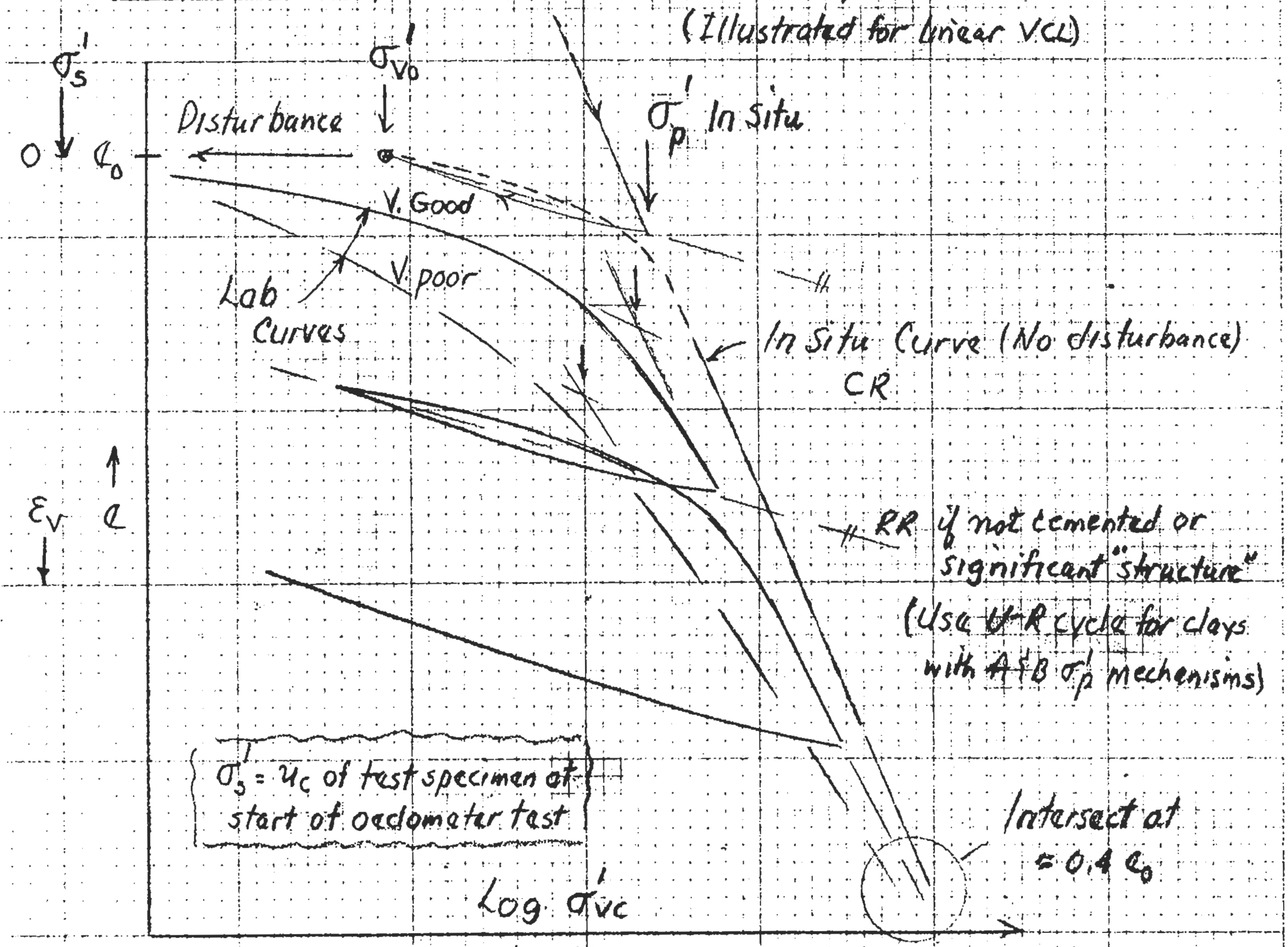
(2) Arthur Casagrande (AC) construction to obtain estimate of σ_p' (Fig. 20.6, L&W)



- AC ≠ Empirical procedure, but quick & simple
- See p7a for STRAIN ENERGY technique that CCL recommends
- Carry lab $\sigma_{vc}' \geq 4\text{-}8 \times \sigma_p'$ to define virgin compression line (VCL)
- Reduce LIR near est. σ_p' to better define curve (esp. if S-shaped)

4.5 Effect of Sample Disturbance on 1-D Compressibility

(Illustrated for linear VCL)



Disturbance → (1) Much higher recompression & increased ε_v at σ_{v0}' (Want < 3±1%)

(2) Probably lower σ_p' (can be much too low)

(3) Reduced CR_{max}, esp. S-shaped VCL''

10/96 10/98

Becker, Crooks, Been & Jefferies (1987) Canadian Geot. J., Vol. 24, No. 4, p 549-564

A. Definition of Strain Energy (SE) ≡ Work Per Unit Volume for 1-D Consolidation Tests

$$SE = \int \sigma_v' \, d\varepsilon_n = \Sigma \left(\text{Ave. } \sigma_v' \times \Delta\varepsilon_n \right) \text{ for each increment,}$$

where ε_n = NATURAL STRAIN = $\Delta H/H = \Delta e/(1+e)$. Plot SE vs. σ_v' at end of increment

B. Basic Assumptions and Application of SE Technique

1) SE vs. σ_v' should be linear up to $\sigma_v' = \sigma_{vo}'$. Use these data to define initial slope = S_1

2) SE vs. σ_v' will be essentially linear for virgin compression line (VCL) having constant C_c or CR ← If based on initial portion of VCL having $\Delta\varepsilon_v <\approx 10\%$ (Needs checking)
Use this slope to define VCL slope = S_2. Note: If S-shaped VCL, use S_2 = max. slope.

3) Preconsolidation pressure (σ_p') = intersection of slopes S_1 & S_2

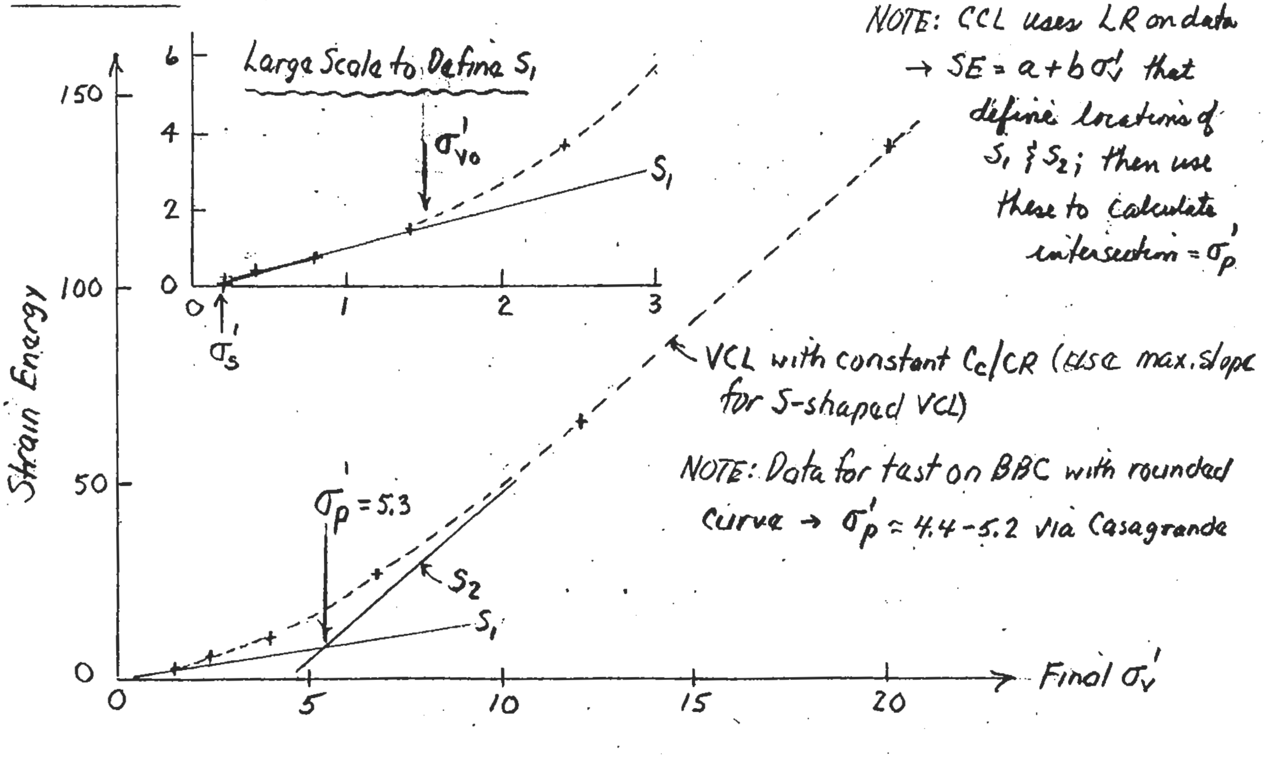
C. Comments

1) Application often requires LIR<1 up to σ_{vo}' to properly define S_1. If test does not give linear SE vs. σ_v' up to σ_{vo}' (as often happens), then select reasonable range for S_1.

2) For "structured" clays with S-shaped VCL, also need LIR<1 (best to have continuous loading à la CRSC test) to define S_2.

3) Method is especially usefully for heavily OC clays/silts with rounded compression curves causing large uncertainty in σ_p' using Casagrande method

D. Illustration



NOTE: CCL uses LR on data → SE = a + bσ_v' that define locations of S_1 & S_2; then use these to calculate intersection = σ_p'

NOTE: Data for test on BBC with rounded curve → $\sigma_p' \approx 4.4-5.2$ via Casagrande

Part IV-4 CLAY BEHAVIOR

5. ESTIMATING 1-D SETTLEMENT IN PRACTICE

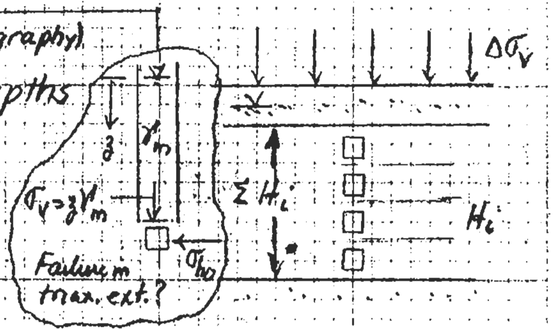
5.1 Steps

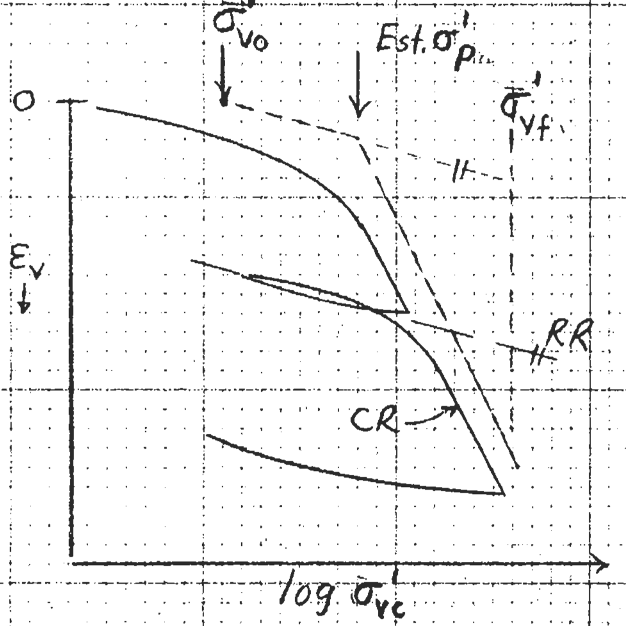
(1) Undisturbed sampling (with fixed piston / drilling mud + radiography) representative depths

(2) Run oedometer tests with U-R cycle
(always with set of Atterberg limits)

NOTE: Need small LIR near σ'_p or continuous loading for clays with S-shaped VCL

(3) Interpretion results





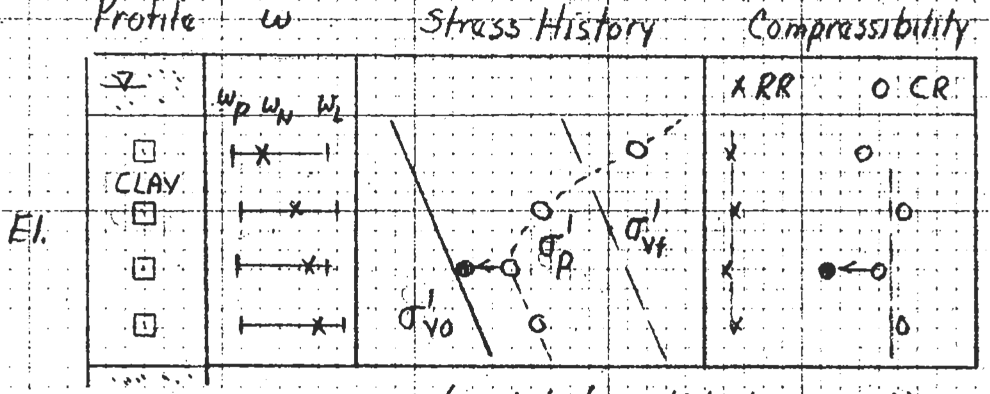
- Casagrande & Strain Energy → σ'_p
- RR → recompression
- CR → virgin (check vs empirical)

MUST KNOW

$$\boxed{\begin{aligned} \varepsilon_{cf} &= RR \log(\sigma'_p/\sigma'_{vo}) && \text{Recompression} \\ &+ CR \log(\sigma'_{vf}/\sigma'_p) && \text{Virgin Compression} \end{aligned}}$$

DON'T USE $\varepsilon - \log \sigma'_{vc}$ DIRECTLY
(due to disturbance effects, etc)

(4) Summary → design parameters $\rho_{cf} = \Sigma (H_i \cdot \varepsilon_{cf})$



- Use geology to help define possible mechanism & hence likely σ'_p trend
- Collective evaluation of RR & CR data → design values

(● data from disturbed sample)

5.2 Comments

(1) Stress history most important since $RR \doteq (\frac{1}{5} - \frac{1}{10}) CR$

(2)

SAND	ρ during $\Delta\sigma_v$	In Situ	D_r (+SH)	Vibration
vs				
CLAY	ρ after $\Delta\sigma_v$	Lab	OCR	Static
	Rate	Parameters from	Compressibility	To densify soil

Part IV-4 CLAY BEHAVIOR

6. DRAINED SHEAR BEHAVIOR OF CLAYS

6.1 Introduction

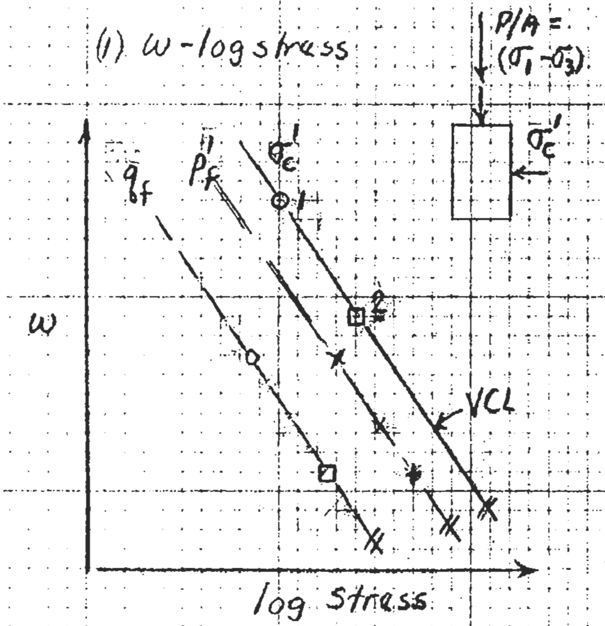
(1) Coverage
- Normalized σ-ε-strength = f(ESP & OCR)
- Unique $w_f - q_f - \bar{p}'_f$ for NC or OC w/ same mpp = σ'_{cm} ↑ (Used since not 1-D σ'_p)

(2) Objectives
- Expected trends + interpolate & extrapolate
- Later tie in with undrained shear

6.2 Normally Consolidated Clay: Std CIDC(L).

Tests at $\sigma'_c = 1$ & 2 ○ □ × +

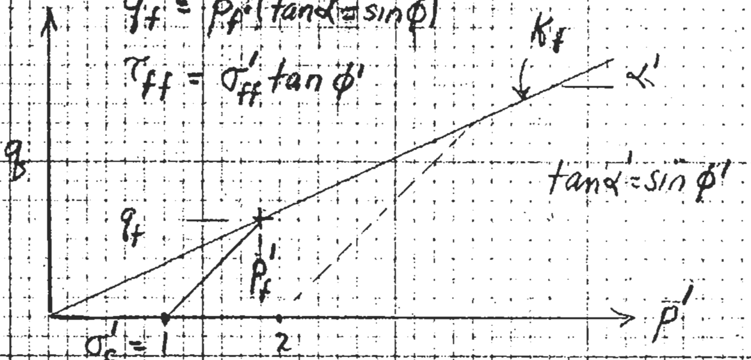
(1) w-log stress



(2) ESP & Failure Envelope

$q_f = \bar{p}'_f (\tan\alpha' = \sin\phi')$

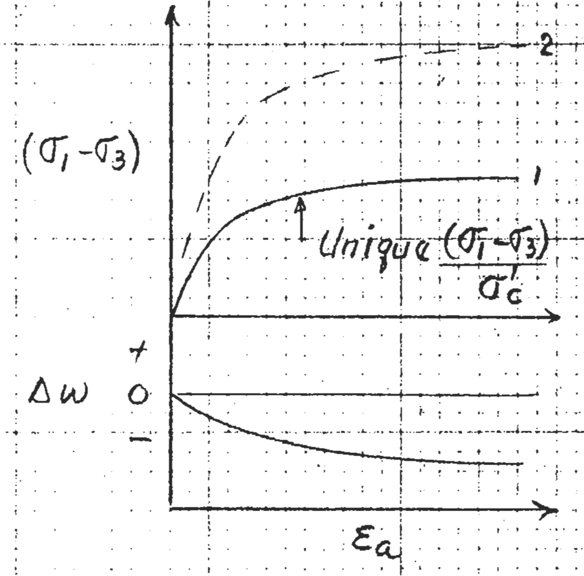
$\tau_{ff} = \sigma'_{ff} \tan\phi'$



$$\bar{p}'_f = \sigma'_c + q_f = \sigma'_c + \bar{p}'_f \sin\phi' = \frac{\sigma'_c}{1-\sin\phi'} \approx (1\tfrac{2}{3}-2)\sigma'_c$$

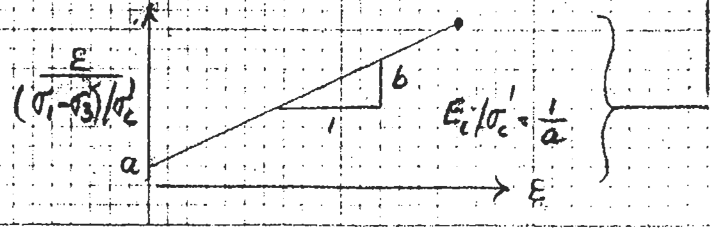
$$\boxed{\frac{q_f}{\sigma'_c} = \frac{\sin\phi'}{1-\sin\phi'}} \approx \frac{2}{3} - 1 \qquad \underbrace{(\sin\phi' \approx 0.4-0.5)}_{\phi' = 23.5-30^\circ}$$

(3) Stress-strain



- Large $\varepsilon_f \approx 20\%$
- Hyperbolic $\frac{(\sigma_1-\sigma_3)}{\sigma'_c} = \frac{\varepsilon}{a+b\varepsilon}$

- Normalized behavior → unique:
  1) ε_a vs $(\sigma_1-\sigma_3)/\sigma'_c$
  2) ε_a vs Δw



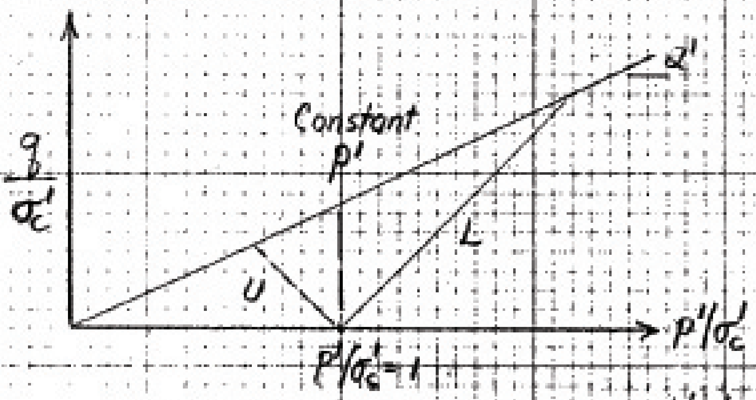
$E_i/\sigma'_c = \frac{1}{a}$

CCL 11/1/83 10/87 10/96 10/27/98 1.361–1.366

Part IV-1 CLAY BEHAVIOR

4.3 N.C. Clay: Other CIDC Stress Paths

(1) ESP: Normalized

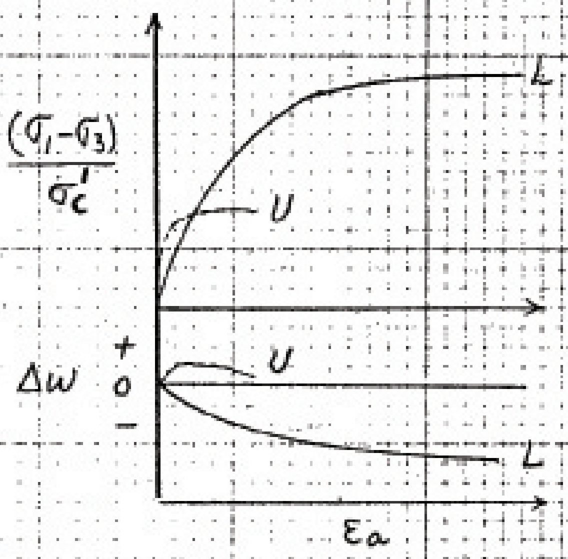


- Unique $q_f = p'_f \sin\phi'$ independent of ESP
- For Unloading

$$\boxed{\frac{q_f}{\sigma'_c} = \frac{\sin\phi'}{1+\sin\phi'}} \approx 0.29 - 0.33$$

$(\sin\phi' = 0.4 - 0.5)$

(2) Normalized stress-strain



- Unloading → lower q_f/σ'_c
  - → lower ε_f
  - → higher E
  - → $+\Delta w$

- Why different Δw behavior?

Loading: $+\Delta p' \rightarrow$

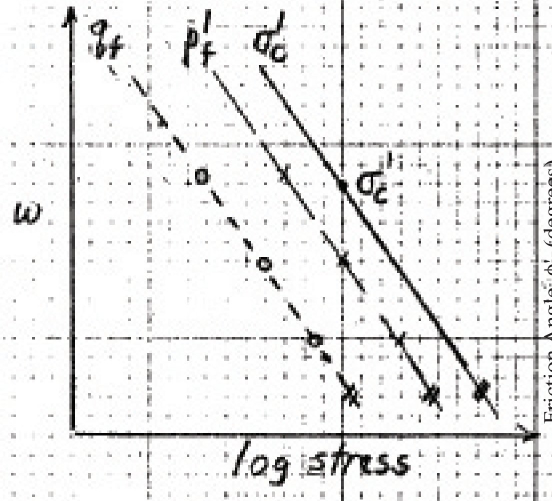
$+\Delta q \rightarrow$ *

Unloading $-\Delta p' \rightarrow$

$+\Delta q \rightarrow$ *

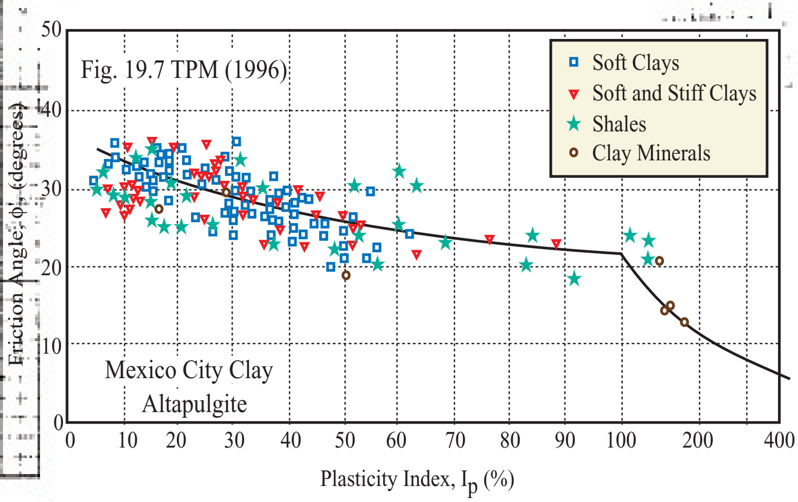
* Shear of NC Clay → volume decrease (more parallel particle orientation)

(3) $w - q - p'$



✱ Unique $w_f - q_f - p'_f$

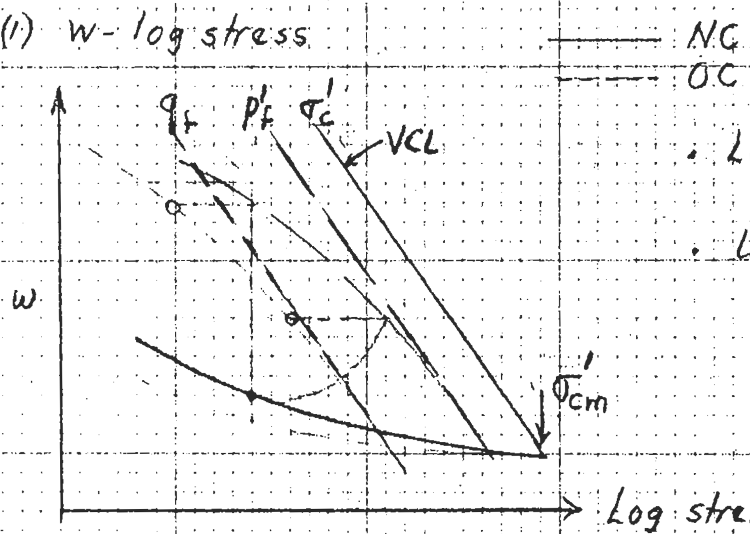
- Show L, U & $\Delta p' = 0$ w vs log p'



K·E ALBANENE GUIDE LINE 10 5841
10 X 10 TO THE INCH

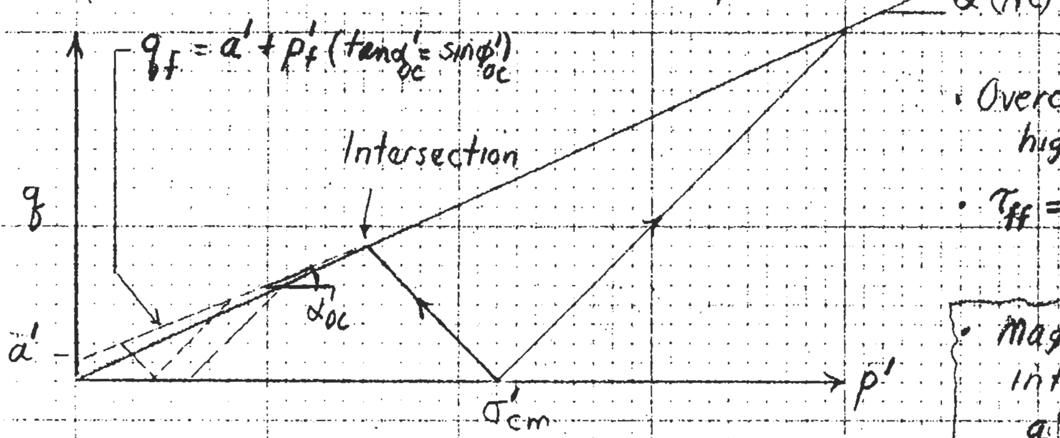
6.4 Overconsolidated Clay (Compared to OCR=1 for CIDC tests)

(1) w-log stress



- Location $w_f - q_f - p'_f$ for O.C.
- Unique $w_f - q_f - p'_f$ independant of ESP for same σ'_{cm}

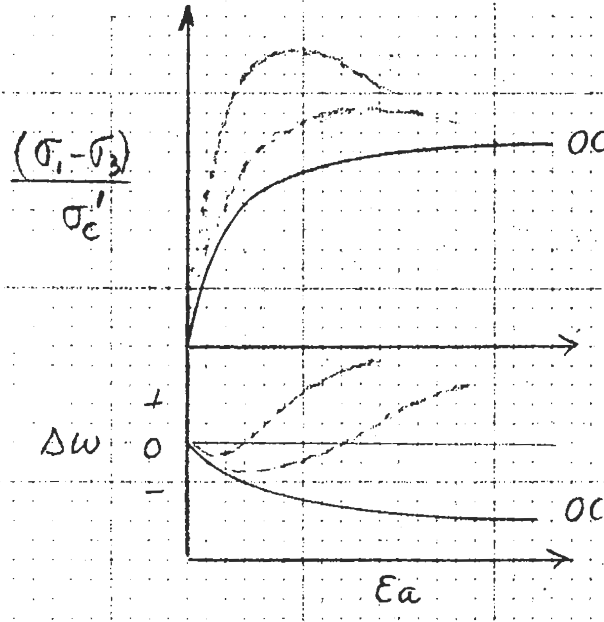
(2) ESP & ESE (Effective Stress Envelope)



- Overconsolidation → higher envelope
- $\tau_{ff} = c' + \sigma'_{ff} \tan\phi'_{oc}$

  $\uparrow = a'/\cos\phi'_{oc}$ (See p12)
- Magnitude cohesion intercept ∝ mpp → $a'/\sigma'_{cm} \approx$ constant
- $\phi'_{oc} < \phi'_{NC}$

(3) Normalized stress-strain CIDC(L)



With increasing OCR:

- Modest incr. $(\sigma_1-\sigma_3)/\sigma'_c$
- Signif. decr. ε_f
- Contractive → dilatant
- Like decreasing ψ for sands

- N.C. Clay ≈ VERY loose sand
- Heavily OC Clay ≈ dense sand (OCR > 10)

6.5 Summary

(1) Variables (restricted to CIDC testing)

- σ'_c
- σ'_{cm}

$\}$ $OCR = \sigma'_{cm}/\sigma'_c$

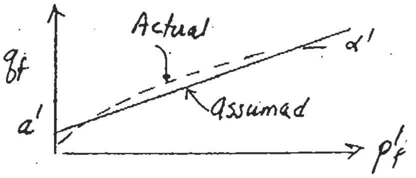
- ESP, i.e., L vs. U vs. constant p'

(2) Unique Failure Envelope

- NC $q_f = p'_f \tan\alpha'_{NC} = p'_f \sin\phi'_{NC}$
- OC (Same σ'_{cm}), $q_f = a' + p'_f \tan\alpha'_{OC} = a' + p'_f \sin\phi'_{OC}$; $a'/\sigma'_{cm} = \text{constant}$

$\}$ Independent of σ'_c & ESP

NOTE: Linear OC ESE is a simplification (See Part II-1, Sheet D2, Fig. 3)



(3) Unique $w_f - q_f - p'_f$ Relationship

- NC Relationship is parallel to VCL
- OC Relationship depends on σ'_{cm} and shifts to left with increasing OCR

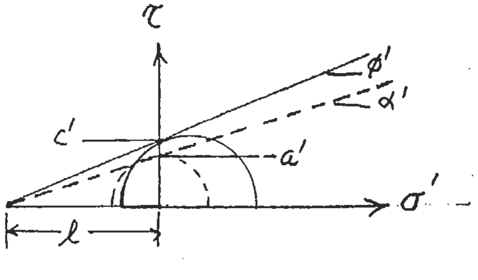
$\}$ Independent of σ'_c and ESP

(4) Normalized stress-strain

- q/σ'_c & ΔW vs ε_a vary with ESP & OCR, but independent of σ'_c (for given ESP & OCR)

(5) Which has higher q_f: NC or OC clay? (Illustrate for $p'_f = \sigma'_c$ tests)

- For same σ'_c and ESP →
- For same w_f →



- $\tan\alpha' = \sin\phi' = a'/l \rightarrow l = a'/\sin\phi'$
- $\tan\phi' = c'/l \rightarrow l = c'/\tan\phi' = c'\cos\phi'/\sin\phi'$

$$\therefore l = \frac{a'}{\sin\phi'} = \frac{c'\cos\phi'}{\sin\phi'} \rightarrow \boxed{c' = \frac{a'}{\cos\phi'}}$$

CCL 10/87
1.361 & 1.366
10/96 10/27/98

1.361-1.366 Part IV-4

Preconsolidation Pressure Mechanisms *

The mechanism(s) responsible for causing the observed preconsolidation pressure of horizontal clay deposits can have several practical implications, as summarized in Table 2-1.— (A3) Mechanical one-dimensional loading-unloading typically leads to a uniform amount of precompression (constant $\sigma'_p - \sigma'_{vo}$) and K_o conditions, although K_o at a given OCR depends on whether or not σ'_{vo} has been increased or decreased to its present value. Desiccation due to drying, freeze-thaw cycles, etc. will usually produce scattered, often difficult to define values of σ'_p and the in situ stresses may deviate from K_o conditions, e.g. isotropic stresses from evaporative drying. The generation of a preconsolidation pressure due to aging, defined as long term one-dimensional drained creep (= secondary compression), is certainly well documented in the laboratory (Leonards and Altschaeffl, 1964) and is supported by case histories (Bjerrum, 1967). It should result in a constant OCR, but whether or not K_o remains constant during secondary compression is in dispute (~~see Section 2.5~~) Probably increases à la Mesri & Castro (1987). Finally, it is now generally accepted that various physio-chemical phenomena can cause an increase in σ'_p, particularly natural cementation due to carbonates, silica, ion exchange, etc. The resultant σ'_p profile is likely to be (A4) variable, as illustrated in Fig. 2-3 for a deposit of James Bay marine clay. Although the in situ K_o may remain constant during development of σ'_p, the yield stress for horizontal loading would presumably increase due to cementation. It should be noted that cementation can be significant in deposits ranging from heavily overconsolidated clay shales (McGowen and Ladd, 1982) to the brittle quick clays of Canada.

(A3)

The fact that the various mechanisms described in Table 2-1 can lead to substantially different σ'_p profiles greatly complicates interpretation of scattered σ'_p data from laboratory tests. That is, it is often difficult to differentiate between scatter due to the effects of sample disturbance on the

* From Ladd, C.C. (1985) "Overview of Clay Behavior" MIT Special Summer Course 1.60S

(A1)

1.361-1.366 Part IV-4

CCL 10/96
10/98

"measured" σ'_p and that due to true spatial variability. When faced with significant scatter (and if the deposit doesn't have shells, sand lenses, etc.), it is often very helpful to analyze results from in situ tests such as the field vane, Dutch cone or piezo-cone, since variations in the derived undrained shear strength (c_u) or penetration pore pressure should reflect similar changes in σ'_p. (~~Also see Professor Baecher's lecture on using statistical techniques to separate "noise" from spatial variability~~).

2.3 ONE-DIMENSIONAL COMPRESSION

2.3.1 Idealized Behavior

See figure on p4 of this handout

Figure 2-4 illustrates the 1-D compression characteristics for idealized behavior, where one observes:

(1) A constant virgin compression index, $C_c = -\Delta e/\Delta \log \sigma'_{vc}$;

(2) The slope of swelling curve (C_s) increases with increasing OCR, but is independent of the maximum past stress;

(3) The recompression index (C_r), which bisects the unload-reload hysteresis loop, follows the same trend as C_s.

Although simplified, the above behavior is reasonably representative of clays of moderate sensitivity and plasticity, i.e. not "highly structured". Many soil models, such as the Modified Cam-Clay, further assume linear elastic behavior for overconsolidated clay such that $C_s = C_r =$ constant (i.e. independent of OCR and without a hysteresis loop).

Computation of the final consolidation strain (ε_{cf}) for loading from point $0 = \sigma'_{vo}$ to point $B = \sigma'_{vf}$, which exceeds the preconsolidation pressure σ'_p at point A, is given by:

$$\varepsilon_{cf} = \frac{\Delta e}{1+e_o} = \underbrace{\frac{C_r}{(1+e_o)}}_{=RR} \log \frac{\sigma'_p}{\sigma'_{vo}} + \underbrace{\frac{C_c}{(1+e_o)}}_{=CR} \log \frac{\sigma'_{vf}}{\sigma'_p} \qquad (2\text{-}1)$$

A2

CCL 10/96
10/98

1.361-1.366 Part IV-4

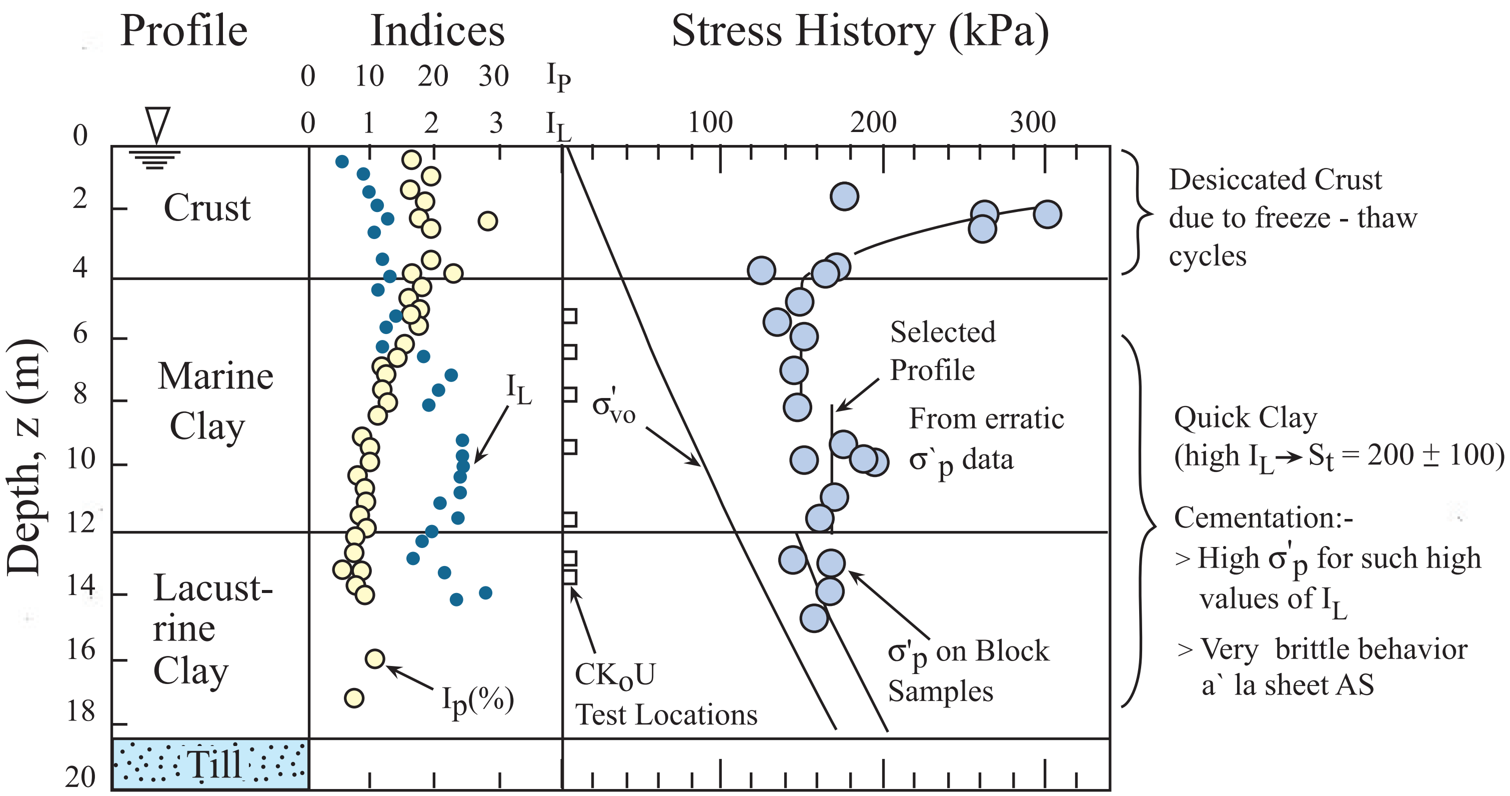
TABLE 2-1 PRECONSOLIDATION PRESSURE MECHANISMS

(For horizontal deposits with geostatic stresses)

Category	Description	Stress History Profile	In Situ Stress Condition	Remarks/References
A) Mechanical One Dimensional	1) Changes in total vertical stress (overburden, glaciers, etc.) 2) Changes in pore pressure (water table, seepage conditions, etc.)	Uniform with constant $\sigma'_p-\sigma'_{vo}$ (except with seepage)	K_o, but value at given OCR varies for reload vs. unload	Most obvious and easiest to identify
B) Desiccation	1) Drying due to evaporation, vegetation, etc. 2) Drying due to freezing	Often highly erratic	Can deviate from K_o, e.g. isotropic capillary stresses	Drying crusts found at surface of most land deposits; can be at depth within deltaic deposits.
C) Drained Creep (Aging)	1) Long term secondary compression	Uniform with constant σ'_p/σ'_{vo}	K_o, but not necessarily normally consolidated value	Leonards and Altschaeffl (1964) Bjerrum (1967) Mesri & Castro(1987) JGE, 113(3)
D) Physico-Chemical	1) Natural cementation due to carbonates, silica, etc. 2) Other causes of bonding due to ion exchange, thixotropy, "weathering" etc.	Not uniform	No information	Poorly understood and often difficult to prove. Very pronounced in eastern Canadian clays, e.g. Sangrey (1972), Bjerrum, (1973) Quigley, (1980)

1.361-1.366 Part IV-4

CCL 10/27/98



Desiccated Crust due to freeze - thaw cycles

Quick Clay (high $I_L \rightarrow S_t = 200 \pm 100$)

Cementation:-

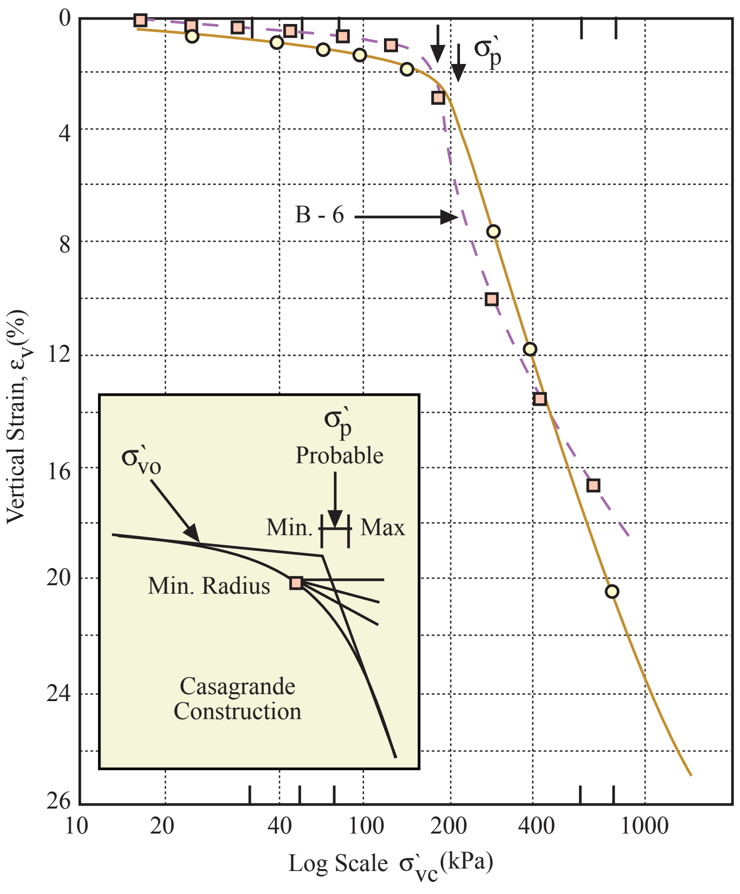
> High σ'_p for such high values of I_L

> Very brittle behavior a` la sheet AS

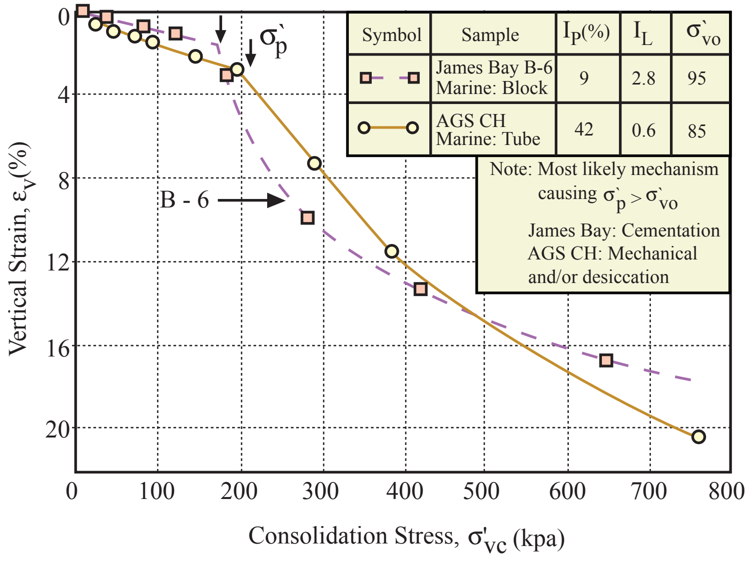
A4

CCL 10/27/98

1361-1366 Part IV-4



(a) Semi log scale



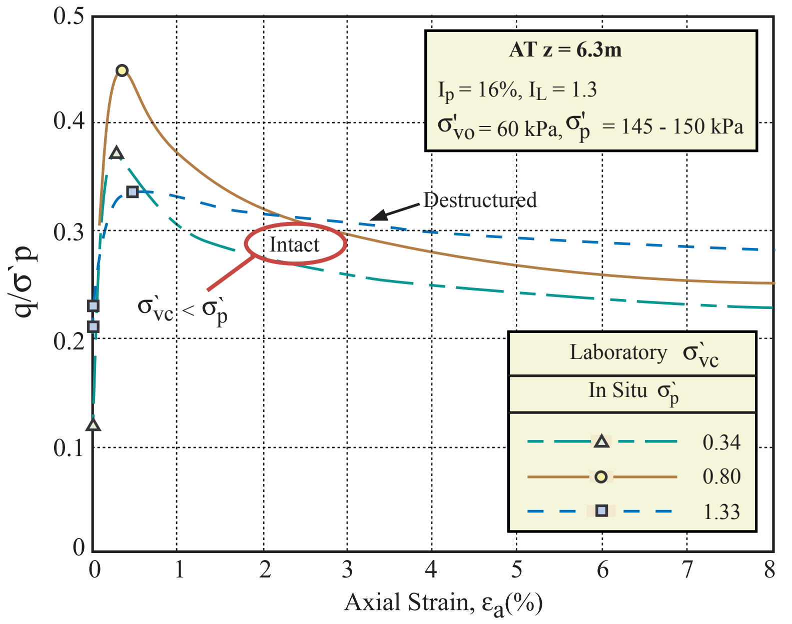
Symbol	Sample	I_P(%)	I_L	σ'_{vo}
□- - -□	James Bay B-6 Marine: Block	9	2.8	95
○——○	AGS CH Marine: Tube	42	0.6	85

Note: Most likely mechanism causing $\sigma'_p > \sigma'_{vo}$

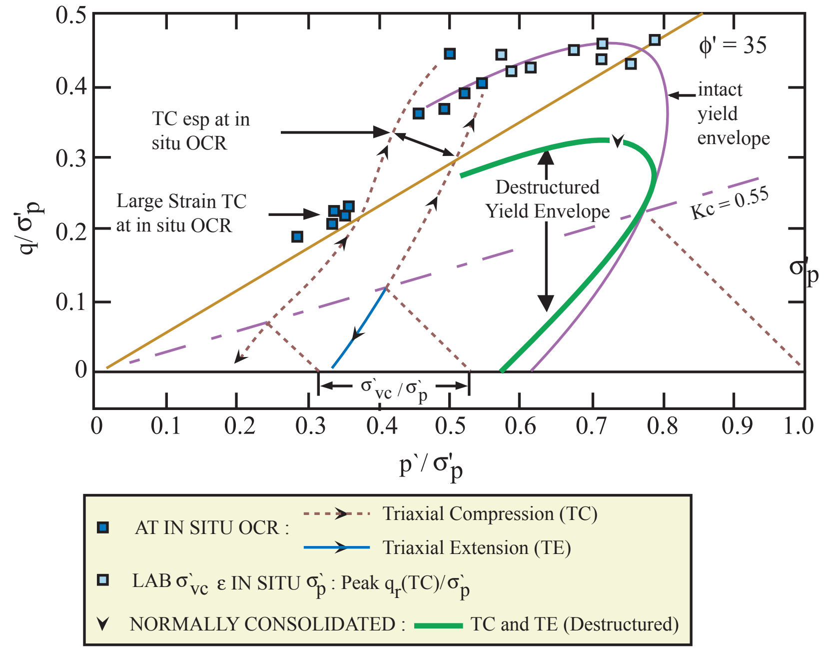
James Bay: Cementation
AGS CH: Mechanical and/or desiccation

(b) Natural scale

B - 6 1-D compression - sharp break in ε_v vs log σ'_{vs} & S-shaped virgin compression



(a) Normalized Stress-strain Data From CkoUC Tests



(a) Normalized Effective Stress Paths and Yield Envelopes

B - 6 Undrained shear - very brittle behavior, i.e. small ε_f followed by high rate of strain softening

Adapted from Jamiolkowski, Ladd, Germaine & Lancellota (1985) 11th ICSMFE, San Francisco

AS