PART V SATURATED SOILS: TRANSIENT FLOW

V-1 INTRODUCTION, PORE PRESSURE PARAMETERS AND UNDRAINED SHEAR

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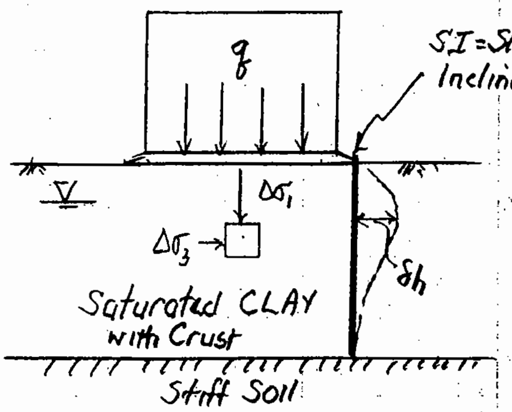
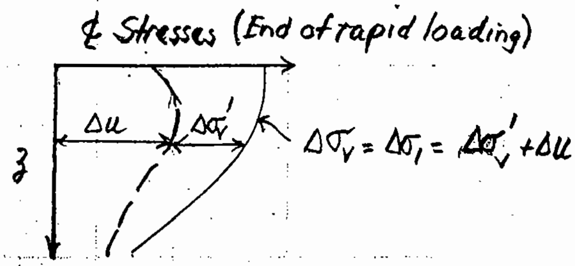
1 INTRODUCTION

Example of Storage Tank & Related Geotechnical Analyses



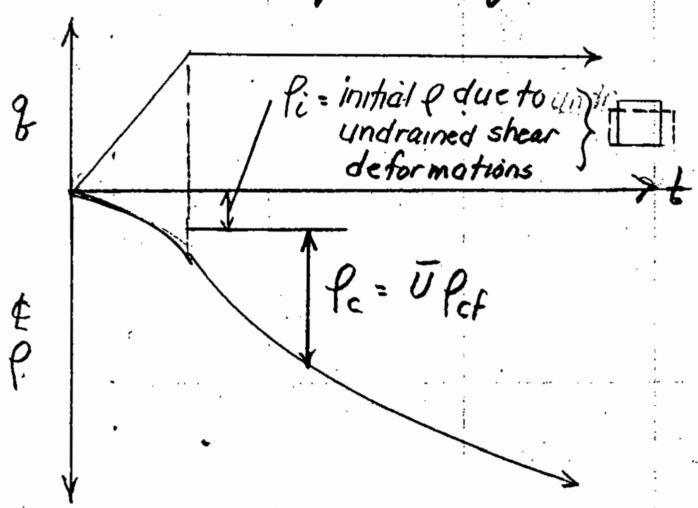


1) Stability at end of rapid loading ("end-of-construction"): In practice, assume zero drainage to be conservative → $\Delta V = \Delta w = 0$

$\therefore FS = q_{ult}/q = 6.2 s_u/q$ (from Part V-3; s_u covered in Part V-4)

NOTE: Potential failures occur during loading

2) Stresses and deformations for undrained loading



a) Change in stresses

- Δu from Skempton A & B parameters (Sect. 2)
- Get $\Delta\sigma'$ without ΔVol for 2 & 3-D loading (in contrast to 1-D loading sat. clay)

b) Initial settlement (ρ_i) due to undrained shear → ΔShape at constant volume → δ_h (Part V-5 for importance & techniques)

3) Consolidation settlement ($\rho_c = \bar{U}\rho_{cf}$; ρ_{cf} = final consolidation settlement

- Consolidation = volume change at constant load due to drainage of pore water = incr. $\Delta\sigma'_v = \Delta\sigma_v$ (constant) $- \Delta u$ (decreasing)
- $\bar{U}$ = ave. degree of consolidation $= \rho_c/\rho_{cf}$
- Usually estimate rate via Terzaghi theory (Part V-2)

$t \propto H_d^2/c_v$ { H_d = max. drainage height; $c_v = k_v/m_v \cdot \gamma_w$ = coef. of consolidation

1. INTRO. (Continued)

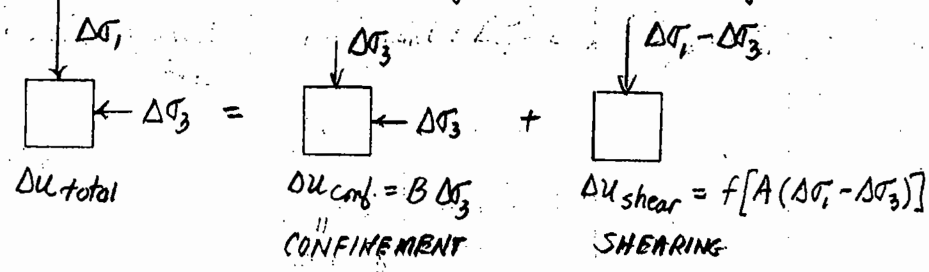
4) Long term (drained) stability
- Analyses covered in Parts IV-6 & 7
- Never critical for LOADING (applied stresses increase in situ p)
- But usually critical for UNLOADING (applied stresses decrease in situ p) } Part V-3

2. SKEMPTON'S PORE PRESSURE PARAMETERS (Chap 26)

2.1 Background

$\Delta\sigma_1 = \Delta\sigma_v$ & $\Delta\sigma_3 = \Delta\sigma_h$

- $\Delta\sigma$ = applied stresses during undrained loading: Illustrated for ℄ element

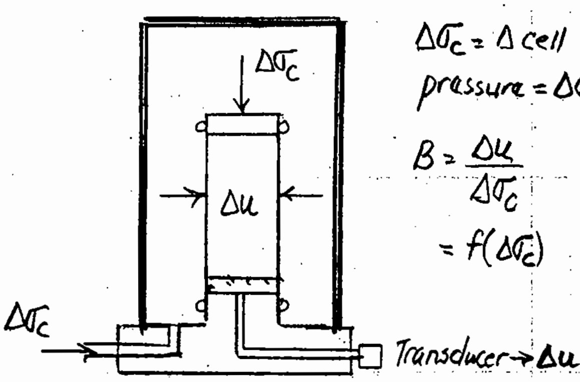


- For S < 100% $\Delta u = B\,\Delta\sigma_3 + B \cdot A(\Delta\sigma_1 - \Delta\sigma_3)$
- For S = 100% $\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$ since $B \approx 1.00$

NOTE: Above for applied $\Delta\sigma_1$ & $\Delta\sigma_3$, which can act in any direction. Also can have negative values of $\Delta\sigma_1$ & $\Delta\sigma_3$ (i.e., for UNLOADINGS)

2.2 B Parameter [for hydrostatic (isotropic) compression]

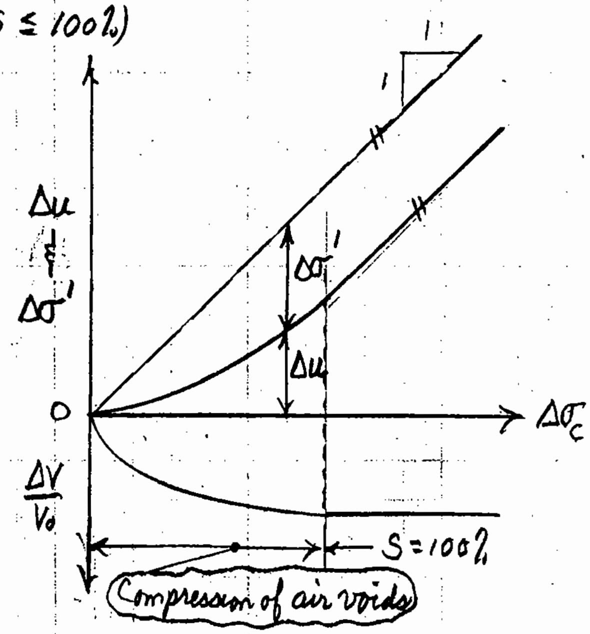
1) Test apparatus and results ($S \leq 100\%$)



$\Delta\sigma_c = \Delta$ cell pressure $= \Delta\sigma_3$

$B = \dfrac{\Delta u}{\Delta\sigma_c} = f(\Delta\sigma_c)$

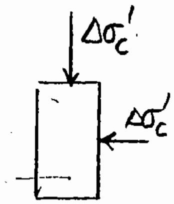
Triaxial Cell: No Drainage
(Δmass = 0)



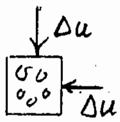
2.2 B Parameter (Continued)

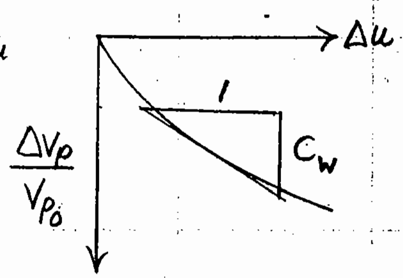
2) Physical interpretation: NOTE Volume decrease is positive

- $\Delta V = \Delta V_p$ (pore fluid) $= \Delta V_{sk}$ (soil skeleton)



Pore Fluid



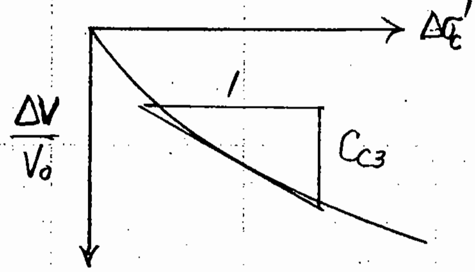


$$C_w = \frac{\Delta V_p / V_{p0}}{\Delta u} = \frac{\Delta V_p / n V_0}{\Delta u}$$

For S = 100% $C_w = 46 \times 10^{-6}\ cm^2/kgf$
$= 47 \times 10^{-6}\ bar^{-1}$

Soil Skeleton

Hydrostatic (Isotropic) consolidation in triaxial cell



$$C_{c3} = \frac{\Delta V / V_0}{\Delta\sigma_0'}$$

- $\Delta V_p = \Delta V_{sk} = \Delta V$

$$\Delta u \cdot C_w \cdot n V_0 = \Delta\sigma_c' \cdot C_{c3} \cdot V_0 \rightarrow \Delta u\, C_w\, n = (\Delta\sigma_c - \Delta u)\, C_{c3}$$

$$\therefore \rightarrow \Delta u\,(n C_w + C_{c3}) = \Delta\sigma_c\, C_{c3}$$

$$\therefore B = \frac{\Delta u}{\Delta\sigma_c} = \frac{1}{\left(1 + n\frac{C_w}{C_{c3}}\right)}$$

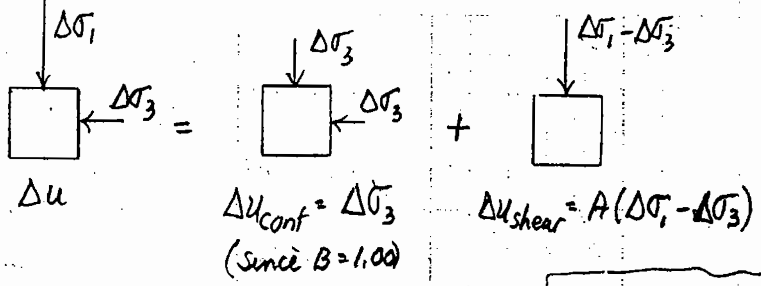
NOTE: (1) Reliable eqn.
(2) also valid for 1-D loading since $C_{c1} = C_{c3}$, where $C_{c1} = m_v = \Delta\varepsilon_v / \Delta\sigma_v' \approx \frac{0.434\,(C_r \text{ or } C_c)}{(1+e_0)\,\sigma_{v\,ave.}'} = \frac{0.434\,(RR \text{ or } CR)}{\sigma_{v\,ave.}'}$

3) Typical values (Table 26.1)

Material	% S	B	Remarks
Soft → Stiff CLAY	100	0.999+	Measure B to check saturation of triaxial specimens
	≈ 99	Signif. < 1	Need backpressure of several atm → S = 100% of "sat." clay
Dense SAND	100	0.99	
ROCK	100	< 0.6	Moderate $\Delta\sigma_c$
Compacted CLAY	90	0.5 ± 0.2	Very approximate & increases with $\Delta\sigma_c$

2.3 A Parameter (For S=100% → B=1.00)

1) Definition

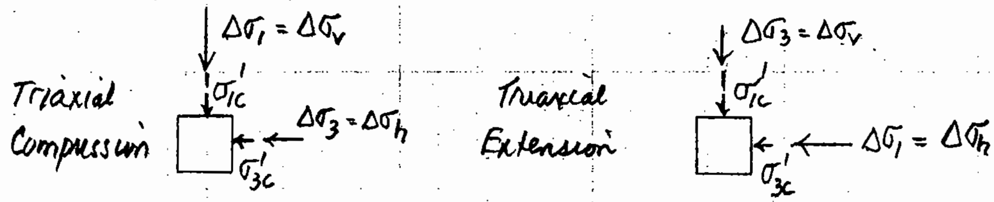


$\therefore \Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \rightarrow \boxed{A = \dfrac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}}$

2) Discussion

a) Values of $A = (\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ are obtained by measurements during undrained shear tests

b) $\Delta\sigma_1$ & $\Delta\sigma_3$ are applied principal stresses during undrained shear, and may differ from directions of σ_1' & σ_3' prior to shear



$A = (\Delta u - \Delta\sigma_h)/(\Delta\sigma_v - \Delta\sigma_h)$ $\qquad$ $A = (\Delta u - \Delta\sigma_v)/(\Delta\sigma_h - \Delta\sigma_v)$

c) In presenting measured data during undrained shear tests, better to plot $(\Delta u - \Delta\sigma_3) = \Delta u_{shear}$, rather than just Δu (See Section 3.4)

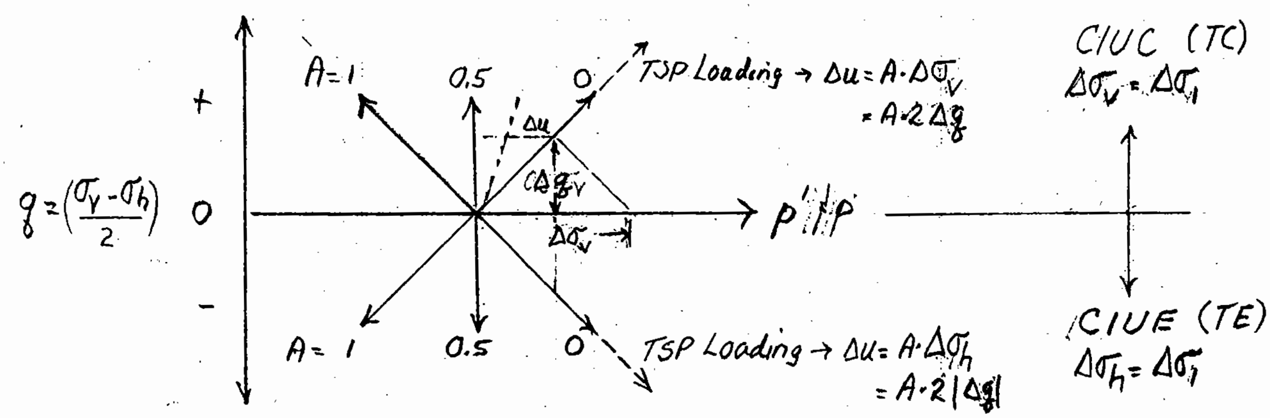
3) Effect of $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$ on A for Linear Elastic Isotropic (LEI) material

- Obtained via superposition as described in Fig. V 1-1 (p.5)

Type of Test	b	Eqn for A	Value of A
Tx. Compr. ($\Delta\sigma_2 = \Delta\sigma_3$)	0	$A = \dfrac{1}{(1 + C_{s2}/C_c')}$	1/3 since $C_{s2} = 2 \times C_c'$
Plane Strain ($\varepsilon_2 = 0$)	0.5	$A = \dfrac{1}{(1 + C_s'/C_c')}$	$\frac{1}{2}$ since $C_s' = C_c'$
Tx. Extension ($\Delta\sigma_2 = \Delta\sigma_1$)	1.0	$A = \dfrac{1}{(1 + C_s'/C_{c2})}$	$\frac{2}{3}$ since $C_s' = \frac{1}{2} \times C_{c2}$

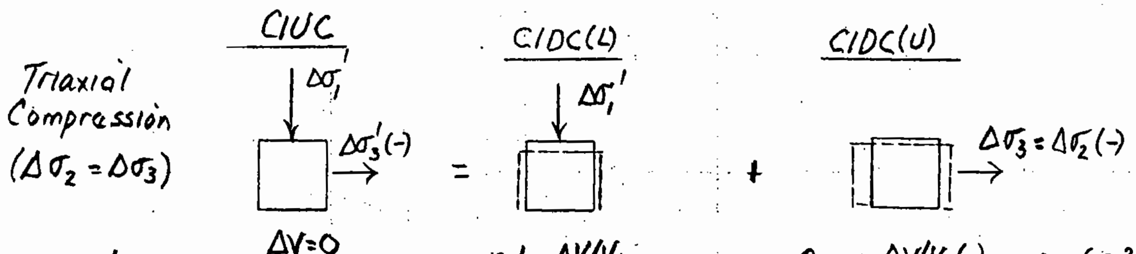
2.3 A Parameter (Continued)

4) Effective Stress Paths (ESP) as f(A) for CIU Triaxial Tests {Very Important}



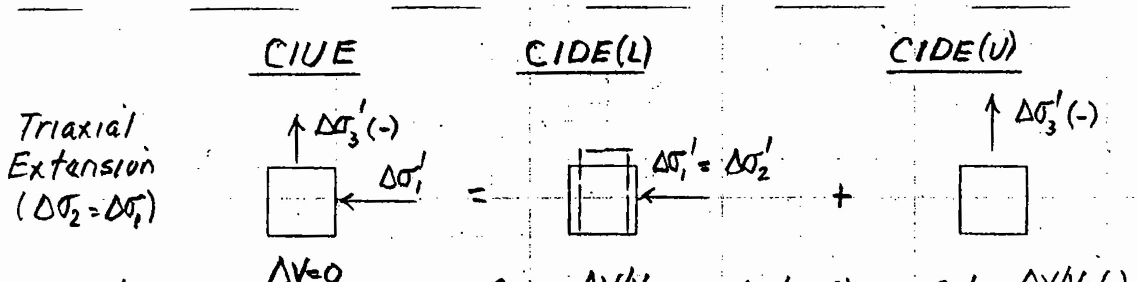
For LOADING tests ($\Delta\sigma_3 = 0$) : $\Delta u = p - p' \rightarrow A = (p-p')/2|\Delta q|$

Undrained Shear = Superposition of Loading / Unloading Drained Shear



$$C_c' = \frac{\Delta V/V_0}{\Delta\sigma_1'} \qquad C_{s2} = \frac{\Delta V/V_0(-)}{\Delta\sigma_3'(-)} = 2\times(C_s^2 = C_s^3)$$

$$A = \frac{1}{(1 + C_{s2}/C_c')}$$



$$C_{c2} = \frac{\Delta V/V_0}{\Delta\sigma_1'} = 2\times(C_c' = C_c^2) \qquad C_s' = \frac{\Delta V/V_0(-)}{\Delta\sigma_3'(-)}$$

$$A = \frac{1}{(1 + C_s'/C_{c2})}$$

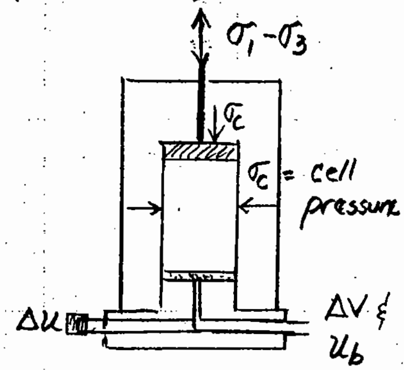
$$\Delta V/V_0 = 0 = C_c\,\underbrace{(\Delta\sigma_1 - \Delta u)}_{\Delta\sigma_1'(+)} + C_s\,\underbrace{(\Delta\sigma_3 - \Delta u)}_{\Delta\sigma_3'(-)} \rightarrow \Delta\sigma_1 - \Delta u = (C_s/C_c)(\Delta u - \Delta\sigma_3)$$

Substract $\Delta\sigma_3$ both sides & rearrange $\rightarrow (\Delta\sigma_1 - \Delta\sigma_3) = (\Delta u - \Delta\sigma_3)(1 + C_s/C_c)$

$$\therefore A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} = \frac{1}{(1 + C_s/C_c)}$$

Fig. VI-1 Derivation of A from Superposition (Only Valid for Elastic Material)

3. TYPES OF SHEAR TESTS, STRENGTH PRINCIPLES AND UNDRAINED SHEAR BEHAVIOR



3.1 Types of Shear Tests ($S \approx 100\%$)

- Will define three basic types of triaxial shear tests that can be run on "undisturbed" samples of clay
- Steps during testing

① Initial condition after trimming specimen and placing in cell ($\sigma_c = 0$, $w_i = w_N$)
$\sigma_s' = \sigma_c - u = -u$; = capillary pressure $u_c = u_a - u_w$

② Apply σ_c (with or w/o drainage) ③ Apply $\sigma_1 - \sigma_3$ (with or w/o drainage)
(May have $K_c \neq 1$)

1) Consolidated-Drained = CD

- C = consolidated wrt $\sigma_c' = \sigma_c - u_b \rightarrow \Delta V \therefore w_c \neq w_N$ (backpressure to saturate)
- D = drained during shear $\rightarrow \Delta u = 0$ & $\Delta w \neq 0$

Examples CIDC/E (L/U) CK_0DC/E (L/U)

$I, K_0 = K_c = \sigma_{hc}'/\sigma_{vc}'$
C, E = Comp., Ext.
L, U = Load, Unload

2) Consolidated-Undrained = CU

- C = as above with $\sigma_c' = \sigma_c - u_b$ & $w_c \neq w_N$
- U = undrained during shear $\rightarrow \Delta w = 0$; $\Delta u \neq 0$ & $w_f = w_c$

} Usually need u_b = several atm. $\rightarrow S = 100\% \rightarrow B = 1.00$

Examples CIUC/E (L/U) CK_0UC/E (L/U)

Measurement of u is standard practice

NOTE: For both CD & CU tests, automated triaxial equipment can follow any stress path during consolidation (i.e., 1-D = K_0) and shear

3) Unconsolidated-Undrained = UU

- 1st U = unconsolidated wrt σ_c, i.e., no drainage allowed
  Preshear $\sigma' = \sigma_s' + \sigma_c - \Delta u = \sigma_s' + \sigma_c(1-B)$
- 2nd U = as above (except usually do not measure u)

} Hence $w_f = w_N$

- Std practice = triaxial compression (loading) ≡ UUC with $d\varepsilon_a/dt = \dot{\varepsilon}_a = 1\%/\text{min}$
- If $\sigma_c = 0$, then call ________________

3.2 Statement of Strength Principles

- For NC (OCR=1) clay or OC clay with same σ'_{cm} and assuming no sample disturbance: } But treat TC & TE separately

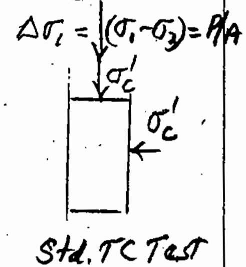
I Unique failure envelope, i.e. q_f vs p'_f } Applies to all 3 types of triaxial tests, i.e., CD, CU & UU

II Unique $w_f - q_f - p'_f$ relationship (that is parallel to VCL = w_c vs $\log \sigma'_c$)

- Will also assume perfect normalized behavior wrt ε_a vs $\frac{(\sigma_1-\sigma_3)}{\sigma'_c}$, Δw, A, etc.

3.3 Comparison of CIU vs CID Standard TC Tests Run on NC Clay

1) See Fig. VI-2 (p8) for results of a CIDC(L) test run on clay with linear VCL and normalized behavior. Hence only 1 test needed to define Princ. I & II relationships



2) Will now illustrate how one can predict either exact or approximate stress-strain-strength properties for a CIUC(L) test starting from the same $\sigma'_c = \sigma_c - u_b$.

a) Values of $s_u = q_f$ and p'_f from Principle ___ = ______

(Note: Typical $s_u/\sigma'_c = 0.30 \pm 0.05$)

b) Total Stress Path (TSP) & TSP$-u_b$ = ______

c) End point for Effective Stress Path (ESP) = ______

d) ESP for $A = 0$ & 1.0

- Initial A for elastic response = ____
- Typical A_f for OCR=1 = 1.0 ± 0.2

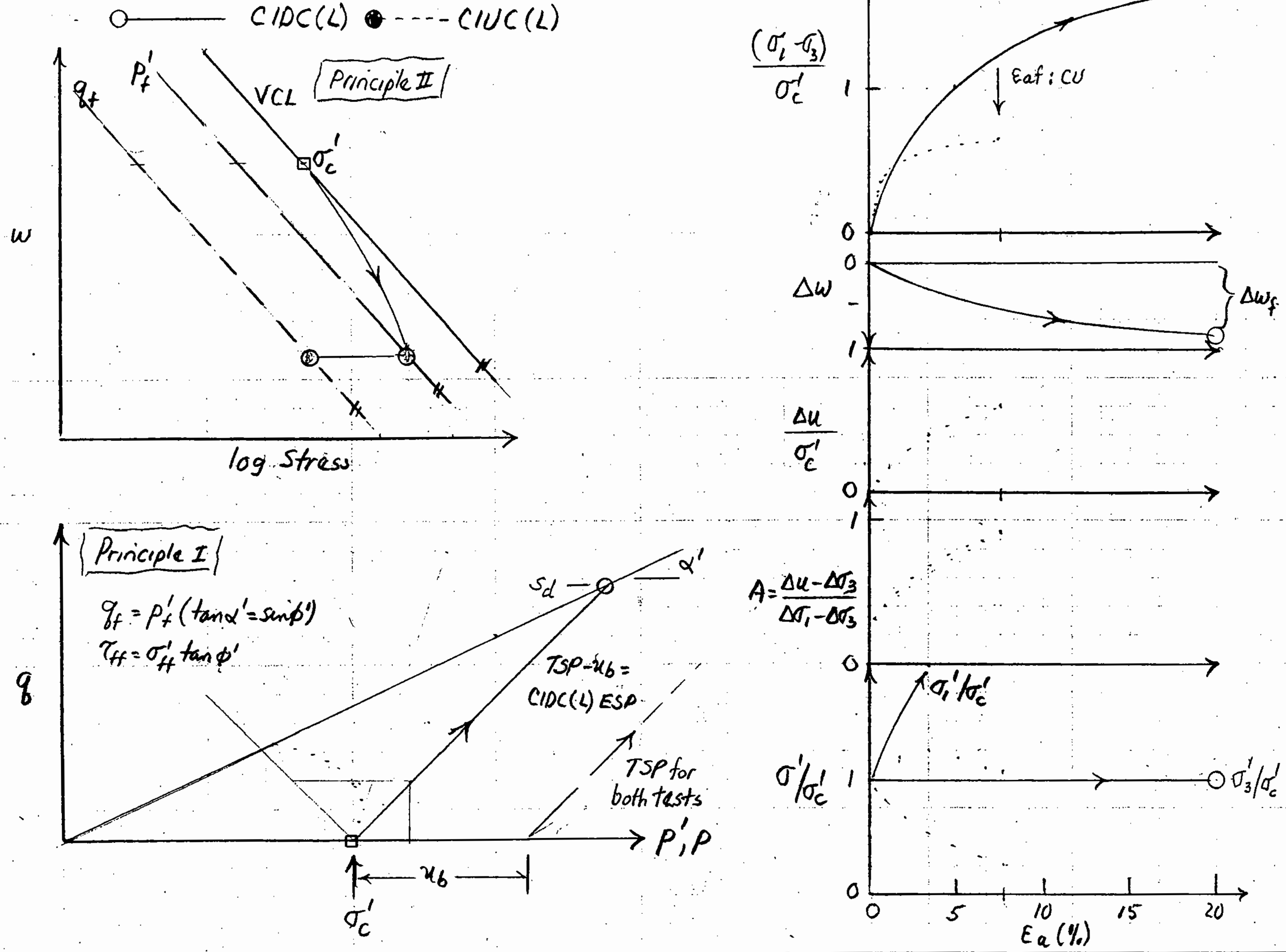
• Typical shape of ESP and determination of A

e) Stress vs strain → $\varepsilon_f \simeq 5$-10%

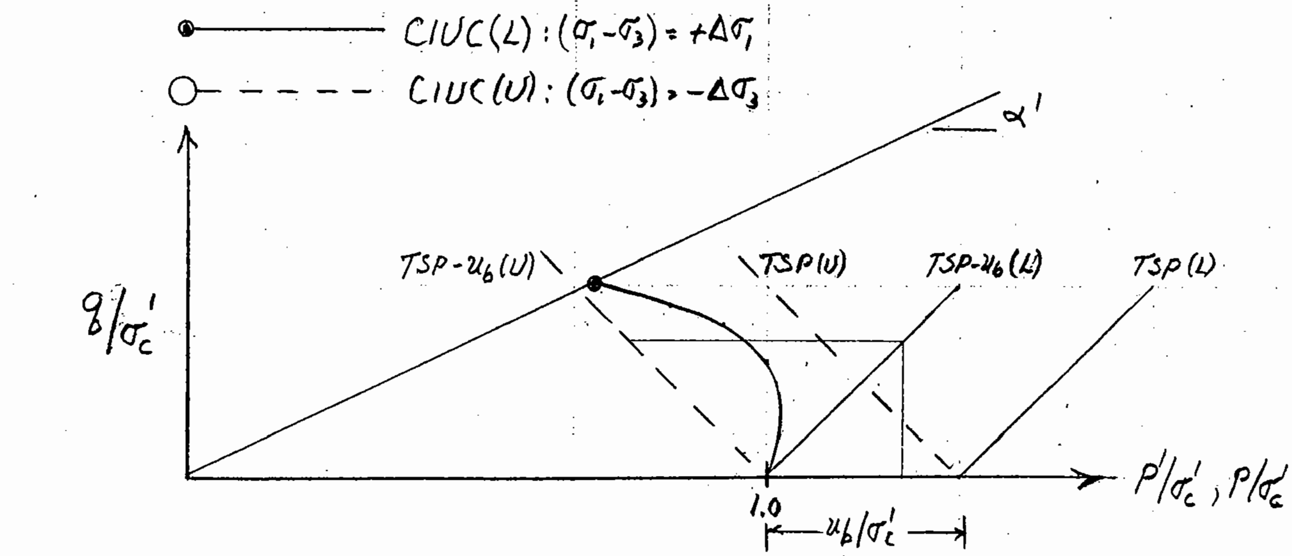
- ε_a vs $(\sigma_1-\sigma_3)/\sigma'_c$, $\Delta u/\sigma'_c$ & A
- $\Delta\sigma'_3 = \Delta\sigma_3 - \Delta u =$ ______ $\times (\sigma_1-\sigma_3)$
- $\Delta\sigma'_1 = \Delta\sigma_1 - \Delta u =$ ______ $\times (\sigma_1-\sigma_3)$

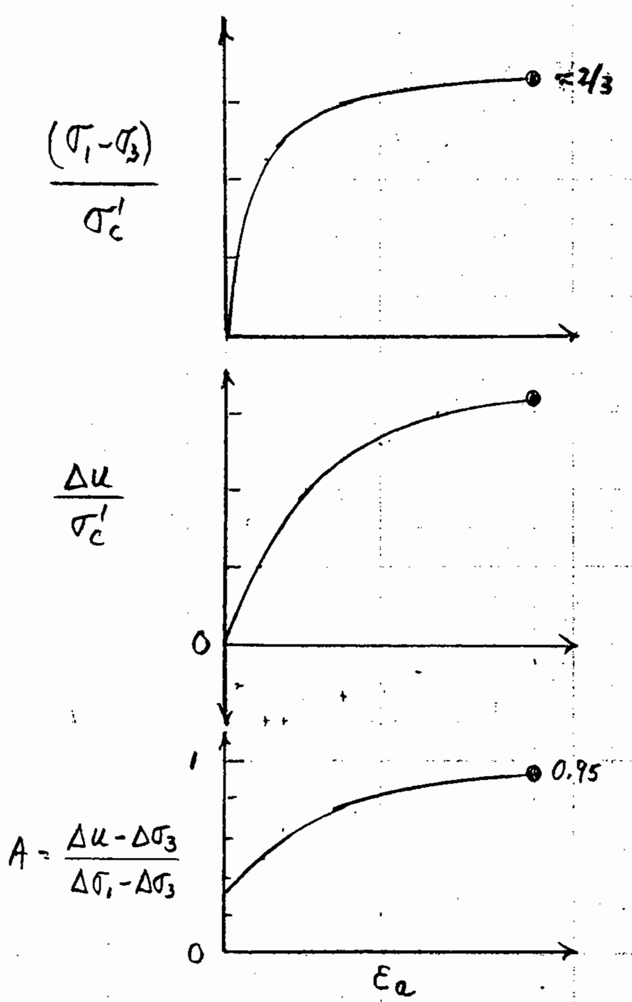
Shape of ESP is uniquely related to values of A

Fig. VI-2: Std. Triaxial Compression Data on NC Clay



3.4 Comparison of CIUC(U) vs. CIUC(L) for B=1.00 (S=100%)





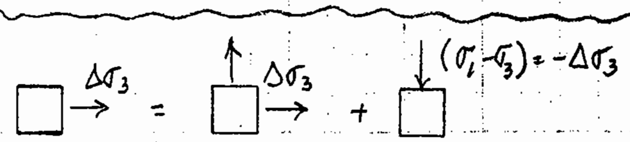
1) CIUC(L) Normalized ESP & stress-strain

2) CIUC(U): TSP & TSP-u_b

3) What changes?

- q_f & p_f'
- ESP & effect of $\pm\Delta\sigma_3$
- $A(U) = \frac{\Delta u_U - \Delta\sigma_3}{-\Delta\sigma_3} =$
- ε_a vs $(\sigma_1-\sigma_3)/\sigma_c'$
- ε_a vs $\Delta u_u/\sigma_c'$
- ε_a vs $(\Delta u - \Delta\sigma_3)/\sigma_c'$

CONCLUSION



3.5 Undrained vs. Drained Shear at High OCR (Fig. V1-3, p11)

1) Review of CIDC behavior for OC clay

a) Summary vs NC

- Increase in ESE (q_f vs p'_f) : Principle I
- $w_f - q_f - p'_f$ shifts to left with increasing OCR : Principle II

} Same for CIDC & CIUC

b) Review CIDC(L) data at OCR ≈ 10 in Fig. V1-3 (p11)

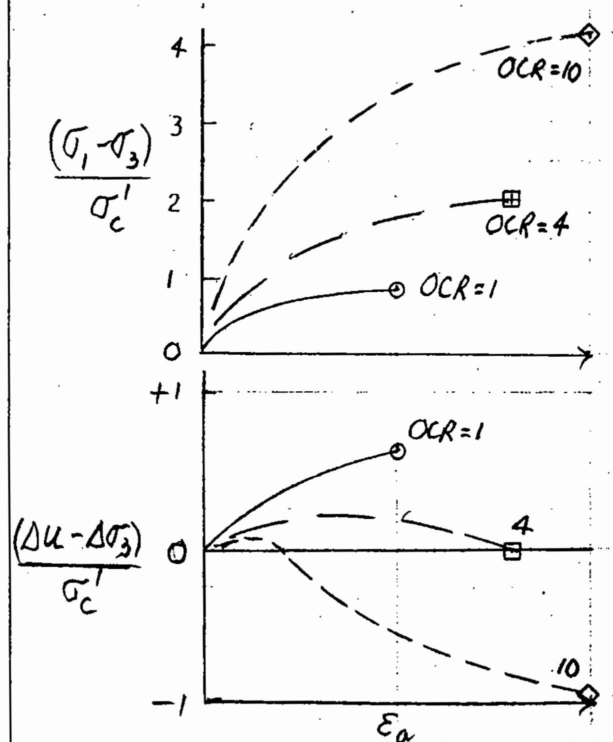
- Clay behavior at high OCR similar to "dense" sand (negative ψ, large I_R — Part III-2)

2) Predict – compare CIUC(L) & CIDC(L) at OCR = 10 (same σ'_{cm} & σ'_c)

- How obtain q_f & p'_f?
- How predict shape of ε_a vs $(\Delta u - \Delta\sigma_3)/\sigma'_c$?
- Resultant shape of ε_a vs $(\sigma_1 - \sigma_3)/\sigma'_c$
  " " " " " A
- Resultant shape of ESP, value of u_f, etc.

3) What changes if run CIUC(U) test at same σ'_c & σ'_{cm}?

3.6 Effect of OCR on CIUC Stress-Strain-Strength Behavior



1) Normalized Stress-Strain Curves

Increasing OCR →

- Increasing stiffness (E_u/σ'_c)
- " s_u/σ'_c
- " ε_f
- Decreasing $(\Delta u - \Delta\sigma_3)/\sigma'_c$

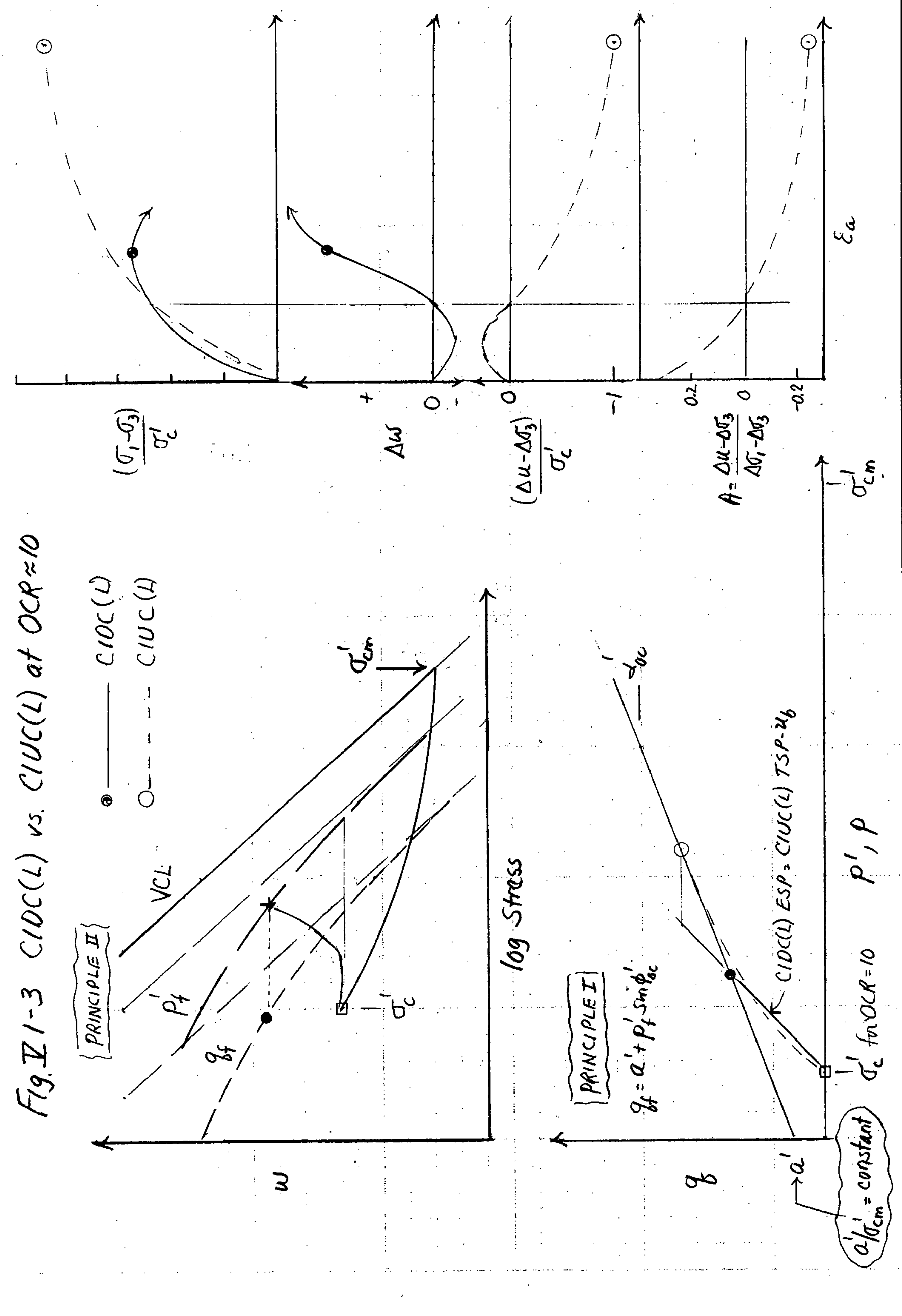
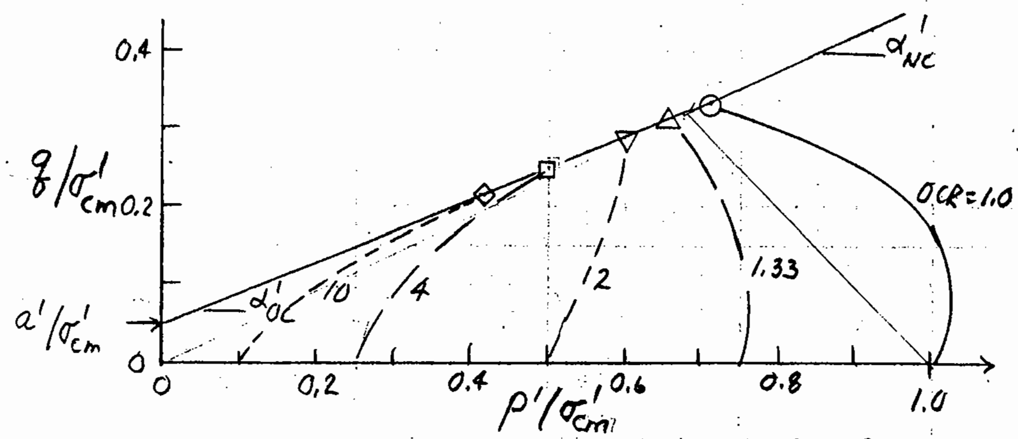


Fig. V 1-3 CIDC(L) vs. CIUC(L) at OCR≈10
CIDC(L)
CIUC(L)
PRINCIPLE II
VCL
p'_f
q_f
σ'_c
σ'_{cm}
log Stress
w
PRINCIPLE I
$q_f = a' + p'_f \sin\phi'_{oc}$
ϕ'_{oc}
CIDC(L) ESP = CIUC(L) TSP-u_b
CIDC(L) for OCR=10
σ'_c
p', p
σ'_{cm}
q
a'
a'/σ'_{cm} = constant
$(\sigma_1-\sigma_3)/\sigma'_c$
Δw
$(\Delta u-\Delta\sigma_3)/\sigma'_c$
$A = \frac{\Delta u-\Delta\sigma_3}{\Delta\sigma_1-\Delta\sigma_3}$
+
0
-
0
-1
0.2
0
-0.2
ε_a

3.6 Cont.

2) Normalized Effective Stress Paths

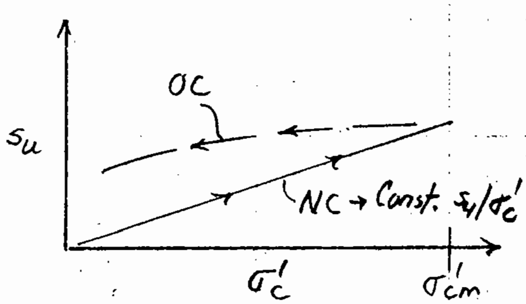


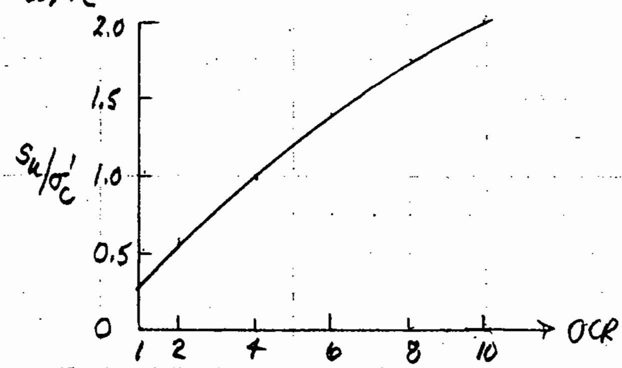
- Increasing OCR: Contractive behavior → Dilatant behavior

	CD Tests	CU Tests
Contractive behavior	Decr. vol	Decr. p'
Dilatant behavior	Incr. vol	Incr. p'

- Very similar to sand behavior covered in Part III-2
  - $+\psi$ or low I_R (low OCR) → contraction
  - $-\psi$ or high I_R (high OCR) → dilation

3) Undrained Strength Ratio, s_u/σ'_c





3.7 Three Factors Controlling s_u (For CIUC or CIUE)

1) Initial $p' = \sigma'_c$

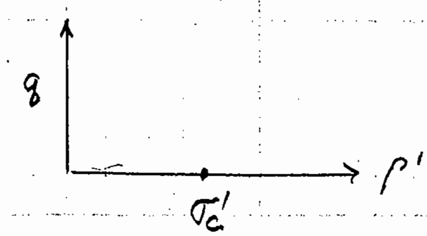
2) $\Delta p'$ during shear = f( )

3) q_f per p'_f = f( )

$$\frac{q_f}{\sigma'_c} = \frac{c'\cos\phi'/\sigma'_c + \sin\phi'}{1+(2A_f-1)\sin\phi'}$$

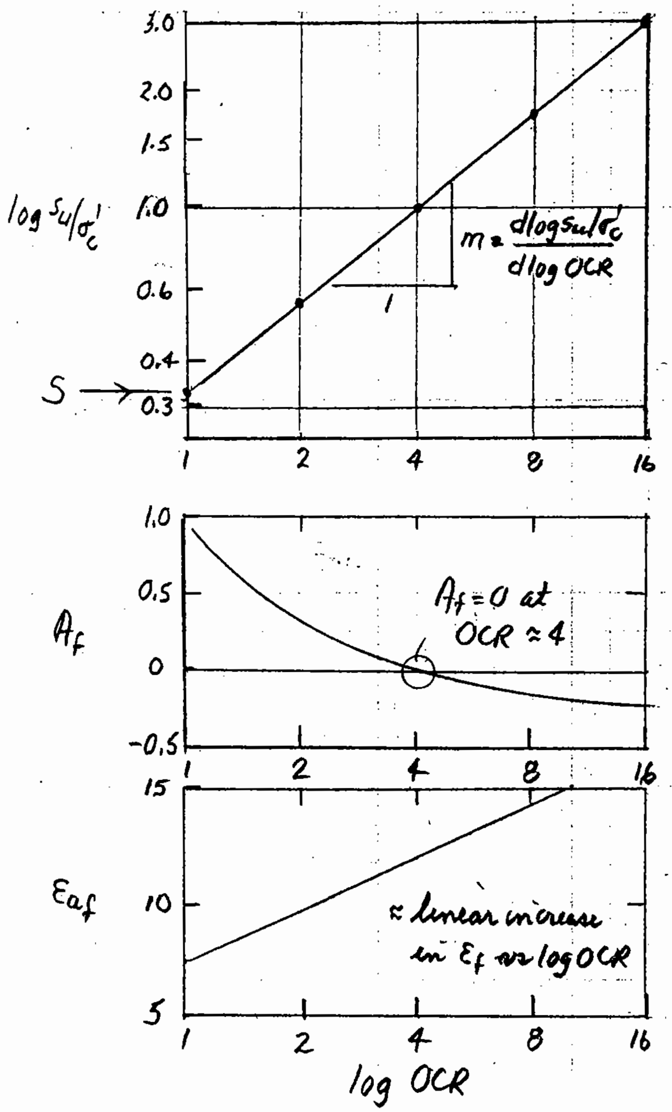
Sheet (A) of II-4 derives for $a' = c'\cos\phi' = 0$

Which is more important in explaining why increasing OCR → much larger s_u/σ'_c?



3.8 Summary of CIU Strength Parameters vs. OCR

1) CIUC trends with log OCR



Notes: • Extensive data from CIU & CK_oU tests on wide variety of clay → linear $\log s_u/\sigma_{vc}'$ vs log OCR (see Part V-4)

• Plot drawn for $S = 0.33$ & $m = 0.8$

3.9 SHANSEP Equation

(SHANSEP = Stress History And Normalized Soil Eng. Properties)
Ladd & Foott (1974), JGED, 100(7) Ladd (1991) JGE 117(4)

1) $\log s_u/\sigma_c' = \log S + m \log OCR$ from above (Use s_u/σ_{vc}' for CK_oU tests)

$\therefore$ $\boxed{s_u/\sigma_c' = S(OCR)^m}$ Must know how to apply (more details in Part V-4)

$S = s_u/\sigma_c'$ at $OCR = 1$
$\approx 0.3 \pm 0.05$ for CIUC
$\approx 0.2 - 0.25$ for CIUE (higher A_f)

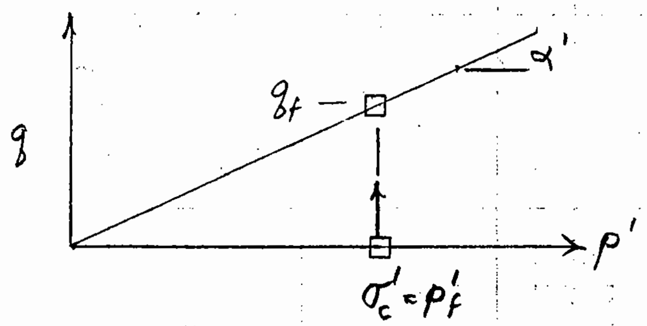
$m = (d \log s_u/\sigma_c') / d \log OCR$
$\approx 0.8 \pm 0.1$ for most clays

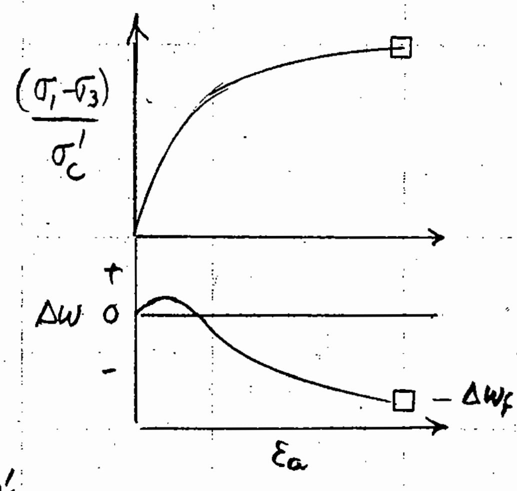
2) Section V-4 will show that for undrained stability analyses of loads on saturated clay: $S = 0.22 \pm 0.03$ & $m = 0.8 \pm 0.1$

3.10 Predict UUC Results From CIDC Test Data

1) Objective: To further illustrate application of Principles I & II

2) Given: Following data from one CIDC test run with $\sigma_c' = p' =$ constant on NC clay having $K_c = 1$ and perfect normalized behavior





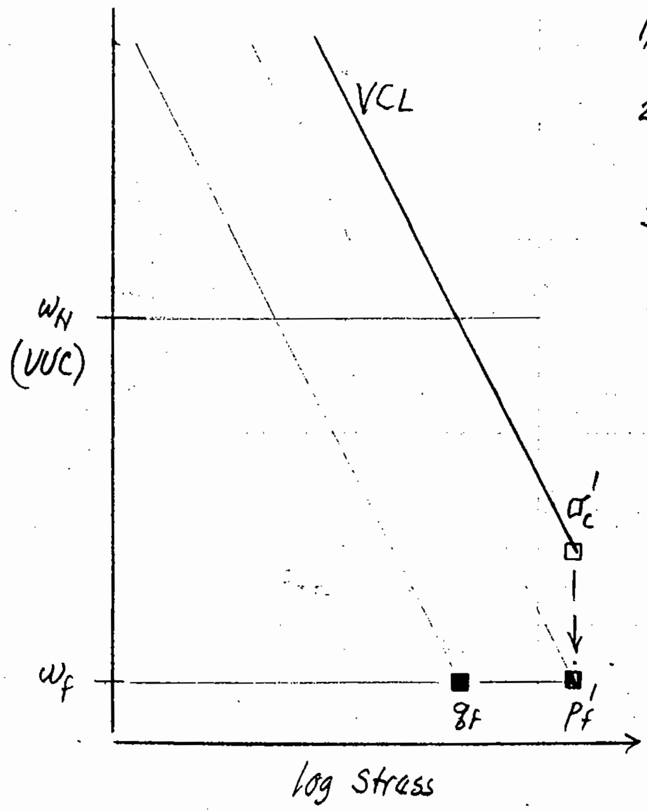
- See Fig V1-4 (p15) for w vs $\log \sigma_c'$ and plotted values of w_f vs q_f & p_f'.

3) Make predictions for unconfined compression test (UUC with $\sigma_c = 0$) run on tube sample with zero disturbance and $w = w_N$ (see listing in c) of Fig V1-4, p15)

NOTE: If you comprehend class discussion, then your understanding of basic strength principles far exceeds that of average practicing geotechnical engineers.

4) For this example, UUC test = CIUC test on sample with unknown σ_c' unless one measured preshear σ_s'. Also, this illustration applies to both NC and OC clay.

a) w - log Stress (Principle II)



c) Predictions for UC Test ($w = w_N$)

1) $\sigma_s' = u_c = \sigma_0'$

2) p_f' & q_f

3) TSP & TSP-σ_0'

4) u_f & Δu_f

5) A_f

6) ESP

7) ε_a vs q

b) Stress Paths (Principle I)

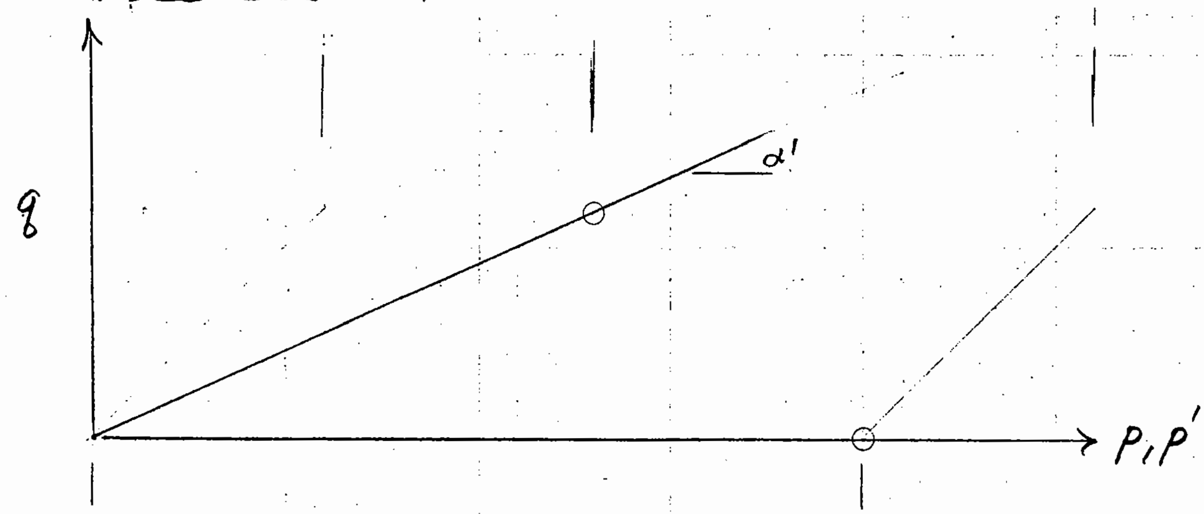


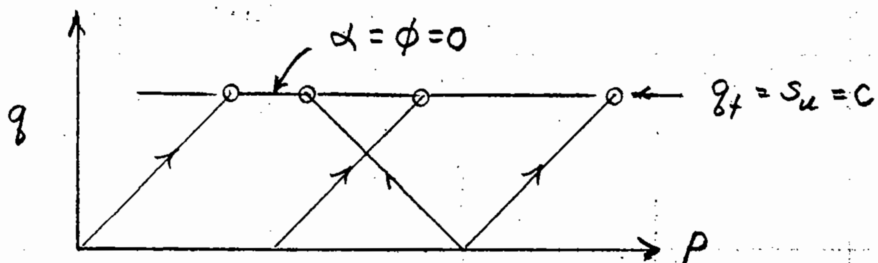
Fig VI-4 Predict Results for Unconfined Compression Test Given CIDC Test Data on NC Clay

3.11 Comments on UUC Testing

1) In reality UUC test results CANNOT be predicted from Principles I & II for following reasons:
   - In situ $K_0 \neq 1$ • Tube sampling always causes some disturbance
   - UUC tests are sheared at faster rate ($\dot{\varepsilon}_a = 1\%/min$) than typical CU tests ($\dot{\varepsilon}_a = 1\%/hr$); higher $\dot{\varepsilon}_a \rightarrow$ higher s_u

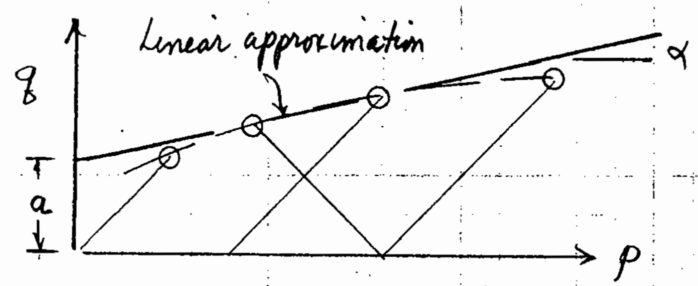
[From Part V-4, UUC tests not reliable method to measure s_u of soft clays for design]

2) For UUC tests on Saturated cohesive soils (so that $B = 1.00$), variations in σ_c and Load vs Unload TSP will not cause changes in the measured $s_u = q_f$ for identical test specimens (i.e. for same reason as covered in Section 3.4)



- This is basis for so-called "$\phi = 0, c = s_u$" stability and bearing capacity analyses discussed in IV-7 (Sec. 1.3) & V-3 (Sec. 3.2)
  e.g. $q_{ult} = \binom{5.14}{6.2} s_u + d \cdot \gamma_t$

3) If $S < 100\% \rightarrow B < 1.00$, then UUC tests with varying levels of confinement will → changes in s_u, e.g., UUC tests on compacted clay



$q_f = a + p_f [\tan\alpha = \sin\phi]$

↓

$\tau_{ff} = c + \sigma_{ff} \tan\phi$

$c = a/\cos\phi$

- $q_{ult} = S_c c N_c + S_\gamma \frac{1}{2} \gamma_t B N_\gamma + S_q \gamma_t d N_q$

  with N_c, N_γ & N_q using measured ϕ